Bridge Performance and Design

A Major Qualifying Project Report
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Abstract

This project studied the structural design of a highway bridge superstructure and substructure. The results were used to develop initial and life-cycle cost estimates. Guidelines are established for young engineers to follow in a preliminary design of these components. Finite element models were developed to study the distribution of loads through superstructures, and stress distributions in bridge connections. Simplified modeling techniques are presented, and provide a basis for capturing the stiffness provided by bracing members in analytical models.

Authorship

All members of the project group made an equal contribution to this project. All members were involved in the development of project objectives, and played a role in the final assembly of the project report. Chapter I (Introduction), Chapter II (Background), Chapter III (Methodology), and Chapter IX (Conclusions) were written by all group members. Primary authors for the remaining chapters are as follows:

Chapter IV (Superstructure): Sezai Emre Gazioglu & Douglas Heath

Chapter V (Substructure): Alejandro Sosa Boyd & Daniel White

Chapter VI (Life-Cycle Cost Analysis of Bearings): Besian Xhixho

Chapter VII (Effect of Bracing on Lateral Load Distribution): Douglas Heath

Chapter VIII (Finite Element of Clip Angle Connection): Besian Xhixho

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Capstone Design

This project considered many of the real world constraints provided by ASCE to fulfill the capstone design requirement. The following list identifies the five constraints considered in this project, and how each one was addressed:

Economic: Several superstructure design options were established and designed; cost analyses were conducted on all designs. The initial construction cost of the designs were compared, and the least expensive option was identified.

Sustainability: Life cycle cost analyses were conducted for the substructure design and bearing type selection. These analyses provided a way for designing a system that will minimize maintenance/additional investment over the life of the structure.

Constructability: Constructability was considered throughout the project. The designs provided consist of standard steel shapes or shapes with regular dimensions (dimensions rounded to the nearest whole number). Also, the constructability of large concrete sections is discussed in Chapter IV.

Ethical: This project considered ethical constraints by identifying potential problems with designs. For example, in Chapter IV, several design alternatives are proposed, however, problems related to cracking of concrete are identified. It is important for engineers to ensure that the limitations and potential problems associated with their designs are clear to the owner.

Health and Safety: These constraints were addressed by basing the designs on the AASHTO bridge design specification. Adhering to this specification provides a reasonable level of confidence that the structure will be structurally sound, and not pose a high level of risk to human life.

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I Introduction

Bridges are important structures in any society; they are especially important to trade by providing a time efficient means of crossing an obstacle. For example, suppose that the only factory in the country that manufactures toothbrushes were located on an island. The only way to get the toothbrushes off of the island and into stores where they could be sold would be to load the toothbrushes onto a ship or airplane that could take them to the mainland, or to build a bridge and transport them by truck. It is likely that the most cost-effective and time efficient option would be to transport the items by truck (neglecting the cost and construction time required to build the actual bridge). This concept of developing a time and cost efficient method of distributing goods is applicable to most of the products purchased today, and the financial savings that distributor generate by means of the bridge gives the structure value. Also, bridges allow easy travel within a region by providing a means to cross a river or gorge, for example. The service provided by bridges to travelers adds even more value to the bridge. The value added to the bridge by the savings of product distributors and travelers makes it a cost-effective and important piece of infrastructure for trade and travel.

As was highlighted in the previous paragraph, bridges are very important structures to a society. Because of this, it is important that they are structurally sound, and that they do not collapse or go out of service for any other reason. This would not only threaten human life due to the danger associated with a collapse, but it would also have severe financial implications, both in terms of the investment in the bridge itself and the loss of an important travel route to product distributors and travelers. To assure the quality of bridges, engineers have studied their behavior, and developed guidelines for designing and constructing them in a structurally sound manner. These guidelines have been made available by the American Association of State Highway and Transportation Officials (AASHTO).

This project studied the basic design of a bridge, particularly a highway overpass. This type of bridge is one of the simplest to design and was a good starting point for a young bridge engineer. The guidelines published by AASHTO were consulted to design various key components of the bridge, such as substructure elements (piers, abutments, and foundations) and superstructure elements (bridge deck, girders, and bracing members). In addition to studying the design of these basic structural components, this project also pursued several topics in depth. These topics included a life-cycle cost analysis of bridge bearings, an analysis of the effect of bracing on laterally distributing deck loads, and an analysis of a typical bridge connection. The project team was able to synthesize the results of all the designs and investigations conducted in the project, and develop an understanding of how bridges behave and how a structural design can affect project constraints (cost, constructability, etc.)

Consulting the AASHTO guidelines for the design of basic bridge components provided the project team with experience in the design of the components, and caused the project team to develop an appreciation for the guidelines published by engineering associations to protect life. By designing basic bridge components, the project team was also required to consider the constructability of a design. Finally, the cost analysis and life-cycle cost analysis activities associated with this project increased the project team's understanding of the importance of cost; not only the initial cost of construction, but also the cost of maintaining a bridge over its lifetime. An understanding of these concepts is not only be valuable to bridge design and construction, but to the design and construction of all structures.

II Background

To design a bridge, a fundamental understanding of its basic structural components and how they behave when loaded is needed. First, different components that make up a bridge are discussed; these are divided into two categories, superstructure components and substructure components. The principles behind life-cycle cost analysis are then investigated. Finally, the fundamental ideas behind finite element analyses are discussed, providing the reader with some background in this powerful tool for analysis.

The Superstructure

To get a better understanding of the components of the bridge, it is divided into two sections, the superstructure and the substructure. The superstructure is generally composed of the deck, girders, and bracing. The superstructure carries the traffic loads on the bridge and transfers them to the substructure. Figure 1, below, shows the different parts of the bridge. Items one and two are part of the superstructure.

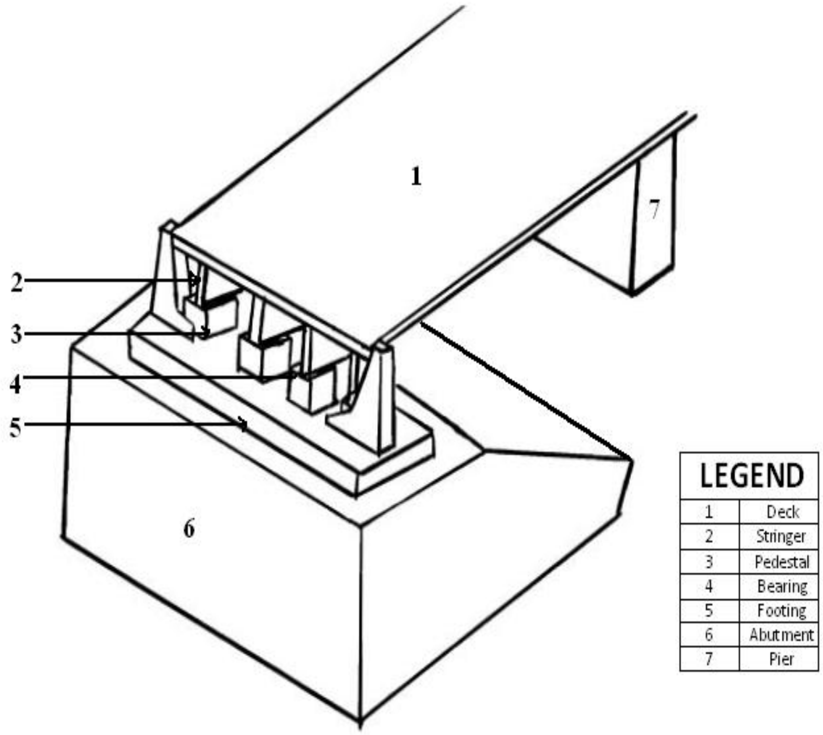


Figure 1: Bridge Components
Carmichael, Adam and Desrosiers Nathan. "Comparative Highway Bridge Design." 28 Feb. 2008. Worcester Polytechnic Institute. 10 Sept. 08 http://www.wpi.edu/pubs/e-project/available/e-project-022608-180459/unrestricted/comparative_highway_bridge_design_lda0802.pdf.

II.1.1 The Deck & Girders

The deck is the topmost part of the bridge, and is the part which comes into direct contact with traffic. It is also referred to as the slab. The deck is generally made from concrete, which is usually cast in place. The deck is supported by the girders, also known as stringers. The girders carry the load from the deck, and transfer it to the substructure at the piers and abutments. The girders are usually made from either reinforced concrete or steel.

In design, the spacing of the girders is often varied. This variation affects the size of the deck and the girders. The most economical spacing option based upon the total cost of the girders and the deck is then chosen. When the spacing between girders becomes large, intermediate beams are added to the structural system. These beams are placed perpendicular to traffic, and they frame into the girders. This prevents a need for a large and heavily reinforced deck (Xanthakos, 1994).

The deck can act compositely with the girders by connecting the elements together with shear studs. This provides extra load carrying capacity to the system because the two members work together to resist loads (Tonias, 1995). There are several design considerations associated with composite deck-girder systems; one consideration is the effect of a change in curvature of the system for continuous girders (Xanthakos, 1994). Despite the complexities associated with the design of composite systems, the American Association of State Highway Transportation Officials recommends their use unless it is prohibited by some factor (AASHTO, 2007).

II.1.2 Bracing Members

Bracing members are often used in girder bridges to help distribute loads. There are many different types of bracing that can be used. Members can be placed between the girders in an “X shape,” in which case the bracing acts like a truss to stiffen the superstructure. Beams are sometimes used instead, and have a similar effect. AASHTO recommends the use of bracing members to help resist wind load and limit lateral deflection. There is also research which indicates that the use of bracing members may help to distribute the applied loads among more girders, which would decrease the maximum girder moment. AASHTO recommends a maximum bracing spacing of 25 feet (Eamon and Nowak, 2002).

Substructure

The substructure supports the superstructure. It carries the loads above it, and transfers them to the foundations, and then to the ground. The substructure is made up of bearings, piers, abutments, and foundations.

II.1.3 Bearings

Bearings connect the girders to the piers and abutments to transmit loads such as the superstructure self-weight, traffic loads, wind loads, and earthquake loads from the superstructure to the rest of the substructure. The bearings allow translational and rotational movement in both the longitudinal and transverse directions. Translational movements are caused by shrinkage, creep, and temperature effects, while rotation movements are caused by traffic loads and uneven settlement of the foundations

Bearings can be classified as fixed bearings, allowing rotations but restricting translational movements, or as expansion bearings, allowing both rotational and translational movements. Sliding, roller, and elastomeric bearings fall into the expansion type, while rocker and pin bearings in the fixed type. In contrast with other expansion bearings, roller and elastomeric bearings are suitable for both steel and concrete girders. Roller bearings can be composed of a single or multiple rollers. Single roller bearings have a low manufacturing cost but at the same time have very little vertical load capacity; in contrast, multiple roller bearings can support large loads but are more expensive. Elastomeric bearings are made of a natural or synthetic rubber called elastomer. They accommodate translational and rotational movements by the deformation of this rubber. Elastomeric bearings are the most common because they are inexpensive and almost maintenance free, while still being tolerant with respect to loads and movements greater than the design values (Chen & Duan, 1999).

II.1.4 Piers

In a basic sense, piers are elements that connect the superstructure to the ground at any point that is not an end of the bridge. They are responsible for providing support for the girders at intermediate points along the bridge, and transferring the load from the superstructure to the foundations. Even though piers are commonly designed to resist vertical loads, design precautions are taken to also resist lateral wind loads (Chen & Duan, 1999).

There are many different types of piers and the selection of a specific pier depends upon what the bridge will be made out of and what it will be used for. The typical pier types for steel bridges are hammerhead, solid wall, and rigid frame piers as shown in the following Figure 2. For concrete bridges, the typical pier types are the bents, and can be designed for pre-cast girders and for cast-in-place girders as shown in Figure 3. The type of pier differs depending upon the material used for the girders because of the difference in the weights of the types of girders. Bents can support more dead load from the superstructure than other types of piers.

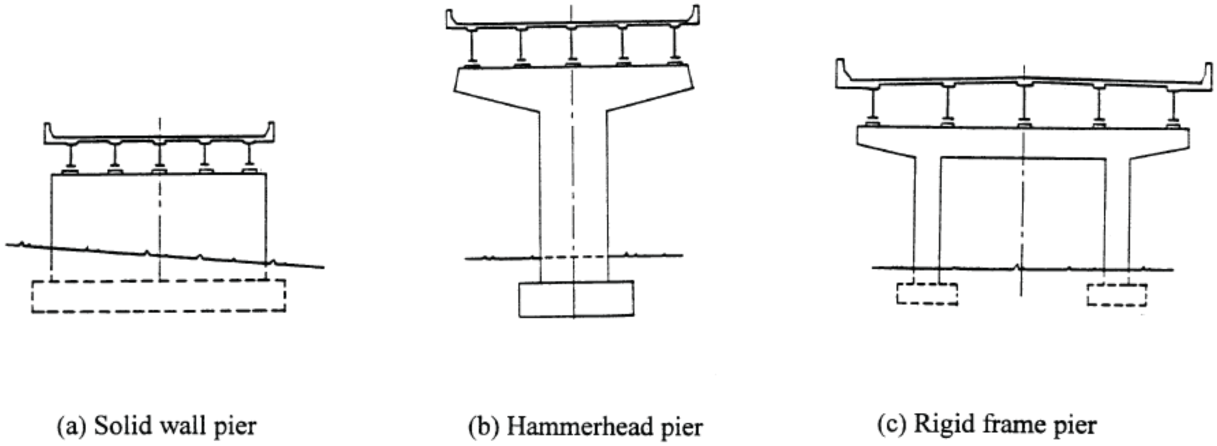


Figure 2: Typical Pier Types for Steel Bridges
Chen, Wai-Fah (Ed.) & Duan, Lian (Ed.) (1999). Bridge Engineering Handbook. Boca Raton, Florida: CRC Press

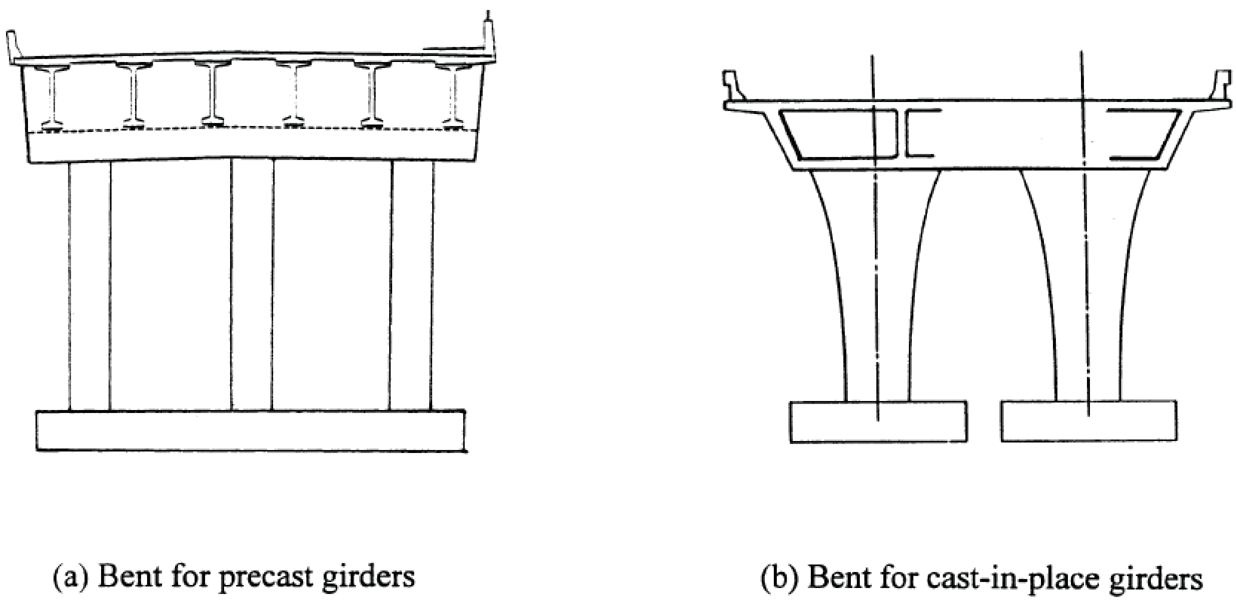


Figure 3: Typical Pier Types for Concrete Bridges
Chen, Wai-Fah (Ed.) & Duan, Lian (Ed.) (1999). Bridge Engineering Handbook. Boca Raton, Florida: CRC Press

II.1.5 Abutments and Retaining Structures

The same way piers provide vertical and lateral support at intermediate points in the bridge superstructure; abutments and retaining structures provide vertical and lateral support at the bridge's ends. In addition, abutments serve as connections between the bridge and the approach roadway, while retaining the roadway materials from the bridge span (Chen & Duan, 1999).

A bridge abutment can be classified as either open-end or closed-end depending on its relation with the roadway it passes over. Open-end abutment have slopes between the bridge abutment face and the edge of the roadway or river canal that the bridge crosses over. Closed-end abutment are high vertical walls that have no slope (Chen & Duan, 1999).

Abutments can also be classified according to the connections between the abutment stem and the bridge superstructure, as monolithic or seat-type abutments (see figure below). The monolithic abutment is built with the bridge superstructure; in contrast, the seat-type abutment is built separately from the bridge superstructure. For monolithic abutments, there is no displacement permitted between the superstructure and the abutment. This means that concrete girders could be cast directly into the abutments. For the seat-type abutments, the superstructure rests on the abutment stem through bearing pads, rock bearings, or other devices (Chen & Duan, 1999).

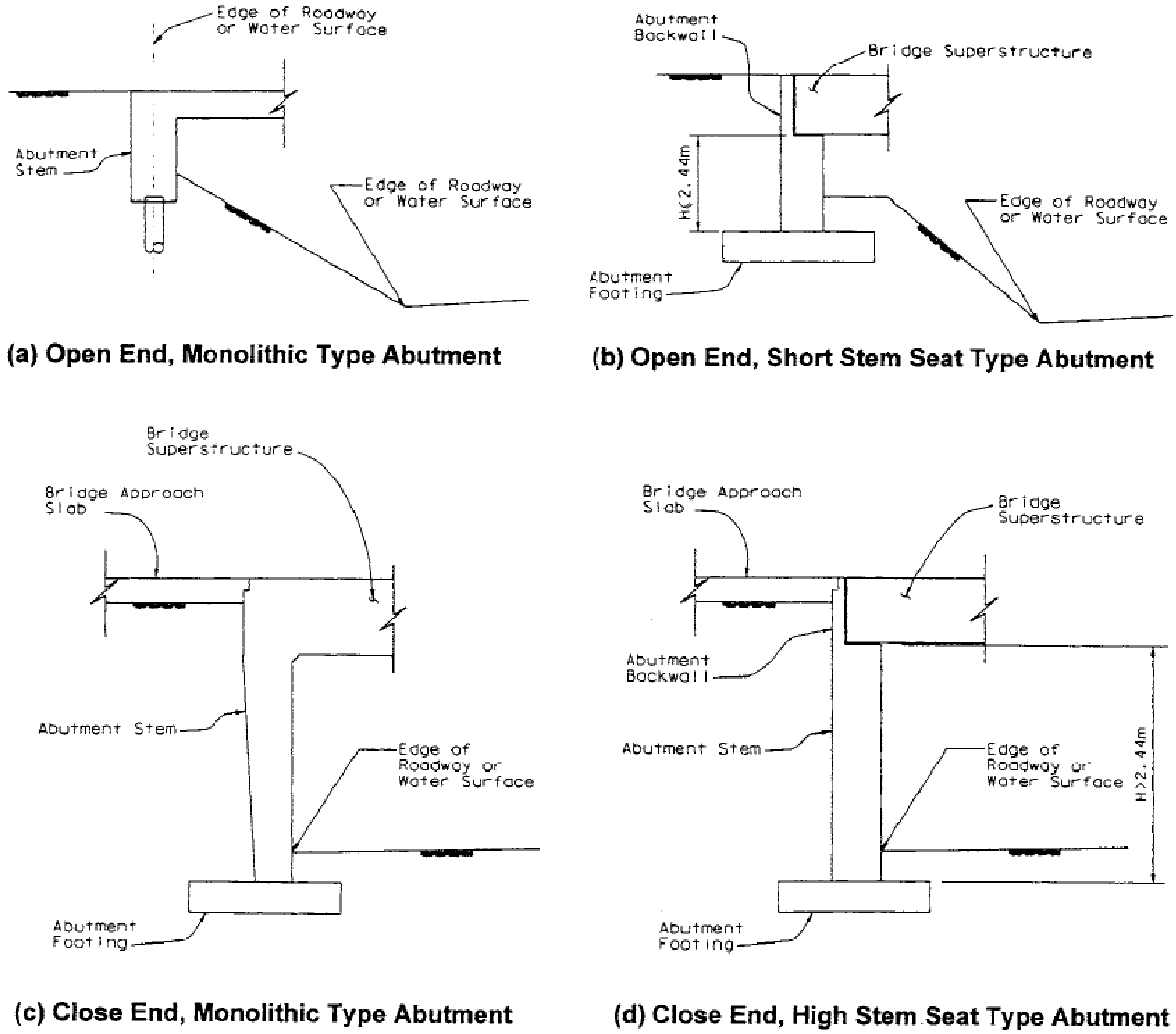


Figure 4: Typical Abutment Types
Chen, Wai-Fah (Ed.) & Duan, Lian (Ed.) (1999). Bridge Engineering Handbook. Boca Raton, Florida: CRC Press

The design of abutments depends in part upon the soil conditions at the project site. If the site is mostly hard bedrock, a vertical, close-end, abutment will be sufficient. If the soil is softer, a sloped, open-end, abutment will most likely be necessary to help counteract settlement. However, the use of sloped abutments usually requires longer bridge spans and extra earthwork; this could increase in the bridge construction cost (Chen & Duan, 1999).

II.1.6 Foundations

Foundations are structural elements that serve as a connection between the bridge substructure and the ground. These structural elements can be classified as either shallow or deep. Shallow foundations include spread footings, which are foundations that transmit the loads to soil near the surface (Figure 5). Deep foundations include piles, drilled shafts, caissons, anchors and others, which transmit all or some of the loads to deeper soils (Figure 6). (Coduto, 2001)

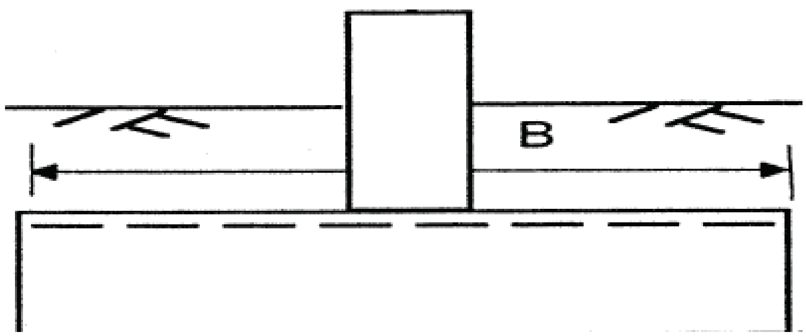


Figure 5: Shallow Foundations
Chen, Wai-Fah (Ed.) & Duan, Lian (Ed.) (1999). Bridge Engineering Handbook. Boca Raton, Florida: CRC Press

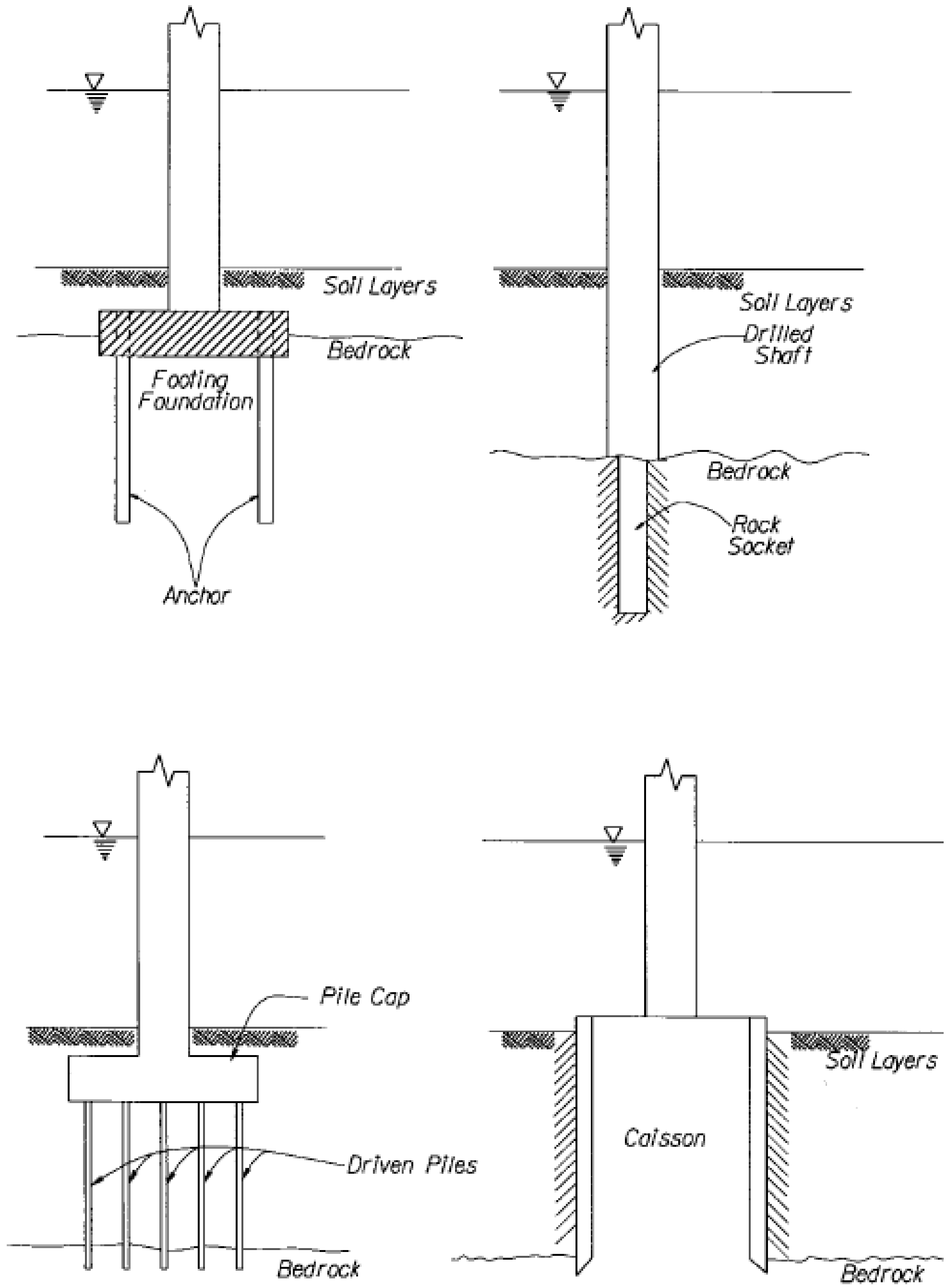


Figure 6: Deep Foundations
Chen, Wai-Fah (Ed.) & Duan, Lian (Ed.) (1999). Bridge Engineering Handbook. Boca Raton, Florida: CRC Press

Shallow foundations are used in good soil conditions. They are able to transfer vertical loads to the soil using bearing pressure. Deep foundations are used when the soil conditions near the surface are poor and the bearing pressure is not sufficient to carry the load. In these cases the foundation needs to extend to a deeper, solid layer of soil and take advantage of side friction in order to transfer the loads.

Design Loads

AASHTO provides many different types of loads to be considered in bridge design. These loads can be classified in one of two categories: permanent (dead) loads and temporary (live) loads. Permanent loads are generally fairly easy to determine because the unit weights of commonly used materials are provided in relevant bridge design codes. Live loads can be broken down into two categories: vehicular live loads and other types of live loads. Vehicular live loads include traffic passing over the bridge. Examples of other types of live loads include wind loads, earthquake load, etc. (AASHTO, 2007). AASHTO categorizes loads in a similar way as ASCE in their specification on <u>Minimum Design Loads for Buildings and Other Structures</u>.

Life-Cycle Cost Analysis

A life-cycle cost analysis is a way to determine the amount of money needed to maintain the bridge for a predetermined amount of time. The life-cycle cost of the bridge is equal to its initial construction cost plus the cost of maintenance. Maintenance will need to be performed on the bridge periodically after it has been completed. To evaluate the cost of this maintenance the type of repair needs to be determined. Once this is done the life cycle cost analysis is a matter of adding up the costs of the initial materials, initial construction, and the cost of repairs once every maintenance period to determine how much the bridge will have cost in 50 or 100 years.

Another way to look at this problem is to perform a present worth estimate. To do this an interest rate must be set. Then, based upon the amount of money needed for the repairs every maintenance period, the amount of money that needs to be set aside now to cover those costs can be determined. This allows for the amount of money that is needed at the time of construction to maintain the bridge for a period of 50 or 100 years to be calculated. In this project, parameters that most strongly influence life cycle cost, e.g. maintenance costs, interest rates, etc., were identified. They were assigned a range of reasonable values to develop an understanding of the range of costs associated with maintaining a bridge over its lifetime.

Finite Element Analysis

Finite element analysis is a mathematical modeling technique that involves representing a structure by a discrete number of elements; these elements are connected to each other by nodes. The type of element used to connect the nodes depends on the needs of the user; typical element types include beam (1 dimensional), plate (2 dimensional) and brick (3 dimensional). Finite element analysis can be used to analyze complicated structures or structures subject to complicated loadings. This is typically done through computer programs, such as ANSYS, which solve for the displacement of the model's nodes. By solving for these displacements, computer software is able to determine other useful information about the model, such as stresses, strains, and forces in members. Finite element analysis is particularly useful for exploring structural behavior, as it has been shown to accurately predict results related to unfamiliar phenomena.

Remarks

This chapter has presented the background information related to bridge superstructures, substructures, life cycle-cost analyses, design loads, and the finite element method. This background research was utilized to achieve the following project goals:

- Develop designs for superstructure elements
    - Define bridge deck, girder, and floorbeam sizes and cross sections
    - Assess construction cost and constructability of designs
- Develop designs for substructure elements
    - Define bridge pier, abutment, and foundation size and cross section
    - Assess life-cycle cost and construction cost of designs
- Develop a reasonable life-cycle cost estimate of bridge bearings
- Investigate load distribution through the bridge superstructure, particularly how bracing can affect load distribution, by the finite element method
- Investigate stress distribution in a typical bridge connection, by the finite element method
- Synthesize the results of the investigations listed above to develop a fundamental understanding of how bridges behave

The following chapters present the methodology followed for achieving these goals, and present a summary of the investigation's results.

III Methodology

This project consisted of five investigations: superstructure design, substructure design, life-cycle cost of bearings, investigation of the effect of bracing on lateral distribution of deck loads, and investigation of the behavior of connections. A brief summary of what was done in each investigation is provided in the following paragraphs.

The superstructure was designed in accordance with the AASHTO LRFD Bridge Specification. In total, 14 different superstructure systems were investigated; each system investigated was based on a bridge that had two 81 foot long spans and carried one lane of traffic in each direction. The following describes the different systems investigated: three different design options were considered, each with a fundamentally different girder arrangement. The cost of each design was assessed by conducting a cost analysis. In addition to exploring the three different girder arrangements, designs were developed for bridges with simple/continuous spans, composite/non-composite deck/girder behavior, and steel/reinforced concrete construction material; each of these additional parameters was investigated for the three different design options. The investigation/design of the superstructure allowed the project team to develop an understanding of how to design superstructure components, and how different design parameters can affect cost.

The substructure design consisted of the design of foundations, abutments, and two different types of piers. In the foundation design process, two different soil types were considered: one soil type that would allow for the use of shallow foundations, and another type that would require the use of deep foundations (piles). A life-cycle cost analysis was conducted for both pier types, and the sustainability of the two designs was assessed. A life-cycle cost analysis was also conducted for the abutment design. This investigation allowed the project team to develop an understanding of the substructure design process, and develop an understanding of the concept of life-cycle cost.

The life-cycle cost analysis of a commonly used type of bearing was conducted, and involved a consultation with bridge engineers at the Connecticut Department of Transportation (CONNDOT). Through library research, the project team identified several parameters that affect the life-cycle cost of bridge bearings. For each of these parameters, the engineers at CONNDOT provided the project team with high, average, and low expected costs. Based on these values, the project team was able to apply bounds to the life-cycle costs associated with bridge bearings.

The investigation of the effect of bracing on lateral distribution of deck loads sought to determine how the inclusion of bracing members in an analytical model could affect the design of various superstructure components (primarily the deck and girders). The investigation assessed the affect of bracing members on reducing the maximum moment in longitudinal girder members and reducing the shear lag effect. To study these phenomena, a brief literature review was conducted, and a simplified finite element model was developed.

A finite element model of a typical bridge connection was developed. Three different modeling techniques were investigated; each one sought to provide a more realistic representation of the phenomena at work in a typical bridge connection, e.g. pretension, friction, etc. Although the more detailed models that were established to capture these phenomena did not produce accurate results, potential sources of error in the modeling process are identified, and alternative modeling strategies are proposed. The simpler modeling techniques investigated provide a basic description of the stress distribution in bridge connections.

The following five chapters provide a detailed methodology for the five investigations conducted in this project. Also, at the end of each chapter, conclusions are drawn from the study. The final chapter of this report presents conclusions drawn from a synthesis of all the studies conducted during this project. These conclusions provide the reader with an enhanced understanding of the behavior and design of bridges. The limitations of the work are discussed, and topics for further study are also presented.

IV Superstructure Design

This section presents the methodology followed to complete the superstructure design. This methodology consisted of sizing the structural members, and performing a cost analysis on all designs. The results of the designs, and the cost analysis are summarized at the end of the chapter.

Design Methodology

The superstructure design was based on a bridge that needed to span six highway lanes. A plan view of the highway the bridge needed to span can be seen in the figure below:

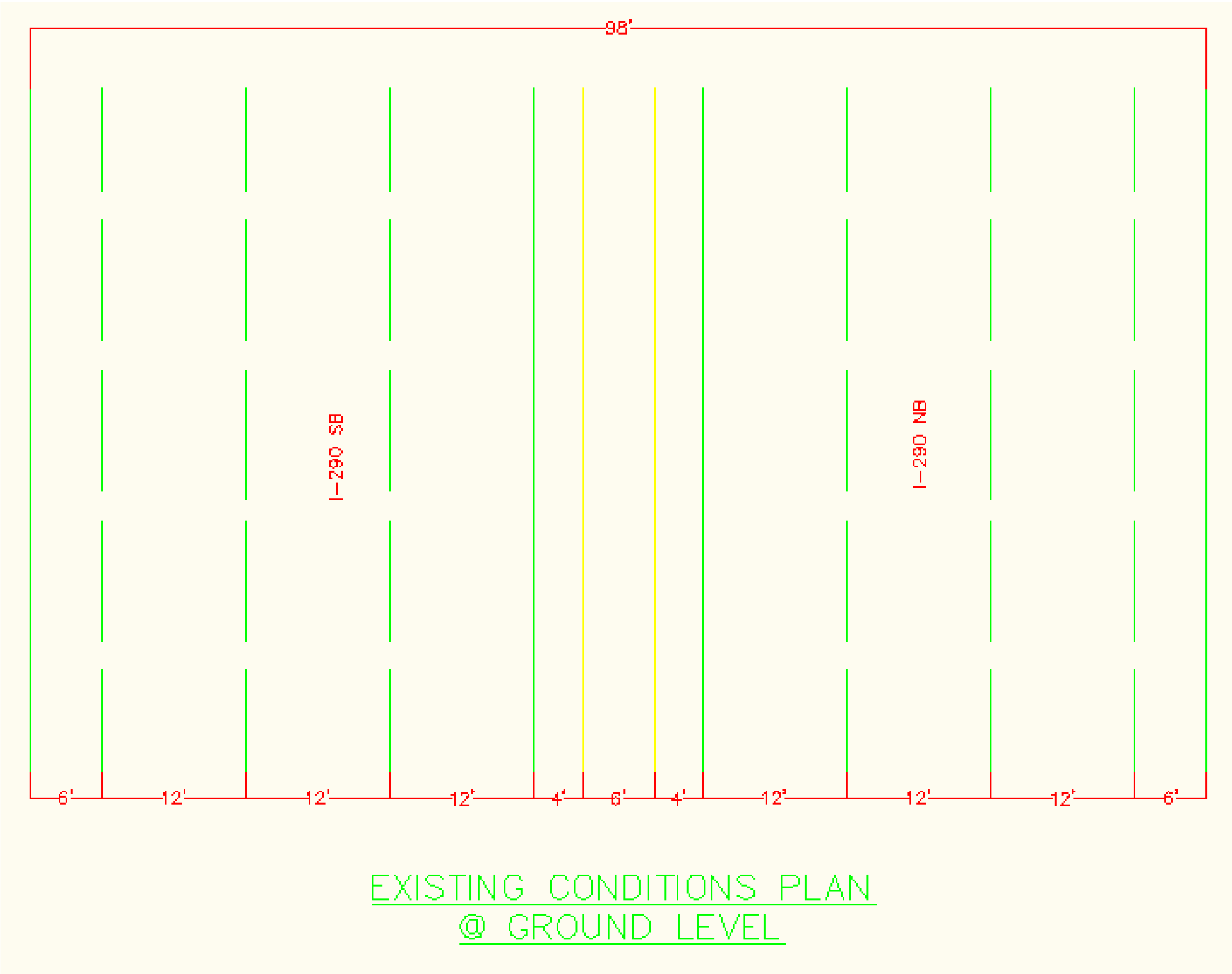


Figure 7: Plan of Highway to be Crossed

To determine the total length of the bridge, a clear space between the highway pavement and the top of the pier was assumed to be 20 feet. Also, a slope of (8/12) was assumed for the abutments. The following figure presents an elevation view of the bridge:

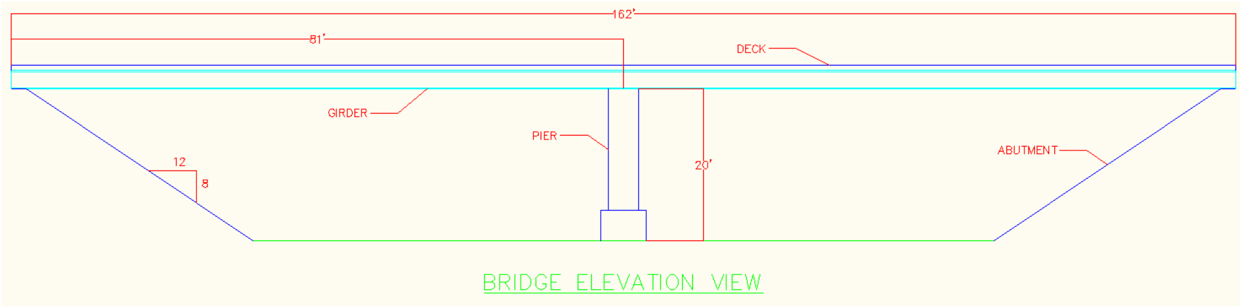


Figure 8: Bridge Elevation View

To determine the width of the bridge, it was assumed that the bridge would carry one lane of traffic in each direction. A three foot buffer zone was also made to allow room for sidewalks/parapets however, additional dead loads or stiffening effects from sidewalks or parapets were not considered in the design of the superstructure. The following figure presents a plan view of the bridge:

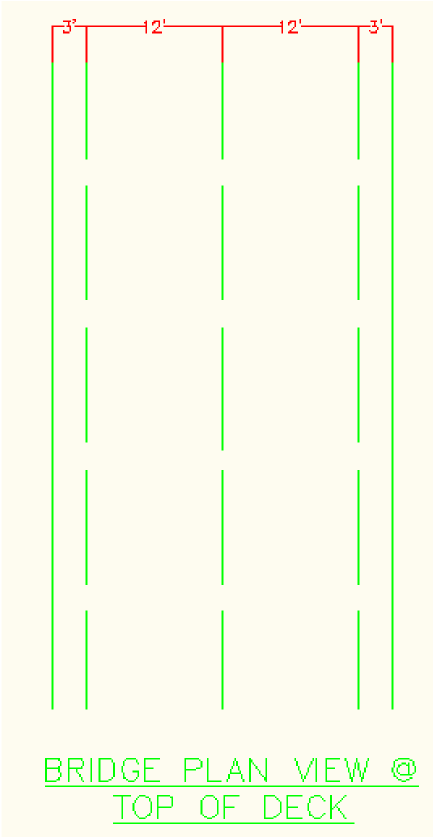


Figure 9: Plan View of Bridge at Top of Deck

The superstructure design consisted of two parts: deck design and girder design. The girders were designed using both hot-rolled steel sections and reinforced concrete sections. Three different design options were investigated to determine the effects of the superstructure layout on the bridge cost. The different options can be seen in the following three figures:

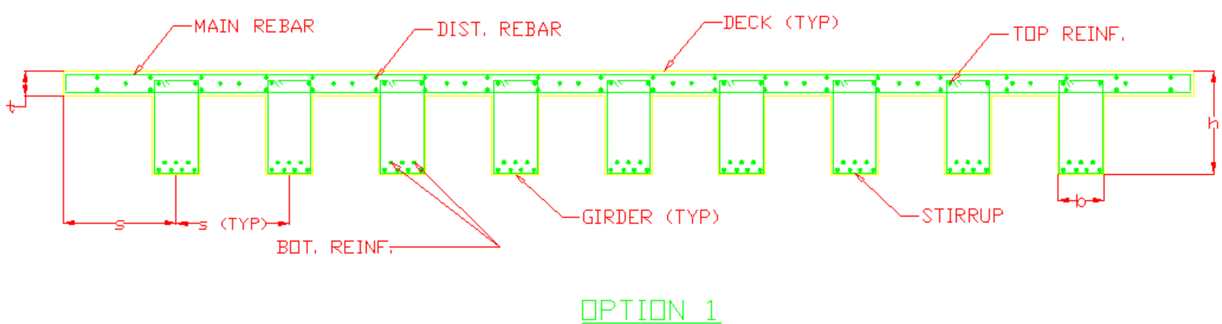


Figure 10: Design Option 1

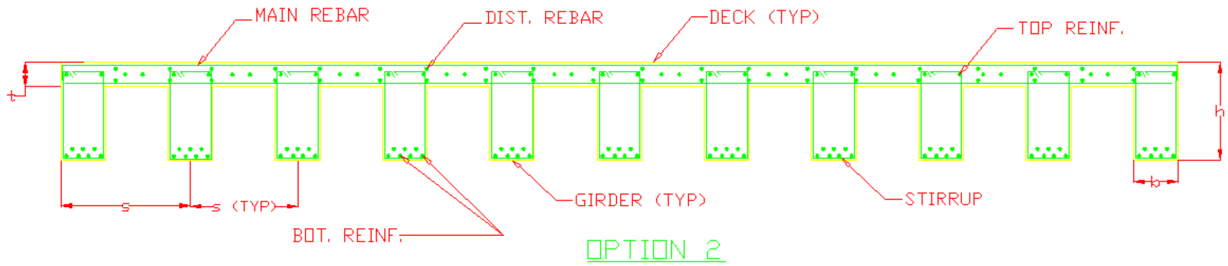


Figure 11: Design Option 2

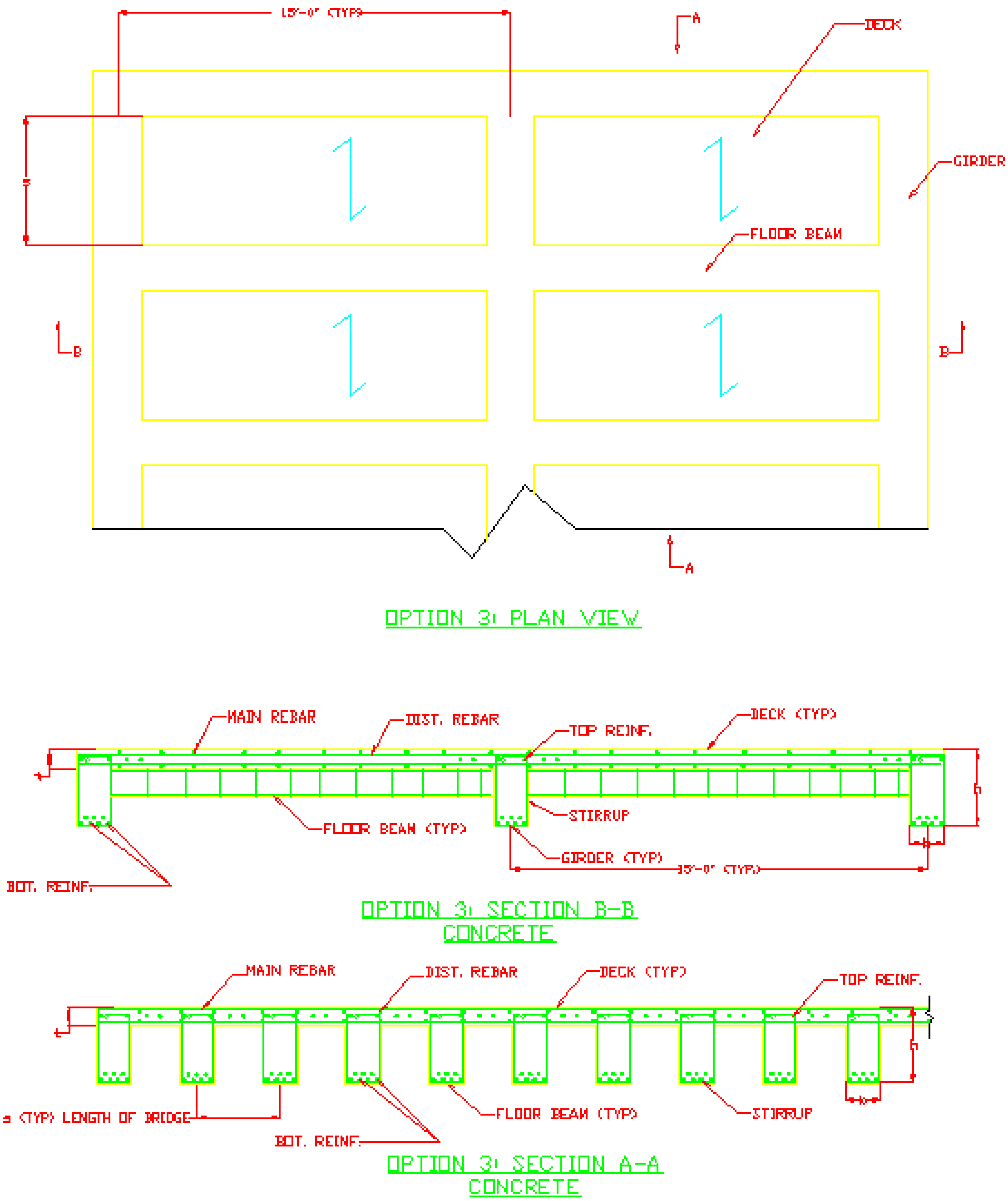


Figure 12: Design Option 3

Option 1 shows a deck cantilevered at each end. The deck spans in the transverse direction. Three girder spacings were used for this option. They were selected to ensure that all the girders could be placed at equal and regular intervals. Option 2 is similar to Option 1, except there are girders at the ends of the slab. Again, the girder spacing was chosen to ensure that the girders would be spaced at equal and regular intervals. The deck in Option 3 spans in the longitudinal direction, and is supported by floor beams spanning transversely, which are supported by girders spanning in the direction of the deck. The floor beam spacing was chosen to ensure that less than five percent of the applied load would be carried by the slab in the transverse direction. To do this, it was assumed that the percentage of load being carried by the slab in the transverse direction was equal to the beam spacing raised to the fourth power divided by the sum of the length of the slab span in the transverse direction (15 feet) raised to the fourth power plus the beam spacing raised to the fourth power. This helps to limit the slab to one way action. The table below summarizes the defining characteristics of each design alternative.

Table 1: Design Option Summary

Option No.	Description	Available Spacings
1	Slab spanning transversely; overhang at end of slab	3ft, 5ft, 7.5ft
2	Slab spanning transversely; no overhang	3ft, 5ft, 6ft, 7.5ft, 15ft
3	Slab spanning longitudinally; no overhang	3ft, 4.5ft

There were five primary goals during the superstructure design. These goals and the methods for achieving them are outlined below:

1. Investigate the advantages of using continuous girders
    a. Design simple span and continuous span superstructures using both steel and reinforced concrete girders; compare the economy of designs; design Options 1 and 2 only
2. Investigate the advantages of using composite sections
    a. Design both composite and non-composite systems using steel girders only;

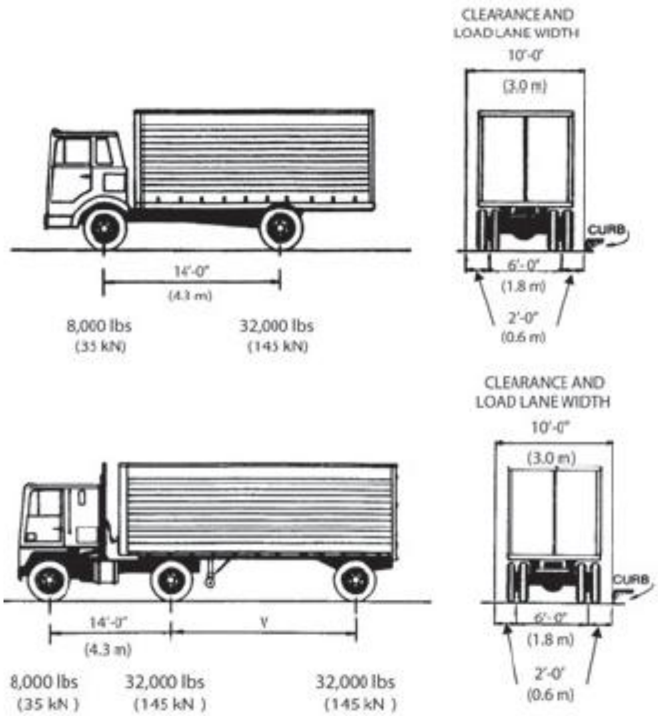
compare economy of designs; design Options 1 and 2 only

3. Investigate effect of material on design/economy
    a. Compare results from goals (1) and (2) for steel and concrete
    b. Design Option 3 in both steel and concrete
4. Investigate the effect of the slab spanning longitudinally versus transversely
    a. Compare the design results of Options 1 and 2 with the results of Option 3
5. Investigate the effect of having an overhang
    a. Compare the design results of Option 1 with Option 2

The comparison of design economy referenced in each of the goals above only refers to cost estimates of the superstructure design. Cost estimates for the substructure design are presented in the next chapter, "Substructure Design."

IV.1.1 Deck Design

To design the deck, several computer models were constructed using Risa-2D. These models represented the different superstructure options shown in the previous section. The computer model consisted of supports at the girder locations, a distributed dead load to represent the deck's self-weight, and a live load of two 32 kip axle loads to represent a truck traveling over the bridge (these represent the rear wheels of the AASHTO design truck shown in the lower half of the following figure). The AASHTO distributed live load was not applied for the design of the deck as permitted by AASHTO. The live load was applied as a moving load which moved at one foot increments along the bridge. This was done in order to determine the critical location of the design truck.



These sketches illustrate the AASHTO-approved live loading specifications for standard H20 and HS20 trucks.
Source: AASHTO Standard Specifications for Highway Bridges.

Figure 13: AASHTO Design Truck
http://www.tfhrc.gov/pubrds/05jul/images/jatrucks.gif

The "Strength I" limit state was analyzed, as it could be seen by inspection to be critical for the deck design. AASHTO does not specify an exact number to use for the dead load factor; 1.2 was chosen because it is used as a dead load factor in other design codes, and because it falls within the bounds specified by AASHTO. The figure below shows a free body diagram used for the deck design. This free body diagram was modified to suit the needs of the individual option being designed:

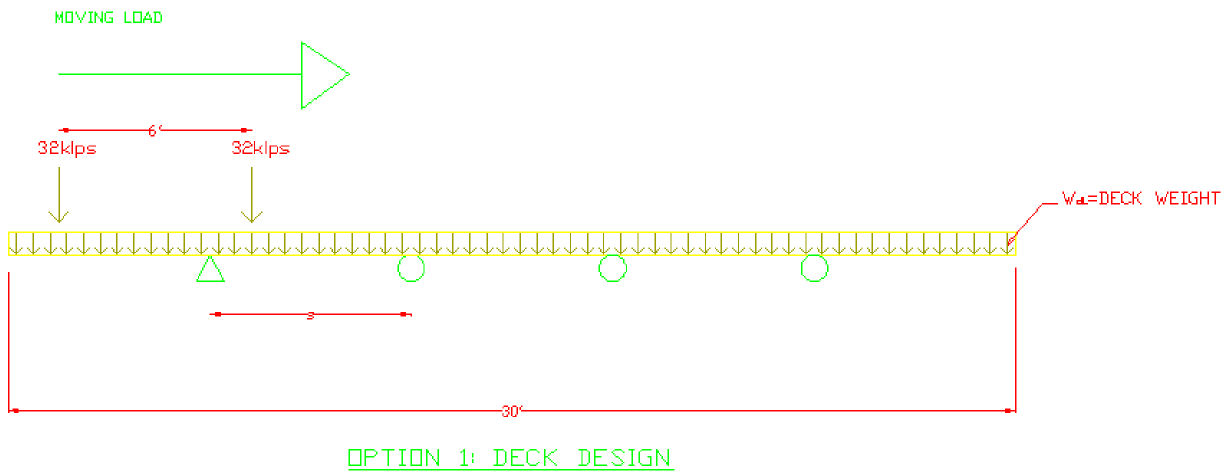


Figure 14: Typical Deck Free Body Diagram

Once the Risa model was solved for each option, the maximum positive and negative moments were recorded. Shear effects were not considered in the design as permitted by AASHTO. An Excel spreadsheet was developed which calculated the required positive and negative reinforcement, and can be found in Appendix B. The deck thickness was adjusted in order to ensure that the deck was tension controlled. AASHTO states that a member is tension controlled if the strain in the extreme tensile reinforcement is greater than 0.005. The required amount of main reinforcement was determined, and the required amount of distribution steel was computed as a percentage of the main reinforcement.

IV.1.2 Girder Design

The following paragraphs describe the design of Options 1 and 2 only. Option 3 will be explained later in the chapter as its design is fundamentally different from Options 1 and 2. The design began with a preliminary analysis. For statically determinate structures (simple spans), this analysis was done by hand; for statically indeterminate structures (continuous spans), this analysis was done in Risa-2D. The analysis consisted of two 81 foot long beams, with pin and roller supports at the pier and abutments. The girder's self-weight and the weight of the deck above it were applied as dead loads. A distributed live load was applied along the length of the girders, and the AASHTO design truck was also applied as a live load. In the girder models, all three of the axle loads shown in Figure 13 were applied. The truck on the bottom was the one used in the design, and the spacing of the rear axles (denoted as "V" in Figure 13) was varied to determine the maximum effect; spacings used were 14, 20, 25, and 30 feet. The following figure shows a typical free body diagram used to design the girders:

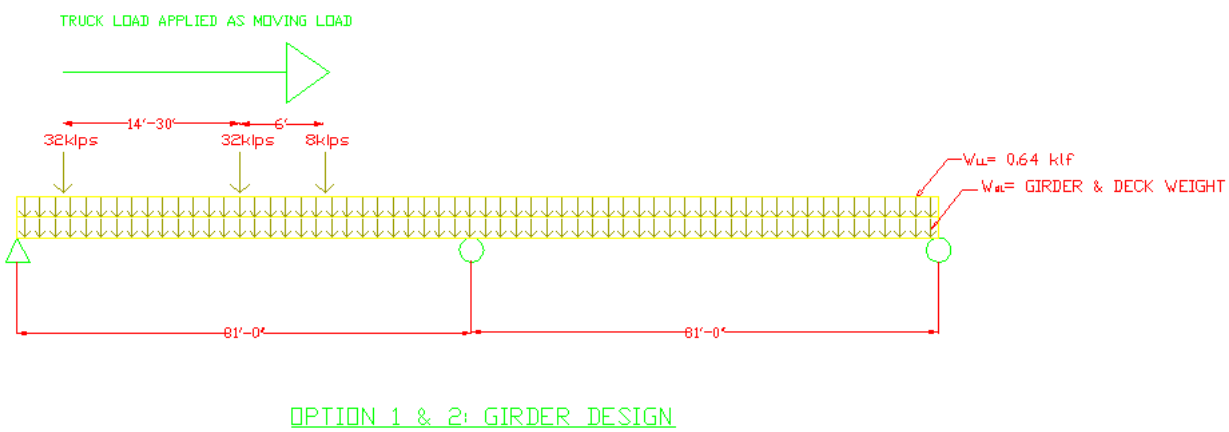


Figure 15: Typical Free Body Diagram for Girder Design

Once the Risa model was constructed, it was solved, and the maximum positive and negative moments, and maximum shears were recorded.

The next step was designing the girders; many different configurations were investigated. For steel girders in Options 1 and 2, simple span composite and non-composite sections were designed, as well as continuous span composite and non-composite sections. The following figure shows the basic procedure followed when designing the steel girders:

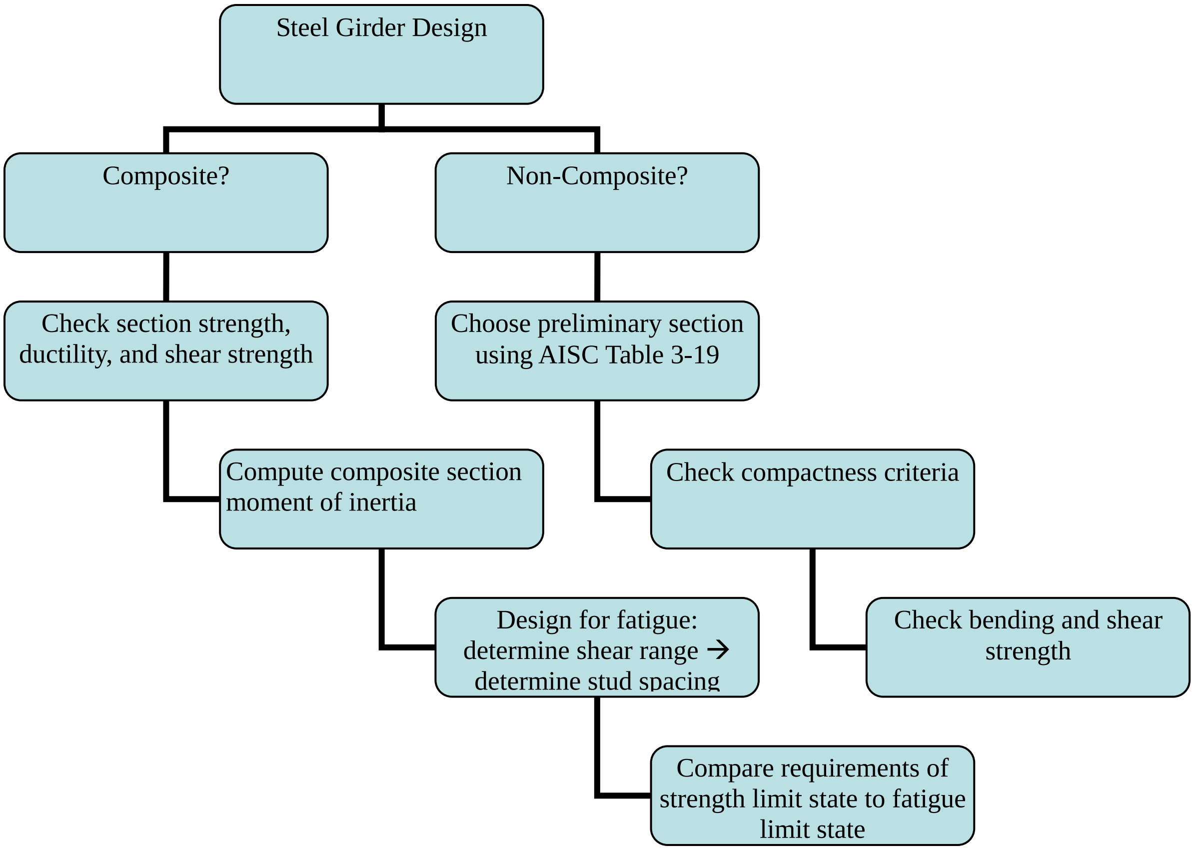


Figure 16: Steel Girder Design Procedure

The design of composite sections in regions of negative moment, which are present in continuous span bridges, were needed to complete the design. For this project these considerations were not taken into account in the design, but the methods for dealing with the situation were researched. There are two alternatives for dealing with composite action in regions of negative moment. The first alternative is to continue the shear reinforcement into the negative moment region. This will allow the bending steel to be used for computing the properties of the section. The other method is to stop the shear reinforcement before it enters the regions of negative moment. In this case the anchorage connectors need to be placed in the area of the point of inflection due to the dead load. If this method is used longitudinal steel cannot be placed in the region of negative moment (Chen, 2003).

Concrete girders were also designed. They were designed to be cast at the same time as the slab to achieve "t-beam" action. The girders were designed using simple and continuous spans for Options 1 and 2. In all cases the girders were designed to be tension-controlled. The following figure shows the basic procedure followed when designing the concrete girders:

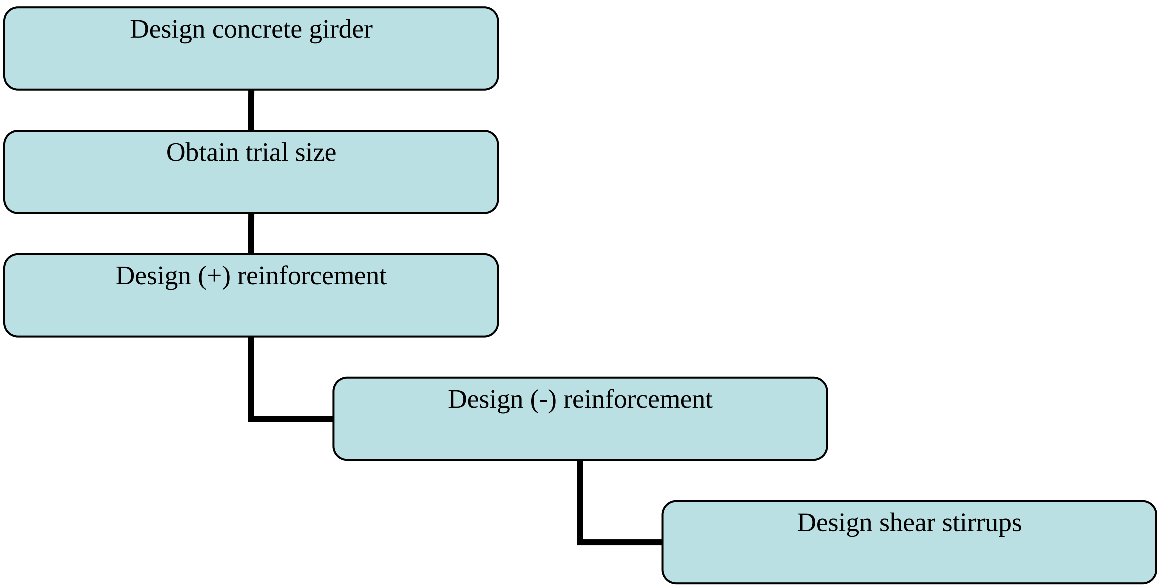


Figure 17: Concrete Girder Design Procedure

In order to correctly evaluate the results from the designs, one needs to consider cutting off reinforcement where it is not needed. In the design of concrete girders in this project, simple span girders, which are not subject to negative moment, only have two reinforcing bars on the top of the beam (provided as supports from which the shear stirrups can be hung), while the continuous girders have many reinforcing bars on the top. This could potentially cause the simple span girders to be more economical than the continuous girders. To determine how large of an impact these extra reinforcing bars have on the economy of the design, one must determine where certain bars can be cut-off in the different designs. Next, a cost estimate should be performed in order to evaluate if a more cost-effective design can be achieved. This investigation was not conducted in this project due to time constraints. Inaccuracies in the data should be minimal because specifying many different cut-off lengths for the negative moment reinforcement in the girders would decrease constructability and increase the amount of time required to place the rebar. This increase in erection time could potentially offset the savings from the decrease in required material.

Although the design of Option 3 followed some of the same guidelines as Options 1 and 2, there were some major differences, particularly in the load path through the superstructure. For design Option 3, two separate Risa models were created; first a model of the floor beam (the beams spanning transversely) was created. For steel floor beams, the beams were considered to be simply supported and exhibit composite action. For concrete floor beams, the beams were considered to be continuous and exhibit "t-beam" action. The floor beams were designed to carry their own dead weight, the weight of the deck, a distributed live load, and the design truck. Next, a Risa file was created to model the girders; the girders were designed to be continuous. The girder models consisted of a dead load representing their own weight, the factored reactions from the floor beam model applied as point loads along the length of the member, a distributed live load, and the design truck load moving across the member. The reactions from the floor beams were from an analysis only involving the dead load of the deck and the floor beam itself, and the point loads were not factored in the girder model. Once the models were created, the maximum shears and moments were recorded and a section was designed to resist the applied loads. The following figure shows the free body diagrams used to design the floor beams and girders:

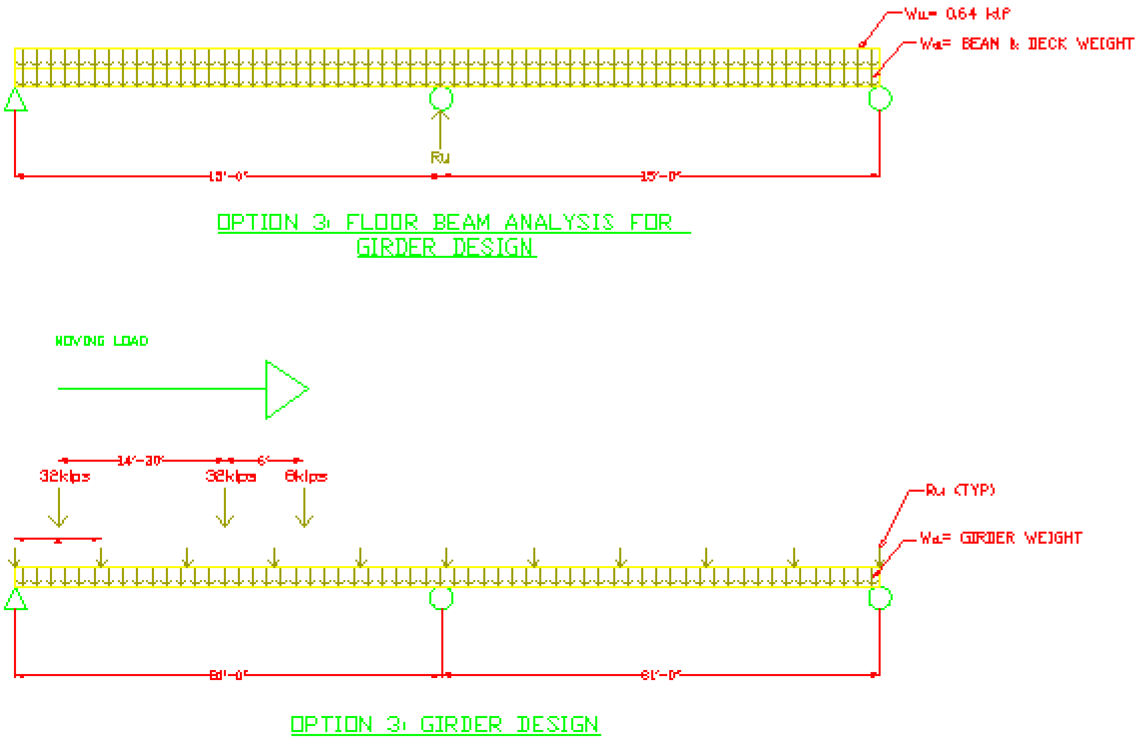


Figure 18: Option 3 Free Body Diagrams

IV.1.3 Cost Estimate

A cost estimate was made for each design alternative that was investigated. The cost data used was from Means Building Construction Cost Data 2006. Using cost data derived from building construction most likely introduced a certain amount of error into the cost estimate. However, the cost data was only meant to give a sense of proportion to material and labor costs. The main purpose of the cost estimate was to evaluate the different design alternatives by seeing if any of them were significantly less expensive than others.

To prepare the estimate, Excel files were created for each design alternative, each building material (steel or concrete), each span type (contiunuous or simple), and composite/non-composite sections. The volume of concrete, linear feet of reinforcement, and tonnage of structural steel were taken from the designs, and entered into the spreadsheet. A sample of the spreadsheet can be seen below:

Table 2: Sample Cost Estimate Sheet

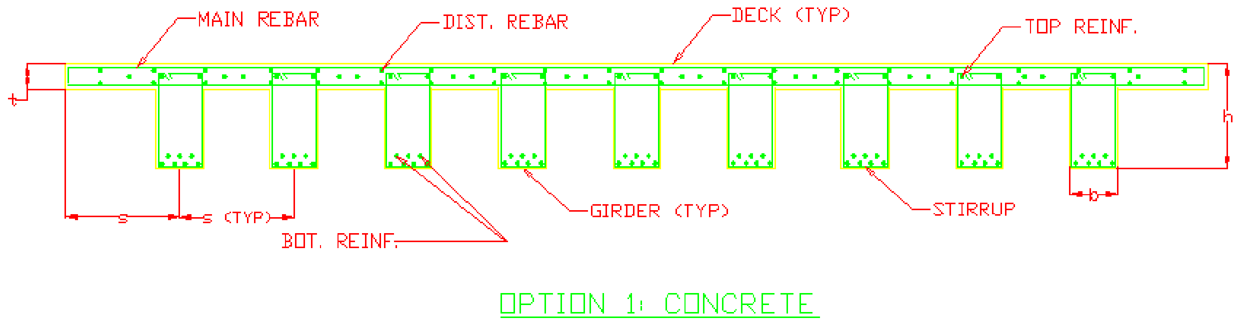
Option 3:								
Concrete:								
s (ft)	Deck Thick (in)	Vol Concrete (yd^3)	Adjust Waste (yd^3)	Conc $/yd^3	Cost Concrete ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	8	120	129.6	100	12960	7.06	39.44	276
4.5	10	150	162	100	16200	8.748	39.44	345
Main Top Reinforcement:								
s (ft)	Main Top	Main Top (lf)	Main Top (lb)	$/lf	Main Top Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#8 @12"	5022	13408	1.15	5775.3	95.418	53.15	5071
4.5	#6 @6"	9882	14843	0.56	5533.92	108.702	53.15	5778

Design Results

This section will present the design results for each option investigated, it will also present the results of the cost estimates.

IV.1.4 Option 1 Design Results

. The following figure shows the results for the design of Option 1 using concrete.

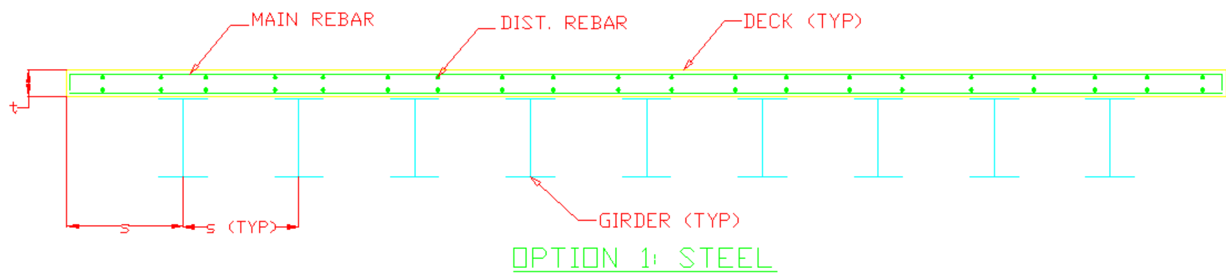


Simple Beams					
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)
3	20	#7 @12"	#10 @12"	#6 @12"	#9 @12"
5	26	#8 @12"	#11 @12"	#7 @8"	#10 @12"
7.5	34	#9 @12"	#14 @12"	#8 @12"	#11 @12"
s (ft)	bxh (in)	Top Rebar	Bot Rebar	Sitrrups	
3	40x70	2 #8	32 #8 (2layers)	126	
5	48x72	2 #8	40 #8 (2 layers)	170	
7.5	50x80	2 #8	46 #8 (2 layers)	203	
Continuous Beams					
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)
3	20	#7 @12"	#10 @12"	#6 @12"	#9 @12"
5	26	#8 @12"	#11 @12"	#7 @8"	#10 @12"
7.5	34	#9 @12"	#14 @12"	#8 @12"	#11 @12"
s (ft)	bxh (in)	Top Rebar	Bot Rebar	Sitrrups	
3	28x55	2 #8	14 #10 (2layers)	243	
5	30x59	2 #8	13 #9 (2layers)	203	
7.5	40x78	2 #8	8 #10 (2 layers)	122	

Figure 19: Option 1 Concrete Design Results

To interpret these results, look at what is being called out in the drawing, and look at the value given in the table. There are main reinforcing bars on the top and bottom of the deck. There also distribution bars on the top and bottom of the deck, although the drawing only shows one layer in order to make the information presented easier to read.

The following figure presents the results for the design of Option 1 using steel:



Simple Beams (non-composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	20	#7 @12"	#10 @12"	#6 @12"	#9 @12"	W44x262
5	26	#8 @12"	#11 @12"	#7 @8"	#10 @12"	W44x290
7.5	34	#9 @12"	#14 @12"	#8 @12"	#11 @12"	W40x503

Continuous Beams (non-composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	20	#7 @12"	#10 @12"	#6 @12"	#9 @12"	W40x199
5	26	#8 @12"	#11 @12"	#7 @8"	#10 @12"	W44x230
7.5	34	#9 @12"	#14 @12"	#8 @12"	#11 @12"	W40x503

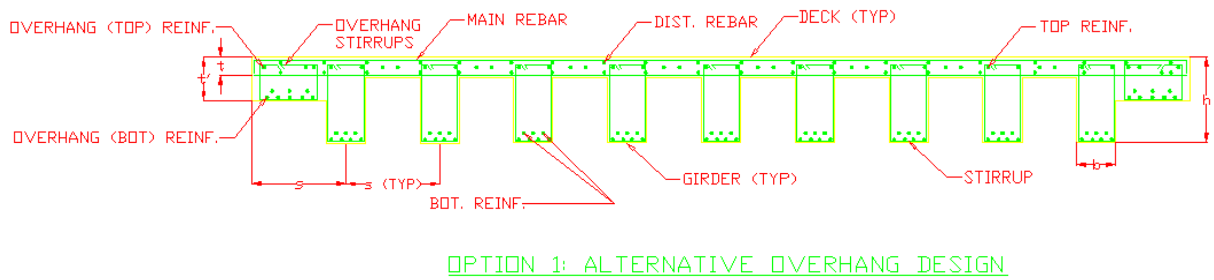
Figure 20: Option 1 Steel Design Results

Simple Beams (composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	20	#7 @12"	#10 @12"	#6 @12"	#9 @12"	W27x178(7776)
5	26	#8 @12"	#11 @12"	#7 @8"	#10 @12"	W24x131 (4148)
7.5	34	#9 @12"	#14 @12"	#8 @12"	#11 @12"	W27x146(1458)

Continuous Beams (composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	20	#7 @12"	#10 @12"	#6 @12"	#9 @12"	W24x104(3888)
5	26	#8 @12"	#11 @12"	#7 @8"	#10 @12"	W24x104(1620)
7.5	34	#9 @12"	#14 @12"	#8 @12"	#11 @12"	W27x178(1458)

Figure 21: Option 1 Steel Design Results (continued)

These results show that the deck slab must be very thick to support the applied loads; this is due to the large negative moment developed over the exterior girder. To decrease this moment, and therefore the deck slab thickness, an alternative design method was used for Option 1. In this method, the deck was designed to resist the maximum moment in the interior spans. Next, the overhang was designed to act like a girder spanning in the longitudinal direction of the bridge. This decreased the volume of concrete needed for the deck because most of the deck slab was made much thinner, only the overhang had a large thickness. The results for this alternative design approach can be seen in the figure below:



Deck Design					
s (ft)	t (in)	Main Rebat (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)
3	8	#9 @12"	#8 @12"	#8 @12"	#7 @12"
5	12	#9 @12"	#7 @6"	#8 @12"	#6 @6"
7.5	16	#8 @6"	#8 @6"	#7 @6"	#7 @6"
Overhang Design					
s (ft)	bxh (in)	Top Rebar	Bot Rebar	Sitrrups	
3	36x42	19 #8	17 #8 (2layers)	126	
5	60x36	22 #8	22 #8	170	
7.5	90x36	40 #8	28 #8	288	

Figure 22: Option 1 Overhang Alternative 1

Design results shown are for the deck and overhang only. The girders were not re-designed for the decrease in dead load caused by the thinner deck because of time constraints. It is likely that the girders would have decreased in size; their size would probably be comparable to the girders of Option 2 because the two systems were designed to carry loads in the same basic manner.

This alternative design method needs to be investigated further. Under this approach, the deck has double curvature over the exterior girder (due to the deck bending in different directions in this region.) The design must provide a way of preventing cracking in this region. One idea to prevent the cracking is to extend the reinforcing bars from the deck into the overhang. This is, however, only a preliminary thought. Also, the girders would need to be re-designed to support the new loading pattern that the overhang would create.

Another design alternative was developed for the design of the deck used in Option 1; it is shown in the figure below:

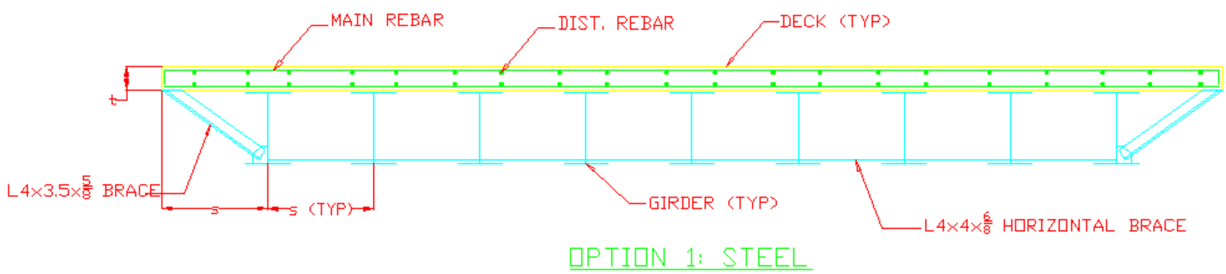


Figure 23: Braced Design Alternative

It is important to note that the angle sizes called out in the previous drawing are based on S=3 feet. In this design alternative, angle sections are used as brace elements to support the free end of the deck; the brace elements were designed to be spaced three feet apart. This allows for the overhang to be supported at its end and makes the deck act much more like the deck in Option 2. The same deck thicknesses could be used that were used for the design of Option 2. The horizontal brace elements shown in the figure were design to be placed in a typical “X-pattern”, as shown in the figure below:

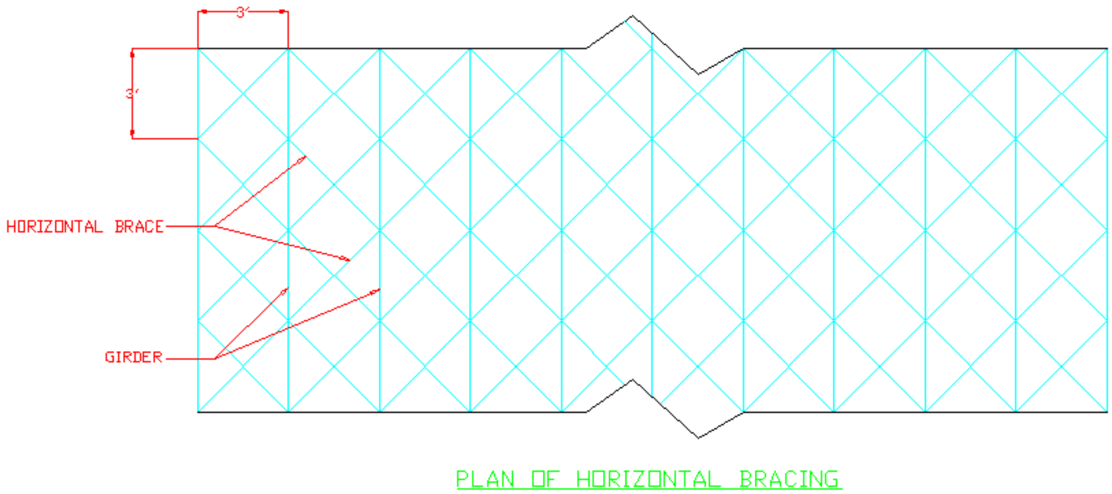


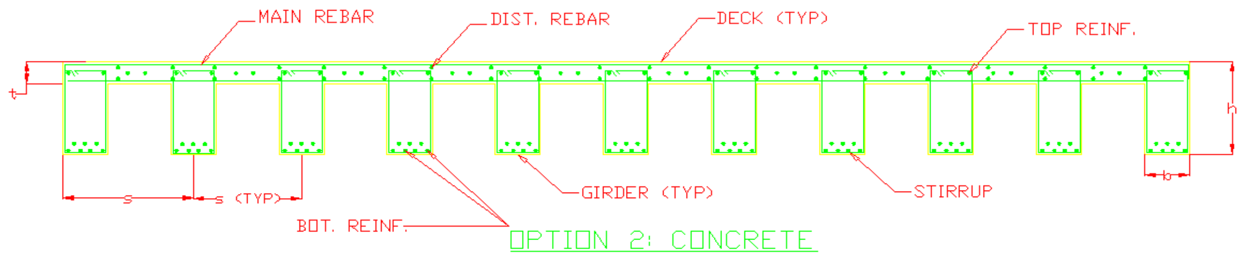
Figure 24: Horizontal Bracing for Deck Alternative 2

The horizontal bracing was provided to transfer the horizontal load from the bracing shown in Figure 23 to the piers or abutments.

This design alternative would need to be investigated further to be used in a real bridge design. An investigation of the cracking phenomena described for the first alternative would need to be conducted. Also, the girders would need to be re-designed to support the vertical load from the bracing element and the updated dead load caused by the thinner slab.

IV.1.1 Option 2 Design Results

The design results for Option 2 can be seen in the following figure:



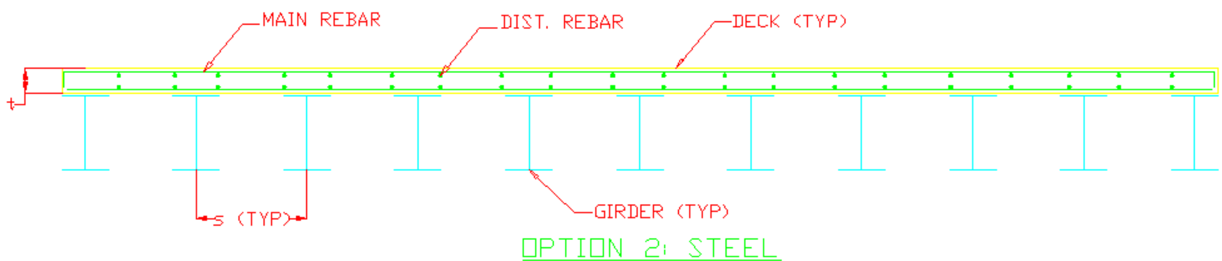
Simple Beams					
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)
3	8	#9 @12"	#8 @12"	#8 @12"	#7 @12"
5	12	#9 @12"	#7 @6"	#8 @12"	#6 @6"
6	14	#8 @6"	#8 @6"	#7 @6"	#7 @6"
7.5	16	#8 @6"	#8 @6"	#7 @6"	#7 @6"
15	22	#9 @6"	#9 @6"	#8 @6"	#7 @6"
s (ft)	bxh (in)	Top Rebar	Bot Rebar	Sitrrups	
3	24x52	2 #8	10 #11	81	
5	24x40	2 #8	12 #11	194	
6	36x34	2 #8	14 #11	139	
7.5	36x32	2 #8	16 #11	194	
15	36x38	2 #8	20 #11	278	

Continuous Beams					
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)
3	8	#9 @12"	#8 @12"	#8 @12"	#7 @12"
5	12	#9 @12"	#7 @6"	#8 @12"	#6 @6"
6	14	#8 @6"	#8 @6"	#7 @6"	#7 @6"
7.5	16	#8 @6"	#8 @6"	#7 @6"	#7 @6"
15	22	#9 @6"	#9 @6"	#8 @6"	#7 @6"
s (ft)	bxh (in)	Top Rebar	Bot Rebar	Sitrrups	
3	24x40	10 #10	8 #11 (2 layers)	194	
5	24x40	10 #11	8 #11 (2 layers)	243	
6	24x34	14 #10	14 #10 (2 layers)	278	
7.5	24x32	14 #11	12 #11 (2 layers)	324	
15	36x38	18 #11	18 #11 (2 layers)	389	

Figure 25: Option 2 Concrete Design Results

These results show that a much thinner deck slab can be used compared to the required deck thickness of Option 1 (not revised to decrease deck thickness). Also, the girder sizes are smaller than those in Option 1. When comparing the cost of each design, it will be important to remember that there are two more girders for each spacing in Option 2. This idea was a main driving force in the development of Option 1 and Option 2; the original investigation was supposed to be to discover if the hypothesized larger girder sizes required for Option 1 would still end up being less expensive than the hypothesized smaller girder sizes of Option 2, because Option 1 would have less girders. This investigation did not work out as well as was hoped because the section sizes for Option 1 were very large, adding large amounts of dead weight to the structure and skewing the results.

The following figure shows the design results for Option 2 using steel girders:



Simple Beams (non-composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	8	#9 @12"	#8 @12"	#8 @12"	#7 @12"	W40x215
5	12	#9 @12"	#7 @6"	#8 @12"	#6 @6"	W44x230
6	14	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W44x262
7.5	16	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W44x262
15	22	#9 @6"	#9 @6"	#8 @6"	#7 @6"	W36x487
Continuous Beams (non-composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	8	#9 @12"	#8 @12"	#8 @12"	#7 @12"	W40x149
5	12	#9 @12"	#7 @6"	#8 @12"	#6 @6"	W36x160
6	14	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W40x199
7.5	16	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W40x215
15	22	#9 @6"	#9 @6"	#8 @6"	#7 @6"	W40x372
Simple Beams (composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	8	#9 @12"	#8 @12"	#8 @12"	#7 @12"	W36x160(2074)
5	12	#9 @12"	#7 @6"	#8 @12"	#6 @6"	W40x167(4147)
6	14	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W40x167(4147)
7.5	16	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W40x183(4147)
15	22	#9 @6"	#9 @6"	#8 @6"	#7 @6"	W40x215(4860)

Figure 26: Option 2 Steel Design Results

Continuous Beams (composite)						
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)	Girder Size
3	8	#9 @12"	#8 @12"	#8 @12"	#7 @12"	W33x118(3402)
5	12	#9 @12"	#7 @6"	#8 @12"	#6 @6"	W33x130(4860)
6	14	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W33x130(4860)
7.5	16	#8 @6"	#8 @6"	#7 @6"	#7 @6"	W33x141(4374)
15	22	#9 @6"	#9 @6"	#8 @6"	#7 @6"	W33x221(5184)

Figure 27: Option 2 Steel Design Results (continued)

IV.1.2 Option 3 Design Results

The following figure shows the design results for Option 3 using concrete girders:

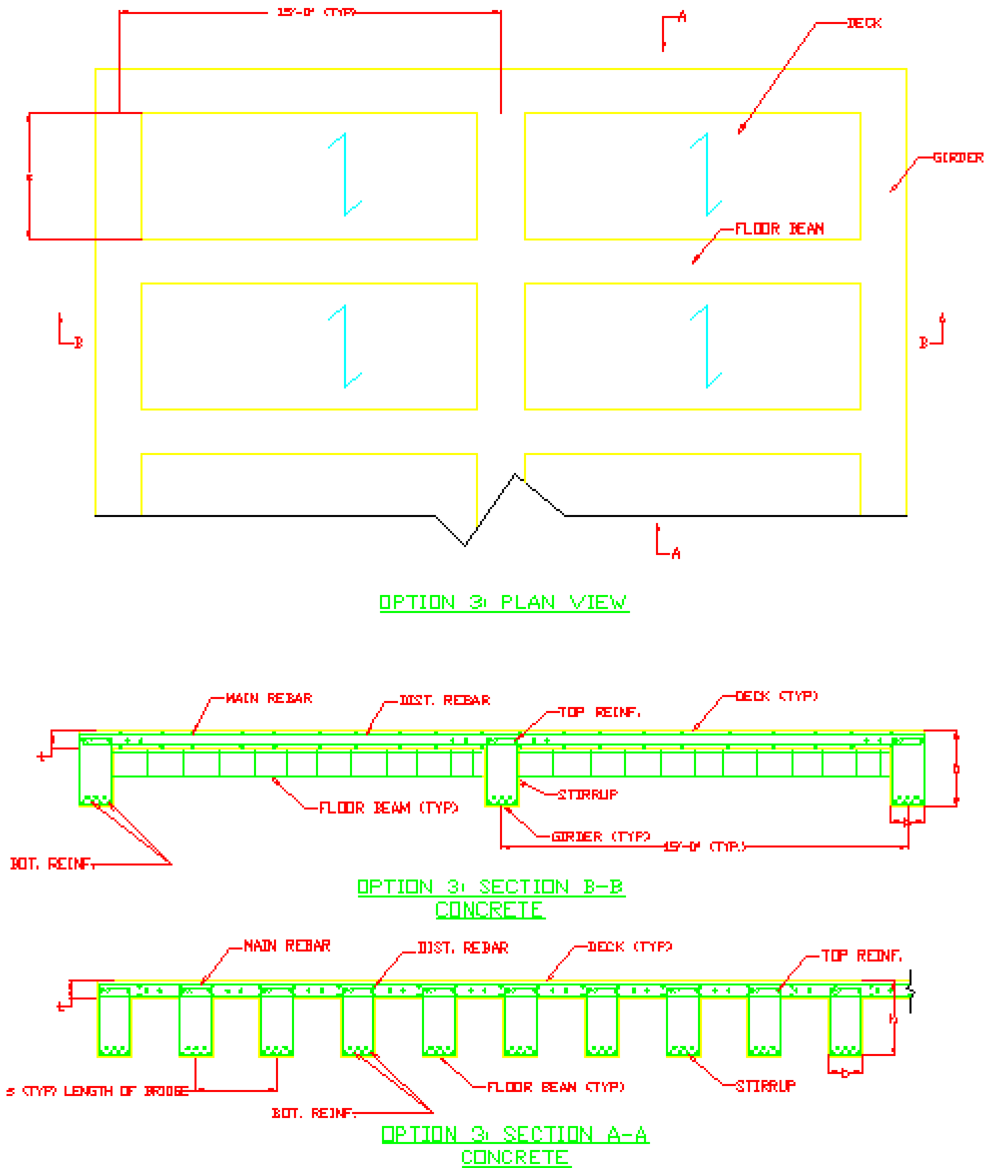
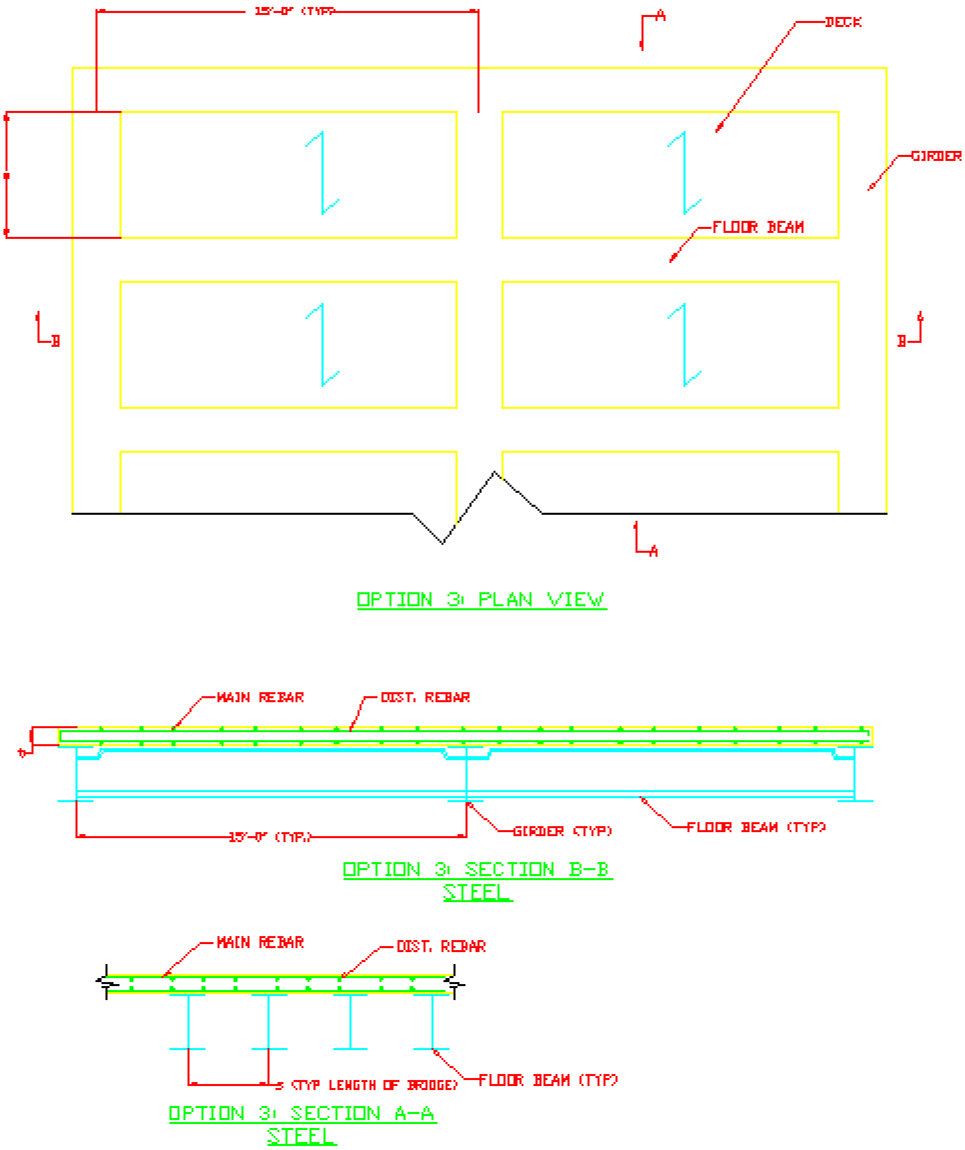


Figure 28: Option 3 Concrete Design Layout

Table 3: Option 3 Concrete Design Results

Deck Design					
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)
3	8	#7 @12"	#8 @12"	#5 @12"	#8 @12"
4.5	10	#8 @6"	#6 @6"	#6 @12"	#6 @12"
Floorbeam Design					
s (ft)	bxh (in)	Top Rebar	Bot Rebar	Sitrrups	
3	15x12	6 #7	4 #8	68	
4.5	26x7	6 #8	7 #8	68	
Girder Design					
s (ft)	bxh (in)	Top Rebar	Bot Rebar	Sitrrups	
3	40x55	38 #8	18 #8	243	
4.5	50x77	62 #8	31 #8	446	

The following figure shows the design results for Option 3 using steel girders:



Deck Design					
s (ft)	t (in)	Main Rebar (Bot)	Main Rebar (Top)	Dist. Rebar (BOT)	Dist. Rebar (Top)
3	8	#7 @12"	#8 @12"	#5 @12"	#8 @12"
4.5	10	#8 @6"	#6 @6"	#6 @12"	#6 @12"

s (ft)	Floorbeams	Girders
3	W16x45(7776)	W36x800
4.5	W18x46(3888)	W36x800

Figure 29: Option 3 Steel Design Results

In the design of the girders for Option 3, the composite action was not assumed. This is because of the possibility of a sizeable gap between the top flange of the girders and the bottom of the deck.

IV.1.3 Cost Analysis Results

It is difficult to compare the cost of Option 3 to Options 1 and 2 by simply comparing the specified section sizes because the structural systems are completely different. The cost analysis however, provides an objective method that takes into account both the total number of required members, and member sizes and cross sections. The cost analysis is discussed in the following paragraphs.

The following table summarizes the total cost of each design. It should be noted that the cost estimate for Option 1 is for the original design only; the costs of the alternatives involving the "girder overhang" or "braced overhang" were not analyzed.:

Table 4: Cost Analysis Summary

Cost	Steel Continuous Composite	Steel Simple Composite	Steel Continuous Non-Composite	Steel Simple Non-Composite	Concrete Continuous	Concrete Simple
Option 1 S=3’	$267,200	$386,600	$397,200	$488,300	$247,500	$220,000
Option 1 S=5’	$291,000	$253,300	$322,300	$370,500	$225,500	$214,400
Option 1 S=7.5’	$271,800	$256,400	$424,800	$424,800	$246,100	$237,600
Option 2 S=3’	$274,600	$345,400	$320,600	$437,400	$168,000	$194,000
Option 2 S=5’	$231,200	$271,000	$252,500	$331,300	$143,400	$159,300
Option 2 S=6’	$236,000	$270,000	$290,100	$350,900	$169,000	$172,000
Option 2 S=7.5’	$226,000	$259,100	$274,200	$312,100	$161,900	$177,000
Option 2 S=15’	$250,700	$247,000	$310,200	$365,700	$188,600	$187,500
Option 3 S=3’	$485,400	-	-	-	$367,500	-
Option 3 S=4.5’	$499,700	-	-	-	$246,300	-

denotes simple is less expensive than continuous
denotes Option 1 is less expensive than Option 2
denotes both blue & red criteria are met

The highlighted cells in the table show design options that did not follow the trend that was expected when the research goals mentioned earlier in this chapter were developed. For example, it was expected that simple spans would be more expensive than their continuous counterpart because continuous spans have smaller absolute values of moment.

These results show that Option 2 with a five foot girder spacing, using continuous reinforced concrete girders yields the most cost-effective design. The composite sections were always less expensive than their non-composite counterparts. Although all of the concrete designs were less expensive than their steel counterparts, the designs for these concrete sections pose constructability issues, as will be discussed in the following sections. Also in the following sections will be a discussion of how the cost analysis answered the research questions proposed earlier in this chapter.

IV.1.4 Investigate Advantages of Continuous Span Girders

In general, simple spans yielded a less cost-effective design than their continuous counterparts. There were a few anomalies. First, the Option 2 steel girder design when spacing was 15 feet required many more shear studs than the simple counterpart, causing it to be a more expensive option. This is most likely due to the geometry of the composite deck/girder section, and can be regarded as an outlier in the data. The concrete continuous sections are more expensive than their simple span counterparts because they required more negative moment reinforcement and more shear stirrups. This highlights the importance of using detailed design methods. As mentioned earlier, cutting off extra negative moment reinforcing steel in regions of positive moment could potentially decrease the cost of construction. Also, varying the shear stirrup spacing during the design phase would allow for fewer shear stirrups near the bridge abutments than at the bridge piers, because the shear force is lower at the abutments than the piers. It should be noted however that these more detailed designs would be less constructible and could increase the chances of a construction error taking place on the job site. Despite the fact that these extra design steps were not taken, most of the continuous concrete spans were less expensive than their simple span counterparts. Based on these results, one can reasonably conclude that continuous spans are generally more economical than simple spans.

IV.1.5 Investigate Advantages of Composite Sections

The cost analysis results clearly show that using composite steel sections can greatly reduce the cost of the superstructure. Because the design of composite sections is more involved than the design of a non-composite section, it is important for the designer to ensure that he or she is aware of all the design considerations associated with composite sections. One example is the design of composite sections in regions of negative moment, as was described earlier in this chapter.

IV.1.6 Investigate the Economy of Different Construction Materials

The designs that used reinforced concrete girders were far more economical than their steel counterparts. This is most likely due to the fact that concrete material and labor costs are generally lower than those for steel. One can reasonably conclude this because the deck cost is the same for any given option and spacing regardless of the material for the girder. This means that the cost of the designs is governed by the girder.

Although reinforced concrete provided the most cost-effective design for all of the options, cost should not be the only consideration in choosing a final design scheme. Many of the reinforced concrete designs call for very large girders, some of which may be so large that they are impractical to construct. Large concrete sections can be subject to thermal cracking during the curing process. This is because at the center of the section, the temperature can be high due to the chemical reaction taking place, but at the edges of the section, the temperature is generally lower; this temperature gradient can cause cracking. To avoid cracking due to temperature gradients in a concrete section, the American Concrete Institute (ACI) has provided guidelines for poring and curing large concrete sections; the technique is called "Mass Concreting." The guidelines are available in ACI 211.1-81 "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete."

ACI's guidelines for mass concreting do not appear to be directly applicable to bridge construction; they are more generally provided for the construction of dams. For example, ACI recommends the use of very large aggregate (up to six inches in diameter) in the concrete mix to decrease the amount of cement required and therefore decrease the heat given off during the curing process. This would not be a viable option in bridge construction because of the large amount of reinforcing steel required in the sections. ACI also recommends the use of Type IV Portland Cement; this type of cement undergoes the chemical reaction that takes place during the curing process much more slowly than typical cement, and therefore a smaller temperature gradient is produced. However, Type IV cement is not readily available, and could be very costly (Kerkhoff, Kosmatka, and Panarese, 2002). One final option is the use of a system that delivers cooling water to the center of the section through a hose of some sort. This option would most likely be very expensive and time consuming to assemble.

Based on the constructability issues that would be associated with constructing a bridge with reinforced concrete listed above, it is recommended that the material not be used for girders in bridge construction, at least when the span being design is comparable to the span investigated in this report. Perhaps if the span was shorter, the section sizes would be more reasonable. Also, pre-stressed concrete could be a viable alternative. In pre-stressed concrete, higher strength concrete and reinforcing steel are generally used. Also, sometimes more efficient, standard "I-Beam" sections are used, which would decrease the required section sizes.

IV.1.7 Investigate the Effect of the Deck Spanning Transversely vs. Longitudinally

The effect of the direction in which the deck spans can be evaluated by comparing the results for Option 3 to the results for Options 1 and 2. These results clearly show that when the deck spans transversely, a more cost-effective design can be achieved. This is most likely not a direct result of the direction in which the deck spans. Instead it is the result of an inefficient layout for transferring the load to the substructure. Option 3 required many more beams than the other options because it required so many floor beams. These extra floor beams caused the cost of the girder/floor beam material and girder/floor beam labor for Option 3 to be nearly double the cost of Options 1 and 2.

It should also be noted that the longitudinal girder sizes chosen for Option 3 are much larger than those chosen for Option 1 and 2. This is possibly due to the large spacing of the girders in Option 3. Perhaps if a smaller girder spacing were used, the layout of Option 3 would become more cost-effective. In fact, the layout of Option 3 could become a much more desirable alternative if the floor beams were able to be treated as floor beams and bracing members, which would help to decrease the maximum girder moment. This idea will be more fully developed in

Chapter VII of this report.

IV.1.8 Investigate the Effect of Having an Overhang

The results of the cost analysis show that having an overhang (Option 1) is generally not as cost-effective as not having an overhang. There were a few design options where the design for Option 1 was less expensive than Option 2 (it should be noted that all of these options were composite sections). This is due to the fact that Option 1 had a much thicker deck than Option 2. The increased deck thickness allowed for a smaller girder section to be used because the thick deck was capable of resisting large amounts of load. This is not a good analysis of results however because the deck thickness specified in the unrevised design for Option 1 is not realistic. It would add a large amount of unnecessary dead weight to the bridge superstructure, and it may experience problems due to thermal cracking. Because of these constructability concerns, the unrevised design for Option 1 does not appear to be a viable option.

The alternatives for Option 1 described earlier in this chapter also have setbacks. The first alternative, in which the overhang acts like a girder spanning in the longitudinal direction, would most likely experience major cracking over the exterior steel/concrete girder. The braced alternative could work; however, the exterior girder would need to be re-designed to resist the vertical component of force being induced by the brace. Also, in the regions between the bracing members, the deck would most likely behave like a short beam spanning in the bridge's longitudinal direction and experience cracking in the same manner as the girder-overhang option. It can therefore be concluded that these options need more consideration before they could be used in an actual design.

The simplest design for a bridge with a layout similar to the one used for Option 1 of this report would be to install a crashworthy barrier over the exterior girder. According to the AASHTO specification, it is permitted to not place the design truck load in regions protected by crashworthy barriers. If a barrier were used, no design truck load would need to be considered over the actual overhang, which would decrease the moment over the exterior support; ultimately the deck would be resisting loads similar in magnitude to those for Option 2. The use of a barrier could only be considered for small girder spacings; for example, if a barrier were used on Option 1, 7.5 foot spacing, there would only be 15 feet of room for vehicular traffic to pass over the bridge; half of the area taken up by the bridge would not be accessible to vehicles.

Remarks

This section presented the methodology followed to complete the superstructure design and summarized the results. It was concluded that simple spans are generally less economical than continuous spans, and that composite sections are less expensive than non-composite sections. It was also shown that the reinforced concrete designs provide more cost-effective designs than the steel superstructures. However, the large concrete sections that would be required to resist the applied loads would introduce constructability concerns. A comparison of the cost analyses for Option 1 and 2 to Option 3 shows that Option 3 is far less economical. This is due to the fact that Option 3 carried load in an inefficient manner, requiring many floor beams. Option 3 could be a more cost-effective option if smaller girder spacing was considered. Finally, it was concluded that for bridges with large overhangs, the installation of a crashworthy barrier over the exterior girder would most likely be required to keep member sizes reasonable. These conclusions provide some basic guidelines for developing an initial layout of the structural system for a girder bridge. More precise and standardized design methods could be applied to the concepts introduced in this chapter to develop the design of an actual bridge.

V Substructure Design

This section presents the methodology followed in the substructure design. This includes the design of the piers and the abutments. This section will also contain the results of the substructure design along with life cycle cost analyses for both parts.

Pier Design

This section will present the design of the bridge pier. It includes two different designs as well as life-cycle cost analyses for both designs. The designs and analyses will be summarized.

V.1.1 Design Background

The pier of the bridge was designed by exploring two different, alternative designs. The first pier consisted of four separate columns, while the second was a single rectangular column. The two designs were decided upon because they are the most common type of piers for highway overpasses. The two piers were compared based upon their initial costs and their life-cycle costs. Multi-column piers generally have a lower initial cost because less material is required and construction is simpler. However, the single leg usually has a lower life-cycle cost because there is less surface area to be affected by the elements. Both piers were designed using the LRFD example found at the US Department of Transportation Federal Highway Administration website. Also the pier was designed by referencing the AASHTO LRFD Bridge Specifications. The appropriate limit states were used in the design of each element of the bridge pier. These limit states are summarized in the following table.

Table 5: Limit states of different pier components

Pier Component	Limit States
Cap	Strength I, Service I
Column	Strength I, Strength III, Strength V
Footing	Strength I, Strength III, Strength V

A sketch of the two pier designs can be seen in the figure below:

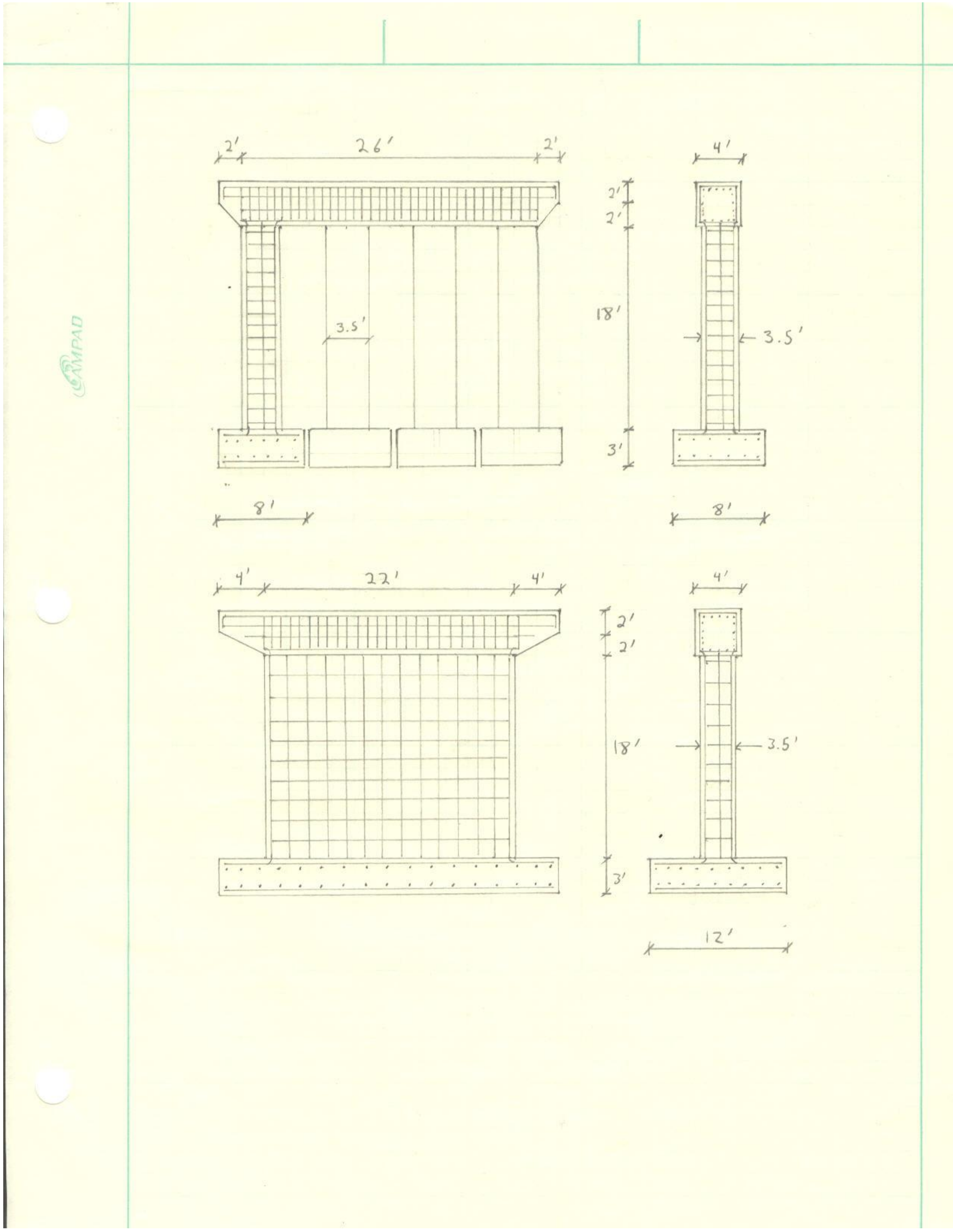


Figure 30: Pier Design Sketch

Both designs have approximately the same cap. It is 4 feet deep by 4 feet wide and spans 30 feet. The multi-column design has four 3.5 feet diameter, round columns while the single leg pier has one large 3.5 feet by 22 feet, rectangular column. The multi-column pier also has four separate footings, one for each column. The single leg pier has a single large footing.

V.1.2 Design Methodology

To design the piers the dead load, the live load, the wind loads, and the braking force were calculated. The dead load had contributions from both the superstructure and the substructure. The superstructure chosen for the design was the composite design with 3 foot girder spacing. This system was chosen because it has one of the largest dead loads to be applied to the pier since it has more girders. This means that most of the other options investigated for the superstructure should work if placed upon the piers that are designed from this analysis. The 3 foot girder spacing also places a fairly even load distribution across the width of the pier because it has eleven contact points to divide the loads between. The maximum live load on the pier was determined by first finding the load from AASHTO's design truck, which occurred when the truck was positioned over the pier. To determine the live load on the pier using the AASHTO Specifications the number of design lanes needed to determined. The number of design lanes is equal to the integer part of the bridge width divided by 12 feet. In this particular case the ratio was 30/12 = 2.5. Therefore there were two design lanes. It was then assumed that a truck load would occupy each of the design lanes. They were spaced two feet apart starting from one side of the bridge. The maximum force on any girder resulting from this loading was applied to all eleven girders.

A typical wind load was applied to both the superstructure and the substructure as a pressure distribution. This caused two different forces on the pier. By varying the angle at which the wind hit the bridge a maximum wind load on the superstructure and the substructure was determined. The wind load also had an effect on the live load. It will move the vehicle loads as they are crossing the bridge causing another type load that needs to be added to the design. Both the wind load on the superstructure and the wind load on the live load resulted in a force on the substructure due to the friction forces between the live load and the superstructure and the connections between the superstructure and the substructure.

The braking force was determined as a force that acts six feet above the pier. From this, a moment that was applied where the girders connect to the pier was calculated. The magnitude of the braking force is the least of a series of four equations that involve the truck load. These equations are given by the AASHTO Specifications.

With all of these forces, the appropriate load combinations for each limit state were applied. This gave the maximum moments, shears, and torsions acting on each part of the pier using the equations from the Federal Highway Administration's website. The cap was designed for Strength I and Service I, while the columns and footings were each designed for Strength I, Strength III, and Strength V. Each limit state has two different load combinations, one is a maximum and one is typical. The cap and the column were designed for the typical load combination, and the footing was designed with the maximum load combination. This is done to ensure that the footing will be able to adequately withstand two-way shear. The column and the cap do not need this consideration. The design process for each piece of the pier is summarized in the three flows charts shown below.

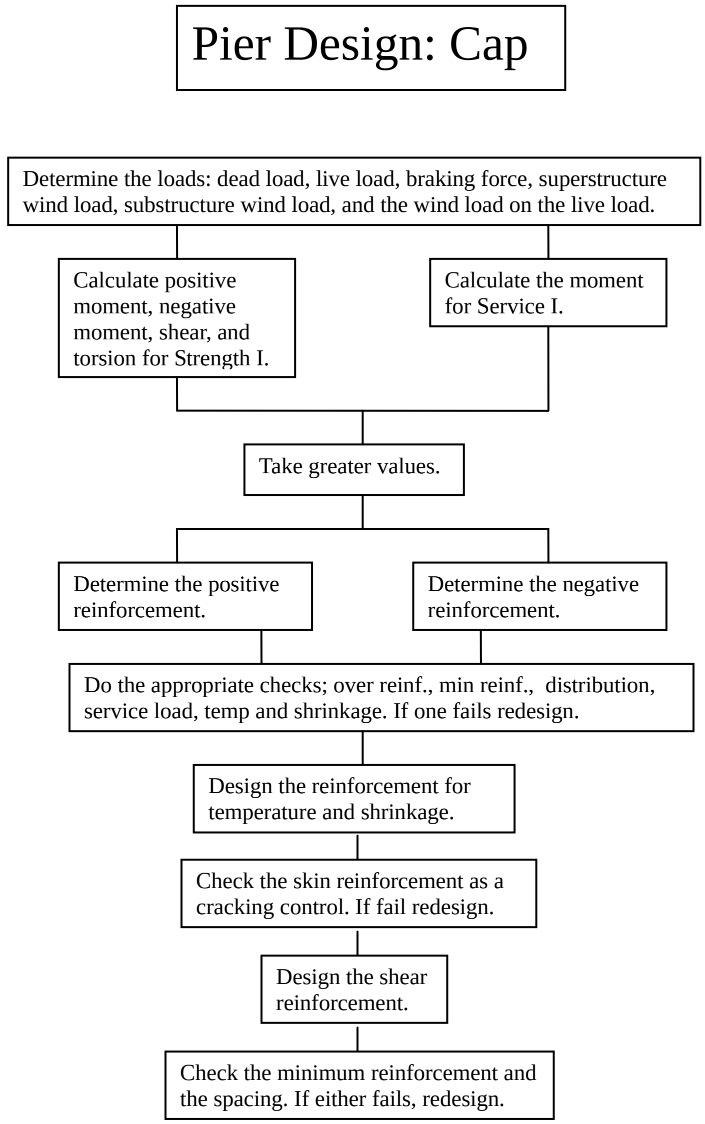


Figure 31: Cap Design Flow Chart

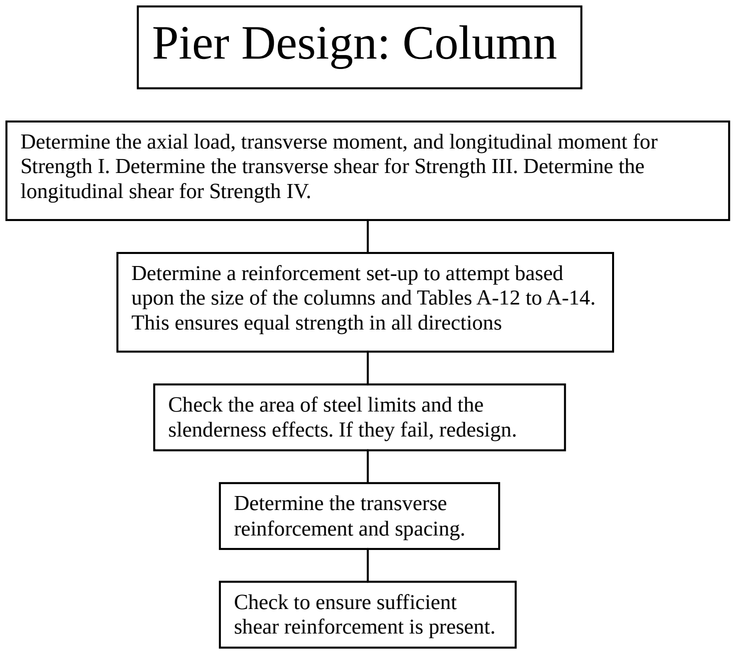


Figure 32: Column Design Flow Chart

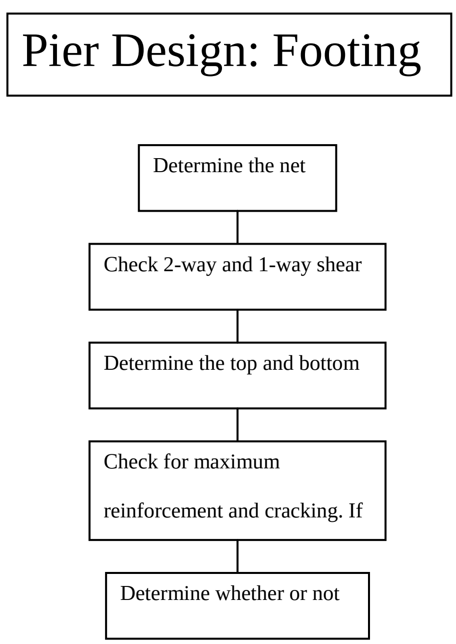


Figure 33: Footing Design Flow Chart

V.1.3 Pier Foundation Design Methodology

For each pier a shallow and a deep foundation were designed. This was done because the soil conditions where the bridge will be built is not specified. Therefore, having a design for both a shallow foundation and a deep foundation will be sufficient for most typical soil conditions. These designs were done using two sets of soil conditions that would allow for the design of each type of foundation. The soil conditions that were used are summarized in the table shown below.

Table 6: Assumed soil conditions for pier foundation design

Shallow		
Unit Weight of Soil	120 lbs/ft^3	
Friction Angle	33°	
Deep		
0-4m	Medium Clay	Side Friction = 25 kPa
4-14m	Silty Sand	Side Friction = 100 kPa
14-15m	Glacial Till	Side Friction = 800 kPa
		Toe Bearing = 4000 kPa

These typical soil conditions were obtained from examples in <u>Foundation Engineering: Principles and Practices</u>. The shallow foundation design was mostly a check to ensure that the footings designed for the piers would act as suitable foundations. The deep foundations were designed as piles that will be driven into the ground and use friction as a way to withstand the forces being applied. A flow chart for the design of the shallow foundation was done as the footing design for the piers earlier. Therefore a flow chart for the design of the deep foundations only will be shown in the following figure.

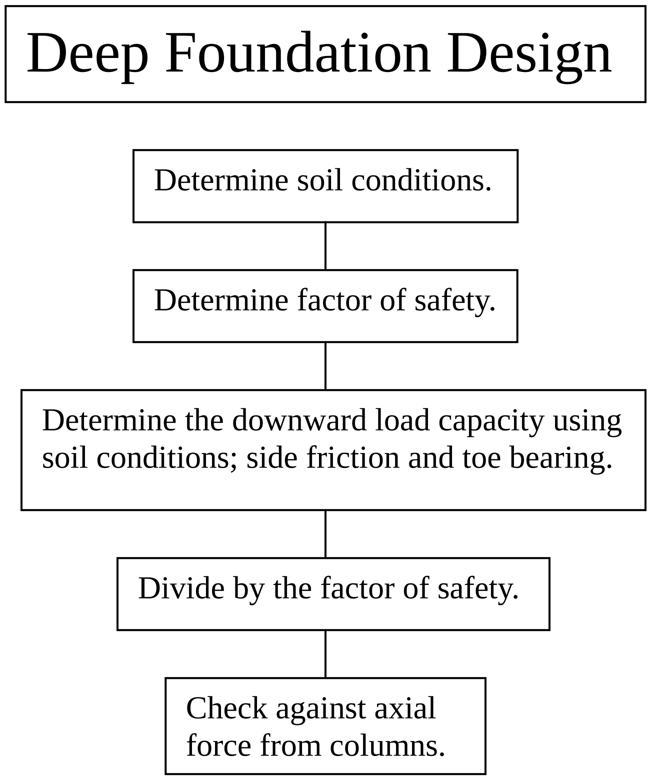


Figure 34: Deep Foundation Design Flow Chart

The layout of the piles for the single leg pier is shown in the following figure.

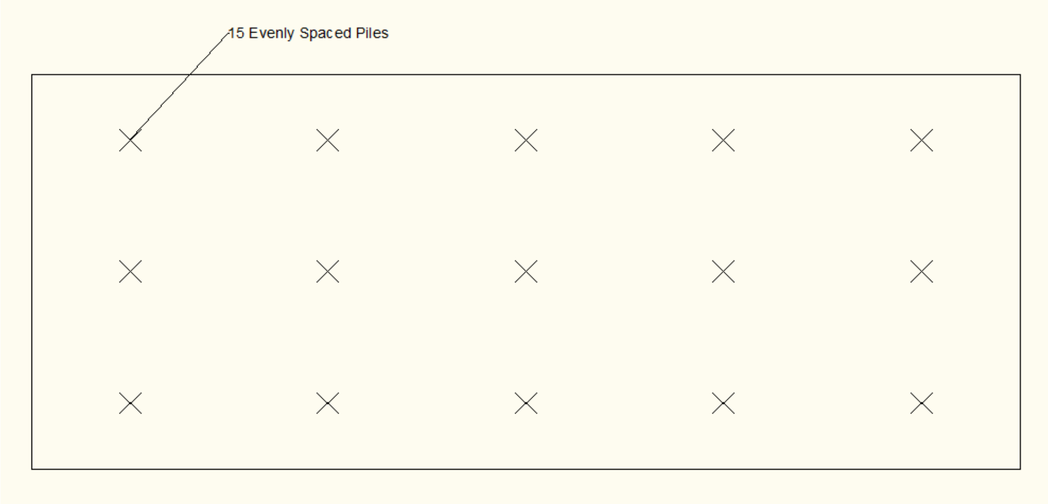


Figure 35: Layout of Piles of the Deep Foundation for the Single Leg Pier

The sizes for the piles were calculated in metric units and then converted to English.

V.1.4 Design Results

This section will present the design results for both pier options. It will also contain the results of the foundation designs.

The results of both pier designs is shown below in the following table. Cross sections of the different pieces of the two piers are also shown in the Figures 36-41.

Table 7: Pier design reinforcement results

Multi-Column Pier Design			
	Type of Reinforcement	Bar Size	Number of bars
	Cap Top Flexural	6	12
	Cap Bottom Flexural	6	12
	Cap Torsional	7	4 (per side)
	Cap Stirrup	5	6" Spacing
	Column Longitudinal	8	16
	Column Transverse	3	12" Spacing
	Footing Top Flexural	6	15
	Footing Bottom Flexural	8	15
Single-Leg Pier Design			
	Type of Reinforcement	Bar Size	Number of bars
	Cap Top Flexural	6	12
	Cap Bottom Flexural	6	12
	Cap Torsional	7	4 (per side)
	Cap Stirrup	5	3" Spacing
	Column Longitudinal	10	90
	Column Transverse	4	12" Spacing
	Footing Short Direction	10	11 (Top and Bottom)
	Footing Long Direction	6	10 (Top and Bottom)

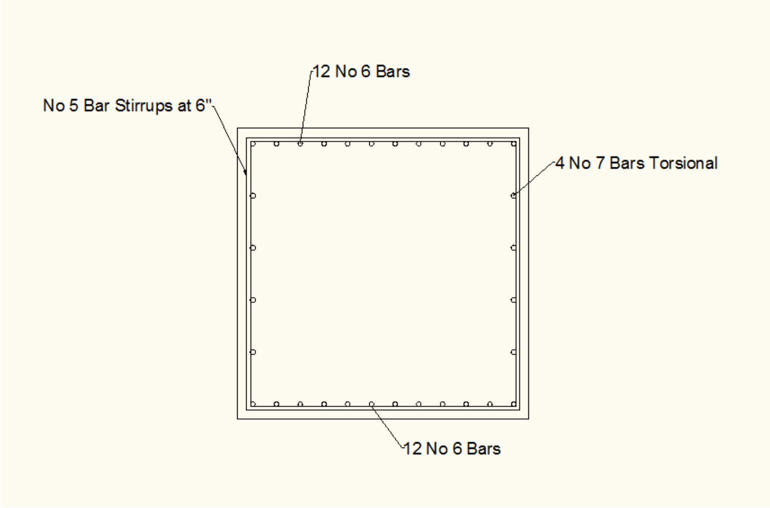


Figure 36: Multi-Column Pier Cap Cross-Section with Reinforcement

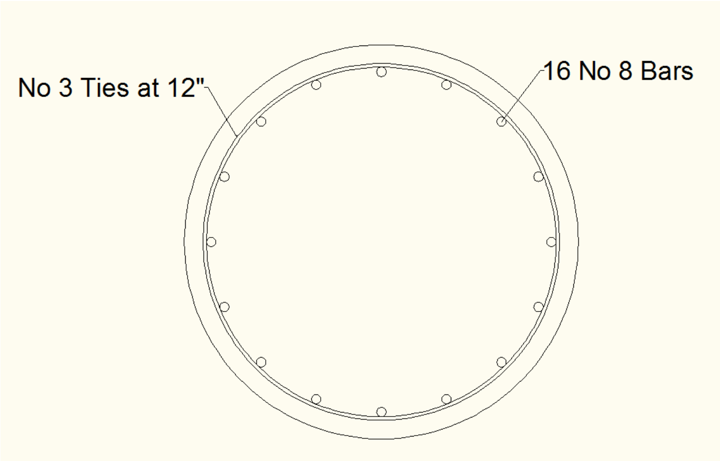


Figure 37: Multi-Column Pier Column Cross-Section with Reinforcement

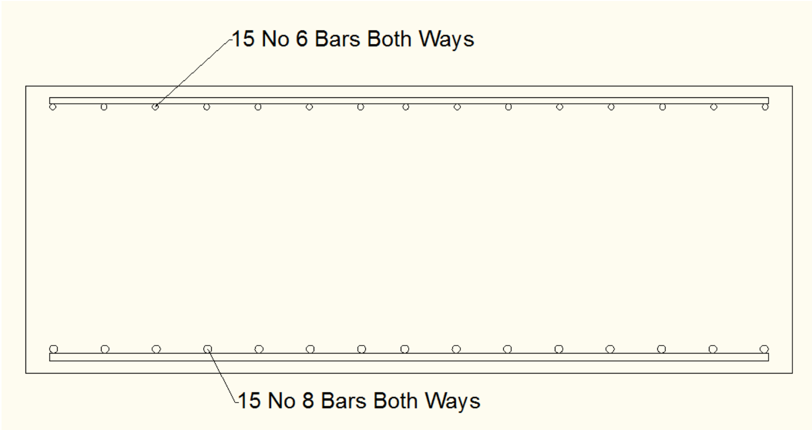


Figure 38: Multi-Column Pier Footing Cross-Section with Reinforcement

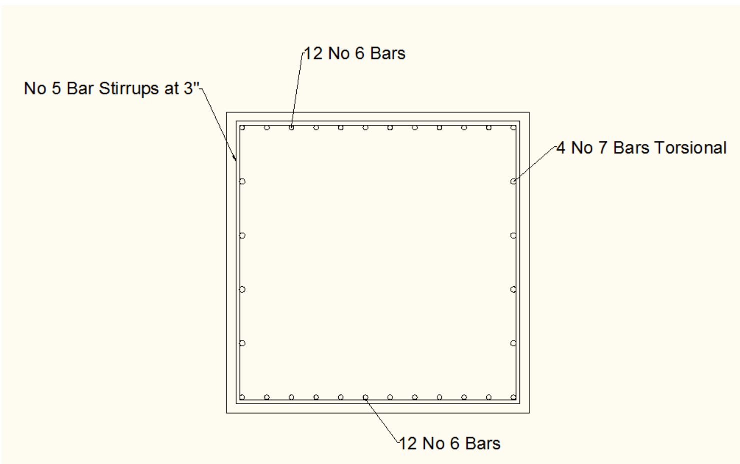


Figure 39: Single Leg Pier Cap Cross-Section with Reinforcement

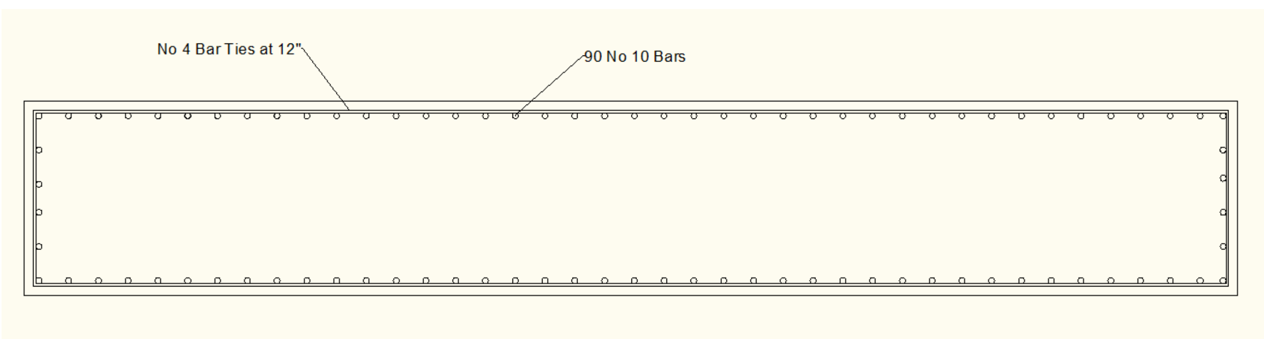


Figure 40: Single Leg Pier Column Cross-Section with Reinforcement

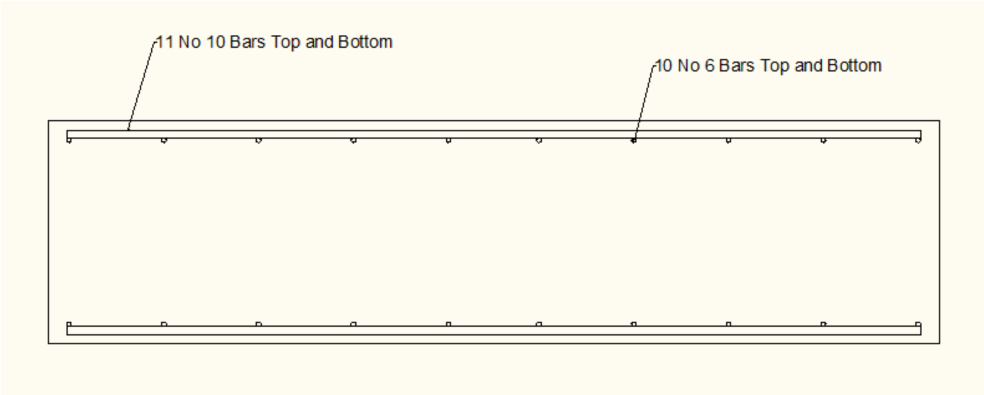


Figure 41: Single Leg Pier Footing Cross-Section with Reinforcement

As can be seen from the preceding table, more reinforcement is needed for the single-leg pier. From the sketches and dimensions given in the design method it is also shown that more concrete will be needed for the single-leg design. Therefore based upon only the amount of material needed the multi-column pier design appears to be the better option.

The results of the foundation design are shown in the table below.

Table 8: Pier foundation results

Multi-Column Pier Design		
	Shallow Foundation	Deep Foundation
	Footing is acceptable	Each column uses a 28" diameter 49.2' pile
Single-Leg Pier Design		
	Shallow Foundation	Deep Foundation
	Footing is acceptable	Use 15 evenly spaced 10" diameter 49.2' piles

For both designs the footing is sufficient as a shallow foundation. The multi-column design requires only one pile per column for the deep foundation, while the single leg design requires an evenly spread pattern of piles. The piles for this particular single leg pier are smaller than those used for the multi-column design, but they are also more numerous.

V.1.5 Life-Cycle Cost Analysis

The life-cycle cost of a pier is equal to its initial cost plus the cost of maintenance. The initial cost is due to the amount of material used and the constructability of the structure. The maintenance cost will be determined by the amount of repairs that are needed. The surface area of the pier will determine this. The more surface area there is, the more area that is susceptible to the elements that can result in damage to the concrete. This could lead to possible corrosion of the reinforcing steel. Repairs are needed when the concentration of chloride ion at the reinforcing bar reaches a certain level (Nishizaki, 2006). These repairs would be performed for surface cracking due to freeze-thaw conditions and road salts. The method of the repair will also affect the cost. According to the research done a multiple column design will be subject to more deterioration during its life-cycle than the single leg design because it has more exposed surface area. However, it is less expensive and easier to construct the single leg design (Faculty, 2009). Inspection costs also affect the maintenance cost, but because inspections are done at a set interval their cost will be the same for both piers and were therefore not taken into account for this analysis.

In the table below is a list of the costs that were used to determine the life-cycle cost of the piers.

Table 9: Costs of pier components (Ito, 2009)

	Item	Cost
Initial	Concrete	5.570 $/ft^3
	Form	0.995 $/ft^3
	Curing	0.159 $/ft^3
	Rebar	1011.600 $/metric ton
	Scaffolding	0.637 $/ft^3
Repair	Patching	10.847 $/ft^2

It is important to note that these costs were converted from Japanese cost data. The error that this presents is negligible since both piers will be subjected to the same values. Therefore the comparison between the costs of the two piers should remain the same.

The amount of material needed for each pier design is summarized in the following table.

Table 10: Material quantities used in pier designs

	Item	**Amount**
Multi-Column	**Concrete**	**1950.0 ft^3**
	Rebar	**4.2 tons**
		19.0 ft^3
	Surface Area	**730.0 ft^2**
Single Leg	**Concrete**	**2950.0 ft^3**
	Rebar	**5.5 tons**
		26.5 ft^3
	Surface Area	**1450.0 ft^2**

The single leg column needs more material to be constructed as was expected. However, it also has more surface area. This is contradictory to what was found in the research. This could be due to the fact that the single leg design needed such a large column to accommodate the size of the cap chosen for the piers.

Repairs need to be done when the chlorine ion concentration at the rebar reaches a certain value. A typical value for this is 1.2 kg/m^3 or 0.0749 lbs/ft^3 (Nishizaki, 2006). Since both piers have the same concrete cover thickness over the reinforcement, the concrete will deteriorate and allow for the chlorine ion concentration to build up at approximately the same rate. This means that both pier designs will need repairs after the same amount of time. Depending upon the climate and the quality of the initial construction it takes between 15 and 30 years to reach the ion concentration that is being used (Nishizaki, 2006). To counteract these affects patching will be done regularly at these intervals. This should ensure that the reinforcing steel will not corrode. A life-cycle cost analysis was performed for both of these repair intervals over service periods of both 50 and 100 years. This was done by using the values from the tables above. An initial cost for each pier was obtained and then the appropriate amount of repair costs were added. The repair cost was determined by multiplying the total surface area for each design by the patching cost per square foot. A present worth analysis was also performed using both a 3% and a 5% interest rate. The present worth of the pier is the amount of money needed now to cover the costs of construction and maintenance. The money not used for the initial construction would gain the given interest amount until it was needed for repairs. The results of these life-cycle cost analyses are given in the following table.

Table 11: Results of the life-cycle cost analyses

Multi-Column	LCC (years)	Repair Interval (years)	Initial Cost ($)	Repair Cost ($)	Total Cost ($)	Present Worth 3% ($)	Present Worth 5% ($)
	50	15	17300	7900	41000	27800	23900
	50	30	17300	7900	25200	20600	19200
	100	15	17300	7900	64700	30500	24600
	100	30	17300	7900	41000	22500	19700
Single-Leg	50	15	25300	15700	72400	46000	38300
	50	30	25300	15700	41000	31800	29000
	100	15	25300	15700	119500	51500	39700
	100	30	25300	15700	72400	35600	30000

V.1.6 Remarks

In all cases the multi-column pier is less expensive than the single leg pier. The single leg pier required more concrete and more steel reinforcement to build. It will also require the use of larger sections, which will be difficult to transport and erect. The multi-column pier seems to be the better option in every respect. It requires less material, it will be easier to construct, it has a lower life-cycle cost, and less piles are required for a deep foundation. If the current money can be put into an account with 5% interest, using the multi-leg design will only require $19700 to be set aside in order for the pier to be maintained for 100 years under the best of conditions. Even if only 3% interest can be obtained and conditions are not ideal, the pier will only require $30500 to be maintained for 100 years. Based upon this design a multi-column pier should be used for the design of a highway overpass bridge.

Abutment Design

This section will present the design of the bridge abutment. It includes a design background, a methodology, and a results section as well as a life-cycle cost analysis for this design. The designs and analyses will be summarized and a complete procedure will be annexed in the appendix.

V.1.7 Design Background

As mentioned earlier in this report, abutments are classified as: a) open end, monolithic type; b) close end, monolithic type; c) open end, short stem seat type; and d) close end, high stem seat type (see figure below). For the design of this project, the different abutment types were evaluated according to its structural support and structure approach. A cantilever abutment, which falls under the close end, high stem seat type, was chosen for this design.

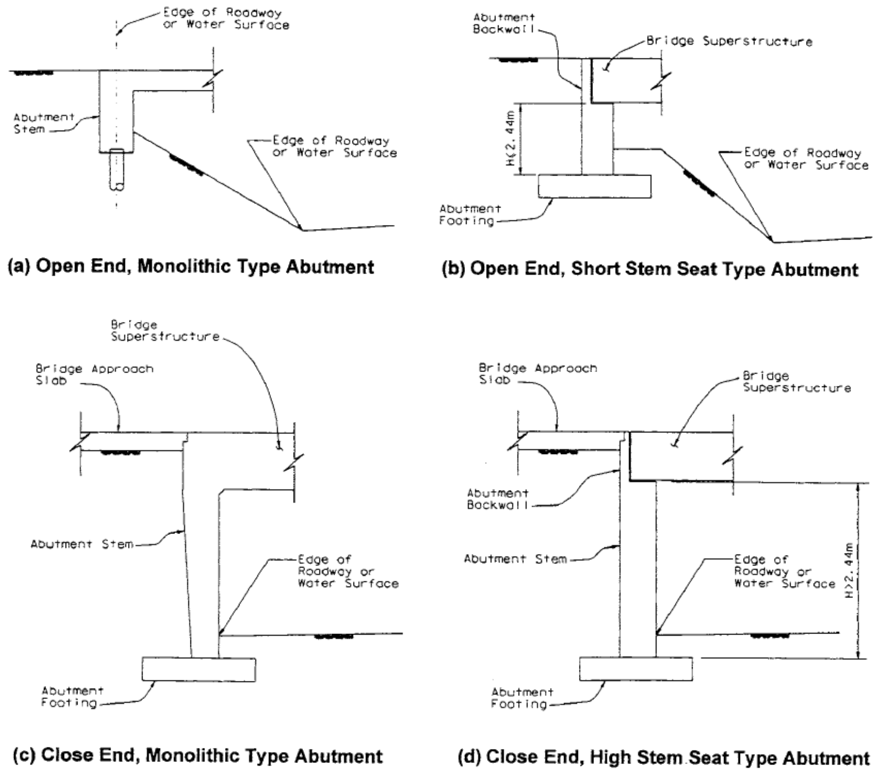


Figure 42: Typical Abutment Types.
Chen, Wai-Fah (Ed.) & Duan, Lian (Ed.) (1999). Bridge Engineering Handbook. Boca Raton, Florida: CRC Press.

As seen in the figure above, close end abutments contain high vertical walls that do not require much space for its construction. They have a high initial cost, but require low maintenance. On the other hand, open end abutments have slopes between the abutment face and the edge of the roadway, which take up a large space for its construction. Also, open end abutments allow water intrusion between the abutment and the approach roadway, causing damage to the approach embankment and pavement, and consequently requiring a continuous maintenance of these areas (Land & Post).

A seat type abutment can be designed to accommodate all imposed forces and allow superstructure movement, since it is an independent component of the bridge. Being an independent component of the bridge, seat type abutments would be suitable for both steel and concrete superstructures. In contrast, a monolithic type abutment is directly connected to the superstructure; and therefore it would be suitable for concrete superstructures only (Land & Post).

Due to the facts mentioned above, a cantilever abutment will be designed to support this bridge at the extreme ends. The design of the abutment was divided into three sections; backwall design, stem design, and footing design. This was done by following the procedure from an LRFD abutment design example from the FHWA's website. The composite superstructure design with 3 foot spacing between girders was also used for the abutment design. Since the dead load produced by this option is one of the largest forces acting on the abutment, the abutment designed in this analysis should be capable of supporting most of the other superstructure systems.

V.1.8 Design Methodology

The dimensions of the abutment can be obtained from design manual's specifications, by trial and error, or from size proportions from previous designs. For this case, the dimensions were estimated by using the guidelines in the book Design of Reinforced Concrete by Jack C. McCormac. A graphical representation of these guidelines can be observed in Figure 43. After making an estimate of the size, the stability of the abutment was checked to obtain the final dimensions. This was based on a factor of safety of 1.5 for sliding and 2.0 for overturning (McCormac). The final dimensions can be seen in Figure 44.

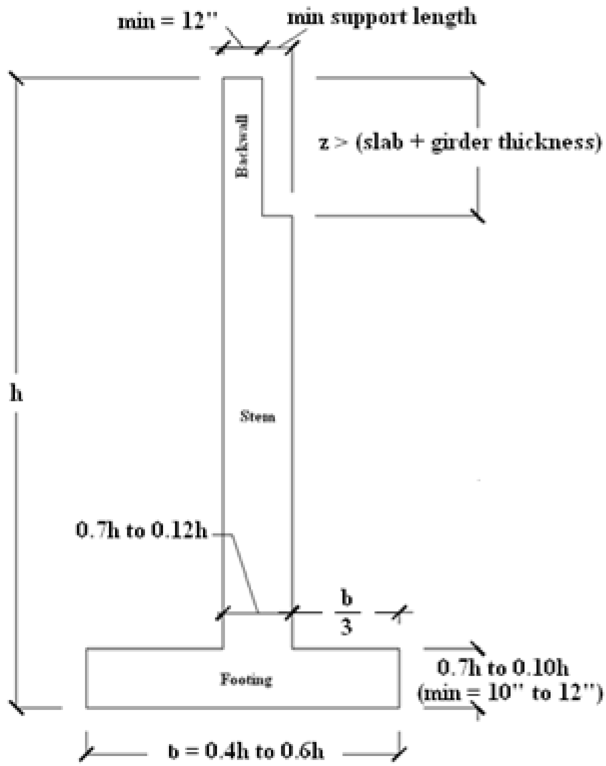


Figure 43: Abutment Size Specifications (McCormac)

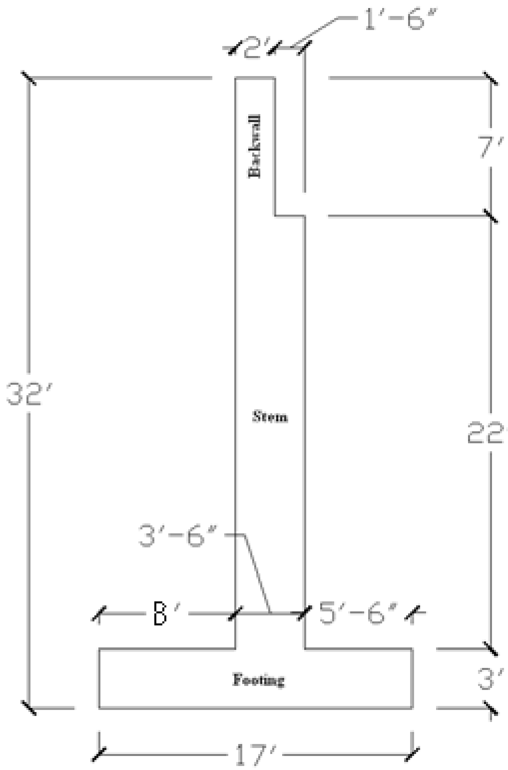


Figure 44: Final Abutment Sizes

The dead load, live load, and wind load acting on the abutment were calculated in a similar way to those acting on the pier (see pier section for more details or appendix for calculations). For this case, it was assumed that the abutment has expansion bearings, therefore, the braking force is not applied at the abutment. It is instead resisted by the fixed bearings located at the pier. Earth loads and temperature loads were also calculated. The earth loads investigated in this design include loads due to lateral earth pressure and live load surcharge loads.

For this design, three critical locations where the force effects needed to be combined and analyzed; the bottom of the backwall: the bottom of the stem, and the bottom of the footing. The maximum moments and shears acting on each part of the abutment were calculated using the appropriate load combination for each limit state. The backwall, the stem, and the footing were designed for Strength I, Strength III, Strength V, and Service I, but in most cases the controlling limit states were Strength I and Service I. The design processes for each of the elements are summarized in the following flow charts:

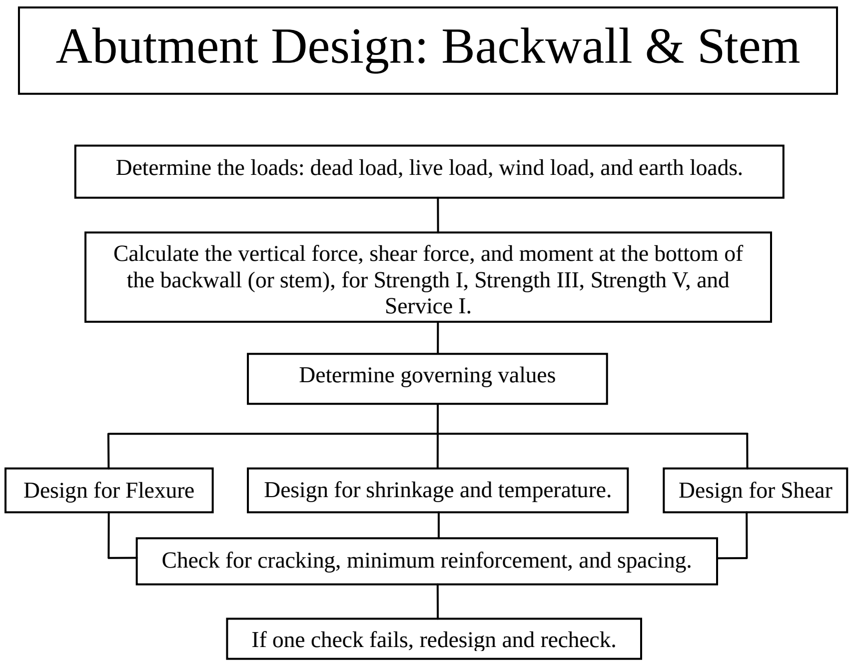


Figure 45: Backwall & Stem Design Procedure

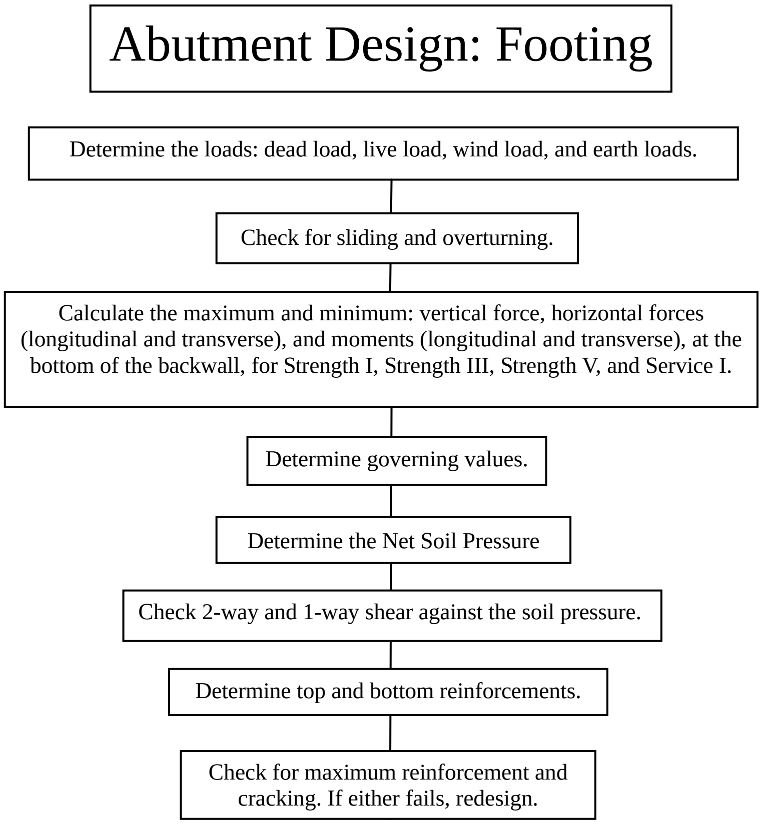


Figure 46: Footing Design Procedure

The figure below shows the different types of loads acting on the abutment. For this case, it was assumed that the approach slab and the roadway will cover the abutment backfill material. Therefore, no uniform load was applied.

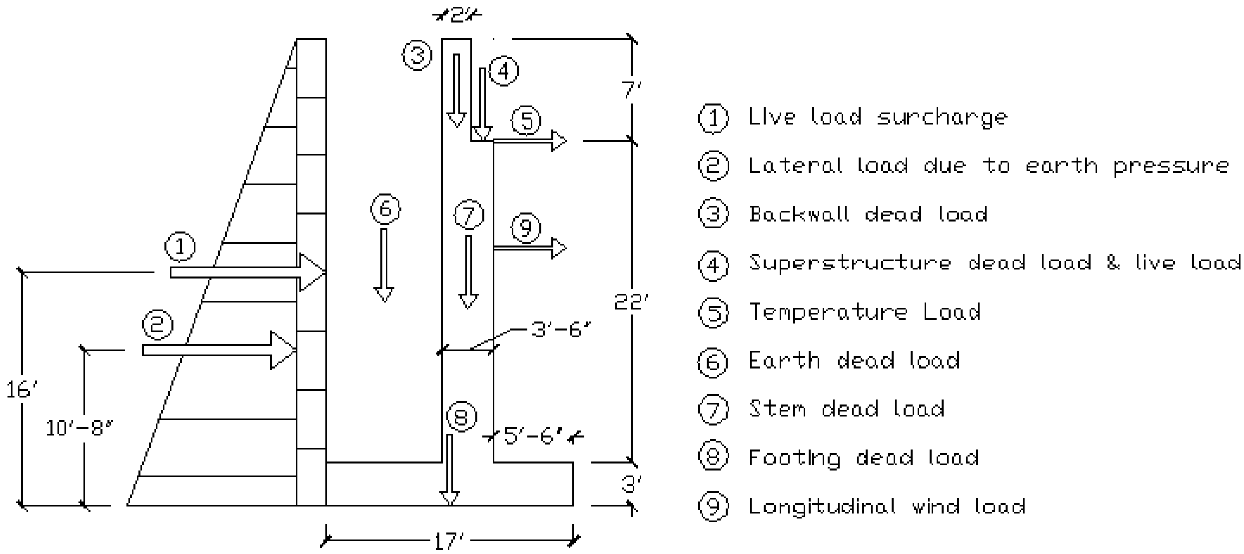


Figure 47: Loads Acting on the Abutment

V.1.9 Abutment Foundation Design Methodology

The soil conditions, under which this bridge will be built, were not specified. Hence, the abutment foundation could be either a shallow foundation or a deep foundation. A footing having the proper proportions can act as a shallow foundation. Therefore, the footing was designed so that it would be suitable for a shallow foundation. A soil unit weight of 120 pcf and a friction angle of 27° were used. If soil conditions were not suitable for a shallow foundation, then a clear procedure can be followed in the pier design chapter and pier design calculations for the design of a pile foundation with a pile cap.

V.1.10 Abutment Design Results

Figures 48-51 show the final abutment design results, including the dimensions and reinforcement schemes for the backwall, stem, and footing.

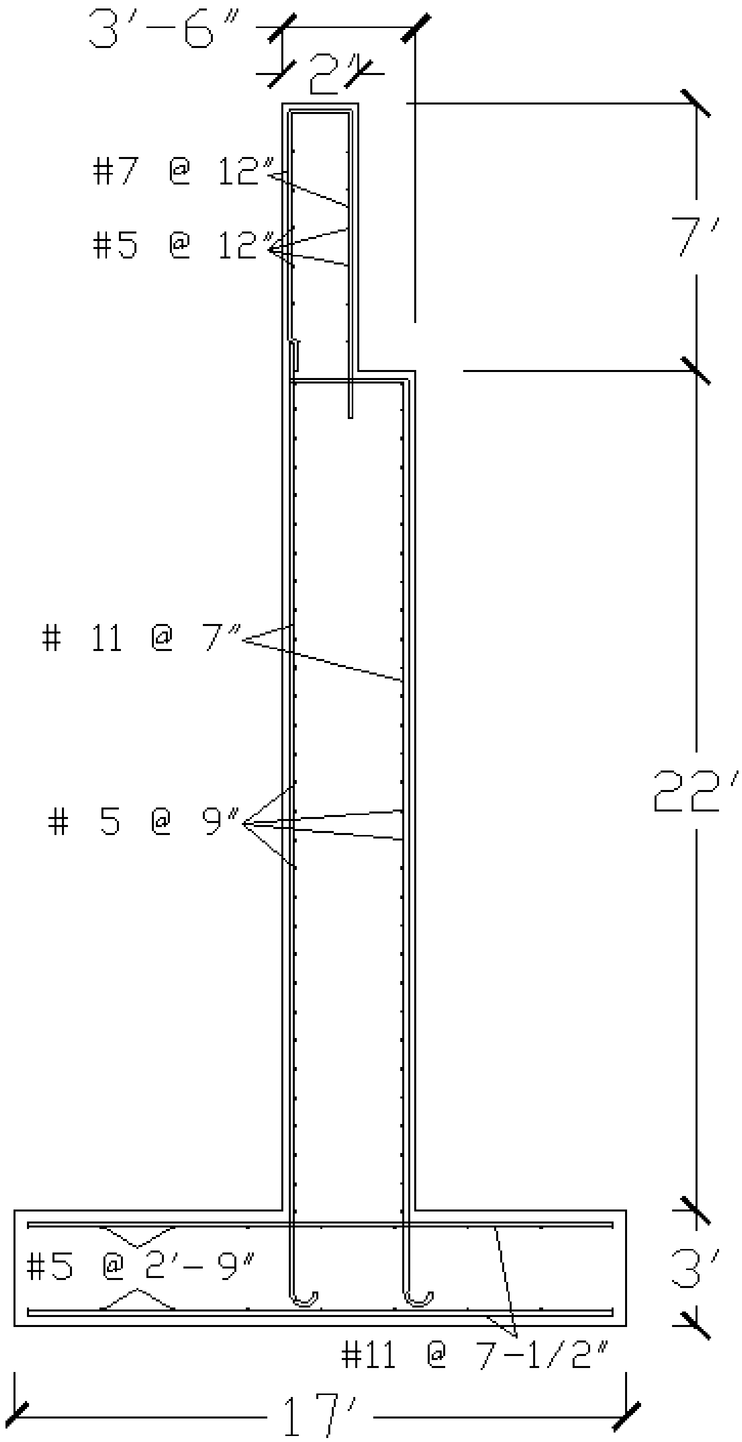


Figure 48: Abutment Reinforcements

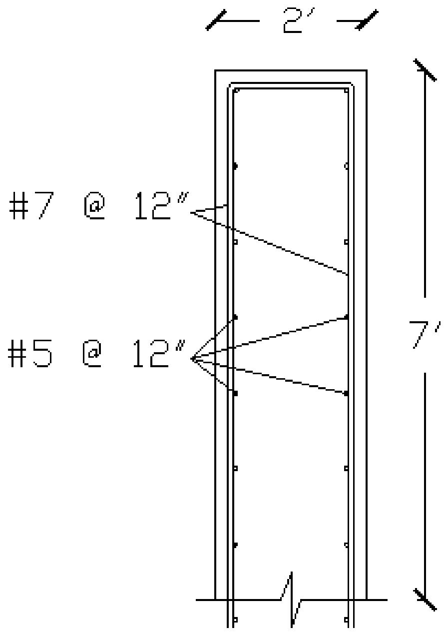


Figure 49: Abutment Stem With Reinforcement

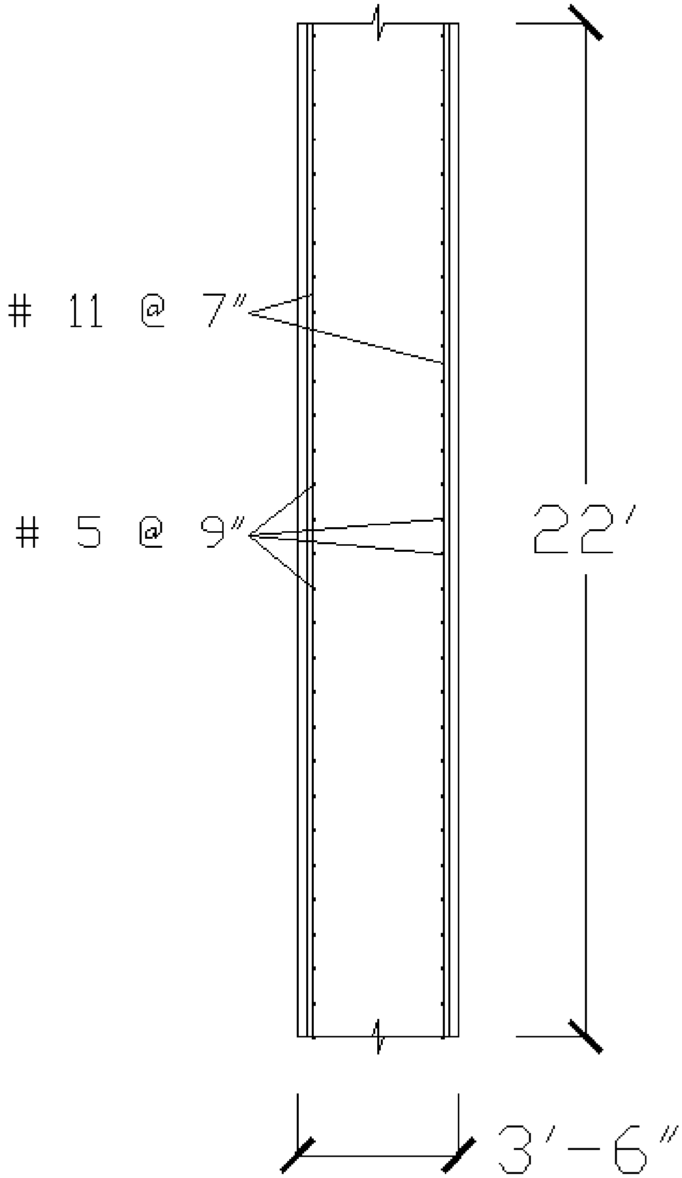


Figure 50: Abutment Backwall with Reinforcement

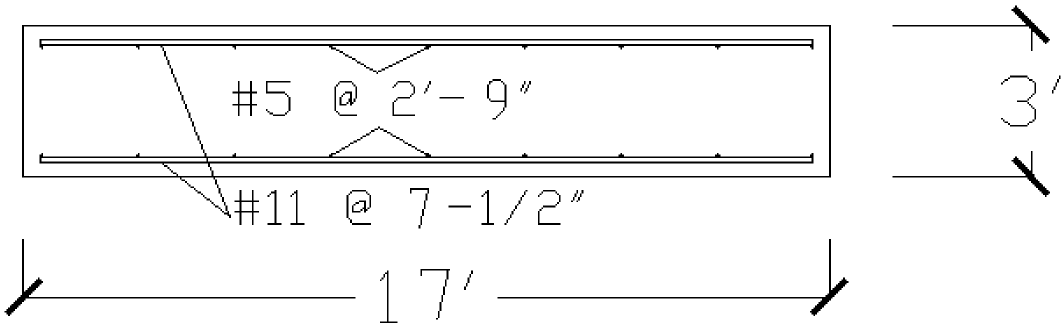


Figure 51: Abutment Footing with Reinforcement

V.1.11 Abutment Life-Cycle Cost Analysis

The life-cycle cost of the abutment is equal to its initial cost plus the cost of maintenance. An approximation of the initial cost was estimated as seen in the figures below.

Table 12: Cost of Materials and Maintenance

Item	Cost	Unit
Concrete	5.57	$/ft³
Form	1.00	$/ft³
Rebar	1,011.60	$/ton
Curing	0.16	$/ft³
Scaffolding	0.64	$/ft³
Patching	10.85	$/ft³

Table 13: Abutment Steel Reinforcement by Weight

Abutment - Reinforcement - Cost							
Backwall	Quantity	Bar Size	Spacing	Length (ft)	Weight (lb/ft)	Total Weight (ton)	Cost (US Dollars)
Longitudinal	14	#5	@ 12"	29	1.043	0.192	$ 194.31
Transverse (front + back)	58	#7	@ 12"	7	2.044	0.376	$ 380.79
Stem	Quantity	Bar Size	Spacing	Length (ft)	Weight (lb/ft)	Total Weight (ton)	Cost (US Dollars)
Longitudinal	60	#5	@ 9"	29	1.043	0.823	$ 832.74
Transverse (front + back)	100	#11	@ 7"	22	5.313	5.302	$ 5,363.37
Footing	Quantity	Bar Size	Spacing	Length (ft)	Weight (lb/ft)	Total Weight (ton)	Cost (US Dollars)
Longitudinal	14	#5	@ 2'-9"	29	1.043	0.192	$ 194.31
Horizontal (top + bot)	94	#11	@ 7'-1/2"	16.5	5.313	3.738	$ 3,781.17
						Total Cost Steel	**$ 10,746.68**

Table 14: Abutment Steel Reinforcement Cost by Volume

Scaffolding					
Steel Reinforcement	Quantity	Area	Length	Volume (ft³)	Cost (US Dollars)
#11	100	1.56	22	23.83	$ 15.25
#11	94	1.56	16.5	16.80	$ 10.75
#7	58	0.60	7	1.69	$ 1.08
#5	88	0.31	29	5.49	$ 3.52
				Total	**$ 30.61**

Table 15: Abutment Concrete Cost by Volume

Abutment - Concrete Volume - Cost					
	Width (ft)	Length (ft)	Height (ft)	Volume (ft^3)	Cost (US Dollars)
Backwall	2	30	7	420	$ 2,339.40
Stem	3.5	30	22	2,310	$ 12,866.70
Footing	17	30	3	1,530	$ 8,522.10
			Total	4,260	$ 23,728.20

Table 16: Repair Cost of Abutment

Abutment - Patching - Exposed Area		
	Surface Area (ft^2)	Cost (US Dollars)
Backwall	298	$ 3,233.30
Stem	814	$ 8,831.90
Footing	0	$ -
	Total	**$ 12,065.20**

It is important to note that for the patching repair only the exposed surface area was included.

With these values a life-cycle cost analysis was performed for the abutments. This analysis involved determining the total cost of the abutment after 50 and 100 years based upon a repair interval of either 15 or 30 years. A present worth analysis was also performed assuming both a 3% and a 5% interest rate, in order to determine how much money needs to be set aside now to maintain the abutments. The results of these analyses can be seen in the following table.

Table 17: Life-Cycle Cost Analysis Results

LCC (years)	Repair Interval (years)	Initial Cost ($)	Repair Cost ($)	Total Cost ($)	Present Worth 3% ($)	Present Worth 5% ($)
50	15	39500	12100	75800	55500	49500
50	30	39500	12100	51600	44500	42300
100	15	39500	12100	112100	59700	50600
100	30	39500	12100	75800	47400	43100

V.1.12 Remarks

Based upon the needs of the bridge, a cantilever abutment was chosen. This design is acceptable for both steel and concrete superstructures. It will also act as a retaining wall to hold back the soil on either end of the bridge. To build each abutment, $39500 will be required initially. Based upon the present worth analysis, the amount of money needed now to maintain the abutment for 100 years under the best of conditions at 5% interest is $43100. Under the worst conditions at 3% interest $59700 is needed. This means that in the next 100 years somewhere between $43000 and $60000 is needed to maintain each abutment.

VI Life Cycle Cost Analysis of Bearings

The Life-Cycle Cost Analysis was performed based on the technical requirements for bearings to be used in Option 2 (5 foot spacing) only. The geometry of the piers on top of which the bearings are to be placed was also taken into consideration. Other design options were not considered since the cost estimate of the superstructure showed that this design option was the most cost effective. In order to establish the type of bearing, initial cost, maintenance cost, and expected economic life several design parameters needed to be determined based upon the superstructure design.

Methodology

First, the most appropriate bearing type was chosen from a design capability perspective. The maximum vertical and horizontal loads on the bearing were determined through a structural analysis of the superstructure option under investigation using RISA 2D. The maximum rotation to be accommodated by the bearings was also determined from this structural analysis software. The maximum horizontal displacement in the longitudinal axis of the stingers was determined using standard procedures form AASHTO LRFD Bridge Design Specifications 4th Edition 2007 as well as FHWA LRFD Design Example for Steel Girder Superstructure Bridge, December 2003, FHWA NHI-04-041. This displacement was obtained by taking into account both displacements due to traffic loading and thermal expansion/contraction. Procedure A from AASHTO LRFD Specification was used to determine the horizontal displacement in the longitudinal axis due to thermal expansion/contraction. This procedure is based upon the fundamental assumption of a uniform temperature distribution throughout the cross-section of the superstructure. A procedure based on considering the effect due to the thermal gradient of the superstructure cross-section was also investigated. The implementation of this method however, was dismissed due to uncertainty in the results and consideration of the AASHTO LRFD Specification guidelines. The horizontal displacement of the stringers in the longitudinal axis was found to be less than the displacement in the longitudinal direction, therefore the latter governed.

The average life span of a bridge bearing is affected by traffic loading and, to a large degree, by corrosion, which diminishes the flexibility and load capacity of the bearing unit. The degree and rate of corrosion depend on two major corroding agents; the relative humidity and the presence of acidic substances on the exposed surface of the bearing. The degree of humidity present on the exposed surface of the bearings is affected in large part by two main factors; the location of the site and the quality of the expansion joints. One of the major factors responsible for the presence of acidic substances is bird excrements, which contain large quantities of substances with high pH levels that act as corroding agents over long periods of time. The location of the construction site was assumed to be Worcester, MA.

Traffic loading is influenced by the number and typical size of the vehicles that frequently use the bridge. The amount of traffic and the size of vehicles passing over the bridge depends upon its location. Bridges located on major traffic arteries leading into densely populated areas, industrial areas, large shipyards, construction areas, airports, etc. are expected to be subjected to a large traffic loading. Such loads will expose the bearings to large fatigue loading, which will cause them to lose elasticity and thus become unable to accommodate the displacement of the superstructure.

Based on Table 1.2 from Chen & Duan, several bearing types satisfied the design requirements obtained above including, steel reinforced elastomeric bearings, rocker bearings, and multiple rollers. Based on the cost range for each bearing type provided in the same table, the steel reinforced elastomeric bearing was considered the most cost effective. The design criteria obtained from the procedure outlined in the previous paragraphs were then provided to several bridge bearing manufacturers in order to establish a realistic price. Approximately 25 manufacturers of steel reinforced elastomeric bearings were contacted, including manufacturers in the US, UK and Canada. Such an approach did not yield the expected results since the information provided by these manufacturers did not include the elements of primary concern to this project. Few of the manufacturers responded to the group's requests. However, the information provided by them, included mostly technical specifications for several of the bearing types they produced. No data of any sort about the costs or life spans of their products were made available.

The Connecticut Department of Transportation, Bridge Design Department was then contacted and all the above information concerning the design criteria was provided to this agency. As a result, information regarding approximate values for life span, maintenance cost, and initial cost for a relatively similar project was obtained. The following Life-Cycle Cost Analysis results were based exclusively on this information. The reader should consider the fact that these results are based on approximate data and therefore will probably include appreciable error. The data, however, is expected to be accurate in a relative way. Please notice that rather than single cost values, cost ranges are graphically displayed in an effort to provide a sense of the magnitude of the possible deviation in the cost results.

Results

Tables 18-20 present the data that was used to perform the Life-Cycle Cost Analysis. First the data was divided into three main categories represented by the three columns in the first table from the left. The three categories consist of Low Estimates, High Estimates and Estimated Average for each parameter. The next tables were obtained by isolating each of the four parameters (Real Rate, Expected Economic Life, Rehabilitation Cost, Maintenance Cost and Inspection Cost) and assuming either a high, low, or average value for each of them. This was done in order to determine the effect that each of these variables have on the life-cycle cost. The isolated data is highlighted in yellow. The Real Rate was taken into account instead of simply considering the Expected Inflation Rate. This was done in order to account for the fact that the funds used to pay for the bearing costs throughout their life-cycle will be deposited in a bank. In this case the inflation rate will need to include the interest rate paid to these funds by the bank. It is also important to provide a short definition of Real Rate, which in this study is not equivalent to the classical definition. This rate is equal to the Expected Inflation Rate less the Interest Rate from the bank where the funds are deposited. In case the funds were loaned from a bank or another financial institution, the Real Rate, as defined in this study, would have to include the Interest Rate to be paid for the loaned funds. Such a scenario was not subject to consideration in this study.

Table 18: Cost Data

				High Labor Cost			Low Labor Cost		
	Low Estimate	High Estimate	Average	Low Estimate	High Estimate	Average	Low Estimate	High Estimate	Average
Initial Cost (C)[1]	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00
Inflation Rate (F)[4]	3.00%	6.00%	4.50%	3.00%	6.00%	4.50%	3.00%	6.00%	4.50%
Interest Rate (I)[2]	2.00%	3.00%	2.50%	2.00%	3.00%	2.50%	2.00%	3.00%	2.50%
Real Rate(R)[1]	-1.00%	-3.00%	-2.00%	-1.00%	-3.00%	-2.00%	-1.00%	-3.00%	-2.00%
Expected Economic Life (n) in yrs.[1]	30	35	32	30	35	32	30	35	32
Number of Girders	6	6	6	6	6	6	6	6	6
Labor Price/girder (LP)[1]	$ 3,000.00	$ 5,000.00	$ 4,000.00	$ 5,000.00	$ 5,000.00	$ 5,000.00	$ 3,000.00	$ 3,000.00	$ 3,000.00
Labor Cost (LC)	$ 18,000.00	$ 30,000.00	$ 24,000.00	$ 30,000.00	$ 30,000.00	$ 30,000.00	$ 18,000.00	$ 18,000.00	$ 18,000.00
Jacking Price/girder (JP)[1]	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00
Jacking Cost (JC)	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00
Transportation Cost etc. (TC)[1]	$ 5,000.00	$ 10,000.00	$ 7,500.00	$ 5,000.00	$ 10,000.00	$ 7,500.00	$ 5,000.00	$ 10,000.00	$ 7,500.00
Rehabilitation Cost (RC)	$ 143,000.00	$ 160,000.00	$ 151,500.00	$ 155,000.00	$ 160,000.00	$ 157,500.00	$ 143,000.00	$ 148,000.00	$ 145,500.00
Maintenance Cost* (MC)	$ 2,000.00	$ 3,000.00	$ 2,500.00	$ 2,000.00	$ 3,000.00	$ 2,500.00	$ 2,000.00	$ 3,000.00	$ 2,500.00
Maintenance Freq. in yrs. (MF)	4	4	4	4	4	4	4	4	4
Inspection Freq. in yrs. (IF)	2	2	2	2	2	2	2	2	2
Inspection Cost (IC)*	$ 1,000.00	$ 2,000.00	$ 1,500.00	$ 1,000.00	$ 2,000.00	$ 1,500.00	$ 1,000.00	$ 2,000.00	$ 1,500.00
Salvage Value	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
Exp. Life span of the bridge in yrs.[3]	150	150	150	150	150	150	150	150	150

Table 19: Cost Data

High Maintenance Cost			Low Maintenance Cost			High Inspection Cost			Low Inspection Cost		
Low Estimate	High Estimate	Average	Low Estimate	High Estimate	Average	Low Estimate	High Estimate	Average	Low Estimate	High Estimate	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00
3.00%	6.00%	4.50%	3.00%	6.00%	4.50%	3.00%	6.00%	4.50%	3.00%	6.00%	4.50%
2.00%	3.00%	2.50%	2.00%	3.00%	2.50%	2.00%	3.00%	2.50%	2.00%	3.00%	2.50%
-1.00%	-3.00%	-2.00%	-1.00%	-3.00%	-2.00%	-1.00%	-3.00%	-2.00%	-1.00%	-3.00%	-2.00%
30	35	32	30	35	32	30	35	32	30	35	32
6	6	6	6	6	6	6	6	6	6	6	6
$ 3,000.00	$ 5,000.00	$ 4,000.00	$ 3,000.00	$ 5,000.00	$ 4,000.00	$ 3,000.00	$ 5,000.00	$ 4,000.00	$ 3,000.00	$ 5,000.00	$ 4,000.00
$ 18,000.00	$ 30,000.00	$ 24,000.00	$ 18,000.00	$ 30,000.00	$ 24,000.00	$ 18,000.00	$ 30,000.00	$ 24,000.00	$ 18,000.00	$ 30,000.00	$ 24,000.00
$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00
$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00
$ 5,000.00	$ 10,000.00	$ 7,500.00	$ 5,000.00	$ 10,000.00	$ 7,500.00	$ 5,000.00	$ 10,000.00	$ 7,500.00	$ 5,000.00	$ 10,000.00	$ 7,500.00
$ 143,000.00	$ 160,000.00	$ 151,500.00	$ 143,000.00	$ 160,000.00	$ 151,500.00	$ 143,000.00	$ 160,000.00	$ 151,500.00	$ 143,000.00	$ 160,000.00	$ 151,500.00
$ 3,000.00	$ 3,000.00	$ 3,000.00	$ 2,000.00	$ 2,000.00	$ 2,000.00	$ 2,000.00	$ 3,000.00	$ 2,500.00	$ 2,000.00	$ 3,000.00	$ 2,500.00
4	4	4	4	4	4	4	4	4	4	4	4
2	2	2	2	2	2	2	2	2	2	2	2
$ 1,000.00	$ 2,000.00	$ 1,500.00	$ 1,000.00	$ 2,000.00	$ 1,500.00	$ 2,000.00	$ 2,000.00	$ 2,000.00	$ 1,000.00	$ 1,000.00	$ 1,000.00
$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -	$ -
150	150	150	150	150	150	150	150	150	150	150	150

Table 20: Cost Data

High Real Rate			Low Real Rate		
Low Estimate	High Estimate	Average	Low Estimate	High Estimate	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00	$ 1,069.00
6.00%	6.00%	6.00%	3.00%	3.00%	3.00%
2.00%	2.00%	2.00%	2.00%	2.00%	2.00%
-4.00%	-4.00%	-4.00%	-1.00%	-1.00%	-1.00%
30	35	32	30	35	32
6	6	6	6	6	6
$ 3,000.00	$ 5,000.00	$ 4,000.00	$ 3,000.00	$ 5,000.00	$ 4,000.00
$ 18,000.00	$ 30,000.00	$ 24,000.00	$ 18,000.00	$ 30,000.00	$ 24,000.00
$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00	$ 20,000.00
$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00	$ 120,000.00
$ 5,000.00	$ 10,000.00	$ 7,500.00	$ 5,000.00	$ 10,000.00	$ 7,500.00
$ 143,000.00	$ 160,000.00	$ 151,500.00	$ 143,000.00	$ 160,000.00	$ 151,500.00
$ 2,000.00	$ 3,000.00	$ 2,500.00	$ 2,000.00	$ 3,000.00	$ 2,500.00
4	4	4	4	4	4
2	2	2	2	2	2
$ 1,000.00	$ 2,000.00	$ 1,500.00	$ 1,000.00	$ 2,000.00	$ 1,500.00
$ -	$ -	$ -	$ -	$ -	$ -
150	150	150	150	150	150

*Rrough estimates

[1]According to data gathered from Connecticut DOT. This umber refers to the cost of the labor needed to jack each bridge girder.

[2]Hanover Insurance Group

[3]California DOT (www.dot.ca.gov/dist4/eastspans/index.html)

[4]inflationdata.com (Based on Inflation Data from the Past 10 years)

Input

Input for Present Worth Formula

First the Life Cycle–Cost Analysis was performed by assuming low, average, and high values for each of the parameters listed on the left-hand side of Table 18. The results were then presented from the perspective of high, average, and low expected economic life. Please note that in all the following graphs the final value representing Rehabilitation Cost was omitted. This was done in order to better display the shape and trends of the graph curves. Since rehabilitation costs are always much higher than other costs, the latter values would not be clearly distinguishable if all the data was to be presented in the same graph. The initial cost values are also not displayed graphically. This was done with the intent to place more emphasis on the trend of the cost data after the bearings have been initially installed. The omission of the initial cost and rehabilitation cost data from the graphs allows the reader to focus on the trend that the data will follow after the bearings have been initially installed, and before they are actually replaced. The data displayed on the graphs provides a clear idea of how the costs are distributed through time, a result that has a relative significance to projects of the type, despite the difference in cost.

Table 21 displays the Present Worth values for Inspection Cost, Maintenance Cost, and Rehabilitation Cost of the bearings evaluated every two years. The first column from the left lists the years at which the Present Worth value of each cost was evaluated. The second column lists the coefficients calculated by the formula P/F = 1/((1-(R))^(x)), where R is the Real Rate and x is the number of years passed since the bearings were initially purchased. P/F denotes the coefficient used to calculate the Present Worth (P) given Future Expense (F) at discount rate (R) for number of years (x). The second column lists the present worth values of the Inspection Cost. As a result of consultations with professional engineers, it was deemed appropriate to assume that inspection of the bearings is to be done at least every two years. The third column lists the present worth values for Maintenance Cost. Maintenance frequency was determined in the same way as inspection frequency. The fourth column lists the present worth values for rehabilitation costs evaluated for Low, Average, and High Economic Life respectively. The same parameters are then evaluated using the high and average estimates from Table 18. The total cost of is then cumulated every two years and is displayed on a graph.

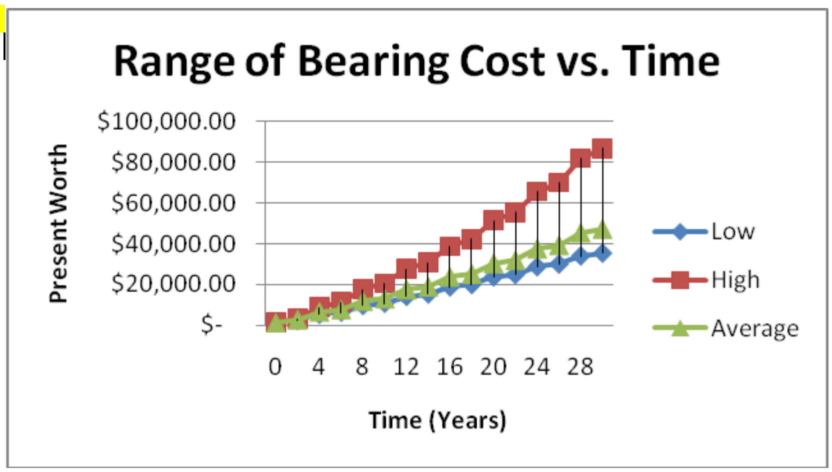
Table 21: Absolute Low, Average and High Estimates

Absolute Low, Average and High Estiamtes												
	Low Estimate				High Estimate				Average Estimate			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,3%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,2%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 1,020.30	$ -	$ -	1.0628	$ 2,125.62	$ -	$ -	1.041233	$ 1,561.85	$ -	$ -
4	1.0410	$ 1,041.02	$ 2,082.04	$ -	1.1296	$ 2,259.14	$ 3,388.71	$ -	1.084166	$ 1,084.17	$ 2,710.41	$ -
6	1.0622	$ 1,062.16	$ -	$ -	1.2005	$ 2,401.04	$ -	$ -	1.128869	$ 1,128.87	$ -	$ -
8	1.0837	$ 1,083.72	$ 2,167.45	$ -	1.2759	$ 2,551.86	$ 3,827.78	$ -	1.175415	$ 1,175.42	$ 2,938.54	$ -
10	1.1057	$ 1,105.73	$ -	$ -	1.3561	$ 2,712.14	$ -	$ -	1.223881	$ 1,223.88	$ -	$ -
12	1.1282	$ 1,128.18	$ 2,256.36	$ -	1.4412	$ 2,882.50	$ 4,323.75	$ -	1.274345	$ 1,274.35	$ 3,185.86	$ -
14	1.1511	$ 1,151.08	$ -	$ -	1.5318	$ 3,063.56	$ -	$ -	1.32689	$ 1,326.89	$ -	$ -
16	1.1745	$ 1,174.46	$ 2,348.91	$ -	1.6280	$ 3,255.98	$ 4,883.98	$ -	1.381601	$ 1,381.60	$ 3,454.00	$ -
18	1.1983	$ 1,198.30	$ -	$ -	1.7302	$ 3,460.50	$ -	$ -	1.438569	$ 1,438.57	$ -	$ -
20	1.2226	$ 1,222.63	$ 2,445.27	$ -	1.8389	$ 3,677.86	$ 5,516.79	$ -	1.497885	$ 1,497.89	$ 3,744.71	$ -
22	1.2475	$ 1,247.46	$ -	$ -	1.9544	$ 3,908.88	$ -	$ -	1.559647	$ 1,559.65	$ -	$ -
24	1.2728	$ 1,272.79	$ 2,545.57	$ -	2.0772	$ 4,154.40	$ 6,231.60	$ -	1.623956	$ 1,623.96	$ 4,059.89	$ -
26	1.2986	$ 1,298.63	$ -	$ -	2.2077	$ 4,415.35	$ -	$ -	1.690916	$ 1,690.92	$ -	$ -
28	1.3250	$ 1,325.00	$ 2,649.99	$ -	2.3463	$ 4,692.69	$ 7,039.03	$ -	1.760637	$ 1,760.64	$ 4,401.59	$ -
30	1.3519	$ 1,351.90	$ -	$ 193,321.52	2.4937	$ 4,987.44	$ -	$ 398,995.46	1.833233	$ 1,833.23	$ -	$ 277,734.84
32	1.3793	$ 1,379.35	$ 2,758.70	$ 197,246.73	2.6504	$ 5,300.72	$ 7,951.07	$ 424,057.24	1.908823	$ 1,908.82	$ 4,772.06	$ 289,186.63
34	1.4074	$ 1,407.35	$ -	$ -	2.8168	$ 5,633.67	$ -	$ -	1.987529	$ 1,987.53	$ -	$ -
35	1.4216	$ -	$ -	$ 203,284.48	2.9040	$ -	$ -	$ 464,632.18	2.028091	$ -	$ -	$ 307,255.72

Table 22 considers Low Expected Economic Life and shows the cumulative total cost at the end of every two years for low, high, and average estimates.

Table 22: Low Economic Life

Low Expected Economic Life		
Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 228,569.46	$ 485,825.05	$ 324,860.71



The same process was followed for Average and High Economic Life respectively and the results are presented below in a similar manner.

Table 23: Average Economic Life

Average Expected Economic Life		
Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 35,247.94	$ 86,829.59	$ 47,125.88
$ 236,632.71	$ 524,138.63	$ 342,993.38

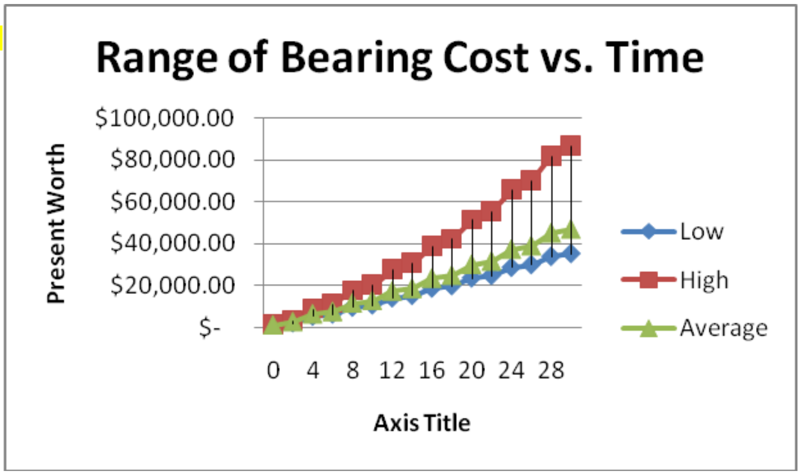


Table 24: High Economic Life

High Expected Economic Life		
Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 35,247.94	$ 86,829.59	$ 47,125.88
$ 39,385.98	$ 100,081.38	$ 53,806.75
$ 40,793.34	$ 105,715.05	$ 55,794.28
$ 244,077.82	$ 570,347.23	$ 363,050.01

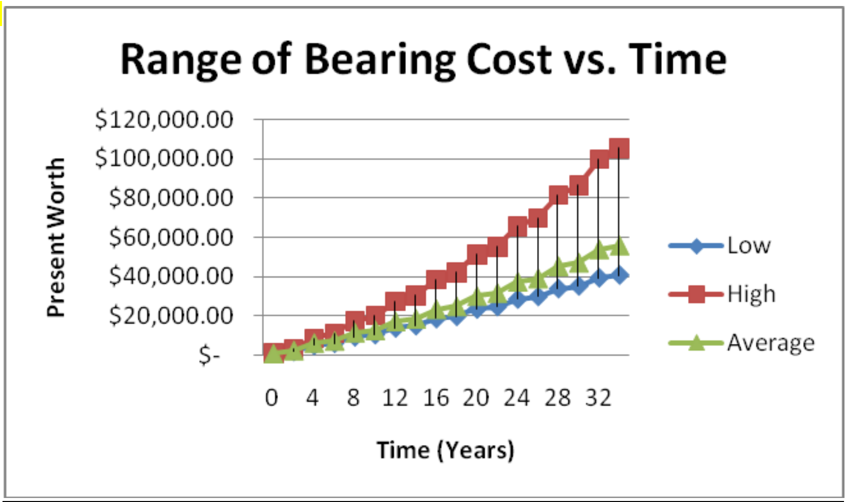


Table 25 concisely displays the low, high, and average estimates for Present Worth of Total Cost based on low, high, and medium expected economic life.

Table 25: Results

Low expexted economic Life			
Present Worth of Total Cost	$ 228,569.46	$ 485,825.05	$ 324,860.71
Medium Expected Economic Life			
Present Worth of Total Cost	$ 236,632.71	$ 524,138.63	$ 341,624.38
High Expected Economic Life			
Present Worth of Total Cost	$ 244,077.82	$ 570,347.23	$ 363,050.01

The next step involved the use of the data presented in Table 18, which is related to high labor cost. The following tables present the results obtained by assuming High and Low Labor Cost. Please note that labor price (LP) is highlighted in Table 18 instead of labor cost (LC). This is irrelevant since (LC) is a function of (LP). (LC) could have just as well have been highlighted instead of (LP). The Present Worth estimates for Low, Average, and High Expected Economic Life are displayed below. The values in Table 26 under Low Estimate were obtained by calculating the Present Worth of Inspection, Maintenance and Rehabilitation Costs based on High Labor Cost (LP). The same procedure was employed for calculating the values under High and Average Estimates respectively.

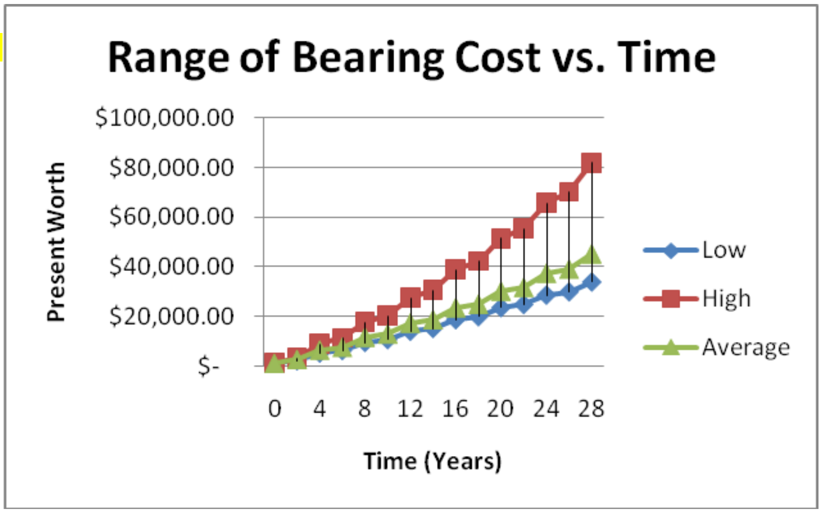
Table 26: High Labor Costs

High Labor Cost												
	Low Estimate				*High Estimate*				*Average Estimate*			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,3%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,2%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 1,020.30	$ -	$ -	1.0628	$ 2,125.62	$ -	$ -	1.041233	$ 1,561.85	$ -	$ -
4	1.0410	$ 1,041.02	$ 2,082.04	$ -	1.1296	$ 2,259.14	$ 3,388.71	$ -	1.084166	$ 1,084.17	$ 2,710.41	$ -
6	1.0622	$ 1,062.16	$ -	$ -	1.2005	$ 2,401.04	$ -	$ -	1.128869	$ 1,128.87	$ -	$ -
8	1.0837	$ 1,083.72	$ 2,167.45	$ -	1.2759	$ 2,551.86	$ 3,827.78	$ -	1.175415	$ 1,175.42	$ 2,938.54	$ -
10	1.1057	$ 1,105.73	$ -	$ -	1.3561	$ 2,712.14	$ -	$ -	1.223881	$ 1,223.88	$ -	$ -
12	1.1282	$ 1,128.18	$ 2,256.36	$ -	1.4412	$ 2,882.50	$ 4,323.75	$ -	1.274345	$ 1,274.35	$ 3,185.86	$ -
14	1.1511	$ 1,151.08	$ -	$ -	1.5318	$ 3,063.56	$ -	$ -	1.32689	$ 1,326.89	$ -	$ -
16	1.1745	$ 1,174.46	$ 2,348.91	$ -	1.6280	$ 3,255.98	$ 4,883.98	$ -	1.381601	$ 1,381.60	$ 3,454.00	$ -
18	1.1983	$ 1,198.30	$ -	$ -	1.7302	$ 3,460.50	$ -	$ -	1.438569	$ 1,438.57	$ -	$ -
20	1.2226	$ 1,222.63	$ 2,445.27	$ -	1.8389	$ 3,677.86	$ 5,516.79	$ -	1.497885	$ 1,497.89	$ 3,744.71	$ -
22	1.2475	$ 1,247.46	$ -	$ -	1.9544	$ 3,908.88	$ -	$ -	1.559647	$ 1,559.65	$ -	$ -
24	1.2728	$ 1,272.79	$ 2,545.57	$ -	2.0772	$ 4,154.40	$ 6,231.60	$ -	1.623956	$ 1,623.96	$ 4,059.89	$ -
26	1.2986	$ 1,298.63	$ -	$ -	2.2077	$ 4,415.35	$ -	$ -	1.690916	$ 1,690.92	$ -	$ -
28	1.3250	$ 1,325.00	$ 2,649.99	$ -	2.3463	$ 4,692.69	$ 7,039.03	$ -	1.760637	$ 1,760.64	$ 4,401.59	$ -
30	1.3519	$ 1,351.90	$ -	$ 209,544.30	2.4937	$ 4,987.44	$ -	$ 398,995.46	1.833233	$ 1,833.23	$ -	$ 287,817.62
32	1.3793	$ 1,379.35	$ 2,758.70	$ 213,798.90	2.6504	$ 5,300.72	$ 7,951.07	$ 424,057.24	1.908823	$ 1,908.82	$ 4,772.06	$ 299,685.15
34	1.4074	$ 1,407.35	$ -	$ -	2.8168	$ 5,633.67	$ -	$ -	1.987529	$ 1,987.53	$ -	$ -
35	1.4216	$ -	$ -	$ 220,343.32	2.9040	$ -	$ -	$ 464,632.18	2.028091	$ -	$ -	$ 318,410.22

Table 27 considers Low Expected Economic Life and shows the cumulative total cost at the end of every two years for low, high and average estimates.

Table 27: Low Economic Life

Low Expected Economic Life		
Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 244,792.24	$ 485,825.05	$ 334,943.50



The same process was followed for Average and High Economic Life respectively and the results are presented below in a similar manner.

Table 28: Average Economic Life

Average Expected Economic Life		
Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 35,247.94	$ 86,829.59	$ 47,125.88
$ 253,184.89	$ 524,138.63	$ 353,491.91

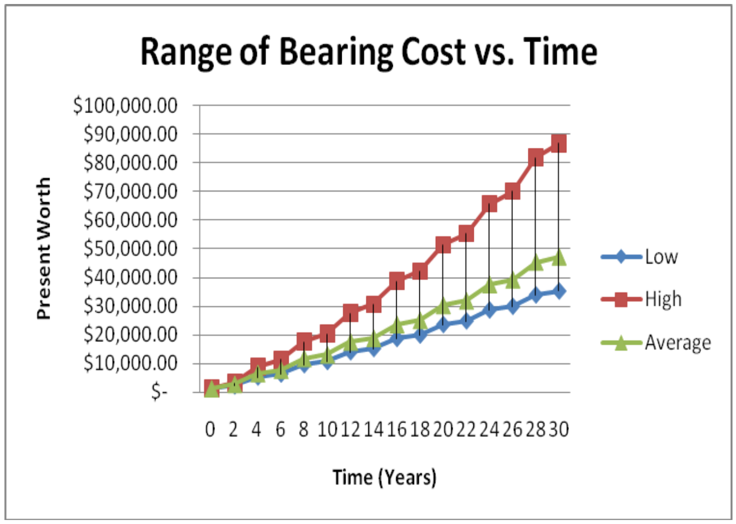


Table 29: High Economic Life

High Expected Economic Life		
Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 35,247.94	$ 86,829.59	$ 47,125.88
$ 39,385.98	$ 100,081.38	$ 53,806.75
$ 40,793.34	$ 105,715.05	$ 55,794.28
$ 261,136.66	$ 570,347.23	$ 374,204.51

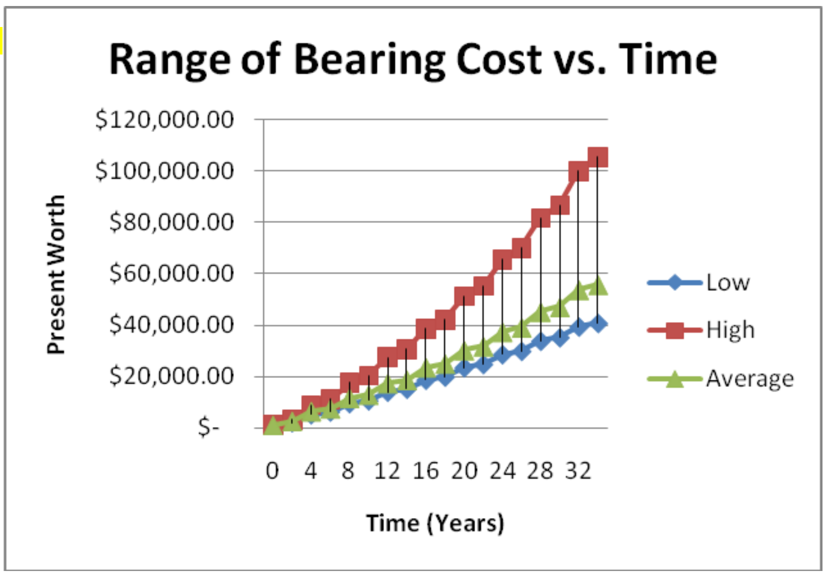


Table 30 concisely displays the low, high, and average estimates for Present Worth of Total Cost based on low, high, and medium expected economic life.

Table 30: Results

Low expexted economic Life

Present Worth of Total Cost	$ 244,792.24	$ 485,825.05	$ 334,943.50

Medium Expected Economic Life

Present Worth of Total Cost	$ 253,184.89	$ 524,138.63	$ 353,491.91

High Expected Economic Life

Present Worth of Total Cost	$ 261,136.66	$ 570,347.23	$ 374,204.51

Essentially the same process was followed in determining the low, high, and average estimates for Present Worth of Total Cost based on low, high, and medium expected economic life for Low Labor Cost, High/Low Maintenance Cost, High/Low Inspection Cost, and High/Low Real Rate. Please referr to the Appendix for tables and results based on the variation of these parameters.

Figure 52 presents all the data gathered in this study. The cost at each two year interval can be estimated from the distribution of the data points. For a specific area on the graph, the denser the distribution of data points, the higher the probability that the cost value is going to fall in that region.

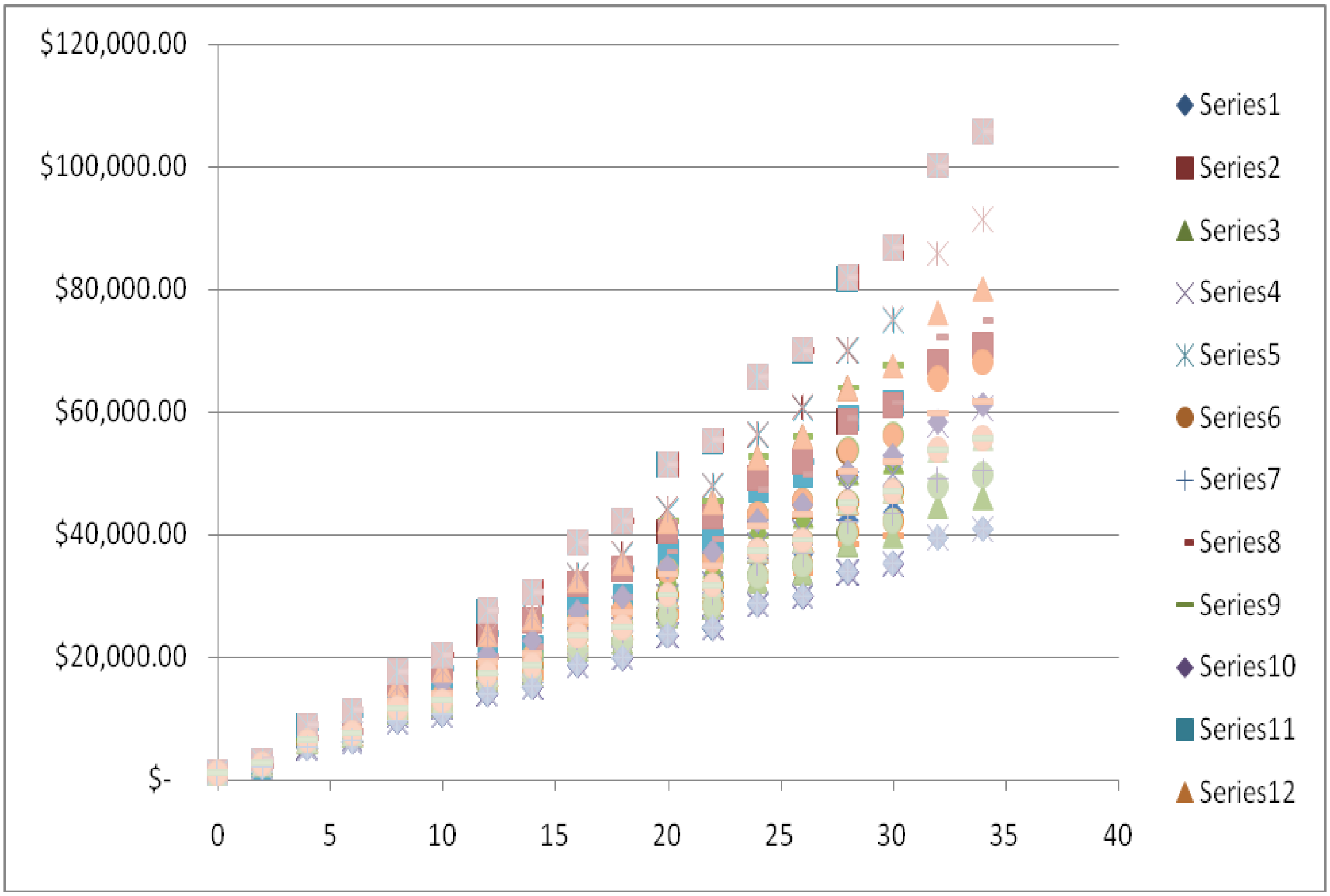


Figure 52: Superimposed Results

The following flow chart provides a clear picture on how each series was obtained. Note that L, A, and H represent High, Average, and Low Estimates respectively. Figure 53 shows how the nine series pertaining to Absolute Cost were obtained.

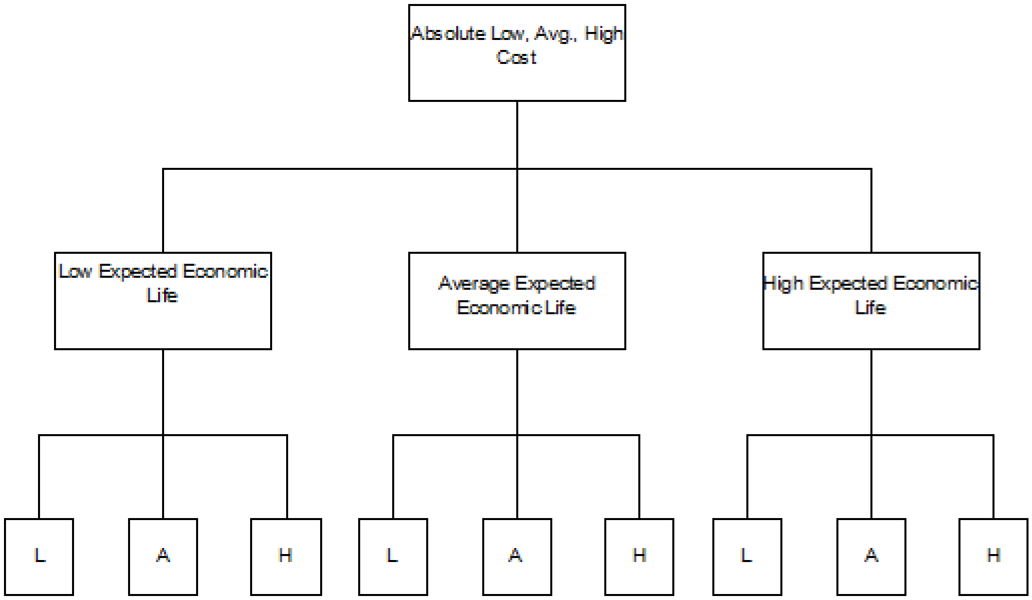


Figure 53: Process of Attaining Absolute Cost

Figure 54 shows how the eighteen series pertaining to Labor Cost were obtained. The series for Maintenance Cost, Inspection Cost, and Real Rate were obtained in a similar way. Each of these three variables produced eighteen series.

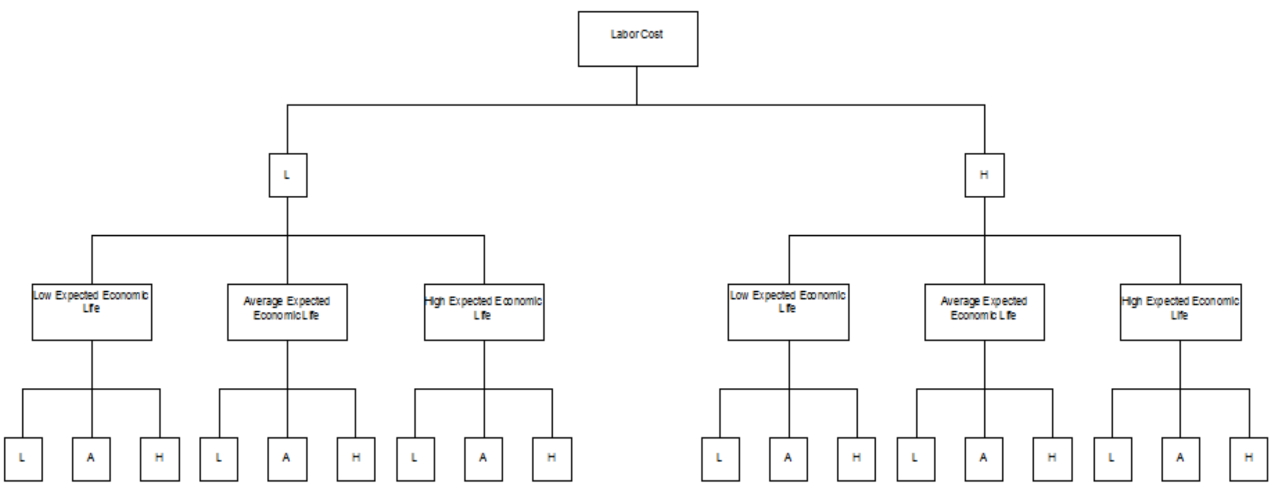


Figure 54: Process of Obtaining Labor Costs

Conclusions

Most of the graphs representing the projected maximum estimates appear to follow exponential curves. This trend becomes more subtle in the values representing average and low projected estimates. As the values approach low estimates, however, the dependence of cumulative cost with respect to time can be best approximated by a straight line. Figure 52 clearly shows the trend of the superimposed results. The concluson is valid for the time interval between the initial installation of the bearings and the replacement at the end of their economic life.

It is important to note that the life-cycle cost model compiled in this study does not account for the fact that due to the location and positioning of the bridge on the site, one or more of the bearings may deteriorate faster than others. Therefore, these bearings will increase costs associated with maintenace and rehabilitation. Since all the stringers of the bridge need to be jacked before any one bearing is removed or rehabilitated, the maximum cost values presented in this report may be exceeded. In such a scenario the rehabilitation costs may almost double the value initially predicted.

Figure 52 shows that as time approaches the end of the bearings' expected economic life, the cumulative cost becomes progressively harder to predict. This is due to the fact that the maximum projected cumulative cost of the bearings increases at a greater rate than the projected minimum and average cost. Each series represents a group of data obtained by assuming a high and low value for a specific variable and evaluating the cumulative costs for low, average, and high estimates at low, average, and high expected economic life. Thus, there are are 81 series in total. Note that not all the series are displayed on the right hand side of the graph due to size limitations. However, all of them appear on the graph. It is important to understand that the series do not represent the only paths that the real cumulative cost can follow. The cumulative cost can follow any path that lies between the maximum and minimum values at each time interval. However, the cost will always be between the minimum and maximum values at each two year interval.

A careful inspection of the curves representing average estimates, reveals that Real Rate is the most influential factor in determining the projected cumulative cost. The effect of Real Rate is also the most sensitive to changes in the expected economic life. This factor also determines the relative impotance of all the other parameters. An increase in the Real Rate will increase all other costs and a decrease of the Real Rate will have the oposite effect. However, changes in the assumed value for Real Rate will have little or no impact on the distribution of the data in Figure 52. Inspection cost turned out to be the second most influential factor in the analysis and the second most sensitive to changes in the expected economic life. See the Appendix for specific results on the Inspection Cost.

At the beginning of this study, the bearing type was chosen based only on two major criteria as specified in Chen & Duan; technical requirements and overrall cost during the bearing’s lifetime. Based on the results from this study, a third major criteria was identified. In chosing a bearing type the owner and the engineer will need to know the total maximum and minimum cost of the bearing, the expected economic life, and the technical requirements. The ability to determine the most probable cumulative cost at each time interval with the highest degree of confidence will also constitutes a crucial factor in the decision-making process. In many instances it would not be wise to pick a bearing type based only on the fact that it provides the lowest maximum cumulative cost throughout its lifetime, if the cost of the bearing throughout its expected economic life is hard to predict. Therefore, a graph such as Figure 52 needs to be obtained, and the distribution of the superimposed results should be carefully examined. If the data points are scattered, then this means that the costs are highly variable, and the most probable Life-Cycle curve will be harder to predict.

Assuming a lower value for the high estimate of the Real Rate will considerably decrease the maximum possible cost at each time interval. Furthermore, by also assuming a larger time interval between inspections and a lower value for the high estimate of the Inspection Cost the probability range of the projected cumulative costs will shrink. In such a situation it will be relatively easier to establish the Life-Cycle Cost curve with the highest probability of occurrence.

It is also important to understand that the cost range for each of the factors that influence life-cycle cost has its own unique significance from a probability perspective. For instance, at a specific point in time, the average value of labor cost may be less likely to occur than the maximum value. This means that the average value at that specific time is less representative than the maximum value. Such a scenario raises the need for a third dimesion. Thus, the model presented in this study can be further improved by assigning a number (weight) to each estimate for every time interval at which they are evaluated. The smaller the ‘weight’ the higher the probability that that will be the true value. The number representing the ‘weight’ can be obtained either from a probability distribution curve of previous data or from previous experience. The weight should be a function of time and type of factor; IC, Real Rate, MC etc. Now, a third dimension can be added to the graph in Figure 52 that can potentially increase the accuracy of the life-cycle cost analysis model. A three dimensional plot made up of straight lines parallel to the third dimesion can then be constructed. The density of the line in the three dimesional space will map out the path of the most probable life-cycle cost curve.

VII Effect of Bracing on Lateral Load Distribution

This chapter presents the methodology and results of a study on how the use of bracing in girder bridges can help to distribute loads across the width of the bridge. The study was conducted using the finite element method; a popular software title (ANSYS) was used. The study had the following goals:

1. Develop a simplified finite element model whose results can be validated by applying basic principles of structural behavior
2. Investigate the effect of different bracing types and spacings on maximum girder moment
3. Investigate the effect of bracing on the shear lag effect

To achieve these goals, relevant literature was consulted. For each of the goals listed above, this chapter presents a summary of the literature consulted, an explanation of the study's modeling and analysis methodology, and the results of the study.

Development of a Simplified Finite Element Model

This project used a simplified finite element model to study the effect of bracing on lateral load distributions. The alternative to using a simplified model would be a detailed model, which would model all parts of the system such as the reinforcing bars in the deck or the shear studs at the girder-deck interface. This type of model would require large amounts of computing power and an in-depth knowledge of modeling techniques. To decrease the required computing power and the required modeling experience, a simplified model was used. This section presents the results of a literature review of simplified modeling techniques, and then presents a description of the model and the modeling techniques used in this study.

VII.1.1 Simplified Model Literature Review

An article was consulted for the development of the simplified finite element model used in this project. This article was written by Wonseok Chung and Elisa D. Sotelino and was titled "Three-dimensional finite element modeling of composite girder bridges". The article investigated the use of four different simplified finite element models, and it discussed the validity of each model's results and the required mesh fineness required to achieve valid results. The authors were particularly interested in the flexural behavior of the bridge. They sought to produce accurate results for the bending stresses and moments in the deck and girders.

One of the models investigated in Chung and Sotelino's article can be seen in the figure below. It should be noted that all of the model's investigated in Chung and Sotelino's article had similar components; the main differences were in the way the girders were modeled, which will be described later in this chapter.

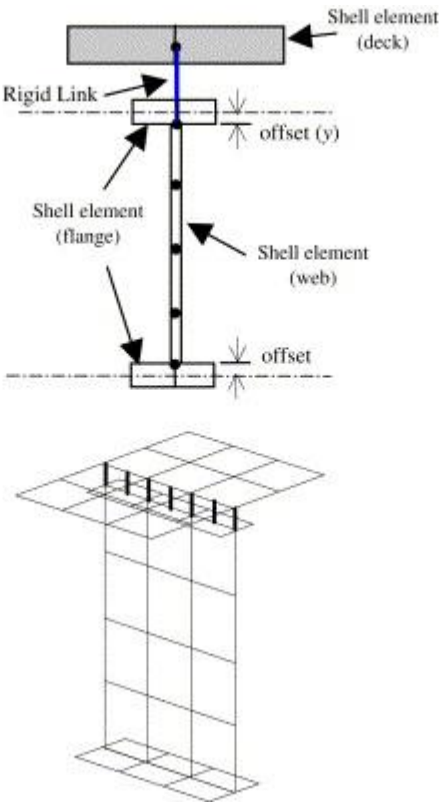


Figure 55: Simplified FEM G1 Diagram
http://www.sciencedirect.com/science?_ob=ArticleURL&_udi=B6V2Y-4GYNY7H-3&_user=74021&_rdoc=1&_fmt=&_orig=search&_sort=d&view=c&_acct=C000005878&_version=1&_urlVersion=0&_userid=74021&md5=cb06bdab77e96bcb52564ef27434c6b7

A key part of the models used by Chung and Sotelino were the shell elements. Although brick elements would provide a more realistic representation, they require twice as many nodes as shell elements and a finer mesh size (Chung & Sotelino, 2005). The use of shell elements requires that the model geometry be laid out in such a way the moment of inertia for a section is modeled properly. For example, having the deck shell elements and the girder shell elements sharing common nodes would not account for the increase in moment of inertia due to the deck's and girder's thicknesses. To properly model the section's moment of inertia, Chung and Sotelino placed the shell elements at the midpoint of the structural element being modeled. This can be seen by observing the "offset" shown in the previous figure.

The technique for properly modeling the section's moment of inertia presented in the previous paragraph requires a gap between the deck and the top flange of the girder. To connect the two elements, Chung and Sotelino propose the use of rigid links. Rigid link elements in ANSYS behave like typical link elements and do not have rotational degrees of freedom. As a result, the use of rigid link elements in this study caused the deck to bend independently of the girders. This caused the deck to behave like a symmetric beam, with its neutral axis located at its mid-height and equal tensile and compressive stresses on the top and bottom faces. To properly model the composite action of the deck and girders, the authors of this project suggest using rigid beam elements (whose nodes have rotational degrees of freedom) to connect the deck to the girders. The use of these elements causes high compressive stresses on the deck's top face, and low tensile stresses on the deck's bottom face, as would be predicted by general composite slab-girder theory. It is therefore possible that this project did not follow the same modeling philosophy as Chung and Sotelino with respect to connecting the bridge girders to the deck. It should be noted however that Chung and Sotelino did not provide documentation on the properties of their rigid links (whether or not they had rotational degrees of freedom). In their article they used another finite element software package, ABAQUS. It is possible that in ABAQUS, rigid link elements behave differently than the rigid link elements in ANSYS.

As mentioned earlier, Chung and Sotelino developed several models and compared the accuracy of the results for each model. The models differed in the way the girders were modeled. For each model, a different combination of shell and beam elements were used to model the girder flanges and web. The following table summarizes the different models investigated:

Table 31: Simplified FEM Summary

Model Name	Girder Part	
	Flange	Web
G1	Shell Element	Shell Element
G2	Shell Element	Beam Element
G3	Beam Element	Shell Element
G4	Beam Element	Beam Element

http://www.sciencedirect.com/science?_ob=ArticleURL&_udi=B6V2Y-4GYNY7H-3&_user=74021&_rdoc=1&_fmt=&_orig=search&_sort=d&view=c&_acct=C000005878&_version=1&_urlVersion=0&_userid=74021&md5=cb06bdab77e96bcb52564ef27434c6b7

The results of Chung and Sotelino's study found that models G1 and G2 require fine mesh sizes to produce acceptable results, while models G3 and G4 required little or no mesh refinement. They also concluded that beam elements are best for capturing the bending effects of the girders (Chung & Sotelino, 2005). Based on the recommendations of Chung and Sotelino, and the geometric requirements for modeling the bridge bracing, the authors of this project decided to use model G3 shown in the previous table.

VII.1.2 Development of the Model

Once an acceptable modeling technique was established, the model was constructed. The model was based on Option 2 (3 foot spacing), from the superstructure chapter. The following figure and table summarize the important characteristics of the model:

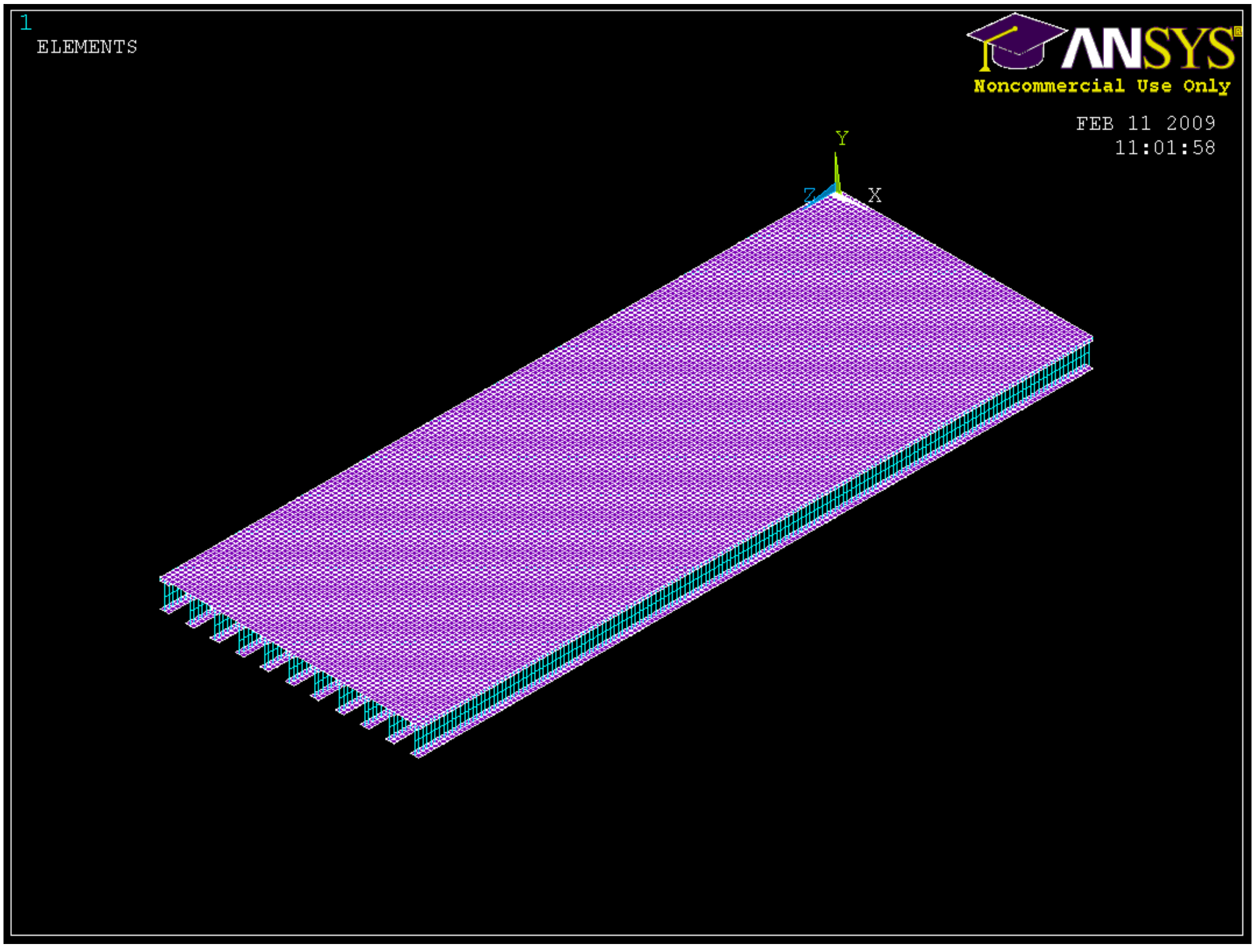


Figure 56: Superstructure Finite Element Model

Table 32: Model Summary

Girder Flange Properties			**Deck Properties**	
Section Size	W36X160		Thickness	8 in
E	29E6 psi		E	3834 psi
Poisson's Ratio	0.3		Poisson's Ratio	0.15
Element Size	6"X6"		Element Size	6"X6"
Girder Web Properties			Length (z direction)	81 ft
Section Size	W36X160		Width (x direction)	30 ft
E	29E6 psi			
Poisson's Ratio	0.3			
Element Size	6"			

As described earlier, the deck was modeled with shell elements, specifically Shell63, as were the girder flanges. The girder web was modeled as a three dimensional beam element, Beam4. The girder web (represented by beam elements) was connected to the flanges (represented by shell elements) by rigid beams; the girder flanges were connected to the deck in the same manner. The element type used for the rigid beams was MPC184. The brace elements were modeled using either beam elements, or link elements (Link8). The following figure provides an enlarged view of the model, and shows how the different elements came together:

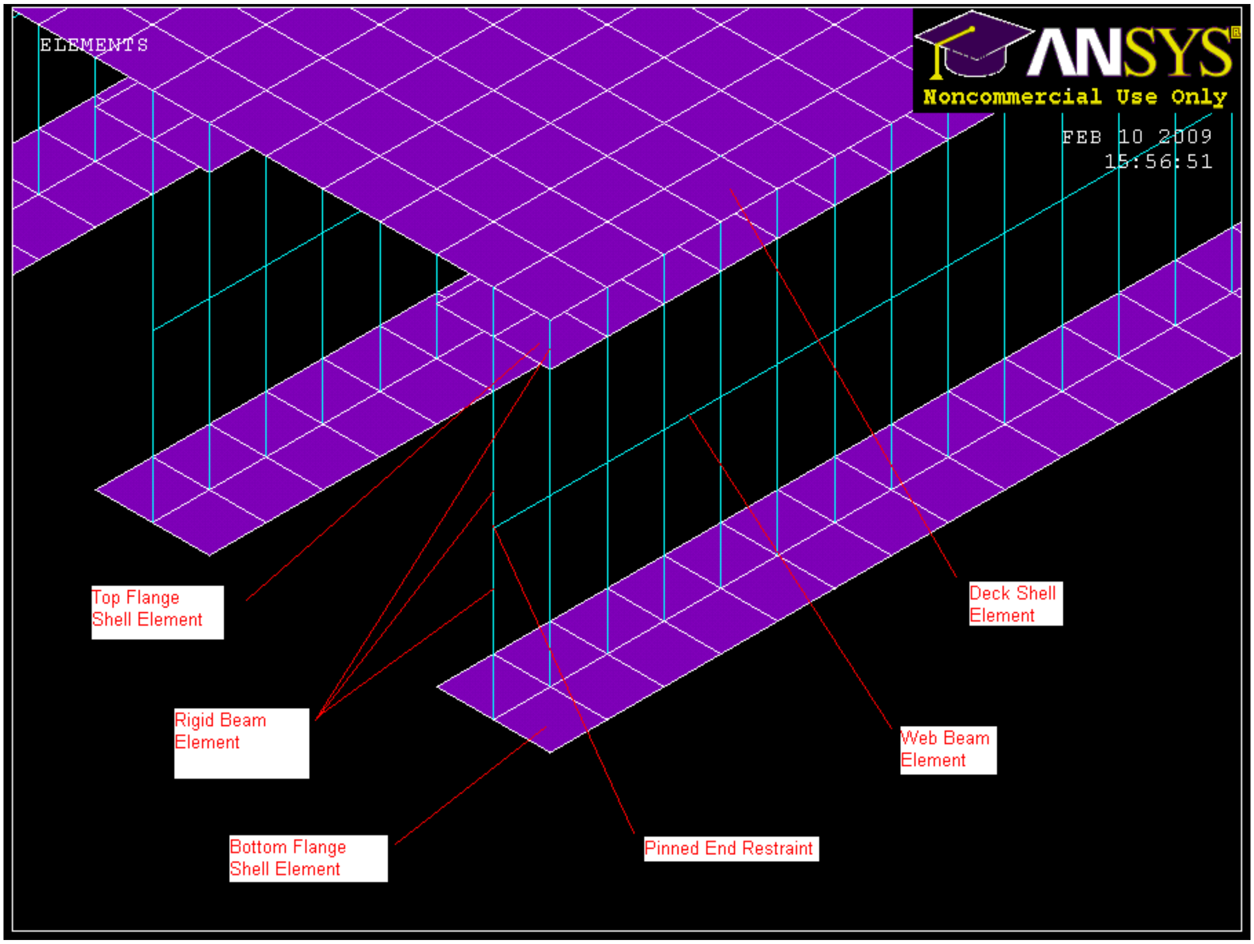


Figure 57: Simplified FEM Components

The model was restrained at the girder's ends. Standard pin and roller boundary conditions were applied at the end node of the girder web, as illustrated in the previous figure. The model was loaded with its own dead weight, the AASHTO specified distributed live load, and the AASHTO specified design truck. The structure's dead load was applied as an area load across the deck, and as a line load along the length of the web of the girders. The AASHTO distributed live load was added to the magnitude of the dead load. The areas to which the distributed live load was applied can be seen in the following figure:

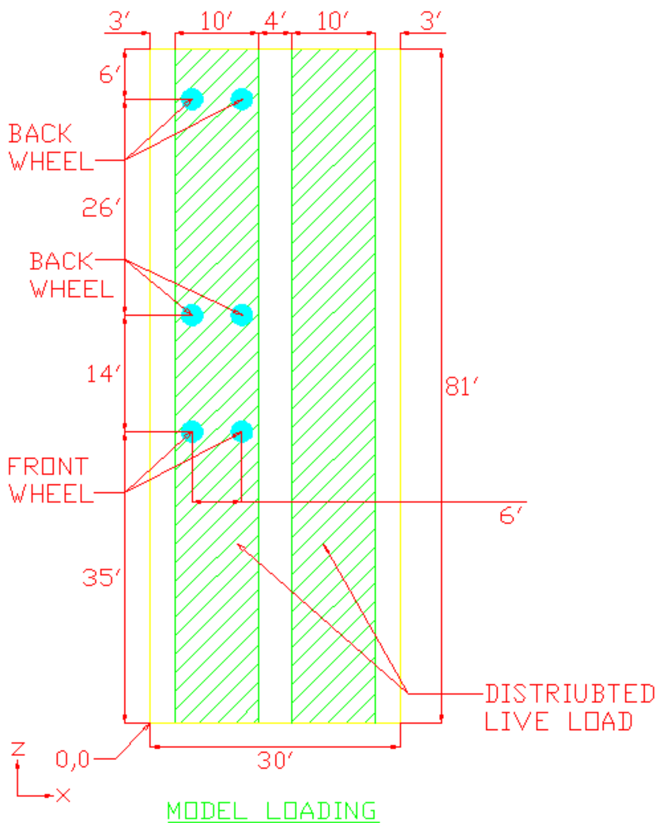


Figure 58: Live Load Location

The figure shows the location of the distributed live load, as well as the location of the design truck. The loads from the wheels of the AASHTO design truck were applied as point loads. Factored loads were used in the model.

Three basic types of bracing were investigated in this project. They can be seen in the following figure:

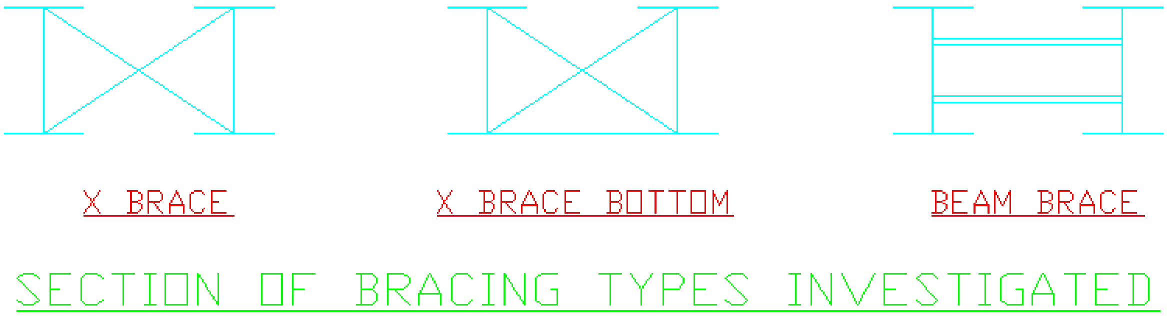


Figure 59: Bracing Types Investigated

Both types of X Bracing were modeled as link elements, while the beam brace was modeled as a beam element. The main difference between the "X Bracing" and the "X Bracing Bottom" was that the "X Bracing" consisted of two diagonal elements, while the "X Bracing Bottom" consisted of two diagonal elements and one horizontal element across the bottom flange of the girder. Two different sizes of each type of bracing were investigated to determine the effect of the bracing stiffness on load distribution. L6X6X1/2 and L2.5X2X3/8 angles were used for the X Bracing, and W12X53 and W8X24 beams were used for the beam bracing. Three different bracing spacings were investigated: 9ft, 18ft, and 27ft; it should be noted that AASHTO recommends a maximum bracing spacing of 25 feet (AASHTO, 2007). For a given spacing, bracing was provided across the width of the bridge (bracing was placed between each girder).

In addition to the bracing types shown above, the effect of horizontal bracing was also investigated. The horizontal bracing used can be seen in the figure below:

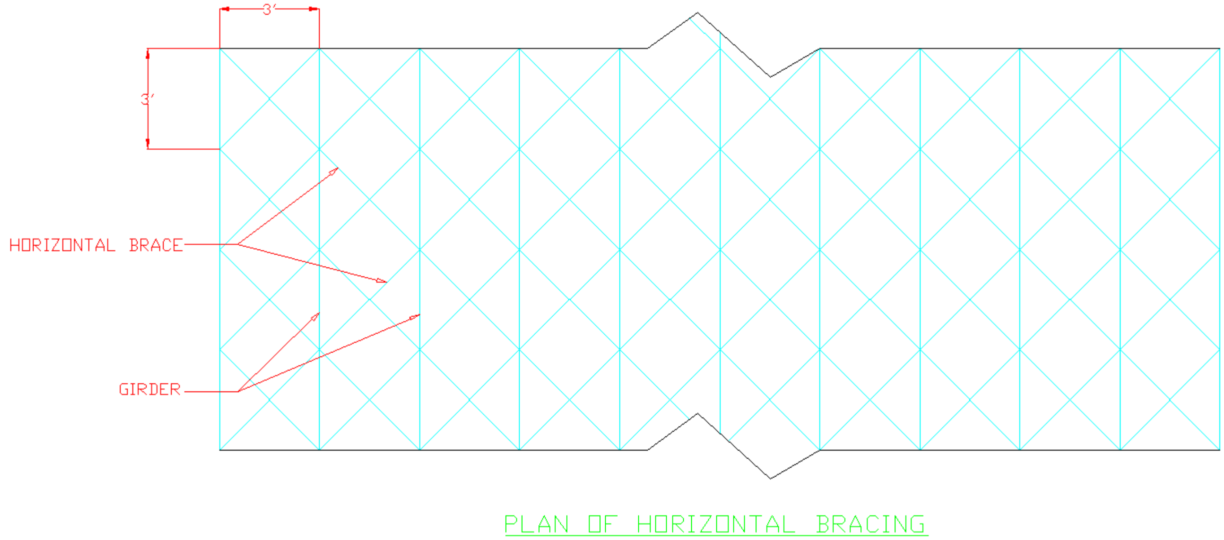


Figure 60: Horizontal Bracing Plan

For each bracing type and spacing described in the previous paragraph, the model was solved with and without the horizontal bracing. The horizontal bracing was modeled with link elements, and the same angle sizes were used as for the X Bracing (L6X6X1/2 and L2.5X2X3/8). When the less stiff bracing members were used, the less stiff horizontal bracing was used; when the more stiff bracing members were used, the more stiff horizontal bracing was used.

VII.1.2.1 Validating the Model

The model was validated by observing basic structural engineering principles. For example, it was expected to find high tensile stresses within the bottom flange of the girders, and compressive stresses within the top flange. The following is a list of different parameters that were checked to validate the model results:

- Plots of deflection
- Contour plots of stress distribution in the transverse and longitudinal directions:

  - On the top part of the deck, tensile stresses over the girders and compressive stresses in between the girders (transverse distribution)
  - Tensile stresses on bottom girder flanges and compressive stresses on top girder flanges (longitudinal distribution)
- Maximum bending stress of a beam in a separate ANSYS file (beam modeled with same philosophy as bridge girders) → comparison of maximum stress from ANSYS to maximum stress predicted by flexure formula

Once the simplified model was validated, the effect of bracing on load distribution could be studied. The remainder of this chapter presents a summary of the relevant literature consulted for this study, and the results of the study itself.

Moment Distribution Factors

One area of lateral load distribution that was studied in this project was the concept of a moment distribution factor. This factor relates the maximum moment in a bridge girder determined from a simple statics analysis of the beam, to the maximum girder moment determined by methods which take the stiffness of the system (bridge deck, adjacent girders, bracing, etc.) into account. This section presents background information on moment distribution factors; both the current factors provided by AASHTO and factors proposed by researchers are discussed. Also, the results of this study, which include a comparison of moment distribution factors for different bracing types and spacings are presented.

VII.1.3 Background

In Chapter IV of this report, “Superstructure Design,” a method for analysis was described for the bridge deck and girders. The analysis consisted of a simply supported or continuous beam, loaded with its own dead weight, the AASHTO distributed live load, and the AASHTO design truck. The beam was analyzed in Risa following standard engineering analysis procedures, and the deck and girders were designed to resist the maximum moment. While this approach provides young engineers with a first step in the analysis and design of bridge components, it grossly underestimates the strength of the bridge system. Because the bridge is made up of many girders, each connected by the bridge deck, the analysis described above does not capture the lateral distribution of the deck loads to multiple girders. Some of the loads will be transferred through the bridge deck into the other girders. To account for this phenomenon, AASHTO has developed distribution factors. These factors are described in their specification for highway bridges, and they depend upon the type of deck used and the girder spacing (Tonias, 2007).

The AASHTO distribution factors are a source of controversy for many bridge engineers. Practicing engineers claim that the factors are too conservative and do not take into account other parameters which affect load distribution (Tonias, 2007). These parameters include the depth of the deck, span length, spacing of secondary members, stiffness of primary members, stiffness of secondary members (e.g. bracing), type of bracing employed, and size and position of loads (Tonias, 2007). Modeling the effect of each of these parameters can be a very complicated task. However, the use of the finite element method provides a reasonably simple solution (Eamon and Nowak, 2002).

There have been many studies conducted on load distribution in girder bridges, many of which use simplified finite element models. The researchers validated their results through the use of detailed finite element models, large scale experimental tests, and case studies of existing bridges. This project studied one research paper, “Effects of Edge-Stiffening Elements and Diaphragms on Bridge Resistance and Load Distribution” by Christopher D. Eamon and Andrez S. Nowak. The paper studied the effect of edge stiffening elements, such as sidewalks and concrete barriers, and diaphragms (or bracing) on load distribution. The paper used a simplified finite element model, similar to the one constructed for this project, for the study.

The following conclusions were drawn from the work:

1. Including edge stiffening elements and diaphragms in an analytical model decreases the maximum girder moment
2. Diaphragms are generally more effective in bridges with wide girder spacings and long spans
3. The number of diaphragms (and therefore diaphragm spacing) has little impact on maximum girder moment

Based on the work of Eamon and Nowak, one can reasonably infer that including diaphragms (bracing) in an analytical model reduces the maximum moment, which could potentially allow the designer to choose a more economical girder section. Although the argument could be made that the cost of adding bracing members would offset any potential savings from smaller girder sizes, in section 6.7.4.1 of the AASHTO bridge design specification, a guideline is provided that recommends that bracing be used to prevent lateral displacement due to wind loads. Therefore, simply including the bracing members in an analytical model could reduce construction costs (Eamon and Nowak, 2002). This project mimicked Eamon’s and Nowak’s results by comparing the maximum girder moments in bridges with a variety of bracing types and spacings, to the maximum girder moment of a bridge without any bracing.

It is important to note that, in the paper by Eamon and Nowak, and in many other papers written about load distribution in girder bridges, the authors caution their readers about their results. They say that load distribution is dependent on many factors, but most studies only investigate a few of them. The researchers suggest that in order to make a universal model that could capture all parameters in all girder bridges, many models would need to be constructed and analyzed, and a general theory would need to be formulated (Eamon and Nowak, 2002). Perhaps this is the reason that AASHTO has not yet accounted for the effect of bracing members on lateral load distribution.

This project conducted a study very similar to the one done by Eamon and Nowak. The maximum girder moment of a model with no bracing was compared to the maximum girder moment of models with a variety of bracing types and spacings. By conducting the study in this manner, Eamon's and Nowak's claim that bracing decreases maximum girder moment could be substantiated. Also, by testing different bracing types, insight could be gained for determining what type of brace is most effective in decreasing maximum moment. Finally, Eamon and Nowak claim that bracing spacing has little impact on maximum moment. However, until recently, AASHTO recommended a maximum spacing of 25 feet. Although AASHTO's recommendation was intended to limit lateral deflection due to wind loads, this project investigated if spacing requirements would increase load distribution and decrease maximum girder moment

VII.1.4 ***Moment Distribution Results***

To determine the effect of bracing on girder moment reduction, the maximum girder moment of a model with no bracing was compared to the maximum moments obtained from models with bracing. This comparison led to the development of a Moment Distribution Factor (MDF), which was taken as the ratio of: (Max. Girder Moment with Bracing)/(Max. Girder Moment without Bracing). Also, the maximum stress in bracing members was recorded; axial stresses were recorded for both types of X Bracing, and bending stresses were recorded for Beam Bracing.

The first analysis investigated the importance of using a MDF to the analysis of a bridge girder. For this analysis, a simple beam was analyzed in Risa, with the truck load positioned on it in the same way as was done in the ANSYS model. The appropriate dead and distributed live loads were also applied, and the maximum moment was recorded. This provided a baseline with which to compare. Next, the AASHTO specified MDF was applied, which established the AASHTO design moment. Finally, the maximum girder moments from the ANSYS model without bracing, and the ANSYS model with Bottom X Bracing spaced at nine foot intervals was established. The following table summarizes the findings:

Table 33: Comparison of AASHTO MDF to MDF Predicted by ANSYS

Distribution Factor Comparison			
	Max. Moment (in*lb)	MDF (No Factor)	MDF (AASHTO Factor)
No Factor	30273420	1.000	1.833
AASHTO Factor	16512774	0.545	1.000
FEM (no bracing)	714260	0.024	0.043
FEM (bracing)	360880	0.012	0.022

This table shows that the AASHTO factor decreases the MDF nearly 50% compared to using no factor as can be seen in the third column; the ANSYS models predicted a very small MDF when compared to an analysis not using any factors. The ANSYS models also predicted a large reduction in girder moment compared to the AASHTO factor, as can be seen in the fourth column. This table highlights the importance of using MDF in the analysis and design of girder bridges. Simply using the AASHTO factors could reduce moments, and therefore section sizes, by nearly 50%. The use of more precise analysis techniques, such as finite element models could justify the use of even smaller sections.

The next analysis compared the MDF for each bracing type and spacing investigated. The following table summarizes the results:

Table 34: Comparison of MDF by Bracing Type

Spacing (ft)	9	18	27
No Bracing	1	1	1
Horizontal Bracing	0.622	0.622	0.622
X Bracing	0.713	0.827	0.887
X Bracing Bottom	0.505	0.629	0.689
Beam Bracing	0.528	0.646	0.728
Hor. Bracing & X Bracing	0.640	0.668	0.616
Hor. Bracing & X Bracing Bottom	0.685	0.754	0.682
Hor. Bracing & Beam Bracing	0.664	0.719	0.598
	denotes lower MDF than Horizontal Bracing		

In this table, the highlighted cells show bracing types and spacings which have a lower MDF than the horizontal bracing alone. It was originally expected that the horizontal bracing would produce a MDF close to 1.0, while other bracing types would produce smaller MDF's. Because most of the bracing types and spacings investigated produced higher MDF's than the horizontal bracing alone, unexpected results were recorded. These results suggest that using horizontal bracing alone provides a more efficient system for carrying load. The results also suggest that the use of other bracing types, even with horizontal bracing, could increase the maximum girder moment. It should be noted that the weight of the bracing members was not modeled, so that could not account for increased girder moment.

The previous table does not show a strong correlation between bracing spacing and MDF. Although some bracing types show an increase in MDF as bracing spacing increases, other bracing types show other trends such as an increase in MDF from 9 foot to 18 foot spacing, and a decrease in MDF from 18 foot to 27 foot spacing. Because there is not a strong correlation between the two parameters, it is difficult to make conclusions regarding the effect of bracing spacing on MDF.

Table 34 clearly shows the importance of bracing to MDF's. The use of bracing provided an average MDF of 0.669, and a value as low as 0.505. These large reductions in maximum girder moment could certainly allow for the use of smaller and more economical girder sections.

The final investigation was related to bracing stiffness. The stiffness of bracing members was reduced for all bracing types spaced at nine foot intervals, and the resulting parameters of interest were recorded. The original bracing members were chosen arbitrarily. The lower stiffness X Bracing and X Bracing Bottom members were chosen based on an analysis of strength requirements to resist wind loads. The ratio of high stiffness X Bracing members to low stiffness was 3.72 to 1. The lower stiffness beam members were chosen arbitrarily. The ratio of high stiffness beam bracing to low stiffness was 5.14 to 1. The following table summarizes the results of this investigation:

Table 35: Comparison of MDF by Bracing Stiffness

	MDF High Stiffness	MDF Low Stiffness	Max. Brace Load High Stiffness (lb X bracing; in*lb beam bracing)	Max. Brace Load Low Stiffness (lb X bracing; in*lb beam bracing)
Horizontal Bracing	0.622	0.493	32444	14727
X Bracing	0.713	0.883	27485	11189
X Bracing Bottom	0.505	0.664	35970	19164
Beam Bracing	0.528	0.634	-118500	-35436
Hor. Bracing & X Bracing	0.640	0.498	33712	14687
Hor. Bracing & X Bracing Bottom	0.685	0.502	-48905	14723
Hor. Bracing & Beam Bracing	0.664	0.648	244640	-63374
	denotes higher MDF than high stiffness			

It was expected that the use of lower stiffness bracing members would yield higher MDF's, however; this was the case only for the bracing types highlighted in yellow. What is particularly interesting is that the low stiffness bracing members actually carried less load than their high stiffness counterparts, while still providing smaller MDF's. These results do not correlate with those found by Eamon and Nowak, who found that increasing the ratio of bracing stiffness to girder stiffness decreased MDF's.

Despite the unexpected results regarding the relative MDF's for various bracing configurations, it can still clearly be seen that the use of some sort of bracing decreases MDF's. This finding has been confirmed by many researchers, including Eamon and Nowak.

Shear Lag and Effective Width

The effect of bracing members on decreasing the shear lag effect in girder bridges was studied in this project. Although there was some discussion in the literature of a possible reduction in the shear lag effect due to bracing members, many of the articles were related to box girder bridges, which were not studied in this report. Nevertheless, relevant background information was gathered, and some change in the amount of observed shear lag was noted in the project results. This section provides a brief description of the shear lag effect and the concept of effective width. Next, an analysis of the ANSYS results is presented, which investigates if bracing members do decrease the shear lag effect.

VII.1.5 Background on Shear Lag and Effective Width

Shear lag is a phenomenon that is caused by a violation of a basic assumption of beam theory: sections that are plane before bending remain plane after bending. Although this assumption holds true for beams, whose widths are much smaller than their lengths, it does not hold true for plates, which can be equally wide as long (Cai, Nie, and Tian, 2004).

The shear lag effect is especially noticeable in composite girder slab systems, in which the slab bends with the girders. The figure below illustrates the shear lag phenomenon. The phenomenon is caused by the shear connectors between the slab and girder restraining the portion of the slab directly over the girder. This zone of the slab cross-section experiences a larger longitudinal strain and a higher stress than those zones farther away from the girder (Cai, Nie, and Tian, 2004). This phenomenon is illustrated in the top portion of the following figure:

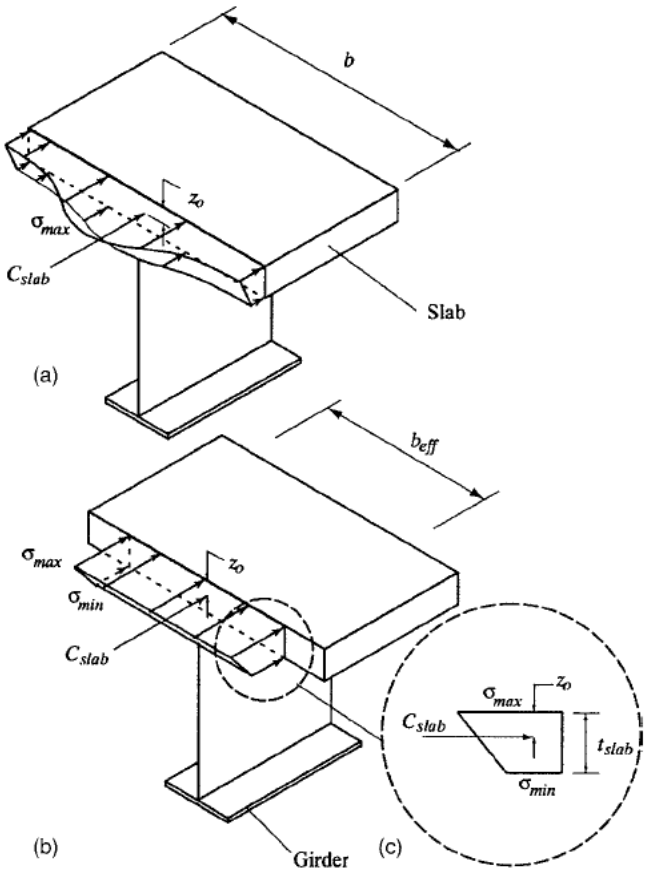


Figure 61: Shear Lag in Composite Sections
http://scitation.aip.org/getpdf/servlet/GetPDFServlet?filetype=pdf&id=JBENF2000012000003000325000001&idtype=cvips&prog=normal

To simplify the treatment of the shear lag phenomenon, design engineers have developed the concept of "effective width," which is illustrated in Part (b) of the previous figure. This concept assumes that the stress in the slab is uniform across the effective width, and that there is no stress outside of the effective width. AASHTO states that the effective width depends on the span length, slab thickness, and girder spacing (Tonias, 2007).

This project studied two papers related to effective width. The first one was called "Proposed Effective Width Criteria for Composite Bridge Girders" and was written by Ahn, Aref, Chen, and Chiewanichakorn. The second was called "Effective width of steel–concrete composite beam at ultimate strength state" and was written by Cai, Nie, and Tian. Both of these papers claim that the established approach for defining effective width is incorrect; the paper by Ahn, Aref, Chen, and Chiewanichakorn went as far as calling the technique "archaic." Both research groups agree that at the ultimate limit state, the effective width should be taken as the physical width of the slab, and Ahn, Aref, Chen, and Chiewanichakorn suggest that the effective width should be taken as the physical slab width at the service limit state.

Because the effective width depends on so many different factors, no one article was found that provides a comprehensive definition of effective width. Although all of the articles made proposals for alternative definitions, they all warned that a more comprehensive study should be undertaken before applying the definitions in practice (Cai, Nie, and Tian, 2007 and Ahn, Aref, Chen, and Chiewanichakorn, 2007). The following conclusions were made in both papers:

1. The effective slab width at the strength limit state should be taken as the physical width
2. The effective width can depend on boundary conditions; many different boundary conditions have not yet been studied
    a. It should be noted that engineers are generally more interested in what is going on at the span midpoint, where the effects of boundary conditions are least significant
3. Effective width depends on the loading condition

This study investigated whether or not bracing helps to decrease the shear lag effect. The results of the study could be applied to background research presented in the previous paragraphs to determine how various bracing types influence the effective width at the service limit state. Although factored loads were used for this model, the effective width at the service limit state can still be evaluated because the analysis conducted was linear.

VII.1.6 ***Results of Shear Lag Investigation***

The effect of bracing on shear lag and effective width was determined by graphing the stress intensity in the longitudinal direction of the bridge as one moved across the bridge in the transverse direction. It was expected that deck stresses would be higher directly over the girders and lower in the regions in between the girders. When interpreting Figures 61, 62, and 63 the following criteria should be considered:

- The flatter the line representing the stress distribution, the smaller the shear lag effect and greater the effective width (this represents smaller stress concentrations)
- The girders are located at three foot intervals (0 in, 36 in, 72 in)
- The stress distribution was recorded over the region from the exterior girder to the region in between the second and third interior girder

For this investigation, all bracing types were investigated; the high stiffness bracings spaced at nine foot intervals were investigated. The following figure shows the stress distribution for the models with no bracing, horizontal bracing, and an ideal stress distribution:

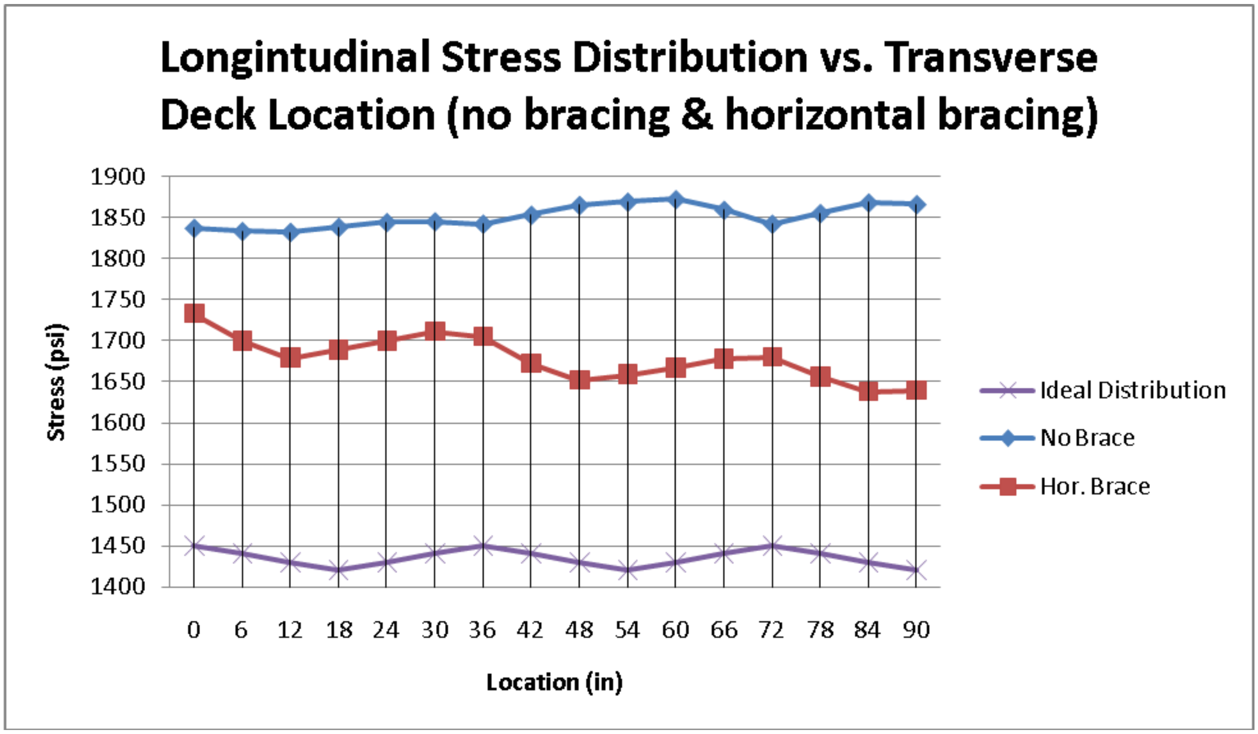


Figure 62: Longitudinal Stress Distribution vs. Transverse Deck Location (no bracing & horizontal bracing)

The ideal stress distribution shows a stress disribution with the highest levels of stress over the girders, and the lowest levels of stress in between the girders; the values shown in the figure are arbitrary, and the variation of the distribution was established based on the shear lag theory presented earlier in this chapter. The ideal distribution is not realistic for this model because in this model the maximum stress levels over the girders should decrease as the "Location" (as shown in the figure above) increases (this corresponds to an increased distance from the applied truck load). By observing this chart, one can conclude that the presense of horizontal bracing increases the shear lag effect and decreases the effective width. This can be seen by observing the large changes in stress intensity from the regions directly over the girders to the regions in between the girders. It should be noted however that the horizontal bracing decreases the overall magnitude of stress.

The following figure shows the stress distribution for the models analyzed with different types of bracing, and horizontal bracing present:

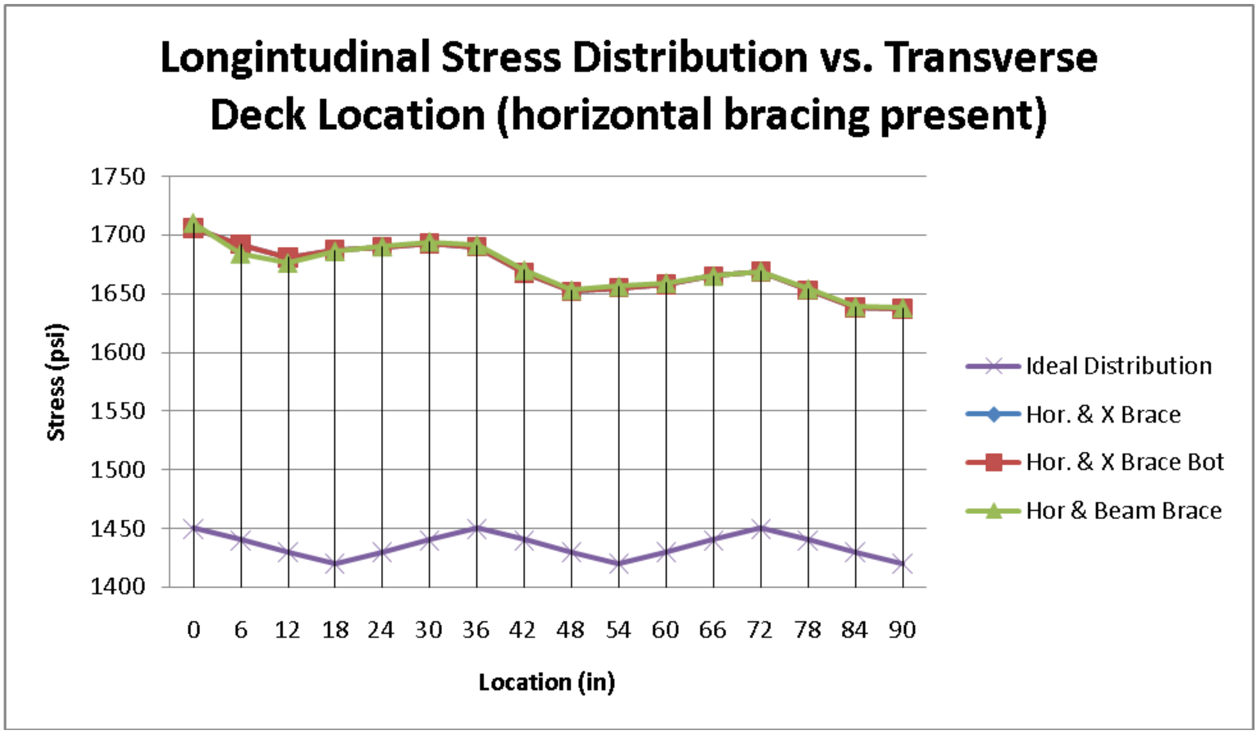


Figure 63: Longitudinal Stress Distribution vs. Transverse Deck Location (horizontal bracing present)

This chart shows that the stress distribution is nearly identical for all bracing types. This means that when used with horizontal bracing, the type of additional bracing used (if any) does not appear to influence effective width.

The following figure shows the stress distribution for the models analyzed with different types of bracing:

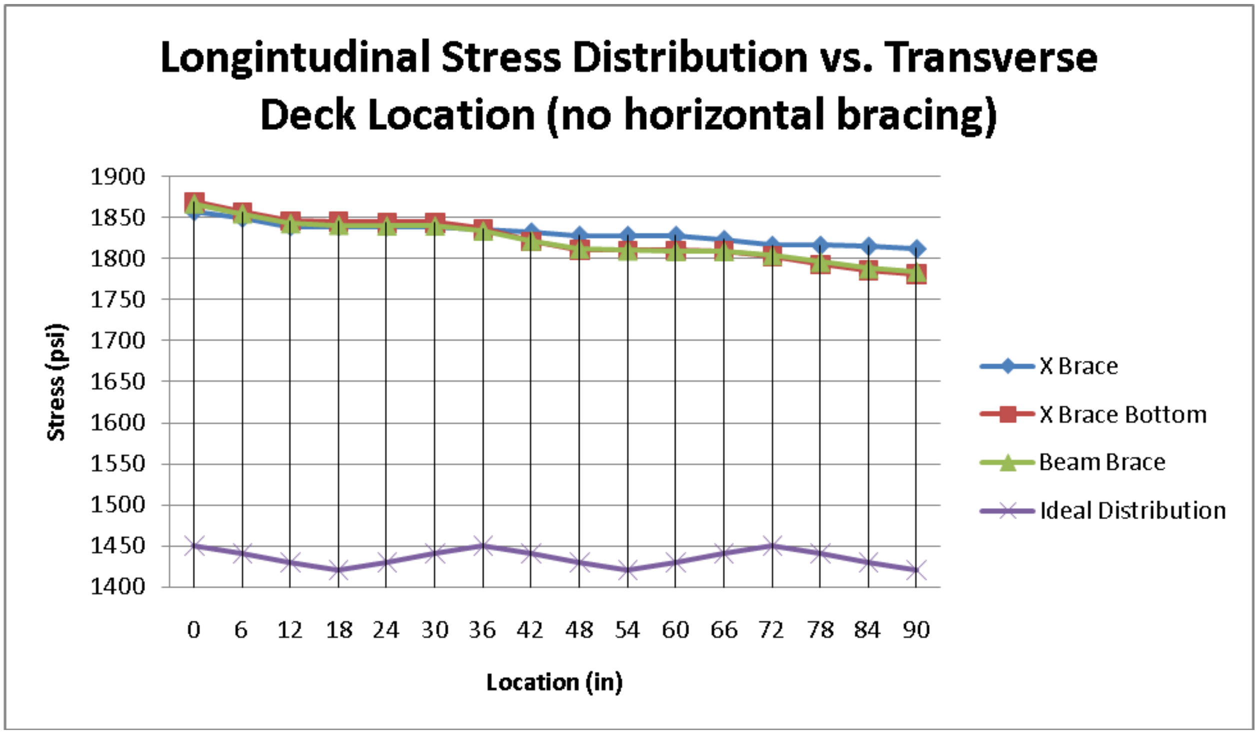


Figure 64: Longitudinal Stress Distribution vs. Transverse Deck Location (no horizontal bracing)

This figure also shows a similar stress distribution for all of the bracing types. This could lead one to conclude that bracing type does not influence effective width when no horizontal bracing is present.

To compare the effectiveness of the various bracing types, it is useful to look at the percent change in stress intensity as one moves from point to point over the region investigated. The following figures present this information. In the plots, the percent change is taken from the more exterior point to the more interior point, e.g. from zero inches to six inches.

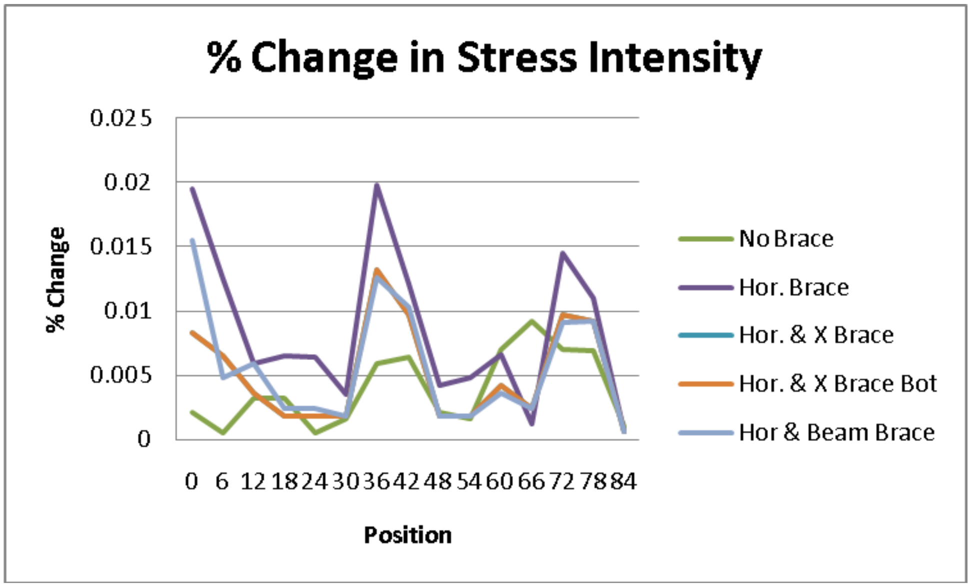


Figure 65: Percent Change in Stress Intensity (horizontal bracing present)

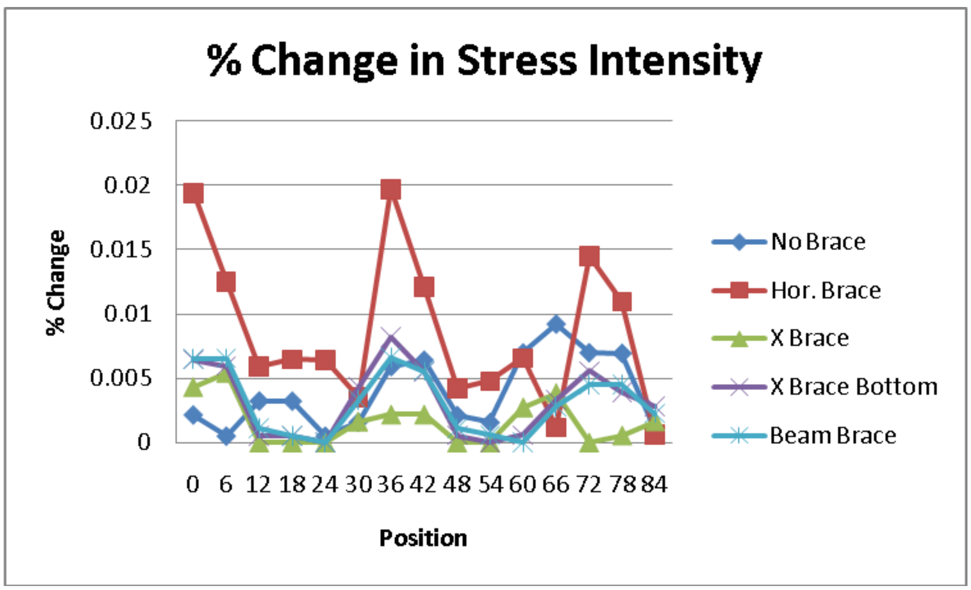


Figure 66: Percent Change in Stress Intensity (no horizontal bracing)

When reviewing these plots, it should be noted that in general, the higher the percent change in stress intensity, the smaller the effective width. If this argument can be accepted, then it can be concluded that providing horizontal bracing could decrease the effective width in composite girder bridges. This can be seen in Figure 64 by comparing the models with bracing to the model with no bracing, and observing that the model with no bracing yielded the smallest percent change in stress intensity. If no horizontal bracing is used, all bracing types appear to increase effective width compared to the model with no bracing. Of all the bracing types investigated, the X Bracing appears to be the most effective in increasing effective width.

Remarks

This chapter presented a study of how bracing affects the lateral distribution of deck loading in girder bridges. The main focuses of the study were reduction in maximum girder moment and increases in effective width. From the results, it can be concluded that using lateral bracing can significantly reduce the maximum girder moment. Conclusions about the effect of bracing on effective width are weaker because not many strong trends were observed. To improve the results used to study effective width, it may be necessary to revise the modeling process. A more detailed model may be required which does not simplify the connection between the girders and the deck.

A review of the literature can lead one to conclude that the availability of finite element software to design professionals is increasing. This increased availability of modelling environments could potentially lead professionals to develop more effeicient and cost effective designs. It is important however for designers to fully understand how the models work and the implications of their results. This is especially important when determining an appropriate MDF. The literature suggests that MDF's are dependent upon many different factors, some of which are easily controlled through individual models. Other factors, particularly loading pattern, are more difficult to model, and especially to post process.

In light of these facts, it may not be advisable for AASHTO to develop models that provide guidelines for determining MDF's which take into account all factors that can affect an MDF, as some researchers have suggested (Eamon and Nowak, 2002). This is because the model would need to be very complex. Instead of developing a general model that could be applied to any bridge, AASHTO could recommend that design engineers develop a finite element model for each individual project they are working on. The use of finite element models would provide a simple way to take into account the many factors that affect the MDF. It would be important for AASHTO to provide guidelines for determining critical load patterns. Also, guidelines for conducting statistical analyses on the MDF's calculated from several different loading patterns would be required. The statistical analyses should provide reasonable safety factors to ensure that the MDF used is not too small. The development of such guidelines could potentially lead to a more accurate depiction of the load distribution, particularly moment reduction, and lead to more efficient and cost-effective designs.

VIII Finite Element Analysis of Clip Angle Connections

Finite element modeling is a widely used tool in performing structural analysis of complex structures and components. The need for this advanced tool arises from the lack of analytical solutions for a variety of problems in fields such as civil engineering, material science, mechanical engineering, etc. Typical applications of FEM include plane stress problems, axi-symmetric problems, fluid flow, heat transfer, etc. It is in these instances that FEA is extremely effective in modeling complex phenomena, such as shear lag, friction, contact, vibration, etc.

The behavior of bolted steel connections in large scale structures such as buildings and bridges is a very complex phenomenon. The complexity of this problem is due to the fact that the component is subjected simultaneously to several different load types, including contact pressure, friction, and moment rotation. The second characteristic that contributes to the problem's complex nature is the fact that the connection is composed of several different parts; including bolts, nuts, welds, and clip angles. Third, the connection parts are often made of materials with different mechanical properties. For example, bolts usually have a higher modulus of elasticity than the other parts of the connection. Therefore, most of the load will be channeled through the bolts, and a large stress range is expected to occur in their vicinity. The presence of threads adds to the complex geometry and accounts for another major difficulty in modeling such connections. In some cases both cold and hot rolled steel may be used in the same connection. All the above mentioned factors give rise to the need for a large number of elements which, in turn, cause extremely large computation time, memory requirement, and inconsistency in results.

The latter may result from numerical errors, which become a significant factor for detailed modeling of contact problems.

It is clear that a FEA model that realistically represents a bolted steel connection would be extremely difficult to construct and analyze. Therefore, simplifications are used to study some of the basic phenomena involved. Such simplifications consist of reducing the object to a two dimensional model, reducing the number of contact pairs, setting the constraints in an appropriate manner, and reducing the number of elements in the areas of the mesh that are not of interest in the analysis. First, the engineer needs to establish the phenomenon of interest and identify what elements are sufficient for its study. Based on this approach, the parts comprising the model can be adequately simplified or omitted.

The following study is a comparison of simplified methods for 3D modeling of single angle steel connections. The study is focused specifically on methods for adequately modeling the stress distribution throughout the leg of a steel angle connecting a floor beam and a stringer within a highway overpass. The design results from Option 3, which are outlined in Chapter IV, were used for sizing the structural components of the model.

Methodology

A steel clip angle with four bolts was designed according to AISC standard procedures. Three limit states including bolt bearing on angle, shear rupture of the angle, and shear yield on the angle were investigated. The clip angle, stringer, and floor beam were designed for a yield strength of 36 ksi; 48 ksi was used for the bolts. The geometry of the model was based on the design results for Option 3. The following table summarizes the main design parameters relevant to this study. Please refer to the Appendix for more details on the design of all the structural components included in this model.

Table 36: Summary of Relevant Design Parameters

Angle thickness	0.65 in
Angle Height	6 in
Angle width	5 in
Yield strength of angle	36 ksi
Yield Strength of bolt	48 ksi
Yield strength of floor beam/stringer	36 ksi
Floor Beam size	W12 x 22
Floor Beam length	15 ft
Stringer size	W36 x 800
Length of modeled stringer section	6 ft
Bolt type	A325-N
Bolt nominal diameter	0.875 in

A large scale, detailed, three-dimensional model was constructed in AutoCAD, and the geometry was exported to ANSYS by using a .SAT file. The geometry included a floor beam, a 6 ft long stringer section, a clip angle, and four bolts. A description of the position of these components on the rest of the bridge structure is necessary. The floor beam runs perpendicular to the direction of traffic and is located between the stringers, which are supported by the pier and the abutment on each end respectively. The floor beam transfers the dead load and traffic load to the stringer through the shear connection. The stringer then transfers the load to the pier and the abutment.

The material properties of all components were set to be linear, elastic, and isotropic with Poisson's Ratio and Young's Modulus of 0.3 and 29000 ksi respectively. The model consisted of five volumes of elements representing the floor beam, stringer, clip angle, and two bolts. Each volume was meshed separately with brick elements.

There are two mesh types: a *free* mesh and a *mapped* mesh, which are automatic capabilities embedded in ANSYS. There are no restrictions on a free mesh in terms of element shapes and no specified pattern applied to it. In the case of a mapped mesh, the element shape it contains and the pattern of the mesh are restricted. A mapped volume mesh contains only hexahedron elements, while a mapped area mesh contains either only triangular or only quadrilateral elements. A mapped mesh typically has a regular pattern, with obvious rows of elements. This type of mesh requires the geometry to be a series of fairly regular volumes and/or areas that can accept a mapped mesh. The free mesh option was used in meshing the model since neither a sweep or mapped mesh could be achieved due to the relatively complex geometry involved. An irregular mesh was obtained, which required further refinement and element size modifications. These adjustments were made in order to better model the contact between the bolts and the angle and to obtain a more detailed stress picture near the bolts and the edges of the angle. The free mesh resulted in excessively large elements in the web of the floor beam. The load scheme used in the FEM was similar to the one employed for the design of Option 3. It involved two 32 kip point loads spaced 6 ft apart and a distributed dead load of 0.009 k/ft^2. All loads were applied on the floor beam. Variations were made from model to model. The following figures show the finite element model for the entire mesh and for the connection details.

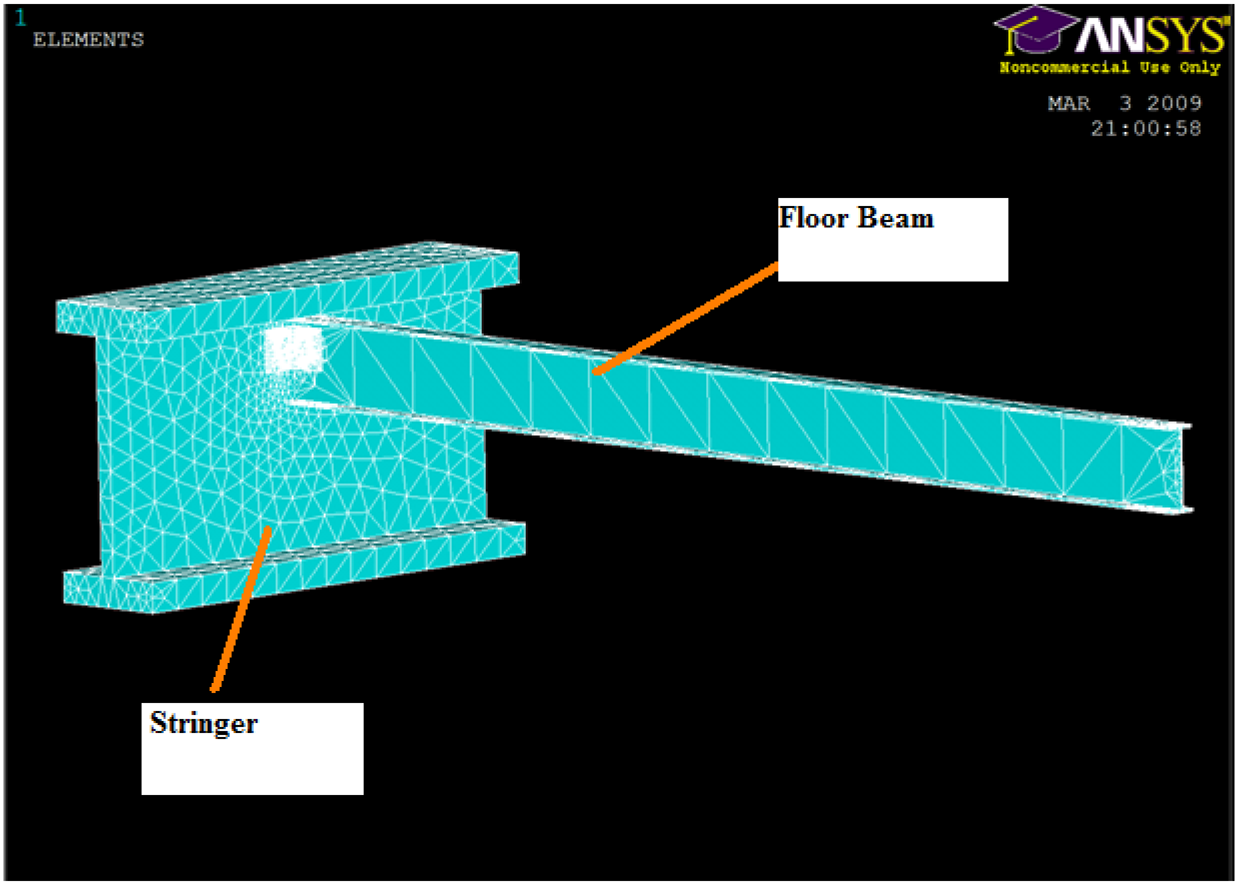


Figure 67: Finite Element Model for the Entire Mesh

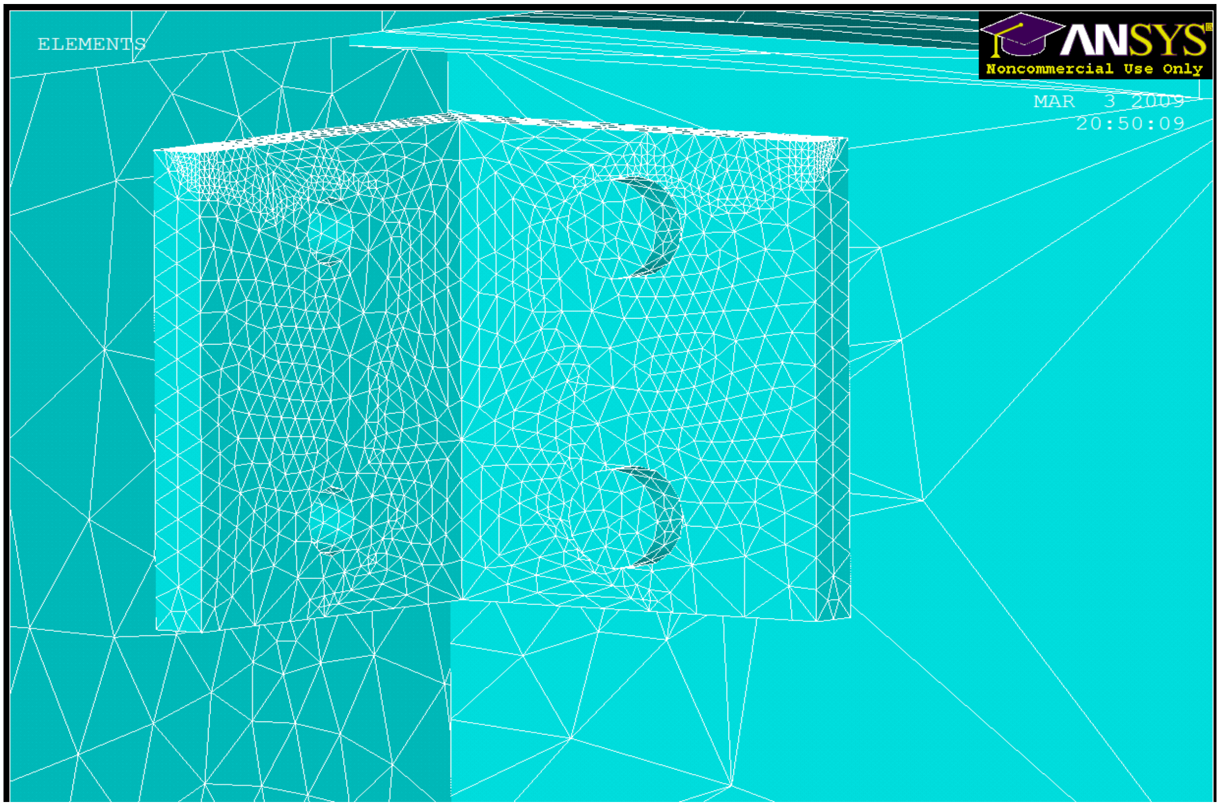


Figure 68: Finite Element Model for the Connection Details

Several simplifications were made to the model based on the objective of the study. First, the contact between the bolt heads and the web and angle legs was not taken into account in order to decrease computational time as much as possible. Three different methods were investigated in modeling the connection bolts.

1. In order to account for the additional friction between the bolt and the contact surfaces between threads, the bolts were made 0.02 inch diametrically larger than the holes. All degrees of freedom were constrained for all nodes on the area of the angle leg on the stringer. The two bolts connecting the web of the stringer to the clip angle were not meshed and no contact pair was created for them. The effect of the stringer bolts on the connection was irrelevant at this point, since focus was placed on the angle leg on the floor beam, which carried the shear load from the

beam to the stringer.

2. The second method involved modeling the bolt with a diameter equal to the diameter of the hole. This method was used in order to eliminate the contact pressure effects resulting from the incompatible geometry between the bolts and the bolt holes.
3. The third method considered all four bolts with diameters 0.02 inches larger than the hole. The constraints on the angle were removed and contact pairs were created for the two bolts connecting the clip angle to the stringer. This method was used to obtain the stress distribution on both angle legs.

Two contact pairs were created corresponding to each bolt. The modeling of the bolts and contact pair was based on a contact tutorial included in the ANSYS software package as well as recommendations from Adriana Hera (ANSYS expert at Worcester Polytechnic Institute) and guidelines on Methods for Modeling Bolts in the Bolted Joint by Jerome Montgomery. A coefficient of friction of 0.2, corresponding to friction between two steel plates, was specified as one of the parameters of the contact pair. The rest of the parameters specified in the ANSYS 'contact wizard' were entered according to the Contact Tutorial. The following table provides a summary of the most important parameters used for the mesh analysis and contact pair.

Table 37: Analysis and Contact Parameters

Analysis Parameters	Element type	Brick elements
	Analysis options	Large static displacement
	Number of substeps	10
	Max. no. of substeps	20
	Min. no. of substeps	10
	Time at end of loadstep	1
	Mesh type	Free mesh
	Max. no. of iterations	Program Chosen
Contact Pair Parameters	Stiffness matrix	Unsymmetric
	Penetration tolerance	0.1
	Coefficient of friction	0.2
	Normal Penalty Stiffeness	0.1

The bottom flange of the stringer, the free end of the floor beam, and the area of the angle leg on the stringer were fixed. Part of the load applied to the top flange of the floor beam travels through the fixed end of the floor beam. The rest of the applied load is transferred through the connection, to the fixed area of the angle. All degrees of freedom were constrained for all the nodes located on these areas. Such an approach was taken in order to decrease the effect of rotation of the floor beam with respect to its longitudinal and transverse axis as much as possible, while still being able to obtain the effect of the deformation of the floor beam on the connection.

A paper on finite element modeling of bridge deck connection details was reviewed as a benchmark for this study. The paper was titled "Finite-Element Modeling of Bridge Deck Connection Details", by DePiero, Anthony H., Robert K. Paasch, and Steven C. Lovejoy. It is a study on FEA modeling of bridge deck connections subjected to fatigue loading. The results from this study were compared with the results obtained from the FEA model and the clip angle design.

Results

The following table provides a summary of the design results and shear capacity for each limit state. The leftmost column identifies the limit state specified by AISC, the center column shows the required angle thickness to satisfy the strength requirements of the given limit state, and the rightmost column shows the capacity of the angle for the given limit state (based on the thickness called out in the center column).

Table 38: Clip Angle Thickness Capacity

Limit State	Angle Thickness (t)	Capacity
Bolt Bearing on Angle	0.55 in	32.0 kips
Shear Rupture of the Angle	0.19 in	31.5 kips
Shear Yield on Angle	0.24 in	31.1 kips

After the FEA solution for the first model was obtained, the principal stresses were plotted as shown in the figure below. It was observed that the stress in both bolts exceeded 80 ksi throughout the contact surface with the angle and web of the floor beam.

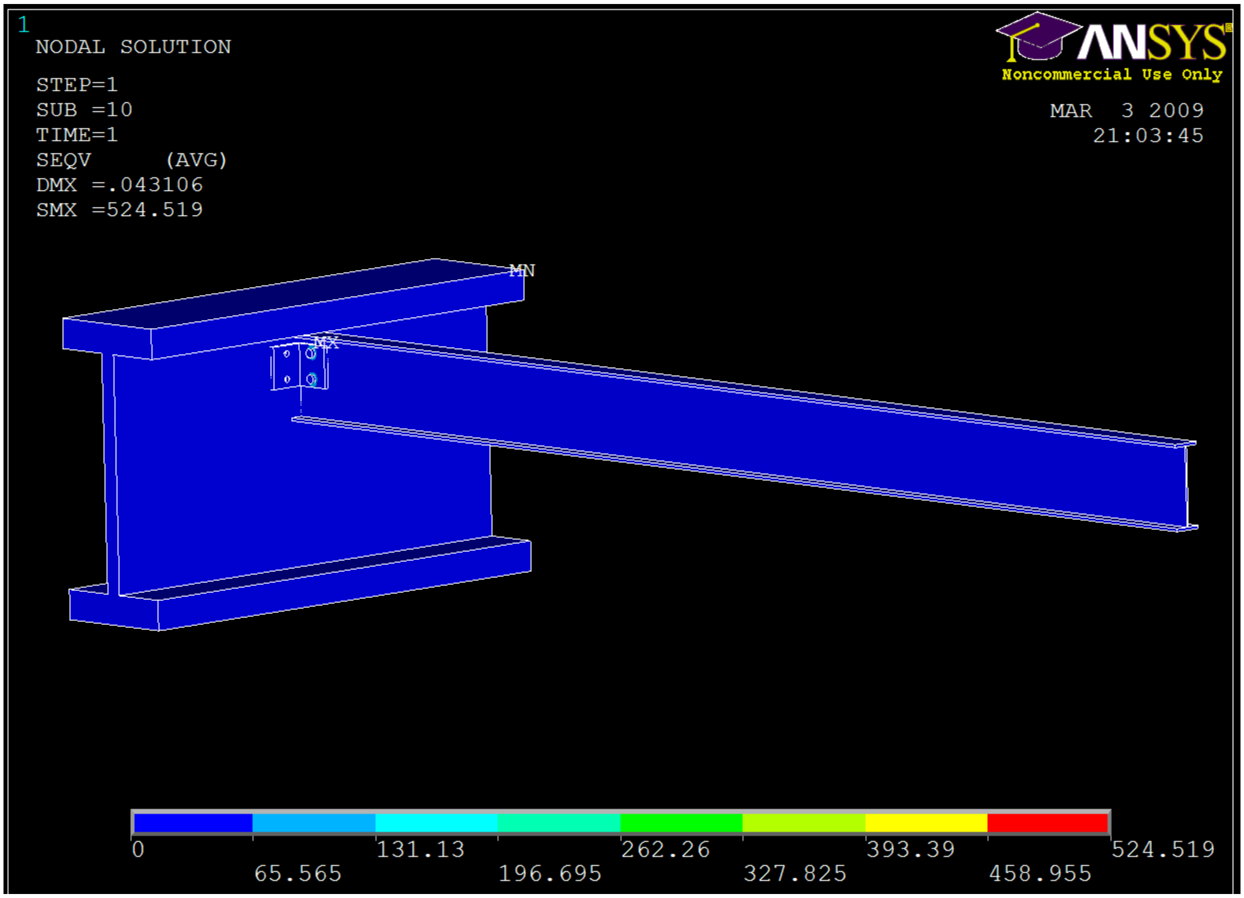


Figure 69: Model 1; Stress Distribution in Entire Model

The following figure shows that the stress level in the contact area on the angle exceeds 200 ksi. The principal stresses exceed 50 ksi in a region extending up to 0.75 inches from the edge of the bolt hole. These values exceed the design capacity of all the structural components. The excessively large stress is mainly due to the fact that the bolt is larger than the hole causing the bolt to push on the edges of the hole and distort the area around it.

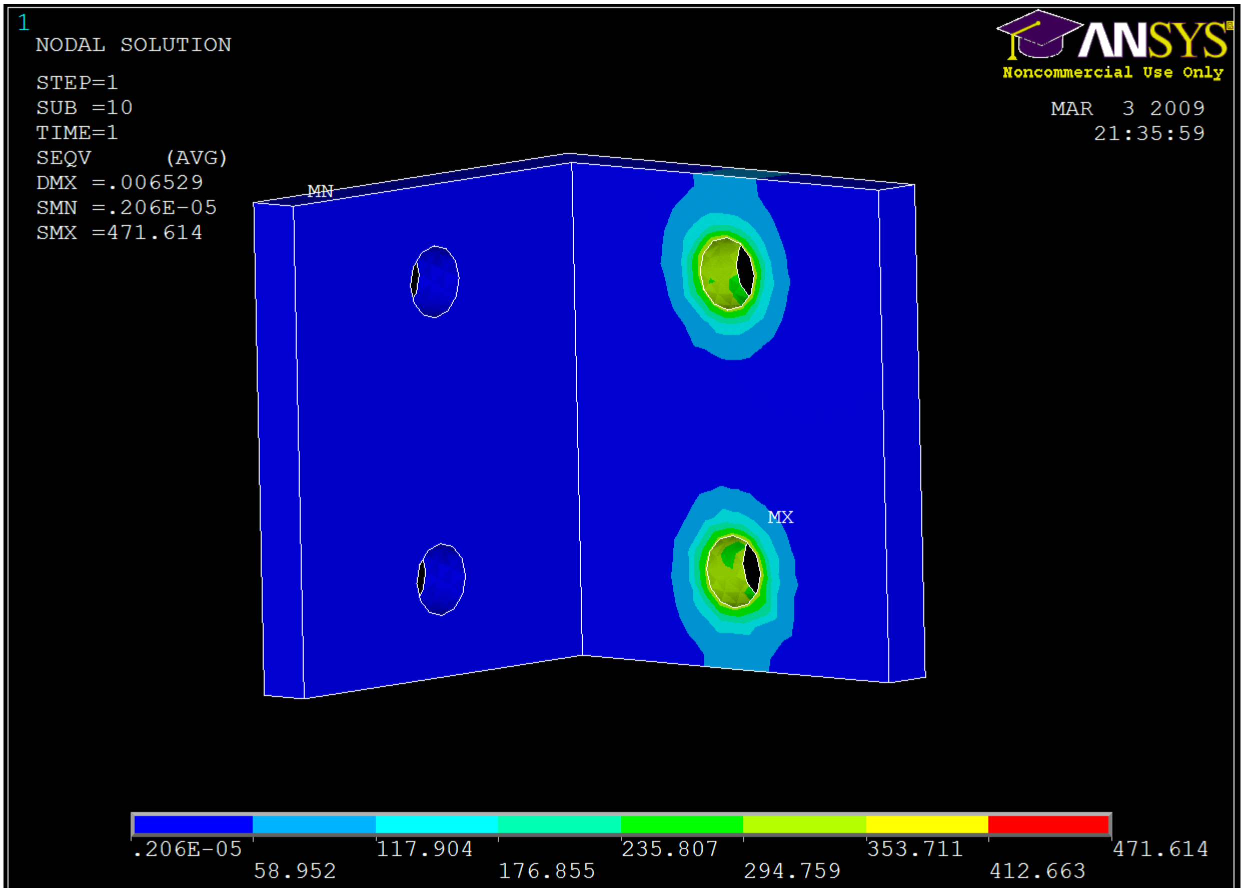


Figure 70: Model 1; Stress Distribution on Clip Angle (Regular Plot Scale)

The plot scale was modified to show only the stresses smaller than 30 ksi. After this modification, a clearer picture of the stress distribution near the edges of the clip angle was obtained. Stress levels reached values above 19 ksi in two areas: around the bolt hole and in the location circled on the following figure.

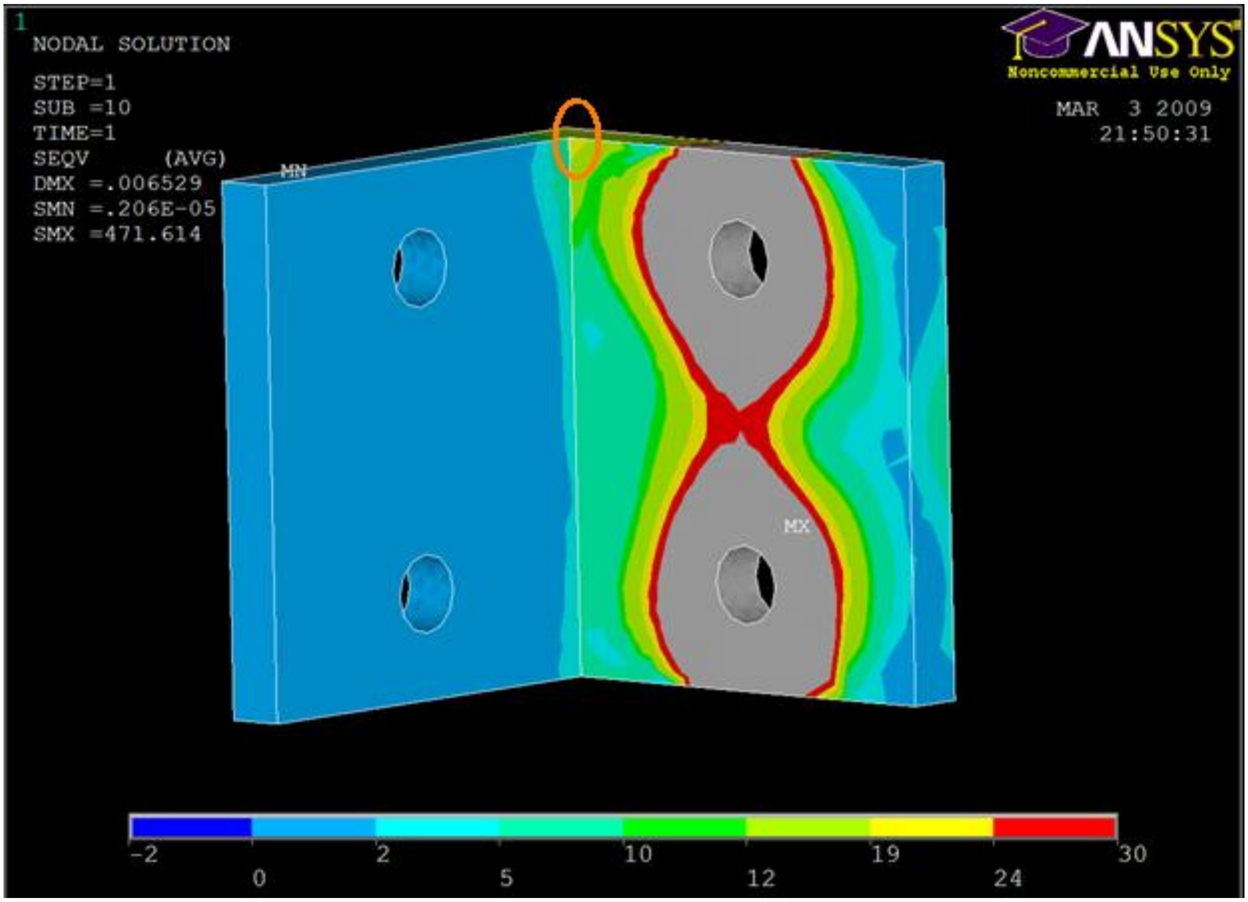


Figure 71: Model 1; Stress Distribution on Clip Angle (modified plot scale)

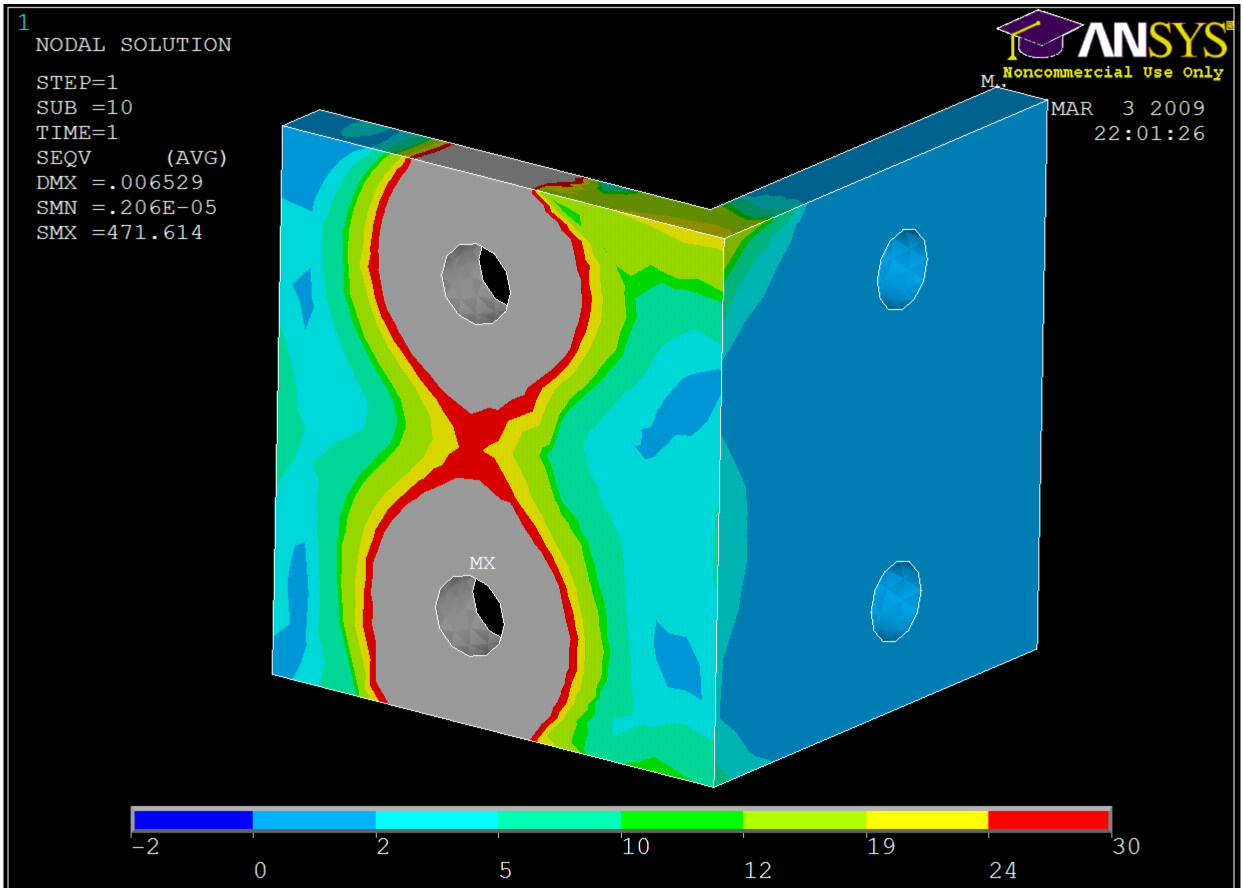


Figure 72: Model 1; Clip Angle's Surface Attached to the Web of the Floor Beam

A look at the stress distribution on the surface of the bolts shows that the stresses exceed 80 ksi throughout the contact surfaces. The stress appeared to be unevenly distributed, which is likely due to element penetration from the angle to the bolt. See Appendix J for figures displaying stress distribution in bolts.

For the second model the stress levels in the majority of the angle's volume did not exceed 36 ksi. The maximum stress occurred in the vicinity of the bolt. The four regions where the stress reached high values relative to threshold are indicated in the following figure.

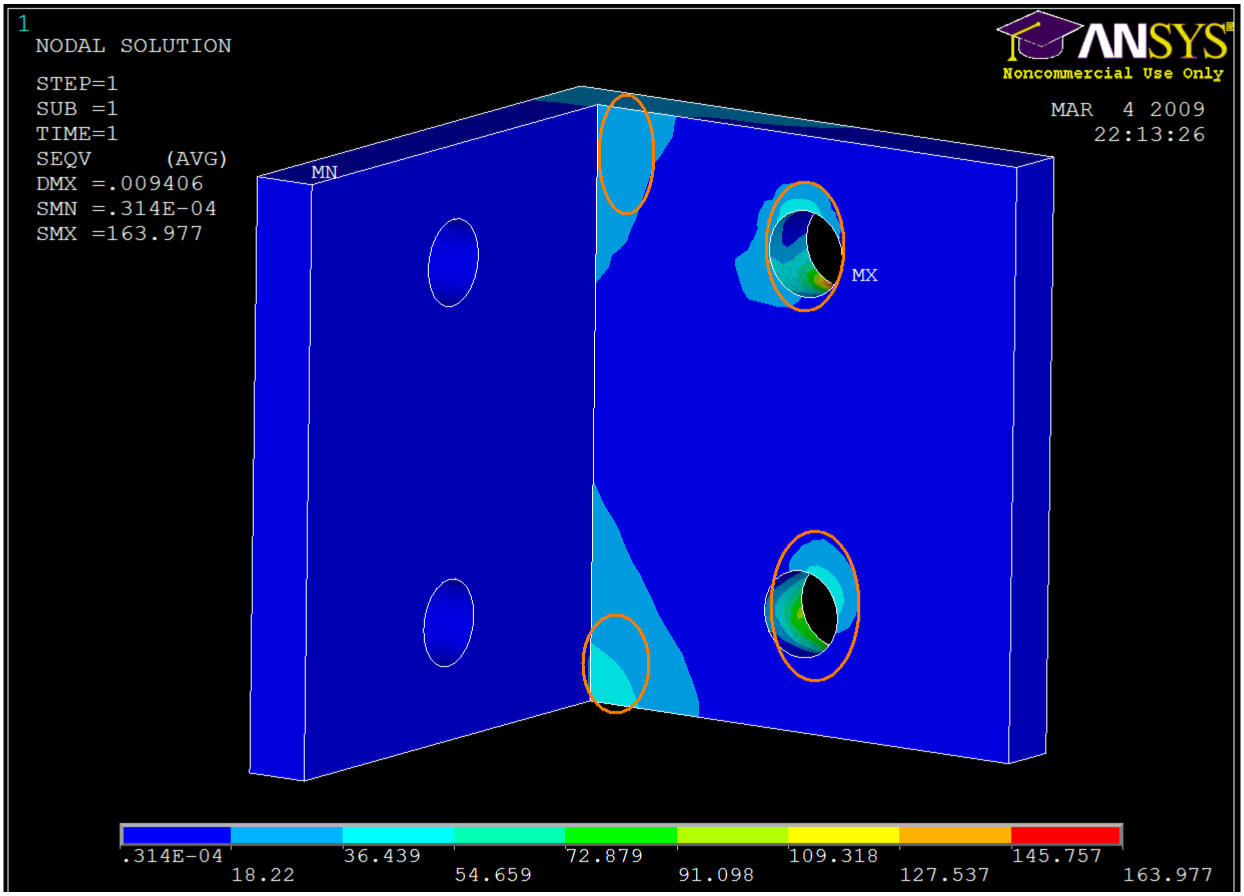


Figure 73: Model 2; Stress Distribution on Clip Angle (regular plot scale)

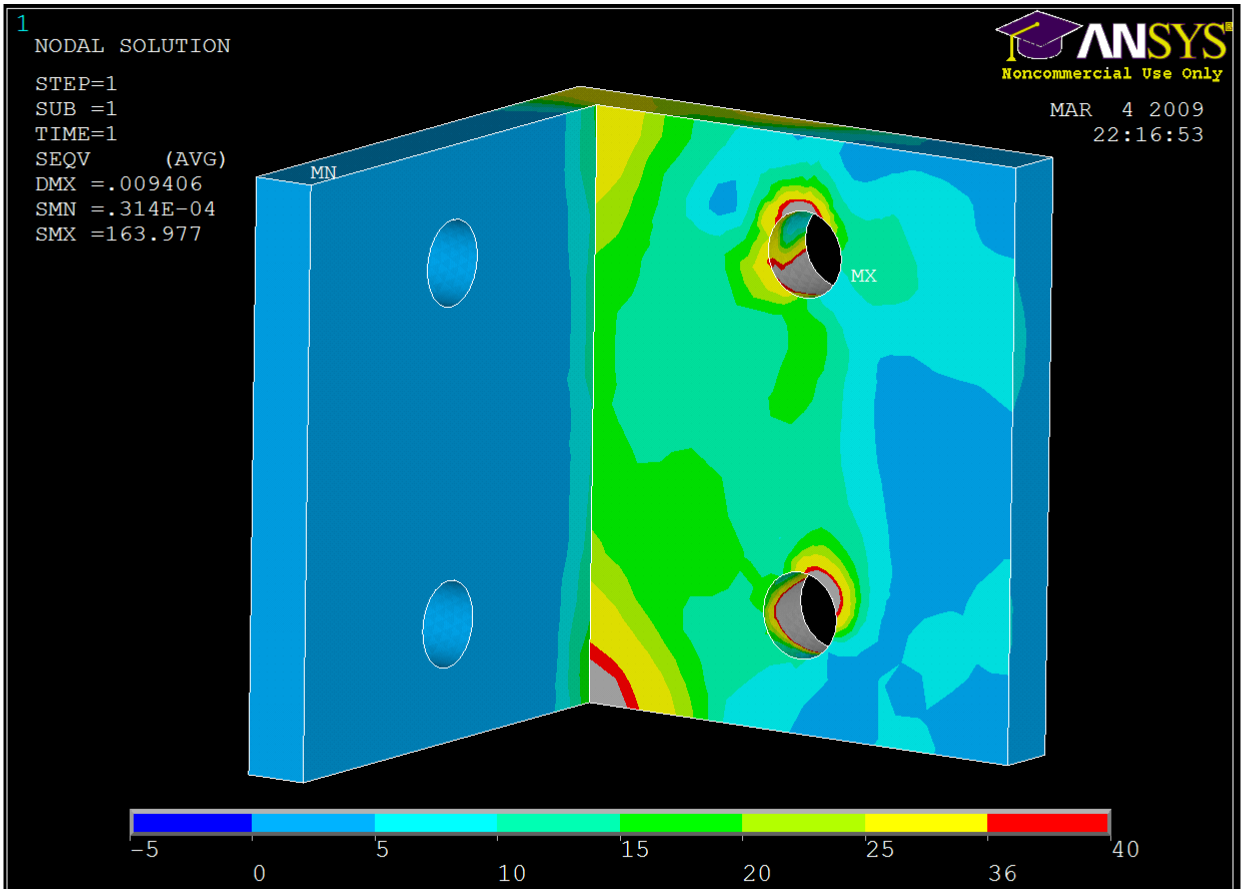


Figure 74: Model 2; Stress Distribution on Clip Angle (modified plot scale)

The stress range is different from the results of the first model. The stress level is approximately 10 kips higher. This effect is due to the larger load applied on the floor beam. However, the stress near the bottom corner exceeds 40 ksi. The excessive stress at this location may be due to rotation of the floor beam with respect to its longitudinal axis, lateral displacements of the bolts (which are due to deformation), and displacement of the floor beam.

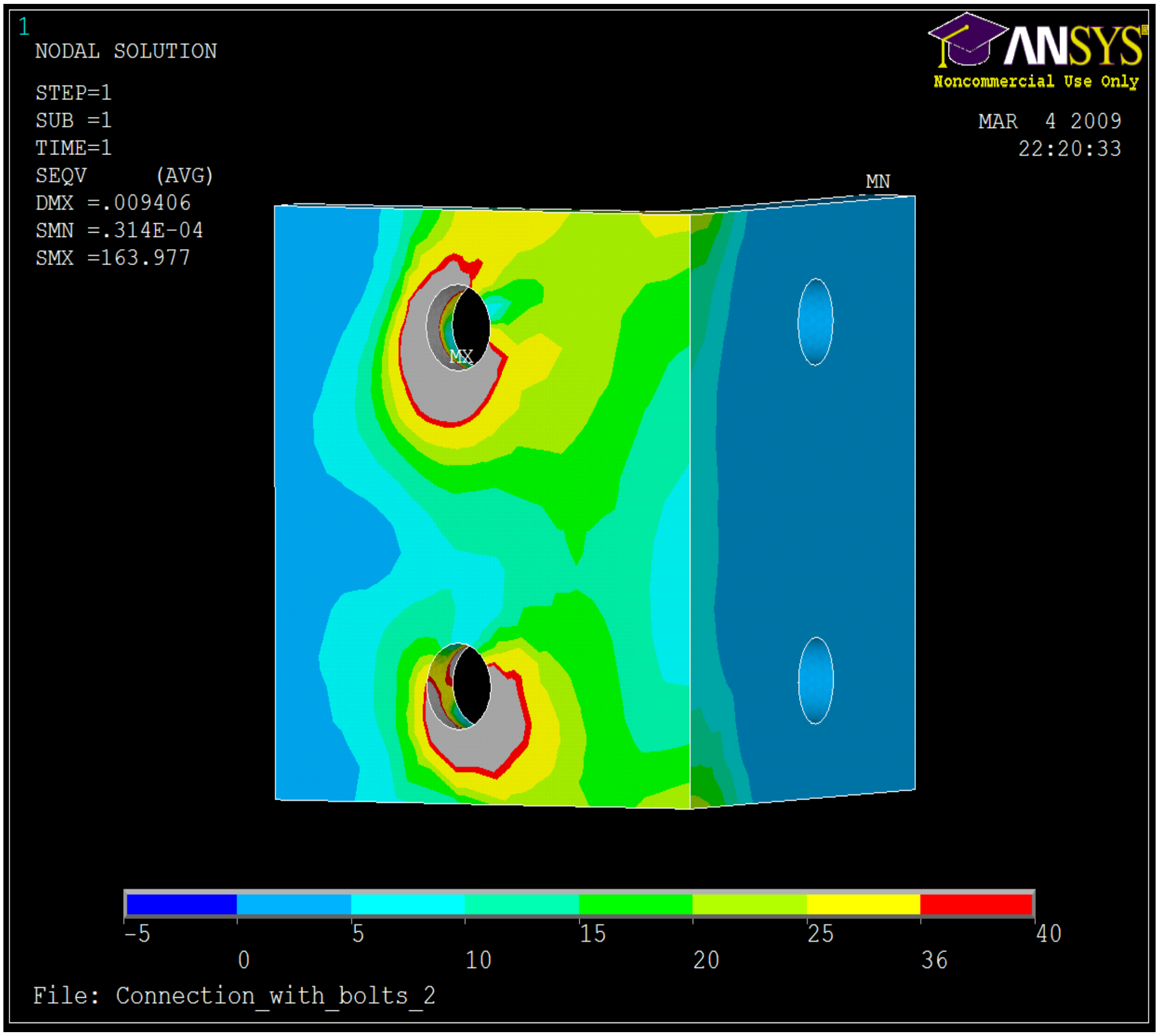


Figure 75: Model 2; Clip Angle's Surface Attached to the Web of the Floor Beam

The third model exhibited a peak stress of 517.684 ksi, the largest of all the models. This value, however, occurred at the fixed support of the floor beam and most probably occurred due to singularities in the mesh.

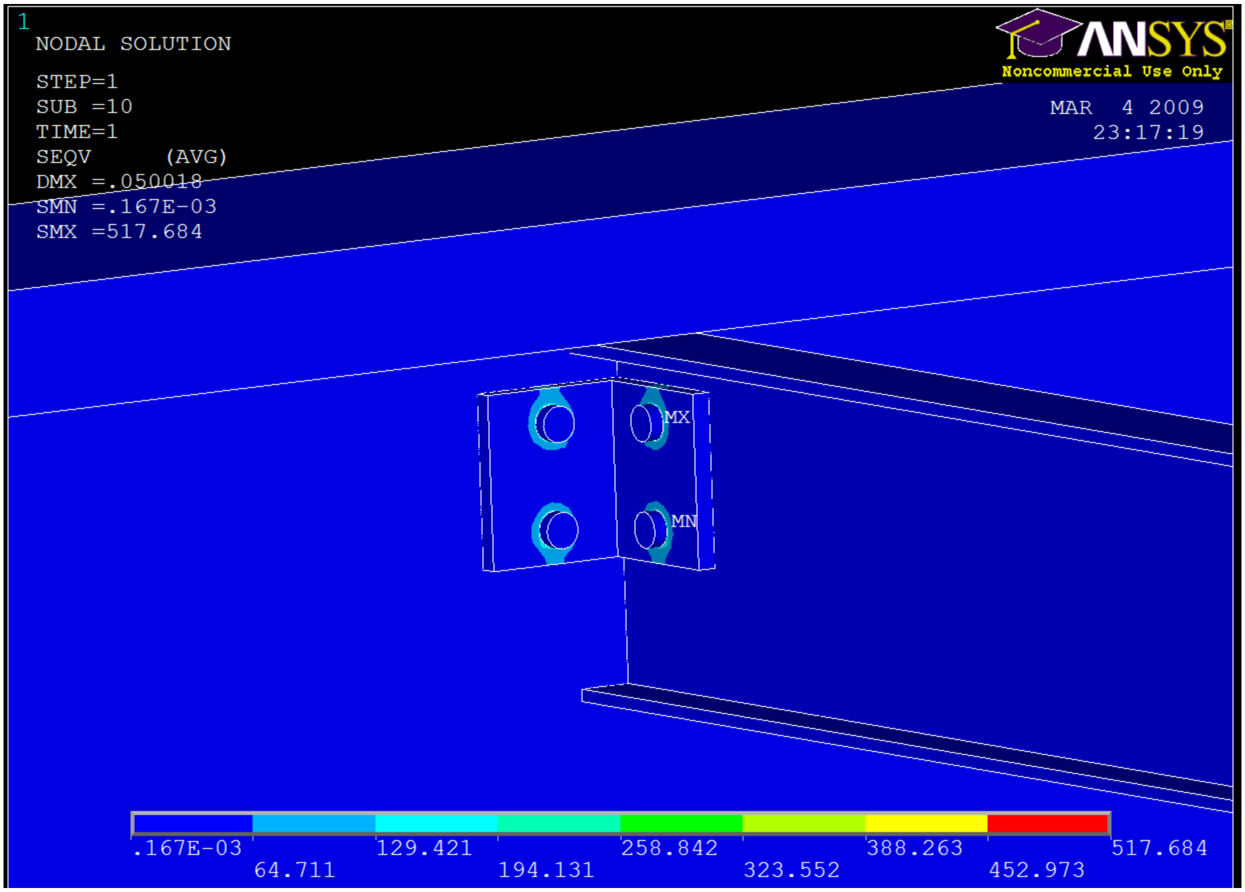


Figure 76: Model 3

Results showed that the maximum stress in the clip angle occurred in the vicinity of the bolt.

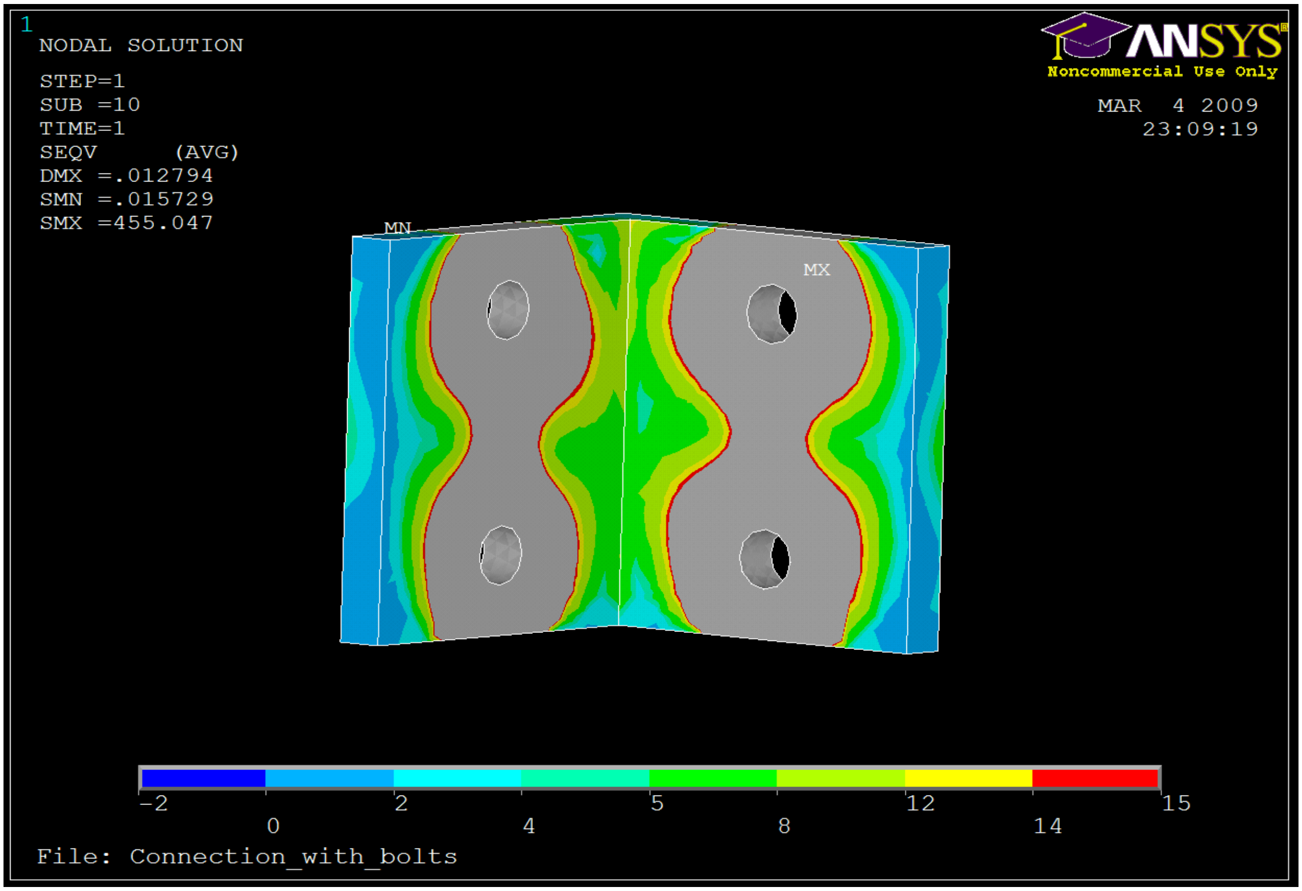


Figure 77: Model 3; Stress Distribution on Clip Angle (modified plot scale)

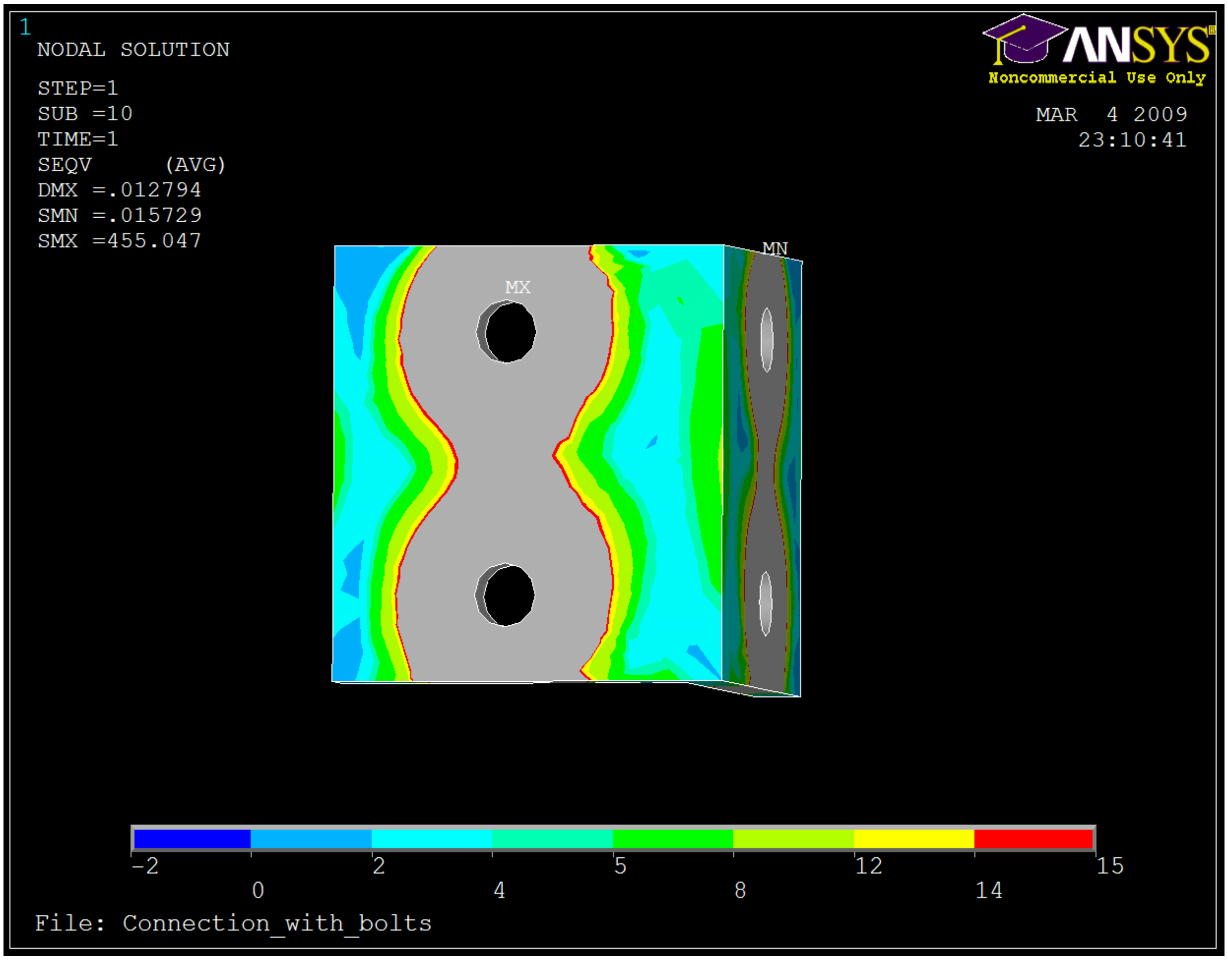


Figure 78: Model 3; Clip Angle's Surface Attached to the Web of the Floor Beam

While running all three analyses, ANSYS reported excessive *initial penetration* in the contact surfaces. Initial penetration refers to the penetration value ANSYS discovers when inspecting the mesh before the first iteration. Penetration between elements happens when the mesh on the contact surfaces is not fine enough. This becomes a major problem for geometries involving arched surfaces in contact. The finer the mesh around the surface of the bolt and bolt hole, the smoother the surface. A coarse mesh with large element sizes will cause penetration between the elements on the surface of the bolt and the elements on the surface of the bolt hole.

A check of the geometry revealed a few inconsistencies with the data from AutoCAD. A check of the coordinates of the *keypoints* (base coordinate points of the geometry) revealed possible misalignments of the bolts. Errors during data transfer may have also caused misalignment between the bolts and the holes. The contact surfaces were refined several times with no success in removing the high stress values. Refinement was done up to the point where the resulting mesh exceeded the allowed number of elements.

The design results displayed in Table 38 showed that bolt bearing governed in determining the thickness of the clip angle legs. The article by DePiero showed that the maximum stress occurred in the indicated regions below as well as at the base of the clip angle where it is attached to the floor beam.

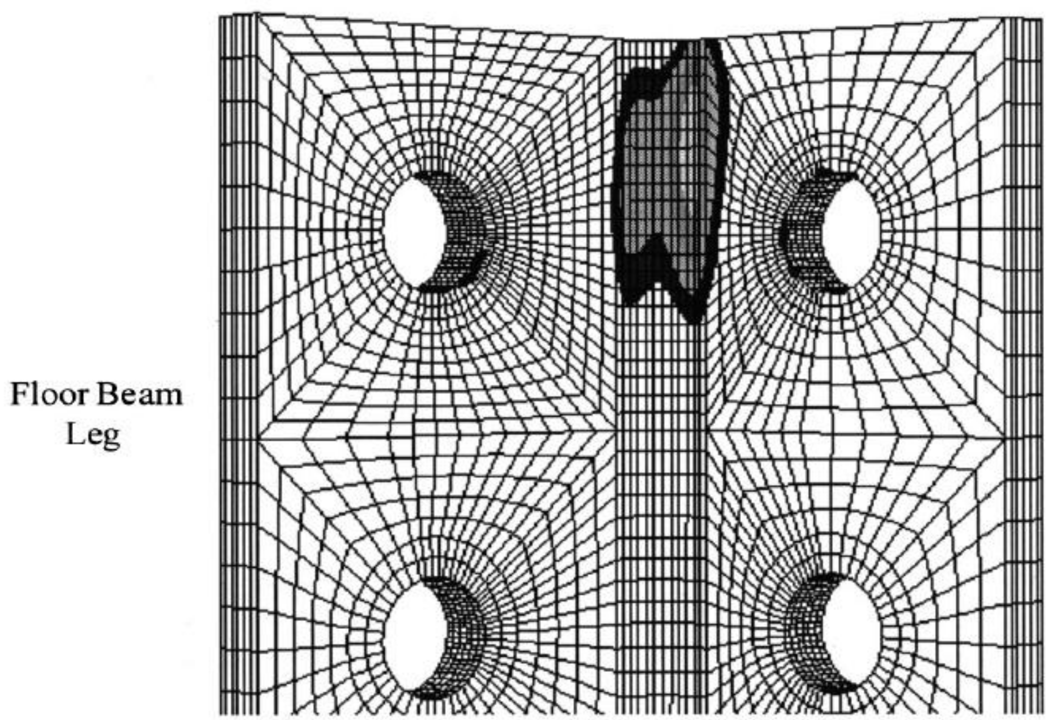


Figure 79: Distribution of Principal Stress from Analysis using Fixed Rotation Model of Floor Beam

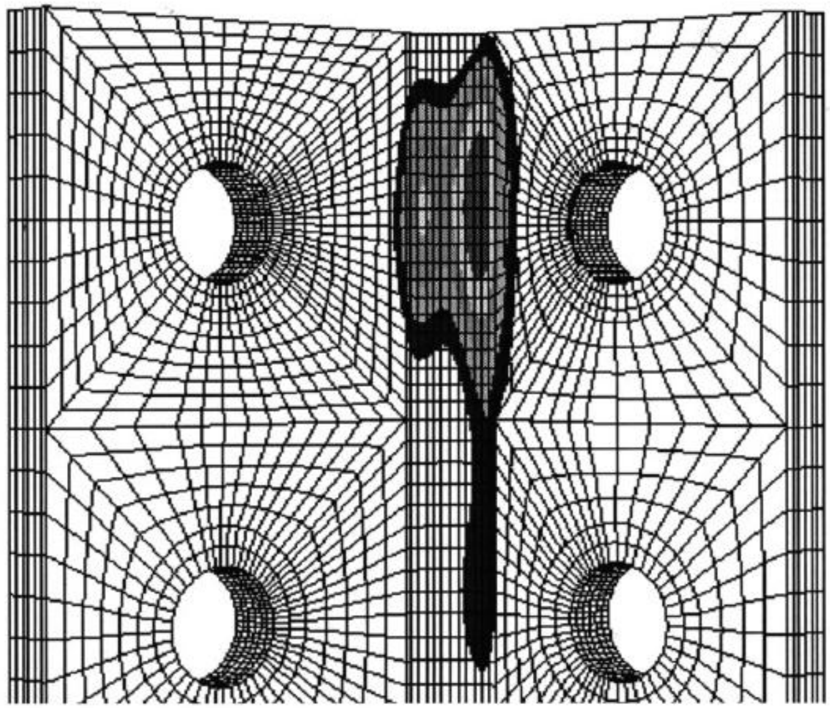


Figure 80: Distribution of Principal Stress from Analysis using Fixed Top Flange Model of Floor Beam

Conclusions

This study is not intended to provide an accurate model of the realistic behavior of a shear connection. It is a comparative approach to simple 3D modeling techniques for large geometries, aimed at developing an understanding of the technical issues involved in modeling problems exhibiting non-linear behavior of this type and scale.

Three FEA models were constructed in this study. The data were compared to results obtained from the clip angle design and to a relevant paper on finite element modeling of steel connections. Three methods were used to model the bolted connection. In the first and second method only the two bolts connecting the clip angle to the floor beam were modeled. Additionally, in the first approach the bolts were modeled with a diameter slightly greater than the bolt holes. In the second approach the diameter of the bolts was equal to that of the bolt holes. In this case, the interaction between the threads was not accounted for. The third method for modeling the connection included all four bolts.

The FEA results showed that modeling the bolts with the same diameter as the bolt holes provides a better picture of the stress distribution in the clip angle in comparison to the other two methods. The model with equal diameter bolts and holes displayed areas of concentrated stress away from the bolts, as in DePiero's results. These stress concentrations were not as visible in the first and second model. However, the bolts' contact with the inner part of the hole was not adequately represented. On the other hand, modeling the bolt somewhat larger than the hole, greatly overestimated the contact force. The latter approach considerably distorted the stress distribution on the clip angle and web.

The accuracy of the second method of modeling the clip angle connection can be improved by refining the mesh near the edges of the holes and clip angle. In addition, positioning the point loads on the center line of the floor beam will eliminate any eccentricities due to the loads and may change the stress picture in the clip angle.

Decreasing the diameter of the hole may improve the results of the second and third model, because the high stresses due to geometric incompatibility between bolts and holes are not present. However, applying pre-tension on the bolt may yield better results since it would provide a more realistic way of modeling the resisting force against slippage.

Overall three methods proved inadequate for modeling bolts. It should be noted, however, that the second method may turn out to be a relatively adequate approach if the errors due to file transferring are eliminated. A different file format should be used to export the data from AutoCAD. A better approach would be to create the model in ANSYS instead of importing the geometry from another software. This, however, becomes impractical for large and complex geometries.

The presence of the contact pair made the analysis a non-linear one. Specifying a contact pair enables ANSYS to account for the frictional forces between the bolt and the angle. Contact problems require more elements and more computation time. The solution may not converge unless the contact surfaces and elements are defined adequately. Further mesh refinement in the contact surfaces may be needed in order to avoid penetration between elements. The maximum number of sub-steps and penetration tolerances are major factors upon which the success of the analysis depends. ANSYS divides the applied load in several parts and applies it step by step. At each step the solution is calculated. If the number of sub-steps is too small the solution will not converge. On the other hand, due to the increased number of iterations, the analysis may considerably increase computation time if the number of sub-steps is too large.

Using a mapped or sweep mesh instead of free meshing will improve the quality of the mesh. In turn, the accuracy of the results will also improve. A free mesh generates a random and relatively irregular mesh. Element sizes and refinement levels should be manually established in order to avoid excessive initial penetration in the contact pairs. If excessive penetration between elements occurs the solution may be less likely to converge.

The data obtained from the finite element analysis showed that the peak stress occurred in the vicinity of the bolts. The design results showed that bolt bearing was the governing limit state. Therefore, the two results are consistent. On the other hand, the results from DePiero's paper showed that the peak stress occurred in two locations: near the middle of the angle and at the base of the clip angle where it is attached to the floor beam. The FEA showed that peak stress values in the vicinity of the bolt hole did occur near the base of the clip angle where it is attached to the floor beam. This peak stress is dismissed in DePiero’s paper, because their model was simplified at that point.

IX Conclusions

This project studied several of the components that comprise a typical highway bridge. These components include superstructure elements (deck, girders, connections, and bracing) and substructure elements (bearings, piers, abutments, and foundations). Design, cost, and constructability studies were conducted for the various bridge components investigated. This section will summarize what was learned from the study of each bridge component. The summary looks at the function of each component in resisting the standard AASHTO design truck load on the bridge deck. The following figure presents the load path, and the subsequent paragraphs detail the function of each component.

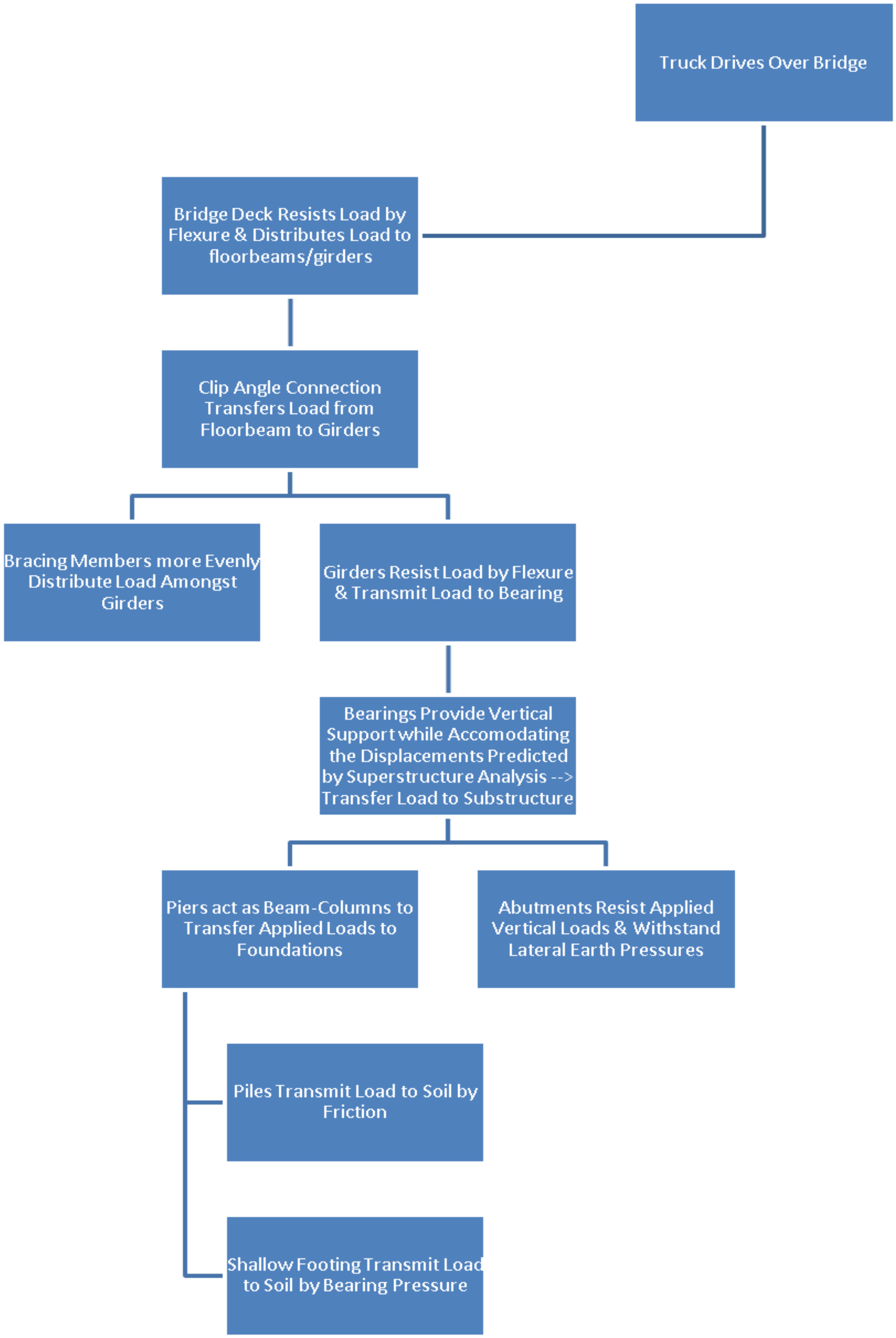


Figure 81: Bridge Load Path Summary

Bridge Deck

The bridge deck is in direct contact with the applied truck load. The design of the deck is most directly impacted by the girder spacing, which essentially dictates the magnitude of the moment the deck is required to resist. Large girder spacings require that the deck slab be thicker and more heavily reinforced than smaller girder spacings. Also, if there is an overhang (part of the deck is cantilevered and not supported at its end) then a large negative moment can develop over the exterior girder, requiring very thick and heavily reinforced sections. To avoid the need for such sections, alternative designs can be established. These include treating the overhanging part of the deck as an exterior girder, bending in the longitudinal direction (the deck typically bends in the transverse direction, perpendicular to the girders), or by applying some sort of bracing to support the deck at its free end. It should be noted however that these alternative designs have potentially serious consequences associated with them, specifically the high likelihood of serious cracking developing over the exterior girder. If these alternatives were proposed to a client in the design of a real bridge, it would be important for the engineer to explain the potential problems associated with the design; knowingly providing a client with a design that could cause serious problems would be highly unethical.

Bracing Members

Bracing members are generally provided to limit the lateral deflection that would be caused by wind loads. Research however indicates that bracing members can help to distribute the vertical loads applied to the deck across the bridge. This load distribution helps to decrease the maximum moment in the girders and allows for smaller, less expensive sections to be specified. Research also indicates that bracing members may be able to reduce the shear lag effect, and allow for a larger effective width in the design of composite girder-and-slab sections. AASHTO does not currently recognize the load distribution effects provided by bracing members; this may be due in part because no comprehensive and conclusive study of these phenomena has been conducted to date.

Connections

When the superstructure has a layout similar to the one used in Option 3 of this report (Chapter IV), shear connections are used to transfer the load from the floor beams (transverse members) to the girders (longitudinal members). The load is transferred through angle connections. The shear from the floor beam is transferred through bolts from the web of the floor beam into the steel angle, which then transfers it to the bolts connecting the angle to the girder. The bolts carry the shear load from the angle to the web of the girder. The bolt configuration also provides some degree of support against twisting of the floor beam.

Girders

The girders are loaded either directly from the deck, or are loaded through a shear connection as described in the previous paragraph. Girders are usually designed to act compositely with the deck; this composite action is achieved by placing shear studs along the top flange of the girder which provide shear resistance at the girder/deck interface. The design of bridge girders is governed by the applied loads; since many of the AASHTO specified live loads are applied in the same manner and magnitude for most bridges, the magnitude of a design load for a particular project is generally based on the dead load from the bridge deck. It is therefore reasonable to conclude that girder sizes are dictated by girder spacing.

The results of the cost analyses conducted in this report show that a girder spacing of five to six feet is most cost effective. Although smaller sections can be used when the girder spacing is less than five to six feet, more girders are required, increasing the cost. Additionally, when large spacings are used, the required section sizes make the design less cost effective. Although girders could be designed using either steel or reinforced concrete, this report has shown that the size of the reinforced concrete sections required to resist the applied loads would not be constructible. It is therefore recommended that for spans similar to the one studied in this project (81 feet), reinforced concrete sections should not be used.

Bearings

The load is transferred from the girders to the bearing at the piers and abutments. The bearing’s main role is to transfer the forces from the girders to the supporting piers and abutments and to accommodate deflections and rotations in the longitudinal and transverse directions. The accommodation of such displacements avoids the buildup of excessive local stresses in the stringers. The displacements are mainly due to dead and live loads, as well as thermal contractions and expansions. Depending on the design of the bearing, the load can be transferred from the girder simply through friction or through a steel plate bolted to the girder’s bottom flange. If an elastomeric bearing is used, the elastomeric pad deforms depending on the direction of movement or rotation of the stringer. The load is then conveyed to the abutment or pier through a bolted or welded steel base plate.

Bearings must be very strong to support the large loads applied by the girders. However, they must also be flexible enough to allow for thermal expansion/contraction, and other displacements. To ensure that bridge bearings retain this flexibility, regular maintenance is required. Life-cycle cost analyses provide clients with a basic idea of how much this maintenance will cost over the life of the bridge. This project conducted a life-cycle cost analysis of bridge bearings to gain skills in conducting such analyses. The results of the analyses show that there can be wide variations in life cycle costs based on whether key factors (maintenance cost, initial cost, and interest rates) are high, low, or average. Based on these variations, it can be concluded that engineers must develop ways to precisely determine the correct values for these factors in order to provide clients with a good estimate of a structure's life cycle cost.

Piers

The bearings transfer the vertical loads to the pier as an evenly distributed load across the area of the bearing. The vertical loads cause the pier cap to act as a beam. It deflects downward between the columns or on either side of the column. The columns themselves take the load from the cap and act in the same manner as a building column. It is important to design the columns so that they will not buckle. The column transfers the vertical loads to a spread footing. The load transferred to the footing has a greater magnitude to account for the fact that the footing must be able to withstand two-way shear.

The life cycle cost of a pier is primarily influenced by its surface area. This project showed that multi-column piers have smaller surface areas than single leg piers, which causes multi-column piers to have a lower life cycle cost. This may not always be the case however; the exact geometry required for each pier type should be checked by the engineer in the design process, and the most cost effective type should be chosen. It should be noted that cost should not be the only factor in the engineer's decision making process; other factors could include strength or performance requirements. These requirements could require that the engineer choose a less cost effective design.

Abutments

The cantilever abutment not only withstands the vertical loads applied by the superstructure, it also retains the horizontal loads due to earth pressures and wind. The back wall and stem design is done in a similar method to a column design, since the abutment design is done on a one-foot strip of abutment across the width of the bridge. A strip footing can serve as shallow foundation if the soil permits it. It will resist the vertical loads through the net bearing pressure of the soil, as well as the longitudinal and transverse forces.

Foundations

The footing receives the load and acts as slab. If the footing also serves as the piers foundation it will resist the vertical loads through the net bearing pressure of the soil. The footing is designed to withstand the forces in both the longitudinal and transverse directions. If a footing is not sufficient to act as the foundation then a deep foundation is utilized. The deep foundation will transfer the load to the soil through side friction and toe bearing pressure.

Final Remarks

This project has illustrated certain factors that need to be considered in the design of highway bridge components. The primary goal of each design activity was to develop a design that could resist the applied loading and that could be constructed, while ensuring that the life-cycle cost and cost of construction was reasonable. Ensuring that these three criteria were met was required for this project, and is important for the design of actual bridges. Due to the fact that bridges play such an important role in society (providing means of traveling from place to place and transporting goods with relative ease) it is essential that they are structurally sound. It is also important to develop cost efficient designs for bridges, and to study ways of decreasing construction and maintenance costs (this could be done, for example, by improving analytical models to capture the extra strength provided by bracing members); this concept of developing a sustainable design would allow for a more efficient use of limited funds. This could potentially decrease taxes required for construction and maintenance activities, or allow for more bridges to be constructed in areas that they are needed. Ultimately, the primary concern of bridge engineers should be life safety; although it is important to develop economic designs, it is essential that those designs will be able to adequately resist applied loads.

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Bridge Performance and Design

Project Appendixes

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March, 2008

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Appendix A

Project Proposal

Bridge Performance and Design

Major Qualifying Project Proposal

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A-Term, 2008

Abstract

This project will study the structural design of highway bridge components. Alternatives will be established to evaluate cost effective designs. Finite element computer modeling will be performed to allow for analysis of complicated phenomena such as stress distribution in connections and seismic impact on structures. This project will consider several real world constraints.

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I Introduction

This project will study the structural design of a highway overpass. There will be three main points of investigation: preliminary design, computer modeling, and discussion of constraints. Because we have little experience in bridge design, background research will be performed to familiarize ourselves with the behavior of highway bridges, and preliminary bridge design practices. We hope that by completing this project, we will develop a fundamental understanding of bridge design and behavior, learn advanced analysis and design techniques, and develop understanding of the different constraints faced by design engineers in practice and their impact on the project.

I.1 Capstone Design

This project will satisfy the capstone design requirements outlined by ABET and the American Society of Civil Engineers (ASCE). The problem that will be investigated will be the design of a highway overpass. Basic bridge design principles will be applied to ensure that standard engineering practice is followed. Also, design alternatives will be established and compared to each other in order to establish a cost effective design. Several of the constraints listed in the ASCE commentary will be addressed. These include: economic, environmental, sustainability, manufacturability, and health and safety. These constraints will be addressed in Chapter III by considering the types of challenges that fall into these categories faced by designers in practice, and incorporating them into our design process.

II Background

To design a bridge, a fundamental understanding of its basic structural components and how they behave under load is needed. First, common materials of construction are discussed. Next the different components that make up a bridge are discussed; these are divided into two categories, superstructure components, and substructure components.

II.1 Materials of Construction

Before starting the preliminary design of a bridge, it is important to understand the differences in the materials from which the bridge may be built. These materials consist mainly of concrete and steel. Concrete performs very well when resisting compression; however, it does not do as well when a tension force is applied. To counter this, steel is added to concrete to provide tensile strength. When the reinforced concrete is loaded the concrete will take the compression load and the steel will take the tension load. The main disadvantage of building a structure out of reinforced concrete is that it has a larger dead load than steel.

Steel is a material that performs well when loaded in either tension or compression. Two popular types of steel girders can be used: hot-rolled sections and plate girders. Compared to concrete, steel is stronger, however material costs are higher, and labor is generally more expensive (Troitsky, 1994).

II.2 Design Loads

AASHTO provides many different types of loads to be considered in bridge design. These loads can be classified in one of two categories: permanent (dead) loads and temporary (live) loads. Permanent loads are generally fairly easy to determine; unit weights of commonly used materials are provided in relevant bridge design codes, providing an easy way of determining the weight of the structure. Live loads can be broken down into two categories: vehicular live loads and other types of live loads. Vehicular live loads include traffic passing over the bridge. Examples of other types of live loads include wind loads, earthquake load etc. (AASHTO, 2007). AASHTO categorizes loads in a similar way as ASCE in their specification on Minimum Design Loads for Buildings and Other Structures.

Vehicular live loads are applied to the bridge in discrete strips, known as design lanes. These lanes include a uniformly distributed load, and a point load to represent a truck; the point load should be placed so as to cause the most critical effect in the member being designed. The loads in the design lanes are increased to account for certain phenomena which commonly occur on bridges. These can include impact, fatigue, centrifugal force, braking force, and vehicle collision. The number of design lanes on a bridge depends on its width; the wider the bridge, the more number of lanes. AASHTO provides reduction factors for the intensity of the load based on the number of design lanes (AASHTO, 2007).

The figure below show how lane loads are applied. It shows one example of applying the lane loads to produce a maximum effect of the phenomenon being investigated. In this figure, a bridge with two lane loads is shown, and they are applied to cause a maximum torque in the deck.

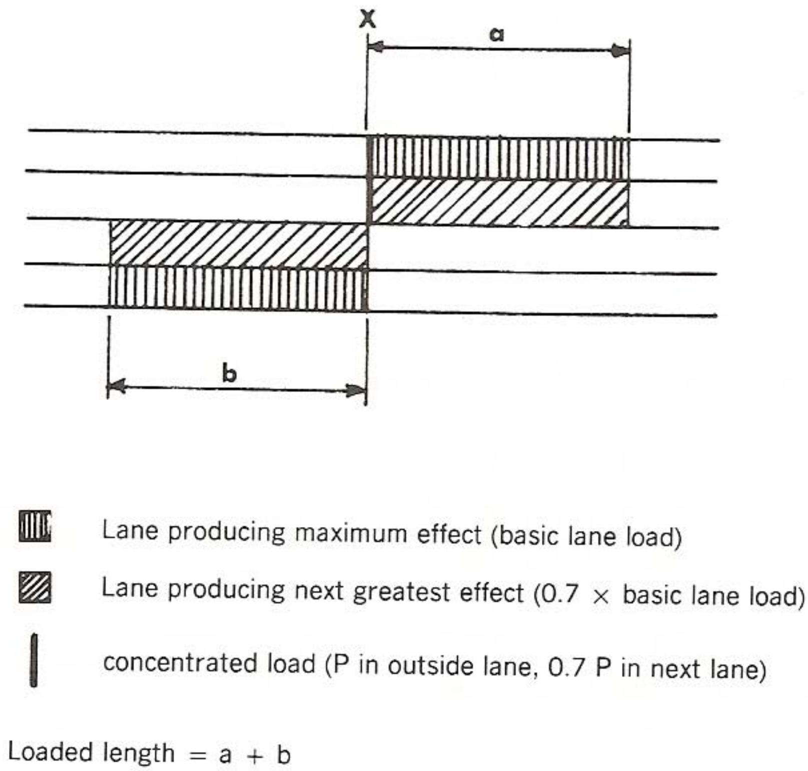

Figure 1: Applying Lane Loads
Xanthakos, Petros P. Theory and Design of Bridges. New York: Wiley-Interscience, 1993.

AASHTO provides four different limit states that bridges should be able to withstand. These include strength, extreme event, service, and fatigue. The limit state being designed for determines the load combination that is to be used, and determines the load factor that is to be applied (AASHTO, 2007).

II.3 The Superstructure

To get a better understanding of the components of the bridge, we divide it into two sections, the superstructure and the substructure. The superstructure is generally composed of the deck, girders, and expansion joints. The superstructure carries the traffic loads on the bridge and transfers them to the substructure (O'Connor, 1971). Figure 2, below, shows the different parts of the bridge, items one and two are part of the superstructure.

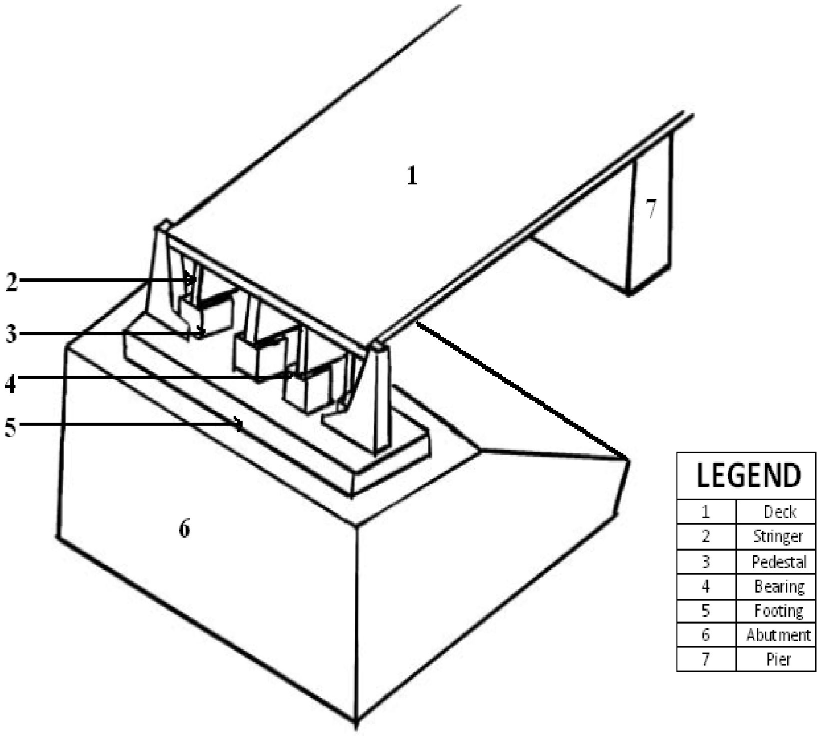

Figure 2: Bridge Components
Carmichael, Adam and Desrosiers Nathan. "Comparative Highway Bridge Design." 28 Feb. 2008. Worcester Polytechnic Institute. 10 Sept. 08 http://www.wpi.edu/pubs/e-project/available/e-project-022608-180459/unrestricted/comparative_highway_bridge_design_lda0802.pdf.

II.4 Substructure

The substructure supports the superstructure. It carries the loads above it, and transfers them to the foundations, and then to the ground. The substructure is made up of bearings, abutments, and piers as seen in Figure 2 as items 4,6, and 7. Foundations are also considered part of the substructure.

III Methodology

This project will investigate the design of a highway overpass. The work will be organized in three discrete sections: preliminary design, computer modeling, and investigation of constraints. The basic design will involve designing structural components of the bridge. Several design alternatives will be established for each component to determine the most cost effective design. The second section will involve computer modeling. The adequacy of the structure subject to earthquake loads will be evaluated by performing finite element analyses on our preliminary design. Finally, we will investigate some of the constraints faced by design consultants in real life. These include constructability, health and safety, economics, sustainability and environmental constraints.

III.1 Preliminary Design

The preliminary design will include the design of all major bridge components. These include determining the design loads, designing the deck, girders, horizontal bracing, bearings, piers, abutments, foundations, and connections. This phase will have two main purposes. First we will learn the procedures that are followed in bridge design. We will become familiar with AASHTO’s specification, and we will review several bridge design textbooks in order to develop a fundamental understanding of how bridges work. Second we will develop several design alternatives for each bridge component. The study of alternatives will allow us to determine the effect of different designs on the economy and constructability of the project. A breakdown of the topics that will be investigated in the basic design can be seen below:

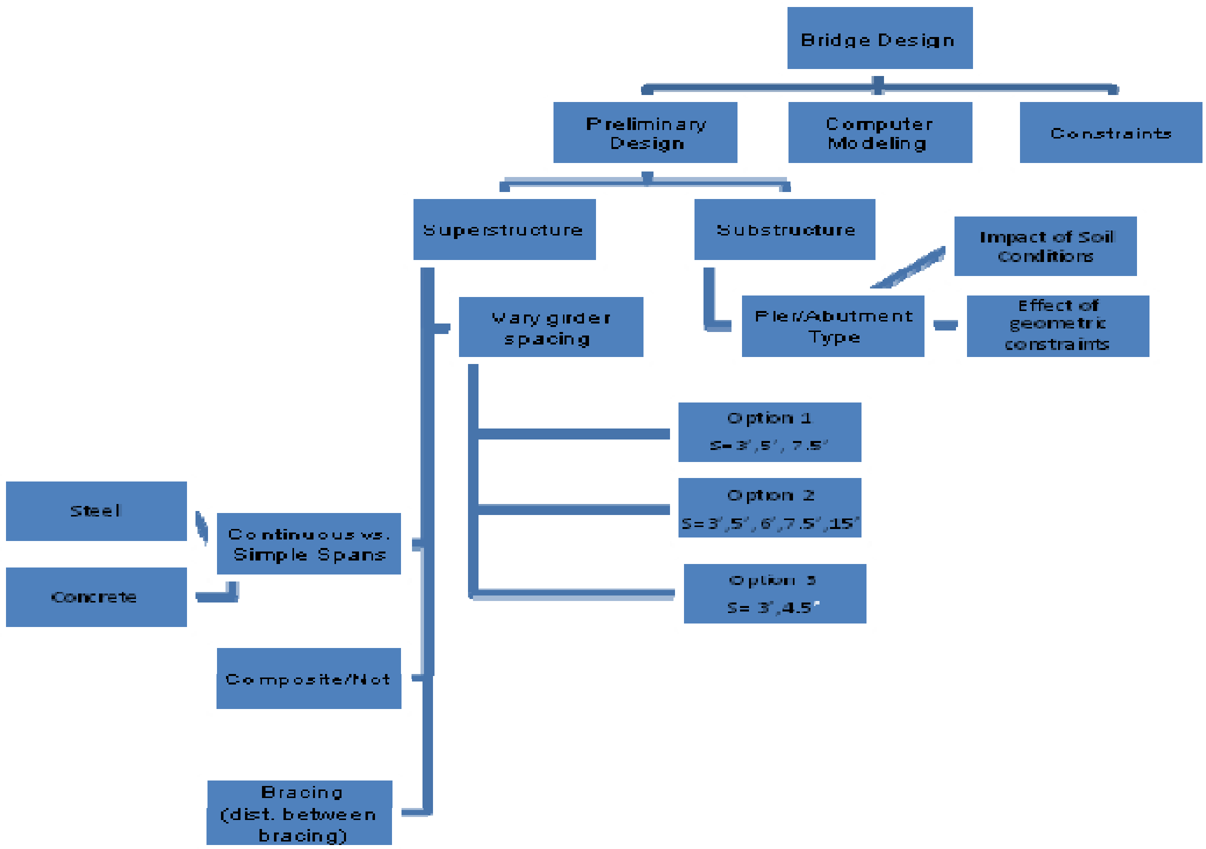

Figure 3: Breakdown of Preliminary Design

III.1.1 Design Loads

The first part of the project will be determining the design loads that the bridge will need to support. These will include dead load, live load, wind load, and earthquake load. The design loads will be determined in accordance with section three of the AASHTO Bridge Design Specification; appropriate load combinations will also be selected from this section. Because computer software allows for a quick and simple way of analyzing a large number of load combinations, all of the limit states will be investigated.

III.1.2 The Deck

Once the design loads have been calculated the deck will be designed. The deck will be designed using reinforced concrete as a material. Appropriate American Concrete Institute (ACI) and AASHTO standards will be applied during the design. Three alternatives will be investigated with the deck design. These alternatives can be seen below:

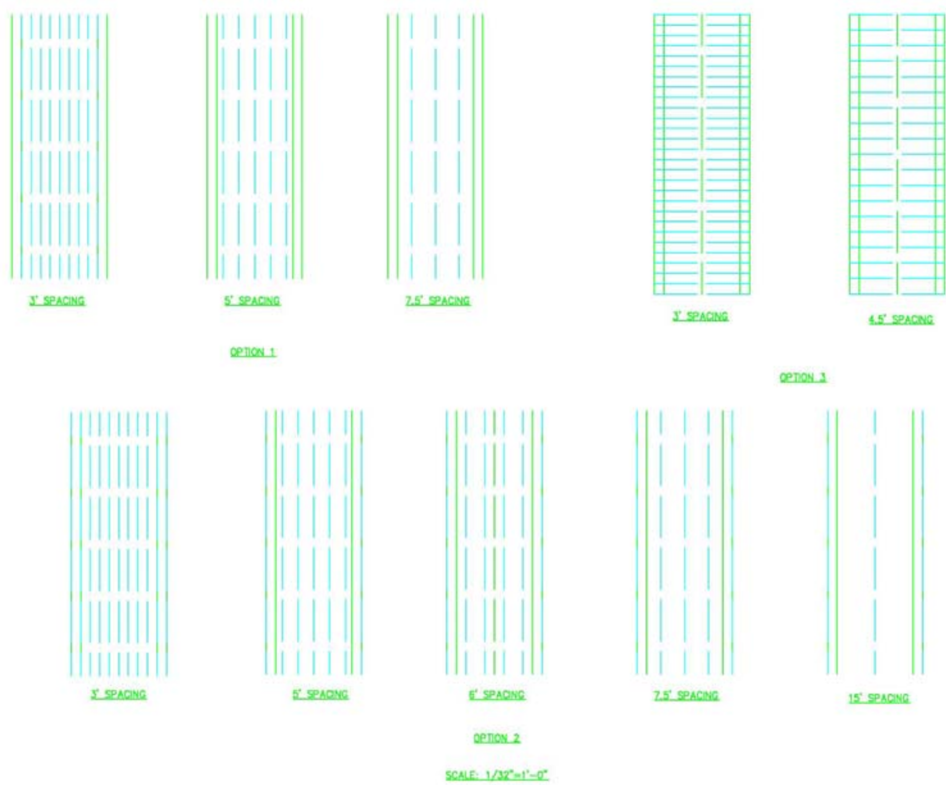

Figure 4: Deck Design Alternatives

The design alternatives are based on using a cantilevered slab at the end, varying the girder spacing, and changing the direction that the slab spans. The different alternatives shown above were chosen to ensure that the slab would only exhibit one way action, and that the spacing of the girders would be equal. The most effective slab design will be evaluated based on the cost estimate developed from the different designs. The girders and deck will be designed at the same time because the different alternatives affect the design of each.

The effect of composite action on the deck's design will also be investigated. For each of the alternatives shown above, the beams and girders will be designed to act compositely and on non-compositely. We will also vary the degree to which the deck and girders exhibit composite action by varying the number of shear studs in our design.

III.1.3 Beams and Girders

In design, the spacing of the girders is often varied; the variation affects the design of the deck and the girders. The most economical spacing option is then chosen. When the spacing between girders becomes large, intermediate beams are added to the structural system. These beams are placed perpendicular to traffic, and frame into the girders. This prevents a need for a large and heavily reinforced deck (Xanthakos, 1994).

The deck can act compositely with the girders by connecting the elements together. This provides extra load carrying capacity to the system because the two members work together to resist loads (Tonias, 1995). There are several design considerations associated with composite deck-girder systems; one consideration is the effect of a change in curvature of the system for continuous girders (Xanthakos, 1994). Despite the complexities associated with the design of composite systems, the American Association of State Highway Transportation Officials

(AASHTO) recommends their use unless it is prohibited by some factor (AASHTO, 2007).

The girders will be designed using both hot rolled steel sections and reinforced concrete sections. Also, continuous spans will be compared to simple spans. The alternatives that will be evaluated in the girder design will be the implications of using two different materials, the implications of using continuous and simple spans, and the implications of the alternatives mentioned in section III.1.2.

III.1.4 Horizontal Bracing

The horizontal bracing will be designed with few alternatives. The spacing between braces will be varied to determine if the spacing plays a large role in the selection of member size. The varied spacing will fall inside the limits set by AASHTO. The primary goal of the horizontal bracing design will be to learn how bridge superstructures resist lateral loads.

III.1.5 Bearings

Usually, the bearings are connected to the superstructure and substructure of the bridge with steel sole plates and a steel masonry plate respectively. The steel sole plates can be bolted or welded, in the case of having steel girders or can be embedded into the concrete with anchor studs, in the case of having concrete girders (Chen & Duan, 1999).

The bearings will be designed using standard engineering practice. The different types of bearings and their effect on superstructure design will be investigated. We will provide general guidelines for choosing what type of bearings to use based on the way the superstructure is design to behave, and the types of conditions the bridge will be subject to. The information on bearing design will be obtained from the Federal Highway Association guidelines and from other relevant literature.

III.1.6 Piers

The selection of proper pier type depends on the type of superstructure, whether the bridge is over a waterway or not, and the height of the piers. It depends on the superstructure since steel girder superstructures are usually supported by cantilevered piers, while cast-in-place concrete superstructures are usually supported by monolithic bents.

Several different pier types will be investigated. We will attempt to determine what sorts of conditions makes the use of different piers appropriate. These conditions could include soil conditions or geometric constraints.

The following figure, which is a table from Chen & Duan's 1999 book summarizes the general guidelines for the selection of pier types.

TABLE 2.1 General Guidelines for Selecting Pier Types

		Applicable Pier Types
		Steel Superstructure
Over water	Tall piers	Pier walls or hammerheads (T-piers) (Figures 2.3a and b); hollow cross sections for most cases; cantilevered; could use combined hammerheads with pier wall base and step tapered shaft
	Short piers	Pier walls or hammerheads (T-piers) (Figures 2.3a and b); solid cross sections; cantilevered
On land	Tall piers	Hammerheads (T-piers) and possibly rigid frames (multiple column bents)(Figures 2.3b and c); hollow cross sections for single shaft and solid cross sections for rigid frames; cantilevered
	Short piers	Hammerheads and rigid frames (Figures 2.3b and c); solid cross sections; cantilevered
		Precast Prestressed Concrete Superstructure
Over water	Tall piers	Pier walls or hammerheads (Figure 2.4); hollow cross sections for most cases; cantilevered; could use combined hammerheads with pier wall base and step-tapered shaft
	Short piers	Pier walls or hammerheads; solid cross sections; cantilevered
On land	Tall piers	Hammerheads and possibly rigid frames (multiple column bents); hollow cross sections for single shafts and solid cross sections for rigid frames; cantilevered
	Short piers	Hammerheads and rigid frames (multiple column bents) (Figure 2.5a); solid cross sections; cantilevered
		Cast-in-Place Concrete Superstructure
Over water	Tall piers	Single shaft pier (Figure 2.4); superstructure will likely cast by traveled forms with balanced cantilevered construction method; hollow cross sections; monolithic; fixed at bottom
	Short piers	Pier walls (Figure 2.4); solid cross sections; monolithic; fixed at bottom
On land	Tall piers	Single or multiple column bents; solid cross sections for most cases, monolithic; fixed at bottom
	Short piers	Single or multiple column bents (Figure 2.5b); solid cross sections; monolithic; pinned at bottom

Figure 5: General Guidelines for Selecting Pier Types
Chen, Wai-Fah (Ed.) & Duan, Lian (Ed.) (1999). Bridge Engineering Handbook. Boca Raton, Florida: CRC Press

III.1.7 Abutments

The design of abutments depends in part upon the soil conditions at the project site. If the site is mostly hard bedrock a vertical, close-end, abutment will be sufficient; if the soil is softer, a sloped, open-end, abutment will most likely be necessary. However, the use of sloped abutments usually requires longer bridge spans and extra earthwork; this could increase in the bridge construction cost (Chen & Duan, 1999).

An abutment needs to be designed to resist loads in three critical locations: bottom of the backwall, bottom of stem or top of footing, and the bottom of the footing. AASHTO does not provide standards for abutment backwall, stem, or footing minimum or maximum dimensions. As a result, the preliminary abutment dimensions will be based on recommendations from relevant literature (FHWA, Bridge Technology).

In this project, we will investigate what sorts of circumstances warrant the use of the different types of abutments listed in the background chapter in a similar way to the piers.

III.1.8 Foundations

This project will study what type of foundations should be used for a given set of conditions. These conditions could include soil type, geometric constraints, or load carrying capacity demands. The types of foundations that will be studied include various types of deep and shallow foundations.

III.1.9 Connections

The connections of the bridge will be designed only for one of the alternatives being investigated for this project. The main purpose of designing the connections is to conduct a finite element analysis of one of them. The design/analysis methods outlined by the American Institute of Steel Construction (AISC) will be followed, and compared to the results of the finite elements analysis.

III.2 Computer Modeling

This project will make use of various types of computer software for engineering design. The use of software for structural design is becoming increasingly common in engineering practice; by using software in this project, we will learn the advantages and disadvantages of using it, and also familiarize ourselves with a few specific programs (IStructE, 2008). Computer modeling will also allow us to conduct a more detailed analysis of complex situations, such as seismic effects on bridge structures. We will also compare the results of an analysis based on the approach outlined in various design specifications, with a more precise analysis from computer software. This will allow us to determine the adequacy of the methods of the design specifications, and better understand them. Finally, computer modeling will be used in the preliminary structural design of our bridge; certain uses could include analyzing indeterminate structures and analyzing structures subjected to many load combinations. A breakdown of how computer modeling will be used in this project can be seen in the figure below:

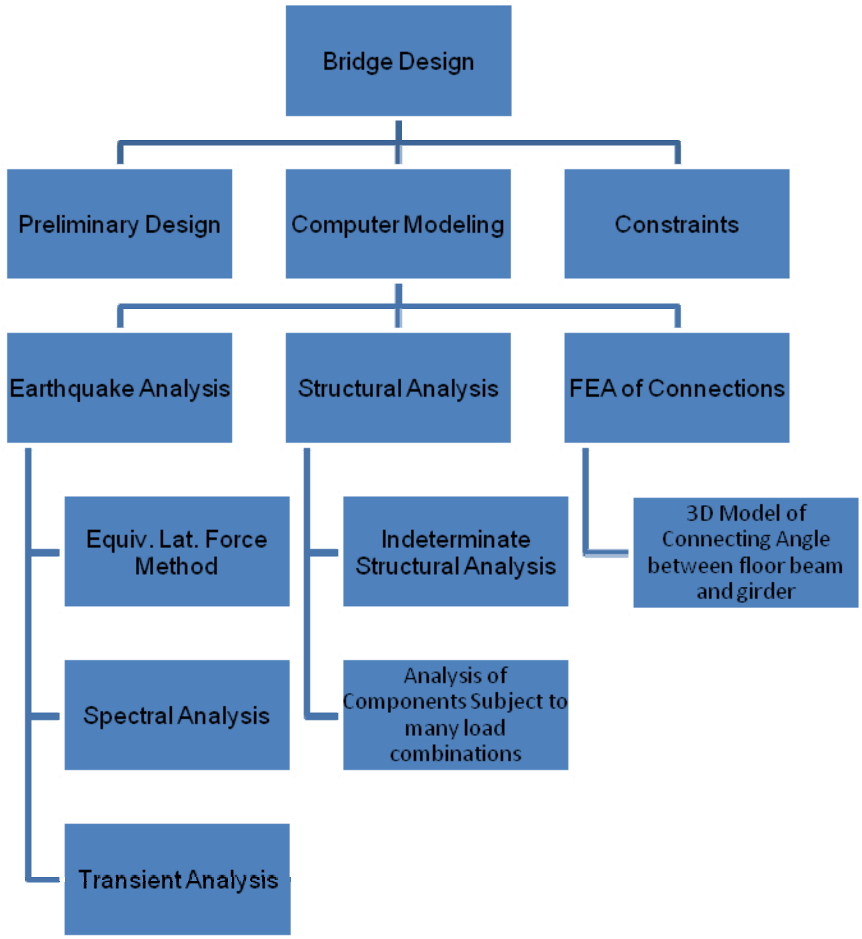

Figure 6: Breakdown of Computer Modeling Tasks

III.2.1 Software to be Used

This project will use several popular structural analysis/design software titles. First, we will use a basic structural analysis program, most likely Risa 2D. Risa will be used in the basic design of the bridge to analyze statically indeterminate structures, and structures subjected to a large number of load combinations. The use of Risa will increase the number of situations we are able to analyze by decreasing the amount of time required for analysis. We may also make use of Risa's design algorithms, which will optimize the selection of structural steel shapes or the reinforcement for concrete members.

We will also make use of finite element programs. Finite element analyses will allow us to obtain fairly accurate results from extremely complex situations. ANSYS will be the finite element program we use the most, however, LDSYNA may also be used for dynamic analyses.

III.2.2 Finite Element Analysis of Connections

Connections will be modeled in ANSYS, and the stress distribution through them will be investigated. A simple connection, such as two plates welded together subject to tensile loads, is the type of connection most likely to be analyzed. The results of this analysis will be compared to the results of the design approach outlined in the AISC specification.

III.2.3 Seismic Load Analysis

This project will investigate the effects of seismic forces on our bridge design. Several different methods for determining the seismic resistance capabilities of a structure will be carried out, and compared. These will include the equivalent later force method, a spectral analysis, and a transient analysis.

The simplest method of analysis is the equivalent later force method. This method is outlined in several specifications, including the AASHTO LRFD Bridge Design Specification and ASCE-7 Minimum Design Loads on Buildings and Other Structures.

The spectral analysis method is the next simplest method. The spectral analysis will be carried out using finite element analysis software, specifically, ANSYS. The general procedure that will be followed is outlined below:

1. Build the model
2. Obtain modal solution
3. Obtain spectrum solution
4. Expand the modes
5. Combine the modes
6. Review the results

The steps listed above will be carried out by consulting relevant literature (Structural Analysis Guide, 2008). Simplified techniques will be applied where appropriate; an example could be constructing a dual stick-beam model, rather than building a detailed model of the bridge (Meng and Lui, 2002).

The transient analysis will also be carried out using ANSYS. Relevant literature will be consulted as a guide for completing this analysis. The primary different between the transient analysis and the spectral analysis is that the transient analysis will carry out a non-linear analysis, potentially leading to more accurate results (Kappos, 2002). Both the time history analysis and the spectral analysis will be of the El Centro earthquake.

The results of these analyses will be compared, and the adequacy of the different methods will be determined. Emphasis will be placed on choosing the simplest method which provides reasonably accurate results. An example could be recommending the use of the simple equivalent later force method in areas of low seismic risk, while using the more complicated transient analysis in areas of high seismic risk.

III.2.4 Computer Modeling in Preliminary Design

Risa will be used in our preliminary design to conduct analyses which would take a long time to do by hand. Examples include indeterminate structures, and structures subject to many different load combinations. We may also use design algorithms in Risa to size structural steel members and determine the required reinforcement of concrete members. By carrying out these repetitive calculations with a computer, we will be able to concentrate our efforts on new topics that we are not familiar with, increasing the amount we can learn from this project.

III.3 Constraints

This project will address several constraints which professional engineers face when working on design projects. We hope that this will allow us to develop a better appreciation for the issues which need to be addressed in engineering projects. We will also satisfy our capstone design requirement by addressing these constraints. The constraints we will look at are economic, environmental, sustainability, manufacturability, health and safety, and political. A breakdown of how we will address each constraint can be seen in the figure below:

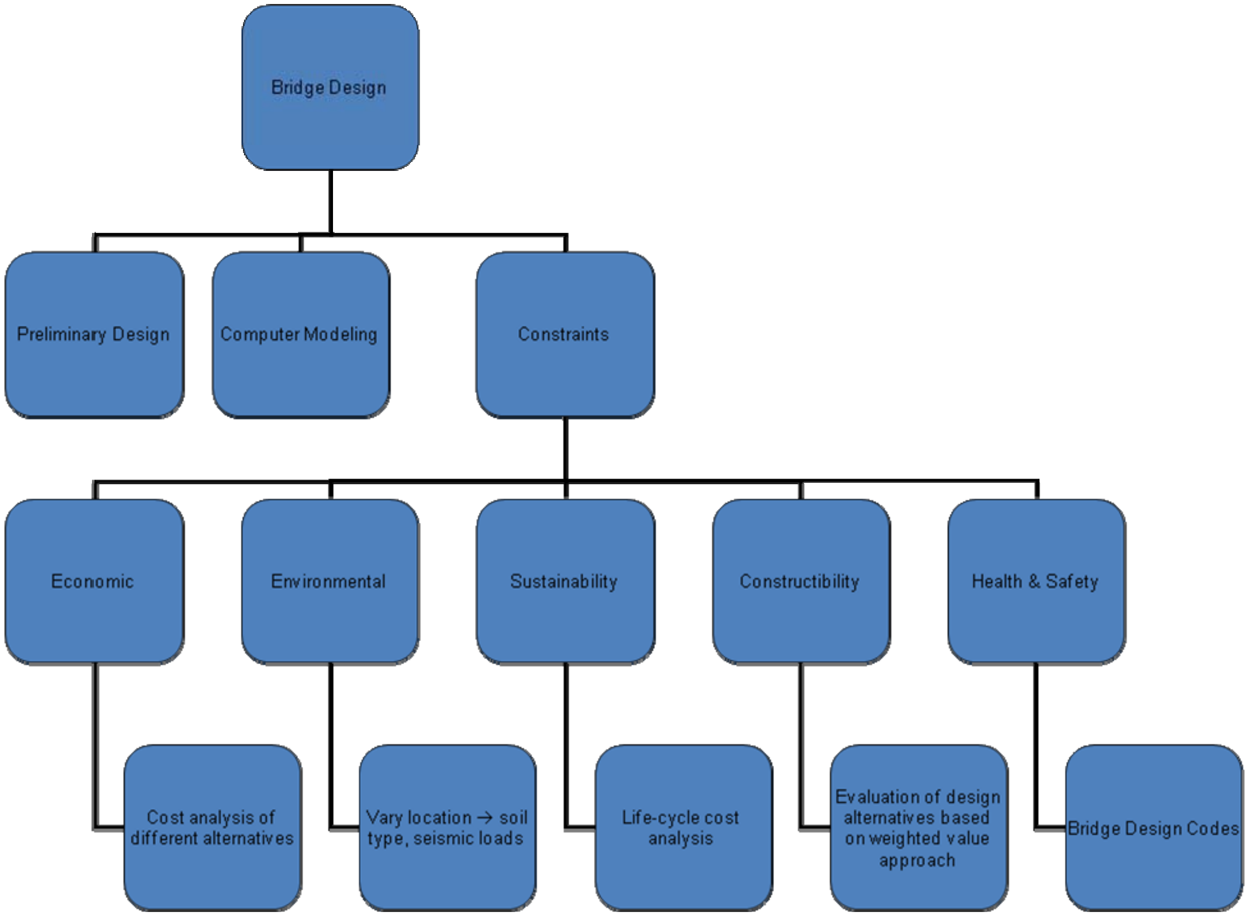

Figure 7: Breakdown of Constraints

III.3.1 Economic Constraints

Because of the high cost of most civil engineering projects, producing an economically viable design is very important. This project will address this issue by investigating several different alternatives and determining the economic implications of each design. We will do a cost analysis of each design by consulting relevant literature.

III.3.2 Environmental Constraints

There are many environmental constraints associated with bridge design. One example could be that the site that the bridge is to be built on is protected land. This project will focus more on the structural design of a highway overpass. Environmental constraints in this project will mostly relate to the location of the project. The seismic forces are heavily dependent on the location of the structure. Also, the location affects the soil profile, which will in turn affect the design of the foundations.

III.3.3 Sustainability Constraints

This project will address sustainability by performing a life-cycle cost analysis. We will study how each design alternative affects the cost to maintain the bridge throughout its life. This is an important exercise because a design which has very low construction costs may cost the owner more money over time than a design with a higher initial construction cost.

III.3.4 Constructability Constraints

Creating a design which can be easily built is often difficult, especially for young engineers. Because of their lack of experience, they do not anticipate some of the problems contractors can face when performing work on the job site. For example, a design might call for a welded connection; to perform that weld the welder may need to stand on a platform high above the ground and weld from underneath the connection rather than level with it. This would make the weld nearly impossible to perform, and a new design would be needed. We will attempt to create a design which is constructible. We will compare each of our design alternatives to each other from a constructability point of view. Each one will be ranked in several different categories, some of which may include: procurement, placement, and safety concerns. The importance of each category to constructability will be determined, and the design alternatives will be ranked in terms of constructability.

III.3.5 Health and Safety Constraints

Health and safety will be addressed by following the applicable bridge design codes. By following the relevant codes, we will ensure that we are designing a structure which is reasonably safe. These constraints are related to the economic constraints because, although we will make a safe design, we will not overdesign, which would lead to additional unnecessary cost.

III.4 Conclusions

This project will have three main goals: learning preliminary bridge design skills, learning the cost effectiveness of different designs, and investigating real world constraints that engineers face when working the field. These goals will be met by working in the areas of basic design, computer modeling, and investigation of constraints.

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Appendix A: Project Schedule

1	**B-Term Schedule & Deliverables**								
2	Task	Week1	Week2	Week3	Week4	Week5	Week6	Week7	Who works on it?
3	Beam&Girder	Active	Active	Active	Not Active	Not Active	Not Active	Not Active	Everyone
4									
5	Bracing	Active	Active	Active	Not Active	Not Active	Not Active	Not Active	Everyone
6									
7	Bearings	Active	Active	Active	Not Active	Not Active	Not Active	Not Active	Everyone
8									
9	Pier	Not Active	Active	Active	Active	Not Active	Not Active	Not Active	Dan
10									
11	Abutment	Not Active	Active	Active	Active	Not Active	Not Active	Not Active	Alejandro
12									
13	Foundation	Not Active	Active	Active	Active	Not Active	Not Active	Not Active	Dan/Alejandro
14									
15	Connection	Active	Active	Active	Not Active	Not Active	Not Active	Not Active	Besian/Emre/Doug
16									
17	Cost Analysis	Not Active	Not Active	Active	Active	Active	Active	Active	Everyone
18									
19	Life-Cycle Analysis	Not Active	Not Active	Not Active	Not Active	Active	Active	Active	Emre/Dan/Alejandro
20									
21	Dynamic Analysis	Active	Active	Active	Active	Active	Active	Active	Doug
22									
23	Constructability Analysis	Not Active	Not Active	Active	Active	Active	Active	Active	Everyone
24									
25	FEA	Active	Active	Active	Active	Active	Active	Active	Besian
26									
27	Report	Active	Active	Active	Active	Active	Active	Active	Everyone
28									
29	* Everyone working on an item means that everyone will investigate at least 1 alternative/work on part of it								
30									
31	Included in Report at the End of B-Term:	Procedure of superstructure design					**A-Term Goals:**		Complete proposal
32		Procedure of substructure design							Determine design loads
33		Design of superstructure alternatives							Begin superstructure design
34		Design of substructure alternatives							Beginning of report
35		Preliminary cost estimate							
36		Preliminary life-cycle cost esitmate							
37		Preliminary constructability analysis							
38		Summary of dynamic analysis progress							
39		Summary of FEA progress							

40	**C-Term Schedule & Deliverables**								
41	Task	Week1	Week2	Week3	Week4	Week5	Week6	Week7	Who works on it?
42	Cost Analysis	Active	Active	Active	Active	Not Active	Not Active	Not Active	Emre/Dan/Alejandro
43									
44	Life Cycle Analysis	Active	Active	Active	Active	Not Active	Not Active	Not Active	Emre/Dan/Alejandro
45									
46	Dynamic Analysis	Active	Active	Active	Active	Active	Not Active	Not Active	Doug
47									
48	FEA	Active	Active	Active	Active	Active	Not Active	Not Active	Besian
49									
50	Constructability Analysis	Active	Active	Active	Not Active	Not Active	Not Active	Not Active	Emre/Dan/Alejandro
51									
52	Report	Active	Active	Active	Active	Active	Active	Active	Everyone

Appendix B

Slab Design

$f'_c = 4$ ksi $f_y = 60$ ksi

- eq.'s same for (+) & (-) steel
- (t) select trial
- $d = t - 1'' - \frac{1}{2} d_b$ (+) steel
- $d = t - 2.5'' - \frac{1}{2} d_b$ (-) steel
- $b = 12''$. $\phi = 0.9$
- M_u from Risa
- $R_n = \dfrac{M_u \times 12{,}000}{\phi b d^2}$
- $\rho = \dfrac{0.85 f'_c}{f_y} \left(1 - \sqrt{1 - \dfrac{2 R_n}{0.85 f'_c}} \right)$
- $A_{s\,req'd} = \rho b d \rightarrow A_s$ based on provided steel
- $a = \dfrac{A_s \times f_y}{0.85 f'_c b}$
- $c = \dfrac{a}{0.85}$
- $\varepsilon = 0.003 (d - c)/c \rightarrow$ TCL if $\varepsilon > 0.005$
- Dist steel$_{req'd}$ if $\dfrac{220}{\sqrt{s}} > 67 \rightarrow A_s \times 0.67 \xrightarrow{\text{else}} \dfrac{220}{100\sqrt{s}} \times A_s$

Option 1

3 ft spacing

	Thicknes s	d	b	phi	Mu	Rn	p	As req'd	Main Reinforcement	As	a
(+)								0.59644			0.8
steel	18	16.5625	12	0.9	43.277	175.2923065	0.003	9	#7 @12" BOT	0.6	8
(-)							0.016	2.96716		3.1	4.5
steel	18	14.865	12	0.9	169.35	851.555316	6	5	#11 @6" TOP	2	9

5 ft spacing

	Thicknes s	d	b	phi	Mu	Rn	p	As req'd	Main Reinforcement	As	a
(+)							0.002	0.75165		0.7	1.1
steel	24	22.5	12	0.9	74.236	162.9322359	8	9	#8 @12" BOT	9	6
(-)							0.013	3.47003		3.8	
steel	24	20.795	12	0.9	284.875	731.9711758	9	2	#10 @4" TOP	1	5.6

7.5 ft spacing

	Thicknes s	d	b	phi	Mu	Rn	p	As req'd	Main Reinforcement	As	a
(+)							0.002	0.90110			1.4
steel	34	32.436	12	0.9	128.84	136.0672549	3	2	#9 @12" BOT	1	7
(-)							0.013	4.55166		4.6	6.8
steel	32	28.6535	12	0.9	518.344	701.4879414	2	5	#11 @4" TOP	8	8

Option 1

3 ft spacing

	c	Strain (s)	TCL ?	Req'd Dist. Steel	Dist. Rebars	Provided Dist. Steel	Volume (yd^3)
(+)	1.03806	0.04486					
steel	2	6	YES	0.402	#6 @12"	0.44	270
(-)	5.39792	0.00526					
steel	4	2	YES	2.0904	#9 @6"	2	

5 ft spacing

	c	Strain (s)	TCL ?	Req'd Dist. Steel	Dist. Rebars	Provided Dist. Steel	Volume (yd^3)
(+)	1.36678	0.04638					
steel	2	6	YES	0.5293	#7 @12"	0.6	360
(-)	6.59169	0.00646					
steel	6	4	YES	2.5527	#10 @6"	2.54	

7.5 ft spacing

	c	Strain (s)	TCL ?	Req'd Dist. Steel	Dist. Rebars	Provided Dist. Steel	Volume (yd^3)
(+)	1.73010	0.05324					
steel	4	4	YES	0.67	#8 @12"	0.79	510
(-)	8.09688	0.00761					
steel	6	6	YES	3.1356	#11 @6"	3.12	

Appendix C

Continuous Concrete Girder

Concrete Girder Design Continuous

0253 $h_f = 8"$, $M_u = 2231$ ft-k $b_e = 36"$

say $b = 24"$ assume 2 layers steel $h = 48"$

$d = 44.5"$

$$A_s = \frac{2231(12000)}{.9(60000)(.95)(44.5)} = 11.73 \text{ in}^2$$

Use 2 rows 4 No 11 bars

$A_s = 12.48$ in^2

Req'd $b_w = 14.0" < 24"$ ✓

$A_{s\,min}$ same as 3' spacing simple design ✓

$$a = \frac{12.48(60000)}{.85(4000)(36)} = 6.12 \text{ in}$$

$$\phi M_n = \frac{.9(12.48)(60000)(44.5 - 6.12/2)}{12000} = 2327.3 \text{ ft-k} > M_u \checkmark$$

$\boxed{\text{Use } b_w = 24",\ h = 48"\ \ h_w = 40" \quad \text{2 rows 4 No 11 bars}}$

0255

$h_f = 8"$ $M_u = 2355.88$ ft-k $b_e = 60"$

Say $b_w = 24"$, $h = 48"$ assume 2 layers steel

$d = 44.5"$

$$A_s = \frac{2355.88(12000)}{.9(60000)(.95)(44.5)} = 12.38 \text{ in}^2$$

Try 2 rows 4 No 11 bars $A_s = 12.48$ in^2

Same checks as 3' spacing

$$a = \frac{12.48(60000)}{.85(4000)(60)} = 3.67"$$

$$\phi M_n = \frac{.9(12.48)(60000)(44.5 - 3.67/2)}{12000} = 2396.1 \text{ ft-k} > M_u \checkmark$$

$\boxed{\text{Use } b_w = 24"\ \ h = 48",\ h_w = 40" \quad \text{2 rows 4 No 11 bars}}$

0256

$h_f = 14"$ $M_u = 2968.10$ ft-k $b_e = 72"$

Say $b_w = 24"$ $h = 48"$ 2 rows steel
$d = 44.5"$

$$A_s = \frac{2968.10(12000)}{.9(60000)(.95)(44.5)} = 15.60 \text{ in}^2$$

Try 2 rows 7 No 10 bars $A_s = 17.78$ in²
Req'd $b_w = 20.5" < 24"$ ✓

$A_{s\,min}$ check same as 3' spacing simple ✓

$$c = \frac{17.78(60000)}{.85(4000)(72)} = 4.36"$$

$$\phi M_n = \frac{.9(17.78)(60000)(44.5 - 4.36/2)}{12000} = 3386.0 \text{ ft-k} > M_u \checkmark$$

Use $b_w = 24"$, $h = 48"$, $h_w = 34"$
2 rows 7 No 10 bars

0257.5

$h_f = 16"$, $M_u = 3462.58$ ft-k $b_e = 90"$
Say $b_w = 24"$, $h = 48"$ 2 layers steel
$d = 44.5"$

$$A_s = \frac{3462.58(12000)}{.9(60000)(.95)(44.5)} = 18.20 \text{ in}^2$$

Try 2 rows 6 No 11 bars $A_s = 18.72$ in²
Req'd $b_w = 15.5" < 24"$ ✓
$A_{s\,min}$ check same as before ✓

$$a = \frac{18.72(12000)}{.85(4000)(90)} = .734"$$

$$\phi M_n = \frac{.9(18.72)(60000)(44.5 - .734/2)}{12000} = 3717.8 \text{ ft-k} > M_u \checkmark$$

Use $b_w = 24"$, $h = 48"$, $h_w = 32"$
2 rows 6 No. 11 bars

02515

$h_f = 22$ $M_u = 6169.57$ ft-k $b_e = 176"$

say $b_w = 36"$ $h = 60"$ 2 layers steel
$d = 56.5"$

$$A_s = \frac{(6169.57)(12000)}{.9(60000)(.95)(56.5)} = 25.54 \text{ in}^2$$

Try 2 rows 9 No 11 bars $A_s = 28.08 \text{ in}^2$
b_w req'd = 28.0" < 36" ✓
$A_{s\,min}$ check same as for 15' spacing simple ✓

$$a = \frac{28.08(60000)}{.85(4000)(176)} = 2.82"$$

$$\phi M_n = \frac{.9(28.08)(60000)(56.5 - 2.82/2)}{12000} = 6961.2 \text{ ft-k} > M_u \checkmark$$

Use $b_w = 36"$ $h = 60"$ $h_w = 38"$
2 rows 9 No. 11 bars

0253

$V_u = 150.36^k$ $V_u/\phi = 253.81^k$ $b_w = 24"$ $d = 44.5"$

$V_c/2 = 67.5^k$

$S_{max} = 14.05"$ $S_{max} = 13.33"$

$S = \frac{.4(90000)(44.5)}{253.81 - 2(67.5)} = 5.66"$ use 5" spacing

$V_n = .75\left(\frac{.4(90000)(44.5)}{5} + 2(67.5)(1000)\right) = 208.1^k > 150.36^k$ ✓

$\frac{81(12)}{5} = 194.4$

$\boxed{\text{Use 194 stirrups each span @ 5"}}$

0255

$V_u = 218.36^k$ $V_u/\phi = 291.15^k$ $b_w = 24"$ $d = 44.5"$

$V_c/2 = 67.5^k$

$S_{max} = 13.33"$

$S = \frac{.4(90000)(44.5)}{291.15 - 2(67.5)} = 4.56"$ Use 4" spacing

$V_n = .75\left(\frac{.4(90000)(44.5)}{4} + 2(67.5)(1000)\right) = 234.8^k > 218.36^k$ ✓

$\frac{81(12)}{4} = 243$

$\boxed{\text{Use 243 stirrups each span @ 4"}}$

0256

$V_u = 238.96^k$ $V_u/\phi = 318.61^k$ $b_w = 24"$ $d = 44.5"$

$V_c/2 = 67.5^k$

$S_{max} = 13.33"$

$S = \frac{.4(90000)(44.5)}{318.61 - 2(67.5)} = 3.88"$ use 3.5"

$V_n = .75\left(\frac{.4(90000)(44.5)}{3.5} + 2(67.5)(1000)\right) = 253.8^k > 238.96^k$ ✓

$\frac{81(3.5)}{12} = 277.7$

$\boxed{\text{Use 278 stirrups each span @ 3.5"}}$

O2S7.5

$V_u = 267.26^k$ $\quad V_c/\phi = 356.3^k$ $\quad b_w = 24"$ $\quad d = 44.5"$

$V_c/2 = 67.5^k$

$S_{max} = 13.33"$

$$S = \frac{.4(40000)(44.5)}{356.3 - 2(67.5)} = 3.22" \quad \text{use } 3" \text{ spacing}$$

$$V_u = .75\left(\frac{.4(40000)(44.5)}{3} + 2(67.5)(1000)\right) = 279.3^k > 267.26^k \checkmark$$

$$\frac{.81(12)}{3} = 324$$

$\boxed{\text{Use 324 stirrups each span @ 3"}}$

O2S15

$V_u = 436.26^k$ $\quad V_u/\phi = 581.68^k$ $\quad b_w = 36"$ $\quad d = 56.5"$

$V_c/2 = 128.6^k$

$S_{max} = 8.85"$

$$S = \frac{.4(40000)(56.5)}{581.68 - 2(128.6)} = 2.79" \quad \text{use } 2.5" \text{ spacing}$$

$$V_u = .75\left(\frac{.4(40000)(56.5)}{2.5} + 2(128.6)(1000)\right) = 464.1^k > 436.26^k \checkmark$$

$$\frac{81(12)}{2.5} = 388.8$$

$\boxed{\text{Use 389 stirrups each span @ 2.5"}}$

0253 $M_{u(-)} = 2146.752$ ft-k

$A_s = \dfrac{2146.752(12000)}{.9(60000)(.95)(44.5)} = 11.55 \text{ in}^2$

Use 2 rows 5 No. 10 bars $A_s = 12.70 \text{ in}^2$

$a = \dfrac{12.70(60000)}{.85(4000)(24)} = 9.34$

$\phi M_n = \dfrac{.9(12.70)(60000)(44.5 - 9.34/2)}{12000} = 2276.3$ ft-k $> M_{u(-)}$ ✓

$\boxed{\text{Use 2 rows 5 No. 10 bars}}$

0255 $M_{u(-)} = 2650.311$ ft-k

$A_s = \dfrac{2650.311(12000)}{.9(60000)(.95)(44.5)} = 13.93 \text{ in}^2$

Try 2 rows 5 No 11 bars $A_s = 15.60 \text{ in}^2$

$a = \dfrac{15.60(60000)}{.85(4000)(24)} = 11.47''$

$\phi M_n = \dfrac{.9(15.60)(60000)(44.5 - 11.47/2)}{12000} = 2721.3$ ft-k $> M_{u(-)}$ ✓

$\boxed{\text{Use 2 rows 5 No. 11 bars}}$

0256 $M_{u(-)} = 2583.834$ ft-k

$A_s = \dfrac{2583.834(12000)}{.9(60000)(.95)(44.5)} = 15.68 \text{ in}^2$

Try 2 rows 7 No. 10 bars $A_s = 17.78$

$a = \dfrac{17.78(60000)}{.85(4000)(24)} = 13.07''$

$\phi M_n = \dfrac{.9(17.78)(60000)(44.5 - 13.07/2)}{12000} = 3037.6$ ft-k $> M_{u(-)}$ ✓

$\boxed{\text{Use 2 rows 7 No 10 bars}}$

0257.5 $M_{u(-)} = 3442.317$ ft-k

$A_s = \dfrac{3442.317(12000)}{.9(60000)(.95)(44.5)} = 18.09 \text{ in}^2$

Try 2 rows 7 No 11 bars $A_s = 21.84 \text{ in}^2$

$a = \dfrac{21.84(60000)}{.85(4000)(24)} = 16.06''$

$\phi M_n = \dfrac{.9(21.84)(60000)(44.5 - 16.06/2)}{12000} = 3584.3$ ft-k $> M_{u(-)}$ ✓

$\boxed{\text{Use 2 rows 7 No 11 bars}}$

02515 $M_{u(-)} = 6179.411$ ft-k

$$A_s = \frac{6179.411\,(12000)}{.9(60000)(.85)(56.5)} = 25.58 \text{ in}^2$$

Try 2 rows 9 No 11 bars $A_s = 28.08$

$$a = \frac{28.08(60000)}{.85(4000)(36)} = 13.76''$$

$$\phi M_n = \frac{.9(28.08)(60000)(56.5 - 13.76/2)}{12000} = 6270.0$$

$\boxed{\text{Use 2 rows 9 No 11 bars}}$

Appendix D

Composite Steel Girder Design

Section	A	d	be	bf	tw	d(deck)	h
W24x131	38.5	24.5	60	12.9	0.605	22.795	26
		Mu:	5560	Vu:	281		

Check Section Strength:			
Strength Deck:	4650.18		
Strength Girder:	1925		
a:	9.4362745		
Mp:	6830.0384		
D:	47.1		
D/tw ok?	Yes		
Dt:	47.295		
Mn:	6354.233		
phi*Mn:	5718.8097	o.k.?	Yes
Ductility o.k.?	Yes		
Shear o.k.? (C=1)	Yes		

Compute Moment of Inertia:	
bf(deck):	7.5
Ideck:	7402.8476

Element	A	Y	AY	AY^2	Io
W24x131	38.5	12.25	471.625	5777.406	4020
Deck	170.9625	41.06373	7020.357	288282	7402.848
Sum:	209.4625	53.31373	7491.982	294059.4	11422.85

Iz:	305482.27
Y':	35.767654
I:	37511.649

Design for Fatigue:			
Shear Range:	x=0	281	
	x=20.25	165	
	x=41	93.33	
	x=60.75	160.453	
	x=81	277	
Q:	1109.3284		
s (three studs)	5.075		
s (four studs)	3.3833333		
Use four studs		*s must be>3.0	

N:	104709375
alpha:	0.1744618
Zr:	1.546875

Point	Vr	Q	I	Sr	p
x=0	281	1109.328	37511.65	8.309986	0.74609
x=20.25	165	1109.328	37511.65	4.879529	1.270614
x=41	93.33	1109.328	37511.65	2.760039	2.246345

x=60.75	160.453	1109.328	37511.65	4.745061	1.306622
x=81	277	1109.328	37511.65	8.191695	0.756864

Compute total studs by hand
Check Strength Limit State

Qn	27.37
# studs req'd	82.744095

Verify that fatigue controls

Appendix E

Non-Composite Steel Girder Design

Option 1

3 ft spacing

Mu	Req'd Zx	Trial Section	Zx	bf/(2tf)	h/tw	Compact?	Mu	Phi*Mn	Vu	Phi*Vn	O.K.?
2950.797	786.8792	W 40X199	869	7.39	52.6	Yes	3057.628	39105	243.511	754	Yes

5 ft spacing

Mu	Req'd Zx	Trial Section	Zx	bf/(2tf)	h/tw	Compact?	Mu	Phi*Mn	Vu	Phi*Vn	O.K.?
3688.691	983.6509	W 44X230	1100	6.45	54.8	Yes	3826.022	49500	290.953	823	Yes

7.5 ft spacing

Mu	Req'd Zx	Trial Section	Zx	bf/(2tf)	h/tw	Compact?	Mu	Phi*Mn	Vu	Phi*Vn	O.K.?
8413.67	2243.645	W 40x503	2310	2.98	22.3	Yes	9046.416	103950	613.276	1940	Yes

Appendix F

Superstructure Cost Estimate

Summary

Option 1

s (ft)	Deck Concrete Material $	Deck Concrete Labor $	Deck Rebar Material $	Deck Rebar Labor $	Girder $	Total $
3	32400	690.04224	43503.78	32653.7655	110777.09	220024.68
5	42120	897.054912	55463.16	38913.7725	76988.41	214382.4
7.5	55080	1173.071808	74532.6	51436.0188	55408.54	237630.23

Option 2

s (ft)	Deck Concrete Material $	Deck Concrete Labor $	Deck Rebar Material $	Deck Rebar Labor $	Girder $	Total $
3	12960	276.016896	23989.74	19984.1874	136790.45	194000.39
5	19440	414.025344	28249.92	24341.637	86869.42	159315
6	22680	483.029568	40607.88	34398.4674	73854.19	172023.57
7.5	25920	552.033792	40607.88	34398.4674	75561.62	177040
15	35640	759.046464	52240.74	42206.7339	56682.38	187528.9

Slab Estimate

Option 1:

Concrete:

s (ft)	Deck Thick (in)	Vol Concrete (yd^3)	Adjust Waste (yd^3)	Conc $/yd^3	Cost Concrete ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	20	300	324	100	32400	17.496	39.44	690.04224
5	26	390	421.2	100	42120	22.7448	39.44	897.05491
7.5	34	510	550.8	100	55080	29.7432	39.44	1173.0718

Main Top Reinforcement:

s (ft)	Main Top	Main Top (lf)	Main Top (lb)	$/lf	Main Top Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#10 @6"	9750	41925	2.06	20085	253.5	53.15	13473.525
5	#11 @6"	9750	51772.5	2.55	24862.5	312	53.15	16582.8
7.5	#14 @6"	9750	74587.5	3.65	35587.5	438.75	53.15	23319.563

Main Bot Reinforcement:

s (ft)	Main Bot	Main Bot (lf)	Main Bot (lb)	$/lf	Main Bot Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#7 @12"	4890	9995.16	0.92	4498.8	68.46	53.15	3638.649
5	#8 @12"	4890	13056.3	1.15	5623.5	92.91	53.15	4938.1665
7.5	#9 @12"	4890	16626	1.63	7970.7	117.36	53.15	6237.684

Dist. Top Reinforcement:

s (ft)	Dist. Top	Dist. Top (lf)	Dist. Top (lb)	$/lf	Dist. Top Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#9 @6"	9882	33598.8	1.63	16107.66	237.168	53.15	12605.479
5	#10 @6"	9882	42492.6	2.06	20356.92	256.932	53.15	13655.936
7.5	#11 @6"	9882	52473.42	2.55	25199.1	316.224	53.15	16807.306

Dist. Bot Reinforcement:

s (ft)	Dist. Bot	Dist. Bot (lf)	Dist. Bot (lb)	$/lf	Dist. Bot Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#6 @12"	5022	7543.044	0.56	2812.32	55.242	53.15	2936.1123
5	#7 @12"	5022	10264.968	0.92	4620.24	70.308	53.15	3736.8702

7.5	#8 @12"	5022	13408.74	1.15	5775.3	95.418	53.15	5071.4667

* 8% waste on concrete material (p.130 CE3021 book)

Option 2:

Concrete:

s (ft)	Deck Thick (in)	Vol Concrete (yd^3)	Adjust Waste (yd^3)	Conc $/yd^3	Cost Concrete ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	8	120	129.6	100	12960	6.9984	39.44	276.0169
5	12	180	194.4	100	19440	10.4976	39.44	414.02534
6	14	210	226.8	100	22680	12.2472	39.44	483.02957
7.5	16	240	259.2	100	25920	13.9968	39.44	552.03379
15	22	330	356.4	100	35640	19.2456	39.44	759.04646

Main Top Reinforcement:

s (ft)	Main Top	Main Top (lf)	Main Top (lb)	$/lf	Main Top Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#8 @12"	4890	13056.3	1.15	5623.5	92.91	53.15	4938.1665
5	#7 @6"	9750	19929	0.92	8970	136.5	53.15	7254.975
6	#8 @6"	9750	26032.5	1.15	11212.5	185.25	53.15	9846.0375
7.5	#8 @6"	9750	26032.5	1.15	11212.5	185.25	53.15	9846.0375
15	#9 @6"	9750	33150	1.63	15892.5	234	53.15	12437.1

Main Bot Reinforcement:

s (ft)	Main Bot	Main Bot (lf)	Main Bot (lb)	$/lf	Main Bot Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#9 @12"	4890	16626	1.63	7970.7	117.36	53.15	6237.684
5	#9 @12"	4890	16626	1.63	7970.7	117.36	53.15	6237.684
6	#8 @6"	9750	26032.5	1.15	11212.5	185.25	53.15	9846.0375
7.5	#8 @6"	9750	26032.5	1.15	11212.5	185.25	53.15	9846.0375
15	#9 @6"	9750	33150	1.63	15892.5	234	53.15	12437.1

* 8% waste on concrete material (p.130 CE3021 book)

Option 2:

Dist. Top Reinforcement:

s (ft)	Dist. Top	Dist. Top (lf)	Dist. Top (lb)	$/lf	Dist. Top Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#7 @12"	5022	10264.968	0.92	4620.24	70.308	53.15	3736.8702
5	#6 @6"	9882	14842.764	0.56	5533.92	108.702	53.15	5777.5113
6	#7 @6"	9882	20198.808	0.92	9091.44	138.348	53.15	7353.1962
7.5	#7 @6"	9882	20198.808	0.92	9091.44	138.348	53.15	7353.1962
15	#7 @6"	9882	20198.808	0.92	9091.44	138.348	53.15	7353.1962

Dist. Bot Reinforcement:

s (ft)	Dist. Bot	Dist. Bot (lf)	Dist. Bot (lb)	$/lf	Dist. Bot Cost ($)	Labor Hrs	$/(Labor*hr)	Labor ($)
3	#8 @12"	5022	13408.74	1.15	5775.3	95.418	53.15	5071.4667
5	#8 @12"	5022	13408.74	1.15	5775.3	95.418	53.15	5071.4667
6	#7 @6"	9882	20198.808	0.92	9091.44	138.348	53.15	7353.1962
7.5	#7 @6"	9882	20198.808	0.92	9091.44	138.348	53.15	7353.1962
15	#8 @6"	9882	26384.94	1.15	11364.3	187.758	53.15	9979.3377

Girders
Estimate

Option 1

s (ft)	b (in)	h-t (in)	Vol. Conc (yd^3)	$/yd^3	Conc. $	Top Rebars	Top Rebars (lf)	$/lf	Top Rebar $
3	40	50	750	112.55	84412.5	2 #8	2916	2.16	6298.56
5	48	46	460	112.55	51773	2 #8	2916	2.16	6298.56
7.5	50	46	287.5	112.55	32358.125	2 #8	2916	2.16	6298.56

s (ft)	Bot Rebars	Bot Rebars (lf)	$/lf	Bot Rebar $	Stirrups	$/Stirrup	$ Stirrup
3	33 #8	48114	2.16	103926.24	126	3.49	439.74
5	40 #8	32400	2.16	69984	170	3.49	593.3
7.5	46 #8	22356	2.16	48288.96	203	3.49	708.47

Option 2

s (ft)	b (in)	h-t (in)	Vol. Conc (yd^3)	$/yd^3	Conc. $	Top Rebars	Top Rebars (lf)	$/lf	Top Rebar $
3	24	44	484	112.55	54474.2	2 #8	2916	2.16	6298.56
5	24	28	196	112.55	22059.8	2 #8	2916	2.16	6298.56
6	36	20	180	112.55	20259	2 #8	2916	2.16	6298.56
7.5	36	16	120	112.55	13506	2 #8	2916	2.16	6298.56
15	36	16	72	112.55	8103.6	2 #8	2916	2.16	6298.56

s (ft)	Bot Rebars	Bot Rebars (lf)	$/lf	Bot Rebar $	Stirrups	$/Stirrup	$ Stirrup
3	10 #11	17820	4.25	75735	81	3.49	282.69
5	12 #11	13608	4.25	57834	194	3.49	677.06
6	14 #10	13608	3.44	46811.52	139	3.49	485.11
7.5	16 #11	12960	4.25	55080	194	3.49	677.06
15	20 #11	9720	4.25	41310	278	3.49	970.22

Pier Design
Multi-Column

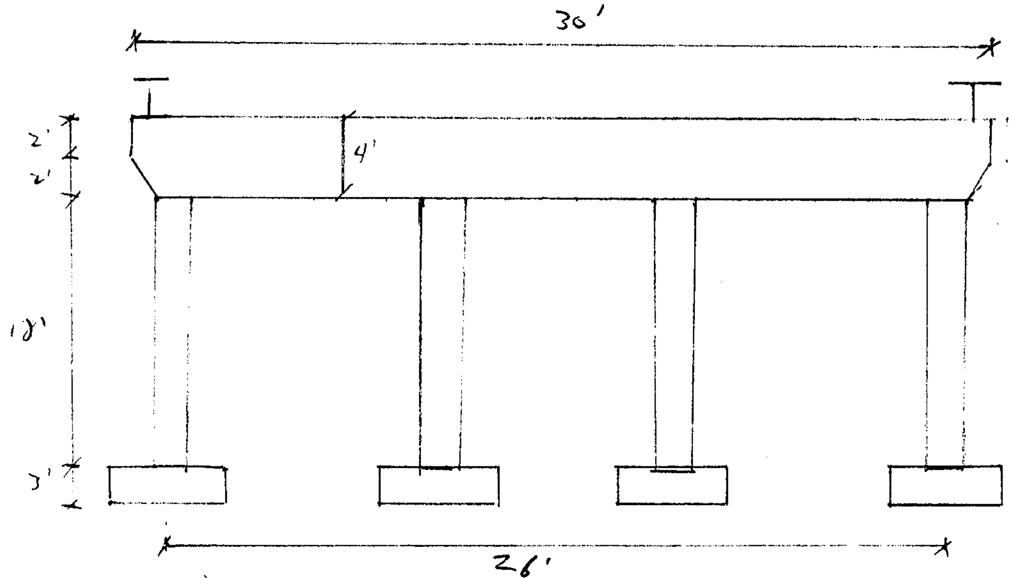
Use 0253 Composite Design

DL

$DL_{slab} = 8/12(30)(.15) = 3\ ^k/_{ft}$

$DL_{girder} = .16(11) = 1.76\ ^k/_{ft}$

$DL_{bracing} = .015\ ^k/_{ft}$



$W_{cap} = 4(4)(.15)(30) = 72^k$

$W_{column} = \pi(3.5/2)^2(.15)(18) = 25.977^k$ per column

$W_{footing} = 6(6)(.15)(3) = 16.2^k$ per footing

LL

$R_{lane} = 64.796^k$

$R_{truck} = 68.599^k$

$P_{wheel} = \frac{R_{truck}}{2}(1+IM)(.90) = \frac{68.599}{2}(1.33)(.90) = 41.057^k$

$W_{lane} = \frac{R_{lane}}{10'}(.90) = \frac{64.796}{10}(.9) = 5.832\ ^k/_{ft}$

w/ both lanes loaded
Max LL reaction = 48.755^k
w/ only lane 2 loaded
Max LL reaction = 48.307^k

both @ girder 6

Braking Force

greatest of:
25% (axle weight of truck or design tandem)
5% (design truck plus lane load)
5% (design tandem plus lane load)

$.25(72)4(.65) = 46.8^k$ ← governs

$.05[72 + 162(.64)] = 8.78^k$

$.05[(25+25) + 162(.64)] = 7.68^k$

$\frac{46.8}{11} = 4.25^k$ per girder   For Torsion DF = $\frac{46.8}{5} = 5.36^k$

Applied 6' above road surface
Moment Arm = 6' + slab thick + girder depth
= 6 + 8/12 + 3 = 9.67'
M = 2(4.25)(9.67) = 81.17 ft-k @ each girder

WL

Check aeroelastic instability

$\frac{L}{W} = \frac{81}{30} = 2.7 < 30$ ✓

$\frac{L}{d} = \frac{81}{3.67} = 22.1 < 30$ ✓

Superstructure

$H_{super} = 3.67'$ $L_{windT} = 81'$ $L_{windL} = 162'$

$A_{wsuperT} = 3.67(81) = 297.27\ ft^2$

$A_{wsuperL} = 3.67(162) = 594.54\ ft^2$

Assume $V_B = 100$ mph

Design wind pressure $P_D = P_B\left(V_{DZ}/V_B\right)^2 = P_B\left(\frac{V_{DZ}^2}{10000}\right)$

H < 30' so assume $V_{DZ} = 100$ mph

$P_D = P_B$

Wind load Transverse

$F_{Tsuper} = P_B A_{wsuperT}$ = .05(297.27) = 14.86^k (0°)
.044(297.27) = 13.08^k (15°)
.041(297.27) = 12.19^k (30°)
.033(297.27) = 9.81^k (45°)
.017(297.27) = 5.05^k (60°)

Wind load Longitudinal

$F_{Lsuper} = P_B A_{wsuperL}$ = 0(594.54) = 0 (0°)
= .006(594.54) = 3.57^k (15°)
.012(594.54) = 7.13^k (30°)
.016(594.54) = 9.51^k (45°)
.019(594.54) = 11.30^k (60°)

P_B values from Table 3.8.1.2.1-1 from AASTO

No Skew so

$F_{Lpier} = F_{Lsuper} = 11.30^k$ $F_{Tpier} = F_{Tsuper} = 14.86^k$

Acts at $\frac{3.67}{2} = 1.83'$

Substructure

$W_{sub} = W_{cap} + W_{column}$

$W_{capT} = .04(\text{cap width}) = .04(4) = .16$ k/ft of height

$W_{capL} = .04(\text{cap length}) = .04(30) = 1.2$ k/ft of height

$W_{column\,T} = .04\left(\frac{\phi}{n}\right) = .04\left(\frac{3.5}{4}\right) = .035$ k/ft of height

$W_{column\,L} = .04(\phi) = .04(3.5) = .14$ k/ft of height

$W_{sub\,T} = .16 + .035 = .195$ k/ft

$W_{sub\,L} = 1.2 + .14 = 1.34$ k/ft

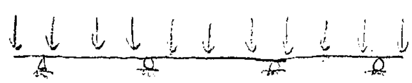
on Live load

$F_{LL_T} = F_{WT} \frac{L}{2}$	$= .100(81) = 8.10^k$	$(0°)$
	$.088(81) = 7.13^k$	$(15°)$
	$.082(81) = 6.64^k$	$(30°)$
	$.066(81) = 5.35^k$	$(45°)$
	$.034(81) = 2.75^k$	$(60°)$
$F_{LL_L} = F_{WL}\,L$	$= 0(162) = 0$	$(0°)$
	$.012(162) = 1.94^k$	$(15°)$
	$.024(162) = 3.89^k$	$(30°)$
	$.032(162) = 5.18^k$	$(45°)$
	$.038(162) = 6.16^k$	$(60°)$

F_w values from Table S3.8.1.3-1 AASHTO

$F_{WLL} = 8.10^k$

Due to symmetry no temp or shrinkage force



① $DL_{super} = 4.775$ k/ft $(81) = 386.775^k = 35.161$ per girder

② $LL = 48.755^k$ per girder

③ Brake force $= 81.17$ ft-k @ each girder

④ Max $WL_{super} = \dfrac{F_{T_{super\,0°}}\left(\frac{H}{2}\right)\cdot 15}{2(15)^2 + 2(3)^2} = .874^k$ per girder $= 1.539^k$

$= \dfrac{F_{L_{super\,60°}}\left(\frac{H}{2}\right)\cdot 15}{2(15)^2 + 2(3)^2} = .665^k$ per girder

⑥ Max $WL_{sub} = .16(4) + .035(18) = 1.27^k$ ⟩ apply to pier

$= 1.2(4) + .14(18) = 7.32^k$

⑤ Max $WL_{LL} = \dfrac{8.10\left(\frac{3.67}{2}\right)(15)}{2(15)^2 + 2(3)^2} = .476^k$ per girder

For Strength I

$M_u^+ = 400.592$ ft-k $\qquad M_u^- = 267.683$ ft-k

$V_u = 278.003^k$

$T_u = 2(1.75\ldots) = 2(1.75)(4.25) = 14.86$ ft-k

For Service I

$M_u = 195.863$ ft-k

	1	2	3	4
Reactions @ columns	407.275^k	338.463^k	414.475^k	261.754^k

$f'_c = 4$ ksi $\quad f_y = 60$ ksi $\quad \beta_1 = .85 \quad$ Stirrups are #5 bars

cap width = 4' $\quad$ cap depth = 4' $\quad$ cover = 2"

$\boxed{\text{Pos}}$

Moment from column weight $= \dfrac{4(4)(.15)(26/3)^2}{8} = 22.533$ ft-k

$M_u = 400.592 + 22.533 = 423.125$ ft-k

$A_s = \dfrac{M_u}{\phi f_y jd} \qquad jd = .875d = .875(46) = 40.25" \qquad \phi = .9$

$= \dfrac{423.125(12000)}{.9(60000)(40.25)} = 2.34$ in^2

Try 12 No 4 bars $\qquad A_s = 2.40$ in$^2 \qquad$ Diameter $= .500"$

$M_r = \phi M_n$

$M_n = A_s f_y (d - a/2) \qquad d =$ cap depth $- CSG_t$

$CSG_t =$ cover + stirrup diameter + ½ bar diameter

$= 2 + .625 + ½(.5)$

$= 2.875"$

$d = 48 - 2.875 = 45.125"$

$a = \dfrac{A_s f_y}{.85 f'_c b} = \dfrac{2.40(60000)}{.85(4000)(48)} = 0.882"$

$M_n = 2.40(60000)(45.125 - .882/2) = 536.208$ ft-k

$M_r = .9(536.208) = 482.587$ ft-k $> M_u$ ✓

Check Over-reinforcement

$c/d_e \le 0.42$

$c = a/\beta_1 = \dfrac{.882}{.85} = 1.038 \qquad d_e = d = 45.125"$

$c/d_e = \dfrac{1.038}{45.125} = .0230 < .42$ ✓

Check min reinforcement

$1.2 M_{cr} = 1.2 f_r S$

$f_r = .24\sqrt{f'_c} = .24\sqrt{4} = .48$ ksi

$S = bh^2/6 = 48(48)^2/6 = 18432$ in^3

$1.2 M_{cr} = \dfrac{1.2(.48)(18432)}{12} = 884.736 > M_r$ X

need additional reinforcement for cracking

$A_s = \dfrac{884.736\,(12000)}{.9(60000)(40.25)} = 4.88 \text{ in}^2$ Try 12 No. 6 bars $A_s = 5.28 \text{ in}^2$

$CSG_r = 2 + .625 + \frac{1}{2}(.750) = 3''$
$\delta = 48 - 3 = 45''$

$a = \dfrac{5.28(60)}{.85(4)(48)} = 1.64$

$\phi M_n = M_u = .9(5.28)(60000)(45 - 1.94/2) = 1046.360$ ft-k

Check over-reinf
$c = \dfrac{1.64}{.85} = 2.25$
$c/d_e = \dfrac{2.25}{45} = .05 < .42$ ✓

Check min reinf
$M_n = 1046.360 \text{ ft-k} > 884.736 = M_{CR}$ ✓

Check dist.
$f_{s\,allow} = \dfrac{z}{(d_c A)^{1/3}} \le 0.6 f_y$ $z = 170$ k/in, $d_c = 2 + \frac{1}{2}(.75) = 2.375''$

$A = \dfrac{2 d_c\,(\text{top width})}{n_{bars}} = \dfrac{2(2.375)(48)}{12} = 19 \text{ in}^2$

$f_{s\,allow} = \dfrac{170}{(2.375(19))^{1/3}} \le .6(60)$
$= 47.75 > 36$ ksi $\therefore f_{s\,allow} = 36$ ksi

Check service load
Trans steel area $= 5.28\left(\frac{36000}{3640}\right) = 42.24$
$42.24(45 - y) = 48y\,(y/2)$ $y = 8.06$
$I_{trans} = A_t(\delta - y)^2 + by^3/3$
$= 42.24(45 - 8.06)^2 + \frac{48}{3}(8.06)^3$

$= 66016.872 \text{ in}^4$

$f_{s\,actual} = \left(\dfrac{Mc}{I}\right)n = \left(\dfrac{423.125(12)(45 - 8.06 - 3)}{66016.872}\right)8$
$f_{s\,actual} = 31.32$ ksi < 36 ksi ✓

Neg

Since M_u is less than M_{CR}, M_{CR} will control reinforcement
Use same as for pos moment

$f_{s\,actual} = \left(\dfrac{290.216\,(12)(45 - 8.06 - 3)}{66016.872}\right)8 = 14.32$ ksi < 36 ksi ✓

Check Temp & shrinkage

$$A_{s\,min} = \frac{.11 A_g}{f_y} \qquad A_g = (4(12))^2 = 2304 \text{ in}^2$$

$$= \frac{.11(2304)}{60} = 4.22 \text{ in}^2$$

Try 4 No 7 bars per face

$$A_s = 4(0.6) = 2.4 \text{ in}^2 > \frac{4.22}{2} = 2.11 \text{ in}^2 \quad \checkmark$$

Skin Reinf

$$A_{sk} = .012(d_c - 30) \le (A_s + A_{ps})/4 \qquad A_{ps} = \text{Area prestress} = 0$$

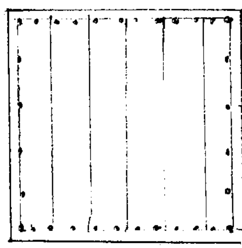
$$= .012(45-30) \le 5.28/4$$

$$= .18 \le 1.32$$

$$= .18(4)$$

$$= .72 \text{ in}^2 < 3.4 \text{ in}^2 \quad \checkmark$$

Cross-Section S. face



Shear Resistance

$$V_r = \phi V_n$$

$$V_c = .0316 \beta_1 \sqrt{f'_c}\, b_v d_v \qquad d_v = d - a/2 = 45 - \frac{1.94}{2} = 44.03$$

$$= .0316(.85)\sqrt{4}\,(48)(44.03)$$

$$= 113.53^k$$

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

$$V_n = \frac{V_u}{\phi} = \frac{278.003}{.75} = 370.761^k \qquad V_c/2 = \frac{113.53}{2} = 56.765^k$$

$$V_n > \frac{V_c}{2} \quad \text{need stirrups}$$

$$S = \frac{.62(60000)(45)}{(370.761 - 113.63)(1000)} = 6.51'' \quad \text{use } 6'' \text{ spacing}$$

$$S_{max} = \frac{.62(60000)}{.75(\sqrt{4000})(45)} = 17.4'' \qquad S_{max} = \frac{.62(60000)}{50(45)} = 16.5'' \leftarrow \checkmark$$

$$V_s = A_s f_y d_v / s = \frac{.62(60)(44.03)}{6} = 272.99^k$$

V_n = lesser of:

$$= V_c + V_s \quad \text{or} \quad .25 f'_c b_v d_v$$

$$= 113.53 + 272.99 \quad \text{or} \quad .25(4)(48)(44.03)$$

$$= 386.52^k \quad \text{or} \quad 2113.44^k$$

↑ governs

$\phi V_n = .75(386.52) = 289.89^k > V_u = 278.003^k$ ✓

Check Min Reinf

$$A_v = \frac{.0316\sqrt{f'_c}\, b_v S}{f_y} = \frac{.0316\sqrt{4}(48)(6)}{60} =$$

$$= .303 \text{ in}^2 < .62 \text{ in}^2 \quad ✓$$

$S_{max} = .4 d_v \le 12'' \quad = .4(44.03) \le 12'' \quad = 17.6'' \le 12''$

$S_{max} = 12'' > 6''$ ✓

Use 6'' stirrup spacing throughout

Column

Strength I
Axial Load: 819.875^k
Transverse Moment: $1.75(48.755)(2) = 170.642$ ft-k
Longitudinal Moment: $11(1.75(4.25)(22 + 5/12) \cdot 2 = 3667.927$ ft-k

Strength III Transverse Shear: $1.40(14.86 + .145) = 21.077^k$

Strength V Longitudinal Shear: $.4(11.30 + 1.34) + 1.00(6.16) + 1.35(11 \cdot 4.25)(3) = 200.554^k$

Try this setup
$\phi = 3.5''$ 2'' side cover
$A_g = 9.62$ ft^2
16 No 8 bars $A_s = 12.64$ in^2 (more than enough base on Tables A-12-A-18

Limits

$A_s/A_g \le .08 \qquad \frac{12.64}{9.62(144)} = .00912 \le .08$ ✓

Slenderness

$k = 1.2 \parallel, \; 2.1 \perp \qquad l_u =$ unbraced length $= 18'$
$r =$ radius of gyration $= 1/4 r = 1/4(3.5) = .875'$

$$\frac{k l_u}{r} \ge 22 \qquad \parallel \; \frac{1.2(18)}{.875} = 24.686' > 22 \quad ✓$$

$$\perp \; \frac{2.1(18)}{.875} \quad 43.2 > 22 \quad ✓$$

Slenderness effects do not need to be considered

Transverse Reinf

B/c used No 8 bars for longitudinal reinforcement, No 3 bar this will be sufficient. Spacing = least spacing of compression member. Use 12'' spacing.

$c_{cv} = .100$ $\quad \mu = 1.00$

$f_y = 60$ $\quad f'_c = 4$ $\quad \phi_v = .9$

$V_{nsf1} = c_{cv} A_g + \mu A_s f_y = .1(1385.28) + 1(12.64)(60) = 896.63^k$

$V_{nsf2} = .2 f'_c A_g = .2(4)(1385.28) = 1108.22^k$

$V_{nsf3} = .8 A_g (1) = .8(1385.28) = 1108.22^k$

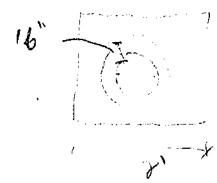
$V_{nsf} = V_{nsf1} = 896.63^k$ $\quad V_{u1} = 200.554^k$

$\phi V_{nsf} = .9(896.63) = 807.237^k > V_{u1}$ ✓

Footing

Net Soil Pressure $= \frac{414.475}{8(8)} = 6.48$ ksf

$d = 36 - 3 - 1 = 32"$ $\quad d/2 = 16"$



2-way

$V_u = 6.48\left(8^2 - \frac{74^2}{12}\right) = 168.3^k$

$b_o = 4(3.5(12) + 32) = 296"$ $\quad \beta_c = 1.0$ $\quad \phi = .75$ $\quad \alpha_s = 40$

$\phi V_c = \phi\left(2 + 4/\beta_c\right)\sqrt{f'_c}\, b_o d = \frac{.75(2+4)\sqrt{4000}(296)(32)}{1000} = 2695.78^k$

$\phi V_c = \phi\left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f'_c}\, b_o d = .75\left(\frac{40 \cdot 32}{296} + 2\right)\sqrt{4000}(296)(32) = 2841.50^k$

$\phi V_c = \phi(4)\sqrt{f'_c}\, b_o d = \frac{.75(4)(\sqrt{4000}(296)(32)}{1000} = 1797.14^k$

$\phi V_c = 1797.14^k > V_u = 168.3^k$ ✓

1-way

$V_u = 6.48\left(8 \times \frac{43}{12}\right) = 185.76^k$

$\phi V_c = \phi 2\sqrt{f'_c}\, b_w d = \frac{.75(2)(\sqrt{4000})(296)(32)}{1000} = 838.69^k$

$\phi V_c > V_u$ ✓

Moment (Top Reinf)

$M_u = 6.48\left(8 \times \left(\frac{27}{12}\right)^2 / 2\right) = 131.22$ ft-k

$A_s = \frac{M_u}{\phi f_y j d} = \frac{131.22(12000)}{.9(60000)(.9)(32)} = 1.01$ in^2

Min $A_s = .0018bh = .0018(96)(36) = 6.22$ in^2 ← governs

Max spacing = 18"

Try 15 No 6 bars each way $A_s = 6.6$ in^2

$$a = \frac{6.60(60000)}{.85(4000)(96)} = 1.21"$$

$a/d = \frac{1.21}{32} = .038 \ll a/d$ allowable from Table A-4 $\therefore \phi = .90$

$$\phi M_n = \frac{.9(6.6)(60000)(32 - 1.21/2)}{12000} = 932.43 > M_u \checkmark$$

Spacing $= (L - 2(\text{side cov.}) - d_{bar})/1 = 8 - \frac{2(3)}{12} - \frac{.75}{12} = 7.43" < S_{max} = 18" \checkmark$

Bottom mat'l

$M_u = (414.975)(4) - 648(8)(4) = 1450.88$ ft k

$$A_s = \frac{1450.88(12000)}{.9(60000)(.9)(32)} = 11.20 \text{ in}^2$$

Try 15 No 8 bars $A_s = 11.85$ in^2

$a = \frac{11.85(60000)}{.85(4000)(96)} = 2.18"$ $a/d \ll$ Table A-4 $\therefore \phi = .9$

$$\phi M_n = \frac{.9(11.85)(60000)(32 - 2.18/2)}{12000} = 1648.3 \text{ ft-k} > M_u \checkmark$$

Spacing $= 8 - \frac{2(3)}{12} - \frac{1}{12} = 7.42" < S_{max} \checkmark$

Check Max Reinf

$c/d_t \le .42$ $c = a/\beta_1 = 1.21/.85 = 1.42$

$\frac{1.42}{32} = .04 < .42 \checkmark$

$c = \frac{2.18}{.85} = 2.56$

$\frac{2.56}{32} = .08 < .42 \checkmark$

Cracking

$f_{s\,allow} = z/(d_c \times A)^{1/3} \le .6 f_y$

$z = 170$
$d_c = $ bot cov $+ \frac{1}{2}\phi$
$= 2 + \frac{1}{2}(1) = 2.5$
$A = 2 d_c s$
$= 2(2.5)(7.42) = 37.1$

$f_{s\,allow} = \frac{170}{(2.5 \times 37.1)^{1/3}} = 37.6 \ge 36$ use 36 ksi

Trans steel Area $= 1.01\left(\frac{36000}{3640}\right) = 9.99$ in^2

$9.99(32 - y) = 7.42y(y/2)$ $y = 8.03$ in

$I_{trans} = A_s(d - y)^2 + by^3/3 =$

$= 9.99(32 - 8.03)^2 + \frac{7.42(8.03)^3}{3} = 7020.51$ in^4

$f_{s\,actual} = \frac{Mc}{I} = \frac{932.43(32 - 8.03)}{7020.51}(9.99) = 31.8 < 36 \checkmark$

Dowels

Load $= 414.975^k$ Load transferred by bearing $= \phi(.85)(f'_c)A_1 = .65(.85)(4)(1.5\cdot12)(4)$
$= 3061.47^k$

$3061.47 > 414.75$ total load transferred by bearing
Dowels not needed.

Appendix H

Foundation Calculations

Shallow

Assume $\phi_s = 33°$ $\gamma = 120$ lbs/ft³ footing: 8x8
Bottom footing 6' below ground

$B' = 8 - 2(126/1374) = 7.82$ ft
$L' = 8 - 2(626/1774) = 7.09$ ft

$q_{nu} = 0.5\gamma B C_{wq} N_{qm} + \gamma C_{wq} D_f N_{\gamma m}$

$D_f = 6'$ $\gamma = 120/2000 = .06$
$B = B' = 7.82'$

$N_{\gamma m} = N_\gamma S_\gamma C_\gamma I_\gamma$ $N_{qm} = N_q S_q C_q I_q d_q$

$N_\gamma = 35$ for $\phi = 33°$ $N_q = 26$ for $\phi = 33°$

$L'/B' = 7.09/7.82 = .91 \approx 1.0$
$S_\gamma = 0.60$ for $L'/B' \approx 1.0$ $S_q = 1.64$ for $L'/B' \approx 1.0$

if $\xi = 0.5$; $\phi_f = 33°$
From Table 10.6.3.1.2c-5 $C_q, C_\gamma = 0.99$

Assume $H/V = 0.1$
$I_\gamma = 0.77$ $I_q = 0.85$

From Table 10.6.3.1.2c-4 $d_q = 1.20$

$N_{\gamma m} = 35(0.6)(0.99)(0.77)$
$= 16.01$

$N_{qm} = 26(1.64)(0.99)(0.85)(1.20)$
$= 44.37$

$q_{ult} = 0.5(.06)(7.82)(1.0)(44.37) + .06(1.0)(6)(16.01)$
$= 16.17$ TSF

$q_r = \phi q_n - \phi q_{ult}$
$= 0.35(16.17) = 5.66$ TSF

Footing Resistance $= q_r L'B'$
$= 5.66(7.09)(7.82)$
$= 313.78$ tons
$= 627.57^K > 414.475^K$ ✓

For shallow foundation footing is adequate

Deep

$P_u = \dfrac{q_t A_t + \Sigma f_s A_s}{F}$ $F = 3.0$ $P_u = 414.475^K$

Assume these soil conditions:
4 m medium clay $f_s = 25$ kPa
10 m silty sand $f_s = 100$ kPa
1 m glacial till $f_s = 800$ kPa
$q_t = 9000$ kPa

$$1843.685 = \frac{4000(\pi)r^2 + 25(4)(2\pi)(r) + 100(10)(2\pi)(r) + 800(1.0)(2\pi)r}{3}$$

$r = .34\ m = 13.4''$

Use 14" radius pile Diam = 28"

$14'' = .3556\ m$

$$P_u = \frac{4000\pi(.3556)^2 + 25(4)(2\pi)(.3556) + 100(10)(2\pi)(.3556) + 800(1.0)(2\pi)(.3556)}{3}$$

$P_u = 1944.7\ kN > 1843.68\ kN = 414.475^k$ ✓

Shallow

$$q_{ult} = c'N_c + \sigma'_{zD} N_q + 0.5\gamma' B N_\gamma$$

Assume $\gamma = 120$ pcf $D = 6'$ water table very much below foundation

$\phi = 38'$ $c' = 150$ psf

$q_{ult} = 150(48.1) + 120(6)(32.2) + 0.5(120)(12)(33.3)$

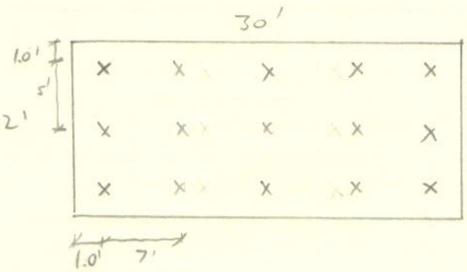
$q_{ult} = 32.347^k$

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

$$32.347 = \frac{P + 162}{12(30)} - 0 \qquad P = 11500^k > A_D = 1686.551^k$$

Therefore for shallow foundation situations the footing is sufficient.

Deep



15 total piles

$$P_u = 15\left(\frac{q_t' A_t + \Sigma f_s A_s}{F}\right)$$ w/ same conditions as for multi-leg pier

$$7502.153 = 15\left(\frac{4000\pi r^2 + 25(4)(2\pi)r + 100(10)(2\pi)r + 800(1)(2\pi)r}{3}\right)$$

$r = .112$ m $= 4.41''$

Use $r = 5''$ $D = 10''$

$5'' = .127$ m

$$P_u = 15\left(\frac{4000\pi(.127)^2 + 25(4)(2\pi)(.127) + 100(10)(2\pi)(.127) + 800(1)(.127)}{3}\right)$$

$P_u = 7682.0$ KN > 7502.153 kN $= 1686.551^k$ ✓

Appendix I

LCC Calculations

Pier LCC

Conversions

Initial
- Concrete 17500 $\text{yen}/m^3 \left(\frac{.01124\ \$}{1\ \text{yen}}\right)\left(\frac{1\ m^3}{35.3147\ ft^3}\right) = 5.570\ \$/ft^3$
- Form 3000 yen/m^3 = $.955\ \$/ft^3$
- Curing 500 yen/m^3 = $.159\ \$/ft^3$
- Rebar 90000 yen/ton = 1011.600 \$/metric ton
- Scaffolding 2000 yen/m^3 = $.637\ \$/ft^3$

Patching 10388 yen/m^2 = $10.847\ \$/ft^2$

Recovery/Repair

Multi-Leg
- Concrete: $1940.72\ ft^3$
- Rebar = 9255.048 lbs = 4.198 tons
- = $18.964\ ft^3$
- SA = $727.02\ ft^2$

Single-Leg
- Concrete = $2946\ ft^3$
- Rebar = 12002.802 lbs = 5.444 tons
- = $26.4749\ ft^3$
- SA = $1442\ ft^2$

Initial Cost Multi = \$17230.55
Repair Cost Multi = \$7885.99

Initial Cost Single = \$25215.08
Repair Cost Single = \$15641.37

Repair when chloride ion concentration at rebar reaches $\approx 1.2\ kg/m^3 = .0749\ lb/ft^3$
Depending on climate this takes 15 to 30 years.
Do estimate for both extremes and for LC of 50 & 100 yrs.

50 yr LCC Multi w/ 15 yr repair = \$40888.52
50 yr LCC Multi w/ 30 yr repair = \$25116.54
100 yr LCC Multi w/ 15 yr repair = \$64546.49
100 yr LCC Multi w/ 30 yr repair = \$40888.52

50 yr LCC Single w/ 15 yr repair = \$72139.11
50 yr LCC Single w/ 30 yr repair = \$40856.37
100 yr LCC Single w/ 15 yr repair = \$119063.22
100 yr LCC Single w/ 30 yr repair = \$72139.11

Interest Rate 3-5% use both
Present worth analysis

Multi-Column

$$PV = \frac{FV}{(1+i)^t}$$

	3%		5%	
15 yrs	$\frac{7900}{(1+.03)^{15}}$ =	5071	$\frac{7900}{(1+.05)^{15}}$ =	3800
30 yrs	$\frac{7900}{(1+.03)^{30}}$ =	3255	$\frac{7900}{(1+.05)^{30}}$ =	1828
45 yrs	$\frac{7900}{(1+.03)^{45}}$ =	2089	$\frac{7900}{(1+.05)^{45}}$ =	879
60 yrs	$\frac{7900}{(1+.03)^{60}}$ =	1341	$\frac{7900}{(1+.05)^{60}}$ =	423
75 yrs	$\frac{7900}{(1+.03)^{75}}$ =	861	$\frac{7900}{(1+.05)^{75}}$ =	203
90 yrs	$\frac{7900}{(1+.03)^{90}}$ =	552	$\frac{7900}{(1+.05)^{90}}$ =	98

Single Leg

15 yrs	$\frac{15700}{(1.03)^{15}}$ =	10077	$\frac{15700}{(1.05)^{15}}$ =	7552
30 yrs	$\frac{15700}{(1.03)^{30}}$ =	6468	$\frac{15700}{(1.05)^{30}}$ =	3633
45 yrs	$\frac{15700}{(1.03)^{45}}$ =	4152	$\frac{15700}{(1.05)^{45}}$ =	1747
60 yrs	$\frac{15700}{(1.03)^{60}}$ =	2665	$\frac{15700}{(1.05)^{60}}$ =	841
75 yrs	$\frac{15700}{(1.03)^{75}}$ =	1710	$\frac{15700}{(1.05)^{75}}$ =	404
90 yrs	$\frac{15700}{(1.03)^{90}}$ =	1098	$\frac{15700}{(1.05)^{90}}$ =	194

Abutments
Present worth

Initial Cost: 10746.68 + 30.61 + 28669.80 = 39447.09

Repair: 12100 Interest 3 - 5%

	3%	5%
15 yr	$\frac{12100}{(1.03)^{15}} = 7767$	$\frac{12100}{(1.05)^{15}} = 5820$
30 yr	$\frac{12100}{(1.03)^{30}} = 4985$	$\frac{12100}{(1.05)^{30}} = 2800$
45 yr	$\frac{12100}{(1.03)^{45}} = 3200$	$\frac{12100}{(1.05)^{45}} = 1347$
60 yr	$\frac{12100}{(1.03)^{60}} = 2054$	$\frac{12100}{(1.05)^{60}} = 648$
75 yr	$\frac{12100}{(1.03)^{75}} = 1318$	$\frac{12100}{(1.05)^{75}} = 312$
90 yr	$\frac{12100}{(1.03)^{90}} = 846$	$\frac{12100}{(1.05)^{90}} = 150$

Appendix J

Abutment Design

Dead Load Effects

Girder

$$R_{DC} = 35.161^k$$

$$R_{DC\,\cancel{tot}} = \frac{(35.161^k)(11)}{30'} = \boxed{12.89\ ^k/_l}$$

Backwall

$$DL_{BW} = [(7')(3.5') - (7')(1.5')](0.15) = \boxed{2.1\ ^k/_l}$$

Stem

$$DL_{stem} = (22')(3.5')(.15) = \boxed{11.55\ ^k/_l}$$

Footing

$$DL_{ftg} = (17')(3')(.15) = \boxed{7.65\ ^k/_l}$$

Earth

$$DL_{earth} = (8')(29')(0.120) = \boxed{27.84\ ^k/_l}$$

Live Load Effects

Dynamic Load Allowance = IM = 0.33

Multiple presence factors: m_1 (1 lane) = 1.20

m_2 (2 lanes) = 1.00

$$RLL_{,bw} = \frac{\overbrace{2}^{\#\text{ of axles}}(\overbrace{32^k}^{\text{truck load}})\cdot(1+\overbrace{IM}^{.33}) + 2(\overbrace{0.64\ ^k/_l}^{\text{lane load}})(\overbrace{2'}^{\text{length}})}{(L_{abut} = 30')} = \boxed{2.92\ ^k/_l}$$

The following loads are obtained from girder design software output for one lane loaded and they are applied at the beam seat or top of abutment stem for the stem design.

$V_{vehmax} = 65^k$ $\qquad V_{lanemax} = 33.5^k$

$V_{vehmin} = -7.5^k$ $\qquad V_{lane\ min} = -5.5^k$

- Max unfactored LL used for abutment stem design:

$$rLL_{max} = V_{vehmax}(1+IM) + V_{lane\ max} = 120^k$$ for one lane

$$RLL_{max} = \frac{2(m_2)(rLL_{max})}{L_{abut}} = \frac{2(1)(120)}{30} = \boxed{8\ ^k/_l}$$

- Min unfactored LL representing uplift used for abutment stem design:

$$rLL_{min} = -15.5^k$$

$$RLL_{min} = \boxed{-1.03\ ^k/_l}$$

The following loads are applied at the beam seat or top of abutment stem for the footing design.

- Max unfactored LL used for abutment footing design:

$$rLL_{max1} = V_{veh\,max} + V_{lane\,max} = 65 + 33.5 = 98.5^k$$ for one lane loaded

$$RLL_{max1} = \frac{2(1)(98.5^k)}{30} = \boxed{6.57\ k/_{l}}$$

- Min unfactored LL for footing design:

$$rLL_{min1} = -7.5 - 5.5 = -13^k$$ for one lane loaded

$$RLL_{min1} = \frac{2(1)(-13^k)}{30} = \boxed{-0.867\ k/_{l}}$$

Wind Load on Abutment (substructure)

$A_{wsubend} = (3.5')(29') = 101.5\ ft^2$ $\Rightarrow$ (Abutment exposed end elevation wind Area)

Wind pressure = 0.040 ksf

for wind attack angle of 0°

$WS_{subtransend\ 0} = (A_{wsubend})(0.040\ ksf)(\cos 0°) = (101.5\ ft^2)(0.040\ ksf)(1) = \boxed{4.06^k}$

$WS_{sublongend\ 0} = (A_{wsubend})(0.040\ ksf)(\sin 0°) = 0^k$

for a wind attack angle of 60°

$WS_{subtransend\ 60} = (101.5)(0.040)(\cos 60°) = 2.03^k$

$WS_{sublongend\ 60} = (101.5)(0.040)(\sin 60°) = 3.52^k$

Wind Loads

Abutment Design Wind Loads From Superstructure		
Wind Attack Angle	Bridge transverse Axis	Bridge* Longitoudinal Axis
Degrees	$WS_{supertrans}$ = (kips)	kips
0	9.93	0
15	6.54	0.8925
30	6.07	1.7825
45	4.09	2.3775
60	2.25	45.2
* Provided But Not Aplicable due to Expansion Bearings at Abutment		

Design Vehicular Wind Loads		
Wind Attack Angle	Bridge Transverse Axis	Bridge* Longitoudinal Axis
Degrees	WL_{trans} = (kips)	kips
0	4.05	0
15	3.565	0.485
30	3.32	0.9725
45	2.675	1.295
60	1.375	1.54
Provided but not applicable due to expansion bearings at abutment		

Vertical Wind Load $= (WL_{vert})(W_{deck}) = (0.02\ ksf)(30) = \boxed{0.6\ k/l}$

Earth Loads

Minimum Support length required

$N = (8 + 0.02L + 0.08H)(1 + 0.000125 S^2)$; $L = 162'$, $H = 29'$, $S = 0$

$N = 13.56'$ USE $\boxed{14' = N}$

$\phi' = 32°$, $K_a = \dfrac{1 - \sin 32°}{1 + \sin 32°} = 0.3$, $\gamma_s = 0.120$ kcf (average of loose and compac gravel)

backwall height $= z = 7'$

$p = K_a \gamma_s z = (0.3)(0.120)(7) = \boxed{0.252 \text{ ksf}}$ → earth pressure

lateral load due to the earth pressure (acts @ H/3 from the bottom of the section investigated)

$h_{backwall} = z = 7'$

* $R_{EH\,bw} = \frac{1}{2} p\, h_{backwall} = \frac{1}{2}(0.252 \text{ ksf})(7) = \boxed{0.882 \text{ k/}}$

bottom of the abutment stem lateral earth load:

$K_a = .3$, $\gamma_s = .120$ kcf , $z = 29'$ (height used for max moment at bottom of abut. stem)

$p = K_a \gamma_s z = (.3)(.120)(29) = 1.044$ ksf

* $R_{EH\,stem} = \frac{1}{2} p\, h_{stem} = \frac{1}{2}(1.044)(29) = \boxed{15.14 \text{ k/}}$

Bottom of footing lateral earth load ($K_a = .3$, $\gamma_s = .120$ kcf , $z = 32'$)

$p = (.3)(.120)(32) = 1.152$ ksf

* $R_{EH\,ftg} = \frac{1}{2}(1.152)(32) = \boxed{18.43 \text{ k/}}$

Loads due Live Load surcharge ($K_a = .3$, $\gamma_s = 0.120\ kcf$)

$\Delta p = K \gamma_s h_{eq}$ (horizontal pressure increase due to LL surcharge)

Backwall

$h_{eq} = 3.6'$ (equivalent height of soil for vehicular loading based on 7' backwall height ($H = 7'$))

$\Delta p = (.3)(.12)(3.6) = 0.130\ ksf$

* $R_{LS,bw} = \Delta p\, h_{backwall} = (.130)(7) = \boxed{0.91\ k/l}$

Bottom of Abutment stem ($H_{stem} = 29'$)

$h_{eq} = 2.0$ for $H \geq 20'$ Table 3.11.6.4-1 (AASHTO)

$\Delta p = (.3)(.12)(2) = 0.072\ ksf$

* $R_{LS,stem} = (0.072)(29) = \boxed{2.088\ k/l}$

Bottom of footing

$h_{eq} = 2\ ft$

$\Delta p = 0.072\ ksf$

* $R_{LS,ftg} = (0.072)(32') = \boxed{2.304\ k/l}$

Loads due to uniform surcharge

It was assumed that the approach slab and roadway will cover the abutment backfill material. Therefore, no uniform load will be applied.

Loads due to temperature (Temp range from 120°F - 0°F),

Expansion

$\varepsilon = 6.5 \times 10^{-6}$ in/in/°F

$t_{set} = 68°F$

$\Delta t_{rise} = 120° - 68° = 52°F$

$$\Delta_{expansion} = \varepsilon \Delta t_{rise} \left(L_{span} \cdot 12''/'\right) = (6.5\times10^{-6})(52)(81)(12) = \boxed{0.328''}$$

contraction

$\Delta t_{fall} = \Delta t_{set} - 0°F = 68°F - 0°F = 68°F$

$$\Delta_{contraction} = (6.5\times10^{-6})(68)(81)(12) = \boxed{0.430''}$$

$H_u = G \cdot A \cdot \frac{\Delta}{h_{rt}}$; $G = 0.095$ ksi (Shear modulus)

$A = 14'' \times 15'' = 210\ in^2$ (Area of bearing pad in plan view)

$h_{rt} = 3.5''$ (elastomer thickness)

Load due to temp rise

$$H_{u,rise}\ G A \frac{\Delta_{exp}}{h_{rt}} = (0.095)(210)\left(\frac{0.328}{3.5}\right) = \boxed{1.87^k} \text{ per bearing}$$

$$H_{u,rise\ TOTAL} = \frac{(H_{u,rise})(\#\text{of Bearings})}{L_{abut}} = \frac{(1.87)(11)}{30'} = \boxed{0.686\ k/'}$$

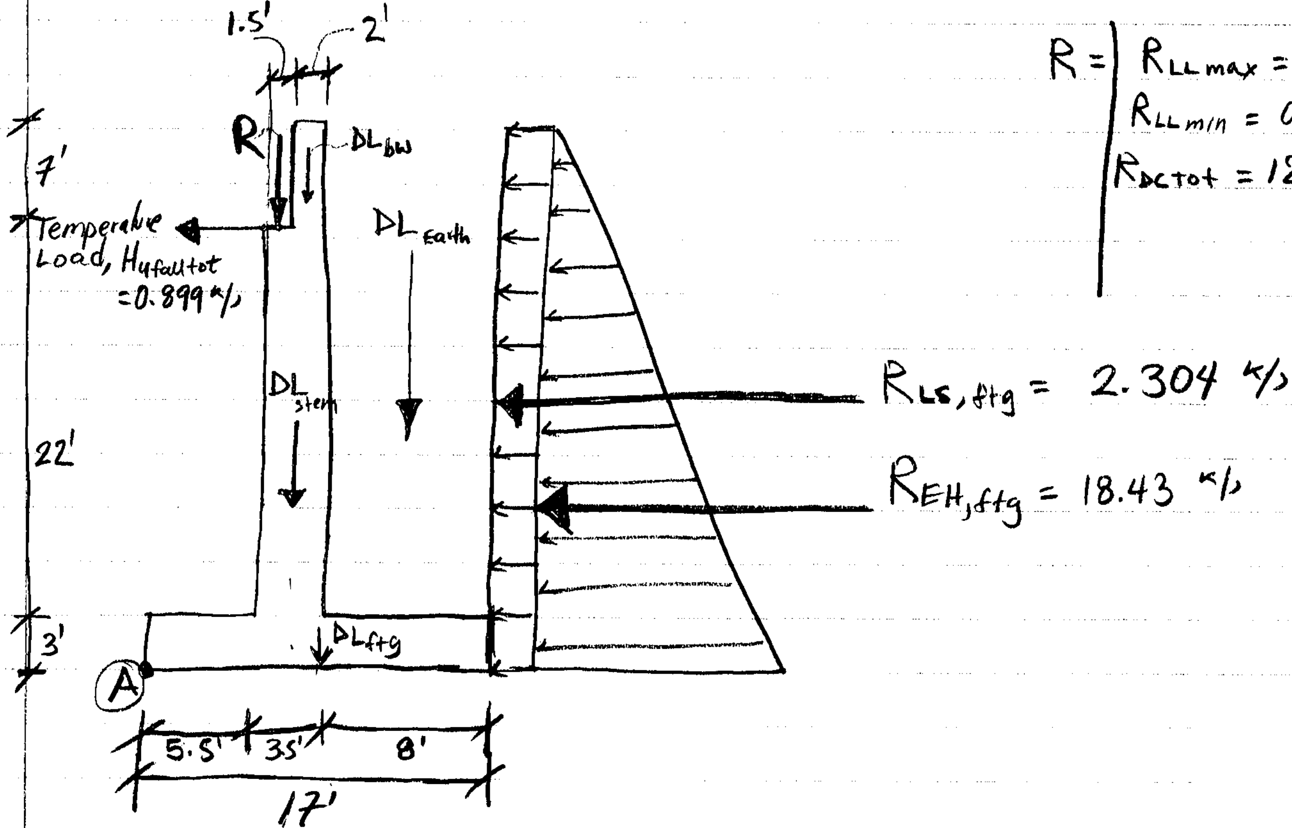
Load due to temperature fall

$$H_{u,fall} = (0.095)(210)\left(\frac{.430}{3.5}\right) = \boxed{2.451^k} \text{ per bearing}$$

$$H_{u,fall\ TOTAL} = \frac{(2.451)(11)}{30} = \boxed{0.899\ k/'}$$

Sliding & Overturning

$R = \begin{cases} R_{LL\,max} = 6.57 \text{ k/l} \\ R_{LL\,min} = 0.867 \text{ k/l} \\ R_{DC\,tot} = 12.89 \text{ k/l} \end{cases}$



$DL_{Earth} = 27.84$ k/l
$DL_{bw} = 2.1$ k/l
$DL_{stem} = 11.55$ k/l
$DL_{ftg} = 7.65$ k/l

Check for Overturning

Overturning Moment

$$M_1 = (R_{EH,ftg})\left(\frac{32}{3}\right)(\gamma_{EH}) = (18.43)\left(\frac{32}{3}\right)(1.5) = \boxed{294.88 \ \frac{k\cdot}{}}$$

$$M_2 = (R_{LS,ftg})\left(\frac{32}{2}\right)(\gamma_{LS}) = (2.304)\left(\frac{32}{2}\right)(1.75) = \boxed{64.51 \ \frac{k\cdot}{}}$$

$$M_3 = (H_{upsalltot})(25')(\gamma_{TU}) = (0.899)(25)(0.5) = \boxed{11.24 \ \frac{k\cdot}{}}$$

$$\text{Total} = \boxed{370.63 \ \frac{k\cdot}{}}$$

Resisting Moment

$$M_4 = (DL_{Earth})(12.5')(\gamma_{DC}) = (27.84)(13')(1.25) = \boxed{452.4 \ \frac{k\cdot}{}}$$

$$M_5 = (DL_{bw})(8')(\gamma_{DC}) = (2.1)(8')(1.25) = \boxed{21 \ \frac{k\cdot}{}}$$

$$M_6 = (DL_{stem})(7.25')(\gamma_{DC}) = (11.55)(7.25)(1.25) = \boxed{104.67 \ \frac{k\cdot}{}}$$

$$M_7 = (DL_{ftg})(8')(\gamma_{DC}) = (7.65)(8.5)(1.25) = \boxed{81.28 \ \frac{k\cdot}{}}$$

$$M_8 = (R_{DC\,Tot})(6.25')(\gamma_{DC}) = (12.89)(6.25)(1.25) = \boxed{100.703 \ \frac{k\cdot}{}}$$

$$M_9 = (R_{LL\,min})(6.25)(\gamma_{LL}) = (0.867)(6.25)(1.75) = \boxed{9.48 \ \frac{k\cdot}{}}$$

$$\text{Total} = \boxed{769.53 \ \frac{k\cdot}{}}$$

$$FS_{overturning} = \frac{769.53 \ \frac{k\cdot}{}}{370.63 \ \frac{k\cdot}{}} = 2.08 > 2.00 \qquad \underline{\underline{OK}}$$

Check for Sliding

Forces causing Sliding

$F_1 = (R_{EH,ftg}) = 18.43\ k/l$

$F_2 = (R_{LS,ftg}) = 2.304\ k/l$

$F_3 = (H_{u,falltot}) = 0.899\ k/l$

$\boxed{Total = 21.633}$

Forces Resisting Sliding

$F_4 = (DL_{Earth}) = 27.84\ k/l$

$F_5 = (DL_{bw}) = 2.1\ k/l$

$F_6 = (DL_{stem}) = 11.55\ k/l$

$F_7 = (DL_{ftg}) = 7.65\ k/l$

$F_8 = (R_{DC,tot}) = 12.89\ k/l$

$F_9 = (R_{LL\,min}) = 0.867\ k/l$

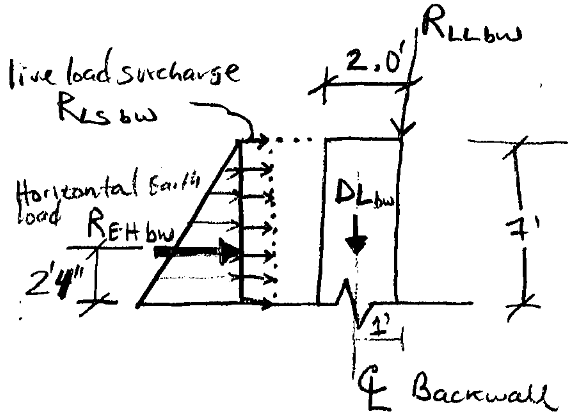
$\boxed{Total = 62.897\ k/l}$

$$FS_{sliding} = \frac{(62.897)(0.55)}{21.633} = \boxed{1.6} > 1.5$$

OK

BACKWALL

Analyze and combine force effects



- **Bottom of abutment backwall**

$DL_{bw} = 2.1\ k/ft$ $\qquad R_{EHbw} = 0.882\ k/ft$

$R_{LL,bw} = 2.92\ k/ft$ $\qquad R_{LSbw} = 0.91\ k/ft$

- Abutment Backwall Strength I force effects

$\gamma DC = 1.25$

$\gamma LL = 1.75$

$\gamma EH = 1.50$

$\gamma LS = 1.75$

- Factored vertical force at the base of the backwall:

$$\oplus\ F_{vbwstrI}\ \gamma DC \cdot DL_{bw} + \gamma LL\ R_{LLbw}$$
$$(1.25)(2.1) + (1.75)(2.92) = \boxed{7.735\ k/ft}$$

- Factored longitudinal shear force at the base of the backwall:

$$\oplus\ V_{ubwstrI} = (\gamma EH)(R_{EHbw}) + (\gamma LS)(R_{LSbw})$$
$$= (1.5)(0.882) + (1.75)(0.91) = \boxed{2.91\ k/ft}$$

- Factored moment at the base of the backwall:

$$\oplus\ M_{ubwstrI} = (\gamma LL)(R_{LLbw})(1') + (\gamma EH)(R_{EHbw})(2.33') + (\gamma LS)(R_{LSbw})\left(\tfrac{7'}{2}\right)$$
$$= (1.75)(2.92)(1) + (1.50)(0.882)(2.33) + (1.75)(0.91)(3.5)$$
$$= \boxed{13.77\ \tfrac{k\cdot ft}{ft}}$$

Abutment backwall strength III force effects:

$\gamma_{DC} = 1.25$ $DL_{bw} = 2.1 \, k/l$

$\gamma_{LL} = 0.00$ $R_{LL\,bw} = 2.92 \, k/l$

$\gamma_{EH} = 1.50$ $R_{EH\,bw} = 0.882 \, k/l$

$\gamma_{LS} = 0.00$ $R_{LS\,bw} = 0.91 \, k/l$

- Factored vertical force at the base of the backwall

$$\oplus \; F_{v\,bw\,str\,III} = \gamma_{DC} \cdot DL_{bw} + \cancel{\gamma_{LL}\,R_{LL\,bw}}^{\,0}$$
$$= (1.25)(2.1) = \boxed{2.625 \tfrac{k}{l}}$$

- Factored longitudinal shear force at the base of the backwall

$$\oplus \; V_{u\,bw\,str\,III} = (\gamma_{EH})(R_{EH\,bw}) + \cancel{(\gamma_{LS})}^{\,0}(R_{LS\,bw})$$
$$= (1.5)(0.882) = \boxed{1.32 \, k/l}$$

- Factored moment at the base of the backwall:

$$\oplus \; M_{u\,bw\,str\,III} = \cancel{(\gamma_{LL})}^{\,0}(R_{LL\,bw})(Arm) + (\gamma_{EH})(R_{EH\,bw})(2.33') + \cancel{(\gamma_{LS})}^{\,0}(R_{LS\,bw})(Arm)$$
$$= (1.50)(0.882)(2.33') = \boxed{3.08 \tfrac{k \cdot l}{l}}$$

• Abutment backwall strength V

$\gamma_{DC} = 1.25$, $\gamma_{LL} = 1.35$, $\gamma_{EH} = 1.50$, $\gamma_{LS} = 1.35$

- Factored vertical force = $\underline{F_{v\,bw\,str\,V}} = (\gamma_{DC})(DL_{bw}) + (\gamma_{LL})(R_{LL\,bw})$
$$= (1.25)(2.1) + (1.35)(2.92) = \boxed{6.57 \, k/l}$$
- Factored longitudinal shear force = $\underline{V_{u\,bw\,str\,V}} = (\gamma_{EH})(R_{EH\,bw}) + (\gamma_{LS})(R_{LS\,bw})$
$$= (1.5)(0.882) + (1.35)(.91) = \boxed{2.55 \, k/l}$$
- Factored moment = $M_{u\,bw\,str\,V}$ =
$$= (1.35)(2.92)(1') + (1.50)(.882)(2.33') + (1.35)(0.91)(3.5')$$
$$= \boxed{11.32 \tfrac{k \cdot l}{l}}$$

- **Abutment backwall service I force effects**

$\gamma_{DC} = \gamma_{LL} = \gamma_{EH} = \gamma_{LS} = 1.0$

- Factored vertical force: $F_{Vbwserv I} = (2.1) + (2.92) = \boxed{5.02 \text{ k/l}}$

- Factored longitudinal shear = $V_{ubwserv I} = (.882) + (.91) = \boxed{1.79 \text{ k/l}}$

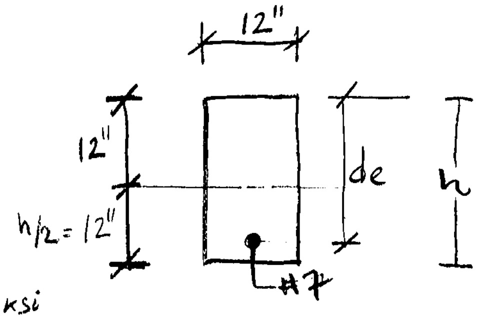
- Factored Moment = $M_{ubwserv I} = (2.92)(1) + (.882)(2.33) + (.91)(3.5) = \boxed{8.16 \frac{\text{k-l}}{\text{l}}}$

- Maximum factored backwall vertical force, shear force, and moment for the strength limit state are:

    - $F_{Vbwmax} = 7.735 \text{ k/l}$

    - $V_{ubwmax} = 2.91 \text{ k/l}$

    - $M_{ubwmax} = 13.77 \frac{\text{k-l}}{\text{l}}$



Abutment Backwall Design

Design for flexure

Cracking strength

$M_{cr} = f_r \frac{I_g}{y_t}$; $y_t = \frac{h}{2} = 12"$; $f_r = 0.24\sqrt{f'_c} = 0.48\,ksi$

$I_g = \frac{bh^3}{12} = \frac{12}{12}(24)^3 = 13{,}824\,in^4$

$M_{cr} = \boxed{46.08 \frac{k\cdot'}{'}} \Rightarrow \boxed{1.2(M_{cr}) = 55.3 \frac{k\cdot'}{'}}$

$M_{ubwmax} = 13.77 \frac{k\cdot'}{'} \Rightarrow \boxed{1.33(M_{ubwmax}) = 18.31 \frac{k\cdot'}{'}}$ → controls the minimum backwall reinforcem

M_{ubwdes} = controlling moment $= \boxed{18.31 \frac{k\cdot'}{'}}$

Assuming #7 bars for reinforcement, $A_{s\,bar} = 0.6$, diam = 0.875

Effective depth $d_e = (t_{bw} = h) - (\text{cover} = 2") - \frac{1}{2}\text{bar diam}$

$= 24 - 2" - .4375" = \boxed{21.56 = d_e}$

Reinforcing steel Rqd: $\phi_f = 0.9$, $b = 12"$

$R_n = \frac{M_{ubwdes}}{\phi b d^2} = \frac{(18.31 \frac{k\cdot'}{'})(12")}{(0.9)(12")(21.56\,in^2)} = \boxed{0.0438\ ^{k}/in^2}$

$\rho = 0.85 \frac{f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85 f'_c}}\right) = \boxed{0.000734}$

$A_{s_1} = \rho b d_e = 0.19\ in^2/ft \Rightarrow$ reqd spacing $= \frac{A_{s\,bar}}{A_{s_1}}(12"/_1) = 38"$

$\boxed{\text{USE \# 7 @ 12"}}$

$A_{s_2} = A_{s\,bar}\left(\frac{12"}{12"}\right) = 0.6\ in^2$ per foot

CHECK

$T = A_{s_2} f_y = 36^k$, $a = \frac{T}{0.85 f'_c b} = 0.882"$, $c = \frac{a}{\beta_1} = 1.03$

$\frac{c}{d_e} = 0.048 < 0.42$ OK

Check crack control

Assumin the backwall will be exposed to deceiting salts, $Z = 130\ ^{k}/_{in}$

Thickness of clear cover

$$d_c = 2'' + \frac{\text{bar diam}}{2} = \boxed{2.437''}$$

Concrete area

$$A_c = 2(d_c)(\text{bar-space}) = 58.5\ in^2$$

Allowable reinforcement service load stress for crack control

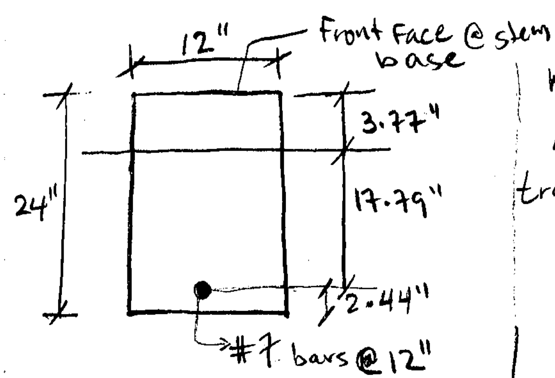
$$f_{sa} = \frac{Z}{(d_c A_c)^{1/3}} = 24.885\ ksi < 0.6 f_y = 36\ ksi \qquad \text{OK}$$

USE → $f_{sa} = 24.885$, $E_s = 29{,}000\ ksi$, $E_c = 3640\ ksi$, $n = \frac{E_s}{E_c} = 8$, $M_{ubwservI} = 8.16\ \frac{k\cdot l}{l}$

$d_e = 21.56''$ $A_{s_2} = 0.6\ in^2$

$$\rho = \frac{A_{s_2}}{b\,d_e} = 0.0023$$

$$k = \sqrt{(\rho \cdot n)^2 + (2\rho n)} - \rho n = 0.175 \implies \boxed{k d_e = 3.77''}$$



$kd_e = 3.77''$

$A_s = 0.6\ in^2$

transformed moment of inertia $I_t = \frac{1}{3}(12''/l)(kd_e)^3 + nA_s(d_e - kd_e)^2$

$$= \frac{1}{3}(12)(3.77)^3 + (8)(.6)(21.56 - 3.77)^2$$

$$= \boxed{1733.45\ in^4}$$

Actual stress in the reinforcement: f_s ; $y = d_e - kd_e = 17.79''$

$$f_s = \frac{n(M_{ubwservI})(12''/l)\,y}{I_t} = \boxed{8.04\ ksi < 23.93\ ksi = f_{sa}} \qquad \text{OK}$$

Design for shear

$V_{ubw\,max} = 2.91$ k/lf

Nominal shear resistance (V_n) = smaller of $\begin{cases} V_{n_1} = V_c + V_s \\ V_{n_2} = 0.25 f'_c\, b_v d_v \end{cases}$

$\beta = 2.0$, $b_v = 12''$, $d_v = \begin{cases} 0.72h = 17.28 \\ 0.9 d_e = 19.40 \\ d_e - \frac{a}{2} = 21.12 \end{cases}$ ← governs → $d_v = 21.12''$

$V_{n_1} = 0.0316\, \beta \sqrt{f'_c}\, b_v d_v$
$= \boxed{32.03 \text{ k/lf}}$ ← governs

$V_{n_2} = 0.25 f'_c\, b_v d_v = 253.44$ k/lf

$\boxed{V_n = 32.03 \text{ k/lf}}$

$\phi = 0.9 \implies V_r = \phi V_n = 28.83$ k/lf $> V_{ubw\,max}$ OK

Shrinkage & temperature reinforcement.

$A_s \geq \left[0.11 \frac{A_g}{f_y} = .528 \text{ in}^2 \right]$

$A_g = (24'')(12''/l) = 288 \text{ in}^2/l$

OR

$A_s \geq \left[0.0015 A_g = 0.432 \right]$ ← governs

The above steel must be distributed equally on both faces of the backwall:
Try 1 horizontal #5 for each face of the backwall @ 12" spacing
$A_{s\,bar} = 0.31 \text{ in}^2$, diam = 0.625"

$A_s = 2A_{s\,bar} = 0.62 > 0.432$ OK

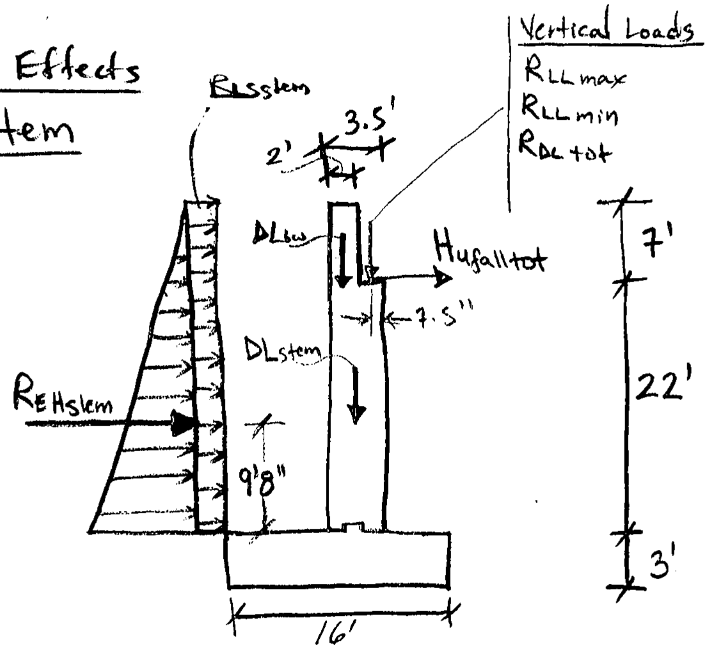
#7 @ 12" for back face flexure reinforcement
#7 @ 12" for front face vertical reinforcement
#5 @ 12" for front & back faces horizontal Temp & Shrinkage reinforcement.

STEM

Analyze & Combine Force Effects

- Bottom of Abutment Stem

$DL_{bw} = 2.1\ ^{k}/_{l}$
$DL_{stem} = 11.55\ ^{k}/_{l}$
$R_{DCTot} = 12.89\ ^{k}/_{l}$
$R_{DWTot} = 0$
$R_{LLmax} = 8\ ^{k}/_{l}$
$R_{EHstem} = 15.14\ ^{k}/_{l}$
$R_{LSstem} = 2.088\ ^{k}/_{l}$
$H_{ufalltot} = 0.899\ ^{k}/_{l}$



Abutment stem Strength I (Strength III + V values < Strength I values)

$\gamma_{DC} = 1.25$
$\gamma_{LL} = 1.75$
$\gamma_{EH} = 1.50$
$\gamma_{LS} = 1.75$
$\gamma_{TU} = 0.5$

- Factored vertical force = $F_{vstemstrI}$

$$F_{vstemstrI} = (\gamma_{DC})(DL_{bw} + DL_{stem} + R_{DCtot}) + (\gamma_{LL})(R_{LLmax})$$
$$= (1.25)(2.1 + 11.55 + 12.89) + (1.75)(8)$$
$$= \boxed{47.175\ ^{k}/_{l} = F_{vstemmax}}$$

- Factored longitudinal shear force = $V_{ustemstrI}$

$$V_{ustemstrI} = (\gamma_{EH})(R_{EHstem}) + (\gamma_{LS})(R_{LSstem}) + (\gamma_{TU})(H_{ufalltot})$$
$$= (1.5)(15.14) + (1.75)(2.088) + (0.5)(0.899)$$
$$= \boxed{26.81\ ^{k}/_{l} = V_{ustemmax}}$$

- Factored Moment about the bridge transverse axis @ the base of the stem:

$$M_{ustemstrI} = (\gamma_{DC})(DL_{bw})(.75) + (\gamma_{DC})(R_{DCTot})(1.125') + (\gamma_{LL})(R_{LLmax})(1.125') + (\gamma_{EH})(R_{EHstem})(9.66') + (\gamma_{LS})(R_{LSstem})(7.33') + (\gamma_{TU})(H_{ufalltot})(11')$$

$$= (1.25)(2.1)(0.75) + (1.25)(12.89)(1.125) + (1.75)(8)(1.125) + \ldots + (1.5)(15.14)(9.66') + (1.75)(2.088)(7.33') + (0.5)(0.899)(11)$$

$$= \boxed{286.95\ \frac{k \cdot l}{l} = M_{ustemmax}}$$

Abutment Stem Design

Design for flexure (USE #11 bars, $A_{sbar} = 1.56\ in^2$, diam = 1.41 in)

Cracking Strength

$M_{cr} = f_r \frac{I_g}{y_t}$; $f_r = 0.24\sqrt{f'_c} = 0.48ksi$; $I_g = \frac{1}{12}(12")(42")^3 = 74088\ in^4$; $y_t = 21"$

$$M_{cr} = 0.48\left(\frac{74088\ in^4}{(12)21"}\right) = \boxed{141.12\ \frac{k \cdot ')}{'}} \Rightarrow \boxed{1.2(M_{cr}) = 169.34\ \frac{k \cdot '}{'}}$$

governs

$$M_{ustemmax} = 286.95\ \frac{k \cdot '}{'} \Rightarrow \boxed{1.33\ M_{ustemmax} = 381.64}$$

$$M_{ustemdes} = 1.2(M_{cr}) = \boxed{169.34\ \frac{k \cdot '}{'}}$$

effective depth $d_e = t_{stem} - cover_s - \frac{bar\_diam}{2}$

$$= 42 - 2.5" - \frac{1.41}{2} = \boxed{38.795 = d_e}$$

reinforcing steel reqd:

$\phi = 0.9$, $b = 12"$, $f'_c = 4ksi$

$$R_n = \frac{M_{ustemdes}}{\phi b d_e^2} = \frac{169.34\ (12")}{(0.9)(12)(38.795)^2} = \boxed{0.125\ k/in^2}$$

$$\rho = 0.85\frac{f'_c}{f_y}\left(1 - \sqrt{1 - \frac{2R_n}{0.85f'_c}}\right) = 0.85\left(\frac{4}{60}\right)\left(1 - \sqrt{1 - \frac{2(.125)}{(.85)(4)}}\right) = \boxed{0.00212}$$

$$A_s = \rho b d_e = (0.00212)(12)(38.795) = \boxed{0.988\ in^2}$$

$$\text{reqd bars spacing} = \frac{A_{s\,bar}}{A_s} = \frac{1.56}{0.988}(12) = 18.94" \Rightarrow \text{USE \#11 bars @ 7"}$$

$$A_s = A_{sbar}\left(\frac{12"}{spacing}\right) = \boxed{2.674 in^2\ \text{per foot}}$$

CHECK

$T = A_s F_y = 160.44$, $a = \frac{T}{0.85 f'_c b} = 3.93"$, $\beta_1 = 0.85$, $c = \frac{a}{\beta_1} = 4.63$

$$\frac{c}{d_e} = 0.119 \Rightarrow \frac{c}{d_e} \leq 0.42 \Rightarrow \boxed{0.119 < 0.42}\quad \underline{OK}$$

Check crack control $\quad (Z = 130\ ^{k}/_{in})$

$$d_c = 2'' + \frac{\text{diam}}{2} = \boxed{2.705''}$$

Concrete area $(A_c) = 2(d_c)$ bar spacing $= 2(2.705)(7) = \boxed{37.87\ in^2}$

allowable reinforcement service load stress for crack control.

$$f_{sa} = \frac{Z}{(d_c A_c)^{1/3}} = \frac{130}{[(2.705)(37.87)]^{1/3}} = \boxed{27.80\,ksi < 0.6 f_y = 36\,ksi} \quad \underline{OK}$$

$$n = \frac{E_s}{E_c} = \frac{29{,}000}{3640} = 8$$

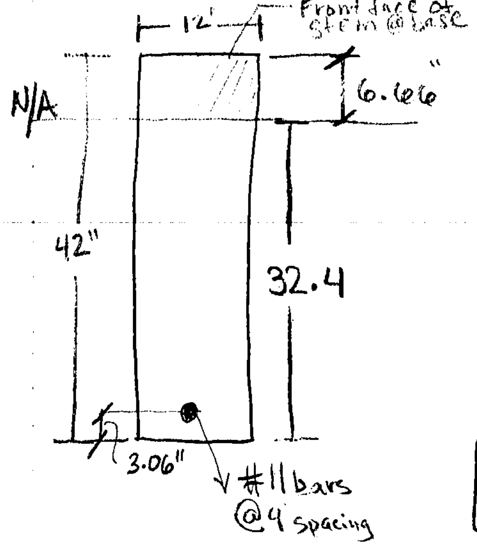
$$M_{u\,stem\,service\,I} = (\gamma_{DC})(DL_{bw})(0.75') + (\gamma_{DC})(R_{DC\,tot})(1.125') + (\gamma_{LL})(R_{LL\,max})(1.125') + (\gamma_{EH})(R_{EH\,stem})(9.6) + (\gamma_{LS})(R_{LS\,stem})(14.5) + (\gamma_{TU})(H_{u\,fall\,tot})(22')$$

$$= (2.1)(0.75) + (12.8)(1.125) + (8)(1.125) + (15.14)(9.6') + (2.088)(14.5') + (0.899)(22') = \boxed{219\ k\cdot'}$$

$M_{u\,stem\,serv\,I} = 221.38\ k\cdot'$
d_e : 38.795 in
A_s : 2.674 in^2
$n = 8$

$$\rho = \frac{A_s}{b\,d_e} = \frac{2.674}{(12)(38.795)} = \boxed{0.00574}$$

$$k = \sqrt{(\rho n)^2 + 2\rho n} - \rho n = \boxed{0.26} \Rightarrow \boxed{k d_e = 10.112\ in}$$



$$I_t = \frac{1}{3}(12''/1)(k d_e)^3 + n A_s (d_e - k d_e)^2 = \boxed{21{,}732.35\ in^4}$$

Actual stress in the reinforcement

$y = d_e - k d_e = 28.683$

$$f_s = \frac{n(M_{u\,stem\,serv\,I})(12''/1)(y)}{I_t} = \boxed{27.66\ ksi}$$

$$\left[f_s = 27.66\,ksi < 27.78\,ksi = f_{sa}\right] \quad \underline{OK}$$

Design for Shear

$V_{ustemmax} = 26.81\ ^{k}/_{l}$, $\beta_1 = 2.0$, $b_v = 12''$

Nominal Shear resistance V_n = lesser of $\begin{cases} V_{n_1} = V_c = 0.0316\,\beta\sqrt{f'_c}\,b_v d_v = \boxed{55.91\ ^{k}/_{l}} \leftarrow \text{governs} \\ V_{n_2} = 0.25 f'_c\, b_v d_v = 441.96\ ^{k}/_{l} \end{cases}$

$d_v = \begin{cases} 0.72(h = 42'') = 30.24 \\ 0.9 d_e = 34.9 \\ d_e - \frac{a}{2} = 36.83 \leftarrow \text{governs} \Rightarrow d_v = 36.83 \end{cases}$

$V_n = 55.91\ ^{k}/_{l} \Longrightarrow V_r = (\phi = 0.9)(V_n) = 50.32\ ^{k}/_{l} > V_{ustemmax} = 26.81\ ^{k}/_{l}$

$\boxed{V_r > V_{ustemmax}}$ OK

Shrinkage + temperature reinforcement

$A_s = 2.674\ in^2$

$A_g = (42'')(12''/_l) = 504\ in^2/_l$

$f_y = 60ksi$

$A_s \geq \left[0.11\left(\frac{A_g}{f_y}\right) = 0.11\left(\frac{504}{60}\right) = 0.924\right]$

$A_s \geq \left[0.0015 A_g = (0.0015)(504) = 0.76\right] \rightarrow$ governs

$A_s \geq 0.76\ in^2/_l$

Try 1 horizontal #5 bars for each face of the stem @ 9 spacing

bar diam = 0.625"

$A_{sbar} = 0.31\ in^2$

$$A_s = 2\left(\frac{A_{sbar}\left(\frac{12''}{\text{Spacing}}\right)}{ft}\right) = 2\left(\frac{(0.31)\left(\frac{12''}{9''}\right)}{ft}\right) = \boxed{0.83\ in^2/_l > 0.76\ in^2/_l}\ \text{OK}$$

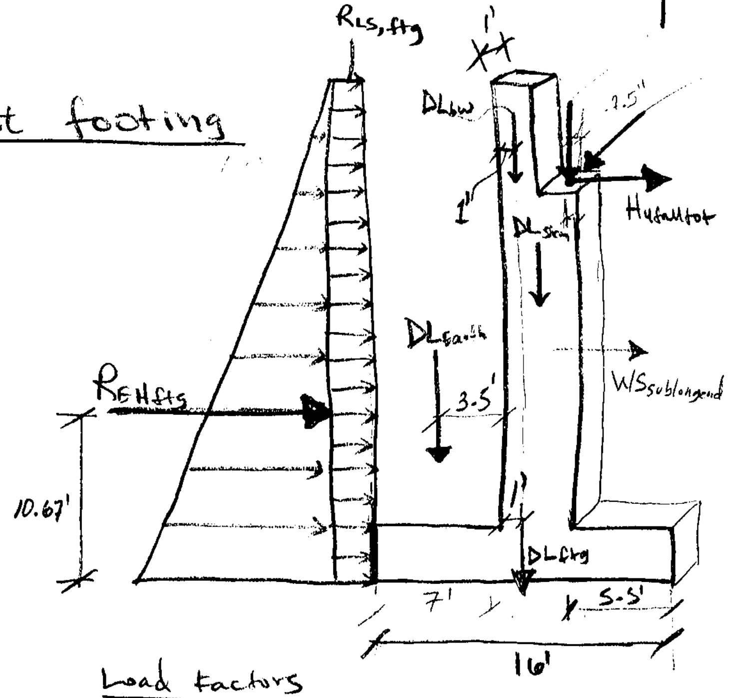
#11 @ 7" for back face flexure reinforcement

#11 @ 7" for front face vertical reinforcement

#5 @ 9" for front + back faces horizontal Temp + shrinkage reinforcement.

FOOTING

Vertical loads	Transverse Horizontal
R_{LLmax}	$WS_{supertr}$
R_{LLmin}	$WS_{subtrans}$
R_{DCTot}	$WS_{subtrans}$
	WL_{trans}



Bottom of abutment footing

Loads

$DL_{bw} = 2.1\ k/l$

$DL_{stem} = 11.55\ k/l$

$DL_{ftg} = 7.65\ k/l$

$DL_{Earth} = 27.84\ k/l$

$R_{DCTot} = 12.89\ k/l$

$R_{LLmin1} = -0.867\ k/l$

$R_{LLmax1} = 6.57\ k/l$

$R_{EHftg} = 18.43\ k/l$

$R_{LSftg} = 2.304\ k/l$

$H_{ufalltot} = 0.899\ k/l$

$WS_{supertrans0} = 9.93^{k}$

$WS_{subtransend0} = 4.06$

$WL_{trans0} = 4.05^{k}$ (6' above roadway)

$WS_{subtransend60} = 2.03^{k}$

$WS_{sublongend60} = 3.52^{k}$

$WS_{supertrans60} = 2.25^{k}$

Load factors

$\gamma_{LS} = 1.75$

$\gamma_{DC} = 1.25$

$\gamma_{LL} = 1.75$

$\gamma_{TU} = 0.5$

$\gamma_{EH} = 1.5$

$\gamma_{EV} = 1.35$

Abutment bottom of footing Strength I (using the maximum loads)

- Factored vertical force : $F_{vftgstrI}$

$$F_{vftgstrI} = (\gamma_{DC})(DL_{bw}) + (\gamma_{DC})(DL_{stem}) + (\gamma_{DC})(DL_{ftg}) + (\gamma_{DC})(R_{DCTot}) + (\gamma_{EV})(DL_{Earth}) + (\gamma_{LL})(R_{LLmax1})$$
$$= 1.25(2.1 + 11.55 + 7.65 + 12.89) + (1.35)(27.84) + (1.75)(6.57) = \boxed{91.82\ k/l}$$

- factored longitudinal horizontal force: $F_{longftgstrI}$

$$F_{longftgstrI} = (\gamma_{EH})(R_{EHftg}) + (\gamma_{LS})(R_{LSftg}) + (\gamma_{TU})(H_{ufalltot}) = (1.5)(18.43) + (1.75)(2.304) + (0.5)(0.899)$$
$$= \boxed{24.97\ k/l}$$

- factored transverse horizontal force: $F_{transftgstrI} = 0$

- Factored Moment about transverse axis:

$$M_{longftgstrI} = (\gamma_{DC})(DL_{bw})(0) + (\gamma_{DC})(DL_{stem})(0.75') + (\gamma_{EV})(DL_{Earth})(-4.5') + (\gamma_{DC})(R_{DCtot})(1.875')$$
$$+ (\gamma_{LL})(R_{LLmax1})(1.875) + (\gamma_{EH})(R_{EHftg})(10.66) + (\gamma_{LS})(R_{LSftg})(16') + (\gamma_{TU})(H_{ufalltot})(25')$$
$$= (1.25)(11.55)(0.75) + (1.35)(27.84)(-4.5') + (1.25)(12.89)(1.875') + (1.75)(6.57)(1.875) + \ldots$$
$$+ (1.5)(18.43)(10.66') + (1.75)(2.304)(16') + (0.5)(0.899)(25') = \boxed{263.91\ k'/l}$$

- Factored moment about longitudinal axis : $M_{transftgstrI} = 0$

Abutment Bottom of footing <u>Strength I</u> (using minimum load factors)

$\gamma_{DC} = .9$ $\quad\gamma_{EV} = 1.0$

$\gamma_{LL} = 1.75$ $\quad\gamma_{EH} = 0.9$

$\gamma_{BS} = 1.75$ $\quad\gamma_{TU} = 0.5$

- Vertical force

$$F_{vftgstrImin} = (0.9)(2.1 + 11.55 + 7.65 + 12.89) + (1.0)(27.84) + (1.75)(6.57)$$
$$= \boxed{70.105 \text{ kips}}$$

- longitudinal horizontal force

$$F_{longftgstrImin} = (0.9)(18.43) + (1.75)(2.304) + (0.5)(0.899)$$
$$= \boxed{21.07 \text{ kips}}$$

- transverse horizontal force = $F_{transftgstrImin} = 0$
- Moment about transverse axis

$$M_{longftgstrImin} = (0.9)(11.55)(.75) + (1.0)(27.84)(-4.5) + (.9)(12.89)(1.875) + (1.75)(6.57)(1.875) +$$
$$+ (0.9)(18.43)(10.66) + (1.75)(2.304)(16) + (0.5)(.899)(25)$$
$$= \boxed{178.39 \text{ k-ft}}$$

- Moment about longitudinal axis = $M_{transftgstrImin} = 0$

MAX	MIN
$F_{vftgstrI} = 91.82$ kips	$F_{vftgstrImin} = 70.105$ kips
$F_{longftgstrI} = 24.97$ kips	$F_{longftgstrImin} = 21.07$ kips
$M_{longftgstrI} = 263.91$ k-ft	$M_{longftgstrImin} = 178.39$ k-ft
$F_{transftgstrI} = 0$	$F_{transftgstrImin} = 0$
$M_{transftgstrI} = 0$	$M_{transftgstrImin} = 0$

Abutment bottom of footing service I (wind @ 0° + max live load)

$\gamma_{DC}, \gamma_{LL}, \gamma_{EH}, \gamma_{EV}, \gamma_{LS}, \gamma_{WL}, \gamma_{TU} = 1.0 \quad ; \gamma_{WS} = 0.3$ (wind @ 0°)

- Vertical Force

$$F_{vftgservI} = (\gamma_{DC})(DL_{bw}) + (\gamma_{DC})(DL_{stem}) + (\gamma_{DC})(DL_{ftg}) + (\gamma_{DC})(R_{DCtot}) + (\gamma_{EV})(DL_{Earth}) + (\gamma_{LL})(R_{LLm}$$
$$= 1.0(2.1 + 11.55 + 7.65 + 12.89 + 27.84 + 6.57)$$
$$= \boxed{68.6 \ k/l}$$

- Longitudinal Shear Force

$$F_{longftgservI} = (\gamma_{EH})(R_{EHftg}) + (\gamma_{LS})(R_{LSftg}) + (\gamma_{WS})\left(\frac{WS_{sublongend0}}{L_{abut}}\right) + (\gamma_{TU})(H_{ufaultot})$$
$$= 1.0(18.43 + 2.304 + 0.899) + 0.3(0) =$$
$$= \boxed{20.51 \ k/l}$$

- Transverse Shear Force

$$F_{transftgservI} = (\gamma_{WS})\left(\frac{WS_{supertrans0}}{L_{abut}}\right) + (\gamma_{WS})\left(\frac{WS_{subtransend0}}{L_{abut}}\right) + (\gamma_{WL})\left(\frac{WL_{trans0}}{L_{abut}}\right)$$
$$= 0.3\left(\frac{9.93 + 4.06}{30}\right) + (1.0)\left(\frac{4.05}{30}\right) = \boxed{0.275 \ k/l}$$

- Moment about transverse axis

$$M_{longftgservI} = (\gamma_{DC})(DL_{bw})(0) + (\gamma_{DC})(DL_{stem})(0.75) + (\gamma_{EV})(DL_{Earth})(-4.5) + (\gamma_{DC})(R_{DCtot})(1.875) +$$
$$+ (\gamma_{LL})(R_{LLmax1})(1.875) + (\gamma_{EH})(R_{EH})(10.66) + (\gamma_{LS})(R_{LS,ftg})(16) + (\gamma_{TU})(H_{ufaultot})(25) +$$
$$+ (\gamma_{WS})\left(\frac{WS_{sublongend0}}{L_{abut}}\right)(10')$$
$$= \boxed{175.67 \ \frac{k \cdot l}{l}}$$

- Moment about longitudinal axis

$$M_{transftgservI} = \gamma_{WS}\left(\frac{WS_{supertrans0}}{L_{abut}}\right)(25') + (\gamma_{WS})\left(\frac{WS_{subtransend0}}{L_{abut}}\right)(10') + \gamma_{WL}\left(\frac{WL_{trans0}}{L_{abut}}\right)(38')$$

32'+6

$$= 0.3\left(\frac{(9.93)(25') + (4.06)(10)}{30'}\right) + 1.0\left(\frac{4.05}{30'}\right)(38') = \boxed{8.018 \ \frac{k \cdot l}{l}}$$

Abutment bottom of footing Service I (wind @ 60° + min live load)

$\gamma_{DC}, \gamma_{LL}, \gamma_{EH}, \gamma_{EV}, \gamma_{LS}, \gamma_{WL}, \gamma_{TU} = 1.0$; $\gamma_{WS} = 0.3$ (wind @ 60°)

- Vertical force

$$F_{vftgservI\,min} = 1.0(2.1 + 11.55 + 7.65 + 12.89 + 27.84 - 0.867) = \boxed{61.16\ \text{k/ft}}$$

- Longitudinal shear force

$$F_{longftgservI\,min} = 1.0(18.43 + 2.304 + 0.899) + 0.3\left(\frac{3.52^k}{30'}\right) = \boxed{20.54\ \text{k/ft}}$$

- Transverse shear Force

$$F_{transftgservI\,min} = 0.3\left(\frac{2.25^k + 2.03^k}{30'}\right) = \boxed{0.143\ \text{k/ft}}$$

- Moment about transverse axis

$$M_{longftgservI\,min} = (11.55)(0.75) + (27.84)(-4.5) + (12.89)(1.875) + (-.867)(1.875) + \ldots$$
$$+ (18.43)(10.66) + (2.304)(16) + (0.899)(25) + 0.3\left(\frac{3.52^k}{30'}\right)(10') = \boxed{165.24\ \frac{\text{k}\cdot\text{ft}}{\text{ft}}}$$

- Moment about Longitudinal Axis

$$M_{transftgservI\,min} = 0.3\left(\frac{(2.25^k)(25') + (3.52^k)(10')}{30'}\right) = \boxed{0.914\ \frac{\text{k}\cdot\text{ft}}{\text{ft}}}$$

Limit state	Vertical Force (kip)	Long. Moment (k·ft)	Trans. Moment (k·ft)	Lateral Load Long. Direction (kip)	Lateral Load Trans. Direction (kip)
Strength I Max/Final	2754.6	7917.3	0	749.1	0
Strength I Min/Final	2103.15	5351.7	0	632.1	0
Service I Max/Final	2058	5270.1	240.5	615.3	8.25
Service I Min/Final	1834.8	4957.2	27.4	616.2	4.29

Abutment footing design

$\delta_s = 1.06$

$F_{uftgstrI} = 2754.6^{k}$

$M_{longftgstrI} = 7917.3^{k\cdot)}$

$M_{trasftgservI} = 240.5$

$P_u = F_{uftgstrI} = 2754.6$

$M_{uL} \quad 1.06(7917.3^{k\cdot)}) = 8392.34^{k\cdot)}$

$M_{uT} \quad 240.5^{k\cdot)}$

Net soil pressure $= \dfrac{2596.8^{k}}{(7)(34)} = \boxed{4.77 \text{ ksi} = q_n}$

$d = 36 - 3'' - 1'' = 32'' \longrightarrow d/2 = 16''$

$V_u = q_n\left(\dfrac{d/2}{12} \cdot 12\right) = 4.77 \text{ ksi}\left(\dfrac{17}{12} \cdot 12\right) = \boxed{81.02^{k}}$

$\phi V_c = 0.75(2)\sqrt{f'_c}\,(b_o)(d) \;;\; b_o = 2(d + t_w) = 2(32'' + 42'') = 148''$

$= 0.75(2)\sqrt{4000}\,(.148)(32'')$

$= \boxed{449.3^{k}} > V_u \rightarrow \underline{OK}$ ✓

Reinforcement short direction ($\phi = 0.9$, $j = 0.9$)

$A_s = \dfrac{M_{uL}}{\phi f_y j d} = \dfrac{9064.59^{k\cdot)}(12''/,)}{(0.9)(60)(0.9)(32)} = 64.76 \text{ in}^2$ ← governs

$A_{s\,min} = 0.0018(148)(32) = 8.52 \text{ in}^2$

USE 47 # 11 → $A_s = 73.32 \text{ in}^2$

$a = \dfrac{(73.32)(60)}{(.85)(4)(148)} = 8.74''$; $a/d = \dfrac{8.74}{32} = 0.27 < 0.5$ ✓

$\phi M_n = \dfrac{0.9(73.32)(60)\left(32 - \frac{8.74}{2}\right)}{12''/,} = 9116.24^{k\cdot)} > 9064.59^{k\cdot)}$ ✓

47 # 11 @ 7-1/2"

Reinforcement Long Direction

$$A_s = \frac{M_{4T}}{\phi f_y jd} = \frac{240.5(12)}{(0.9)(60)(.9)(32)} = \boxed{1.85 \text{ in}^2}$$ use $7 \# 5$ bars $A_s = 2.17 \text{ in}^2$

$$a = \frac{(2.17)(60)}{(.85)(4)(360)} = 0.105'' \quad ; \quad \frac{a}{d} = \frac{.105}{32} = 0.0033$$ ✓

$$\phi M_n = \frac{.9(1.85)(60)(32 - \frac{.105}{2})}{12} = 265.96^{k'} > 240.5^{k'}$$ ✓ ok

USE $7 \# 5$ bars

Check Max reinforcement

$$c/d_c < 0.42 \Rightarrow c = \frac{a}{\beta_1} = \frac{8.74}{.85} = 10.28 \Rightarrow \boxed{\frac{c}{d} = \frac{10.28}{32} = 0.32 < 0.42}$$ ok

Long Direction → use $7 \# 5$ bars @ $2'\text{-}9''$
Short direction → use $47 \# 11$ bars @ $7\frac{1}{2}''$

Shallow Foundation

$$q_{ult} = c' N_c + \sigma'_{zD} N_q + 0.5 \gamma'_s B N_\gamma$$

$\gamma_s = .120\text{kcf}$; $D = 4'$ $\phi' = 27$, $c' = 0$

$N_q = 15.9$ $N_\gamma = 12.5$ $N_c = 29.2$

$$q_{ult} = (0)(29.2) + (.120)(4)(15.9) + (0.5)(.120)(16)(12.5)$$
$$= 19.63 \text{ k/ft}^2$$

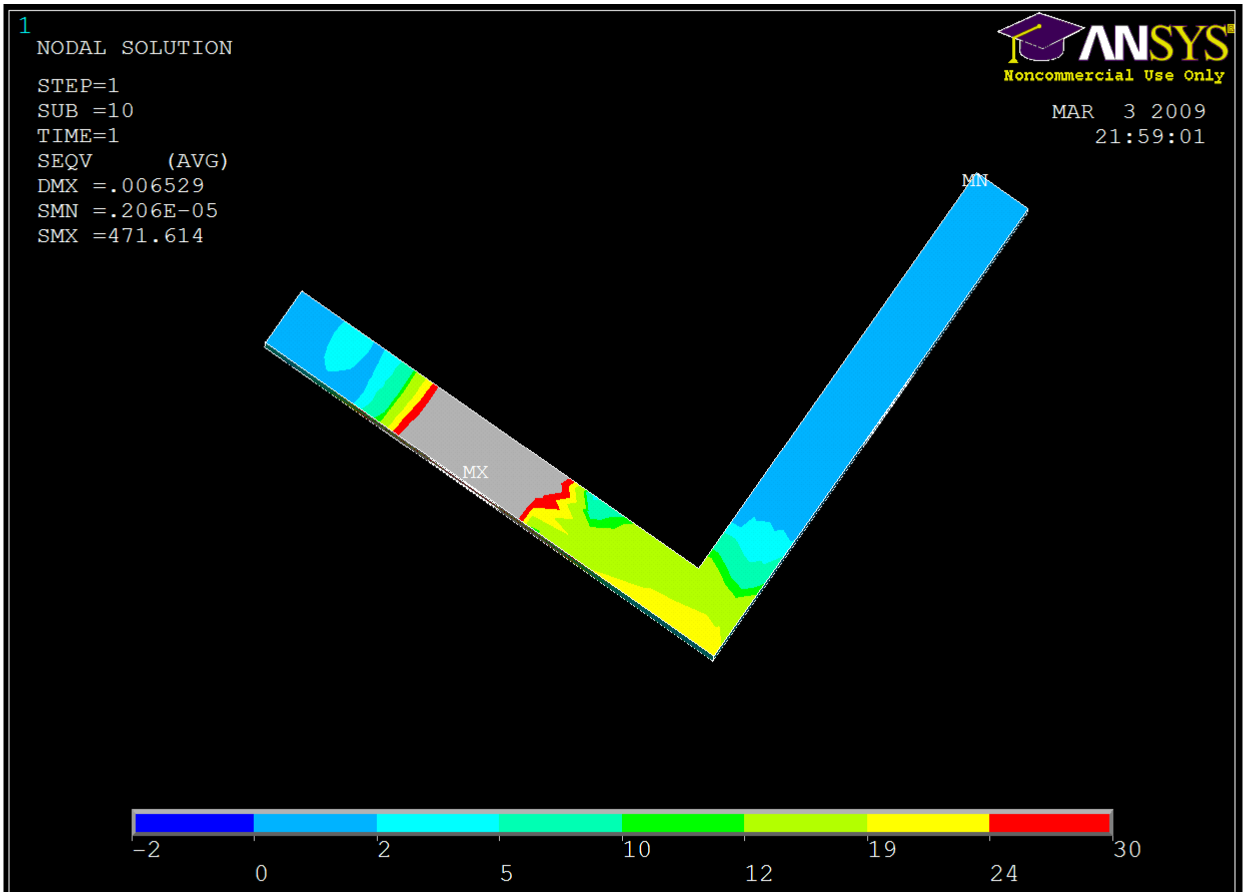
$$q = \frac{P + W_f}{A} - u$$

$$19.63 \text{ k/ft}^2 = \frac{P + [W_f = (16' \times 30' \times 3')(.150)]}{(16)(30)}$$

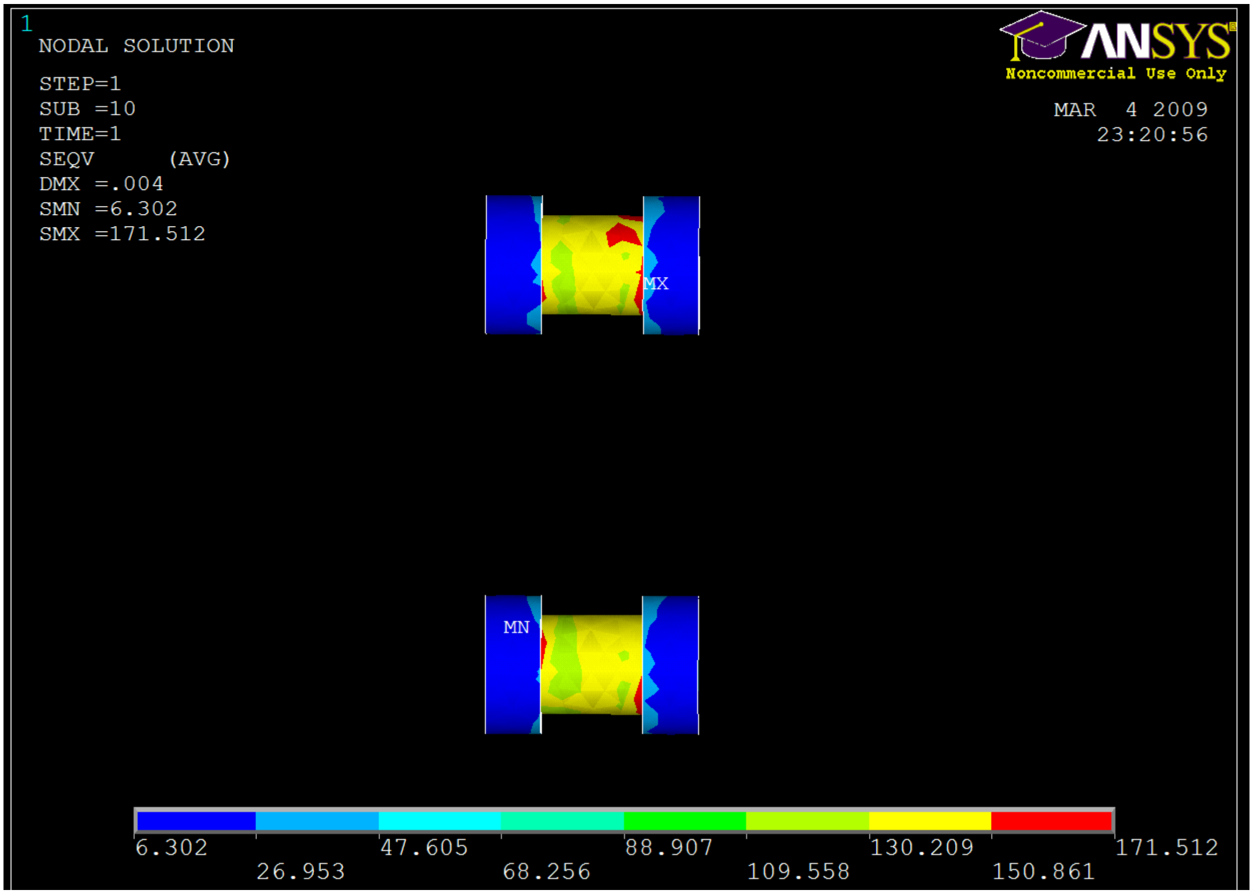
$P = 9207.36^k > 2596.8^k$ <u>OK</u>

Appendix J

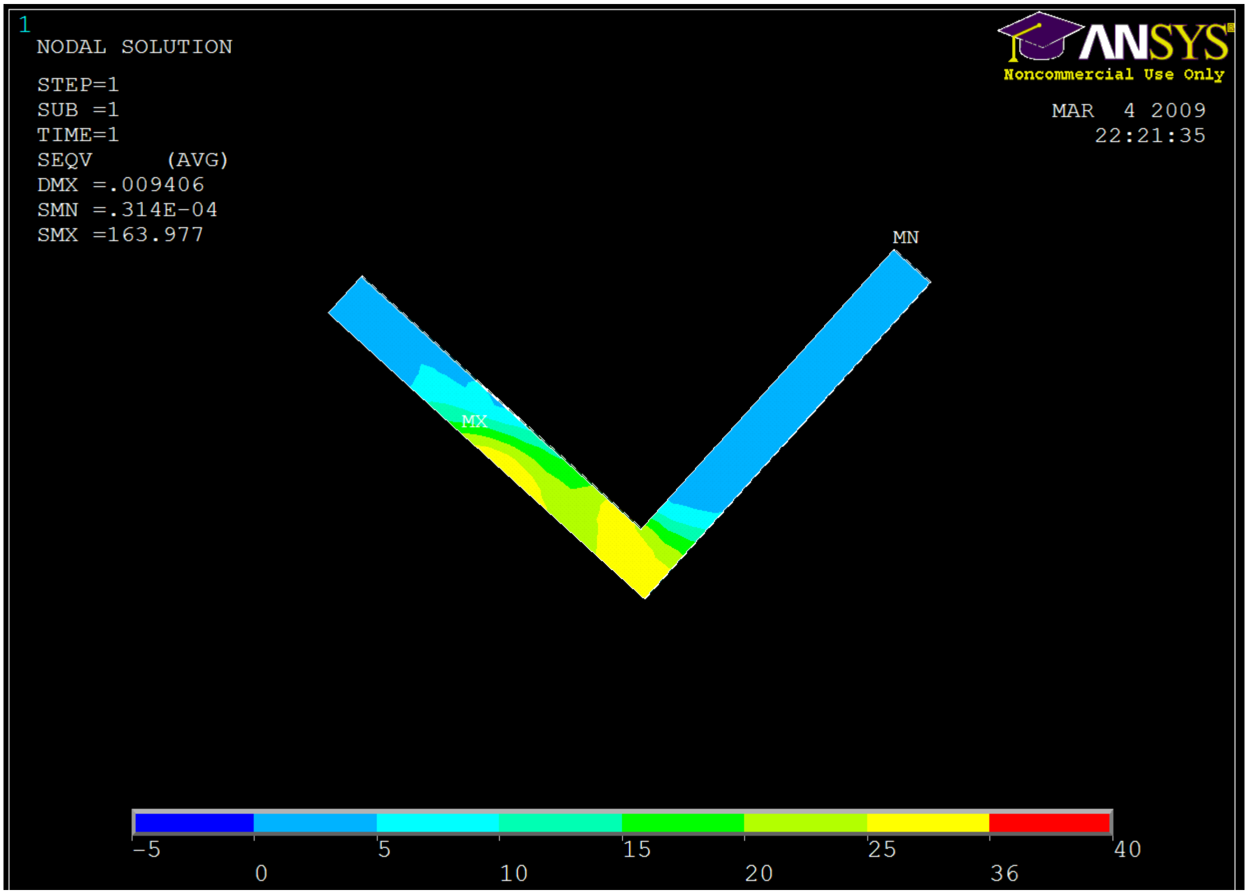
FEM of Connections



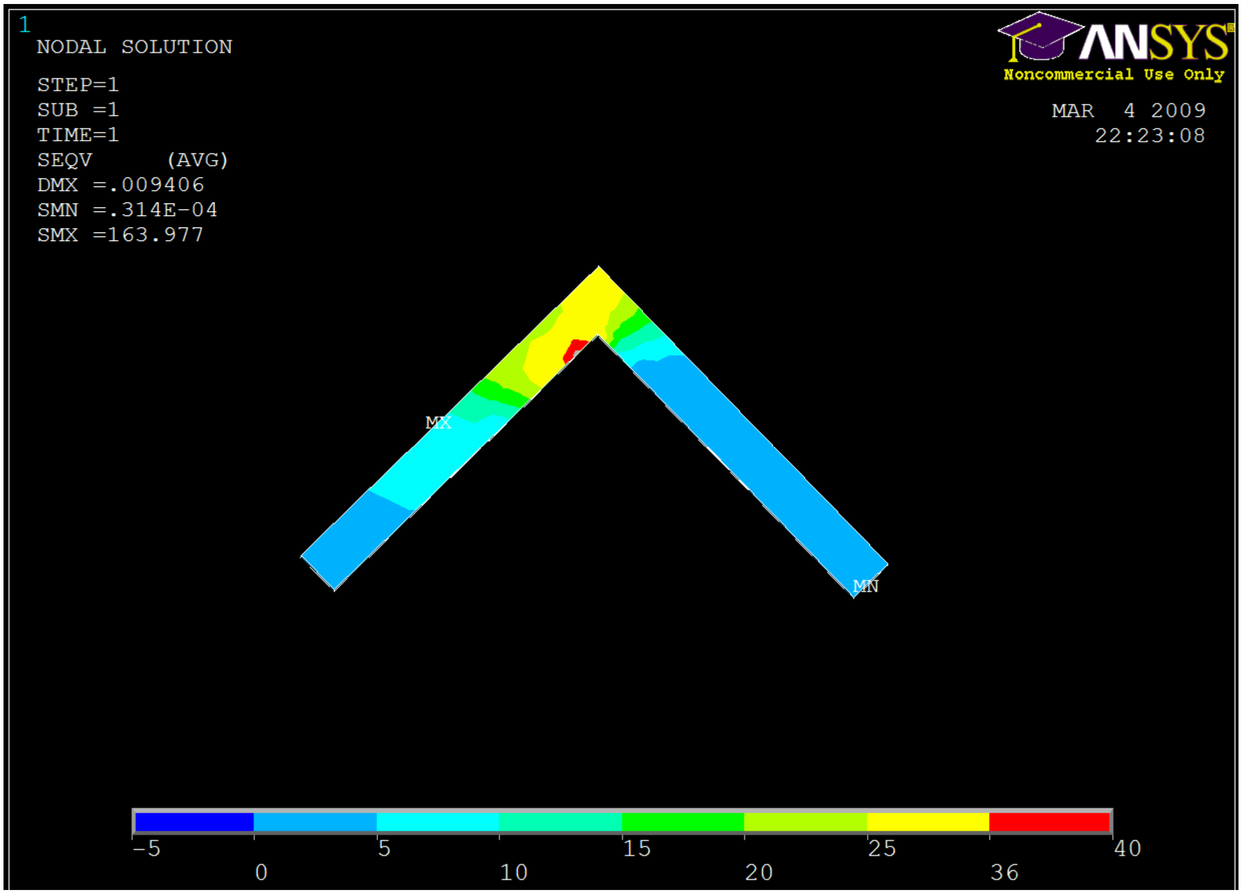
Model 1. Top view of the clip angle



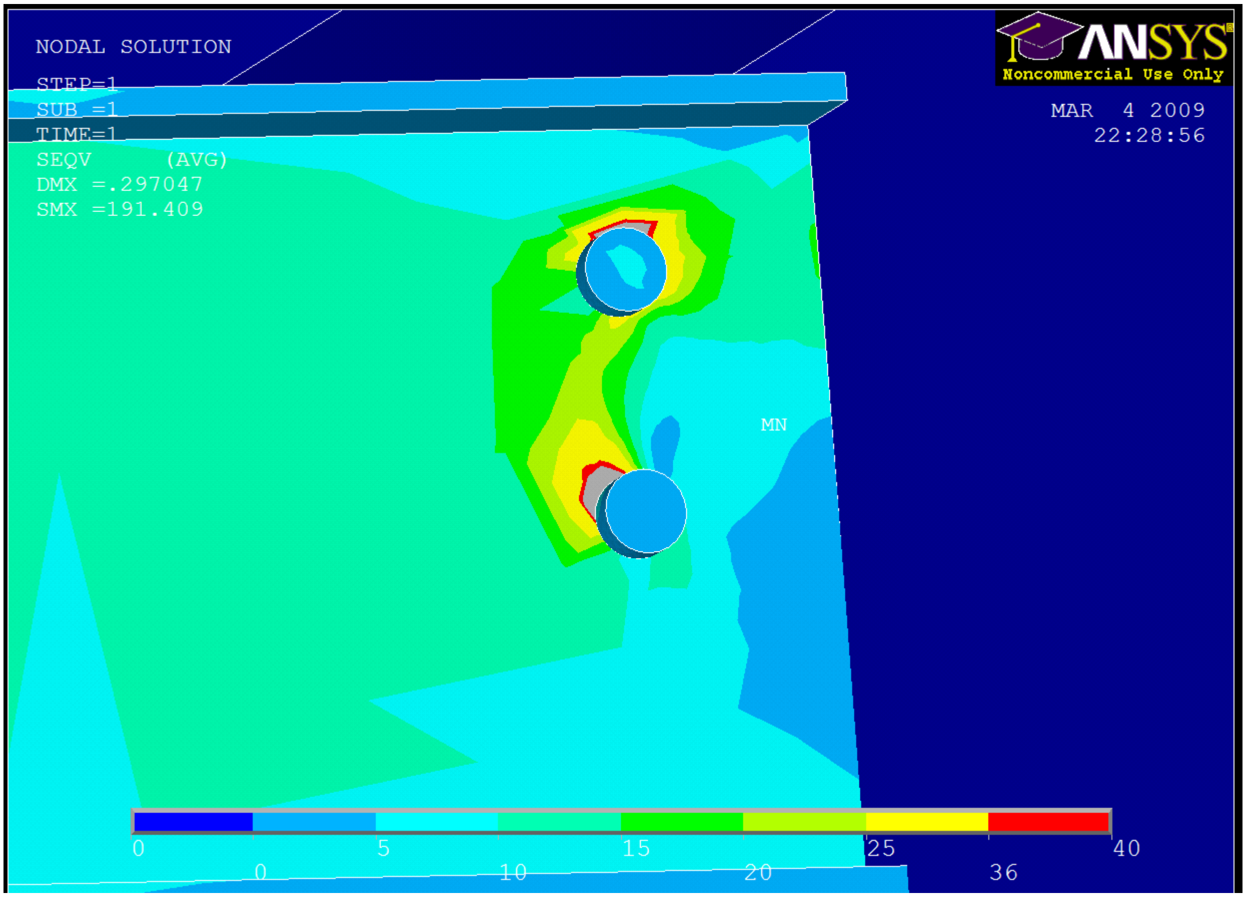
Model 1. Stress distribution in bolts.



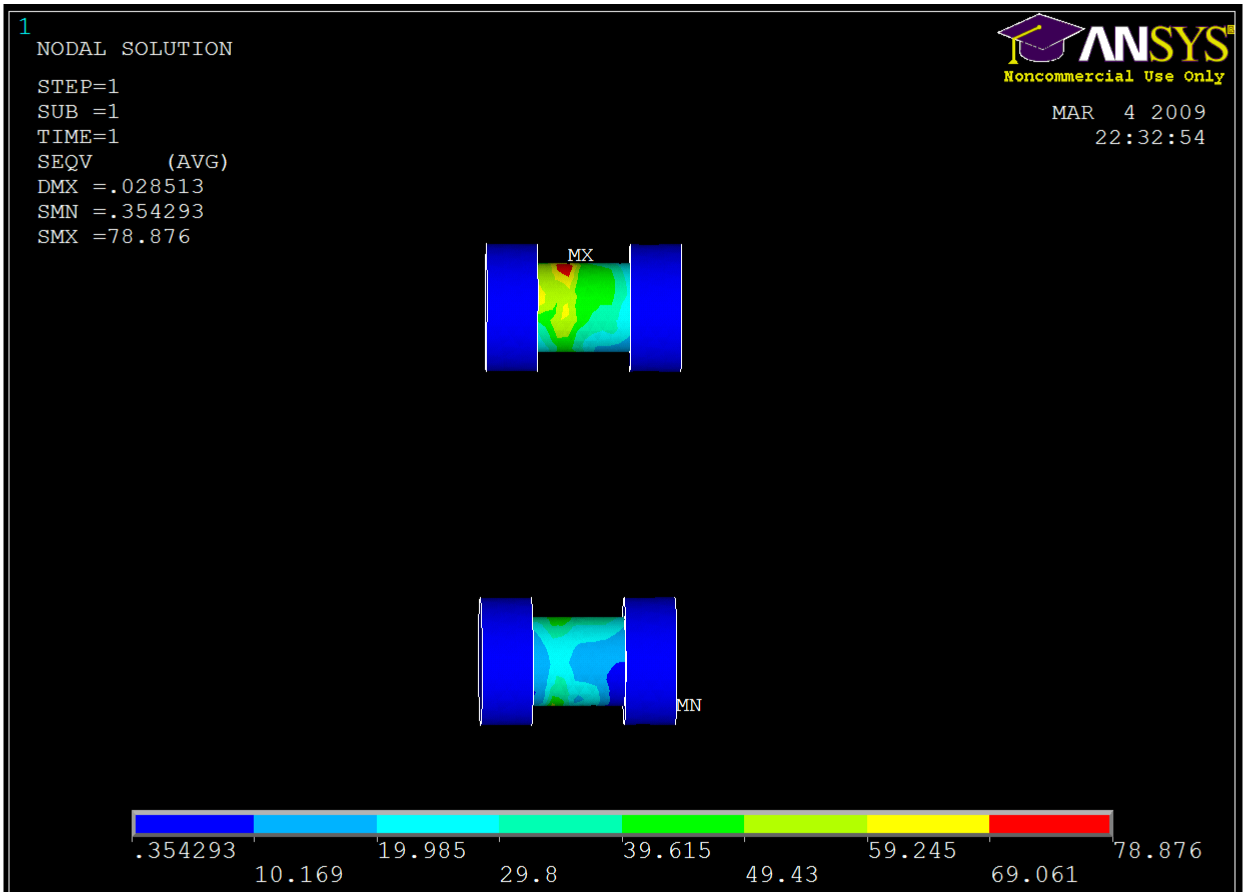
Model 2. Top view of clip angle.



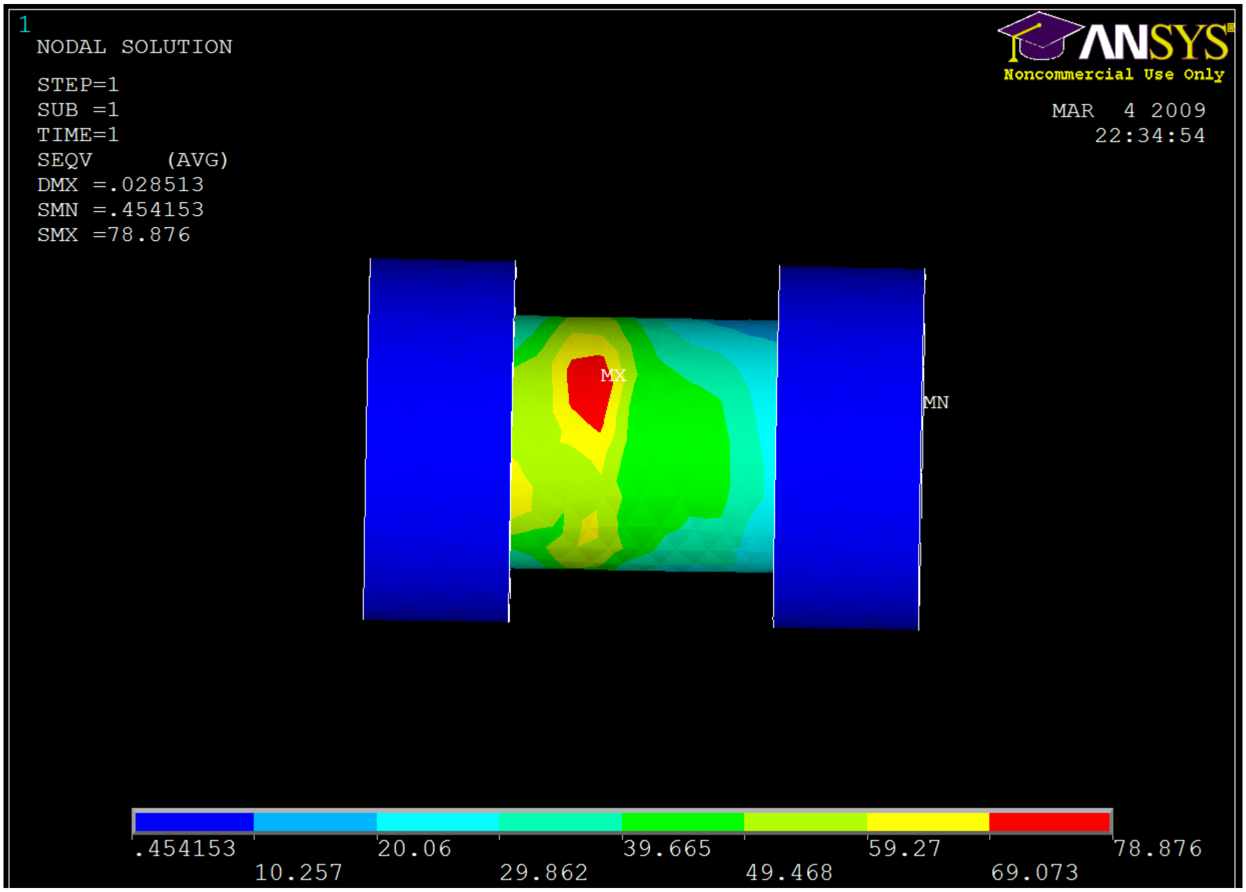
Model 2. Bottom view of clip angle.



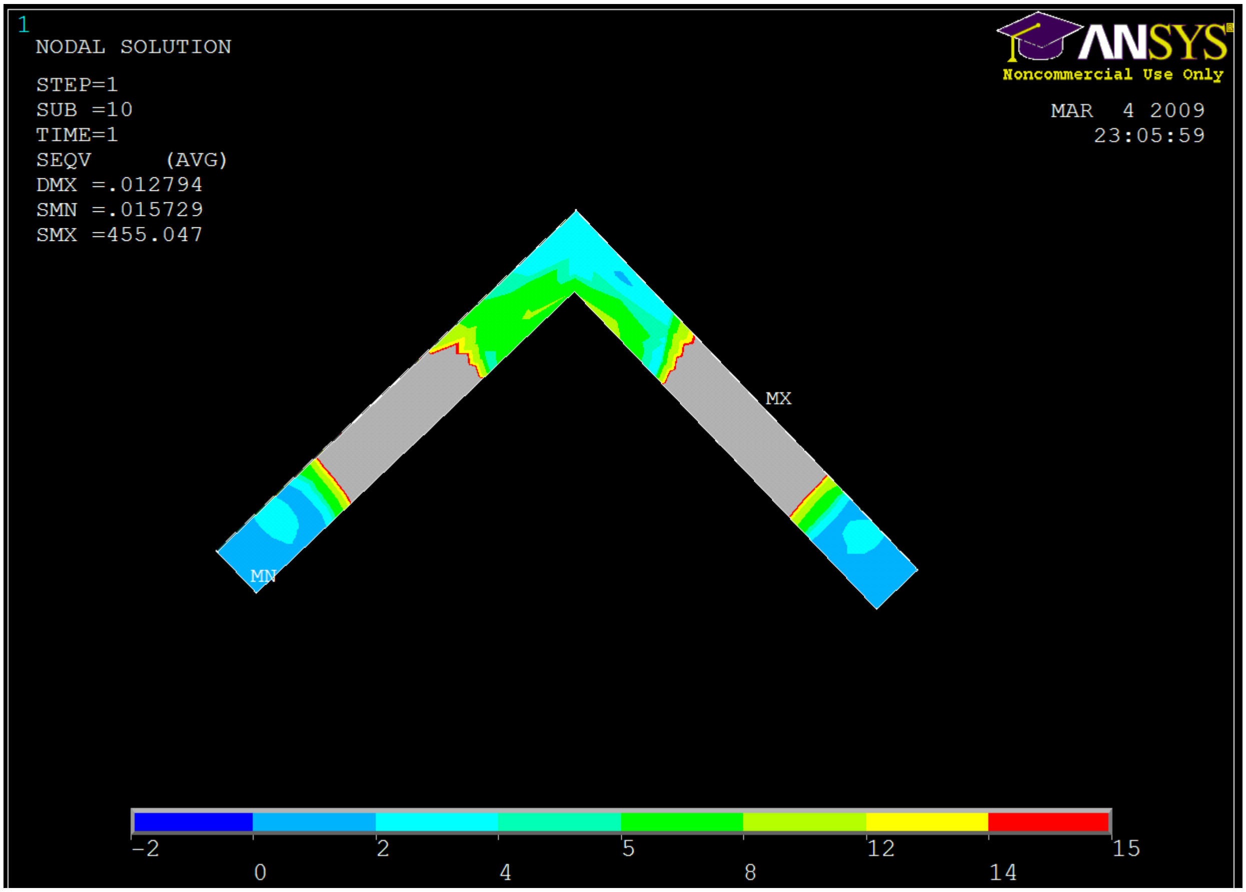
Model 2. Web of floor beam.



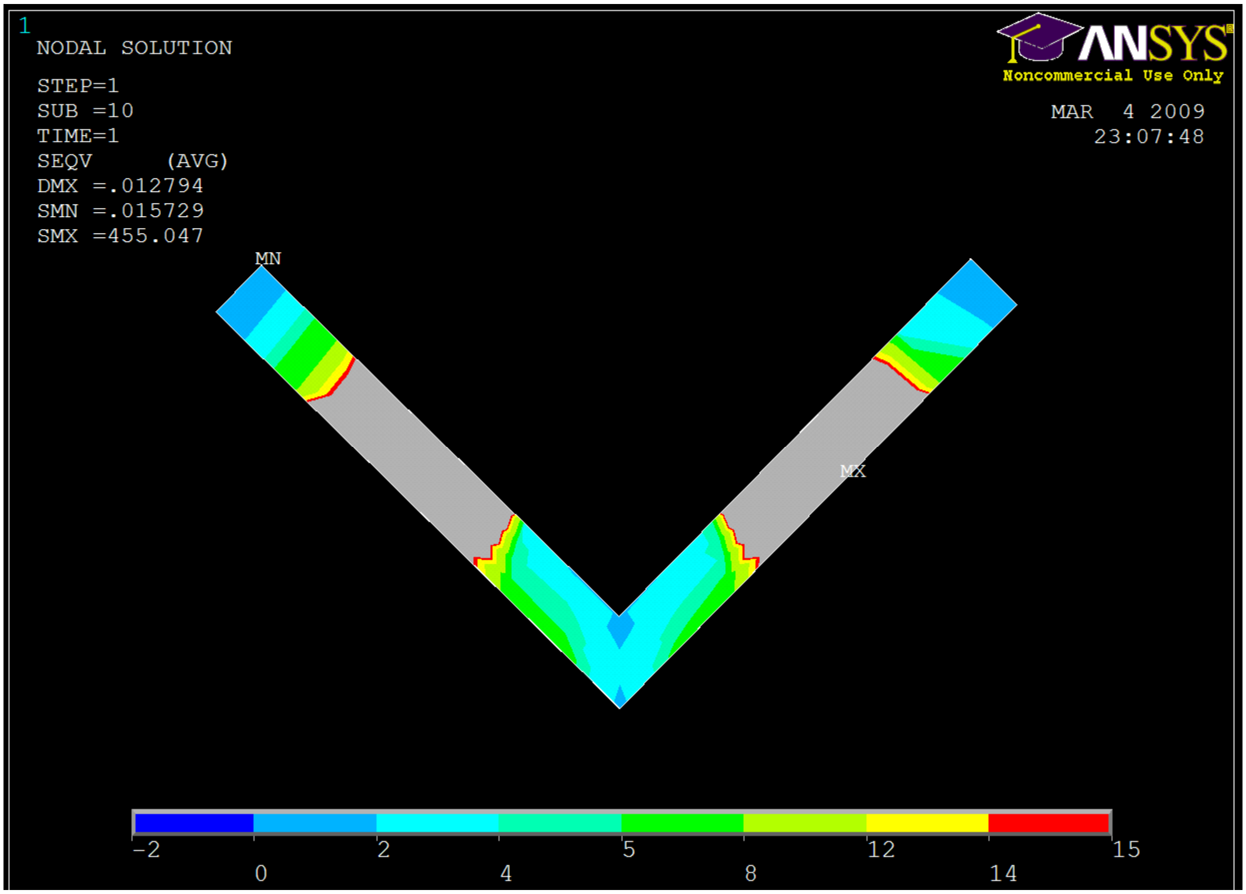
Model 2. Stress distribution in bolts.



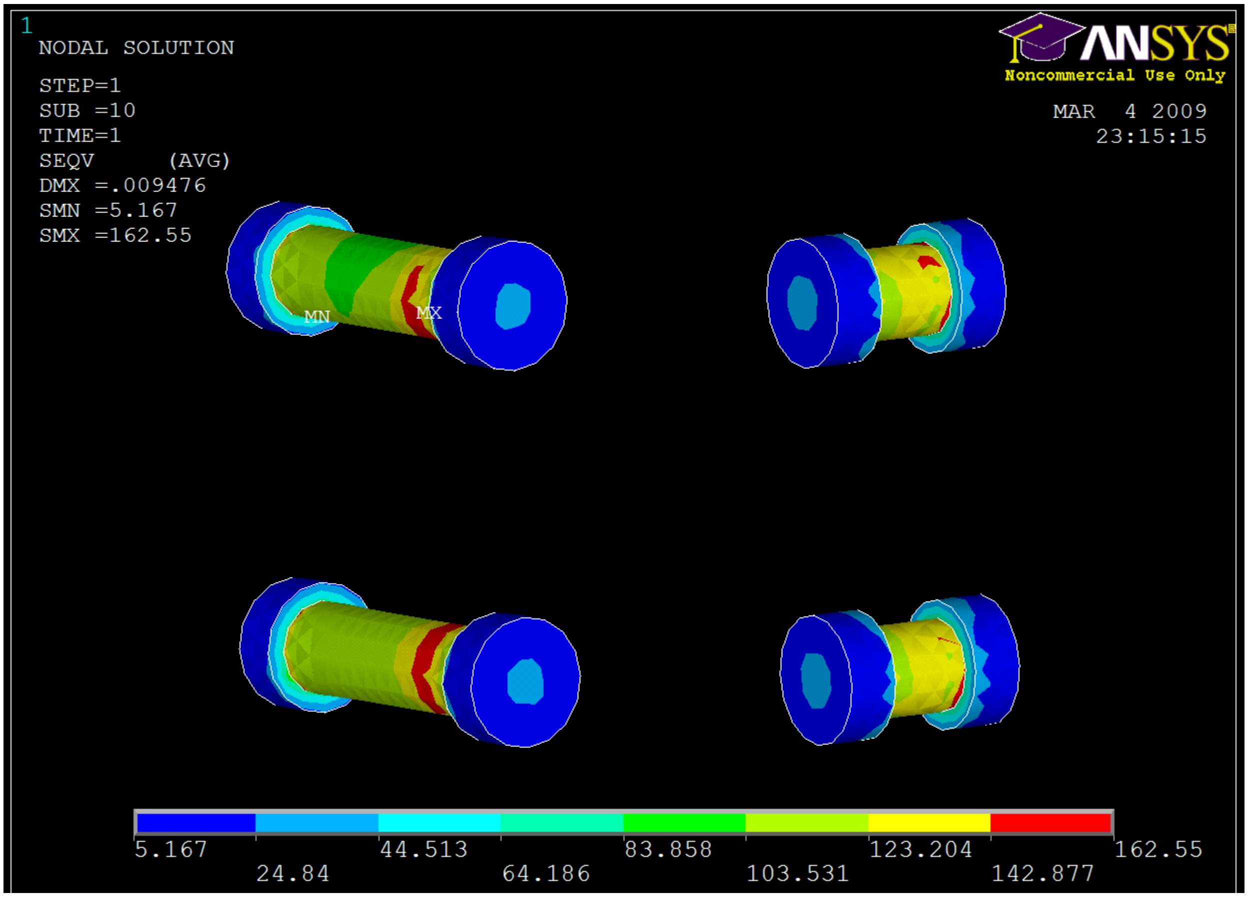
Model 2. Top bolt



Model 3. Top view of angle



Model 3. Bottom view of angle



Model 3. Bolts

Design of Single Angle Steel Connection

$R_n = 32^k$

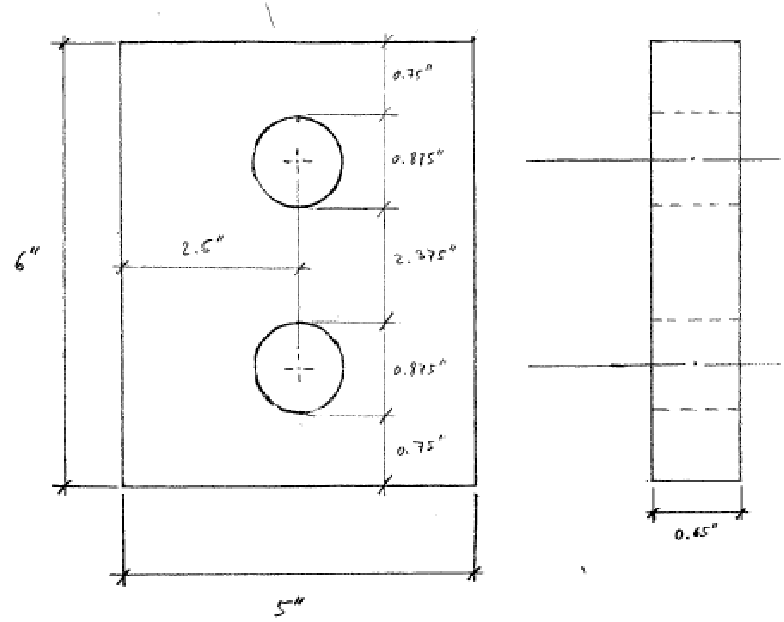
$F_y = 36$ ksi

$L = 6$ in

$F_u = 65$ ksi

$F_{nv} = 48$ ksi   Table J 3.2   AISC Manual

Geometry of connecting plate



Number of Bolts

Use 7/8" A 325 -N Bolts

$\phi R_n = \phi F_{nv} A_b = (0.75)(48)\left(\frac{\pi}{4}\right)\left(7/8\right)^2 = 21.6^k$

Number of Bolts $N = \dfrac{32^k}{21.6^k} = 1.48 \longrightarrow$ 2 Bolts

Angle thickness

- Bolt bearing on angle :

$$R_n = 1.2\, L_c\, t\, F_u \leq 2.4\, d_b\, t\, F_u$$

$$t = \frac{32}{(1.2)(0.75)(65)} = 0.55'' \longrightarrow \text{governs}$$

$$t = \frac{32}{(2.4)(7/8)(65)} = 0.23''$$

- Shear rupture of the angle :

$$R_n = 0.65 \times F_u \times \left[L - (N \times d_b)\right] \times t$$

$$t = \frac{32}{(0.6)(65)(6 - [2 \times 0.875])} = 0.19''$$

- Shear yield on angle :

$$R_n = 0.6 \times F_y \times L \times t$$

$$t = \frac{32}{(0.6)(36)(6)} = 0.24''$$

$$\boxed{\text{Use } t = 0.65''}$$

Appendix K

Life-Cycle Cost Analysis of Bearings

The following tables present the results obtained by assuming Low Labor Cost. The Present Worth estimates for Low, Average and high Expected Economic Life are also displayed below.

Low Labor Cost												
	Low Estimate				*High Estimate*				*Average Estimate*			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,3%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,2%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 1,020.30	$ -	$ -	1.0628	$ 2,125.62	$ -	$ -	1.041233	$ 1,561.85	$ -	$ -
4	1.0410	$ 1,041.02	$ 2,082.04	$ -	1.1296	$ 2,259.14	$ 3,388.71	$ -	1.084166	$ 1,084.17	$ 2,710.41	$ -
6	1.0622	$ 1,062.16	$ -	$ -	1.2005	$ 2,401.04	$ -	$ -	1.128869	$ 1,128.87	$ -	$ -
8	1.0837	$ 1,083.72	$ 2,167.45	$ -	1.2759	$ 2,551.86	$ 3,827.78	$ -	1.175415	$ 1,175.42	$ 2,938.54	$ -
10	1.1057	$ 1,105.73	$ -	$ -	1.3561	$ 2,712.14	$ -	$ -	1.223881	$ 1,223.88	$ -	$ -
12	1.1282	$ 1,128.18	$ 2,256.36	$ -	1.4412	$ 2,882.50	$ 4,323.75	$ -	1.274345	$ 1,274.35	$ 3,185.86	$ -
14	1.1511	$ 1,151.08	$ -	$ -	1.5318	$ 3,063.56	$ -	$ -	1.32689	$ 1,326.89	$ -	$ -
16	1.1745	$ 1,174.46	$ 2,348.91	$ -	1.6280	$ 3,255.98	$ 4,883.98	$ -	1.381601	$ 1,381.60	$ 3,454.00	$ -
18	1.1983	$ 1,198.30	$ -	$ -	1.7302	$ 3,460.50	$ -	$ -	1.438569	$ 1,438.57	$ -	$ -
20	1.2226	$ 1,222.63	$ 2,445.27	$ -	1.8389	$ 3,677.86	$ 5,516.79	$ -	1.497885	$ 1,497.89	$ 3,744.71	$ -
22	1.2475	$ 1,247.46	$ -	$ -	1.9544	$ 3,908.88	$ -	$ -	1.559647	$ 1,559.65	$ -	$ -
24	1.2728	$ 1,272.79	$ 2,545.57	$ -	2.0772	$ 4,154.40	$ 6,231.60	$ -	1.623956	$ 1,623.96	$ 4,059.89	$ -
26	1.2986	$ 1,298.63	$ -	$ -	2.2077	$ 4,415.35	$ -	$ -	1.690916	$ 1,690.92	$ -	$ -
28	1.3250	$ 1,325.00	$ 2,649.99	$ -	2.3463	$ 4,692.69	$ 7,039.03	$ -	1.760637	$ 1,760.64	$ 4,401.59	$ -
30	1.3519	$ 1,351.90	$ -	$ 193,321.52	2.4937	$ 4,987.44	$ -	$ 369,070.80	1.833233	$ 1,833.23	$ -	$ 266,735.44
32	1.3793	$ 1,379.35	$ 2,758.70	$ 197,246.73	2.6504	$ 5,300.72	$ 7,951.07	$ 392,252.95	1.908823	$ 1,908.82	$ 4,772.06	$ 277,733.69
34	1.4074	$ 1,407.35	$ -	$ -	2.8168	$ 5,633.67	$ -	$ -	1.987529	$ 1,987.53	$ -	$ -
35	1.4216	$ -	$ -	$ 203,284.48	2.9040	$ -	$ -	$ 429,784.76	2.028091	$ -	$ -	$ 295,087.18

Low expexted economic Life

Present Worth of Total Cost	**$ 228,569.46**	**$ 455,900.39**	**$ 313,861.32**

Medium Expected Economic Life

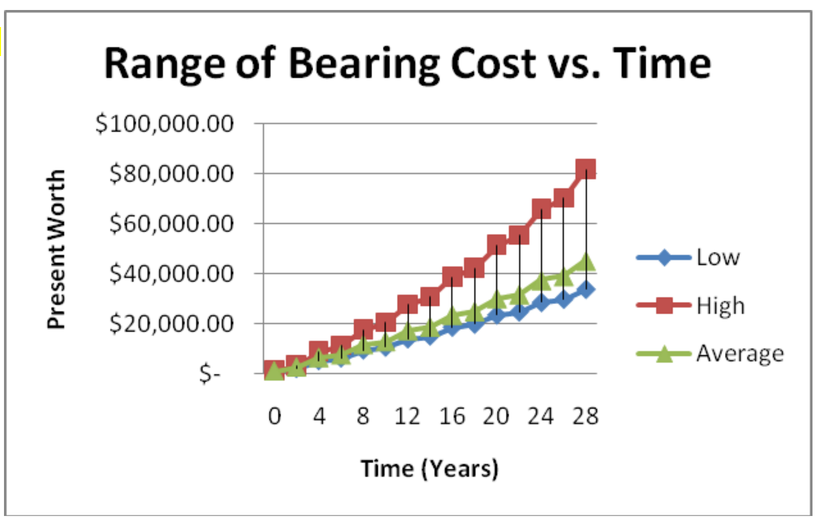
Present Worth of Total Cost	**$ 236,632.71**	**$ 492,334.33**	**$ 331,540.45**

High Expected Economic Life

Present Worth of Total Cost	**$ 244,077.82**	**$ 535,499.81**	**$ 350,881.46**

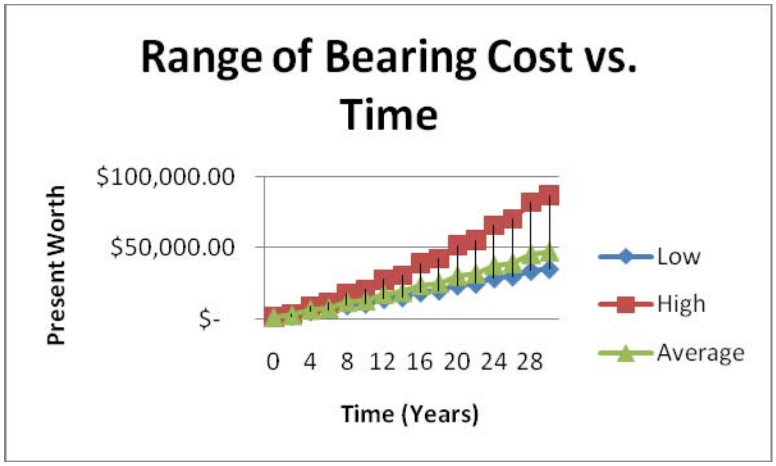
Low Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 228,569.46	$ 455,900.39	$ 313,861.32



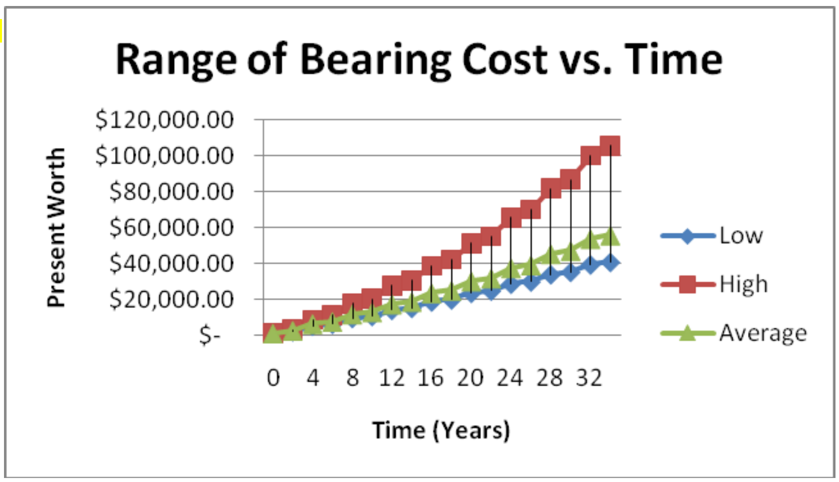
Average Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 35,247.94	$ 86,829.59	$ 47,125.88
$ 236,632.71	$ 492,334.33	$ 331,540.45



High Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 8,842.47	$ 6,425.43
$ 6,274.52	$ 11,243.51	$ 7,554.30
$ 9,525.69	$ 17,623.15	$ 11,668.25
$ 10,631.42	$ 20,335.30	$ 12,892.13
$ 14,015.95	$ 27,541.54	$ 17,352.34
$ 15,167.04	$ 30,605.10	$ 18,679.23
$ 18,690.41	$ 38,745.06	$ 23,514.84
$ 19,888.71	$ 42,205.56	$ 24,953.41
$ 23,556.61	$ 51,400.21	$ 30,196.00
$ 24,804.07	$ 55,309.09	$ 31,755.65
$ 28,622.42	$ 65,695.09	$ 37,439.50
$ 29,921.05	$ 70,110.44	$ 39,130.41
$ 33,896.04	$ 81,842.15	$ 45,292.64
$ 35,247.94	$ 86,829.59	$ 47,125.88
$ 39,385.98	$ 100,081.38	$ 53,806.75
$ 40,793.34	$ 105,715.05	$ 55,794.28
$ 244,077.82	$ 535,499.81	$ 350,881.46



The following tables present the results obtained by assuming High and Low Maintenance Cost. The Present Worth estimates for Low, Average and high Expected Economic Life are also displayed below.

High Maintenance Cost												
	Low Estimate				*High Estimate*				*Average Estimate*			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,3%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,2%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 1,020.30	$ -	$ -	1.0628	$ 2,125.62	$ -	$ -	1.041233	$ 1,561.85	$ -	$ -
4	1.0410	$ 1,041.02	$ 3,123.06	$ -	1.1296	$ 2,259.14	$ 3,388.71	$ -	1.084166	$ 1,084.17	$ 3,252.50	$ -
6	1.0622	$ 1,062.16	$ -	$ -	1.2005	$ 2,401.04	$ -	$ -	1.128869	$ 1,128.87	$ -	$ -
8	1.0837	$ 1,083.72	$ 3,251.17	$ -	1.2759	$ 2,551.86	$ 3,827.78	$ -	1.175415	$ 1,175.42	$ 3,526.25	$ -
10	1.1057	$ 1,105.73	$ -	$ -	1.3561	$ 2,712.14	$ -	$ -	1.223881	$ 1,223.88	$ -	$ -
12	1.1282	$ 1,128.18	$ 3,384.53	$ -	1.4412	$ 2,882.50	$ 4,323.75	$ -	1.274345	$ 1,274.35	$ 3,823.04	$ -
14	1.1511	$ 1,151.08	$ -	$ -	1.5318	$ 3,063.56	$ -	$ -	1.32689	$ 1,326.89	$ -	$ -
16	1.1745	$ 1,174.46	$ 3,523.37	$ -	1.6280	$ 3,255.98	$ 4,883.98	$ -	1.381601	$ 1,381.60	$ 4,144.80	$ -
18	1.1983	$ 1,198.30	$ -	$ -	1.7302	$ 3,460.50	$ -	$ -	1.438569	$ 1,438.57	$ -	$ -
20	1.2226	$ 1,222.63	$ 3,667.90	$ -	1.8389	$ 3,677.86	$ 5,516.79	$ -	1.497885	$ 1,497.89	$ 4,493.66	$ -
22	1.2475	$ 1,247.46	$ -	$ -	1.9544	$ 3,908.88	$ -	$ -	1.559647	$ 1,559.65	$ -	$ -
24	1.2728	$ 1,272.79	$ 3,818.36	$ -	2.0772	$ 4,154.40	$ 6,231.60	$ -	1.623956	$ 1,623.96	$ 4,871.87	$ -
26	1.2986	$ 1,298.63	$ -	$ -	2.2077	$ 4,415.35	$ -	$ -	1.690916	$ 1,690.92	$ -	$ -
28	1.3250	$ 1,325.00	$ 3,974.99	$ -	2.3463	$ 4,692.69	$ 7,039.03	$ -	1.760637	$ 1,760.64	$ 5,281.91	$ -
30	1.3519	$ 1,351.90	$ -	$ 193,321.52	2.4937	$ 4,987.44	$ -	$ 398,995.46	1.833233	$ 1,833.23	$ -	$ 277,734.84
32	1.3793	$ 1,379.35	$ 4,138.04	$ 197,246.73	2.6504	$ 5,300.72	$ 7,951.07	$ 424,057.24	1.908823	$ 1,908.82	$ 5,726.47	$ 289,186.63
34	1.4074	$ 1,407.35	$ -	$ -	2.8168	$ 5,633.67	$ -	$ -	1.987529	$ 1,987.53	$ -	$ -
35	1.4216	$ -	$ -	$ 203,284.48	2.9040	$ -	$ -	$ 464,632.18	2.028091	$ -	$ -	$ 307,255.72

Low expexted economic Life

Present Worth of Total Cost	$ 236,817.25	$ 485,825.05	$ 329,759.72

Medium Expected Economic Life

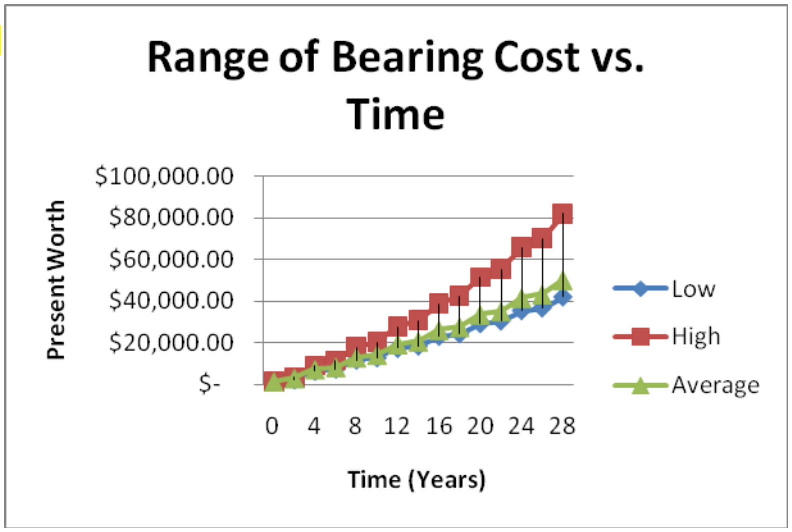
Present Worth of Total Cost	$ 246,259.85	$ 524,138.63	$ 348,846.80

High Expected Economic Life

Present Worth of Total Cost	$ 253,704.96	$ 570,347.23	$ 368,903.42

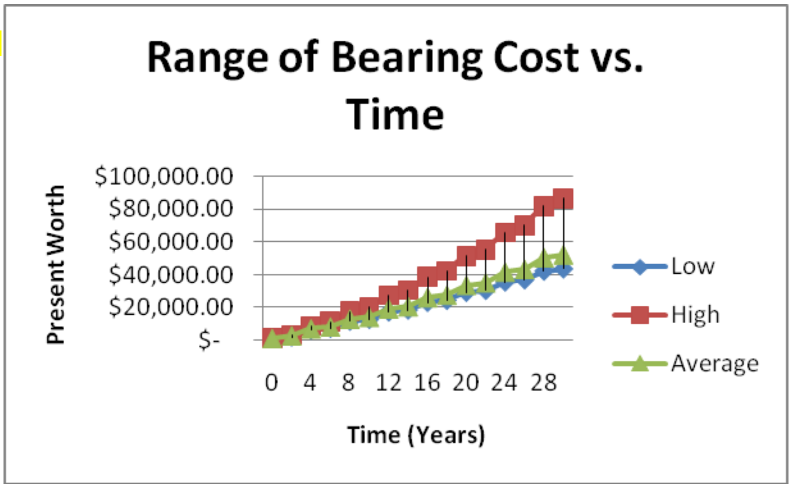
Low Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 6,253.39	$ 8,842.47	$ 6,967.51
$ 7,315.54	$ 11,243.51	$ 8,096.38
$ 11,650.44	$ 17,623.15	$ 12,798.04
$ 12,756.16	$ 20,335.30	$ 14,021.92
$ 17,268.88	$ 27,541.54	$ 19,119.31
$ 18,419.96	$ 30,605.10	$ 20,446.20
$ 23,117.79	$ 38,745.06	$ 25,972.60
$ 24,316.09	$ 42,205.56	$ 27,411.17
$ 29,206.62	$ 51,400.21	$ 33,402.71
$ 30,454.08	$ 55,309.09	$ 34,962.36
$ 35,545.22	$ 65,695.09	$ 41,458.18
$ 36,843.85	$ 70,110.44	$ 43,149.10
$ 42,143.83	$ 81,842.15	$ 50,191.64
$ 236,817.25	$ 485,825.05	$ 329,759.72



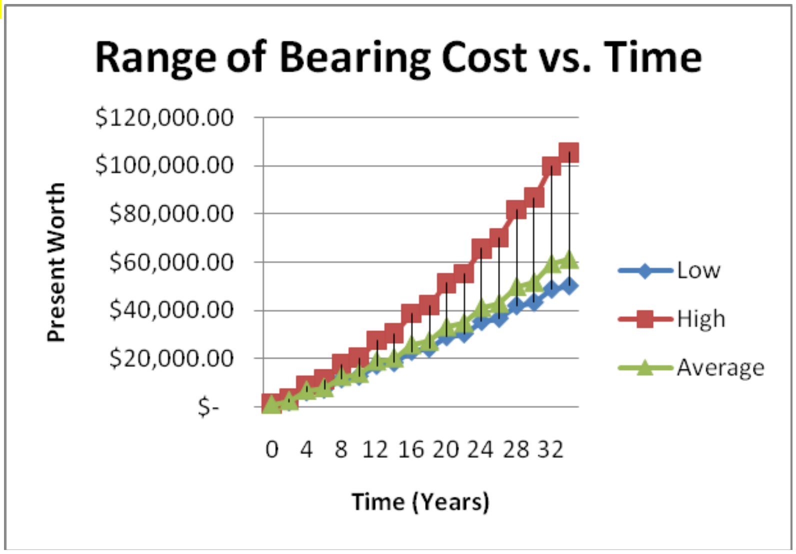
Average Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 6,253.39	$ 8,842.47	$ 6,967.51
$ 7,315.54	$ 11,243.51	$ 8,096.38
$ 11,650.44	$ 17,623.15	$ 12,798.04
$ 12,756.16	$ 20,335.30	$ 14,021.92
$ 17,268.88	$ 27,541.54	$ 19,119.31
$ 18,419.96	$ 30,605.10	$ 20,446.20
$ 23,117.79	$ 38,745.06	$ 25,972.60
$ 24,316.09	$ 42,205.56	$ 27,411.17
$ 29,206.62	$ 51,400.21	$ 33,402.71
$ 30,454.08	$ 55,309.09	$ 34,962.36
$ 35,545.22	$ 65,695.09	$ 41,458.18
$ 36,843.85	$ 70,110.44	$ 43,149.10
$ 42,143.83	$ 81,842.15	$ 50,191.64
$ 43,495.73	$ 86,829.59	$ 52,024.88
$ 246,259.85	$ 524,138.63	$ 348,846.80



High Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 6,253.39	$ 8,842.47	$ 6,967.51
$ 7,315.54	$ 11,243.51	$ 8,096.38
$ 11,650.44	$ 17,623.15	$ 12,798.04
$ 12,756.16	$ 20,335.30	$ 14,021.92
$ 17,268.88	$ 27,541.54	$ 19,119.31
$ 18,419.96	$ 30,605.10	$ 20,446.20
$ 23,117.79	$ 38,745.06	$ 25,972.60
$ 24,316.09	$ 42,205.56	$ 27,411.17
$ 29,206.62	$ 51,400.21	$ 33,402.71
$ 30,454.08	$ 55,309.09	$ 34,962.36
$ 35,545.22	$ 65,695.09	$ 41,458.18
$ 36,843.85	$ 70,110.44	$ 43,149.10
$ 42,143.83	$ 81,842.15	$ 50,191.64
$ 43,495.73	$ 86,829.59	$ 52,024.88
$ 49,013.12	$ 100,081.38	$ 59,660.17
$ 50,420.48	$ 105,715.05	$ 61,647.70
$ 253,704.96	$ 570,347.23	$ 368,903.42



Low Maintenance Cost

	Low Estimate				High Estimate				Average Estimate			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,3%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,2%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 1,020.30	$ -	$ -	1.0628	$ 2,125.62	$ -	$ -	1.041233	$ 1,561.85	$ -	$ -
4	1.0410	$ 1,041.02	$ 2,082.04	$ -	1.1296	$ 2,259.14	$ 2,259.14	$ -	1.084166	$ 1,084.17	$ 2,168.33	$ -
6	1.0622	$ 1,062.16	$ -	$ -	1.2005	$ 2,401.04	$ -	$ -	1.128869	$ 1,128.87	$ -	$ -
8	1.0837	$ 1,083.72	$ 2,167.45	$ -	1.2759	$ 2,551.86	$ 2,551.86	$ -	1.175415	$ 1,175.42	$ 2,350.83	$ -
10	1.1057	$ 1,105.73	$ -	$ -	1.3561	$ 2,712.14	$ -	$ -	1.223881	$ 1,223.88	$ -	$ -
12	1.1282	$ 1,128.18	$ 2,256.36	$ -	1.4412	$ 2,882.50	$ 2,882.50	$ -	1.274345	$ 1,274.35	$ 2,548.69	$ -
14	1.1511	$ 1,151.08	$ -	$ -	1.5318	$ 3,063.56	$ -	$ -	1.32689	$ 1,326.89	$ -	$ -
16	1.1745	$ 1,174.46	$ 2,348.91	$ -	1.6280	$ 3,255.98	$ 3,255.98	$ -	1.381601	$ 1,381.60	$ 2,763.20	$ -
18	1.1983	$ 1,198.30	$ -	$ -	1.7302	$ 3,460.50	$ -	$ -	1.438569	$ 1,438.57	$ -	$ -
20	1.2226	$ 1,222.63	$ 2,445.27	$ -	1.8389	$ 3,677.86	$ 3,677.86	$ -	1.497885	$ 1,497.89	$ 2,995.77	$ -
22	1.2475	$ 1,247.46	$ -	$ -	1.9544	$ 3,908.88	$ -	$ -	1.559647	$ 1,559.65	$ -	$ -
24	1.2728	$ 1,272.79	$ 2,545.57	$ -	2.0772	$ 4,154.40	$ 4,154.40	$ -	1.623956	$ 1,623.96	$ 3,247.91	$ -
26	1.2986	$ 1,298.63	$ -	$ -	2.2077	$ 4,415.35	$ -	$ -	1.690916	$ 1,690.92	$ -	$ -
28	1.3250	$ 1,325.00	$ 2,649.99	$ -	2.3463	$ 4,692.69	$ 4,692.69	$ -	1.760637	$ 1,760.64	$ 3,521.27	$ -
30	1.3519	$ 1,351.90	$ -	$ 193,321.52	2.4937	$ 4,987.44	$ -	$ 398,995.46	1.833233	$ 1,833.23	$ -	$ 277,734.84
32	1.3793	$ 1,379.35	$ 2,758.70	$ 197,246.73	2.6504	$ 5,300.72	$ 5,300.72	$ 424,057.24	1.908823	$ 1,908.82	$ 3,817.65	$ 289,186.63
34	1.4074	$ 1,407.35	$ -	$ -	2.8168	$ 5,633.67	$ -	$ -	1.987529	$ 1,987.53	$ -	$ -
35	1.4216	$ -	$ -	$ 203,284.48	2.9040	$ -	$ -	$ 464,632.18	2.028091	$ -	$ -	$ 307,255.72

Low expexted economic Life

	Low	High	Average
Present Worth of Total Cost	$ 228,569.46	$ 474,087.84	$ 319,961.71

Medium Expected Economic Life

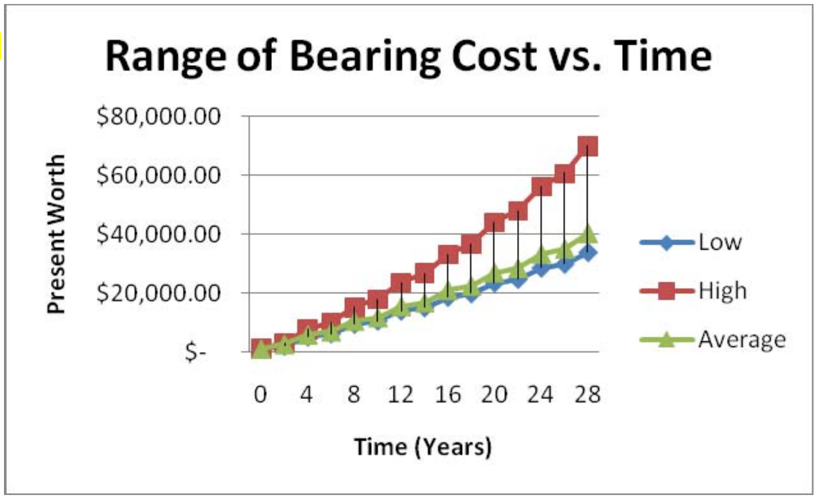
	Low	High	Average
Present Worth of Total Cost	$ 236,632.71	$ 509,751.06	$ 337,139.97

High Expected Economic Life

	Low	High	Average
Present Worth of Total Cost	$ 244,077.82	$ 555,959.65	$ 357,196.59

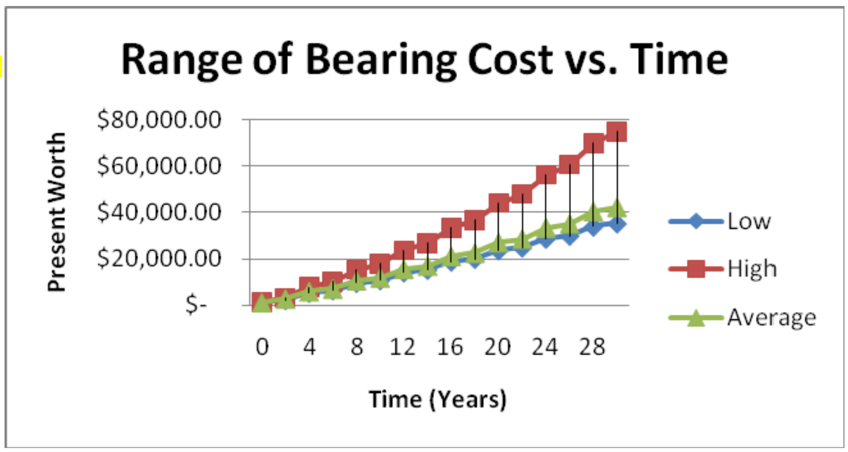
Low Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 7,712.90	$ 5,883.35
$ 6,274.52	$ 10,113.94	$ 7,012.22
$ 9,525.69	$ 15,217.66	$ 10,538.46
$ 10,631.42	$ 17,929.80	$ 11,762.34
$ 14,015.95	$ 23,694.80	$ 15,585.38
$ 15,167.04	$ 26,758.35	$ 16,912.27
$ 18,690.41	$ 33,270.32	$ 21,057.07
$ 19,888.71	$ 36,730.82	$ 22,495.64
$ 23,556.61	$ 44,086.54	$ 26,989.30
$ 24,804.07	$ 47,995.42	$ 28,548.94
$ 28,622.42	$ 56,304.22	$ 33,420.81
$ 29,921.05	$ 60,719.57	$ 35,111.73
$ 33,896.04	$ 70,104.94	$ 40,393.64
$ 228,569.46	$ 474,087.84	$ 319,961.71



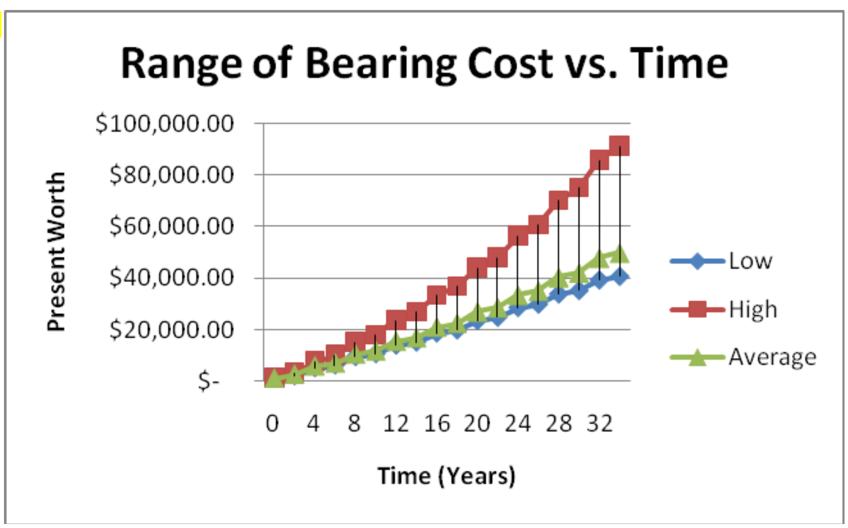
Average Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 7,712.90	$ 5,883.35
$ 6,274.52	$ 10,113.94	$ 7,012.22
$ 9,525.69	$ 15,217.66	$ 10,538.46
$ 10,631.42	$ 17,929.80	$ 11,762.34
$ 14,015.95	$ 23,694.80	$ 15,585.38
$ 15,167.04	$ 26,758.35	$ 16,912.27
$ 18,690.41	$ 33,270.32	$ 21,057.07
$ 19,888.71	$ 36,730.82	$ 22,495.64
$ 23,556.61	$ 44,086.54	$ 26,989.30
$ 24,804.07	$ 47,995.42	$ 28,548.94
$ 28,622.42	$ 56,304.22	$ 33,420.81
$ 29,921.05	$ 60,719.57	$ 35,111.73
$ 33,896.04	$ 70,104.94	$ 40,393.64
$ 35,247.94	$ 75,092.38	$ 42,226.87
$ 236,632.71	$ 509,751.06	$ 337,139.97



High Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,194.62	$ 2,630.85
$ 5,212.37	$ 7,712.90	$ 5,883.35
$ 6,274.52	$ 10,113.94	$ 7,012.22
$ 9,525.69	$ 15,217.66	$ 10,538.46
$ 10,631.42	$ 17,929.80	$ 11,762.34
$ 14,015.95	$ 23,694.80	$ 15,585.38
$ 15,167.04	$ 26,758.35	$ 16,912.27
$ 18,690.41	$ 33,270.32	$ 21,057.07
$ 19,888.71	$ 36,730.82	$ 22,495.64
$ 23,556.61	$ 44,086.54	$ 26,989.30
$ 24,804.07	$ 47,995.42	$ 28,548.94
$ 28,622.42	$ 56,304.22	$ 33,420.81
$ 29,921.05	$ 60,719.57	$ 35,111.73
$ 33,896.04	$ 70,104.94	$ 40,393.64
$ 35,247.94	$ 75,092.38	$ 42,226.87
$ 39,385.98	$ 85,693.81	$ 47,953.34
$ 40,793.34	$ 91,327.48	$ 49,940.87
$ 244,077.82	$ 555,959.65	$ 357,196.59



The following tables present the results obtained by assuming High and Low Inspection Cost. The Present Worth estimates for Low, Average and high Expected Economic Life are also displayed below.

High Inspection Cost												
	Low Estimate				*High Estimate*				*Average Estimate*			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,3%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,2%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 2,040.61	$ -	$ -	1.0628	$ 2,125.62	$ -	$ -	1.041233	$ 2,082.47	$ -	$ -
4	1.0410	$ 2,082.04	$ 2,082.04	$ -	1.1296	$ 2,259.14	$ 3,388.71	$ -	1.084166	$ 2,168.33	$ 2,710.41	$ -
6	1.0622	$ 2,124.31	$ -	$ -	1.2005	$ 2,401.04	$ -	$ -	1.128869	$ 2,257.74	$ -	$ -
8	1.0837	$ 2,167.45	$ 2,167.45	$ -	1.2759	$ 2,551.86	$ 3,827.78	$ -	1.175415	$ 2,350.83	$ 2,938.54	$ -
10	1.1057	$ 2,211.45	$ -	$ -	1.3561	$ 2,712.14	$ -	$ -	1.223881	$ 2,447.76	$ -	$ -
12	1.1282	$ 2,256.36	$ 2,256.36	$ -	1.4412	$ 2,882.50	$ 4,323.75	$ -	1.274345	$ 2,548.69	$ 3,185.86	$ -
14	1.1511	$ 2,302.17	$ -	$ -	1.5318	$ 3,063.56	$ -	$ -	1.32689	$ 2,653.78	$ -	$ -
16	1.1745	$ 2,348.91	$ 2,348.91	$ -	1.6280	$ 3,255.98	$ 4,883.98	$ -	1.381601	$ 2,763.20	$ 3,454.00	$ -
18	1.1983	$ 2,396.61	$ -	$ -	1.7302	$ 3,460.50	$ -	$ -	1.438569	$ 2,877.14	$ -	$ -
20	1.2226	$ 2,445.27	$ 2,445.27	$ -	1.8389	$ 3,677.86	$ 5,516.79	$ -	1.497885	$ 2,995.77	$ 3,744.71	$ -
22	1.2475	$ 2,494.91	$ -	$ -	1.9544	$ 3,908.88	$ -	$ -	1.559647	$ 3,119.29	$ -	$ -
24	1.2728	$ 2,545.57	$ 2,545.57	$ -	2.0772	$ 4,154.40	$ 6,231.60	$ -	1.623956	$ 3,247.91	$ 4,059.89	$ -
26	1.2986	$ 2,597.26	$ -	$ -	2.2077	$ 4,415.35	$ -	$ -	1.690916	$ 3,381.83	$ -	$ -
28	1.3250	$ 2,649.99	$ 2,649.99	$ -	2.3463	$ 4,692.69	$ 7,039.03	$ -	1.760637	$ 3,521.27	$ 4,401.59	$ -
30	1.3519	$ 2,703.80	$ -	$ 193,321.52	2.4937	$ 4,987.44	$ -	$ 398,995.46	1.833233	$ 3,666.47	$ -	$ 277,734.84
32	1.3793	$ 2,758.70	$ 2,758.70	$ 197,246.73	2.6504	$ 5,300.72	$ 7,951.07	$ 424,057.24	1.908823	$ 3,817.65	$ 4,772.06	$ 289,186.63
34	1.4074	$ 2,814.71	$ -	$ -	2.8168	$ 5,633.67	$ -	$ -	1.987529	$ 3,975.06	$ -	$ -
35	1.4216	$ -	$ -	$ 203,284.48	2.9040	$ -	$ -	$ 464,632.18	2.028091	$ -	$ -	$ 307,255.72

Low expexted economic Life

Present Worth of Total Cost	**$ 246,252.81**	**$ 485,825.05**	**$ 345,381.34**

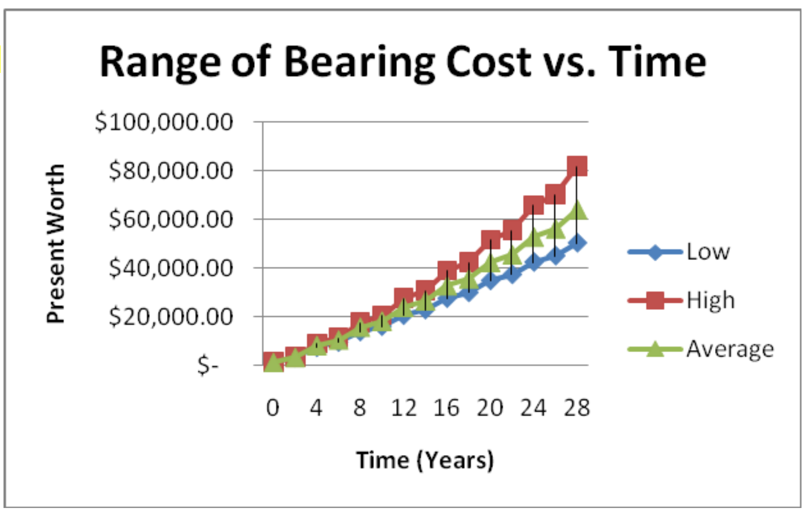
Medium Expected Economic Life

Present Worth of Total Cost	**$ 255,695.41**	**$ 524,138.63**	**$ 365,422.84**

High Expected Economic Life

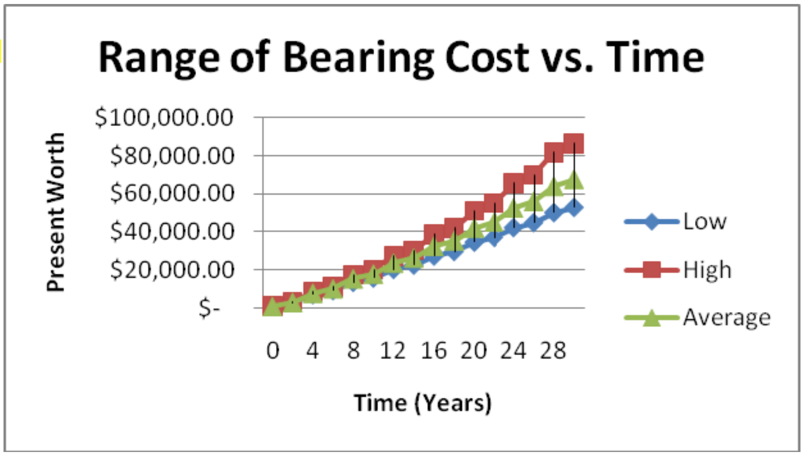
Present Worth of Total Cost	**$ 264,547.87**	**$ 570,347.23**	**$ 387,466.99**

Low Expected Economic Life		
Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 3,109.61	$ 3,194.62	$ 3,151.47
$ 7,273.69	$ 8,842.47	$ 8,030.21
$ 9,398.00	$ 11,243.51	$ 10,287.95
$ 13,732.90	$ 17,623.15	$ 15,577.32
$ 15,944.35	$ 20,335.30	$ 18,025.08
$ 20,457.06	$ 27,541.54	$ 23,759.63
$ 22,759.23	$ 30,605.10	$ 26,413.42
$ 27,457.06	$ 38,745.06	$ 32,630.62
$ 29,853.66	$ 42,205.56	$ 35,507.76
$ 34,744.20	$ 51,400.21	$ 42,248.24
$ 37,239.11	$ 55,309.09	$ 45,367.54
$ 42,330.25	$ 65,695.09	$ 52,675.34
$ 44,927.51	$ 70,110.44	$ 56,057.17
$ 50,227.50	$ 81,842.15	$ 63,980.04
$ 246,252.81	$ 485,825.05	$ 345,381.34



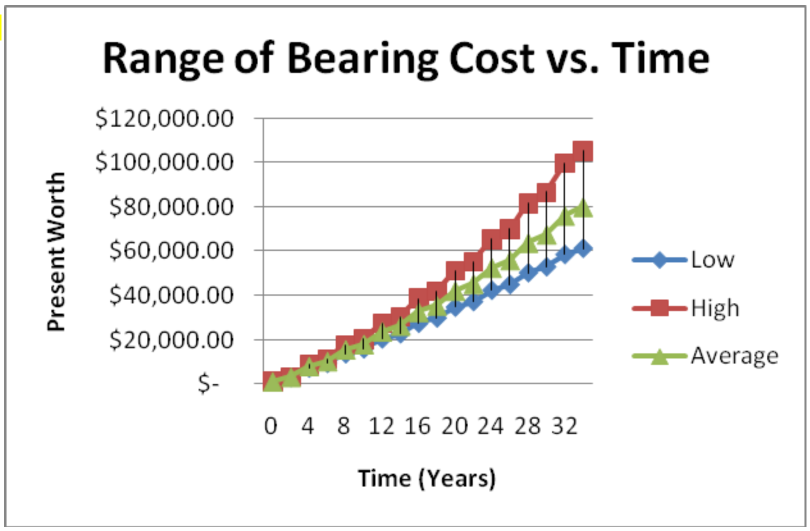
Average Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 3,109.61	$ 3,194.62	$ 3,151.47
$ 7,273.69	$ 8,842.47	$ 8,030.21
$ 9,398.00	$ 11,243.51	$ 10,287.95
$ 13,732.90	$ 17,623.15	$ 15,577.32
$ 15,944.35	$ 20,335.30	$ 18,025.08
$ 20,457.06	$ 27,541.54	$ 23,759.63
$ 22,759.23	$ 30,605.10	$ 26,413.42
$ 27,457.06	$ 38,745.06	$ 32,630.62
$ 29,853.66	$ 42,205.56	$ 35,507.76
$ 34,744.20	$ 51,400.21	$ 42,248.24
$ 37,239.11	$ 55,309.09	$ 45,367.54
$ 42,330.25	$ 65,695.09	$ 52,675.34
$ 44,927.51	$ 70,110.44	$ 56,057.17
$ 50,227.50	$ 81,842.15	$ 63,980.04
$ 52,931.29	$ 86,829.59	$ 67,646.50
$ 255,695.41	$ 524,138.63	$ 365,422.84



High Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 3,109.61	$ 3,194.62	$ 3,151.47
$ 7,273.69	$ 8,842.47	$ 8,030.21
$ 9,398.00	$ 11,243.51	$ 10,287.95
$ 13,732.90	$ 17,623.15	$ 15,577.32
$ 15,944.35	$ 20,335.30	$ 18,025.08
$ 20,457.06	$ 27,541.54	$ 23,759.63
$ 22,759.23	$ 30,605.10	$ 26,413.42
$ 27,457.06	$ 38,745.06	$ 32,630.62
$ 29,853.66	$ 42,205.56	$ 35,507.76
$ 34,744.20	$ 51,400.21	$ 42,248.24
$ 37,239.11	$ 55,309.09	$ 45,367.54
$ 42,330.25	$ 65,695.09	$ 52,675.34
$ 44,927.51	$ 70,110.44	$ 56,057.17
$ 50,227.50	$ 81,842.15	$ 63,980.04
$ 52,931.29	$ 86,829.59	$ 67,646.50
$ 58,448.68	$ 100,081.38	$ 76,236.20
$ 61,263.39	$ 105,715.05	$ 80,211.26
$ 264,547.87	$ 570,347.23	$ 387,466.99



Low Inspection Cost												
	Low Estimate				High Estimate				Average Estimate			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,3%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,2%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 1,020.30	$ -	$ -	1.0628	$ 1,062.81	$ -	$ -	1.041233	$ 1,561.85	$ -	$ -
4	1.0410	$ 1,041.02	$ 2,082.04	$ -	1.1296	$ 1,129.57	$ 3,388.71	$ -	1.084166	$ 1,084.17	$ 2,710.41	$ -
6	1.0622	$ 1,062.16	$ -	$ -	1.2005	$ 1,200.52	$ -	$ -	1.128869	$ 1,128.87	$ -	$ -
8	1.0837	$ 1,083.72	$ 2,167.45	$ -	1.2759	$ 1,275.93	$ 3,827.78	$ -	1.175415	$ 1,175.42	$ 2,938.54	$ -
10	1.1057	$ 1,105.73	$ -	$ -	1.3561	$ 1,356.07	$ -	$ -	1.223881	$ 1,223.88	$ -	$ -
12	1.1282	$ 1,128.18	$ 2,256.36	$ -	1.4412	$ 1,441.25	$ 4,323.75	$ -	1.274345	$ 1,274.35	$ 3,185.86	$ -
14	1.1511	$ 1,151.08	$ -	$ -	1.5318	$ 1,531.78	$ -	$ -	1.32689	$ 1,326.89	$ -	$ -
16	1.1745	$ 1,174.46	$ 2,348.91	$ -	1.6280	$ 1,627.99	$ 4,883.98	$ -	1.381601	$ 1,381.60	$ 3,454.00	$ -
18	1.1983	$ 1,198.30	$ -	$ -	1.7302	$ 1,730.25	$ -	$ -	1.438569	$ 1,438.57	$ -	$ -
20	1.2226	$ 1,222.63	$ 2,445.27	$ -	1.8389	$ 1,838.93	$ 5,516.79	$ -	1.497885	$ 1,497.89	$ 3,744.71	$ -
22	1.2475	$ 1,247.46	$ -	$ -	1.9544	$ 1,954.44	$ -	$ -	1.559647	$ 1,559.65	$ -	$ -
24	1.2728	$ 1,272.79	$ 2,545.57	$ -	2.0772	$ 2,077.20	$ 6,231.60	$ -	1.623956	$ 1,623.96	$ 4,059.89	$ -
26	1.2986	$ 1,298.63	$ -	$ -	2.2077	$ 2,207.67	$ -	$ -	1.690916	$ 1,690.92	$ -	$ -
28	1.3250	$ 1,325.00	$ 2,649.99	$ -	2.3463	$ 2,346.34	$ 7,039.03	$ -	1.760637	$ 1,760.64	$ 4,401.59	$ -
30	1.3519	$ 1,351.90	$ -	$ 193,321.52	2.4937	$ 2,493.72	$ -	$ 398,995.46	1.833233	$ 1,833.23	$ -	$ 277,734.84
32	1.3793	$ 1,379.35	$ 2,758.70	$ 197,246.73	2.6504	$ 2,650.36	$ 7,951.07	$ 424,057.24	1.908823	$ 1,908.82	$ 4,772.06	$ 289,186.63
34	1.4074	$ 1,407.35	$ -	$ -	2.8168	$ 2,816.83	$ -	$ -	1.987529	$ 1,987.53	$ -	$ -
35	1.4216	$ -	$ -	$ 203,284.48	2.9040	$ -	$ -	$ 464,632.18	2.028091	$ -	$ -	$ 307,255.72

Low expexted economic Life

Present Worth of Total Cost	$ 228,569.46	$ 460,550.58	$ 324,860.71

Medium Expected Economic Life

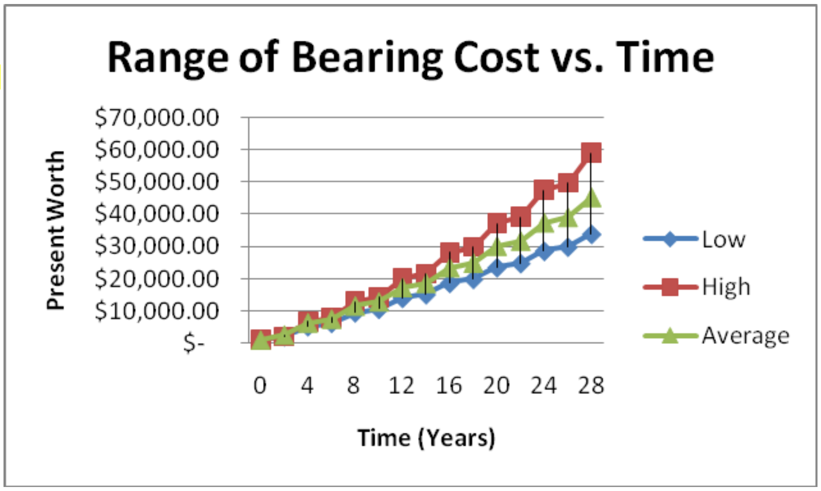
Present Worth of Total Cost	$ 236,632.71	$ 496,213.79	$ 342,993.38

High Expected Economic Life

Present Worth of Total Cost	$ 244,077.82	$ 539,605.56	$ 363,050.01

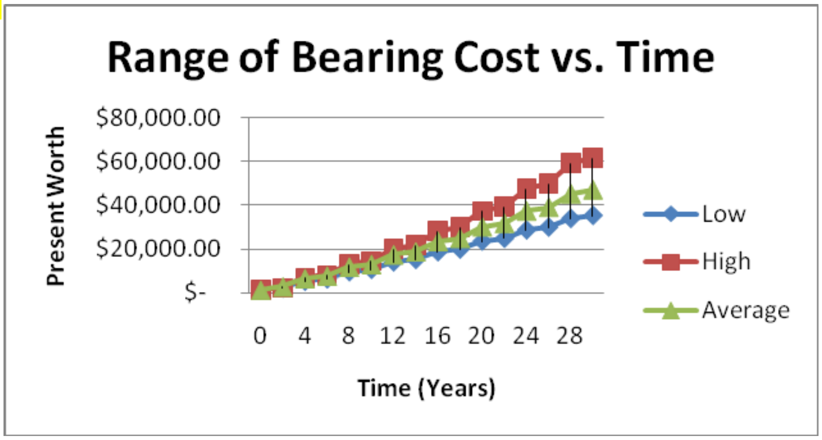
Low Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 2,131.81	$ 2,630.85
$ 5,212.37	$ 6,650.09	$ 6,425.43
$ 6,274.52	$ 7,850.61	$ 7,554.30
$ 9,525.69	$ 12,954.32	$ 11,668.25
$ 10,631.42	$ 14,310.40	$ 12,892.13
$ 14,015.95	$ 20,075.39	$ 17,352.34
$ 15,167.04	$ 21,607.17	$ 18,679.23
$ 18,690.41	$ 28,119.14	$ 23,514.84
$ 19,888.71	$ 29,849.39	$ 24,953.41
$ 23,556.61	$ 37,205.11	$ 30,196.00
$ 24,804.07	$ 39,159.55	$ 31,755.65
$ 28,622.42	$ 47,468.35	$ 37,439.50
$ 29,921.05	$ 49,676.02	$ 39,130.41
$ 33,896.04	$ 59,061.39	$ 45,292.64
$ 228,569.46	$ 460,550.58	$ 324,860.71



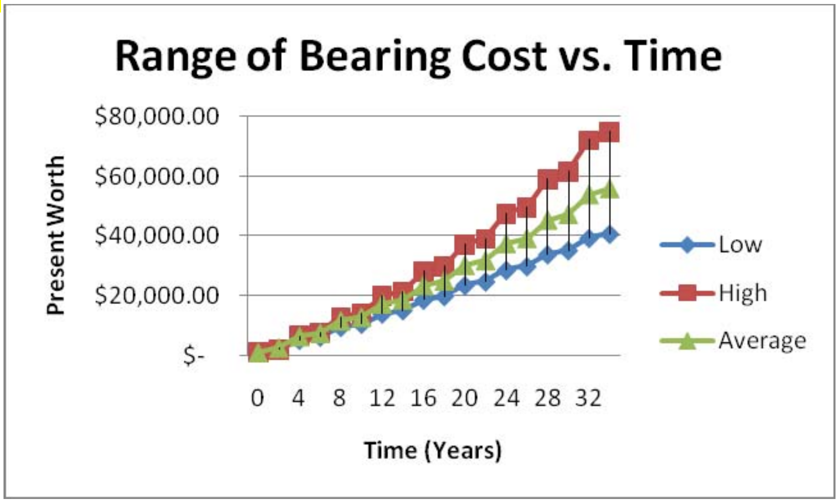
Average Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 2,131.81	$ 2,630.85
$ 5,212.37	$ 6,650.09	$ 6,425.43
$ 6,274.52	$ 7,850.61	$ 7,554.30
$ 9,525.69	$ 12,954.32	$ 11,668.25
$ 10,631.42	$ 14,310.40	$ 12,892.13
$ 14,015.95	$ 20,075.39	$ 17,352.34
$ 15,167.04	$ 21,607.17	$ 18,679.23
$ 18,690.41	$ 28,119.14	$ 23,514.84
$ 19,888.71	$ 29,849.39	$ 24,953.41
$ 23,556.61	$ 37,205.11	$ 30,196.00
$ 24,804.07	$ 39,159.55	$ 31,755.65
$ 28,622.42	$ 47,468.35	$ 37,439.50
$ 29,921.05	$ 49,676.02	$ 39,130.41
$ 33,896.04	$ 59,061.39	$ 45,292.64
$ 35,247.94	$ 61,555.12	$ 47,125.88
$ 236,632.71	$ 496,213.79	$ 342,993.38



High Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 2,131.81	$ 2,630.85
$ 5,212.37	$ 6,650.09	$ 6,425.43
$ 6,274.52	$ 7,850.61	$ 7,554.30
$ 9,525.69	$ 12,954.32	$ 11,668.25
$ 10,631.42	$ 14,310.40	$ 12,892.13
$ 14,015.95	$ 20,075.39	$ 17,352.34
$ 15,167.04	$ 21,607.17	$ 18,679.23
$ 18,690.41	$ 28,119.14	$ 23,514.84
$ 19,888.71	$ 29,849.39	$ 24,953.41
$ 23,556.61	$ 37,205.11	$ 30,196.00
$ 24,804.07	$ 39,159.55	$ 31,755.65
$ 28,622.42	$ 47,468.35	$ 37,439.50
$ 29,921.05	$ 49,676.02	$ 39,130.41
$ 33,896.04	$ 59,061.39	$ 45,292.64
$ 35,247.94	$ 61,555.12	$ 47,125.88
$ 39,385.98	$ 72,156.55	$ 53,806.75
$ 40,793.34	$ 74,973.38	$ 55,794.28
$ 244,077.82	$ 539,605.56	$ 363,050.01



The following tables present the results obtained by assuming High and Low Real Rate. The Present Worth estimates for Low, Average and high Expected Economic Life are also displayed below.

High Real Rate												
	Low Estimate				***High Estimate***				***Average Estimate***			
Years (x)	P/F,4%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,4%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,4%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0628	$ 1,062.81	$ -	$ -	1.0628	$ 2,125.62	$ -	$ -	1.062812	$ 1,594.22	$ -	$ -
4	1.1296	$ 1,129.57	$ 2,259.14	$ -	1.1296	$ 2,259.14	$ 3,388.71	$ -	1.12957	$ 1,129.57	$ 2,823.92	$ -
6	1.2005	$ 1,200.52	$ -	$ -	1.2005	$ 2,401.04	$ -	$ -	1.200521	$ 1,200.52	$ -	$ -
8	1.2759	$ 1,275.93	$ 2,551.86	$ -	1.2759	$ 2,551.86	$ 3,827.78	$ -	1.275928	$ 1,275.93	$ 3,189.82	$ -
10	1.3561	$ 1,356.07	$ -	$ -	1.3561	$ 2,712.14	$ -	$ -	1.356072	$ 1,356.07	$ -	$ -
12	1.4412	$ 1,441.25	$ 2,882.50	$ -	1.4412	$ 2,882.50	$ 4,323.75	$ -	1.44125	$ 1,441.25	$ 3,603.12	$ -
14	1.5318	$ 1,531.78	$ -	$ -	1.5318	$ 3,063.56	$ -	$ -	1.531778	$ 1,531.78	$ -	$ -
16	1.6280	$ 1,627.99	$ 3,255.98	$ -	1.6280	$ 3,255.98	$ 4,883.98	$ -	1.627992	$ 1,627.99	$ 4,069.98	$ -
18	1.7302	$ 1,730.25	$ -	$ -	1.7302	$ 3,460.50	$ -	$ -	1.73025	$ 1,730.25	$ -	$ -
20	1.8389	$ 1,838.93	$ 3,677.86	$ -	1.8389	$ 3,677.86	$ 5,516.79	$ -	1.83893	$ 1,838.93	$ 4,597.33	$ -
22	1.9544	$ 1,954.44	$ -	$ -	1.9544	$ 3,908.88	$ -	$ -	1.954438	$ 1,954.44	$ -	$ -
24	2.0772	$ 2,077.20	$ 4,154.40	$ -	2.0772	$ 4,154.40	$ 6,231.60	$ -	2.0772	$ 2,077.20	$ 5,193.00	$ -
26	2.2077	$ 2,207.67	$ -	$ -	2.2077	$ 4,415.35	$ -	$ -	2.207674	$ 2,207.67	$ -	$ -
28	2.3463	$ 2,346.34	$ 4,692.69	$ -	2.3463	$ 4,692.69	$ 7,039.03	$ -	2.346343	$ 2,346.34	$ 5,865.86	$ -
30	2.4937	$ 2,493.72	$ -	$ 356,602.19	2.4937	$ 4,987.44	$ -	$ 398,995.46	2.493722	$ 2,493.72	$ -	$ 377,798.83
32	2.6504	$ 2,650.36	$ 5,300.72	$ 379,001.16	2.6504	$ 5,300.72	$ 7,951.07	$ 424,057.24	2.650358	$ 2,650.36	$ 6,625.89	$ 401,529.20
34	2.8168	$ 2,816.83	$ -	$ -	2.8168	$ 5,633.67	$ -	$ -	2.816833	$ 2,816.83	$ -	$ -
35	2.9040	$ -	$ -	$ 415,265.01	2.9040	$ -	$ -	$ 464,632.18	2.903951	$ -	$ -	$ 439,948.59

Low expexted economic Life

Present Worth of Total Cost	**$ 406,420.10**	**$ 485,825.05**	**$ 434,016.74**

Medium Expected Economic Life

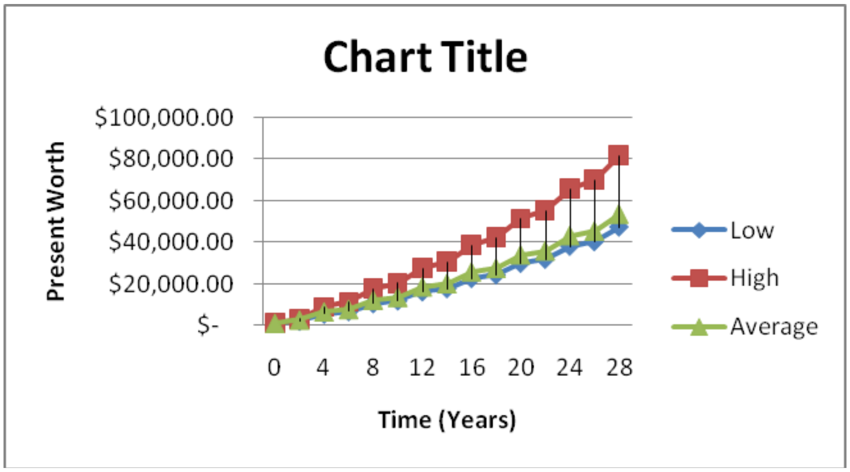
Present Worth of Total Cost	**$ 436,770.14**	**$ 524,138.63**	**$ 467,023.37**

High Expected Economic Life

Present Worth of Total Cost	**$ 475,850.82**	**$ 570,347.23**	**$ 508,259.59**

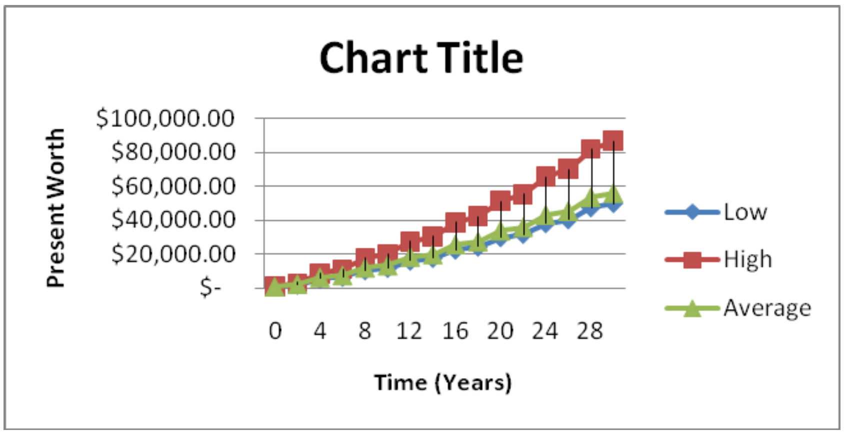
Low Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,131.81	$ 3,194.62	$ 2,663.22
$ 5,520.52	$ 8,842.47	$ 6,616.71
$ 6,721.04	$ 11,243.51	$ 7,817.23
$ 10,548.83	$ 17,623.15	$ 12,282.98
$ 11,904.90	$ 20,335.30	$ 13,639.05
$ 16,228.65	$ 27,541.54	$ 18,683.43
$ 17,760.42	$ 30,605.10	$ 20,215.20
$ 22,644.40	$ 38,745.06	$ 25,913.18
$ 24,374.65	$ 42,205.56	$ 27,643.42
$ 29,891.44	$ 51,400.21	$ 34,079.68
$ 31,845.88	$ 55,309.09	$ 36,034.12
$ 38,077.48	$ 65,695.09	$ 43,304.32
$ 40,285.15	$ 70,110.44	$ 45,511.99
$ 47,324.18	$ 81,842.15	$ 53,724.19
$ 406,420.10	$ 485,825.05	$ 434,016.74



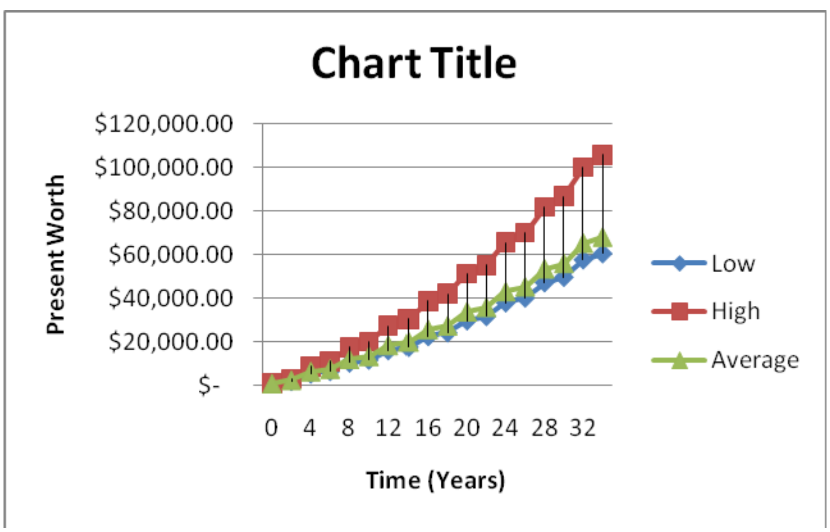
Average Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,131.81	$ 3,194.62	$ 2,663.22
$ 5,520.52	$ 8,842.47	$ 6,616.71
$ 6,721.04	$ 11,243.51	$ 7,817.23
$ 10,548.83	$ 17,623.15	$ 12,282.98
$ 11,904.90	$ 20,335.30	$ 13,639.05
$ 16,228.65	$ 27,541.54	$ 18,683.43
$ 17,760.42	$ 30,605.10	$ 20,215.20
$ 22,644.40	$ 38,745.06	$ 25,913.18
$ 24,374.65	$ 42,205.56	$ 27,643.42
$ 29,891.44	$ 51,400.21	$ 34,079.68
$ 31,845.88	$ 55,309.09	$ 36,034.12
$ 38,077.48	$ 65,695.09	$ 43,304.32
$ 40,285.15	$ 70,110.44	$ 45,511.99
$ 47,324.18	$ 81,842.15	$ 53,724.19
$ 49,817.90	$ 86,829.59	$ 56,217.92
$ 436,770.14	$ 524,138.63	$ 467,023.37



High Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,131.81	$ 3,194.62	$ 2,663.22
$ 5,520.52	$ 8,842.47	$ 6,616.71
$ 6,721.04	$ 11,243.51	$ 7,817.23
$ 10,548.83	$ 17,623.15	$ 12,282.98
$ 11,904.90	$ 20,335.30	$ 13,639.05
$ 16,228.65	$ 27,541.54	$ 18,683.43
$ 17,760.42	$ 30,605.10	$ 20,215.20
$ 22,644.40	$ 38,745.06	$ 25,913.18
$ 24,374.65	$ 42,205.56	$ 27,643.42
$ 29,891.44	$ 51,400.21	$ 34,079.68
$ 31,845.88	$ 55,309.09	$ 36,034.12
$ 38,077.48	$ 65,695.09	$ 43,304.32
$ 40,285.15	$ 70,110.44	$ 45,511.99
$ 47,324.18	$ 81,842.15	$ 53,724.19
$ 49,817.90	$ 86,829.59	$ 56,217.92
$ 57,768.98	$ 100,081.38	$ 65,494.17
$ 60,585.81	$ 105,715.05	$ 68,311.00
$ 475,850.82	$ 570,347.23	$ 508,259.59



Low Real Rate												
	Low Estimate				High Estimate				Average Estimate			
Years (x)	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC	P/F,1%,x	P.Worth IC	P.Worth MC	P.Worth RC
0	0.0000	$ -	$ -	$ -	0.0000	$ -	$ -	$ -	0	$ -	$ -	$ -
2	1.0203	$ 1,020.30	$ -	$ -	1.0203	$ 2,040.61	$ -	$ -	1.020304	$ 1,530.46	$ -	$ -
4	1.0410	$ 1,041.02	$ 2,082.04	$ -	1.0410	$ 2,082.04	$ 3,123.06	$ -	1.04102	$ 1,041.02	$ 2,602.55	$ -
6	1.0622	$ 1,062.16	$ -	$ -	1.0622	$ 2,124.31	$ -	$ -	1.062157	$ 1,062.16	$ -	$ -
8	1.0837	$ 1,083.72	$ 2,167.45	$ -	1.0837	$ 2,167.45	$ 3,251.17	$ -	1.083723	$ 1,083.72	$ 2,709.31	$ -
10	1.1057	$ 1,105.73	$ -	$ -	1.1057	$ 2,211.45	$ -	$ -	1.105727	$ 1,105.73	$ -	$ -
12	1.1282	$ 1,128.18	$ 2,256.36	$ -	1.1282	$ 2,256.36	$ 3,384.53	$ -	1.128178	$ 1,128.18	$ 2,820.45	$ -
14	1.1511	$ 1,151.08	$ -	$ -	1.1511	$ 2,302.17	$ -	$ -	1.151085	$ 1,151.08	$ -	$ -
16	1.1745	$ 1,174.46	$ 2,348.91	$ -	1.1745	$ 2,348.91	$ 3,523.37	$ -	1.174456	$ 1,174.46	$ 2,936.14	$ -
18	1.1983	$ 1,198.30	$ -	$ -	1.1983	$ 2,396.61	$ -	$ -	1.198303	$ 1,198.30	$ -	$ -
20	1.2226	$ 1,222.63	$ 2,445.27	$ -	1.2226	$ 2,445.27	$ 3,667.90	$ -	1.222633	$ 1,222.63	$ 3,056.58	$ -
22	1.2475	$ 1,247.46	$ -	$ -	1.2475	$ 2,494.91	$ -	$ -	1.247457	$ 1,247.46	$ -	$ -
24	1.2728	$ 1,272.79	$ 2,545.57	$ -	1.2728	$ 2,545.57	$ 3,818.36	$ -	1.272786	$ 1,272.79	$ 3,181.96	$ -
26	1.2986	$ 1,298.63	$ -	$ -	1.2986	$ 2,597.26	$ -	$ -	1.298629	$ 1,298.63	$ -	$ -
28	1.3250	$ 1,325.00	$ 2,649.99	$ -	1.3250	$ 2,649.99	$ 3,974.99	$ -	1.324996	$ 1,325.00	$ 3,312.49	$ -
30	1.3519	$ 1,351.90	$ -	$ 193,321.52	1.3519	$ 2,703.80	$ -	$ 216,303.80	1.351899	$ 1,351.90	$ -	$ 204,812.66
32	1.3793	$ 1,379.35	$ 2,758.70	$ 197,246.73	1.3793	$ 2,758.70	$ 4,138.04	$ 220,695.64	1.379348	$ 1,379.35	$ 3,448.37	$ 208,971.19
34	1.4074	$ 1,407.35	$ -	$ -	1.4074	$ 2,814.71	$ -	$ -	1.407354	$ 1,407.35	$ -	$ -
35	1.4216	$ -	$ -	$ 203,284.48	1.4216	$ -	$ -	$ 227,451.17	1.42157	$ -	$ -	$ 215,367.82

Low expexted economic Life

Present Worth of Total Cost	$ 228,569.46	$ 277,482.88	$ 244,694.65

Medium Expected Economic Life

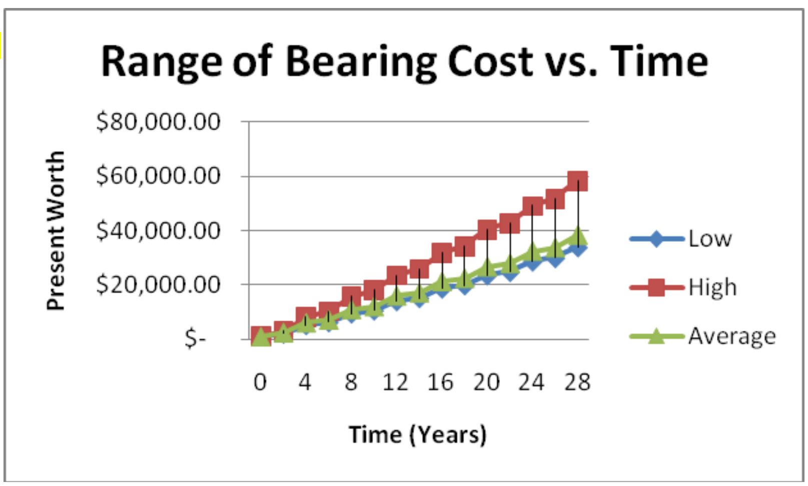
Present Worth of Total Cost	$ 236,632.71	$ 288,771.47	$ 253,680.89

High Expected Economic Life

Present Worth of Total Cost	$ 244,077.82	$ 298,341.70	$ 261,484.88

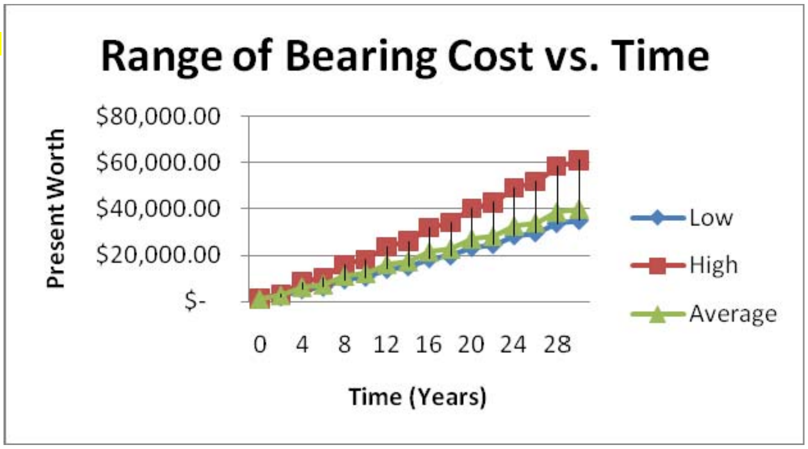
Low Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,109.61	$ 2,599.46
$ 5,212.37	$ 8,314.71	$ 6,243.03
$ 6,274.52	$ 10,439.02	$ 7,305.18
$ 9,525.69	$ 15,857.64	$ 11,098.22
$ 10,631.42	$ 18,069.10	$ 12,203.94
$ 14,015.95	$ 23,709.99	$ 16,152.57
$ 15,167.04	$ 26,012.16	$ 17,303.65
$ 18,690.41	$ 31,884.44	$ 21,414.25
$ 19,888.71	$ 34,281.04	$ 22,612.55
$ 23,556.61	$ 40,394.21	$ 26,891.77
$ 24,804.07	$ 42,889.12	$ 28,139.22
$ 28,622.42	$ 49,253.05	$ 32,593.97
$ 29,921.05	$ 51,850.31	$ 33,892.60
$ 33,896.04	$ 58,475.29	$ 38,530.09
$ 228,569.46	$ 277,482.88	$ 244,694.65



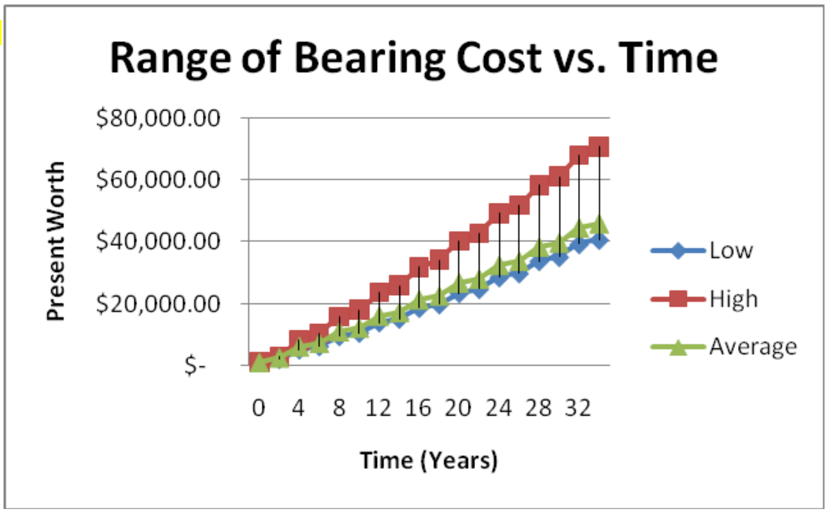
Average Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,109.61	$ 2,599.46
$ 5,212.37	$ 8,314.71	$ 6,243.03
$ 6,274.52	$ 10,439.02	$ 7,305.18
$ 9,525.69	$ 15,857.64	$ 11,098.22
$ 10,631.42	$ 18,069.10	$ 12,203.94
$ 14,015.95	$ 23,709.99	$ 16,152.57
$ 15,167.04	$ 26,012.16	$ 17,303.65
$ 18,690.41	$ 31,884.44	$ 21,414.25
$ 19,888.71	$ 34,281.04	$ 22,612.55
$ 23,556.61	$ 40,394.21	$ 26,891.77
$ 24,804.07	$ 42,889.12	$ 28,139.22
$ 28,622.42	$ 49,253.05	$ 32,593.97
$ 29,921.05	$ 51,850.31	$ 33,892.60
$ 33,896.04	$ 58,475.29	$ 38,530.09
$ 35,247.94	$ 61,179.09	$ 39,881.99
$ 236,632.71	$ 288,771.47	$ 253,680.89



High Expected Economic Life

Low	High	Average
$ 1,069.00	$ 1,069.00	$ 1,069.00
$ 2,089.30	$ 3,109.61	$ 2,599.46
$ 5,212.37	$ 8,314.71	$ 6,243.03
$ 6,274.52	$ 10,439.02	$ 7,305.18
$ 9,525.69	$ 15,857.64	$ 11,098.22
$ 10,631.42	$ 18,069.10	$ 12,203.94
$ 14,015.95	$ 23,709.99	$ 16,152.57
$ 15,167.04	$ 26,012.16	$ 17,303.65
$ 18,690.41	$ 31,884.44	$ 21,414.25
$ 19,888.71	$ 34,281.04	$ 22,612.55
$ 23,556.61	$ 40,394.21	$ 26,891.77
$ 24,804.07	$ 42,889.12	$ 28,139.22
$ 28,622.42	$ 49,253.05	$ 32,593.97
$ 29,921.05	$ 51,850.31	$ 33,892.60
$ 33,896.04	$ 58,475.29	$ 38,530.09
$ 35,247.94	$ 61,179.09	$ 39,881.99
$ 39,385.98	$ 68,075.82	$ 44,709.71
$ 40,793.34	$ 70,890.53	$ 46,117.06
$ 244,077.82	$ 298,341.70	$ 261,484.88



Calculations of Displacements to be accomodated by Bearings

(Option II, 3ft spacing, continuous, steel, composite)

Method A from AASHTO Bridge Design manual was used

$E = 29000$ ksi , $\varepsilon = 6.5 \times 10^{-6} \frac{in/in}{°F}$ (S 6.4.1)

Expansion Calculation:

$\Delta_T = \alpha L (T_{Max\,Design} - T_{Min\,Design})$ (S3.12.2.3-1)

$t_{set} = 60°F$ (Assumed steel girder setting temp. Temp Range = 0°F - 120°F)

$\Delta_{exp} = \varepsilon \Delta t (L_{span}) = (6.5 \times 10^{-6} \frac{in/in}{°F})(120°F - 60°F)(162 ft \times 12 \frac{in}{ft}) =$

$= 0.76$ in

Contraction Calculations: $\boxed{\Delta_T = 0.76 \text{ in}}$

$\Delta_{contr} = \varepsilon \Delta t L_{span}$

$\Delta t_{fall} = t_{set} - 0 = 60°F$

$\Delta_{contr} = (6.5 \times 10^{-6})(162 \times 12)(60 - 0) =$

$= 0.76$ in

Max. Horizontal Displacement of the bridge superstructure (Δ_o):

$\Delta_o = 0.65 \Delta_T$ (S 14.7.5.3.4)

$\Delta_o = (0.65)(0.76) = 0.5$ in

Total horizontal displacement to be accommodated by bearing $= \Delta_o + \Delta_T =$
$= \boxed{1.26 \text{ in}}$

Max. rotation to be accomodated by bearing $= \boxed{0.05 \text{ rad}}$