Project Number: LDA - 0903

Analysis of Bridge Connections

A Comparative Study of Bridge and Joint Design

A Major Qualifying Project submitted to the faculty of
Worcester Polytechnic Institute in partial fulfillment of the requirements for the Degree of Bachelor of Science in Civil Engineering by

Jennifer A Himottu

Jennifer Himottu

Allyson Stuer

Allyson Stuer

Approved:

Leonard D. Albano

Prof. Leonard Albano, Major Advisor

March 6th, 2009

Abstract

This project examined bridge design in terms of strength, cost and sustainability for the reconstruction of a bridge in Worcester, MA. Both a prestressed concrete and a steel girder bridge were designed and four different types of joints were analyzed through laboratory tensile testing. Additionally, one joint was compared through a computer software model analysis. The report culminates with conclusions and recommendations, determined by research and testing, for an appropriate solution for bridge redesign.

Capstone Design

As part of the Major Qualifying Project this report focuses on capstone design including the influences of economic, environmental, sustainability, constructability, ethical, and health and safety concerns. These considerations allowed for a full understanding of an engineering project from various aspects.

Economic Considerations

There is not an unlimited budget for projects including bridge replacement, and engineers must look to the cost effectiveness of their design. Not only should an engineer look at the initial construction cost of a bridge but also the potential maintenance that the bridge will need over its life. This Major Qualifying Project determined which type of joint and bridge is most cost effective for use in Massachusetts. Depending on geography different assemblies can require varied maintenance. In New England the snow and sanding can cause corrosion and introduce additional maintenance requirements to parts such as the joints. This was taken into account for determining cost effectiveness and maintenance costs that can be predicted. Also, the project's estimated construction cost will be compared to the available posted value for the actual project.

Environmental Considerations

Environmental considerations were examined so that the bridge structure does not make a significant impact on the environment in which it is constructed. This was addressed by recycling the materials used for joint tensile testing and examining computer analysis as an alternate method of testing. For this project 30 feet of steel angle was purchased as well as 6 square feet of steel plate. To help divert this waste from landfills, all of the steel was recycled after use except for a few joint models that were kept for presentation. By recycling steel, iron ore, coal, and limestone are all limited resources that are conserved (*Steel Recycling Institute*, 2009). In additional to recycling steel, computer analysis was examined to determine if it was a feasible comparison to the laboratory tensile testing results. By using computer analysis, fewer materials, and therefore, less waste are required for laboratory testing. Although two-thirds of steel material is typically recycled, the remaining amount of steel is mined from limited natural resources (*Steel Recycling Institute*, 2009). When computer analysis is used to obtain testing results, there is no material, and therefore no waste, requirement.

Sustainability Considerations

When communities plan the replacement of a bridge they also need to consider how sustainable it will be when proposing a future maintenance schedule and budget. The budget can be included as part of the overall town budget or a separate account can be set up in advance for the purpose of having a designated amount of money available to perform maintenance. If maintenance is not performed the life cycle of a bridge can shorten considerably, causing more sustainability problems. The purpose of maintenance is to ensure that the bridge is performing correctly and to find problems before failure. One example of a problem is joint corrosion, which can be alleviated initially by designing the appropriate drainage system. By finding the most appropriate materials and methods, maintenance can be reduced due to the versatility of the bridge. In this project, the most sustainable joint (or easiest to maintain) is recommended and the future bridge maintenance cost is explored.

At present, there are no sustainability standards (such as the LEED ratings for building design) for bridge construction. The United States Green Building Council (USGBC) awards LEED ratings to several types of buildings that implement a certain amount of energy and environmental design (*USGBC: U.S. Green Building Council*, 2008). Although the USGBC does not examine bridge construction, bridges have been constructed with similar LEED characteristics. The Eleanor Schonell Bridge in Brisbane, Australia is also known as the "Green Bridge." This bridge has taken a sustainable initiative by including the following (*Eleanor Schonell Bridge Project Overview*, n.d.):

- A more direct connection between destinations which reduces congestion on other streets,
- Bioretention ponds
- A solar roof for bridge lighting,
- Lanes for pedestrian and cyclist traffic.

By considering characteristics such as these during bridge design, bridges can become more self-sufficient while improving the surrounding environment. Within the conclusions and recommendations chapter, the idea of green design was examined, and the most feasible sustainability initiatives, such as the creation of a bike path were described.

Constructability Considerations

Constructability is the most important aspect of a design project. Bridge design includes a product of construction; therefore the constructability of the element must be considered. An engineer must determine how the design will be constructed—what materials will be used and how they will fit together with connections. In this project, we additionally considered how the connections were constructed for laboratory testing. In testing considerations it is important to be representative of the actual large-scale design; however, it is equally important to do this cost effectively while still gaining an accurate representation of the as-built structure.

There are several different manuals and inspection texts that apply to bridge construction. The texts that apply to different designs, depends initially on bridge location, type and purpose. In this project, a steel and concrete continuous girder bridge was designed to be constructed on Grafton Street over Route 20 in Worcester, Massachusetts. To aid in the design, the project team is mainly using the following references: *AASHTO LRFD Design Specifications*, *Mass Highway Bridge Manual*, *LRFD Design Examples* published by the FHWA, Mass Highway Bridge Standards and General Laws, respectively. As shown in the listed references, the bridge design must consider both federal and local laws and be able to apply to both.

Ethical, Health and Safety Considerations

As an engineer there is a responsibility to the public for the design to be to current standards. This is part of the ethical obligation to designing at any level. Each situation and project has a unique set of parameters and an engineer must ensure that the design is appropriate for the given use to provide safety to any users of the facility. By making a conscious ethical design a priority, the engineer is allowing for the health and safety considerations to be met. For the given project standards from AASHTO and the Massachusetts Highway Department were utilized.

Acknowledgements

Several parts of this project have never been undertaken before in a WPI Major Qualifying Project; therefore, we would like to take this space to thank those that have helped us take our ideas and goals in March of 2008 and achieve them within the next year.

The laboratory portion of our report required a lot of time and effort and could not have been done without the help of Don Pellegrino and Dean Daigneault. Don was always available (even through snowstorms) to help us use the Instron tester, or drive us in search of a machinist. Dean was constantly our go-to guy for all things machining. Thank you for dealing with all our joint design scribblings and re-scribblings as well as engineering proper molds for all 23 joints! Without Don and Dean, our testing portion—our favorite portion of this MQP—wouldn't have been a reality.

We would additionally like to thank the "providers." First, we want to thank Professor El Korchi for allowing us to purchase all the necessary steel and sealant as well as purchasing off-site machinist services. The sealant provider—Watson Bowman Acme Corporation—was kind enough to send us initial free sealant samples and lend us pneumatic caulking gun for as long as we needed it. Our off-site machining was unfortunately very last minute, and fortunately Tom Gravel of Hydro-Cutter was able to quickly and efficiently machine all our finger plate joints. Lastly, we want to extend our thanks to all users of Kaven Hall's basement throughout the past several months. Our supplies and joint materials have taken up a great portion of space, and we thank those who were able to work around our "mess."

Finally, and most importantly, we want to acknowledge and thank our faculty advisor—Professor Albano. Every Friday at 11AM you sat through our ramblings about our project and our ramblings about life and actually seemed to enjoy it. We can only imagine how much effort you put into editing *every* page of our report as we sent you drafts throughout the year. You have given us an immeasurable amount of advice for our project and throughout our past four years as WPI civil engineering students. We will miss our Friday morning meetings. Thank you.

Authorship

The sections of this major qualifying project report were contributed equally by Jennifer Himottu and Alyson Stuer. The following sections were written by the specified person:

Section	Author
Capstone Design	Alyson Stuer
Introduction	Jennifer Himottu
Background	Jennifer Himottu & Alyson Stuer
Methodology	Jennifer Himottu & Alyson Stuer
Design Analysis	Alyson Stuer
Laboratory Analysis	Jennifer Himottu
Computer Analysis	Jennifer Himottu
Cost, Funding, Maintenance	Alyson Stuer
Conclusions	Jennifer Himottu & Alyson Stuer
Recommendations	Jennifer Himottu & Alyson Stuer

Table of Contents

List of Figures

List of Tables

Nomenclature

AASHTO – American Association of State Highway and Transportation Officials

BMS – Bridge Management System

CAD – Computer Aided Design

DCR – Department of Conservation and Recreation

FHWA – Federal Highway Administration

LEED – Leadership in Energy and Environmental Design

LRFD – Load Resistance Factor Design

MHD – Massachusetts Highway Department

NCHRP – National Cooperative Highway Research Program

NEBT – New England Bulb Tee

NTRB – National Transportation Research Board

USGBC – The United States Green Building Council

WBA – Watson Bowman Acme

Chapter 1 – Introduction

A significant portion of Massachusetts's bridges were constructed in the early 1900s; because the state continually neglected maintenance of these bridges until it was absolutely necessary to address, there are currently over 500 structurally deficient bridges in Massachusetts. According to the state highway department, this number could increase to nearly 700 structurally deficient bridges by the year 2016 if no maintenance work is completed now (Nicodemus, 2008). In the case where the bridge is improperly maintained or inspected, the result may be bridge failure. Failure can be caused by several factors; however, the factors that are specifically observed in the following report are detail deficiency (such as joint connections) and inadequate maintenance. Recent tragedies such as the I-35 Bridge in Minneapolis have stimulated the transportation community to further address bridge connections and maintenance.

The national transportation community such as the National Transportation Research Board (NTRB) and the National Cooperative Highway Research Program (NCHRP) has been continually making advancements in bridge and connection design. Unfortunately, the funding and research increases most after a major failure occurs or as funding becomes available rather than consistently. In the past few years, the NCHRP has published numerous reports, including one on bridge deck joint performance and performance testing for joint systems. However, the research done by the NCHRP focuses on the entire system failure rather than focusing on specific types of joint failure.

This project simplifies the broad testing that the NCHRP has researched to specifically examine how the sealant performs within four types of bridge joints. The overall goal of this project was to focus on the four types of bridge joints in tension and examine two types of bridge design to address the project's capstone requirements. Additionally, as part of capstone design, the project considers topics of sustainability, the environment, health and safety, economics, and constructability.

As context for this project, the reconstruction of a bridge on Grafton Street over Route 20 in Worcester, Massachusetts was examined in terms of strength, cost and sustainability. To accomplish this, two different bridges were designed—a prestressed concrete and a steel girder bridge. Additionally, four different types of joints were analyzed through laboratory tensile testing, and one joint was investigated through a computer software model analysis. The computer model was then compared with the test data and observations. Resources used within this report include *AASHTO LRFD Bridge Design Specifications*, ANSYS software, and a series of bridge design and inspection manuals, in addition to several other sources relevant to bridge failure, cost estimation, and previous research in bridge design. Conclusions and recommendations determined by research and testing have provided an appropriate solution for bridge redesign.

Chapter 2 – Background

The topic of bridge design and construction is a popular discussion among the government and communities with the failures that continue to occur. In order to design and maintain a structurally sound bridge, the engineer must understand a variety of subjects. For the design phase, the engineer must have knowledge of bridge types, connection methods and how failures have occurred in the past. Once the bridge is constructed, the owner (i.e. the city government) must ensure that the bridge is maintained through inspection and maintenance practices. The following background study focuses on each of these topics to provide a base for the project—analyzing bridge design and connections.

A bridge must be designed in accordance with standards, and the components, bearings, joints, girders, deck, etcetera, serve to various capacities depending on the potential usage situations presented. From bridge superstructure selection to the modification of expansion joints, each part helps to keep the collective whole of the bridge safe for the public while connecting roadways over a variety of terrain. When a bridge fails to meet its given purpose, such as exceeding its designed maximum deflection while carrying the weight for which it was designed, it is imperative to look back and learn where things went wrong in the process—what physical component(s) failed and how it could have been prevented. Whether something was chosen in error or lack of maintenance caused the problem, it must be investigated to prevent future repeats of these events.

2.1 Types of Bridges

There are several types of bridges used throughout the world; however, the two types of bridges that will be examined within this project are concrete and steel girder bridges. These bridges are most typical of those seen as overpasses on the highways in the region. Girder bridges typically are designed to be incorporated with the roadway. This can cause the bridge to go unnoticed by several travelers, while arch and suspension bridges may receive more publicity and notice due to greater size and cost. Girder bridges are typically less than 50 meters in span but range up to approximately 150 meters. They have developed over time into the current bridge design standards and visual appearance. Originally, because the spans were limited to be short, the bridges did not have the aesthetically pleasing look of the other bridges; over time this has been alleviated with the girder geometry and construction methods being developed (Barker, R. & Pucket, J., 2007). Girder designs are less effective in resisting loads in comparison with truss bridges of longer spans, but because of the stiffness the girders provide, they are more commonly used to reduce vibrations (Barker, R. & Pucket. J., 2007). Although these types of bridges are numerous throughout United State's roadways, people may not realize the vital role they play in the transportation system because of their commonality.

Steel girder bridges became popular in the nineteenth century while the reinforced concrete type was not used until the middle of the twentieth century. The steel girder bridge designed in this project is a plate girder, which has the steel plates connected in a variety of fashions including welds, bolts and rivets. This design can be modified in several ways to ensure that excess material is not used which allows for weight and cost reductions. The reinforced concrete girder bridge also has a variety of shapes and sizes for the appropriate amount of materials to be used. This design takes the strength of concrete in compression when loaded and the strength of steel in tension when loaded to create a combined beam. Each bridge type has applications and is most cost-effective in certain scenarios. The goal of designing both types of bridges is to investigate the differences in cost and constructability.

In Massachusetts, for a span of the length being designed (less than 100 feet) there is a small variety of girder bridges used. These include the following:

- Adjacent prestressed concrete,
- Spread prestressed concrete,
- Steel stringer and prestressed concrete NEBT girders with a composite concrete deck,
- Special prefabricated bridge panels with concrete decks and steel beams.

Bridges that are placed in the 100 to 140 feet length range include the following:

- Steel plate girder and steel box girders with composite concrete deck
- Prestressed concrete NEBT girders with a composite concrete deck

This report focuses on a near 100-foot span steel girder bridge that needs to be replaced. The designs for replacement model both a concrete and steel a single span bridge. Because the bridge span is nearly 100 feet, the types of bridges between 100 and 140 feet in length were examined because they are less complex to design, the beams can be shipped to the construction site in one piece, and the beam sizes do not change throughout the span. These must be used in accordance with the specifications given in the *Massachusetts Highway Department Bridge Manual* outlined in Section 2.6 of this report (Mass Highway, 2005). A variety of other bridge types can be considered for spans of other lengths but these also have specific guidelines. The individual use of the bridge usually determines the type that will be designed and constructed.

2.2 Types of Joints

Joints are used to connect bridge spans and to connect bridges to the roadway. These joints serve the purpose of accommodating expansion and contraction of the bridge with temperature fluctuations while providing a smooth connection between the deck of the bridge and the roadway. Through the construction of these joints a variety of movements of a bridge can be offset. Joints also must be designed to allow for minimal penetration of fluids through the joint which can contribute to the deterioration of the substructure; therefore, if a joint is open (meaning fluids and debris can fall within the joint) there must have an appropriate drainage system installed to properly remove fluids from the substructure.

There are two main categories of bridge joints—open and closed. An open joint does not protect the substructure below the joint from water and debris whereas a closed joint protects these components. The type of joint used is determined by the designer to follow the given specifications. The information for the joints is found in the drawings located in Part II of the *Mass Highway Bridge Manual*; in the *Mass Highway Bridge Manual* Part I, little is stated as direct specifications of these joints. These depictions of selected joints have a limited selection of notes from which to determine the joint construction methods.

2.2.1 Open Joints

Open joints are expansion joints that contain an opening between the deck and the substructure. Because of this opening, the joints can allow water and corrosive contamination to pass through; however, some open joints provide greater longitudinal movement than closed joints provide. Unfortunately, the opening can accelerate degradation of the bridge deck, bearing, and substructure elements which is why open joints are rarely used in new construction (Milla, Shaw, et. al., 2007). The open joints that provide a wide span of longitudinal movement are those that consist of fitting plates, such as the finger joint and sliding plate joint—the two types of open joints that were examined in this project (Tonias, D., & Zhao, J., 2006).

2.2.1.1 Finger Plate Joint

One joint being examined is the finger plate joint, shown in Figure 1, which is used when up to 2 feet of movement is expected. When a large amount of expansion can be expected, the finger plate joint is ideal because it provides the largest expansion out of the four joints this project examines. The finger plate joints are considered open joints, and therefore a drainage system must be installed underneath the joint to protect the substructure. However, the drainage system can become clogged and fail to function if it is not maintained properly. The finger plate joint has several known problems including differential settlement that can lead to the locking of the joint making it ineffective. Differential settlement additionally creates difficulty for bicycle and motorcycle travel, and snow plows can easily damage the joint (Tonias & Zhao, 2006). These problems will be looked at in detail during the laboratory analyses.

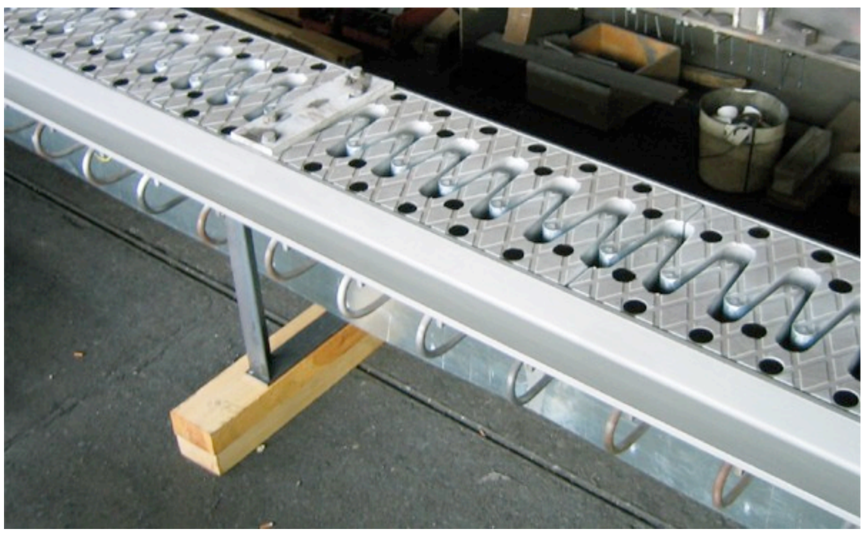


Figure 1: Finger Joint (*Structurae: TENSAFINGER Type RSFD*, 2008)

2.2.1.2 Sliding Plate Joint

The sliding plate joint has a plate that covers the joint and has a gap to allow for movement as shown in the schematic in Figure 2. This joint requires maintenance because if the gap is filled with debris, it does not allow for the sliding which makes the joint completely ineffective. This joint is similar to the finger joint since it also needs a drainage system underneath to provide protection to the substructure. The constraint of movement is greatly different from the finger joint because it can only permit movements up to four inches (Tonias & Zhao, 2006).

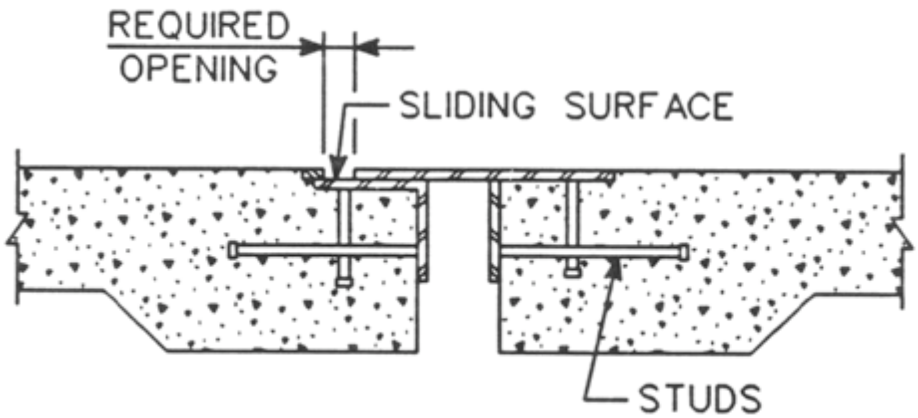


Figure 2: Sliding Plate Joint (*Bridge Deck Joint Performance*, 2003)

2.2.2 Closed Joints

Closed expansion joints, unlike open joints, can prevent the runoff from going to substructure elements; although, they have generally failed to show durability and longevity compared to the other noted joints (Tonias & Zhao, 2006). The substructure is protected by the layer of connecting sealant that is within the joint; however, the sealant can easily deteriorate over time from collecting water and debris. Because of the connecting sealant, the closed joints may need to be replaced or maintained more often than the noted open joints (Tonias & Zhao, 2006). In this project, two types of closed joints were examined—the strip seal joint and the compression joint.

2.2.2.1 Strip Seal Joint

The strip seal joint, shown in Figure 3, is the most simple of the joints. An elastomeric material or sealant is placed between two steel rails that are anchored to the deck mechanically. These joints allow for approximately 4 inches of movement and last anywhere from 10 to 20 years. This joint has an additional backer rod to serve as a support to the joint overall (Tonias & Zhao, 2006).

Figure 3: Strip Seal Joint (*Bridge Deck Joint Performance*, 2003)

2.2.2.2 Compression Joint

The compression joint consists of a sealant, sometimes with an open cross sectional strip, and angles for protection of the joint. Figure 4 shows an example of a compression joint used in the field. These are used for expansions between 0.5 inches to 2.5 inches as well as compressions. The space between the joint is filled with silicone sealant, similar to a rubber material, which can be both stretched in tension, and compressed. The main mode of failure in this joint is loosening due to a release of compression or loss of adhesion to the bridge surfaces. Additionally, the compressive loading can force the sealant to extend vertically above the deck surface. This extreme vertical expansion can cause damage by traffic. The compression joint is more limited than the strip seal joint in movement but is simpler to make because of the lack of the backer rod. The overall lifetime of the compression joint is from 10 to 15 years. (Tonias & Zhao, 2006). The laboratory testing of this joint will show how the sealant adheres to the steel angles in tension (Section 5.2), and the computer analysis will briefly focus on the joint in compression (Section 6.4).

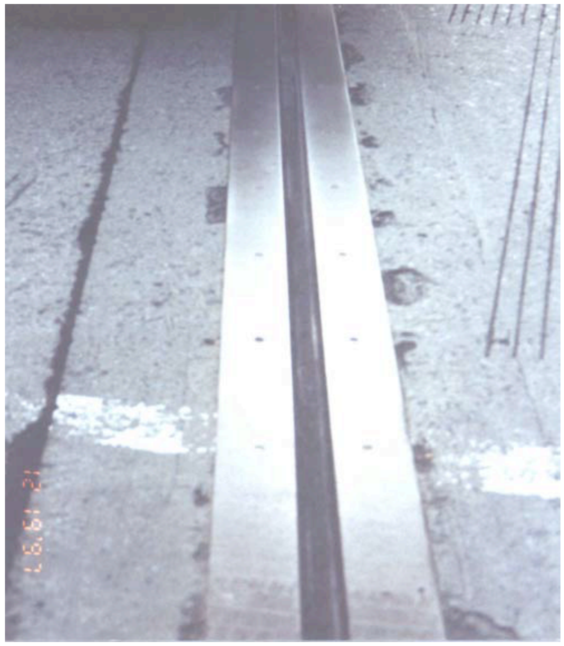


Figure 4: Compression Joint (*Bridge Deck Joint Performance*, 2003)

2.3 Silicone Sealant

This project focused on using a type of silicone sealant bond in each expansion joint because it would protect the substructure as well as adhere to the steel angles forming the joint. The silicone sealant selected for this project is provided by the WBA Corporation—the Wabo Silicone Seal. Other joint sealants exist, but would not be ideal in the conditions for this project (mentioned in the next paragraph). For example, the WBA Corporation also provides the following bridge joint seals:

- Jeene – Bridge sealing system
- Wabo Compression Seal Bridge Series
- WaboEvazote UV
- Wabo H Seal
- WaboInverSeal

Typically all sealants are created with silicone or neoprene—two materials representing similar characteristics of rubber. A comparison of the two materials is shown in Table 1. Silicone performance betters neoprene in characteristics of heat aging, dialectic strength, and temperature extremes. Neoprene performance betters silicone in characteristics of elongation, flame and tear resistance, and abrasion resistance.

Table 1: Comparison of Material Characteristics (*Elastomers*, 2009)

Characteristics	Neoprene	Silicone
ASTM D-2000 Classification	BC	GE
Elongation	Excellent	Fair
Heat Aging	Very Good	Excellent
Flame Resistance	Good	Fair
Tear Resistance	Good	Poor
Abrasion Resistance	Excellent	Poor
Compression Set Resistance	Fair	Fair
Dialectic Strength	Fair	Good
Max Temperature (F)	269	559
Min. Temperature (F)	-50	-150

Several of the sealants (other than Wabo Silicone Seal) provided by WBA Corporation use a neoprene seal that is connected to the joint with an adhesive. For example, the Jeene-Bridge sealing system includes a pre-made seal of neoprene, which is then placed within the joint and connected with an epoxy seal as shown in Figure 5. Although this system is also recommended for expansion joints within bridges (like the Wabo Silicone Seal), it would require examining two materials (the neoprene seal and the epoxy adhering it to the joint) in the laboratory and computer analysis sections of this report (*WBA Corp*, 2007).

Figure 5: Jeene-Bridge Sealing System (*WBA Corp*, 2007)

An appropriate sealant to be used in this project needed to be researched for the joints to be modeled as well as the geographic location which the bridge would be designed for. Because of the extreme weather variations in the Northeast, both expansion and contraction occurs regularly in outside materials including bridge joints. The chosen sealant of Wabo Silicone Seal is an approved adhesive by the Mass Highway Department; additionally, it is recommended by the WBA Corporation for expansion joint applications with a large movement range (+ 100% / -

50%) (*WBA Corp*, 2007). Table 2 organizes the physical properties of the sealant. All additional properties of the Wabo Silicone Seal can be found in Appendix B (*WBA Corp*, 2007).

Table 2: Wabo Sealant Properties (*WBA Corp*, 2007.)

Physical Properties

PHYSICAL PROPERTIES	ASTM TEST METHOD	PART A	PART B
Color		White	Gray
Viscosity		88,000 cps	34,000 cps
Leveling	C 639	self levels	self levels
Extrusion rate ml/min.	C 1183	200-600	200-600

PHYSICAL PROPERTIES	ASTM TEST METHOD	REQUIREMENTS
Leveling	C 639	self levels
Tack free time	C 679	60 minute max.
Joint elongation	D 5329 (1) (2)	600% min.
Joint Modulus, 100%	D 5329 (1) (2)	15 psi (.10MPa) max
Cure evaluation	D 5893	Pass @ 4 hrs, max
Ultimate Elongation	D 412 Die C (1)	1000% min.
Stress @ 150%	D 412 Die C (1)	25 psi max. (.17 Mpa)
Shore Hardness, 00	C 661 (1)	40 - 80
Specific Gravity	D 792 (1)	1.20 - 1.40
(1) Specimens cured at 77 +/-3 F and 50 +/-5% R.H. For 7 days (2) Specimens size is 1/2"wide x 1/2" deep x 2" long.		

2.4 Failures

Bridge failure today occurs most times due to "design methods that were not sophisticated enough to account for subtle conditions and secondary loads" (Carper, K. & Feld, J., 1996). Because bridge design includes hundreds of factors, equations, and loading situations, there are several areas that could be incorrectly designed. Other main types of failure include (Carper, K. & Feld, J., 1996):

- Inadequate wind and thermal effects
- Detail deficiency (within joints, bearings, welds, etc)
- Impact loading from collisions
- Inadequate maintenance

This project is focused on the failure surrounding details such as joints. Failure has occurred for every type of joint; therefore, there is no "perfect" joint that will automatically result in a structurally efficient bridge (Carper, K. & Feld, J., 1996). It is the job of the engineer to design appropriately, and the job of the community to maintain the bridge to avoid any potential failure.

2.4.1 Failures Examples

Widely known bridge failures typically occur in large scale bridge designs, because, unfortunately, they have the ability to cause a large number of fatalities due to size. Although bridge failures can be tragic to families as well as government budget, they have been the main reasons why the standards of inspection and design continually are changing to encompass all areas of where and how possible failure can occur. One example of this reactive approach to bridge regulations is the aftermath of the failure of the Hackensack River Bridge in New Jersey (1928). The cause of the failure was attributed to unaccounted dynamic effects in a moving structure. The bridge was a drawbridge; however, it was not designed as a dynamic object. Since this failure, dynamics and the science of vibrations have been included within the development of bridge engineering.

The National Bridge Inspection Standards were passed by Congress after the Silver Bridge in West Virginia collapsed in 1967. The collapse was caused due to an eyebar failure. Because of the bridge's specific connection detail, inspection of the eyebar would be impossible. Consequently, bridges must now be designed with appropriate accessibility and inspection methods.

Methods of redundancy were highlighted after the Mianus River Bridge in Connecticut collapsed in 1983. Fractured pins due to rust caused all loading to shift to one pin on one of the expansion joints. The rust occurred from ten years of blocked drains, and the bearings were difficult to view during inspection. The reaction from this failure made designers realize the importance of redundancy throughout bridge design. Having multiple girders and beams allows loads to be more distributed and offers multiple load paths; therefore, when one assembly fails, the load will not immediately fall upon one critical element. The collapse also notified states of the need for more engineers to be available for inspection. Connecticut was then in need of thousands of bridge inspections with only a few engineers (Carper, K. &Feld, J., 1996).

2.4.2 Implications of Failures

Bridge failures though often tragic and devastating demonstrated the need for standards to be established across the board. These first standards were produced for railway bridges and then carried into the highway and automobile bridges as the awareness of perceived need for public safety increased. In 1912 the U.S. Department of Agriculture, Office of Public Roads made the original effort for defining loads, materials, and design procedures for highway bridges with the publication of its Circular No. 100, *Typical Specifications for the Fabrication and Erection of Steel Highway Bridges* (Barker, R. & Puckett, J. 2007). Over time, the Office of Public Roads has evolved into the Federal Highway Administration (FHWA), which continues to aid in the regulation of bridge design methods, materials, and loading. Presently, AASHTO (the nonprofit association representing state highway and transportation departments) controls most of the bridge design criteria and issued the first edition of the *Standard Specifications for Highway Bridges* in 1931 (Barker, R. & Puckett, J. 2007). These specifications have been revised several times and are now at the 17th edition to show the development of the highway bridge design practices.

In addition to the standards created in response to bridge failures, the study of failures can teach the engineer about performance and phenomena. Lessons can include how natural disasters impact certain bridges, how flaws can be recognized, and how human incidents such as car accidents can affect a bridge.

2.4.3 Preventative Measures

By observing the problem before failure, both bridges and possibly lives can be saved. Once a bridge has been inspected and determined structurally unstable, it will be closed until the structural capacity is regained through rebuilding or retrofitting. Inspecting, as well as closing a bridge, typically costs time and money for the community. This is one of the reasons why, unfortunately, bridges are not inspected as much as necessary. Inspection and maintenance are vital to increase the lifespan of the bridge and continue to keep the community safe. They are further explained in the following sections.

2.5 Maintenance

The failures that occur usually stem from a problem within the maintenance and preservation of the structure. The collapse of the I-35W bridge in Minneapolis brought attention to the lack of inspection and maintenance on bridges throughout the United States. The Boston Globe read, "Local transportation specialists and engineers said the Minnesota collapse, while still under investigation, underscored the dangers of delaying maintenance on critical bridges …" (Ebbert, 2007). Massachusetts, specifically, has an old infrastructure system with 588 bridges listed as structurally deficient in February 2007 that will not be maintained until funding is available (*Boston Globe*, 2007). According to Stephanie Ebbert, writer for the *Boston Globe*, "Massachusetts has one of the oldest transportation infrastructures in the nation, including 200 bridges that were built in the 19th century… About a third of the state's bridges were built between 1900 and 1950, and 42 percent were built between 1950 and 1970" (Ebbert, 2007). The structurally deficient bridges will be maintained, but, unfortunately, only as the funding becomes available.

Many times disrepair and the delay of maintenance are correlated to the lack of funding available. In addition to the direct costs for manpower and materials, the bridge repair process is lengthy in the design phases and can be extremely expensive. In the 1950s, new structures were designed with 10 or 11 plan sheets; whereas presently, nearly 40 sheets are used for repairs (Tonias & Zhao, 2006). With the growth of regulations both nationally and at the state level it has become a large undertaking to organize and initiate maintenance projects.

Today many projects use "an integrated design-maintain-rehab approach" to relate the design to the rehabilitation and maintenance since many designers see them as completely separate. Not only do bridges need to be designed to carry loads but also to last for an extended lifetime. This works hand in hand with the sustainability issue now being raised. With less maintenance costs due to better design comes a savings to the party responsible for roadway upkeep. (Tonias, Zhao, 2006).

2.6 Mass Highway Bridge Standards

Bridge design in Massachusetts relies heavily on specifications prescribed in the *Massachusetts Highway Department Bridge Manual* (Mass Highway, 2005). It is necessary that this manual be continually referenced to ensure all guidelines are being followed. The Massachusetts Highway Department takes the AASHTO national standards and modifies them further to apply to the location and become more descriptive. This allows for a final appropriate design to be established. Because the AASHTO standards cover a variety of environmental differences, it is an appropriate guide throughout the United States.

The *Massachusetts Highway Department Bridge Manual* is broken into two main parts: design guidelines and standard detail drawings (Mass Highway, 2005). Within the design guidelines there are major sections including site exploration, engineering guidelines, design guidelines, construction drawing standards, then post design estimation and construction. These sections provide an overview of the general parameters needed for bridges with specific cases cited for various design elements. Also, in the second part of the manual, drawings detail more of the specifications than the written, prescriptive elements that comprise the first part of the manual. These drawings are used to guide designers visually. A design that is considered to meet public safety standards can be developed with the bridge manual (Mass Highway, 2005).

2.7 Bridge Inspection

Bridge inspection details as described in the *Massachusetts Highway Department Bridge Manual* are vague; but the manual does provide an overall inspection survey that rates each component of the bridge (Mass Highway, 2005). This allows the engineer to compare the current conditions of the bridge to those from the original design and construction. According to the Massachusetts Highway Department "The main purpose of a bridge inspection is to assure the safety of a bridge for the travelling public by uncovering deficiencies that can affect its structural integrity. The results of a bridge inspection are used to initiate maintenance activities and/or a load rating" (Mass Highway, 2005). These inspections become a fundamental part of the bridge maintenance prioritization since they are the only way of knowing the present deficiencies or flaws in a bridge.

In the United States, inspections have two sets of standards—federal and state regulations. State regulations must have the federal regulations as initial boundaries, and can then add state-specific stipulations. Typically the standards are inspected once every two years with procedural aid of the *Bridge Inspector's Reference Manual* and the *AASHTO Manual for Condition Evaluation of Bridges*. The Transportation Research Board defines eight types of inspections that are shown in Table 3. These inspections each occur depending on the situation. A routine inspection is done consistently to view the entirety of the bridge, and other inspections such as initial inspections and special inspections are completed at the beginning of the bridge's life and the discretion of the owner, respectively.

Table 3: Inspection Types (*Bridge Inspection Practices, 2007)*

Inspection Type	Description
Damage Inspection	An unscheduled inspection to assess structural damage resulting from environmental factors or human activities
Fracture Critical Member Inspection	A hands-on inspection of a fracture-critical member or member components that may include visual and other nondestrictive evaluation.
Hands-On Inspection	Inspection within arm's length of the component. Inspection uses visual techniques that may be supplemented by NDT.
In-Depth Inspection	A close-up inspection of one or more members above or below the water level to identify any deficiencies not readily detectable using routine inspection procedures; hands-on inspection may be necessary at some locations.
Initial Inspection	First inspection of a bridge as it becomes a part of the bridge inventory to provide all structure inventory, appraisal data and other relevant data, and to determine baseline structural conditions
Routine Inspection	Regularly scheduled inspection consisting of observations/measurements needed to determine the physical and functional condition of the bridge, to identify any changes from initial or previously recorded conditions, and to ensure that the structure continues to satisfy present service requirements.
Special Inspection	An inspection scheduled at the discretion of the bridge owner, used to monitor a particular known or suspected deficiency.
Underwater Inspection	Inspection of the underwater portion of a bridge substructure and the surrounding channel that cannot be inspected visually at low water by wading or probing, generally requiring diving or other appropriate techniques.

Most of these inspections listed by the Transportation Research Board are based on visual inspection (with the exception of the hands-on inspection)—inspecting bridge elements by sight with the visual knowledge of a passing and a failing inspection. However, visual inspection can limit description of the inspection because the entirety of the bridge may not be seen. This type of inspection is most suitable recognizing visible deterioration of components that are typically beyond easy repair. Presently, limitations of the current visual inspection practice are becoming better known and new methods are being developed. With the advancing technology other methods have been developed to aid in the visual inspection such as nondestructive testing (Wang, Swanson, et.al., 2007).

2.8 Summary

The previous section is preparatory to understanding this specific project—bridge design and laboratory analysis of bridge joint connections. The project was initiated due to the large number of structurally unsound bridges throughout the United States and specifically Massachusetts. By understanding failure patterns as well as bridge and connection types, the project team developed two bridge designs to compare the cost amount and constructability ease, and further study joint connections in a laboratory and computer analysis experience.

Chapter 3 – Methodology

This project included a variety of areas of study ranging from research to design and laboratory analysis. The project scope is shown in Figure 6. Within each area is a method used to contribute to the overall design project. The research area needed to allow for a comprehensive understanding of bridge failure and the current standards. From this research a focus could be made on the testing and analysis that was to be performed and would eventually lead to the development of a method for the lab and computer simulation. In addition the research guided a design process that was then followed in the design of the two bridges. These procedural steps give an overall understanding of the goals and outcomes for the project.

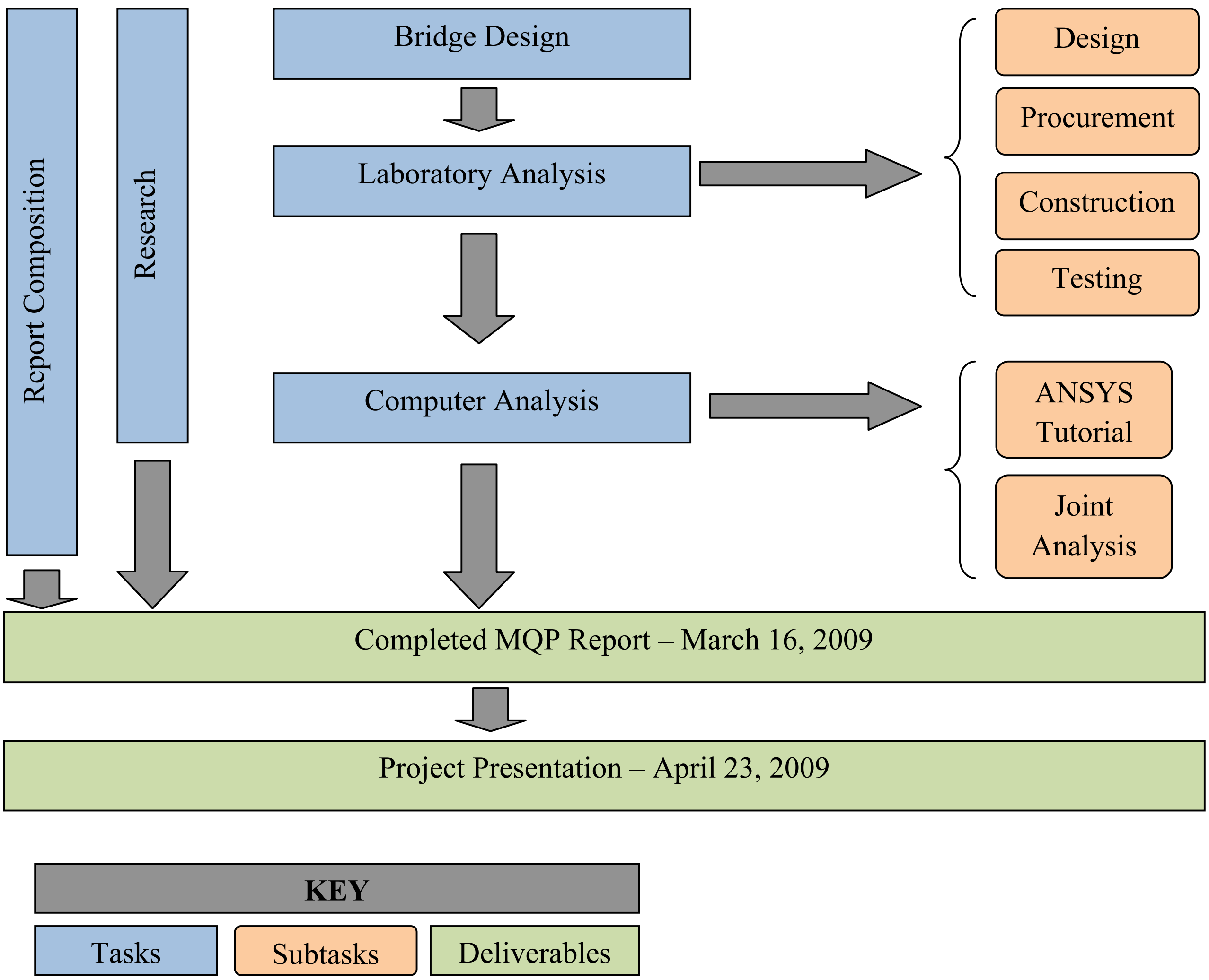


Figure 6: Scope Flowchart

3.1 Background Research

The background research included many textbook, journal articles, and other website references. This basic library style research was the first step to the development of the further background needed. To understand the design process initial research was conducted to determine both national and more local standards. These standards, *AASHTO LRFD Design Standards* and the *Massachusetts Highway Department Bridge Manual*, had many common components and were sorted to create a basis for design. Within researching the history of the bridge design it was discovered that the initial design was trial and error with failure showing the engineers how to design better (Barker, R. & Pucket, J., 2007). From these failures designers began to understand bridge design and developed a series of standards that have evolved to the current ones in use. This basis with the recent failures of bridges led to the focus of finding failure methods, specifically those in the expansion joints leading to the research to develop the lab and computer models to be tested and the tests to be performed. This research enabled the overall development of the design project into the focus needed.

Once the overall educational research was completed, research was done to find a plausible case using the Massachusetts Highway Department bid website. The parameters found were crucial to creating a realistic case study with capstone development aspects such as cost and constructability included.

3.2 Bridge Design

The design of the two bridges followed LRFD, AASHTO, and MassHighway consideration. The requirements studied included type of bridge used for the span, the capacity based on factored loading, and limits of tension and compression forces. Today designs use a load and resistance factor method to ensure over estimation of loads and underestimation of capacities resulting in a probabilistic and consistent approach to structural safety. The government has hired several contractors to develop sampled designs meeting the various specifications seen in the heavy construction industry. Two of these designs were used as the original basis for the design completed from the FWHA (*LRFD Design Examples*, 2006). These designs were supplemented by various text examples, design aids, and prior class knowledge to establish a fully understood design within the scope.

The design performed was a noncomposite design to determine how the deck would be affected by the girder sizing. The process began by using the parameters researched for the replacement of the Grafton Street Bridge over Route 20 including that the replacement be similar to the existing structure which determined the deck and girder basic geometries. Also a variety of parameters such as the parapet and overhang considerations were adapted from the sample designs to fit the proposed bridge design. In order to develop accurate loading combinations for the bridge a mix of AASHTO design guidelines were used for the deck design and analytical techniques were employed for the girder design to determine internal forces and moments. Various loadings were considered and factored moments were found using the basic methods of analysis including statics and the moment distribution method. These were used when design tables from the AASHTO design guide were not applicable. The girder design was completed using assumptions about the deck design then the deck was designed using the geometry of the girders. Each of these processes was iterative with different variations in the steel girder design and the prestressed concrete girder design arising. Once the trial and error of design was completed for the deck and girders to make one complete depiction of the bridge then the bearings were proportioned. The bearing design was simply completed to get an idea of the sizing of typical bearings for the selected designs.

Structural design was not completed for all structural components of the bridge but was focused on the superstructure, the girder and the deck, with a bearing design. The focus was narrowed to allocate time to study and complete laboratory testing of expansion joints, which occur in the deck. The design also provided a general cross section as well as a series of figures to depict the structure. Through the design iterations various limiting parameters were identified and design changes were made, showing the need for adaptability of a preliminary design.

3.3 Laboratory Testing and Analysis

The laboratory testing and analysis process occurred throughout the three terms that the Major Qualifying Project was completed. The types of joints to be tested were researched and an initial design was drawn in AutoCAD. Materials such as steel angles, plates and silicone sealant were procured and the joints were constructed with varying modifications from their initial designs. Tensile testing occurred to monitor how joint silicone sealant fails in four different joint types. All testing took place from December 2008 until mid-February 2009. The output data was copied into Microsoft Excel to analyze and is further explained in Chapter 5 of this report.

3.3.1 Expansion Joint Design

This project focuses on four joint types—strip seal, finger plate, compression, and sliding plate. Several other bridge joints exist, such as integral abutment or polymer modified asphalt joints; however, the four used were consistently shown and described throughout literature review, are constructed in the Worcester area, and use materials that were able to be procured. A summary of joint performance described in Section 2.2 is shown in Table 4. Despite the two

Table 4: Joint Performance (Tonias& Zhao, 2006)

Joint Category	Joint	Permitted Maximum Movement	Additional Notes
Open Joints	Finger Plate	2 feet	Known problems can cause differential settlement leading to joint locking
	Sliding Plate	4 inches	Maintenance is necessary for required opening to remain clean
Closed Joints	Strip Seal	4 inches	10-20 year lifespan, backer rod serves as an overall support
	Compression	2.5 inches	10-15 year lifespan, main failure is loosening from compression/adhesion to bridge surface loss

categories of open and closed joints shown in this summary, all the joints tested in this project would be considered closed. Because the reaction of the silicone sealant was being tested within these four different designs, each joint was designed and constructed to be mostly filled with sealant, therefore becoming a "closed joint", before testing.

The designs of the testing joints were initially based on a variety of plans found in the literature review. These designs were studied and simplified to test the principle failure phenomena of the connecting silicone sealant. In addition to literature research, joints in local roadways were studied and viewed to determine some of the characteristic dimensions in application. *Bridge Engineering* was the primary resource for this project's joint design (Tonias, D. & Zhao, J., 2006). The initial designs shown in Figure 7 were created depending on the text's joint schematics, field research, testing setup, and materials available. The "full scale" joints indicate that they are designed with similar dimensions of a bridge joint in the field; whereas, a "half scale" joint is approximately half the size of a realistic bridge joint in anticipation of available material and testing capabilities.

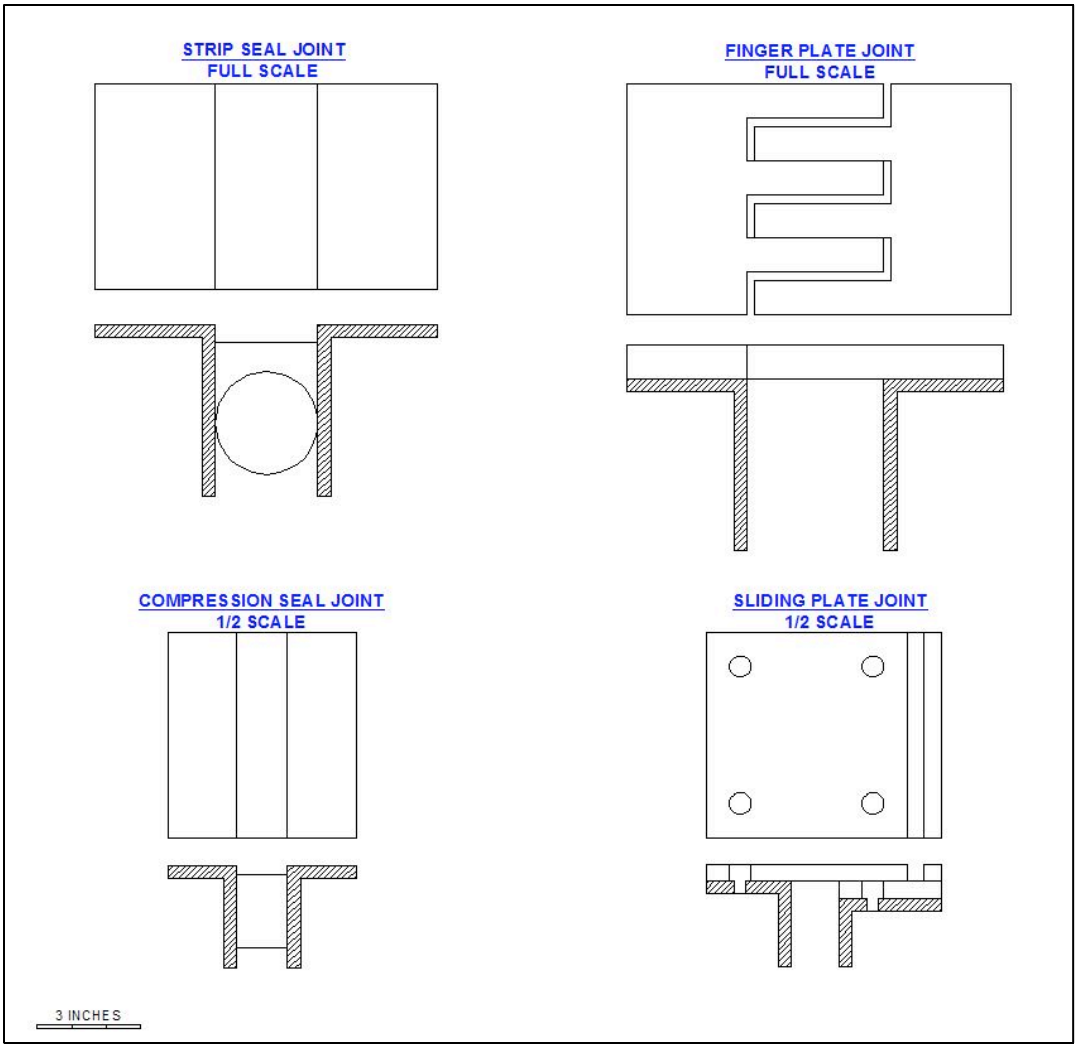


Figure 7: Initial Joint Design

Although the four joint types remained the same, the initial joint designs (Figure 7) needed to be modified further to fit the sizing criteria of the Instron model 8803 Dynamic tester as well as be constructible with the provided machinery before testing. Modifications were made with the comments of the project advisor, lab manager and lab machinist. Main modifications included replacing the sliding plate joint bolts with welds and welding additional thinner grips to the sides of the sliding plate and finger plate joints. By welding thinner grips, the joint would be able to properly fit into the Instron testing machine. The as-built joint designs with descriptions are shown in Figure 8.

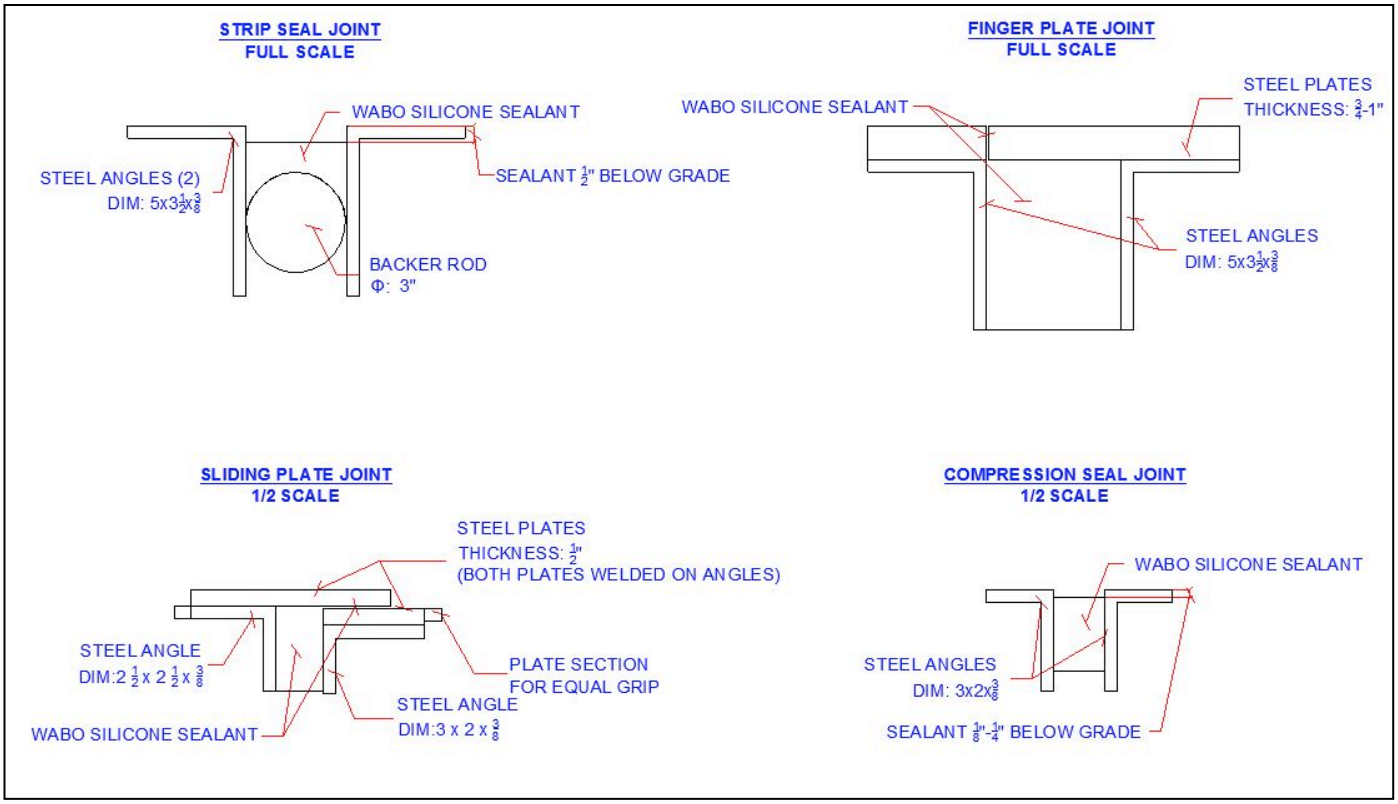


Figure 8: As-built Joint Design

3.3.2 Material Procurement

The material procurement began once the initial joint designs were established. Obtained materials included steel angles, plates and bars as well as silicone sealant and a pneumatic caulking gun for application of the sealant. A complete material takeoff list with expenses is shown in Table 5.

Table 5: Material Takeoff

Material		Amount Used	Cost
Steel Angles	3x2x(3/8)	10FT Length	$965 for Steel (Outsourced Machining $487)
	2(1/2)x2(1/2)x(3/8)	3FT Length	
	3(1/2)x5x(3/8)	10FT Length	
Steel Plate	½”	2.22 SF	
	1”	2.58 SF	
Steel Backer Rod		3FT Length	
Pneumatic Caulking Gun		1 (rented)	Free
WABO Silicone Sealant		0.55CF (11Sample Containers)	$450
Total Expense = $1,902 (not including labor cost from lab technicians)			

The original sealant selected in the project proposal was the DOW Corning 888 Silicone Sealant. After consulting the supplier, it was discovered that this type of sealant is specified for concrete structures and may have difficulty adhering to steel. Therefore, the final sealant choice was Wabo Silicone Seal Provided by the Watson Bowman Acme (WBA) Corporation. The Wabo sealant was selected because of its adherence to steel and recommended use for sealing horizontal bridge joints in the Northeast (*WBA Corp,* 2007). The sealant is a 1:1 mixture that was purchased in 50.75 oz dual cartridge kits. The purchased kits require a pneumatic caulking gun to apply the sealant; therefore, a pressurized gun was rented from WBA Corporation throughout the laboratory process (*WBA Corp*, 2007). More background on the WABO silicone sealant can be found in Section 2.3 and detailed specifications are in Appendix B.

A variety of steel materials were ordered for the four joint types. The angles, backer rod, and steel plates were ordered simultaneously from Peterson Steel in Worcester, Massachusetts.

3.3.3 Expansion Joint Construction

The expansion joint construction was a trial and error process which led to an established procedure. The construction began as materials were received and designs were modified in order to create a testable specimen. The first joint to be completed was the most complicated joint to construct—the sliding plate joint. This joint was the most difficult to construct because it required the most steel pieces to weld together of any of the other three joints. Because the sliding plate joint took longer to machine and construct, the first joint was put aside, and the complete set of compression and strip seal joints were next constructed. The fingerplate joint required an exterior machinist and were constructed and tested last.

All joint construction followed the listed procedure:

1. Machine steel materials to fit joint specifications (Figure 8)
2. Grinding machine used to rough steel areas where sealant would be placed
3. Joint constructed in appropriate mold for containing sealant
4. Steel surfaces wiped with alcohol to ensure clean interface with sealant
5. Sealant injected into joint
6. Curing period prior to mold removal
7. Mold removed
8. Joint tested in Instron model 8803 Dynamic Tester

The steel was mostly machined in-house—in the WPI Civil Engineering machine shop; however, the finger joint plates were brought to Hydro Cutter (North Oxford, MA) to be shaped by a precision water jet. The surfaces where the sealant was going to be placed needed to be grinded for better adhesion and rubbed with alcohol to provide a clean surface before the sealant was placed. An appropriate mold was created for each joint and is shown with brief descriptions in Figure 9.

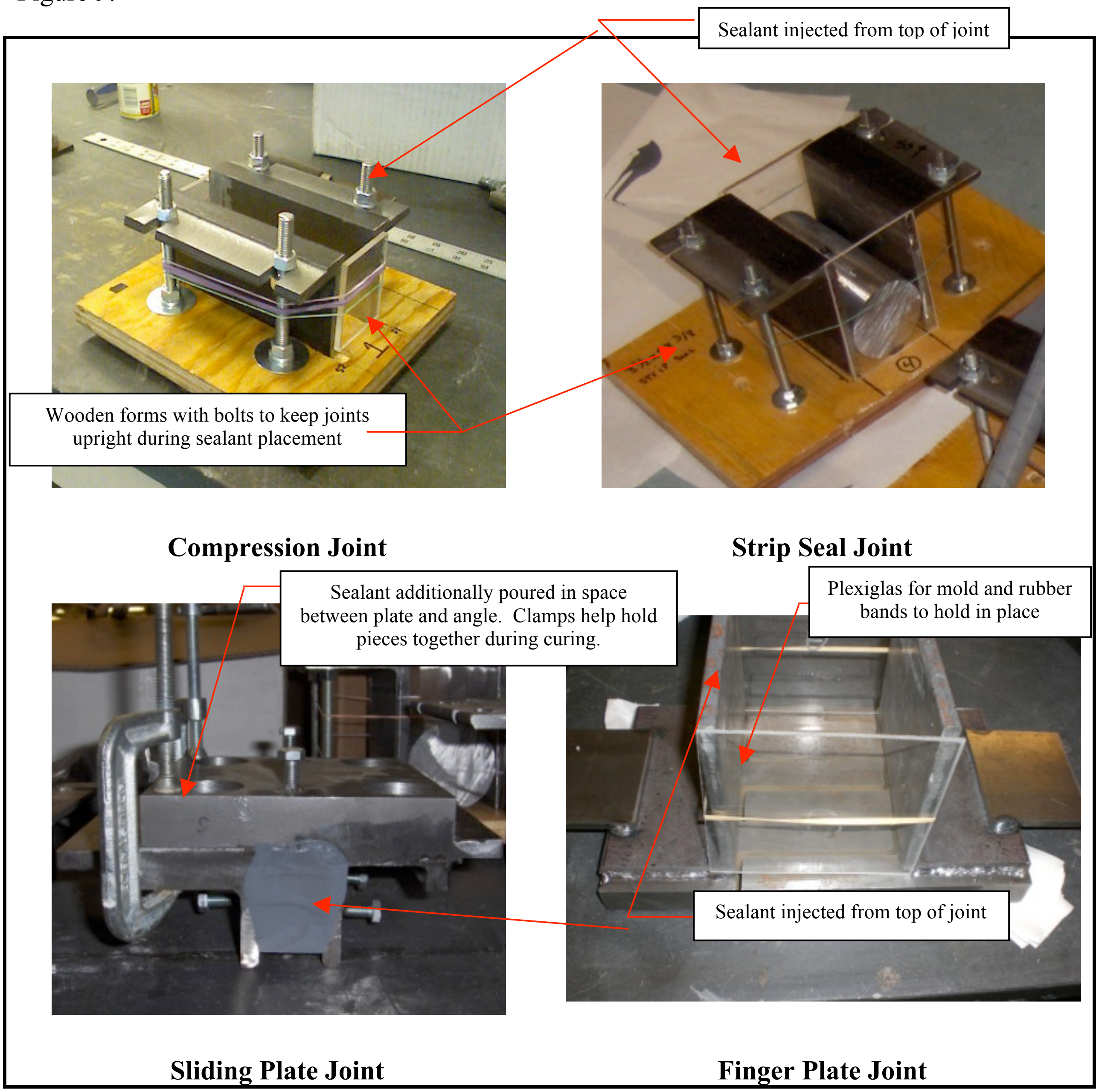


Figure 9: Joint Construction Molds

Once the molds were created and the surfaces cleaned, the sealant was injected into the joint model. Figure 10 shows Alyson using the pneumatic caulking gun to place sealant in a compression joint. The trial and error process included determining the appropriate time period in which the sealant cured before the mold was removed as well as how long the sealant should cure between the removal of the mold and testing. The summary of all constructed joints and curing times are shown in Table 6. All four joints removed from the mold are shown in Figure 11.

Figure 10: Sealant Pouring Method

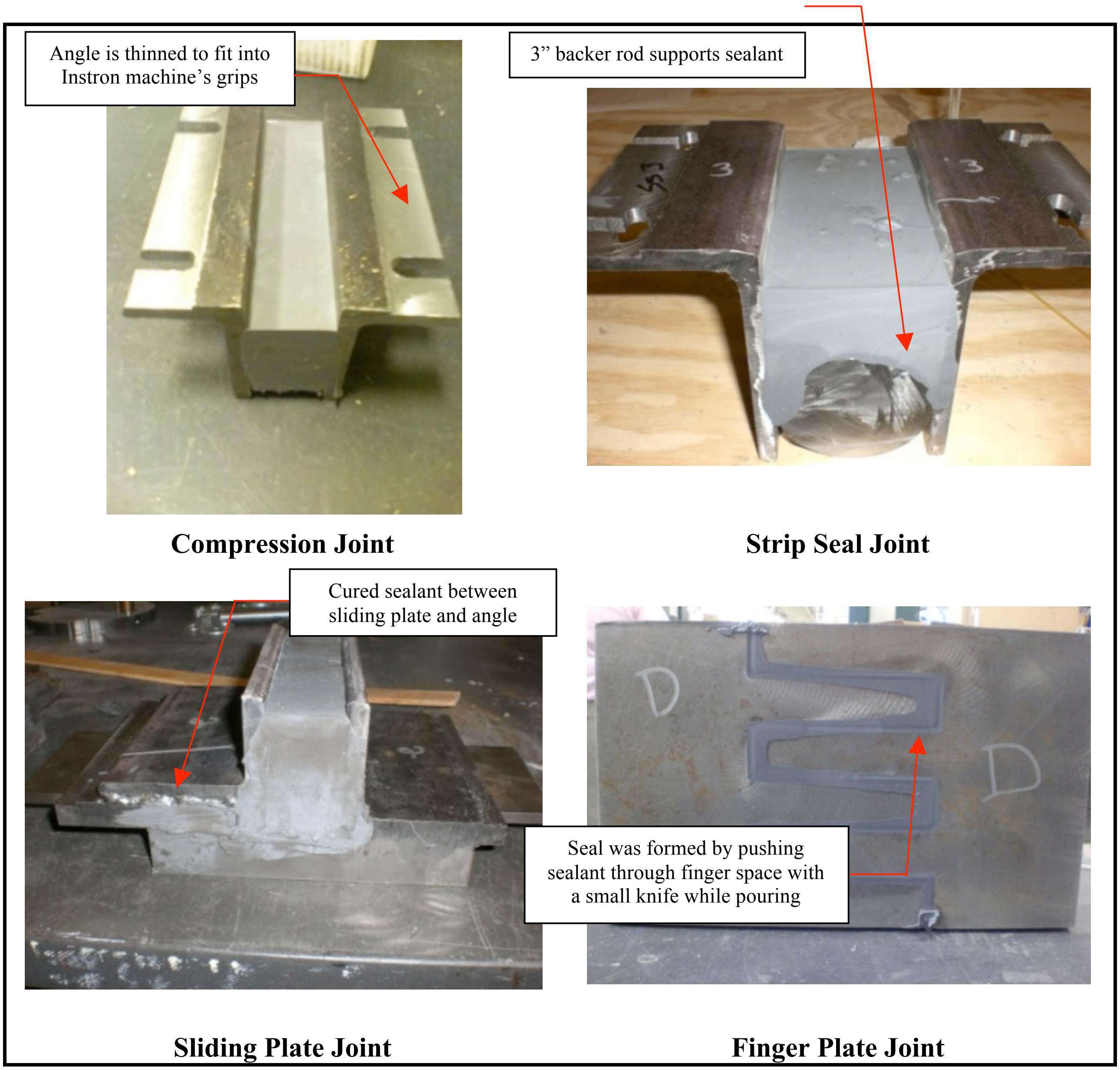


Figure 11: Joint Construction outside Mold

Table 6: Joint Construction Summary

Joint ID	Time in Mold(days)	Time out of Mold(days)	Mold Components
C1	1	7	Glass sides, wood bottom
C2	7	None	
C3	6	8	Glass on all 3 sides, sealant poured from top of joint
C4	6	9	
C5	6	9	
C6	5	7	
SS1	6	8	Glass on 2 sides, sealant poured from top of joint
SS2	7	15	
SS3	7	8	
SS4	7	8	
SS5	7	8	
SS6	5	7	
F1	6	4	Glass on 3 sides, sealant poured into bottom of joint
F2	6	4	
F3	6	4	
F4	6	4	
F5	6	4	
S1	6	8	Glass on 2 sides, sealant poured into bottom of joint
S2	5	14	Glass on 2 sides, joint lifted for sealant to form between sliding plate and angle. Poured from bottom of joint
S3	5	14	
S4	5	14	
S5	5	10	
S6	5	14	

3.3.4 Construction Considerations

Throughout construction, several complications were noted and fixed throughout the entire joint construction proves. The first construction dilemma was creating molds to hold in the sealant while it cured. The first two compression joints constructed had a problem involving the sealant bonding to the bottom of the wood mold. This adhesion created an uneven sealant surface on the bottom of the joint which caused instant deformations when tested in tension. Within the same mold, the plexiglass was found to help form the sealant but not bond to it. Therefore, a piece of plexiglass was placed on the bottom of the mold between the wood and the sealant for the next four compression joints created. The comparison in the surface texture is shown in Figure 12 with the left photograph being peeled from a wood bottom and the right photograph being peeled from plexiglass.

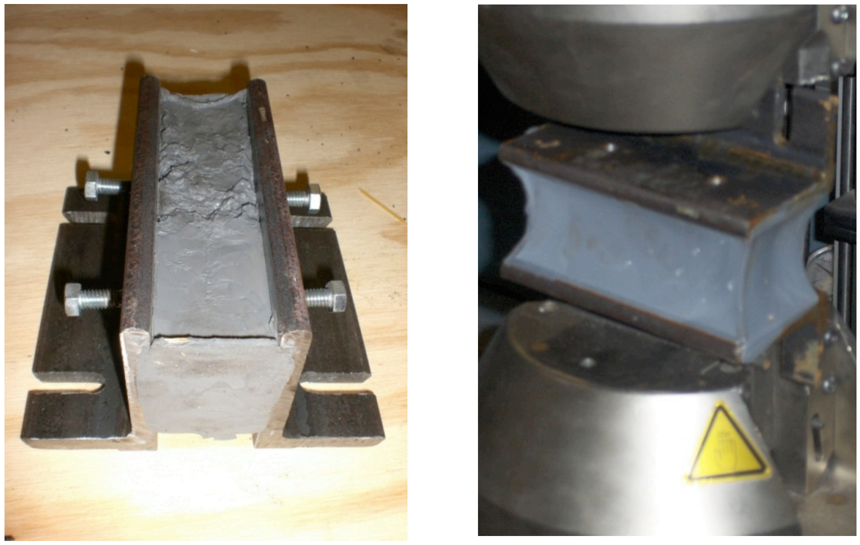


Figure 12: Wood vs Plexiglass texture comparison

3.3.5 Test Configurations

Each joint was tested in the Instron model 8803 Dynamic tester after construction. Concepts for the original tensile testing configurations are shown in Figure 13. The loading schemes were intended to mimic tensile testing along the roadway surface. These initial test configurations evolved as the abilities of the laboratory equipment were understood. Dynamic loading was originally included; however, due to the number of joints that were constructed, all efforts were focused on a control of tensile testing. Upon completion of all tensile tests, the steel was recycled by Dennis Gentile and thena computer analysis model was created to simulate the testing configuration and the tensile test itself.

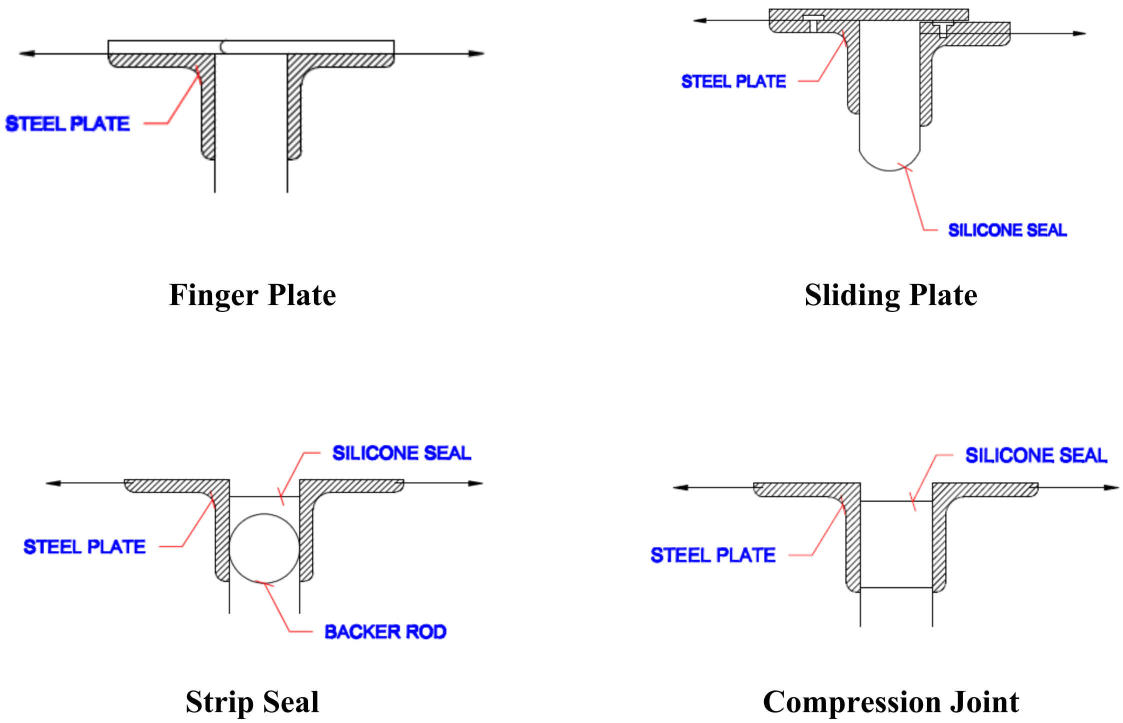


Figure 13: Tensile Test Configuration

3.4 Computer Testing and Analysis

The computer analysis was completed in order to simulate the compression joint sealant in testing. This analysis was performed in ANSYS 11.0 for academic use. Most of the components of ANSYS were self-taught; however, Adriana Hera (WPI Software Instructor) provided a tutorial which helped the beginning processes of the analysis. The ANSYS analyses required three major steps—create the shell and mesh to represent the sealant, apply the correct loading, and procure the appropriate results.

3.4.1 Element Creation

Other than creating a title and saving the project, the first step in ANSYS was to create the model. ANSYS assumes metric units for its numerical input; therefore, all dimensions needed to be converted into metric units. The dimension of the compression silicone sealant from the side perpendicular to the angles is 38.1 mm width and 69.9 mm height, forming a rectangular shape. This rectangle was created as an area in the modeling folder as shown in a screen shot in Figure 14.

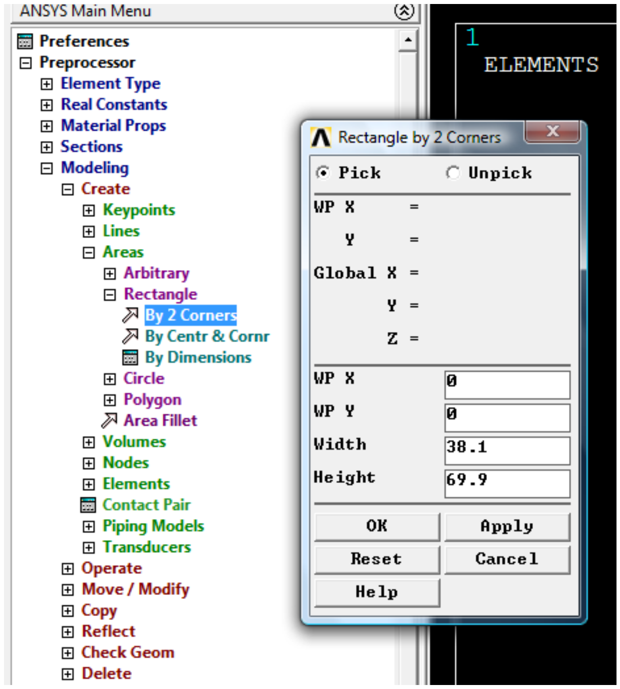


Figure 14: Model Creation

After the shape was formed, the properties were defined. In this project, the sealant was defined as a linear isotropic material. The modulus of elasticity (0.3447 MPa or 50 psi) and poisson's ratio (0.4) were determined from the sealant property information given by the WBA Corporation (*WBA Corp*, 2007). Once the properties were inputted, the element type was chosen—an elastic 8 node shell, coded "Shell 281". The shell thickness was then determined in the Real Constants folder. The thickness in the z-direction was not an important factor in the ANSYS simulation because the goal was to observe a cross-section of the sealant as a plane stress model; because the model situation required a thickness, it was set to a small, 1 mm dimension. The ANSYS screen shots of these three processes are shown in Figures 15, 16 and 17.

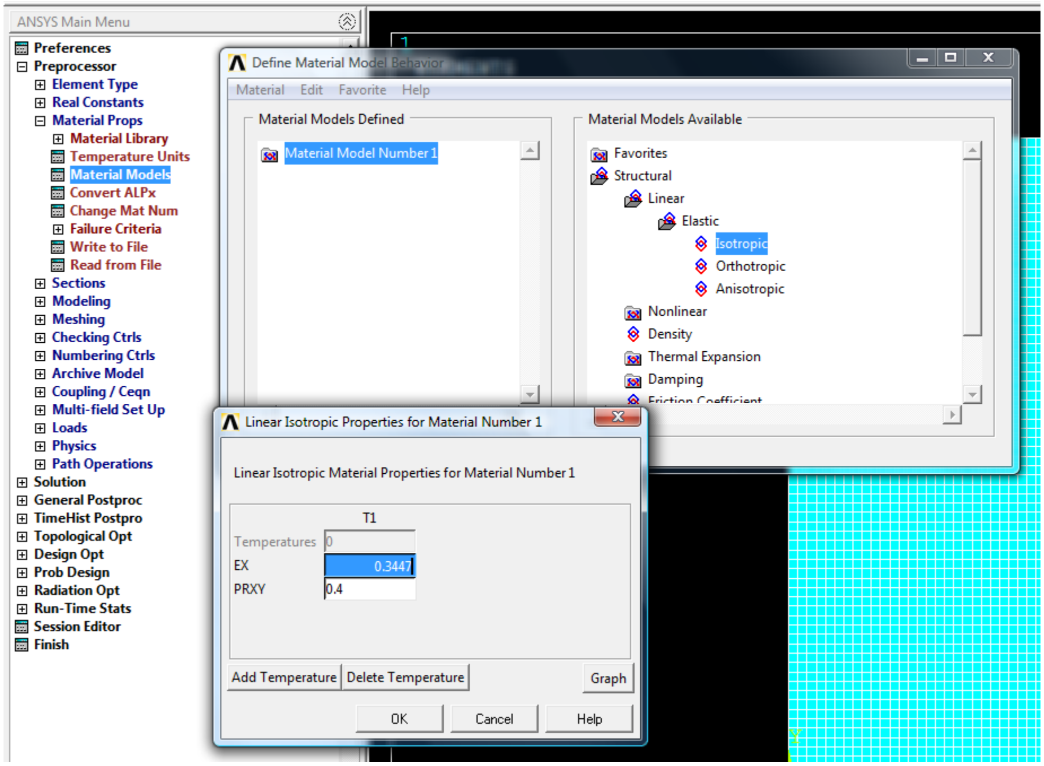


Figure 15: ANSYS Element Properties

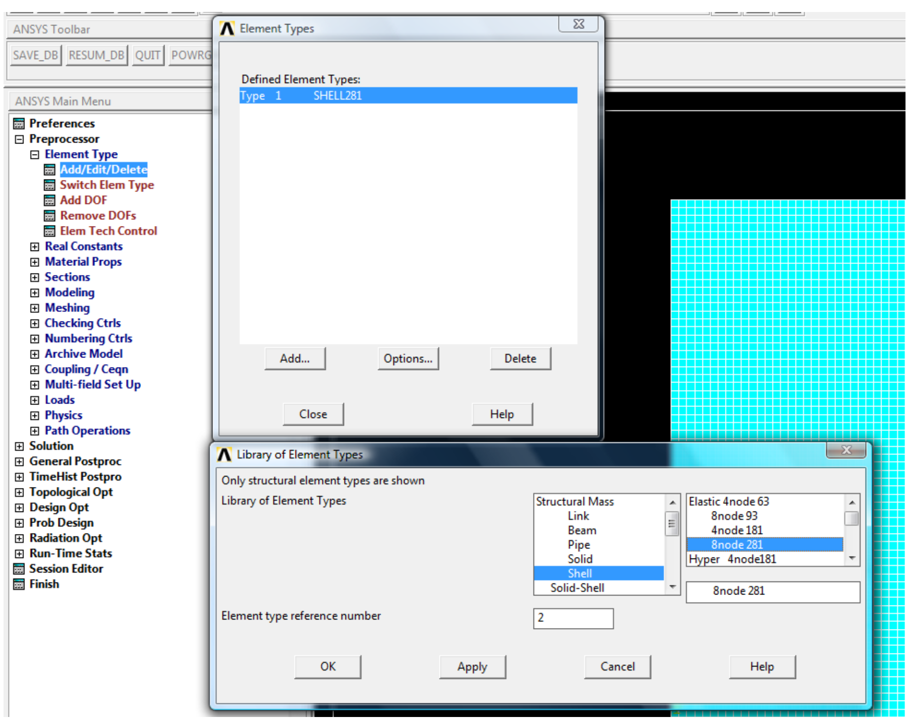


Figure 16: ANSYS Element Type

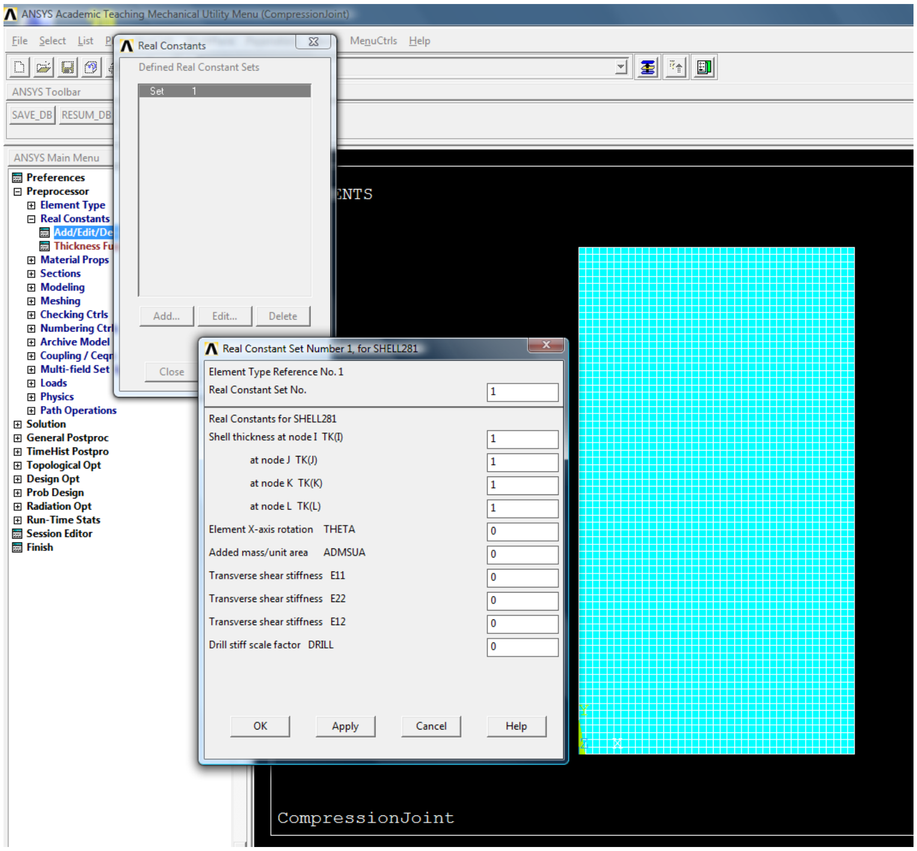


Figure 17: ANSYS Determined Thickness

When the sealant model was created and the properties defined, the mesh was then formed. Meshing separates the sealant volume into elements that simulate material behavior throughout the material model. The smaller the mesh is, the more comprehensive the ANSYS output will be. When the model is solved, ANSYS evaluates each node and how it reacts to the loading and constraints. Individual nodal displacements are used to determine the internal forces and stresses within the element. Therefore, a finer mesh may take longer to evaluate than a course mesh. For this project's model, the mesh thickness was set at 1 mm. A small, 1 mm size mesh will produce an adequate and accurate representation for this project's model. This mesh size produced around 5,538 nodes in a model with an area of 4.13 square-inches. The process for determining mesh size in ANSYS is shown on the screen shots in Figure 18. Once the mesh was created, the model was complete and the loading was then established.

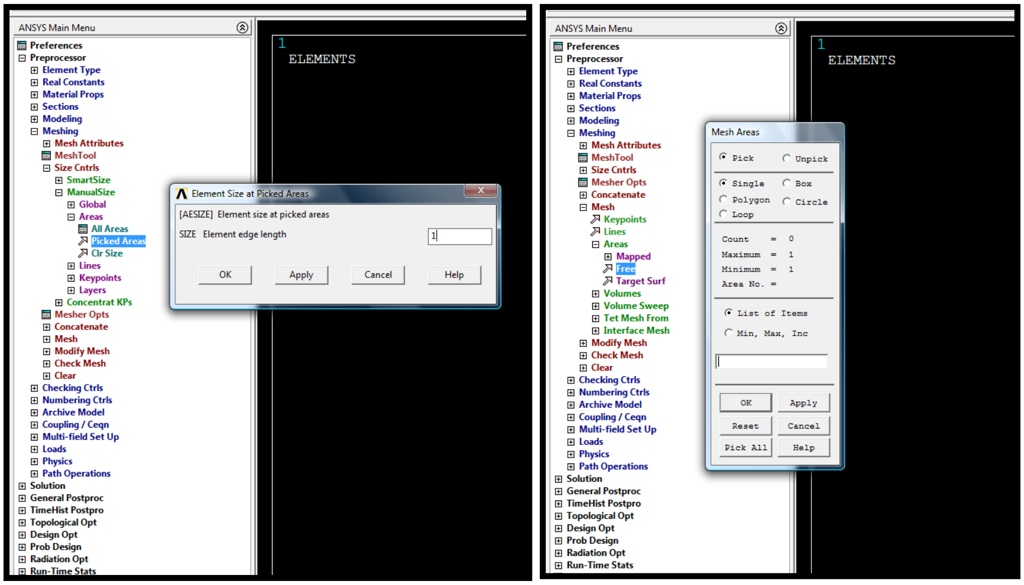


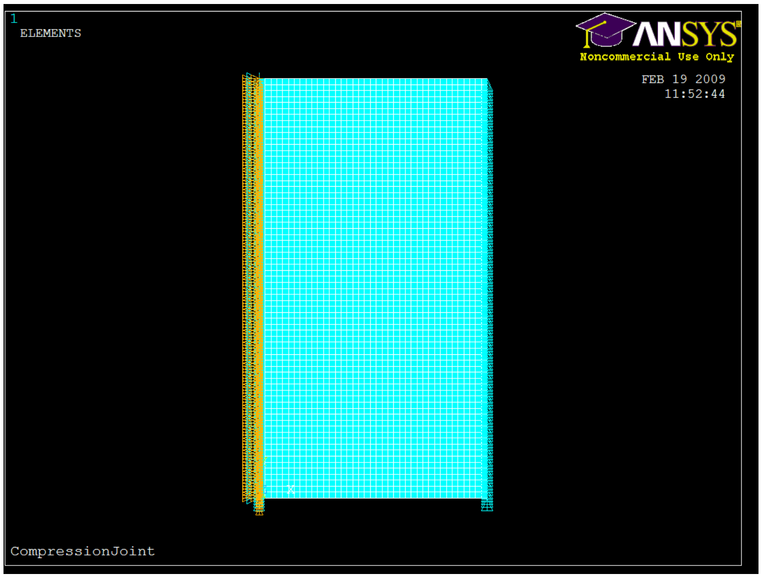
Figure 18: Forming the Element Mesh in ANSYS

3.4.2 Defining Loads and Displacement

The loading process with this model took four steps:

1. Apply displacement loadings or constraints as boundary conditions
2. Apply pressure loading as additional boundary conditions
3. Solve model
4. Continue to increase pressure loading and solve till the maximum pressure is determined.

Constraints were placed on both right and left sides of the sealant model to represent the connection to the steel angles. However, the left side was constrained on all degrees of freedom, whereas the right side was constrained in only the vertical, y, direction. The constrained model is shown in Figure 19. The left side is representing the bottom half of the joint in testing, which is gripped to the Instron tester and remains unmoving. The right side of the sealant model represents the top half of the joint in testing that is pulled away from the bottom half; therefore, the nodes on the far right of the model were constrained in the vertical, y, direction representing the sealant's connection to the angle. The far right nodes were not constrained in the horizontal, x, direction because that is the direction in which the pressure was applied.



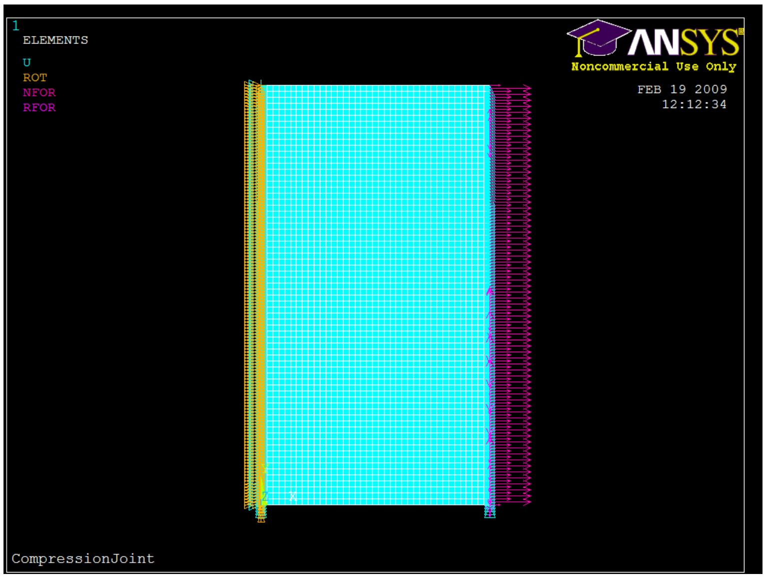


Figure 19: Sealant Constraints in ANSYS

The pressure loading was applied to the nodes on the far right of the model in the positive x-direction. The initial pressure was set at 100 Pascals (0.01psi) and increased until the deformation constraints (when $\Delta L = \sim 4mm$) did not allow the model to be solved. To appropriately compare the computer model to the laboratory model, it was necessary for the model to experience the maximum amount of load for the deformation constraints that were placed on it. Loadings less than the maximum could also produce the same strain in the sealant; therefore, the maximum allowable load was applied. The maximum pressure loading used in this project was 9,500 Pa (1.4 psi). Further results are shown in Chapter 5 of this report.

3.5 Cost, Funding & Maintenance Process

The cost, funding, and maintenance considerations were a special case where data was gathered instead of an overall process followed. Initially the costs for the construction of the bridge were determined using the *RS Means Heavy Construction Cost Data* from 2005 to establish a base cost (Reed Construction Data, 2005). Then the maintenance required was examined to determine what would be needed after construction. This included the overall picture of the life-cycle of a bridge, bridge inspection procedures, and Bridge Management Systems where all of the information is gathered and analyzed. The focus of the investigations into maintenance was not on the individual tasks that would be completed but on how they could be organized to be cost effective using the various resources. From the discussion of cost effective design and maintenance came funding. Today in Massachusetts the funding is in deficit creating hardships when trying to receive funding for projects. Two options were considered looking at the MHD website, the Accelerated Bridge Program and the new economic stimulus bill (*MHD Information Page*, n.d.). Both of these take federal funding and allot it directly to bridges because of the aging network that has an increasing number of structural deficiencies each year. Each consideration examined in this study was part of an overall picture of the Massachusetts Highway Department's procedure for bridge funding, design, construction, and maintenance.

3.6 Summary

This major qualifying projects process included three large tasks that applied to an overall theme of understanding bridge and connection design. The followings tasks of:

- Design of 2 girder bridges,
- Design, construction and testing of 23 bridge joints,
- Modeling a tensile test in a computer simulation program,

are described in the previous sections. The following sections will outline the results obtained from these tasks including their cost analysis, conclusions, and recommendations.

Chapter 4 – Bridge Design

All of the bridge design was completed using the standards researched during the preliminary stages of the project. These designs were established from various resources including the example designs published by the FHWA (*LRFD Design Examples*, 2006) and also those published in the *Bridge Engineering* textbook (Tonias, D., & Zhao, J., 2006). Design calculations were initially completed by hand then adapted in Excel spreadsheets for ease of use and to capture various possibilities and changes. The scope of this project included design of both a steel and a prestressed concrete girder bridge, and these encompassed the girders, deck, and bearings.

The prestressed concrete and steel girder designs were selected due to their use in the region as well as the information available of the MHD website on the design of the proposed bridge. This included a steel girder design, and it seemed to fit that the counterpart would be a concrete girder design. After much trial and error it was determined that a conventionally reinforced concrete girder would not meet the necessary capacity, and therefore the prestressed concrete girder was identified as a candidate. As the design progresses it can be easily noted that the concrete did have some tension capacity problems but bonding the reinforcement could solve them. This bonding is typical for portions of reinforcement and in this case is used for all reinforcement.

In addition to the structural design elements, each design needed to take into account non-structural components including the sidewalk, parapet, and one and a half inch future wearing surface. The basic geometry of the bridge as determined by the Massachusetts Highway Department designates the overall form of the bridge and components. The overall design needed to be a bridge with a width of approximately 59 feet and a span of nearly 92 feet. This was to include two travel lanes with almost equivalent length shoulders and two sidewalks to be 3 feet wide each. (Project, n.d.) In this general description it was taken that for the edge regions the parapet would sit on the sidewalk, and the sidewalk would extend 3 feet from the base of the parapet shown in Figure 20.

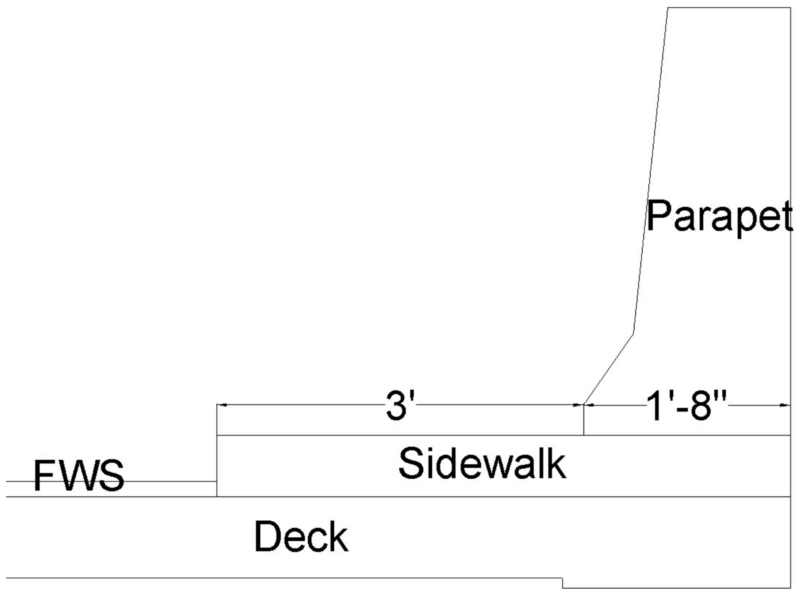


Figure 20: Non-Structural Component Configuration

Figure 21 is a representation of the parapet used in the design. The geometry and construction of the parapet and other structural elements had to be examined before the design was initialized because of the important role they play in the loading of the bridge, especially in the deck overhang.

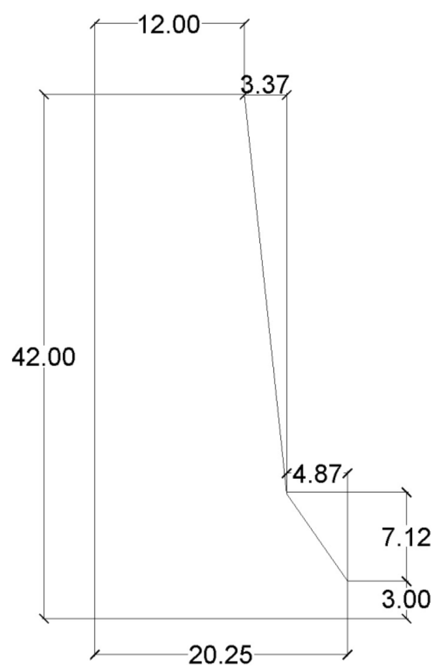


Figure 21: Parapet Dimensions

In the design it was assumed that this parapet is positioned on the edge of the deck due to the desire for extra space for the travel lanes and shoulders. By putting the parapet on the edge the sidewalk then extended 3 feet out where it met the future-wearing surface. This future wearing surface extends for a shoulder, two travel lanes, and a second shoulder. Upon further investigation of the parapet design and attachment to the deck it was found that the parapet is reinforced with many bars including ones that go directly into the slab. Below in Figure 22 is a depiction of a typical attachment of the parapet to the deck to ensure full crash rating.

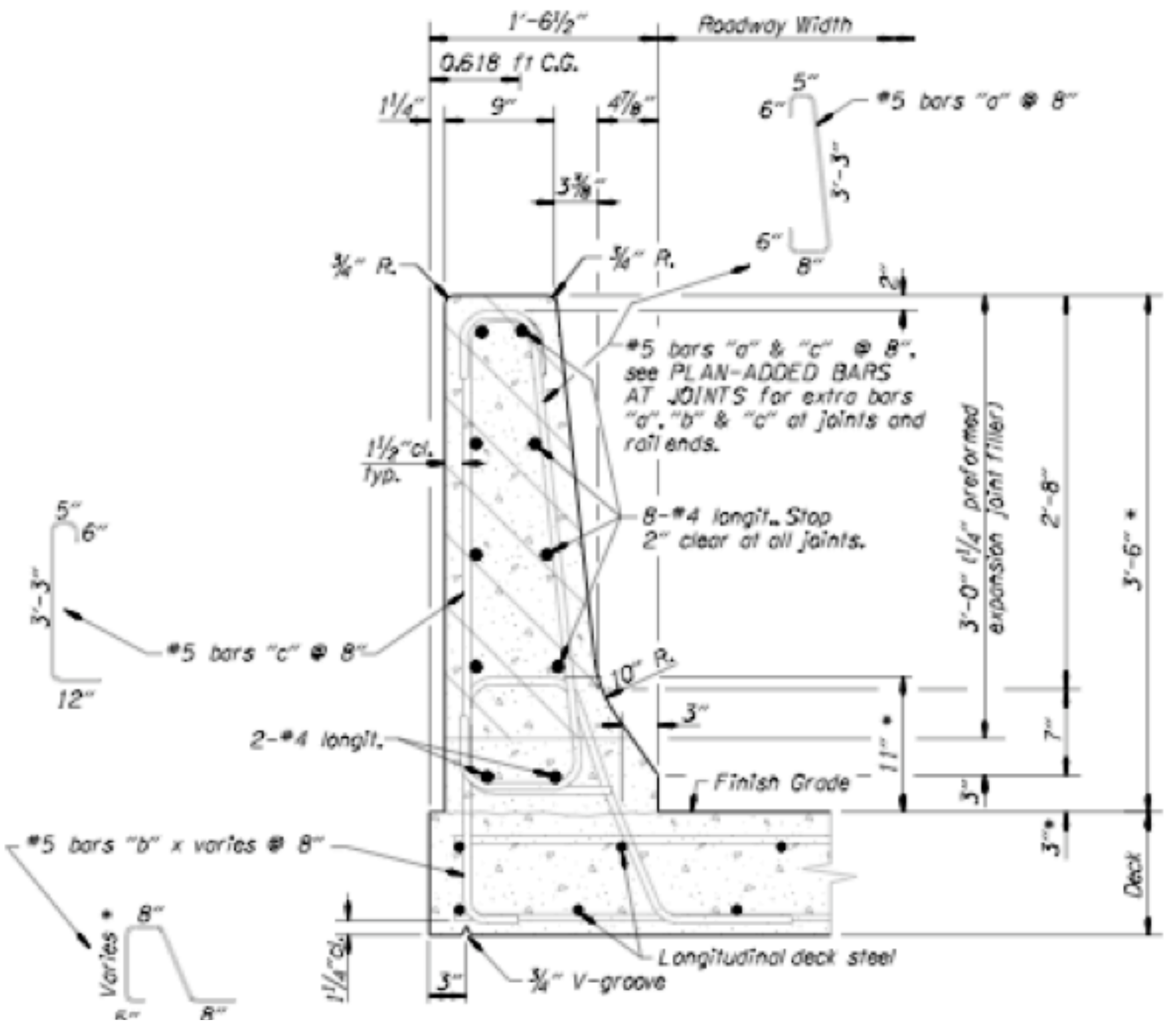


Figure 22: Typical Parapet Detail (3'-6" Type "F" Rail, 2009)

4.1 Loading Cases

When designing bridges various loading conditions must be considered. In the most basic terms dead loads and live loads must be established and then analyzed in terms of internal forces and moments to determine the member sections in accordance with the *Massachusetts*

Highway Department Bridge Manual (Mass Highway, 2005) and the *AASHTO LRFD Bridge Design Guidelines* (AASHTO, 2005). In order to calculate dead and live load effects many loading assumptions were made. Table 7 shows the cases that were considered to establish envelope values for internal moments and shear forces. All loadings were taken from Section 3 of the AASHTO LRFD Bridge Design Guidelines.

Table 7: Design Cases (AASHTO, 2005)

Scenario	Loading Model	Force Effect Under Study
Fully Loaded with Trucks	3 truck fronts and 2.5 trailers fit on length bridge (two lanes)	Max Moments and Shear
Two trucks bumper to bumper over pier (both lanes)	2 feet between trucks at center (4 trucks- one in each lane of each span)	Maximum Shear
One truck and one car on one span (both lanes) since two trucks do not fit	10 feet between truck and car	Maximum Positive Moment
One truck at each end of bridge (both lanes)	Trucks 1 foot from edge of bridge	Maximum Negative Moment
Fully Loaded with Cars	4 foot spacing between cars (6 car lengths on first span and 5.5 car lengths on other span)	Max Moments and Shear
Two cars bumper to bumper over pier (both lanes)	2 feet between cars at center (4 cars- one in each lane of each span)	Maximum Shear
Cars on one span (both lanes)	6 car lengths on one span with 4 foot spacing between each car	Maximum Positive Moment
One car at each end of bridge (both lanes)	Cars 1 foot from edge of bridge	Maximum Negative Moment

With these loading assumptions made, drawings were made in AutoCAD to show clearly where specific loads are being applied and the magnitude of the loads. From the drawings the moment distribution method was used to calculate moments to then begin design of the girders. These moments were used where the AASHTO design table loadings were not applicable in the design process. The AASHTO tables provided pre-factored values and were used as values when designs such as the deck followed the FHWA examples (*LRFD Design Examples*, 2006). The moment distribution values were used in comparison to the values achieved using a simplified process using the girder design examples in the *Bridge Engineering* text (Tonias, D., & Zhao, J., 2006). It was known that in the comparison the loadings from the moment distribution were very conservative.

4.2 Concrete Girder Bridge Design

The design of the concrete girder bridge was of prestressed girders, reinforced concrete deck, and bearings. Figure 23 is the overall design solution cross section.

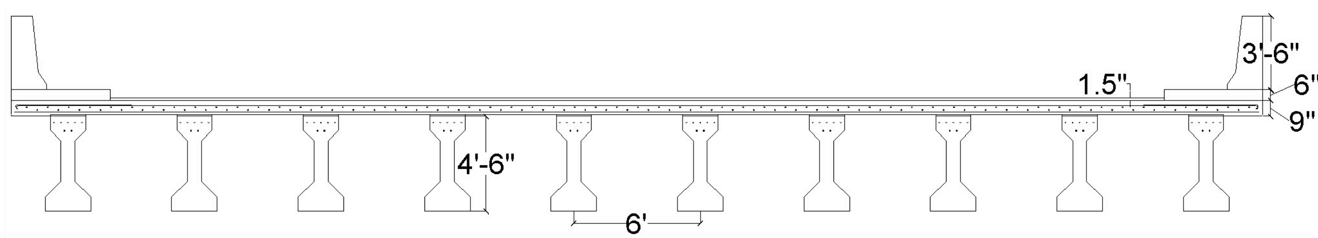


Figure 23: Prestressed Concrete Girder Cross-Section

The general bridge design steps are shown as a flow chart in Figure 24. These steps are the overall procedure to complete the portion of bridge design studied in this project when using prestressed concrete girders.

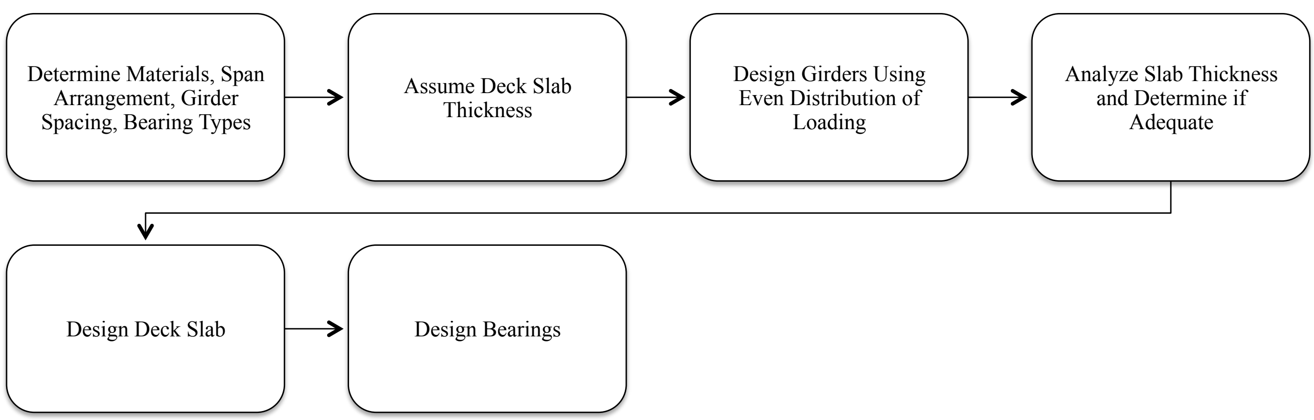


Figure 24: Overall Concrete Bridge Design

Each step used the resources and design examples as specified above. The specific loadings for each design were as follows. AASHTO tables were used for the deck reinforcement design and identified as "Design Deck Slab" in the figure. The "Design girders Using Even

Distribution of Loading" used the simplified method found in *Bridge Engineering* and was compared to the moment distribution loading (Tonias, D., & Zhao, J., 2006).

The key to note in any design process is the interlinking of designs and the iterative process. If the girders do not work given the deck thickness the deck design must be adjusted. In this specific case the deck is an added dead load with no effect on the strength of the girders. Since the deck is such a major component this dead load can affect the number of girders needed to uphold the loading, which in turn determines the spacing of the girders. If the deck design does not meet capacity because of the girder spacing or overhang, the girder design must then be adjusted. This flowchart represents an overall view, and these steps were performed many times before a final design was determined.

4.2.1 Girder

The girder design is the first major component being designed for the concrete bridge and consists of the girder design in flexure and shear. To begin the girder design for a prestressed member typical AASHTO Type Beams can be selected for various spans. Loading is then checked against the capacities of the selected beam. For the design a Type IV beam was selected because it had the span range of 85 to 120 feet and the span of the proposed bridge was approximately 90 feet. In the first attempt it did not meet the required temporary tensile strength for the prestressed member in the top or bottom fibers because of the significant moment and substantial prestressing force needed for the length of the span. Therefore the continuous girder system was changed into a non-continuous design with two simply supported spans having a bearing on the central pier. The original idea was to have a continuous spanning bridge but due to the span and width it was impossible to satisfy all criteria. Figure 25 shows the design process in a flow chart as an overview of the design steps.

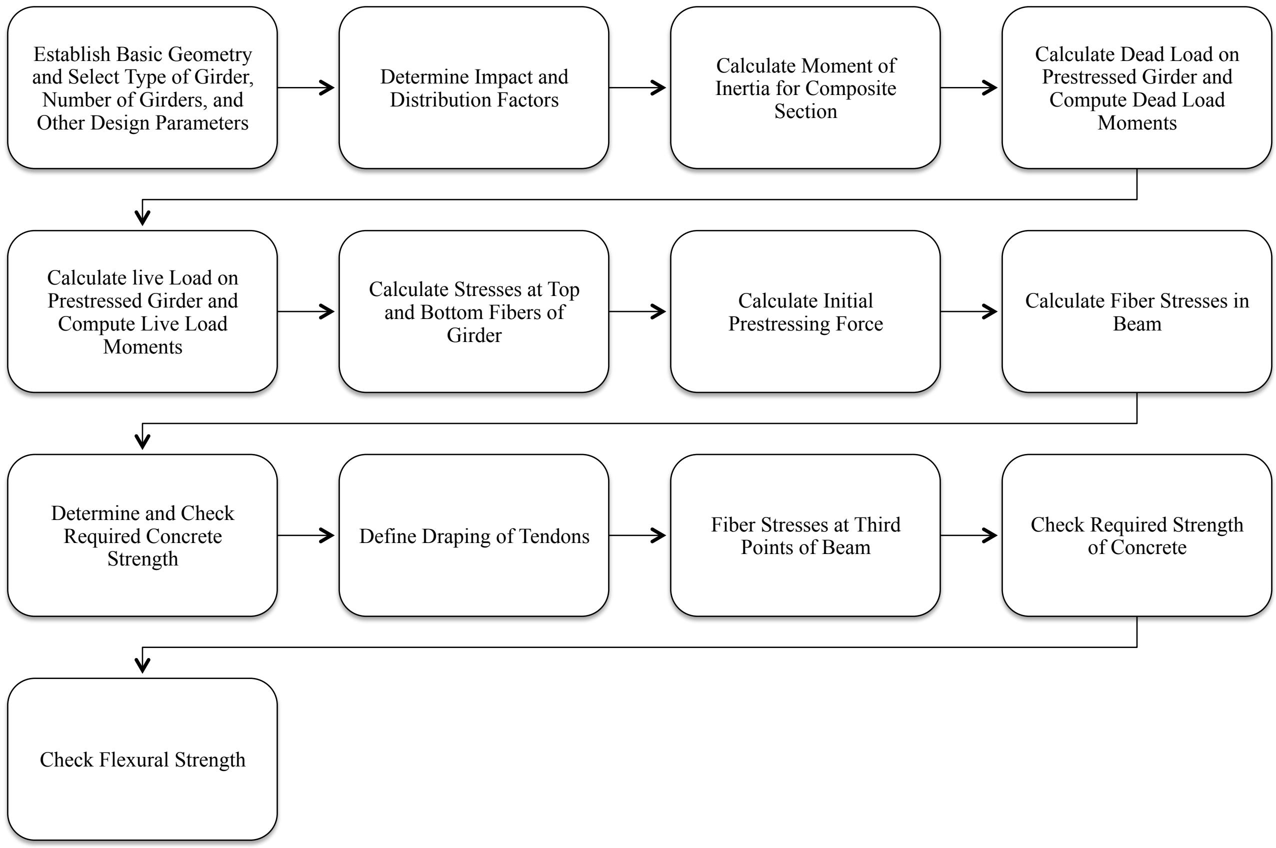


Figure 25: Concrete Girder Design - adapted from *Bridge Engineering* design steps (Tonias, D., & Zhao, J., 2006)

After initially completing all of these steps through hand calculations, an Excel spreadsheet was prepared to aid the design process. This spreadsheet used non-continuous design and included a comparison of two configurations. The two configurations both utilized Type IV girders: one had 12 girders, and the other had 10 girders. The two designs needed similar reinforcing with comparable capacities. Therefore the option with 10 girders was selected because of cost considerations of creating two additional beams. The geometry in Figure 26 is the basic geometry of an AASHTO Type IV girder. This girder is one of several basically defined shapes that are typically used in bridge design. Since these geometries are available as pre-sized typical girders, only the reinforcement must be detailed rather than defining overall geometry in addition to the reinforcement.

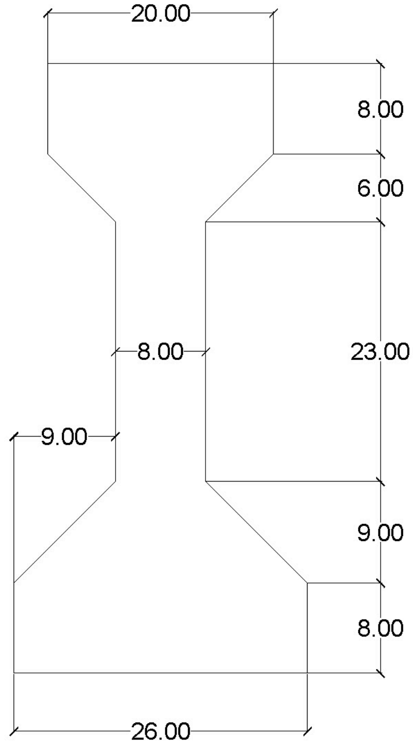


Figure 26: Concrete Girder Geometry

A depiction of the final girder design is shown in Figures 27, 28, and 29. Figure 27 is a side view along the span and the others are cross-sections in order to display the detail of reinforcement and dimensioning.

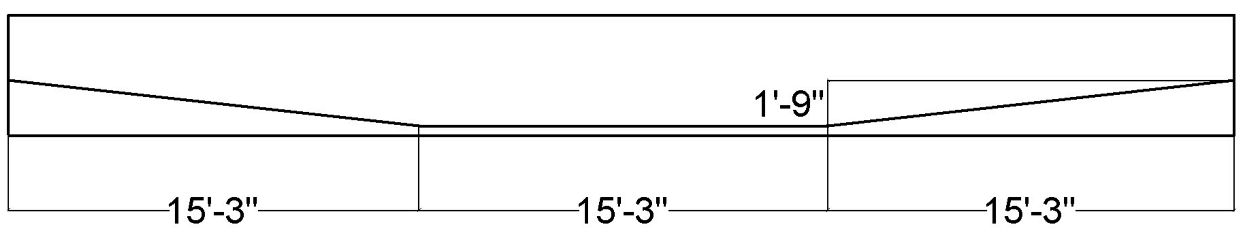


Figure 27: Draping of Prestressing Tendons

Figure 27 represents the draping of the prestressing tendons where the difference in starting and finishing height of the center of gravity of the strands is the eccentricity. The draping is similar to that found in the *Bridge Engineering* text (Tonias, D., & Zhao, J., 2006) and was assumed to be a general draping configuration. This configuration uses the eccentricity and girder bottom flange geometry to establish the draping heights. This difference in height occurs over 1/3 the span of the girder. The middle part of the span has the tendons in the center of the bottom flange of the girder. The configuration of the prestressing reinforcement is detailed below in Figures 28 and 29; Figure 28 refers to the ends of the girder and Figure 29 is a section at the midspan of the girder. The X represents the bonded prestressed strands versus the circular conventional reinforcement also shown. The prestressed strands are not always bonded in which case the symbols would be significantly more important. Due to the nature of the tension force in the designed member, bonded strands are required.

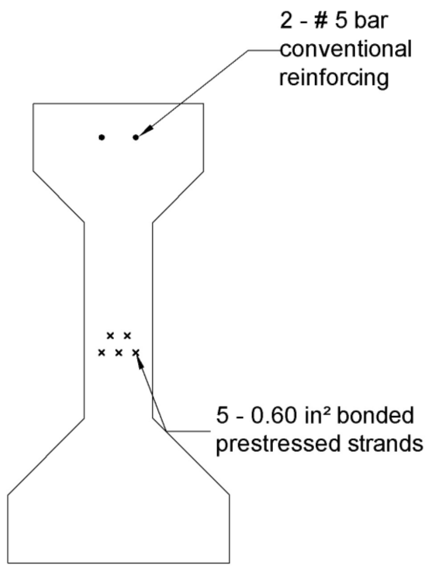


Figure 28: Prestressing Detail at Ends of Girder

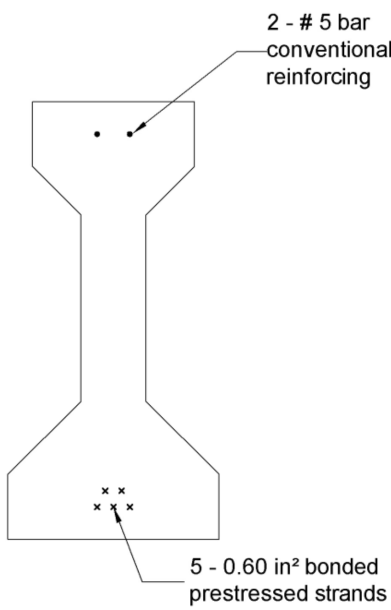


Figure 29: Prestressing Detail at Middle of Girder

4.2.2 Deck

The design of the deck takes into consideration many assumptions that lead to iterations of the design process until all assumptions are proven with the design checks. Shown in Figure 30 is a flowchart detailing the design process to develop a trial design, which is then verified in a systematic manner. Many of the steps become repetitive in the checks of various capacities, and therefore the spreadsheets were extremely useful for the iterations.

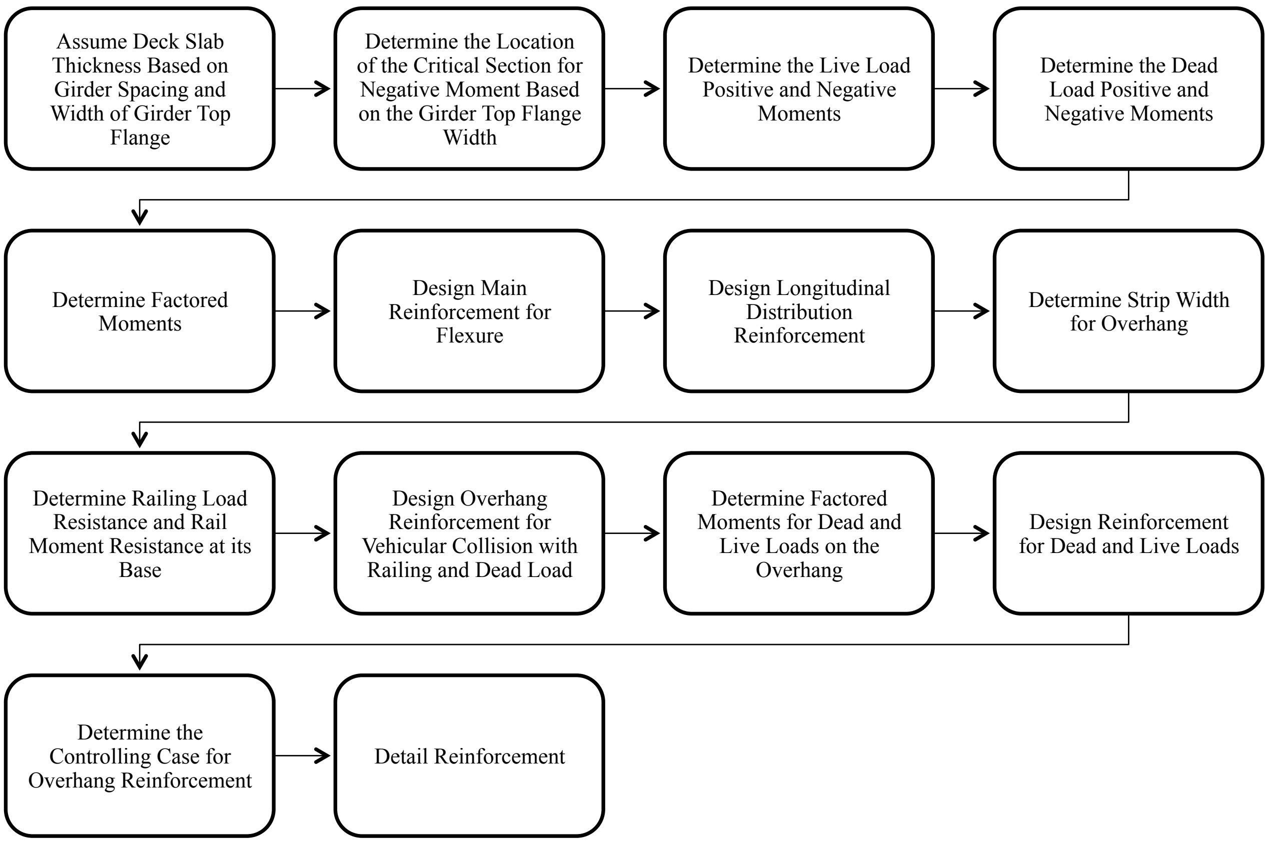


Figure 30: Concrete Deck Design (adapted from *LRFD Design Examples*, 2006)

From the girder selection a finalized deck design was determined. Below is the detail of this design, which has a variety of reinforcing. The transverse reinforcing which goes along the entire deck is #5 bars spaced at 12 inches in the top for negative moment reinforcement, and #5 bars at 4 inch spacing in the bottom for the positive reinforcement. These were located at the edge of the cover, which was 2.5 inches for the bottom and top. This cover was typical throughout references utilized to complete the design and allows for the concrete exposed to the temperature fluctuations, vehicle exhausts and other natural elements to protect the steel reinforcing. In addition to this typical transverse loading are supplementary bonded #5 bars in the overhang region. Each of the original reinforcing bars gained two extra # 5 bars, which were bundled to the overall transverse top reinforcing. The overhang design needed additional reinforcement because of its cantilever nature. This reinforcement needed to develop properly before reaching the overhang and therefore extends 36 inches past the centerline of the exterior girder. In addition to the transverse reinforcement there is longitudinal reinforcement, which spans in the same direction as the bridge. The bottom longitudinal reinforcement is calculated as a percentage of the transverse reinforcement. This design needed # 5 bars at 12 inch spacing for the bottom of the deck to suffice the requirements. The longitudinal reinforcement provided also helps with the temperature and shrinkage capacities of the concrete. The area of steel needed to meet the minimum requirements is eleven percent of the area of the section divided by the yield strength of the reinforcing bars. The top longitudinal reinforcement provides this minimum and is to be # 4 bars every 12 inches as is typical for many states. All of these details are illustrated in Figure 31 showing the deck detail from the overhang to the first interior girder.

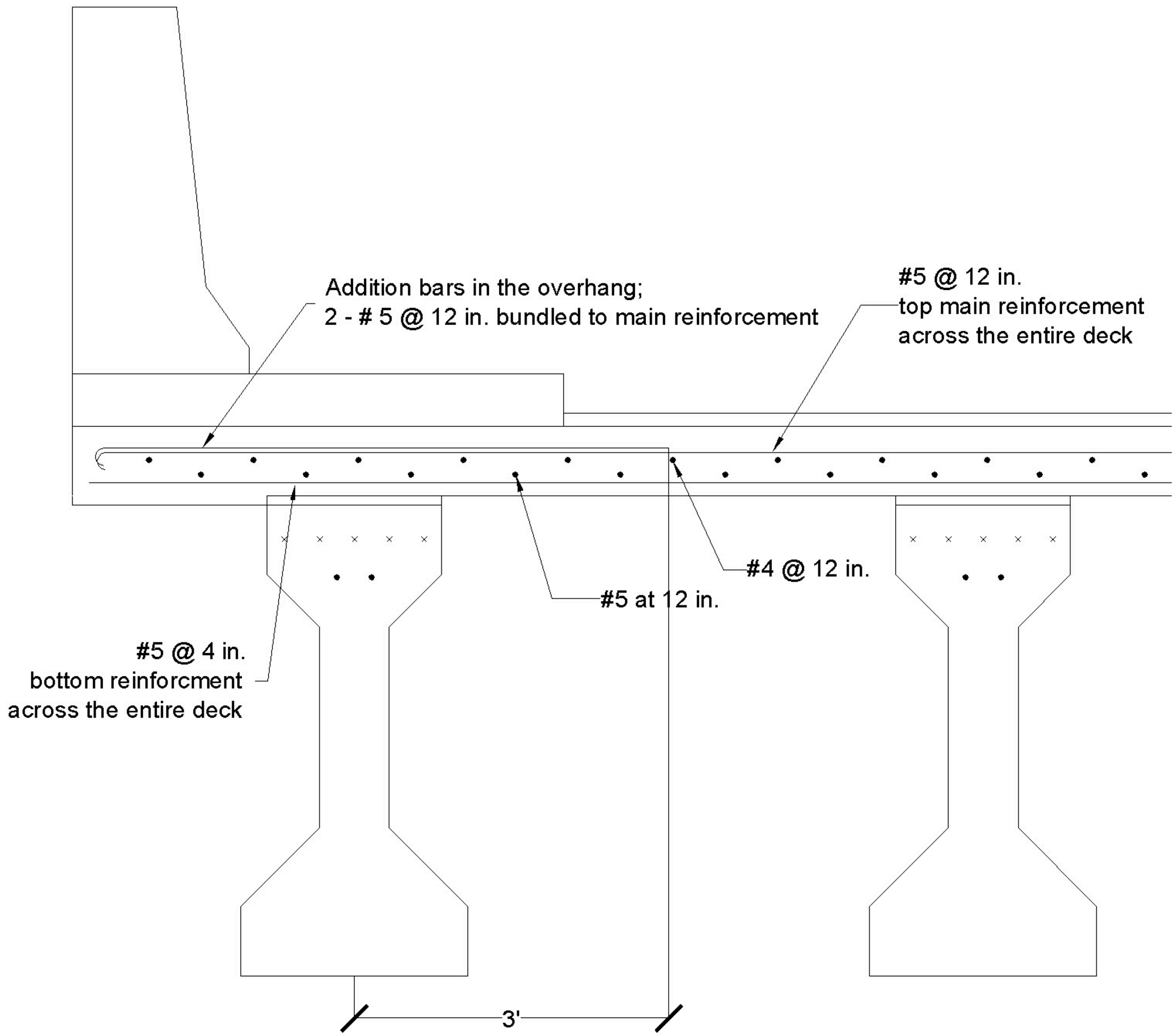


Figure 31: Overhang and Deck Detail

4.2.3 Bearings

Once designs for the deck and girders had been established the bearings were designed. Figure 32 is a design flowchart showing the overall steps that were established from the text *Design of Modern Concrete Bridges* (Heins, C., 1984).

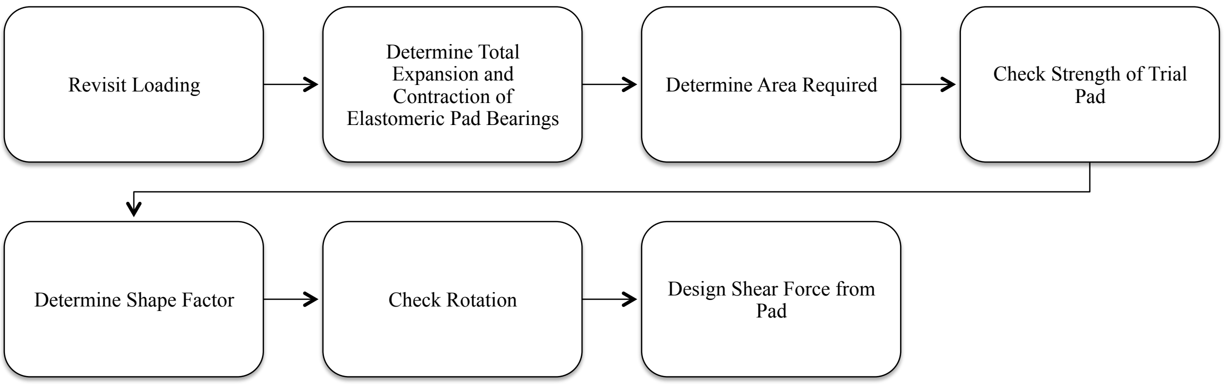


Figure 32: Bearing Design for Prestressed Concrete Girders

The design for the bearings included calculating the reactions that would occur at the pier and abutment. These reactions were the same and therefore the same bearing system can be used for all four of the locations. Each girder has a bearing pad which was designed for a temperature change of 120°F to account for the variety of weather in the region. The final design consisted of a 4" x 26" x 1" beveled sole plate with two laminations. This design took into account deformations, rotations, and other forces acting in the area of the bearing. Included in the design were rotations that the factory prestressing process would create along with the additional rotations from time due to the girder and the slab. These rotations were then used to find the offset, which determined the type of plate to be used. Following the selection of the type of plate, the shear force of the bearing was determined to finalize the dimensions.

The bearing design would be somewhat effected based on the pinned versus roller connections. The pinned locations would have bearings fixed against shear deformation, which would allow a greater compressive stress than that of the bearings subject to shear deformation. These calculations would affect the shape factor, which is determined by the geometric properties. A detailed further study could be performed to investigate the impacts of the various bearing configurations available to suit the differences between the pinned and roller connections (*AASHTO LRFD Bridge Design Specifications*, 1998).

4.3 Steel Girder Bridge Design

The design of the steel girder bridge involved a design of the deck, girders, and bearings. Figure 33 is a depiction of the finalized bridge design drawn in CAD.

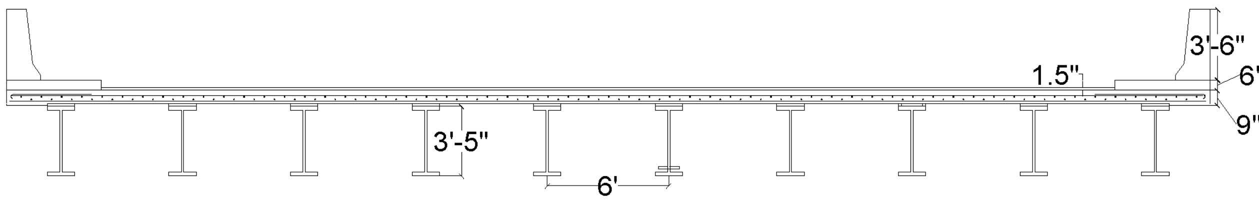


Figure 33: Steel Girder Cross-Section

An overall steel bridge design flowchart to show the process is in Figure 34. Similar to the concrete design it is iterative as a whole and takes much back and forth between the girder and deck designs.

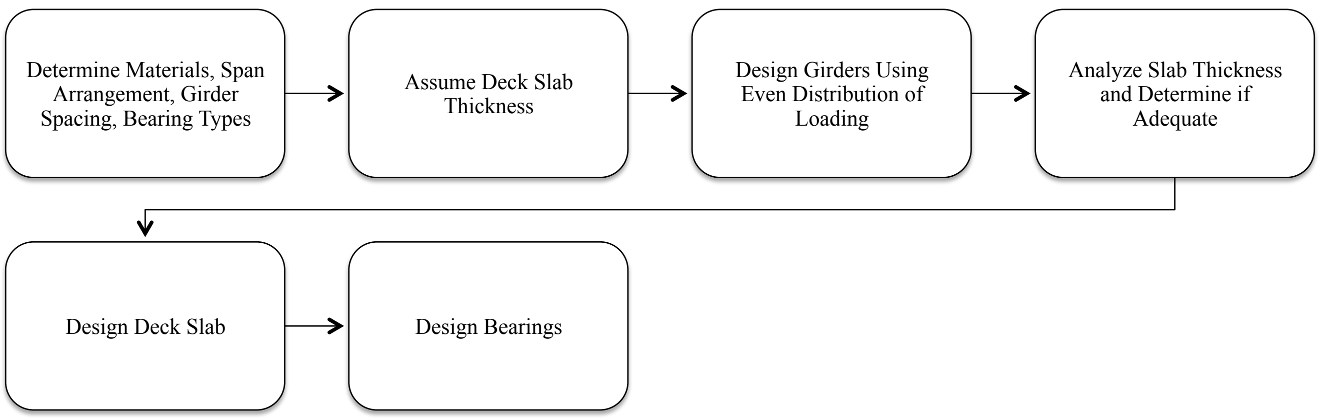


Figure 34: Overall Steel Bridge Design (adapted from *LRFD Examples*, 2006)

Each step used the various resources and design examples. The specific loadings for each design were as follows. AASHTO tables were utilized during the deck reinforcement design identified as "Design Deck Slab" in the figure. The "Design Girders Using Even Distribution of Loading" used the simplified method found in *Bridge Engineering* and was compared to the moment distribution loading (Tonias, D., & Zhao, J., 2006).

4.3.1 Girder

The girder design of the steel differed greatly from that of the concrete design. This design required more design effort directed at the shear forces compared to the overall moment design of the concrete girders. Balancing and iterations performed were fewer during this design, which eliminated some of the repetitive steps. The design was more of a trial and error process trying the varying W-Shapes to fulfill the capacity requirements. This was helped because the general capacities of W-Shapes are known and basic properties are continuous unlike the concrete girder where changes in the area of reinforcing steel can completely change the strength properties. When reinforcing was added to the concrete the tension and compression blocks needed to be examined with the boundary significantly moving depending on the location and amount of reinforcement. This did not occur with the steel because once a member was determined to be used the properties were easily found since non-composite design was completed. Note that this design process individually addressed the positive and negative flexural designs as shown in Figure 35.

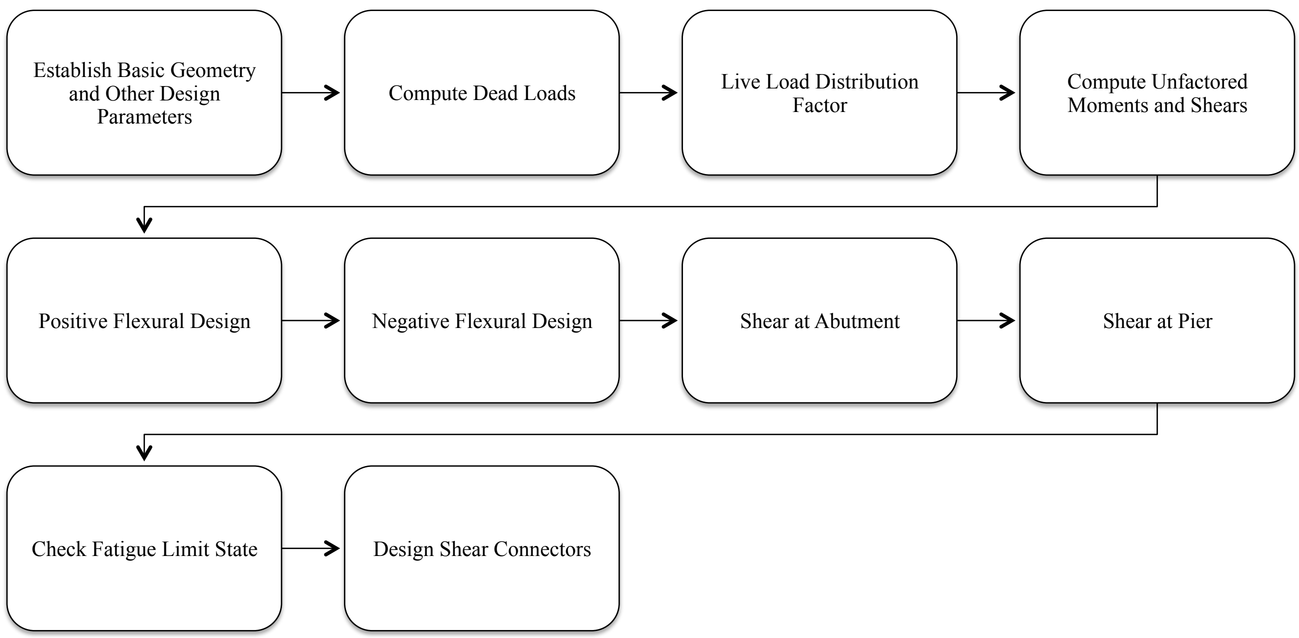


Figure 35: Steel Girder Design - adapted from *Bridge Engineering* text (Tonias, D., & Zhao, J., 2006)

The final completed design was a W 40 x 431 and is dimensioned in Figure 36. This design was non-composite and therefore did not require shear studs to connect to the deck. The possible use of shear stiffeners were also investigated, and it was determined that none were needed at the abutment.

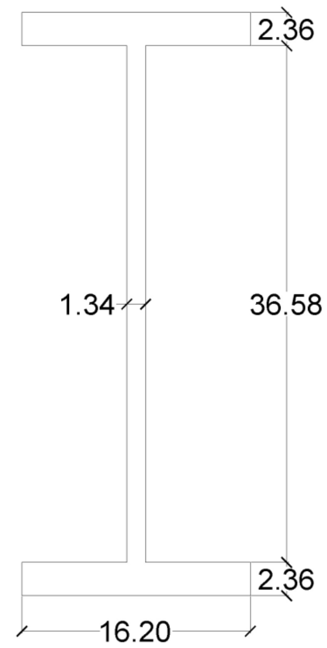


Figure 36: Steel Girder Geometry

4.3.2 Deck

The deck design for the steel girder bridge is the same as the concrete girder bridge but uses the superstructure specified by the steel girders. This design was performed in the Excel spreadsheets developed as part of the design of the concrete girder bridges. The resulting slab designs were similar with the same development lengths needed, and the steel system overall needed comparable reinforcement. The steel girder deck had the same reinforcing as when the design for the prestressed concrete girder everywhere except in the top transverse direction. Instead of requiring a # 5 bar every 12 inches the spacing decreased to 10 inches. This is in part due to the cover, which created different compression and tension zones. After trying various cover thicknesses, it was determined that thinner the cover was the less the reinforcement was needed for the steel girder design due to the location of the girders. Again this shows how much a design can be influenced by the simplest geometries such as cover. Beyond the reinforcement for the entire bridge, the overhang needed two additional # 5 bars, which are bundled to the other top reinforcement. At one point in the design a spacing of 22 inches was required but due to the maximum spacing limit of 12 inches that was utilized. The longitudinal reinforcement was then calculated the same as the first deck design. Because of the similarities of the transverse reinforcement design the longitudinal reinforcement is identical for both. Figure 37 shows the detailed section of the overhang and first span.

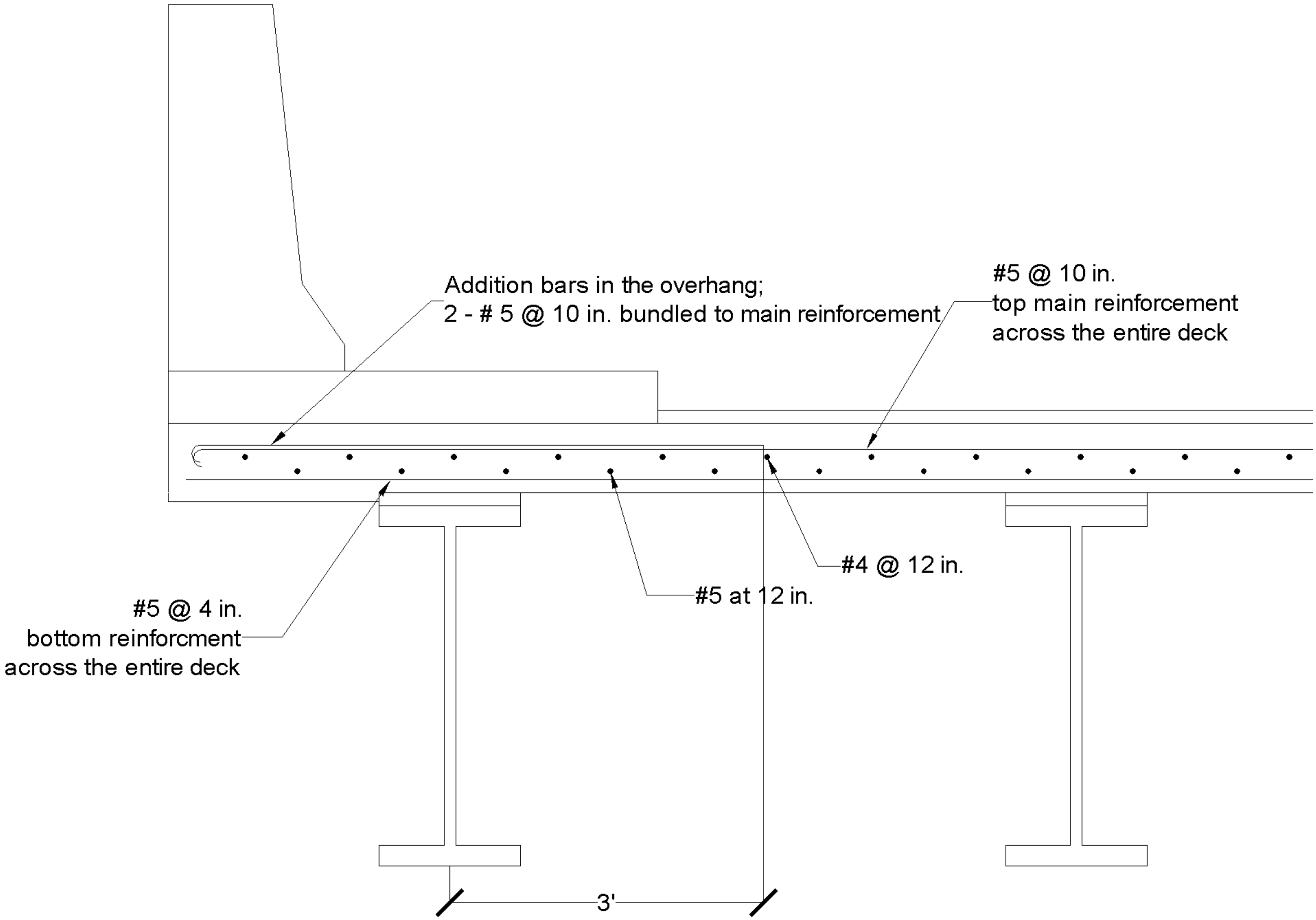


Figure 37: Steel Girder Deck Design

4.3.3 Bearings

The bearing design for the steel girders is nearly identical to the design process for the bearings of the concrete girders. The design process was adapted from the *Design of Modern Steel Bridges* (Heins, C., 1979) and is shown in Figure 38.

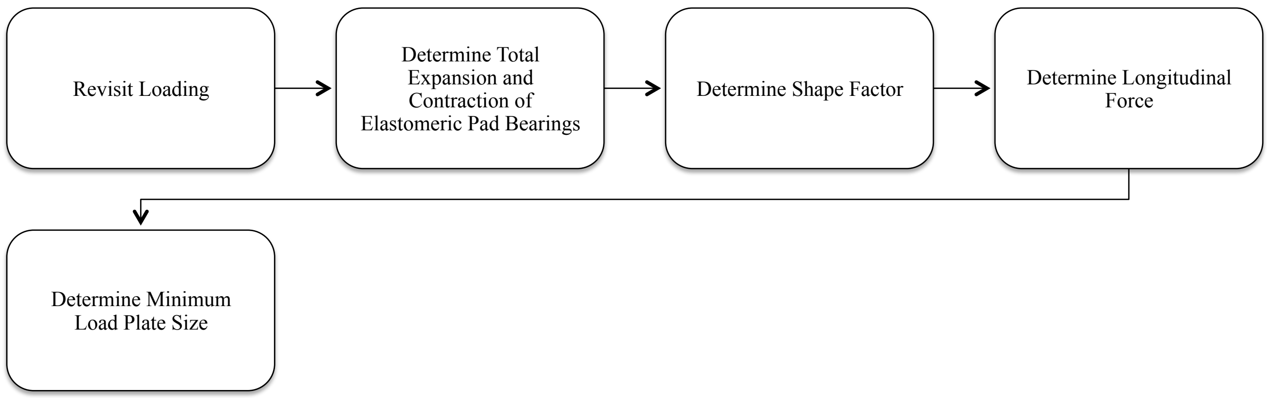


Figure 38: Bearing Design

The final design was a minimum load plate size of 13-¼ x 21-¼" x 1-¼" which is used for each girder. This bearing is very different from that designed for the concrete and was designed for the same conditions. The bearing design depends on the distribution of the load from the girder to the plate. For each design the width dimension must be at minimum as wide as the bottom flange. This was the first point of major difference in the steel and concrete girders. The thickness of both plates is similar but the depth is nearly doubled from the way the steel can create more shear in the bearing. These differences show that the substructure parts of the bridge depend heavily on the girder load distribution and if the rest of the substructure were designed the differences between concrete girder and steel girder design would become more apparent.

4.4 Design Conclusions

As a whole the bridge components selected create similar enough designs that one cannot be chosen as superior. The only significant difference lies in the girder material and girder configurations, but with such similar deck and nonstructural components, without a more detailed substructure design the overall designs are nearly equivalent. The limited design will provide the setting for which the expansion joints tested in the laboratory will be installed. The expansion joint recommended would have tested well in the laboratory but also be cost effective. Laboratory testing results follow this chapter and assist in the determination of the final design selection.

Chapter 5 – Laboratory Testing and Data Analysis

Once the laboratory process begun, there were several small changes that were made in order to obtain accurate results. These included changes to the original joint design, mold construction and testing methods. The testing process was organized so that each joint was assigned an ID, curing times were recorded, and testing was documented through computer analysis and photographs. The results were originally compiled and presented as load versus extension of the silicone sealant. These results were analyzed as well as other comparisons made.

5.1 Joint Testing Summary

Each joint type had six models constructed except for the finger joint which had five models constructed. When joint testing began, each joint was labeled with an identification letter and number to facilitate tracking. The identification letter signified the joint type as shown in the following list:

- Compression – "C"
- Strip Seal – "SS"
- Sliding Plate – "S"
- Finger Plate – "F."

The number corresponded to the order the joint type was tested. For example, "C1" was the first compression joint tested and "SS3" was the third strip seal joint tested. A summary of all tested joints is shown in Table 9 which includes the following:

- Identification,
- Time spent in mold,
- Time spent outside of the mold before testing,
- The mold components, and
- Testing characteristics and observations.

Figure 39: Tensile Compression Joint Test

According to the supplier the sealant curing time is one hour (*WBA Corp*, 2007); however, test results showed that a longer curing period increased the stress and strain the joint could withhold. The first three joints made (C1, C2 & C3) had inconsistent curing times to observe their different tensile test results. Joint C3, which was set in the mold for 6 days and cured outside of the mold for 8 days produced the best results; therefore, all joints thereafter were set in the mold and outside of the mold for around a week each.

The tensile tests were conducted on the Instron model 8803 Dynamic tester in the WPI Civil Engineering Department. The joint was placed in the machine as shown in Figure 39. The machine's control setting was the speed of the top arm. The speed was set to 0.30 inches per minute while the load increased or decreased depending on the strength of the sealant. This speed was in order to better observe the sealant's failure process and reaction to a slow and consistent tensile force.

A variety of observations were noted throughout each joint's tensile test. A summary of the joints and their observations during laboratory testing is shown in Table 8.

Table 8: Joint Summary

Joint ID	Observations
C1	Wood created several indentations on bottom of sealant
C2	Sealant more tacky than C1, perhaps due to less time out of mold
C3	Great results, practically no flaws in poured sealant
C4	Near perfect sealant fill, placed in freezer. Ti= -14C, Tf= 5C
C5	Placed in water for 24 hours. "popped" when failed.
C6	Near perfect sealant fill, great results
SS1	Air bubbles helped increase sealant "caves" and failure
SS2	Partial failure around rod when taking out of mold. Tested with fair results.
SS3	Instant fail, not due to air bubbles
SS4	Similarly an instant fail but slower than SS3
SS5	Perfect pouring (no air bubbles), turned into a compression joint and didn't fail
SS6	First failed in middle, then sealant peeled off each end.
F1	Took on large initial load due to joint steel weight. Did not fail.
F2	Took on large initial load due to joint steel weight. Did not fail.
F3	Took on large initial load due to joint steel weight. Did not fail.
F4	Took on large initial load due to joint steel weight. Did not fail.
F5	Took on large initial load due to joint steel weight. Small section of sealant peeled between the corner of the angle and the plate. Did not fail.
S1	Slowly came apart rather than failing immediately, left some sealant pieces on bare angle, sliding plate was NOT connected with sealant
S2	Plate sealant failed before testing, sealant against plate failed first, final failure was sealant wedged into corner of angle and plate
S3	Very thin strip of sealant that didn't fail between plate.
S4	Sealant initially pulled away from intersection of sliding plate and steel angle
S5	Wasn't welded correctly, results are inadequate

S6	Sealant between plate intact at beginning of testing

5.2 Compression Joints

The compression joint was the simplest and easiest to construct of the four joints used in this project. Figure 40 shows the results produced from the tensile testing, Figure 41 shows the results in terms of stress and strain, and Figure 42 shows photographs of each compression joint while in tension.

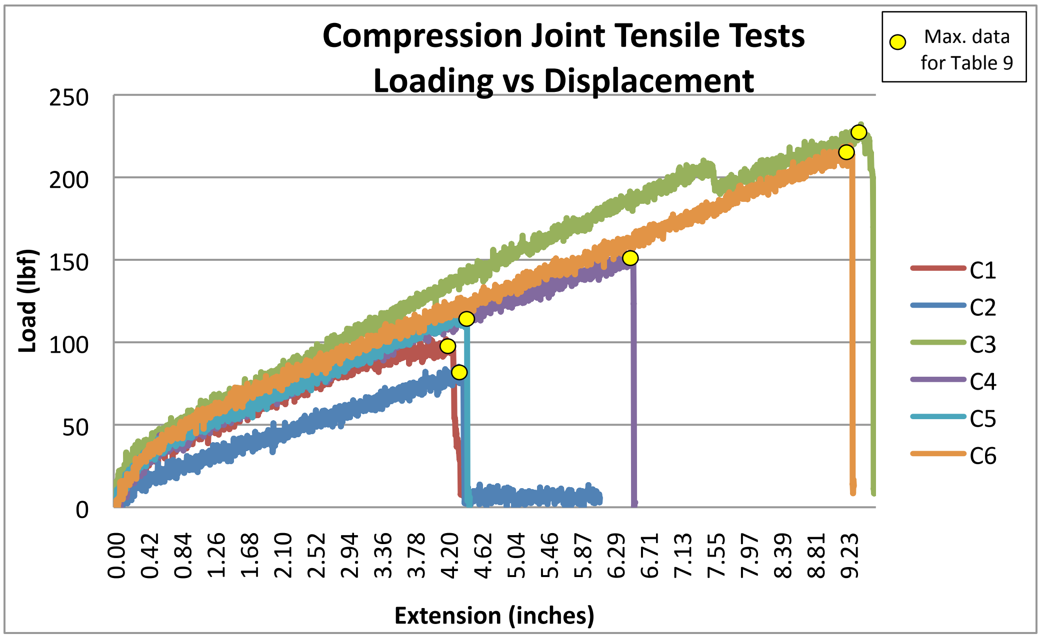


Figure 40: Compression Joint Test Results

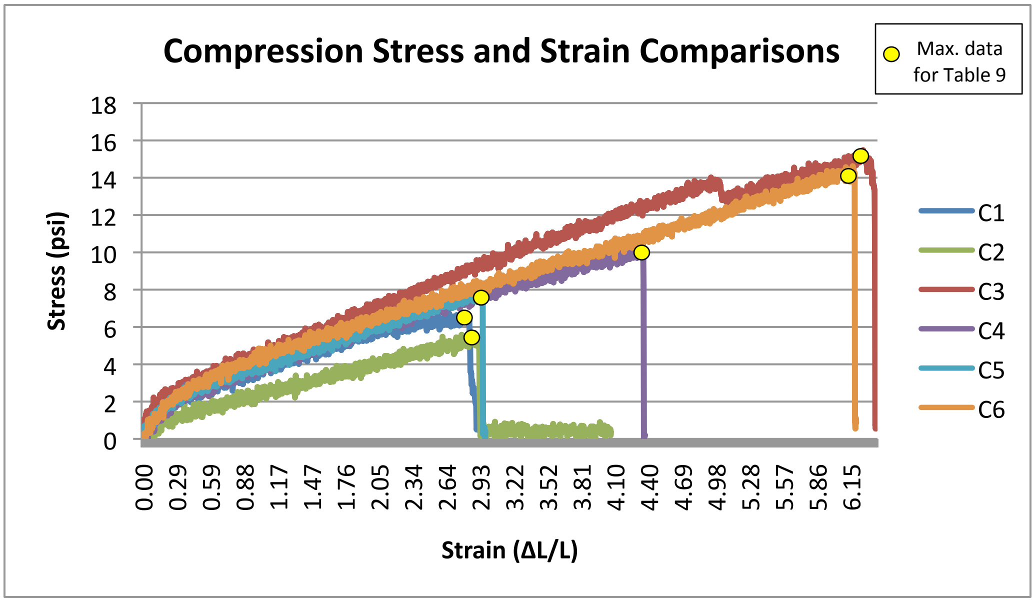
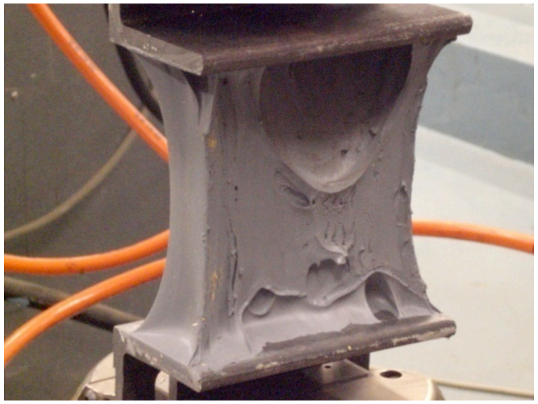


Figure 41: Compression joint stress and strain comparisons



C1

C2

C3

C4 (Frozen)

C5 (Water Submerged)

C6

Figure 42: Compression Joints during Testing

Because the compression joints were the first joints to construct and test, there were inconsistencies with the testing procedure including curing time as well as added experimental variables. Both C1 and C2 had the wooden base versus plexiglass which created more flaws in the sealant during testing (explained in Section 3.3.4). This, added with less curing time, resulted in the least loading/stress and extension/strain in testing compared to joints C3 through C6 (shown in the graphs in Figures 40 and 41). Experimental variables included freezing and water submerging two compression joints.

New England weather is variable in the United States and bridge joints must endure extreme temperature and moisture. To simulate these experiences, joint C4 was placed in a freezer for 24 hours before testing, and joint C5 was submerged in water for 24 hours before testing. Shown in Figure 40, C4 continued to expand with little initial flaws or failure, whereas C5's initial failure began at the edge of the steel angle where water may have penetrated and weakened the sealant. Rather than failing completely by the sealant slowly peeling away (like most other joints), C5 failed instantly—the sealant "popped" from the bottom angle and separated from the steel.

The graphs in Figures 40 and 41 shows the comparison between the strength and deformations of each compression joint tested. The yellow dots on Figure 38 are where the maximum stress and strain data were defined for each specimen. Table 9 summarizes the maximum results for each compression joint tensile test. The last row only summarizes the averages of joints C3 and C6 because they had consistent controls of curing time and no additional variables of freezing or water submersion. The averages of C3 and C6 will be used to compare data with the other types of joints

Table 9: Compression Joint Maximum Testing Data

Joint	Load (lbf)	Extension (in)	Stress (psi)	Strain ($\Delta L/L$)
C1	93.9	4.1	6.5	2.8
C2	78.5	4.3	5.1	2.9
C3	227.8	9.4	15.2	6.3
C4	147.6	6.4	10.2	4.3
C5	117.6	4.4	7.6	2.9
C6	211.9	9.3	14.0	6.2
TOTAL AVG.	**146.2lbf**	**6.3 in**	**9.8 psi**	**4.2 $\Delta L/L$**
AVG. of C3, C6	**219.9 lbf**	**9.4 in**	**14.6 psi**	**6.3 $\Delta L/L$**

As mentioned previously, C1 and C2 sustained the lowest maximum strength and deformation of the six tested compression joints. The best performing compression joints in load capacity and deformation were C3 and C6. Neither experienced an experimental variable and both were cured for two weeks from when the sealant was poured to the testing. Both C3 and C6 held a tensile load greater than 200 pounds and deformed over 9.2 inches before the sealant fully failed (no sealant connected to the angles).

5.3 Strip Seal Joints

The strip seal joint constructed in this project had approximate dimensions of a strip seal joint used in the field; therefore, it consisted of a greater sealant volume than the compression joint, but was the same construction as a compression joint with a steel backer rod. Figure 43 and 44 show the tensile test results and the stress/strain comparisons, respectively. Figure 45 shows photographs of each joint during the tensile test.

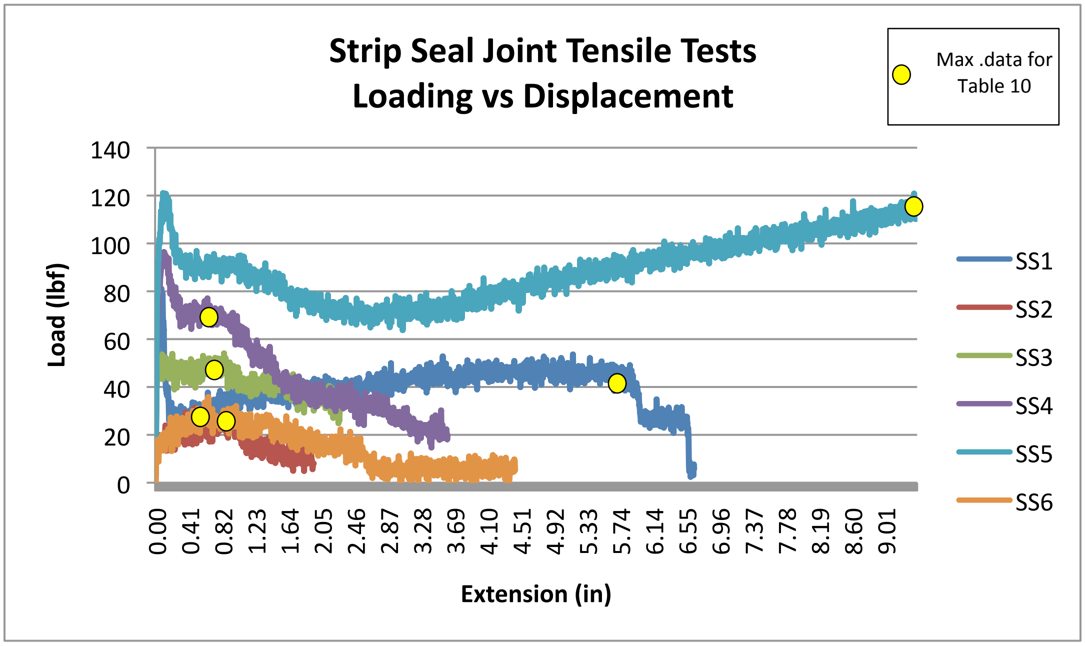


Figure 43: Strip Seal Joint Test Results

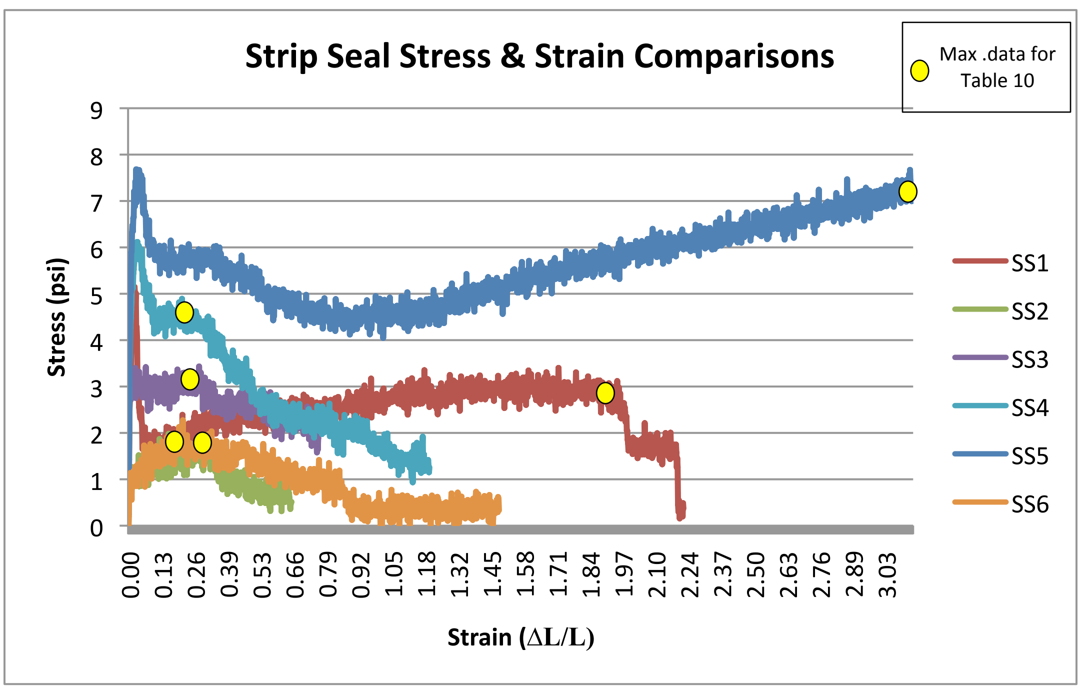
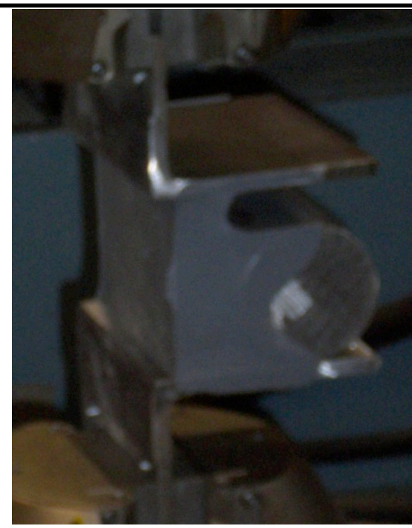


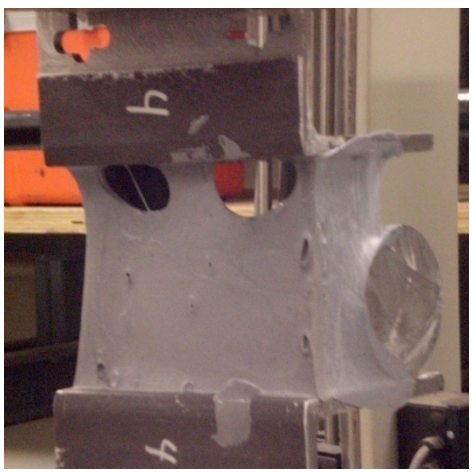
Figure 44: Strip Seal joint stress and strain comparisons

SS1

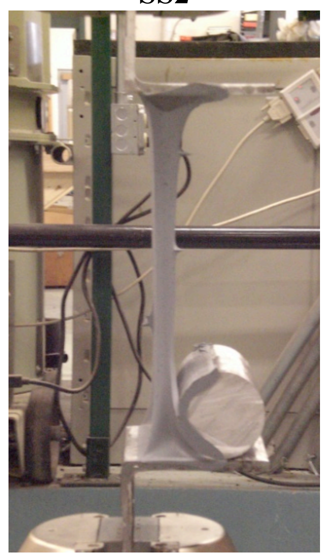


SS2

SS3



SS4



SS5

SS6

Figure 45: Strip Seal Joints during Testing

The strip seal joint data is summarized numerically in Table 10.

Table 10: Strip Seal Joint Maximum Testing Data

Joint	Load (lbf)	Extension (in)	Stress (psi)	Strain (ΔL/L)
SS1	47.1	5.7	3.0	1.9
SS2	29.6	0.9	1.9	0.3
SS3	50.1	0.8	3.2	0.3
SS4	72.0	0.6	4.6	0.2
SS5 (no failure)	117.3	9.3	7.4	3.1
SS6	28.3	0.6	1.8	0.2
AVERAGE	**57.4 lbf**	**3.0 in**	**3.7 psi**	**1.0 ΔL/L**

The maximum values are taken where the yellow dots appear on the graphs in Figures 43 and 44. The ultimate maximum stress and load in the trend line is not always considered, because there was an initial large load or stress needed to handle the gravitational force of the heavy steel angle. Once this angle load became consistent, the highest load and stress throughout the rest of the tensile test was used as the load and stress shown in Table 10.

The strip seal bridge joint was designed with a backer rod to support the overall joint and therefore the bridge connection. However in these project tests, it was discovered that the circular shape of the rod makes it is easier for the sealant to peel away if a tensile load is applied. The test photographs in Figure 45 show how the joint initially fails between the sealant and the rod on one side. Once the sealant pulls away from the rod, in most cases, it fails in the center of the angle and continues to strip away towards the angle sides. Different situations included SS2 which was partially failed prior to testing (while taking the joint out of the mold) and SS5 which showed similar characteristics to a compression joint once the sealant stripped from the backer rod and did not fail (the Instron machine reached its maximum deformation length).

Figure 43 compares each of the six strip seal joint's results in loading capacity and deformation, while Figure 44 shows the stress versus strain values for the joints. Because the strip seal joint has a larger space between angles than the compression joint, a larger volume of silicone sealant was used for each joint. The larger amount of sealant used increased the probability of air voids which most of the strip seal joints contained and were noticeable on the top of the joint. All joints except SS2 and SS5 had air bubbles visible on top of the cured sealant. SS5's loading versus displacement line mirrors that of the compression joints (Figure 40) after a deformation of 2.7 inches. At that point the sealant had peeled from the backer rod and the strip seal joint was behaving like a compression joint in tension. SS2 was the weakest joint because it was partially failed before testing. The other joints followed similar graphing trends in that they held a large load initially, which then dropped as the sealant peeled from the rod. The load capacities for SS3, SS4, and SS6 continue to decrease as the sealant fails in the middle of the angle and then moves to the angle sides. Joint SS1 produces a different graph that begins to slowly increase in load until an approximate deformation of 5.10 inches. This may be because the sealant did not all initially fail at the backer rod. Although the sealant begin to fail at the middle of the angle like SS3, SS4, and SS6, more sealant continued to make contact with the backer rod—creating an overall stronger bond.

5.4 Sliding Plate Joints

The sliding plate joints were the most complicated to construct and produced the largest variance in results. Figure 46 and Figure 47 show the testing results and the stress versus strain relationships, respectfully. Figure 48 shows each sliding plate joint during the tensile test.

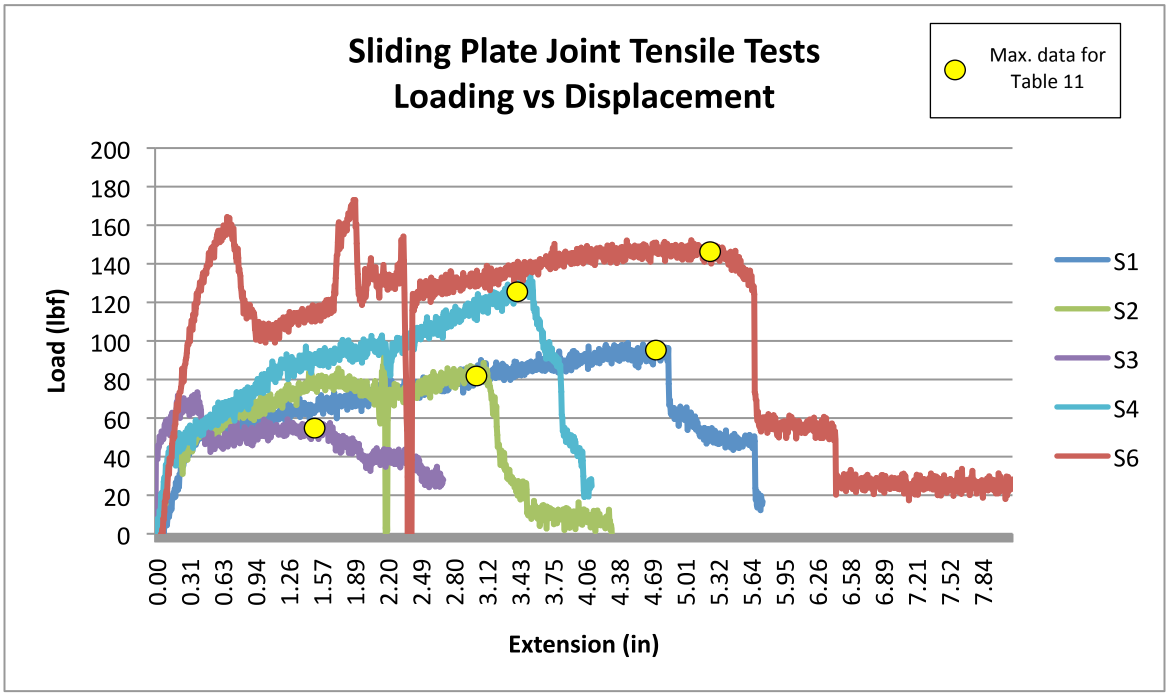


Figure 46: Sliding Plate Joint Test Results

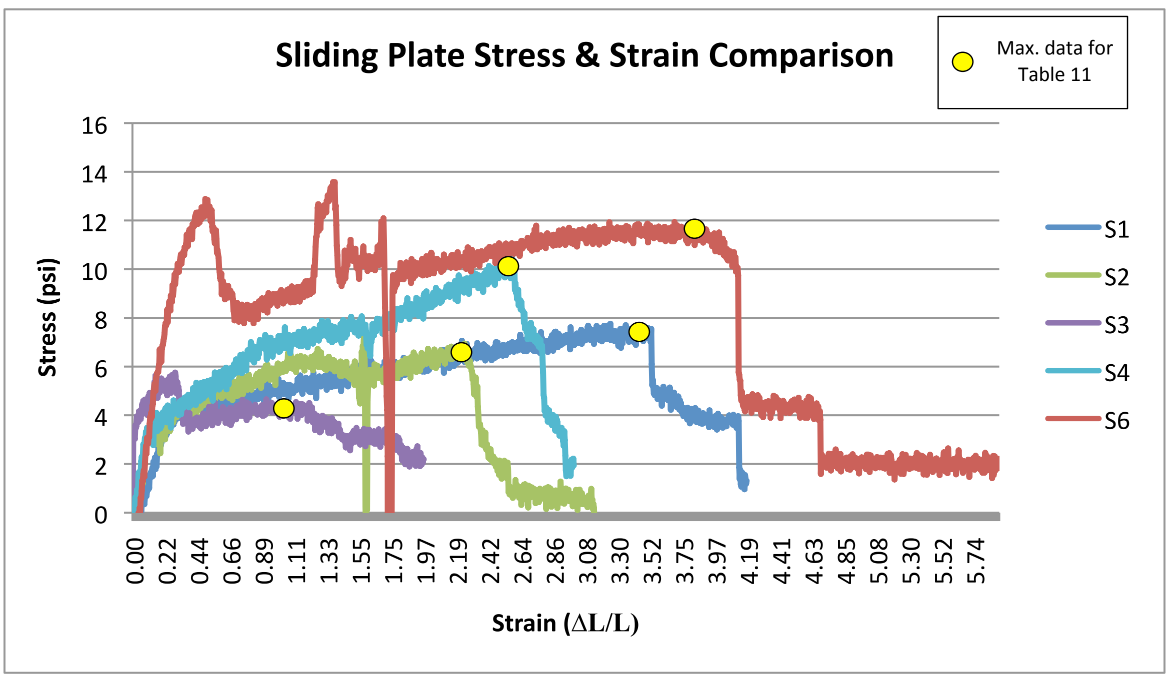


Figure 47: Sliding Plate joint stress and strain comparisons

S1

S2

S3

S4

S5

S6

Figure 48: Sliding Plate Joints during Testing

The final joint design for this project included sealant between the two top sliding plates. The first sliding plate joint constructed—S1—did not include the sealant between the plates. S2 through S6 did include the small sealant strip; however, by the time these joints were tested the sealant had mostly failed between the sliding plates. This premature failure was from accidentally putting too much tensile load on the sealant while moving the joints around and from the strip of sealant not being able to handle the steel weight when initially being placed in the Instron machine. Joint S5 is not included in the results because it was improperly constructed. As shown in Figure 48, the angle was not welded to the plate above; therefore, the joint was never connected initially. Looking at Figure 48 there is not a consistent pattern with how the sealant reacts to the tensile loading which transforms to the varied graph results shown in Figure 46. The inconsistent failure pattern could be because of the number of stresses applied to the sealant within the strip seal joint. These stresses include the following forces between the sealant and the following:

- Angle,
- Top Plate,
- Wedge between angle and plate,
- Plates that create the sliding plates on top of the joint,
- The weight that the previous four elements bore on the sealant in addition to the Instron loading,
- Compressive force from the sliding plate.

The compressive force is created from the sliding plate when the joint deforms past the plate length (such as S2 and S3 shown in Figure 48).

These stresses created by the sliding plate joint construction may have produced the varied results in the graphs of Figures 46 and 47. Where the displacement becomes larger than the sliding plate, most graph lines (S2, S4, S6) drop dramatically in load. This rapid drop involves the compressive force from the sliding plate. Once the sliding plate is no longer resting on the other top plate, it slopes in towards the sealant and causes a compressive force. The inward sloping is a result of torque—most sliding plate joints were unintentionally created with slightly uneven grips that the Instron tester compresses.

The summary of the maximum loads and stresses with corresponding extensions and strains (respectively), are shown in Table 11.

Table 11: Strip Seal Joint Maximum Testing Data

Joint	Load (lbf)	Extension (in)	Stress (psi)	Strain (ΔL/L)
S1	94.6	4.8	7.4	3.5
S2	88.5	3.1	6.9	2.3
S3	57.4	1.7	4.5	1.2
S4	130.5	3.6	10.2	2.6
S5 (no results)	----	----	----	----
S6	146.8	5.2	11.5	3.8
AVERAGE	**103.4lbf**	**3.7 in**	**8.1 psi**	**2.7ΔL/L**

The maximum points are taken where the yellow dots are placed on Figures 46 and 47. Similar to the strip seal joints, the maximum load was taken after a steady incline rather than the initial loading caused by the weight of the steel. According to this data thus far, the sliding plate joint performs better in tension than the strip seal joint but does not exceed the results for the compression joint.

5.5 Finger Joints

The finger joints were the last to construct because they could not be machined immediately in-house. The steel plates were taken to a local water jet machinist (Hydro-Cutter of North Oxford, Massachusetts) to form the fingers before they could be welded to the steel angles and the sealant was then placed. Figure 49 shows the tensile test results of loading versus displacement, whereas Figure 50 applies the testing data to the sealant dimensions and area to produce stress versus strain graphs. Additionally, Figure 51 shows each finger joint during tensile testing.

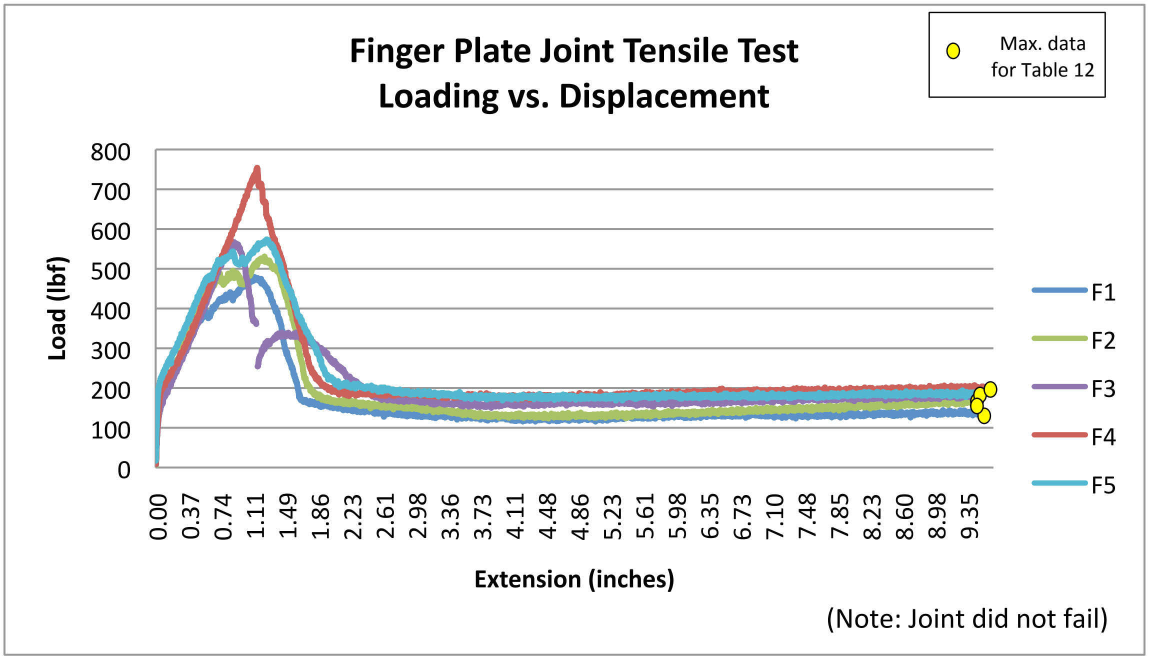


Figure 49: Finger Plate Joint Test Results

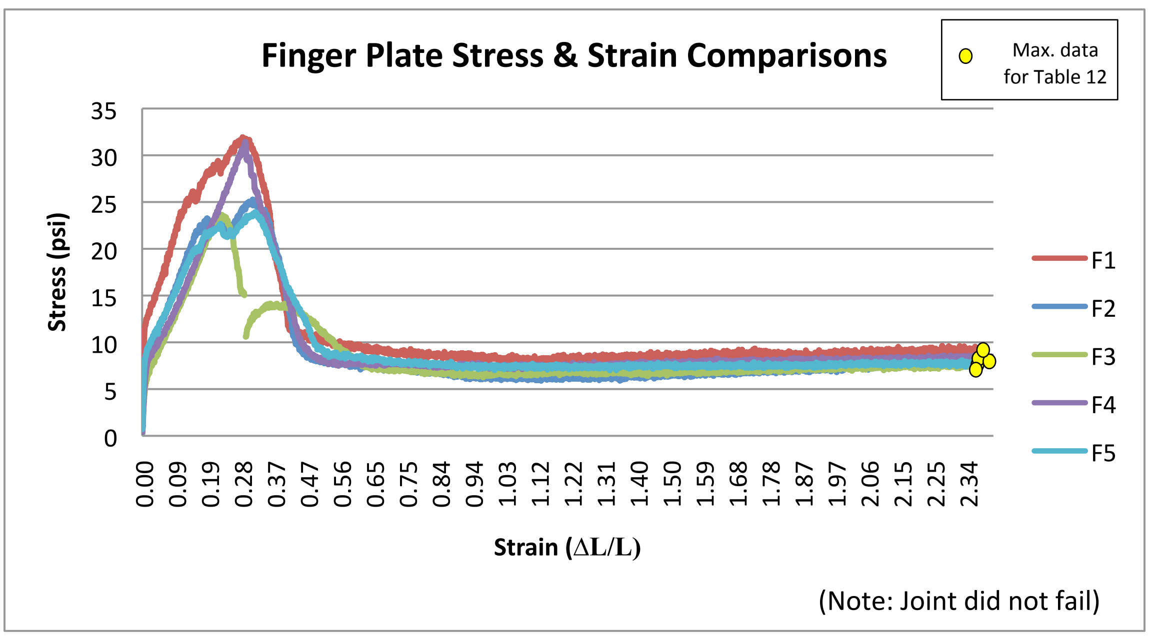


Figure 50: Finger Plate joint stress and strain comparisons

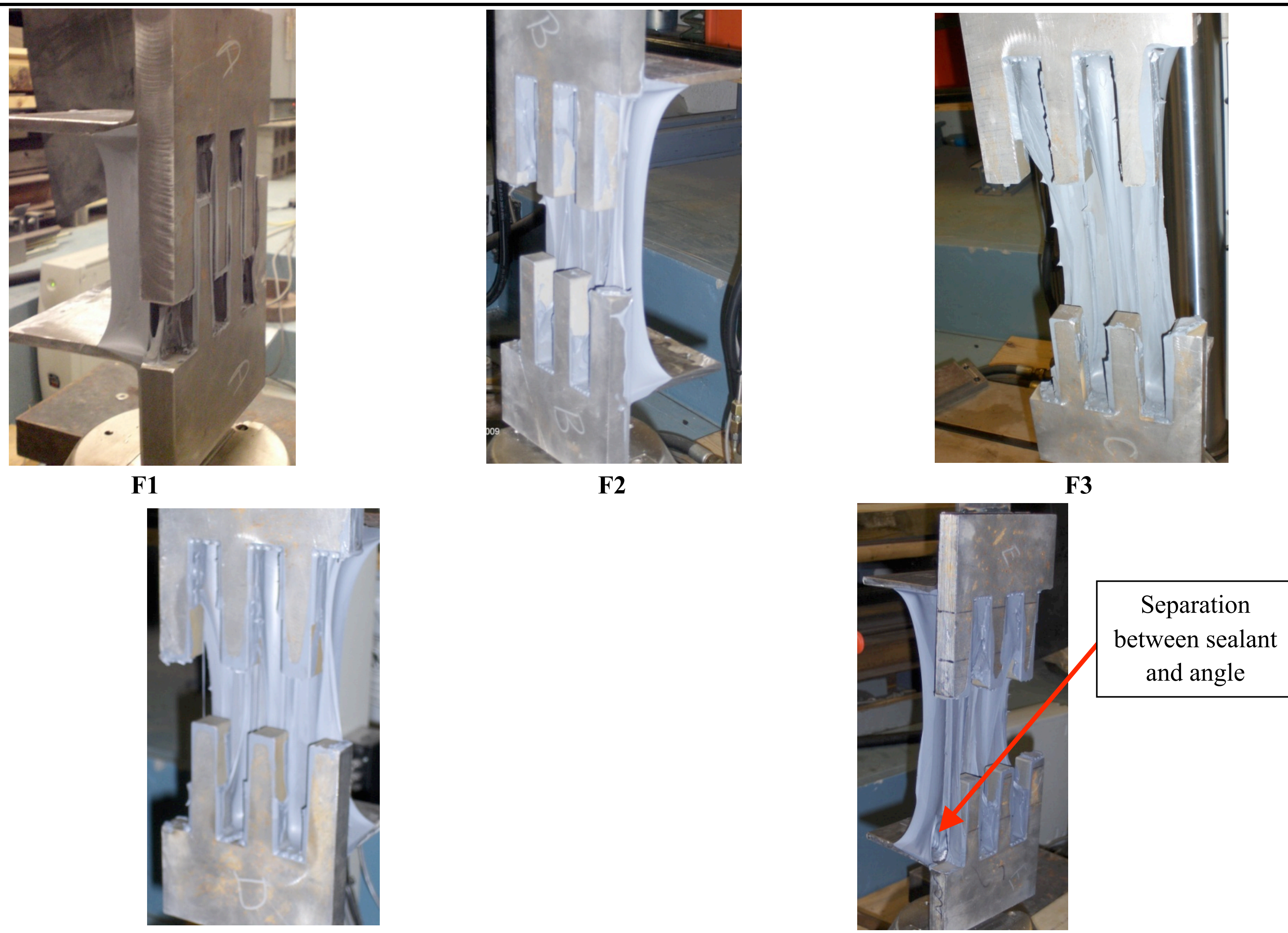


Figure 51: Sliding Plate Joints during Testing

As shown in both graphs, there is a peak load in the beginning of the testing and then the load increases slightly until the end of the graph; none of the finger plate tests' loading quantities decline to zero at the end of and testing because none of the finger plate joints failed (when the steel angles are no longer connected). For each finger joint tested, the sealant between the fingers failed first and then the sealant began tearing from the top finger plate. The Instron testing machine could expand the tensile joint to a maximum of 9.4 inches. At this point, finger plate joints, F1, F2, F3, and F4 were still completely adhered to each angle face. Joint F5 had a small section on the bottom angle where the sealant had torn away from the steel angle (marked in Figure 51); but, the sealant was still fully connected to each angle everywhere else. The initial peak load was caused by the weight of the joint. Similar to the strip seal joint, the angles were larger than those for the compression and sliding plate; and in addition to the strip seal joint, the finger plate joint had a one-inch thick steel plate welded to the angles which required a greater initial force from the Instron testing machine. Once the peak load and stress were reached, they declined until the finger plates were pulled apart from each other (when the extension equals 4 inches). Once this point was reached, the load and stress slowly inclined until the tensile machine could not extend further.

The maximum loading and stress occurring in each test is summarized in Table 12.

Table 12: Finger Plate Joint Maximum Testing Data

Joint	Load (lbf)	Extension (in)	Stress (psi)	Strain ($\Delta L/L$)
F1	136.5	9.4	9.2	2.4
F2	168.7	9.4	8.0	2.4
F3	182.7	9.4	7.6	2.4
F4	197.8	9.4	8.2	2.4
F5	181.8	9.4	7.6	2.4
AVERAGE	**173.5lbf**	**9.4 in**	**8.1 psi**	**2.4$\Delta L/L$**

The limitations on the Instron testing machine did not allow any of the joints to fail; therefore, this data is not representative of the constructed joint. Maximum values were taken at the end of the graph rather than the initial peaks because the initial loading was due to the steel weight. The following Section 5.6 will take this data and the average data in the previous sections and compare each type of joint.

5.6 Summary of Tensile Testing

Table 13 summarizes the average maximum load, extension, stress and strain (from Tables 9, 10, 11 & 12) for each joint type that was tested. Additionally, the maximum loading and maximum stress are shown in Figures 52 and 53, respectively. Stress and strain are considered because it normalizes the testing data to more easily compare each joint. Each type of joint has a different original sealant length and area which the load and extension data do not account for.

Table 13: Joint Average Maximum Testing Data

Joint	Load (lbf)	Extension (in)	Stress (psi)	Strain (ΔL/L)
Compression (C3 & C6)	219.9	9.4	14.6	6.3
Strip Seal	57.4	3.0	3.7	1.0
Sliding Plate	103.4	3.7	8.1	2.7
Finger Plate	173.5	9.4	8.1	2.4

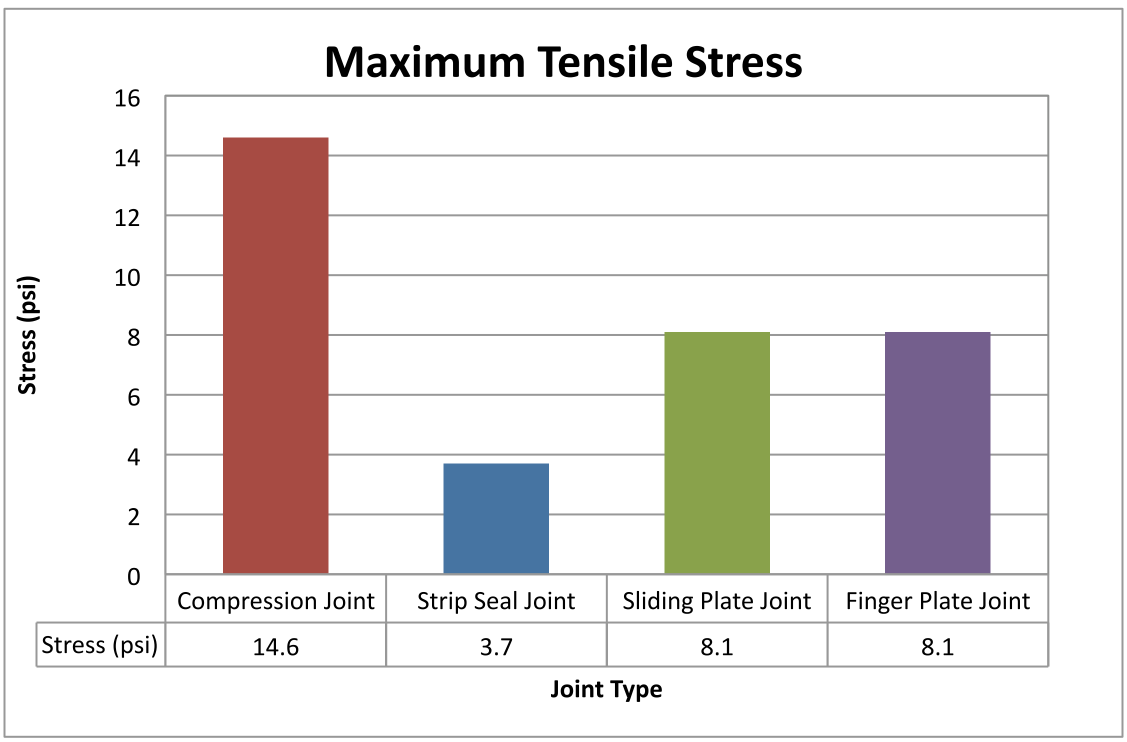


Figure 52: Maximum stress comparison

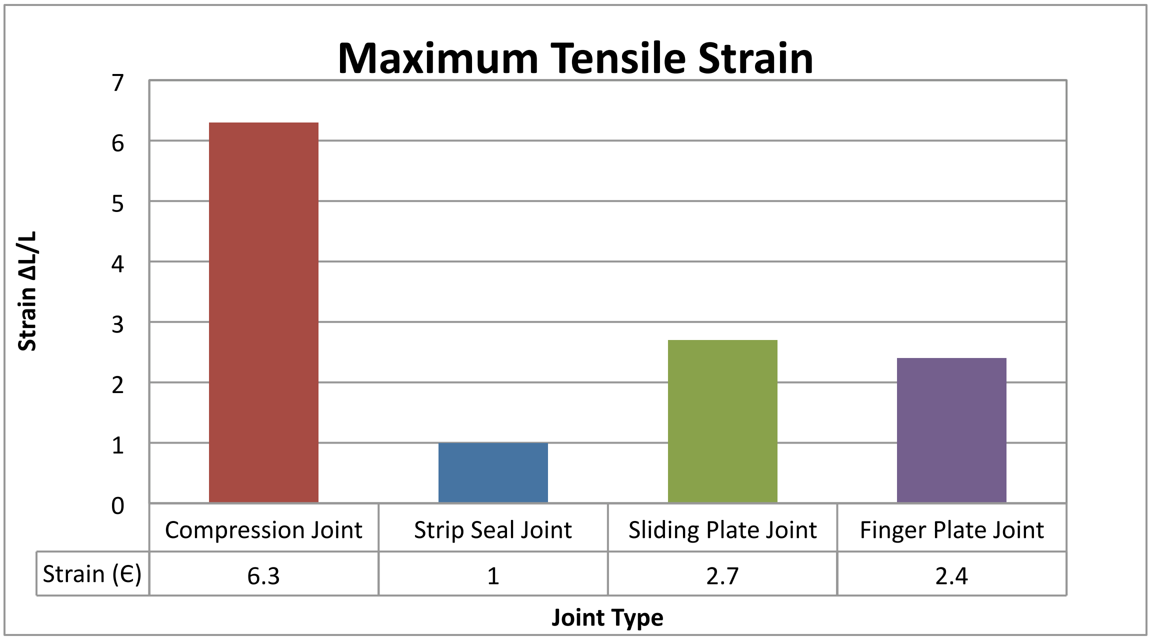


Figure 53: Maximum Strain Comparison

From largest strain to smallest strain the results are the following:

1. Compression Joint
2. Sliding Plate Joint
3. Finger Plate Joint
4. Strip Seal Joint

The maximum tensile stress follows the same trend as above except that the sliding plate joint and the finger plate joint have equal values of 8.1 psi. Because the finger plate joint results are not representative of failure, their proper placement among the other types of joints is difficult to establish. For instance, in order for the finger plate joint to have received a similar average strain of the compression joint, it would have had to extend over 25 inches (the testing stopped after 9 inches).

Disregarding the finger plate results, the compression joint noticeably surpassed the average maximum stress and strain values of the other joints. This may be because of the simplicity of the joint. The compression joint's sealant was only being pulled by the steel angle on each side; whereas, the strip seal and sliding plate joints had other steel rods or plates that may have helped to initiate the sealant pulling away from the angles. The computer analysis of the compression joint sealant mold further examines the stresses and strains occurring in the sealant.

Chapter 6 – Computer Analysis

The ANSYS sealant model was created to better understand what occurred in the material during the laboratory testing. The process of using ANSYS is explained in Section 3.2. Once the model is created and loads applied, the solve command created several types of output data. This project focused on, the following results:

1. The comparison of ANSYS to the laboratory section, based on
   a. Deformed shape
   b. Stress and strain
2. Strain contours
3. Stress contours

6.1 ANSYS Simulation Compared to Laboratory Data

The deformed sealant produced in ANSYS mimics the laboratory data visually and numerically. Figure 54 compares the ANSYS deformation to the actual sealant deformation observed in the tensile test. The ANSYS picture on the left shows the sealant deformed around 4 mm (0.15 inches). This deformation would simulate the tensile test 30 seconds after starting. After approximately 90 seconds from the start of testing, the photograph on the right was taken. The deformation is more defined in the laboratory photograph; however, it is similar to the shape produced with computer simulation.

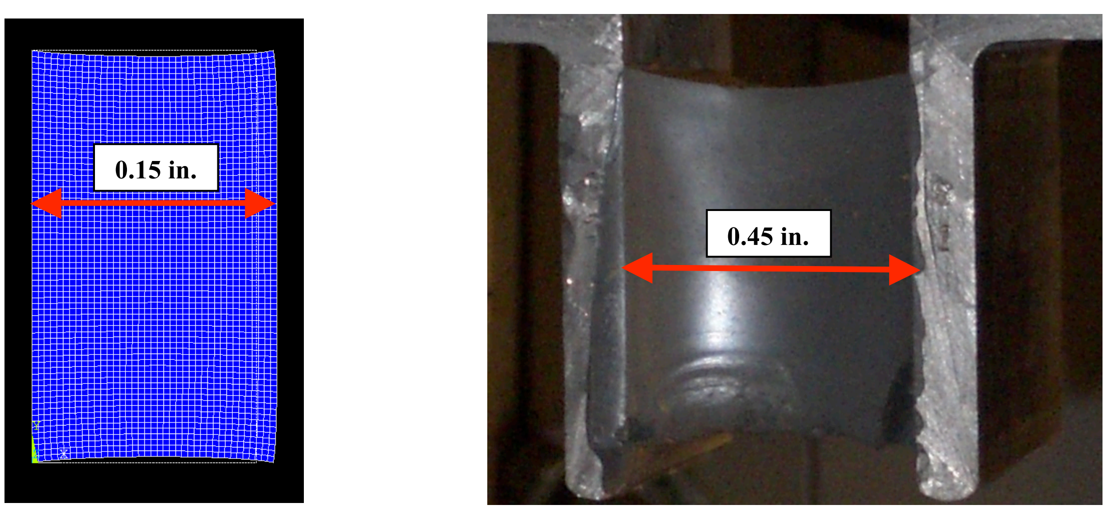


Figure 54: Sealant deformation Comparison, ANSYS vs. Laboratory

The ANSYS simulation was completed after 1 time step which equates to approximately 30 seconds in the laboratory testing (as mentioned in the previous paragraph); the stress and strain that the computer model produces is similar to the average data received in the tensile testing. The maximum pressure load applied to the ANSYS sealant model was 9,500 Pa (1.4 psi). The horizontal, x-direction strain produced by this load was 0.10 (4mm deformation divided by 38.1mm original width). Figure 55 shows this ANSYS data applied to the results from the laboratory data. The window on the right is an enhanced version (0 to 110 seconds) of the laboratory tensile test. The red arrows mark the ANSYS stress, strain data point—(0.10, 1.4). This point is located close to the average of all tensile testing data points at a 0.10 strain which demonstrates the following:

1. The ANSYS model produces accurate results to simulate laboratory testing
2. The tensile tests are more legitimate from this additional data

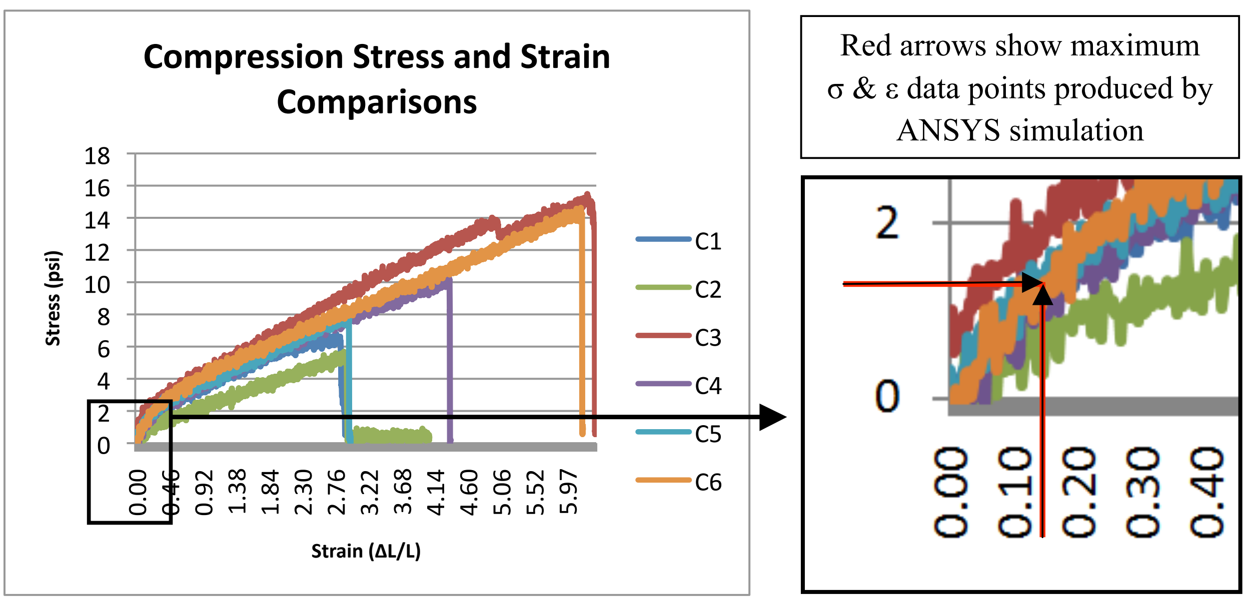


Figure 55: Laboratory Data to compared to ANSYS Analysis

6.2 Strain Analysis

ANSYS's finite element simulation additionally produced results that were not recorded in the laboratory tensile tests, such as strain and stress contouring of the sealant material. Figure 56 illustrates how the contour graphs can be created, as well the strain occurring in the x-direction in the sealant model. As shown in this figure, the strain is largest where the lightest blue sections are—two sections in the center of the sealant, as well as in the corners on the left side. This can be compared with the photograph of the compression joint shown previously in Figure 54, where the bottom corner of the sealant is beginning to separate from the steel angle.

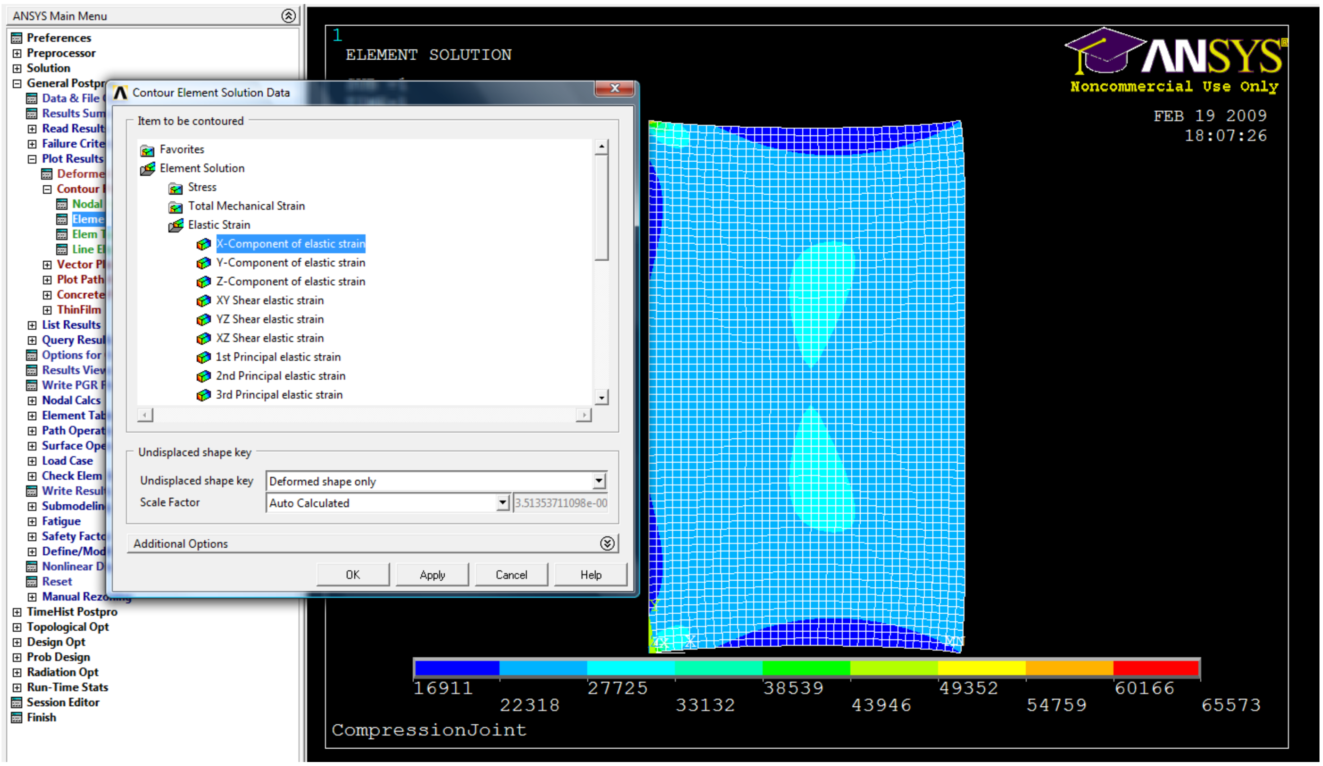


Figure 56: ANSYS Element Solution - X-Component of elastic Strain

The strain can also be viewed by solely examining the vertical, y, direction of this model as shown in Figure 58. The vertical strain is calculated from the given Poisson's ratio of 0.4 for the sealant [$\mu=\varepsilon(y)/\varepsilon(x)$]; therefore, the vertical strain equals the horizontal strain multiplied by a factor of 0.4. However, the adhesion of the sealant to the plates creates a more complex state of stress and strain, and therefore, the portion of the sealant near the center would best exhibit the equation [$\varepsilon(y)=(-\mu)*\varepsilon(x)$]. The red contours on the right and left sides illustrate where the greatest vertical strain occurs under the given loading conditions. This phenomenon could further explain the reaction of the sealant peeling from the middle of the angle as shown in the SS3 Joint in Figure 57.

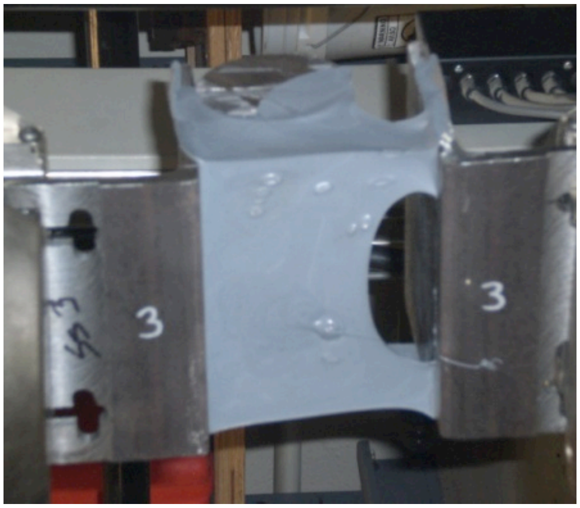


Figure 57: Strip Seal joint showing peeling mid-angle

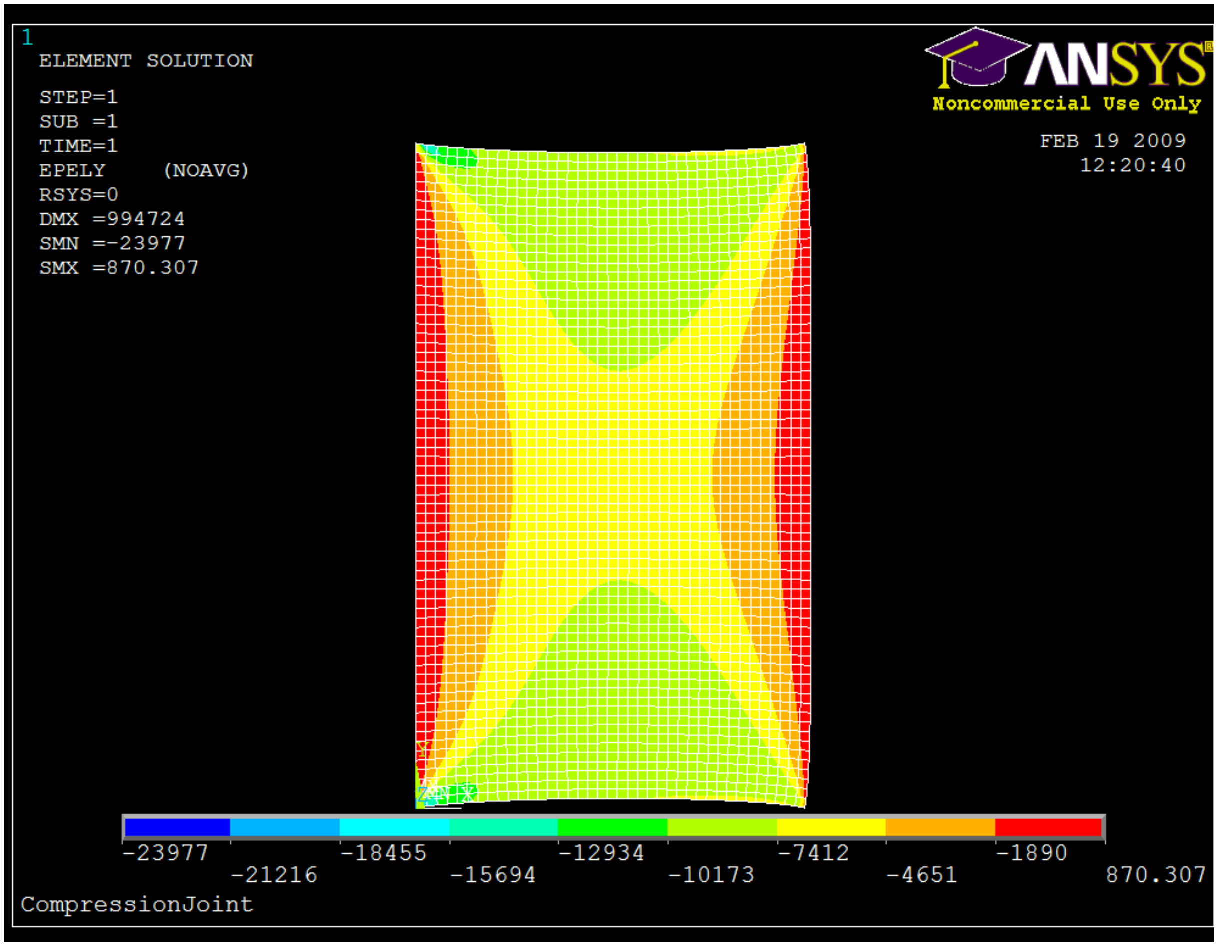


Figure 58: ANSYS Element Solution - Strain in Y-Direction

6.3 ANSYS Stress Analysis

Stress analysis, similar to strain analysis, can help understand the reactions occurring within the sealant. ANSYS provides an output solution that allows the user to query stress at any nodal point within the model. The contour plot shows an overall illustration of where the maximum stress is found; however, the query allows one to find the calculated stress occurring at any node. An example of this output it shown in Figure 59 showing stress values in the x-direction. The stress is shown as 9,500 psi at the far right, considering that is the pressure that was applied to that side. The stress is 9% higher in the center of the sealant model—10,312 Pa. Additionally the stress at the top left corner of the sealant—26,263 Pa, is 176% greater than the applied pressure. This corresponds with the strain contours in Figure 56, showing the highest strain in the left corners and the center of the model.

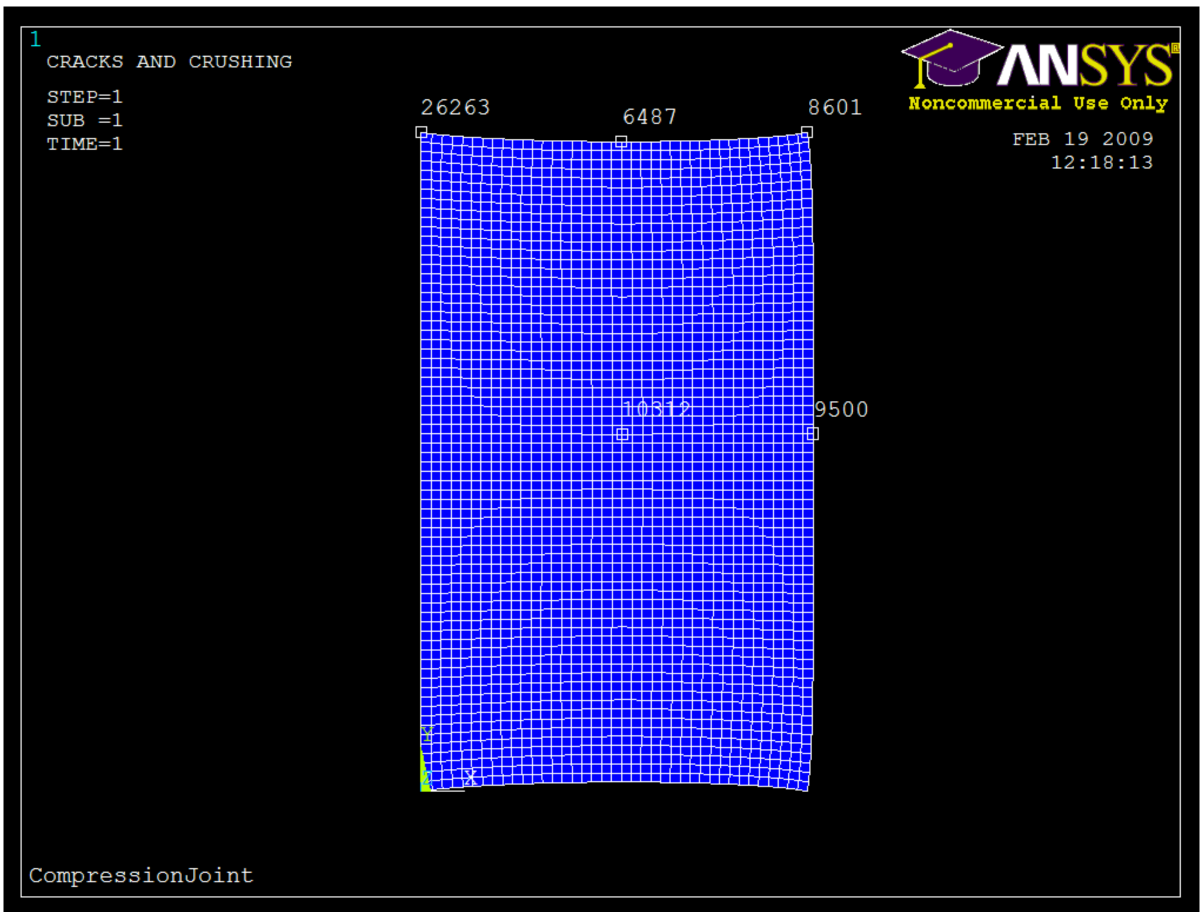


Figure 59: ANSYS Stress Points in the X-Direction

Similar to the strain contours, stress contour graphs produced in ANSYS help visualize where the sealant is experiencing the high and low stresses. Figure 60 illustrates the element solution for the x-direction of stress, and Figure 61 shows the element solution for the y-direction of stress. The contours are similar to the strain contours shown in Figure 56 and 58: the stress in the x-direction is greater in the center and the left corners of the sealant model (~12,283 Pa), and the stress in the y-direction is greatest at the center of the right and left sides of the sealant model (~2,644 Pa).

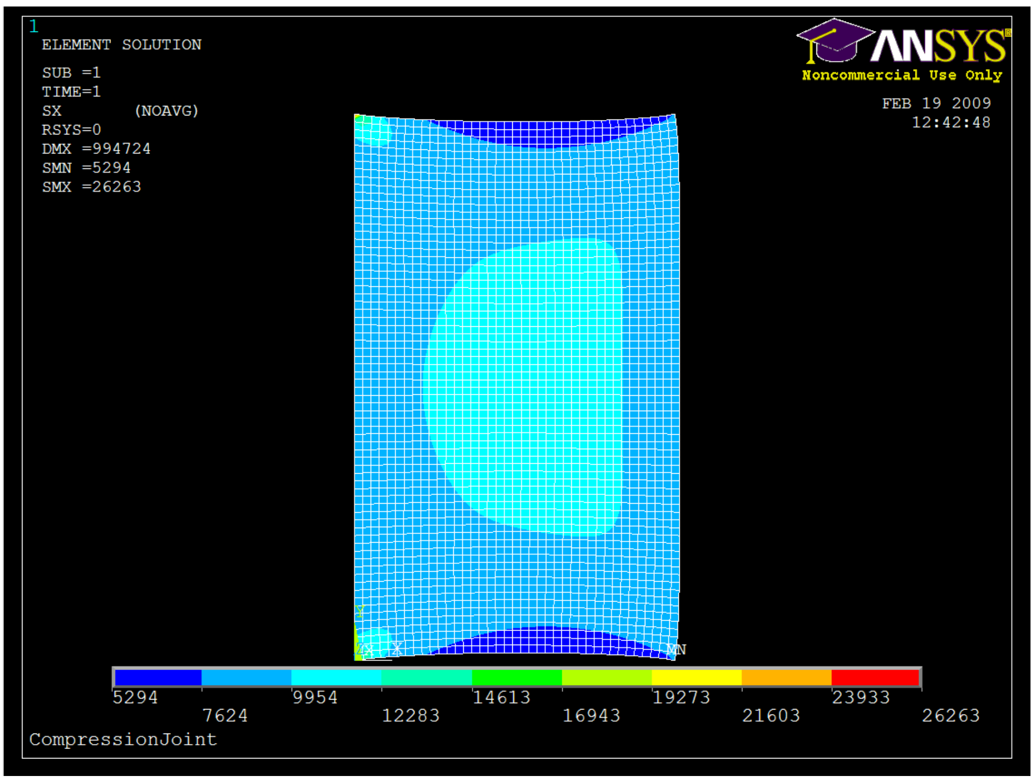


Figure 60: ANSYS Element Solution - X Component of Stress

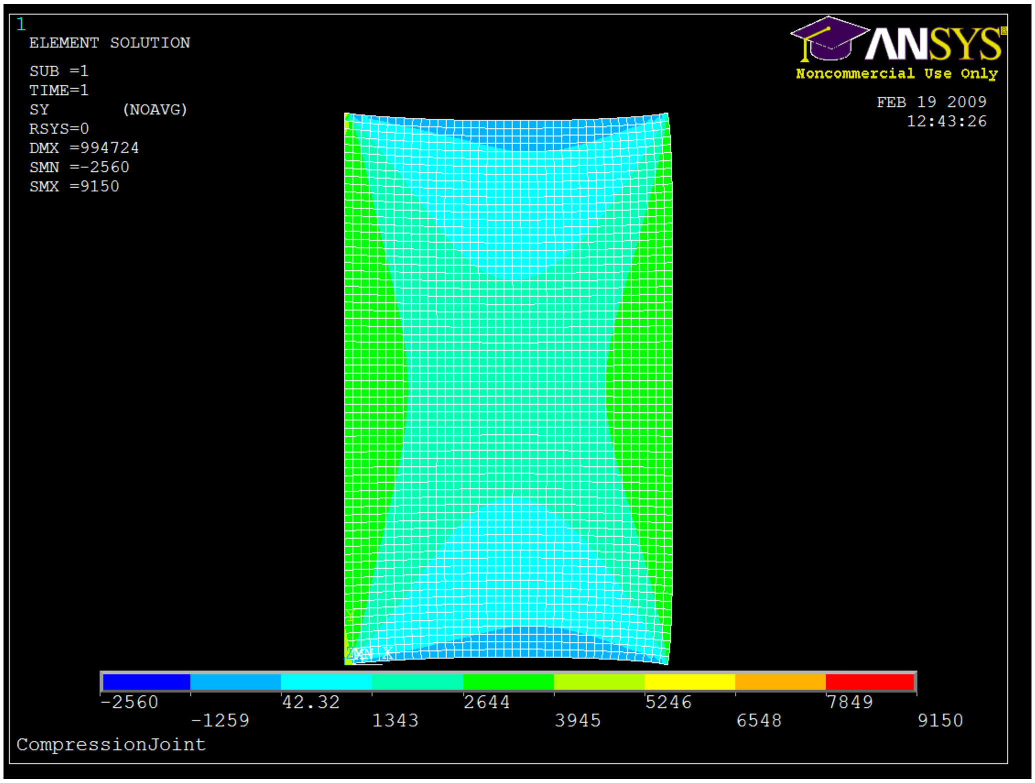


Figure 61: ANSYS Element Solution - Y Component of Stress

6.4 Additional ANSYS Capabilities

In addition to obtaining the stress and strain reactions that pertain to this project, ANSYS is capable of simulating experiments that were not completed due to time and budget constraints. An example is applying compressive pressure loading to the sealant, as shown in Figure 62—an experiment that this project was unable to complete. This showed a similar strain to the tensile loading, however, in the opposite direction. The deformation in the x-direction is approximately -4mm, and the deformation in the y-direction is approximately +1mm. This knowledge can assist in future experimentations to lessen project costs and time.

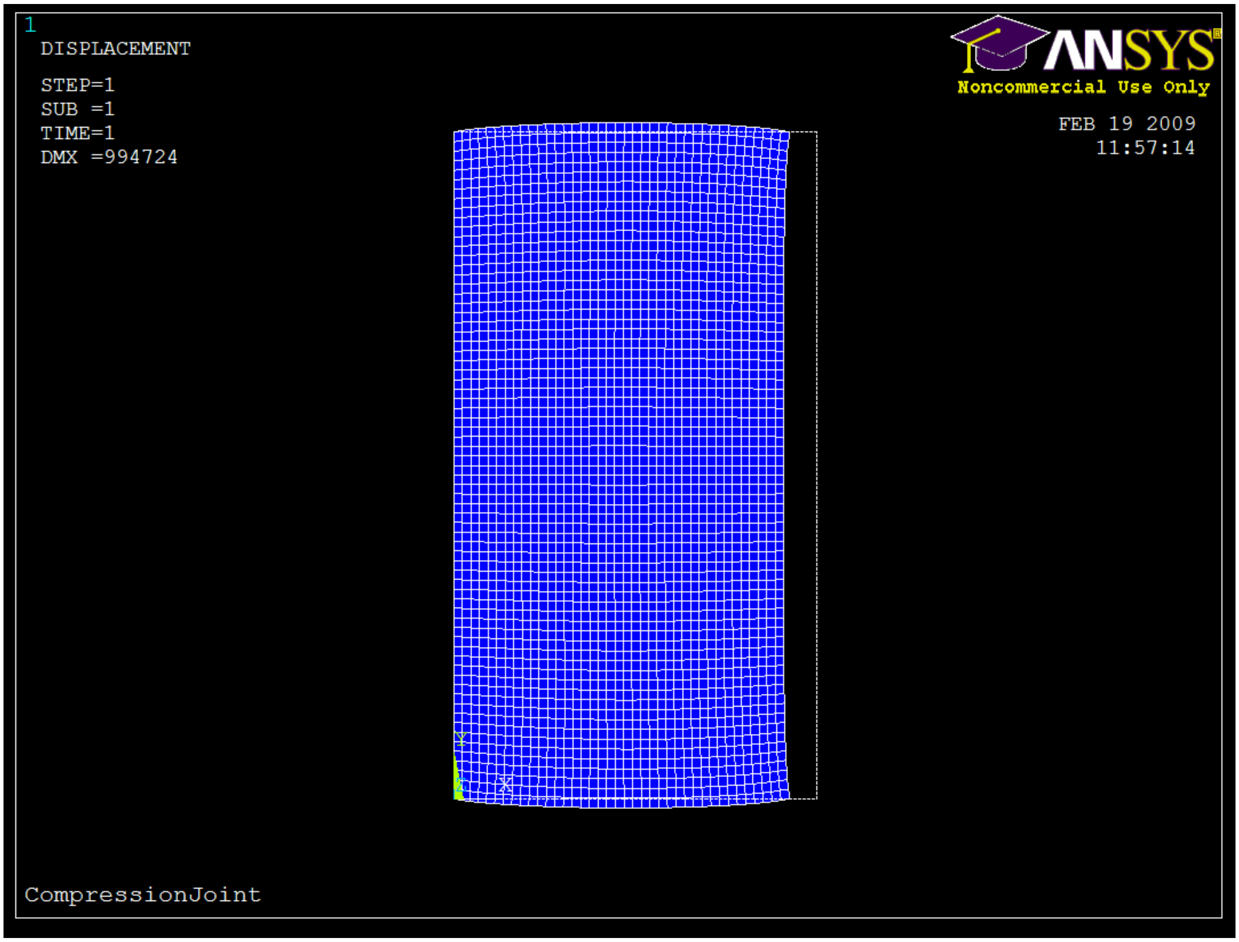


Figure 62: ANSYS Deformation from 9,500 Pa compression pressure

6.5 Summary

Based on the computer analysis and its comparison to the tensile testing data, ANSYS has shown to be a reliable model for understanding certain aspects of the compression joint's behavior. ANSYS may limit other loadings by the inherent assumptions placed on the model, such as what occurred in this model (not being able to extend past 4 mm). However, the results were still able to be compared with the initial data from the tensile testing laboratory.

Chapter 7 – Cost, Funding & Maintenance

To determine the economic factors related to the design of the bridge, basic cost estimates were completed to compare the steel and prestressed concrete designs. In addition to the cost estimate, future maintenance needs were identified as well as the funding for both the original construction and the future maintenance. This study of funding of future maintenance ties into the sustainability of the bridge in the sense of longevity of life through preservation.

7.1 Construction and Materials Cost Estimate

The costs of construction and materials were collected from *RSMeans Heavy Construction Cost Data* (Reed Construction Data, 2005). Below in Table 14 is the summary for the cost of the prestressed concrete girder bridge and in Table 15 is the cost summary for the steel girder bridge. The cost of the miscellaneous deck components were the same for both bridges but the girders, deck, reinforcement, and expansion joints varied.

Table 14: Prestressed Concrete Bridge Costs

Bridge Component	*Total Cost*
Concrete Girders	$225,244.03
Deck	$57,305.68
Miscellaneous Surface Components	$20,968.10
TOTAL COST OF PRESTRESSED CONCRETE BRIDGE	**$282,549.71**

Table 15: Steel Bridge Costs

Bridge Component	*Total Cost*
Steel Girders	$483,499.86
Deck	$58,290.58
Miscellaneous Surface Components	$20,968.10
TOTAL COST OF STEEL BRIDGE	**$541,790.44**

Notice also that the cost of the deck is very close but the cost of the girders determines which bridge is more cost effective. In this case the prestressed concrete girder bridge would cost less to construct. This is not necessarily true if the costs associated with constructing the rest of the bridge were also considered. The pier, abutments, and other substructure components would be very different because of the dead load of the concrete girders versus the load of the steel system.

7.2 Maintenance, Preservation, and Rehabilitation

Maintenance of a bridge can occur at two defined times, at regular scheduled intervals to ensure the most preservation and when failure or possible failure is eminent. Many times maintenance needs are determined by the bridge inspections performed by the government agency overseeing the bridge or a consultant that has been hired. All of the information gathered is put into some form of a Bridge Management System, which allows engineers to determine the needs of the bridges in their network.

7.2.1 Bridge Inspection

Bridge inspections are fundamental to the safety of the general public utilizing the structure. The purpose of the inspection is to find deficiencies and determine present conditions of the structure. These are then used in the Bridge Management System and analyzed to determine the appropriate action. The *Mass Highway Bridge Manual* specifically states "all structural components of a bridge must be accessible for a hands-on inspection" (*Bridge Manual*, 2005).

Conducting a bridge inspection includes the use of ladders, bucket trucks, rigging, platforms, walkways, scaffolding, and barges. Each of these aids has limitations in their use making a use of multiple modes of accessing the bridge typical, especially in the case of non-standard design. The inspection must also follow procedures depending on the type of failure that is critical or most likely to occur. These include fatigue and fracture of various structural components. The most important part of the bridge inspection is the documentation through detailed descriptions and photographs to gain a full understanding of the overall condition of the bridge (*Bridge Manual*, 2005).

Generally the bridge inspection includes a condition rating of the substructure, superstructure appurtenance, and site-related elements. Each of the following must be inspected and rated according to the established system of the agency:

- Joints
- Bearings
- Bridge Seats
- Pedestals
- Concrete Elements
- Steel Elements
- Timber Elements
- Embankment
- Deck
- Wearing Surface
- Primary Members
- Secondary Members
- Railings

- Drainage Systems
- Utilities
- Lighting and Signing
- Channel

This list is an overview of the components being rated to show the scope of what is performed and does not touch upon the methods of rating each element in detail. In addition the bridge inspection includes an overall site inspection to identify any environmental impacts, soil capacity, and various site conditions that could affect the integrity of the bridge. (Tonias, D., & Zhao, J., 2006) The bridge inspection is a large task, which requires many resources to complete. This inspection is a visual inspection used for bridges with maintenance needs or on a schedule. When bridges need rehabilitation additional testing is required and can include coring, delamination testing, testing for cover, measuring the steel thickness, and detection of fatigue cracking. Each of these tests includes special equipment specifically created to identify key conditions of the bridge to determine structural integrity. (*Bridge Inspection Unit*, n.d.)

In Massachusetts the Mass Highway Department has created a Bridge Inspection Unit to inspect the 2,900 Mass Highway owned and 1,500 municipally owned bridges. In addition several consulting contracts assist in the inspections. During these inspections laptops are used to input data immediately and to formulate reports which are then sent to the Bridge Management System. This allows the process to be paperless and easily accessed from all computers in the system.

7.2.2 Bridge Management Systems

Bridge Management Systems (BMS) hold all the information about a select grouping of bridges whether it is a state or regional area. This system is typically a computer database containing conditions data for each bridge in the network to help in the determination of maintenance and rehabilitation measures. This database also allows for the prioritization of projects based on the deteriorating conditions found in the inspection. Many agencies have been using these Bridge Management systems in one form or another not necessarily in a computer system but through an organization of the information known about the network, which they readily access for the needed information. Due to the nature and age of the infrastructure system it has been more commonly seen that the Bridge Management Systems have been computerized as the networks of bridges grow in number.

The BMS should not just be information provided by the condition assessment but also with programming to aid in the determination of maintenance, rehabilitation, and associated costs. Each bridge provides an individual set of components and conditions making it a challenge to compare all of the different variations seen in a network. The two major components of the BMS are an inventory database and a maintenance database. The inventory database presents the findings of the bridge inspections and overall inventory as bridges are constructed. The maintenance database is a record of all work done on the bridge components and their schedule of what is presently planned.

After both of these database components are generated the software analyzes these conditions to create an analysis of present conditions, predictions of future conditions, cost models, and optimization models. All of these analyses are to assist in the decisions an engineer must make after the absorption of the information presented. The present conditions are determined at both an individual level and a network level to determine the level of repairs, maintenance and rehabilitation to be considered by the engineers. Prediction of the future bridge conditions helps to analyze the different scenarios that can occur with a bridge: no maintenance is performed, partial or interim measures are implemented, or a full repair is completed to eliminate all deficiencies. Computer software takes the current conditions and applies deterioration as determined by the agency's knowledge base in the given region through historical experiences and inspection data. These models are not perfectly accurate because conditions continuously change and many factors are involved but they can be used to assist in an overall decision of which route is best at any given point in time. These analyses create models of cost and optimization. The cost model is important to many agencies as funding is a major issue and is discussed further in the next section. Cost models show the costs associated with the necessary work needed to be performed on the structure. The optimization model takes the cost model to the next level by adding a component of optimization of life-cycle costs over and indefinite period of time to show the best options for maintenance over time (Tonias, D., & Zhao, J., 2006). Figure 63 is a depiction of the overall process in which the BMS system can be used at all stages of a bridge life.

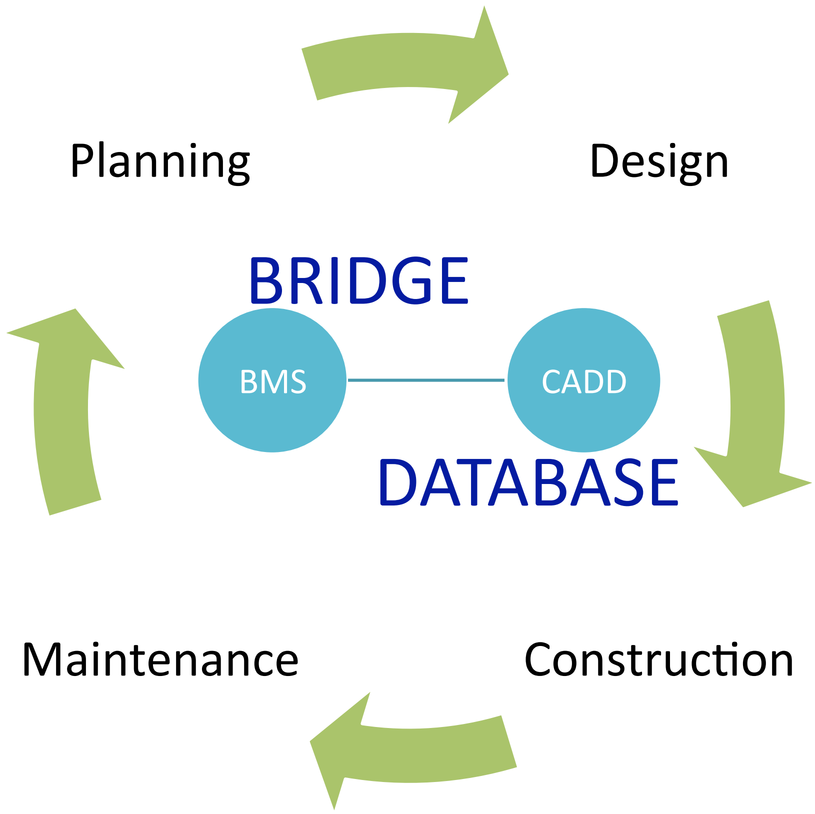


Figure 63: BMS Usage (adapted from *Bridge Engineering (*Tonias, D., & Zhao, J., 2006))

The ideal usage of a BMS would be to integrate it with other management systems relating to the infrastructure to ensure a full knowledge of conditions and potential needs of the system. (Tonias, D., & Zhao, J., 2006)

7.2.3 Types of Bridge Work

Once it is determined that a bridge has the need for work to be performed to ensure the condition of the bridge is safe, it is then categorized. These categories are based on the different types of work performed including repairs and maintenance, preservation, rehabilitation, and replacement. Bridge replacement is the current state of the bridge we are designing; this bridge design is a replacement design for the original bridge. This replacement occurs when the deterioration of the bridge cannot be solved through repairs, maintenance, preservation, or rehabilitation. This is also typically used when rehabilitation is more expensive due to the nature of the work needed. Once the bridge is constructed it will need repair and maintenance regularly to remain in a good working condition. Activities associated with this include washing, cleaning, painting, lubrication of bearings, sealing of joints, and wearing surface repairs. These are all performed when the bridge is still structurally sound. Once the bridge has a structural issue it moves into the category of preservation, which deals with minor structural deficiencies. This step is a key cost savings because problems are found before major design or engineering is needed. Beyond this level is the rehabilitation that typically includes major structural deficiencies and become very costly. Also included in this category are upgrades because of different usage, capacity, and bridge codes. (*About Bridge Projects*, n.d.) Once again this is a cyclical process illustrated in Figure 64.

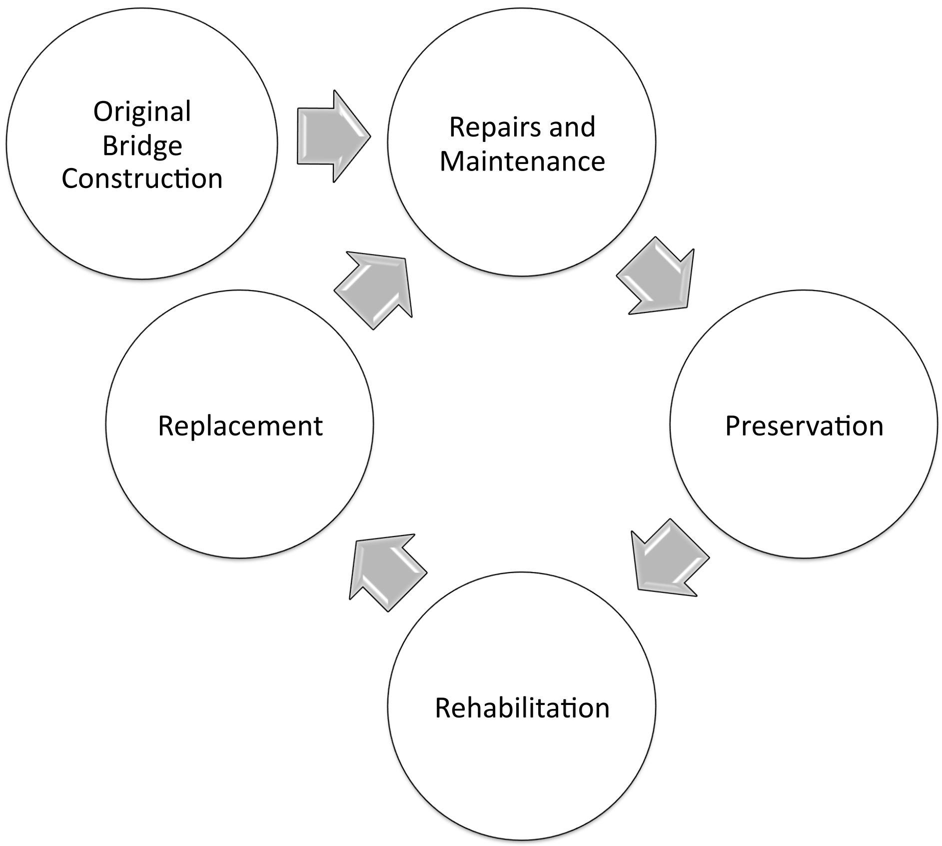


Figure 64: Bridge Work Cycle

7.3 Funding

The funding of transportation systems overall is poor and has many financing problems which are recognized by the Massachusetts Transportation Finance Commission's Report, *Transportation Finance in Massachusetts: An Unsustainable System*. The funding of projects is a difficult balance since many transportation systems are aging and need more work than is available in the budget. Today the Mass Highway Department prepares information and cost evaluation of the life-cycle of bridges being designed and constructed. Many projects are using short-term fixes that hide the larger problems since funding is not available. In particular Mass Highway has had budget cuts which "keep it from effectively carrying out its core mission of overseeing and maintaining the highway system" including bridges (*Transportation Finance in Massachusetts*, 2007). The Federal Highway Administration (FHWA) agreed with the commission stating that staffing was" well below the minimum needed to fulfill the necessary construction and materials testing functions…[and] there are a significant number of personnel who lack the necessary training and qualifications to perform inspection…" (*Transportation Finance in Massachusetts*, 2007). The funding for personnel is limited and begins to show the lack of funding available for projects. The state has constructed projects that will take 1.5 billion dollars to repay the deficit leading to a repetitive cycle of spending future resources before they have been collected by the state. In addition to Mass Highway overseeing bridges the Department of Conservation and Recreation (DCR) is responsible for nearly 200 bridges, which they also cannot afford to maintain and therefore transfer the responsibilities to the Massachusetts Highway Department (*Transportation Finance in Massachusetts*, 2007). Overall the funding system is failing due to the fact "MassHighway is underfunding upkeep and rehabilitation of its highways and bridges [and] the bridges and parkways of DCR are in severe neglect and facing immediate needs…" (*Transportation Finance in Massachusetts*, 2007). All of the needs and problems add up to a need for 15 to 19 billion dollars over the next two decades, money the state clearly does not have.

Overall the state and federal funding available needs to be better managed and distributed in a way where the most improvements can come out of the limited budget. This goes along with the optimization models created by BMS. The best places to put the money need to be identified and acted upon. Below in Table 16 is the distribution of funding from the FHWA in Massachusetts.

Table 16: FHWA Funding to Massachusetts (*Transportation Finance in Massachusetts*, 2005)

FHWA Obiligation Authority	Total Dollars (in Millions)	MHD Roads and Bridges Portion	Central Artery Project Portion
ISTEA (Average of $819.5 Million per Year)			
1992	733	252	481
1993	944	281	663
1994	1040	262	778
1995	756	204	551
1996	730	210	520
1997	714	209	505
TEA-21 (Average of $534 Million per Year)			
1998	579	177	402
1999	528	154	374
2000	481	154	327
2001	515	208	307
2002	562	261	301
2003	537	353	184
SAFETEA-LU (Average of $584 Million per Year)			
2004	591	408	184
2005	605	433	172
2006	633	503	131
2007 - Forecast	549	439	110
2008 - Forecast	559	442	117
2009 - Forecast	564	437	127

As can be seen, a large portion of the funding goes to the Big Dig, which has over the years taken away significant amounts of funding for the existing bridge and highway network. The gap as of 2007 for state controlled bridges was estimated to be 2.4 billion dollars but the overall budget dedicated to bridge repairs continues to increase because of the deteriorating conditions of the infrastructure (*MHD Information Page,* n.d.). An example of the budget increase is in 2004, the Massachusetts Highway Department expanded the basic bridge program due to the growing number of structurally deficient bridges by allotting 100 million more dollars annually (*Transportation Finance in Massachusetts*, 2007). Most of the funding for today comes from the Accelerated Bridge Program, which encompasses the statewide network and could potentially fund the bridge designed in this project.

The Accelerated Bridge Program was announced in May of 2008 by the Patrick-Murray Administration as a promise to repair the worse bridges in Massachusetts in a timely manner. Nearly 3 billion dollars were set aside to work on the 543 structurally deficient MassHighway and DCR bridges. This program overall hopes to address between 250 and 300 of the most rundown bridges. To date two hundred projects have been identified to complete construction over the next six years. Selection includes focus on bridges that need the repair, have weight restrictions, are closed because of structural problems, and bridges not expected to be repaired until after 2011. It is also expected that the recent passage of an economic stimulus package by the federal government will provide the state with money for various infrastructure work. Included in this bill is $25.7 billion for bridge repair, which will be disbursed in the upcoming months. (Berman, J., 2009) The funding for projects at this time is not always defined but the state is working to find funding for one of the most important parts of the infrastructure, the bridge network.

Chapter 8 – Conclusions and Recommendations

This project's purpose was to look at parts of a bridge design for comparison and then with more focus examined at bridge expansion joints. To begin such an endeavor a series of background investigations had to be performed to show why bridges are failing so frequently. The background also consisted of gaining knowledge of the design process and steps surrounding the design such as costs and funding. After gathering this literature review two designs were then formulated, and four types of bridge joints were designed and tested for comparison. Additionally computer analysis allowed for the examination of the accuracy of the tensile testing and comparison to the sealant specifications. The formulation of results created a comparison of design and methods used in bridge construction.

The results for the bridge design gave two different designs; however, several comparisons could be drawn. One example of a similarity, in the cross-section, is in the deck design where the reinforcement was nearly identical despite the two different types of girders. Because the girder haunch thickness, haunch width, and overall girder height were different, it was assumed that the need for negative and positive flexure reinforcement would be different for each girder depending on the location of the neutral axis. Additionally the non-structural components of the cross section are the same, which created the same, unfactored loading conditions. Through these similarities in the cross-section it could be determined that the only major design variation was in the girders. The project group would recommend examining different non-structural components (i.e. bike path loading) on the top of the bridge.

The laboratory testing encompassed the design and tensile testing of four specific joint types chosen for their common placement in bridges in the Northeast. These designs were modified to fit within the parameters of construction and testing in Kaven Hall. The results showed that the compression joint had the greatest stress capacity for a given strain. Also to be noted is the finger joints, which did not fail due to the limitation of the Instron testing machine. These joints performed well to the limiting strain with the potential to exceed the stress capacity of the compression joints. For the future study of joints the project group would recommend changing the definition of failure in tensile testing to be when the joint is no longer safe to drive on rather than the separation of the steel components. The failure definition of an "un-drivable" joint would need to be researched or identified by the author. The project group would additionally recommend changing variables (i.e. corrosion, temperature, saturation) on one type of joint for a more thorough study.

The computer analysis of a compression joint in tension was completed using ANSYS software. It was determined that the computer analysis results mimicked the laboratory testing results within the software restrictions. Additionally different loading configurations outside of the tensile laboratory testing could be performed with little additional cost. This method of analysis is a beneficial alternative to laboratory testing to reduce costs and waste of materials. The recommendations of the project team include investigating the software's capacities further to model other joints and perform dynamic testing similar to vehicles driving on the joint.

The cost, funding, and maintenance considerations are a significant part to any construction project, and in this report these considerations aided the overall recommendation for bridge design. The funding of bridges is complicated due to how funding is disbursed from the federal to the state government, and funding sources within the state itself (i.e. gas taxes, tolls, etc). This funding organization can result in significant deficits. Additionally, the total cost of a bridge is not only in its original construction but also in its life-cycle maintenance. The costs examined in detail were for the original construction of both bridge designs and totaled to the same approximate cost. The project group would recommend a more comprehensive design and cost analysis of additional bridge components (i.e. pier, abutments) to determine a more accurate final cost.

The recommendation for the final design would be a steel girder system, which was specified in the Massachusetts Highway Department Bridge Projects Website (Projects, n.d.). The main reason for the recommendation is the fact that the alternative investigated, the prestressed concrete girder bridge, was not more cost effective or easier to construct. This bridge would also utilize compression joints due to the testing results, with the finger joints as a viable candidate. The finger joints, although well performing, would require additional maintenance and construction costs; therefore the compression joint is initially recommended. Throughout the life cycle of the bridge it is recommended that a preventative maintenance plan be implemented. This is especially important because the sealant in a compression joint needs to be performing to minimum standards without corrosion.

In addition to the design and maintenance recommendations for the structure the project team recommends exploring a system similar to LEED design for buildings. This would entail the discussion of “green” components of the bridge, using recycled materials and environmentally friendly designs. The system could be used as a basis for the nation to use to become fully aware of environmental considerations for bridges. For the particularly designed bridge there is the possibility of adding a bicycling lane to the excessively wide bridge shoulder. This lane could be extended the four miles towards Worcester center and would encourage sustainable methods of commuting. Small considerations such as these could lead to a more “green” design in Worcester’s future.

Works Cited

AASHTO LRFD Bridge Design Specifications. (1998). Washington, D.C.: American Association of State Highway and Transportation Officials

About Bridge Projects. (n.d.). Retrieved February 26, 2009, from http://www.mhd.state.ma.us/acceleratedbridges/pr_aboutprojs.htm

Accelerated Bridge Program. (n.d.). Retrieved February 26, 2009, from http://www.mhd.state.ma.us/acceleratedbridges/program.htm

Barker, R., & Puckett, J. (1997*). Design of Highway Bridges: Based on AASHTO LRFD, Bridge Design Specifications.* New York: Wiley-Interscience.

Berman, J. (2009, February 17). *Transportation policy: Obama signs economic stimulus package into law*. Retrieved February 26, 2009, from www.logisticsmgmt.com/article/CA6638050.html

Bridge Deck Joint Performance. (2003). Washington D.C.: Transportation Research Board.

Bridge Inspection Practices. (2007). Washington D.C.: Transportation Research Board.

Bridge Inspection Unit. (n.d.). Retrieved February 26, 2009, from http://www.mhd.state.ma.us//default.asp?pgid=content/bridge_inspection&sid=about

Carper, K., &Feld, J. (1996).*Construction Failure, 2nd Edition*. New York: Wiley-Interscience.

Ebbert, Stephanie. 588 Mass. Bridges 'Deficient.' (2007, August 3). *Boston Globe*.

Elastomers | Silicone, Neoprene, EPDM | Rubber Elastomer | Dybrook Products, Inc.. (n.d.). Retrieved February 26, 2009, from http://www.dybrook.com/elastomers

Eleanor Schonell Bridge Project Overview. (n.d.). Retrieved January 28, 2009, from www.brisbane.qld.gov.au/BCC:BASE:732668508:pc=PC_1723

Heins, C. (1984). *Design of Modern Concrete Highway Bridges*. New York, NY: Wiley.

Heins, C. (1979). *Design of Modern Steel Highway Bridges*. Hoboken, New Jersey: John Wiley & Sons Inc.

LRFD Design Examples. (July 28, 2006). Retrieved September 25, 2008, fromhttp://www.fhwa.dot.gov/bridge/lrfd/examples.htm

Massachusetts Highway Department- Bridge Home. (n.d.). Retrieved Oct. 6, 2008, from http://www.mhd.state.ma.us//default.asp?pgid=BridgeIndex&sid=level2.

Massachusetts General Laws - Part 1 - Table of Contents. (n.d.). Retrieved Oct. 6, 2008, from http://www.mass.gov/legis/laws/mgl/gl-pt1-toc.htm.

Mass Highway (2005). *Massachusetts Highway Department Bridge Manual*.

MHD Information Page. (n.d.). Retrieved February 26, 2009, from http://www.eot.state.ma.us/default.asp?pgid=content/releases/pr051308_Bridge&sid=release

Nicodemus, Aaron. Span Dough. (2008, December 11). *Worcester Telegram and Gazette*.

Projects. (n.d.). Retrieved March 2, 2009, from http://www.mhd.state.ma.us/ProjectInfo/Main.asp?ACTION=ViewProject&PROJECT_NO=603516#

Reed Construction Data Inc. (2005). *RS Means Construction Cost Data*. 63rd Annual Addition. Kingston, MA.

Steel Recycling Institute. (n.d.). Retrieved March 2, 2009, from http://www.recycle-steel.org/

Tonias, D., & Zhao, J. (2006). *Bridge Engineering*. New York: McGraw-Hill Professional.

Transportation Finance in Massachusetts: An Unsustainable System. (2007, March 28). Retrieved February 26, 2009, from www.eot.state.ma.us/downloads/tfc/TFC_Findings.pdf

USGBC: U.S. Green Building Council. (n.d.). Retrieved January 28, 2008, from http://www.usgbc.org/

Wang, X., Swanson, J., Helmicki, A., Hunt, V. (2007).Development of Static-Response-Based Objective Functions for Finite-Element Modeling of Bridges. *Journal of Bridge Engineering*, 12(5), 544-551.

Appendix A – Proposal

LDA - 0903

Analysis of Bridge Connections Proposal

A Major Qualifying Project proposal to the faculty of Worcester Polytechnic Institute in partial fulfillment of the requirements for the Degree of Bachelor of Science

Submitted by:

Jennifer Himottu

Alyson Stuer

Submitted to:

Project Advisor - Prof. Leonard Albano

October 6th 2008

Abstract

This project will determine the best design in terms of strength, cost and sustainability for the reconstruction of Grafton Street bridge over Route 20 in Worcester, MA. To accomplish this, two different bridges will be designed—a concrete and a steel single-span girder bridge. Additionally, five different types of joints will be analyzed through computer software and laboratory testing. Resources used within this report will include AASHTO LRFD Bridge Design Specifications, RISA structural engineering software, and a series of bridge design and inspection manuals, in addition to several other sources. Conclusions and recommendations will be determined by research and testing to determine an appropriate solution for bridge redesign.

Table of Contents

List of Figures

List of Tables

1. Problem

As a society we have progressed greatly in the area of bridge design. In the 1870s, 25% of bridges failed—a rate of 40 bridges each year (Barker & Puckett, 1997). Today not nearly as many bridges fail; however, it is continues to exist and affect several communities. As technology advances and allows for the analysis of bridge stresses to be calculated easily and stress monitoring systems are developed, it is hoped that one day all failures can be prevented. One mode of failure is in the expansion joints which can be caused by movement, chemical degradation of joint, snow plowing, traffic, structural defections, poor design, and/or poor installation. This project includes researching and examining the failure of joints to discover why they occur as technology advances.

2. Objective

The purpose of this project is to examine two designs of highway overpasses and specifically look at modes of failure in the expansion joints. The specific failures to be studied in a laboratory component are the tension failures due to the roadway surface pulling on the joint and the compressive and shear failures due to loading by vehicles. To begin studying bridge joints, two basic highway overpasses will be designed to include a single span continuous plate girder bridge and a single span pre-stressed concrete girder bridge. Through this study we hope to develop theories and recommendations for a better performing design of the highway overpass and its connections.

3. Background

Bridges are a fundamental part of the transportation system in America. With the United States having a variety and a large number of bridges (approximately 590,000) they become a familiar part of transportation (Tonias & Zhao, 2006). These bridges allow for the interstate highways to be connected and allow for the travel of people, commodities, and necessities across our country. Bridges create limitations for the transportation system such as capacity and cost. They connect a system but with certain capacities based on width and number of lanes. The capacity of a bridge determines the capacity of the roads surrounding it, as all have to pass over the bridge to continue within the transportation system. Also, bridges are the most expensive part of a transportation system. The time, effort, and supplies for building a bridge costs multiple times what it costs to create a roadway (Barker & Puckett, 1997).

The most common, simplistic highway bridges seemingly go unnoticed by drivers because most all Americans are familiar with these bridges. These bridges, though somewhat unnoticed, are crucial to our transit system. The original need for uniform highway bridges resulted through the interstate system (Tonias & Zhao, 2006). This system of roads crossing the vast extents of the country needed to be connected over other roads, water, and differing environmental landscapes.

With an emerging uniform system of bridges, a national level of standards was needed (Tonias & Zhao, 2006).Presently, the AASHTO Standard Specifications for Highway Bridges are used as the basis for all standards in the United States. The state takes these specifications and details them to further fit the needs of the state based on climate and other differing factors. For Massachusetts-based standards, we will be using the Massachusetts Highway Department Bridge Manual. AASHTO LRDF Bridge Design Specifications are used in addition for design methodology. These various standards are vital for regulation and safety purposes.

3.1 Bridge Types

The most common bridges we see today are slab-on-stringer structures, and in particular the project will be studying the continuous plate girder bridge and the pre-stressed concrete girder bridge. The first bridge is a steel based bridge and is commonly recognized by the "green beams" one sees under many of the highway overpasses. The concrete girder bridge has similar components but is primarily concrete.

The steel plate girder bridge has multiple I cross-sections to support the concrete deck. These bridges are typically welded and can be used to reduce the amount of steel needed to support the bridge. This type of steel bridge typically allows for longer spans to be designed. The pre-stressed concrete girder bridge has six basic AASHTO geometry types that are also all I-shaped. This concrete is opposite to the steel because it is strongest under compressive loading and weak in tension; therefore, it is usually reinforced with steel to offset the opposing weaknesses. Both of these designs of bridges have the goal of using the least amount of material while maintaining strength (Tonias & Zhao, 2006).

3.2 Inspection

Many of these common highway overpasses were originally constructed during the 1950's through the 1970's. This presents the problem that many of these bridges are aging quickly and may become structurally deficient—needing maintenance or replacing. Because of the uniformity of highway overpass bridges, the rehabilitation process includes many of the same maintenance needs. As these bridges age at rates in which the government cannot control, many fall into disrepair and may ultimately create failures.

When a bridge fails or is taken out of service for maintenance or reconstruction the transportation systems has an additional limitation. Detours are typically lengthy because groups of bridges are not found whether it be over rivers, highways, or other landscapes. With road closures and traffic problems arising from bridge closures, it is important to keep them maintained to avoid failure altogether. This requires an extensive method of bridge inspection to keep records on conditions and necessary maintenance. These inspections occur at different levels of detail and at varying frequencies. The inspector will look at the various components of the bridge and rate them accordingly. This includes the joint elements, the focus of our analysis.

3.3 Joints

As written in *Bridge Engineering*, "Joint elements are particularly critical because they: prevent leakage of runoff and deicing chemicals from rusting and corroding substructure elements below the deck, provide a smooth transition from approach to bridge deck, [and] allow for longitudinal movement of the structure" (Tonias& Zhao, 2006). The joint helps to protect what is below and has to endure through conditions other parts of the substructure do not face. This makes the joints susceptible to various methods of failures. As a critical part of the bridge it is important to study various connection types and examine how various conditions affect the structural integrity of the joint. In particular we will be studying the strip seal joint, compression joint, finger plate joint, and sliding plate joint.

The joints discussed are some of the most common joints in highway bridges today. These include the compression seal joint, the strip seal joint, the sliding plate joint, and the finger plate joint. The compression and strip seal joints have a dependency primarily on the elastomeric material. The other two joints, the sliding plate joint and the finger plate joint, are primarily steel dependent.

Each of the joints described have a series of problems that need to be analyzed to see the effect on the bridge itself. These joints can lead to additional stresses or alleviate some stress due to expansion and compression from the season and temperature changes. These stresses can lead to a variety of overall structural problems within the bridge. By studying the effects these joints have on the bridge structure, they can help determine possible remedies and solutions to common bridge problems.

4. Methodology

In order to complete this project, it has been divided into three major components. Below, in Table 1, the three component activities are listed with the available resources.

Table 1: Scope Activities and Resources

Activities	Resources
Design	AISC handbook and additional design standards
Computer Analysis	Software, Departmentprofessors
Laboratory Testing	Materials, Don Pellegrino, CE Structural lab

4.1 Schedule

These components will form the Major Qualifying Project and will take place over an eight-month time period. In September, 2008 the project will be formulated, the proposal will be written, and the research and design components will begin. After the initial research and design is completed, a computer analysis component will be observed and applied to the proposed design. When the computer analysis is substantially complete, a laboratory component will begin to confirm the previously obtained analysis data. During the eight-month period, the report will be drafted; and finally, in April the project will be presented to the Worcester Polytechnic Institute Civil Engineering Department. A summary of this schedule is shown in Table 2 and a process flowchart is shown in Figure 1.

Table 2: Project Timeline

TASK		Month							
		Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr
Research of bridge failure, design & connections									
Highway bridge design									
Risa 2D	Risa 2D Familiarity								
	Joint Analysis								
Laboratory Analysis	Material procurement								
	Joint design								
	Joint construction								
	Testing								
Report composition									
Project presentation									

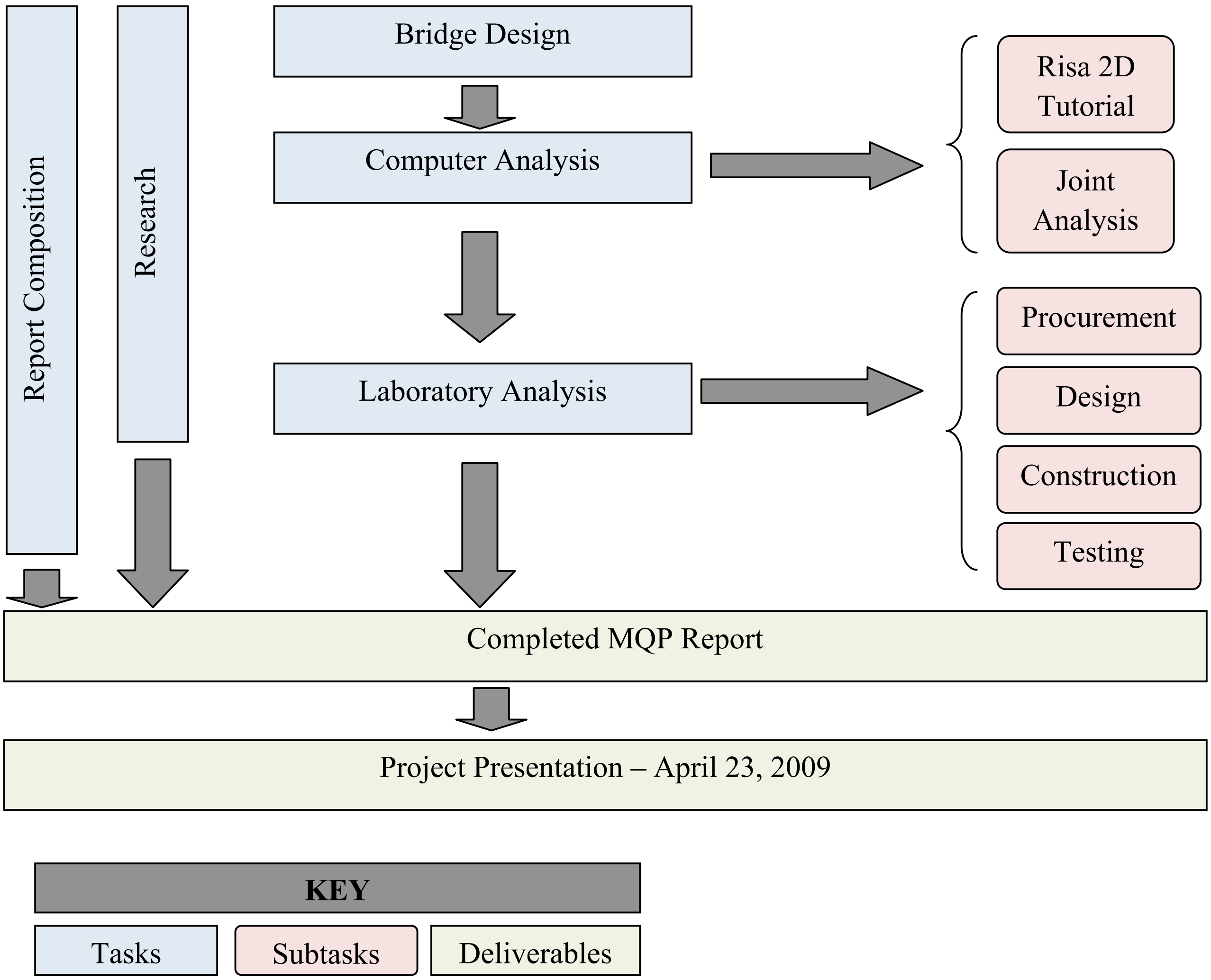


Figure 1: Scope Flowchart

4.2 Bridge Design

In order to study the connections in depth a bridge design must be established to use as a basis for analysis. To allow for realistic design parameters local projects were looked at to determine if any fit with the project. A local bridge replacement fits to the type of design desired. This bridge is in Worcester, W-44-063, at the crossing of Route 122 (Grafton Street) over Route 20. Based on the specifications given on the Massachusetts Highway Department website a design will be established for the bridges to be designed; a single span continuous plate girder bridge and a single span pre-stressed concrete girder bridge as simplified in Table 3.

Table 3: Bridge Design

	Single Span	Dual Span (Time Permitting)
Concrete Girder	X	
Steel Girder	X	

4.3 Connection Analysis

Once a bridge is designed it must be tested thoroughly to ensure its success. To test the connections we will input the specific joint designs into computer models to test after the bridge design has been completed. In addition, a laboratory test will be performed on the joint designs for comparison of data from the computer simulation. The analysis of the connections will provide information of what inspectors should be looking for when inspecting bridges in addition to possible design changes. Beginning with a computer analysis will allow for many different variables to be tested in a limited time frame, and by adding laboratory testing, data is available to show where the computer is lacking information such as inelastic behaviors. CAD renderings of the different types of joints to be constructed and how they will be loaded are shown below in Figures 2-5.

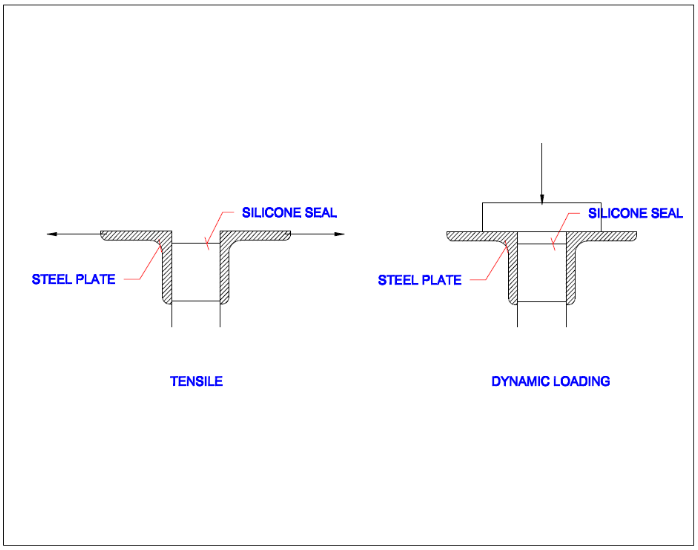


Figure 2: Compression Seal Joint

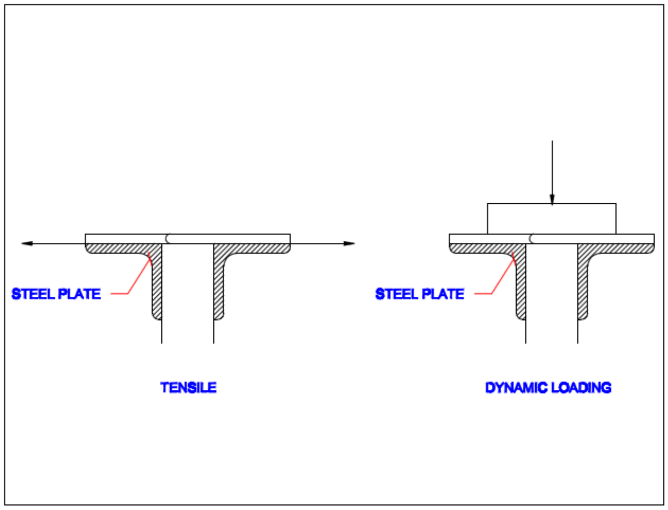


Figure 3: Finger Plate Joint

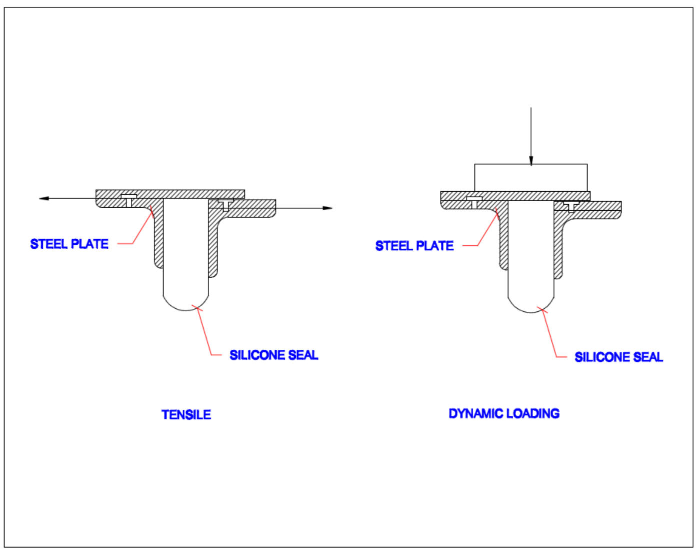


Figure 4: Sliding Plate Joint

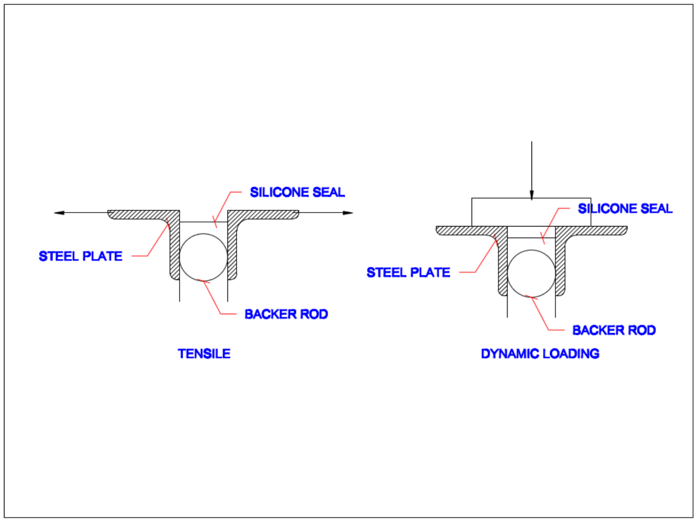


Figure 5: Strip Seal Joint

4.4 Computer Analysis

Using the available programs on campus—Risa 2D and the AutoCAD package of programs—an analysis will be completed on the various connections being studied. These analyses will include information about stresses, shear, and loading. It is intended that we will explore various rates of corrosion based on salt and chemical levels seen in the area. To model and determine the various stresses, shears, and loads, a replica of each bridge design will be modeled in the software program. Through this model various loading situations can be applied and the effects can be seen. In addition to analyzing the bridge as a whole, it is intended that we take the individual joints and model them as well. These joints will then have loads applied as they see in their lifespan. As analysis of the joints in “new” condition occurs, an analysis of joints with less cross-sectional area will be performed to replicate corrosion. Assumptions will be made that corrosion is similar to the loss of cross-sectional area because corrosion can be difficult to replicate. The results from these computer analyses will provide an understanding of where the bridge and connections are most likely to fail and present other possible problems that occur due to select loading situations. A summary of the described computer analysis methods is shown below in Table 4.

Table 4: Computer Analysis Methods

	Joint Alone	Concrete Girder Bridge	Steel Girder Bridge
Compression Seal	X	X	X
Strip Seal	X	X	X
Finger Plate	X	X	X
Sliding Plate	X	X	X

4.5 Laboratory Analysis

The purpose of the laboratory testing is to observe phenomena and compare data to the information provided through the computer analysis. This testing shall show us what the computer is missing in its analysis such as possible inelastic behavior. For the laboratory testing, each of the joints will be built in duplicate to perform two tests. The laboratory testing will include tensile testing because the road puts tensile loads on the connection itself. Also included will be the dynamic loading to replicate the traffic over a given connection.

In order to complete the laboratory analysis, Mr. Don Pellegrino and Mr. Dean Daigneault, will be assisting with procurement of materials, building and machining joints, and laboratory procedures. Through the use of the materials lab all testing shall be able to be completed relatively easily once the joints have been physically created. The laboratory analysis is summarized below in Table 5.

Table 5: Laboratory Analysis

	Joint Design	Joint Construction	Tensile Loading	Repeated Dynamic Loading
Compression Seal	X	X	X	X
Strip Seal	X	X	X	X
Modular	X	X	X	X
Finger Plate	X	X	X	X
Sliding Plate	X	X	X	X

5. Capstone Design

As part of the Major Qualifying Project we will be focusing on capstone design ranging from economic, environmental, sustainability, constructability, ethical, and heath and safety concerns. These considerations will allow for a full understanding of an engineering project from all aspects.

5.1 Economic Considerations

There is not an unlimited budget for projects including bridge replacement and engineers must look to the cost effectiveness of their design. Not only should an engineer look at the cost of the original building of a bridge but also the potential maintenance that the bridge will need over its life. As part of the project, the determination of which joint type and bridge type is the most cost effective for use in Massachusetts. Depending on geography different assemblies can required varied maintenance. In New England the snow and sanding require additional maintenance to parts such as the joints since this can cause corrosion. This will be taken into account for determining cost effectiveness and maintenance costs that can be predicted. Also from the information provided on the bid website an estimated construction cost is available to make comparisons.

5.2 Environmental Considerations

Environmental considerations are examined so that the structure can easily co-exist within the environment that it is placed. Applying the environmental issue to this project, we must consider the impact that the designed bridge will have on the environment and how the design will affect the landscape. Presenting one consideration—being a part of a New England roadway, the bridge could be exposed to salt in the winter to decrease roadway icing. However, salt contaminates the surrounding land and will also flow through the drainage system, contaminating water run-off. Because the air flow is underneath as well as above a bridge, bridges typically freeze before the remaining roadway, creating a prime target for salting. Ultimately, the community must decide how they will decrease ice accidents, whether through salt distribution, increased signage, or both.

A second consideration is how the structure will fit into the surrounding environment both aesthetically and with an environmental consideration. For example, if a bridge and roadway are being constructed along a severe topography, drainage becomes an area of concern. By building a roadway along a hill, the water will flow faster over a smooth surface and can create flooding without proper drainage.

5.3 Sustainability Considerations

When communities plan the replacement of a bridge they also need to think about future budgets for the maintenance of the system. This can be included as part of the overall town budget or a separate account can be set up in advance for the purpose of having a designated amount of money available to perform maintenance. When maintenance is not looked at in the economic and sustainability considerations it is not planned for and then can lead to issues of not being able to maintain the bridge. If maintenance is not performed the life cycle of a bridge can shorten considerably causing more sustainability problems. Also a reduction in necessary maintenance can be designed into a bridge structure. By finding the most appropriate materials and methods, maintenance can be reduced due to the versatility of the bridge. We may find one type of bridge or connection can withstand more of the conditions than others before it need maintenance. This can become a very important part of the design and thinking about the issue of sustainability.

In addition to maintenance, sustainability considerations include thinking about how to avoid corrosion. Corrosion is a concern that makes a bridge need more maintenance and is typically a problem that can be alleviated by appropriate drainage design. Drainage problems and methods will be studied to determine an appropriate drainage system for reducing joint corrosion.

5.4 Constructability Considerations

Constructability is the most important aspect of a design project. Bridge design includes a product of construction; therefore the constructability of the element must be considered. An engineer must determine how the design will be constructed—what materials will be used and how they will fit together with connections. In this project, we must additionally consider how the connections will be constructed for laboratory testing. In testing considerations it is important to be representative of the actual large-scale design; however, it is equally important to do this cost effectively while still gaining an accurate representation of the as-built structure.

5.5 Ethical, Health and Safety Considerations

As an engineer there is a responsibility to the public for the design to be to current standards. This is part of the ethical obligation to designing at any level. Each situation and project has a unique set of parameters and an engineer must ensure that the design is appropriate for the given use to provide safety to any users of the facility. By making a conscious ethical design a priority the engineer is allowing for the health and safety considerations to be met. For the given project standards from AASHTO and the Massachusetts Highway Department will be utilized.

6. Conclusion

During the MQP it is expected that deliverables be produced at varying intervals throughout to show progress. These will include everything listed in Table 6. These deliverables will be in addition to the MQP paper which will use these materials to support information presented.

Table 6: Deliverables

Deliverable:	Two Bridge Designs	Computer Simulation Data & Analysis	Lab Data and Analysis
Presentation of Deliverable through:	• CAD Drawings • Hand Calculations	• Design Printouts • Analysis Printouts • Summary Data • Analysis of Data	• As-Designed Drawings of joints being tested • As-Built Drawings of joints tested • Data Printouts & Summaries from Testing • Analysis of Data

Works Cited

Barker, R., & Puckett, J. (1997*). Design of Highway Bridges: Based on AASHTO LRFD, Bridge Design Specifications.* New York: Wiley-Interscience.

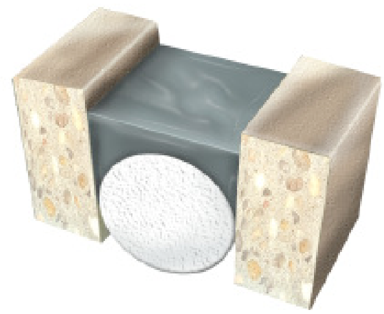
Tonias, D., & Zhao, J. (2006). *Bridge Engineering*. New York: McGraw-Hill Professional.

Appendix B – Wabo Silicone Seal Properties

Wabo®SiliconeSeal

Two Part Silicone Joint Seal
(Licensed for use under US Patent No. 5.190.395)

Features	Benefits
• Unique Design	Designed to provide a watertight seal and minimize debris, the seal will easily bond to itself. No priming of interfaces is required, which simplifies and accelerates the installation process
• Rapid Installation	A two-part cold applied, self-leveling, ambient cure sealant assuring ease of installation and providing minimal downtime in the field.
• Versatility	Suitable for concrete, steel, or elastomeric concrete header applications, while allowing multidirectional movements of the structure.



DESCRIPTION:

Wabo®SiliconeSeal is a dynamic two-part sealant designed for horizontal expansion joints on bridges, parking decks, and open air structures. The cold applied, self-leveling, low modulus sealant is ideally suited for the new construction or repair of existing expansion joints. When properly mixed, the sealant cures rapidly to form a well bonded elastomeric seal capable of accommodating movements + 100% /- 50% of the joint opening. Wabo®SiliconeSeal does not require the use of any primers.

RECOMMENDED FOR:

- Sealing horizontal joints on bridges and parking garages
- Expansion joint applications with a maximum movement range of +100% / -50% of the joint opening.
- Structures where minimum construction closure time is a factor.
- New construction or repair and maintenance of existing joints.

PACKAGING/COVERAGE:

- Wabo®SiliconeSeal is a 1:1 mix and available in:
  - Standard 50.72 oz dual cartridge kit (Part A – 25.36oz; Part B – 25.36oz)
  - 10 gal unit (Part A – 5 gal; Part B – 5 gal)
- Yield will depend on joint design, tooling, backer rod placement, waste, and experience.

JOINT WIDTH	YIELD/KIT (10 Gal Unit)	YIELD/UNIT (50.72 oz Cartridge)
1" (25mm)	317 LF (96.6m)	12.6 LF (3.8m)
1.5" (38mm)	194 LF (59.1m)	7.8 LF (2.4m)
2" (50mm)	135 LF (41.1m)	5.3 LF (1.6m)
2.5" (64mm)	100 LF (30.5m)	3.9 LF (1.2m)
3" (76mm)	78 LF (23.8m)	3.0 LF (0.9m)

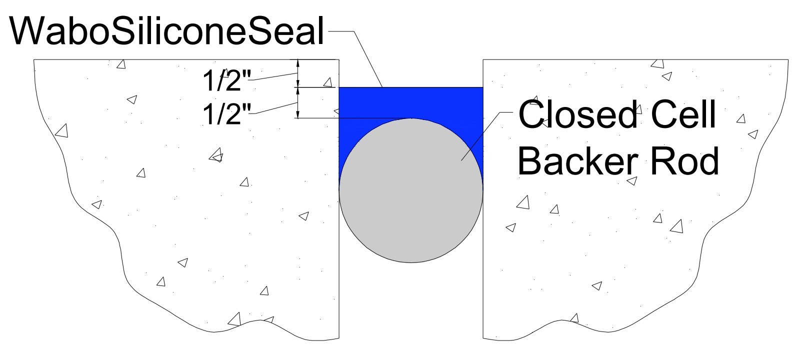
The Chemical Company

TECHNICAL DATA:

Design Information

Wabo®SiliconeSeal is capable of accommodating movement +100%/-50% of the joint opening at the time of installation. Backer rod should be sized 25% greater to ensure proper joint configuration of the sealant and positioned 1-inch from top of riding surface. Regardless of opening, the sealant shall be 1/2" thick, not to exceed 5/8" in the center.

Wabo®SiliconeSeal is not recommended for joint openings that will exceed 3.25" during the movement cycle.



Physical Properties

PHYSICAL PROPERTIES	ASTM TEST METHOD	PART A	PART B
Color		White	Gray
Viscosity		88,000 cps	34,000 cps
Leveling	C 639	self levels	self levels
Extrusion rate ml/min.	C 1183	200-600	200-600

PHYSICAL PROPERTIES	ASTM TEST METHOD	REQUIREMENTS
Leveling	C 639	self levels
Tack free time	C 679	60 minute max.
Joint elongation	D 5329 (1) (2)	600% min.
Joint Modulus, 100%	D 5329 (1) (2)	15 psi (.10MPa) max
Cure evaluation	D 5893	Pass @ 4 hrs, max
Ultimate Elongation	D 412 Die C (1)	1000% min.
Stress @ 150%	D 412 Die C (1)	25 psi max. (.17 Mpa)
Shore Hardness, 00	C 661 (1)	40 - 80
Specific Gravity	D 792 (1)	1.20 - 1.40

(1) Specimens cured at 77 +/-3 F and 50 +/-5% R.H. For 7 days
(2) Specimens size is 1/2"wide x 1/2" deep x 2" long.

APPLICATION:

Installation Summary:

- **Newly placed concrete**: joint interface must be dry and clean (free of dirt, coatings, rust, grease, oil, and other contaminants), sound, and durable. New concrete must be cured (14 day minimum).
- **Aged concrete**: loose, contaminated, weak, spalled, deteriorated and/or delaminated concrete must be removed to sound concrete and repaired prior to placement.
- **Steel**: steel substrates should be sound, steel surfaces must be abrasive blasted SP-10 near white metal, immediately prior to installation.
- The joint opening must be abrasive blasted to remove all latencies and contaminants which may cause bonding problems. The joint opening should be blown clean using compressed air (>90psi).
- A non-gassing closed-cell expanded polyethylene foam rod, approximately 25% larger in diameter than the joint gap is positioned in the joint opening such that the top of the rod is 1" (25mm) below the riding surface.
- The Wabo®SiliconeSeal system shall be applied in one direction only to a thickness of 1/2" (13mm) minimum, while not exceeding 5/8" (16mm) and maintaining a 1/2" (13mm) recess from the riding surface.
- Clean all excess material from the edges of the joint opening as soon as possible. DO NOT allow the silicone to cure before removing it.
- Wabo®SiliconeSeal will begin to cure and form a surface skin within 20 minutes of application. The seal will be ready to accept traffic generally within one hour after installation.

For Best Results:

- Repair any spalls, voids, or structural cracking at the joint interface.
- Do NOT use for joint gap openings that exceed 3.25".
- Do NOT allow any of the components to freeze prior to installation. Store all components out of direct sunlight in a clean, dry location between 50˚F (10˚C) and 90˚F (32˚C). Do not store in high humidity.
- Shelf life of chemical components is approximately 12 months.
- Do NOT install when surface temperature are less than 40˚F (4˚C)
- Periodically inspect the applied material and repair localized areas as needed. Consult a Watson Bowman Acme representative for additional information.
- Make certain the most current version of the product data sheet is being used. Please consult the website (www.wbacorp.com) or contact a customer service representative.

Options/Equipment:

- Dual component pump (for 10 gal kit). Contact WBA for recommended manufacture.
- Well Made AG400 Pneumatic Caulking Gun (for 50.72 oz cartridges) provided by WBA.
- VAS #750.720 DL Mixing Nozzle. Note: material should be mixed using 1/2" dia. 18 element static mixer or greater.
- Closed cell polyethylene backer rod 25% greater than joint opening.

Related Documents:

- Material Safety Data Sheets
- Wabo®SiliconeSeal Specification
- Wabo®SiliconeSeal Sales Drawings
- Wabo®SiliconeSeal Installation Procedure
- WaboCrete SiliconeSeal Joint System Datasheet

LIMITED WARRANTY:

Contact

Watson Bowman Acme Corp. 95 Pineview Drive, Amherst, NY 14228

phone:716-691-7566 / fax: 716-691-9239 / web site: http://www.wbacorp.com

INSTALLATION PROCEDURE

Wabo®SiliconeSeal Joint System
Silicone Expansion Joint Sealant
for Bridge & Highway Applications

A. General

The work shall consist of furnishing and installing a Wabo®SiliconeSeal joint system in accordance with the details shown on the plans and the requirements of the specifications. Placement of the Wabo®SiliconeSeal joint system shall consist of proper surface preparations, material and application of materials.

B. Joint Preparation

The concrete joint interface must be clean (free of dirt, coatings, rust, grease, oil and other contaminants), sound and durable. New concrete must be cured (minimum of 14 days) and all latinance removed.

Durable Concrete- sound and durable concrete should have a cap pull-off strength that meets or exceeds ACI 503R, Appendix A.

Unsound Concrete – Loose, contaminated, weak, spalled, deteriorated and/or delaminated concrete must be removed to sound concrete and repaired prior to placement of Wabo®SiliconeSeal. Any patching materials must be approved by manufacturer prior to use.

Joint interfaces should be sandblasted to remove any residue that may be present. It is recommended that the sandblasting operation be performed in two steps, one step per joint interface.

The joint should be blown cleaned using compressed air (>90 psi). The compressed air shall be free of moisture and oil. The joint interfaces should be checked for cleanliness. Should any contaminates remain, then the joint must be re-blasted and blown clean. To insure cleanliness, each joint interface should be wiped clean with a clean rag without solvents to remove any dust remaining after sandblasting.

C. Backer Rod Placement

A closed-cell, expanded polyethylene foam rod is recommended. The rod acts as a bond breaker, preventing the Wabo®SiliconeSeal from bonding to the bottom of the joint and preventing the flow of the material through the joint itself. The size of the backer rod should be 25% greater than the joint opening to be sealed. The backer rod should be positioned in the joint such that the top of the rod is one 1" (25 mm) below the riding surface.

INSTALLATION PROCEDURE

Wabo®SiliconeSeal Joint System
Silicone Expansion Joint Sealant
for Bridge & Highway Applications

D. Mixing of Wabo®SiliconeSeal

The Wabo®SiliconeSeal should be applied using proper dispensing equipment such as Johnston or Graco/Pyles extrusion pump for 10-gallon kits or dual component pneumatic powered double piston guns for 25.36-oz cartridges.

Regardless of equipment used, the material shall be mixed using ½ dia. 18 element static mixer or greater.

Caution: Do not allow material to remain static in mixer longer than five (5) minutes without changing to a new static mixer. Uncured sealant can be removed with the use of solvents such as naphtha or mineral spirits

E. Wabo®SiliconeSeal Placement

Application of the Wabo®SiliconeSeal should be in one direction only and from the bottom of the joint up. The Wabo®SiliconeSeal shall be applied to a thickness of ½' (13mm) minimum while not exceeding 5/8" (16 mm) and maintaining a ½" (13 mm) recess from the riding surface.

Wabo®SilconeSeal will begin to cure and form a surface skin within 20 minutes of application. The seal will be ready to accept traffic generally within one hour after installation

INSTALLATION PROCEDURE

Wabo®SiliconeSeal Joint System
Silicone Expansion Joint Sealant
for Bridge & Highway Applications

A. General

The work shall consist of furnishing and installing a Wabo®SiliconeSeal joint system in accordance with the details shown on the plans and the requirements of the specifications. Placement of the Wabo®SiliconeSeal joint system shall consist of proper surface preparations, material and application of materials.

B. Joint Preparation

The concrete joint interface must be clean (free of dirt, coatings, rust, grease, oil and other contaminants), sound and durable. New concrete must be cured (minimum of 14 days) and all latinance removed.

Durable Concrete- sound and durable concrete should have a cap pull-off strength that meets or exceeds ACI 503R, Appendix A.

Unsound Concrete – Loose, contaminated, weak, spalled, deteriorated and/or delaminated concrete must be removed to sound concrete and repaired prior to placement of Wabo®SiliconeSeal. Any patching materials must be approved by manufacturer prior to use.

Joint interfaces should be sandblasted to remove any residue that may be present. It is recommended that the sandblasting operation be performed in two steps, one step per joint interface.

The joint should be blown cleaned using compressed air (>90 psi). The compressed air shall be free of moisture and oil. The joint interfaces should be checked for cleanliness. Should any contaminates remain, then the joint must be re-blasted and blown clean. To insure cleanliness, each joint interface should be wiped clean with a clean rag without solvents to remove any dust remaining after sandblasting.

C. Backer Rod Placement

A closed-cell, expanded polyethylene foam rod is recommended. The rod acts as a bond breaker, preventing the Wabo®SiliconeSeal from bonding to the bottom of the joint and preventing the flow of the material through the joint itself. The size of the backer rod should be 25% greater than the joint opening to be sealed. The backer rod should be positioned in the joint such that the top of the rod is one 1” (25 mm) below the riding surface.

Wabo®SiliconeSeal Joint System
Silicone Expansion Joint Sealant
for Bridge & Highway Applications

D. Mixing of Wabo®SiliconeSeal

The Wabo®SiliconeSeal should be applied using proper dispensing equipment such as Johnston or Graco/Pyles extrusion pump for 10-gallon kits or dual component pneumatic powered double piston guns for 25.36-oz cartridges.

Regardless of equipment used, the material shall be mixed using ½ dia. 18 element static mixer or greater.

Caution: Do not allow material to remain static in mixer longer than five (5) minutes without changing to a new static mixer. Uncured sealant can be removed with the use of solvents such as naphtha or mineral spirits

E. Wabo®SiliconeSeal Placement

Application of the Wabo®SiliconeSeal should be in one direction only and from the bottom of the joint up. The Wabo®SiliconeSeal shall be applied to a thickness of ½' (13mm) minimum while not exceeding 5/8" (16 mm) and maintaining a ½" (13 mm) recess from the riding surface.

Wabo®SilconeSeal will begin to cure and form a surface skin within 20 minutes of application. The seal will be ready to accept traffic generally within one hour after installation

Material Safety Data Sheet

WABO® SILICONE SEALANT, PART B

Version 1.2 06/30/2006

1. PRODUCT AND COMPANY INFORMATION

Company	:	**Watson Bowman Acme Corporation** 95 Pineview Drive Amherst, NY 14228
Telephone	:	716-691-7566
Emergency telephone number	:	(800) 424-9300 (703) 527-3887 (Outside Continental US)
Product name	:	WABO® SILICONE SEALANT, PART B
MSDS ID No.	:	10521
TSCA Inventory	:	All components of this product are included, or are exempt from inclusion, in the EPA Toxic Substances Control Act (TSCA) Chemical Substance Inventory.
Canadian DSL	:	This product contains material not included on the Canadian Domestic Substance List (DSL).
Product Use Description	:	Sealant

2. HAZARDOUS INGREDIENTS

Chemical	CAS No.	TLV	STEL	PEL	CEIL	Weight %
POLY(DIMETHYLSILOXANE)	63148-62-9		N.E.	N.E.	N.E.	30.00 - 60.00 %

3. HAZARDS IDENTIFICATION

	HEALTH	FLAMMABILITY	PHYSICAL HAZARD
HMIS® Rating	1	1	0

WHMIS Class	:	D2B
Primary Routes of Entry	:	Eye contact Skin contact Ingestion

Effects of Overexposure

Inhalation	:	Can cause slight irritation.
Skin	:	Can cause slight irritation.
Eyes	:	Can cause slight irritation.
Ingestion	:	Can cause slight irritation.
Chronic exposure	:	No known information available.

Carcinogenicity

	ACGIH	IARC	NTP	OSHA
POLY(DIMETHYLSILOXANE)	N.E.	N.E.	N.E.	N.E.

4. FIRST AID MEASURES

Eye contact	:	Flush eyes with water, lifting upper and lower lids occasionally for 15 minutes. Seek medical attention.
Skin contact	:	Remove contaminated clothing. Wash thoroughly with soap and water. If irritation persists seek medical attention. Wash contaminated clothing before reuse.
Ingestion	:	Do not induce vomiting without medical advice. If conscious, drink plenty of water. If a person feels unwell or symptoms of skin irritation appear, consult a physician. If a person vomits, place him/her in the recovery position. Never give anything by mouth to an unconscious person.
Inhalation	:	Remove victim from exposure. If difficulty with breathing, administer oxygen. If breathing has stopped administer artificial respiration, preferably mouth-to-mouth. Seek immediate medical attention.

5. FIRE-FIGHTING MEASURES

Flash point	:	> 200.01 °F (93.34 °C)
Autoignition temperature	:	no data available
Lower explosion limit	:	no data available
Upper explosion limit	:	no data available
Suitable extinguishing media	:	water fog foam water spray carbon dioxide (CO_2) dry chemical
Fire and Explosion Hazards	:	Containers can build up pressure if exposed to heat (fire). Cool closed containers exposed to fire with water spray.
Special Fire-fighting Procedures	:	As in any fire, wear pressure demand self-contained breathing apparatus (NIOSH approved or equivalent) and full protective gear.

6. ACCIDENTAL RELEASE MEASURES

Methods for cleaning up	:	Wear appropriate protective equipment (refer to section 8). Take action to eliminate source of leak; prevent from entry into open streams or sewers; contain spill by diking; vacuum up liquid or use absorbent media; remove to storage for disposal and rinse residual stain with water.

7. HANDLING AND STORAGE

Handling	:	Keep out of reach of children. For personal protection see section 8.
Storage	:	Keep tightly closed.

8. EXPOSURE CONTROLS / PERSONAL PROTECTION

Eye protection	:	Wear as appropriate: safety glasses with side-shields goggles face-shield
Hand protection	:	Wear as appropriate: impervious gloves
Body Protection	:	Wear as appropriate: impervious clothing preventive skin protection
Respiratory protection	:	In case of insufficient ventilation wear suitable respiratory equipment. When workers are facing concentrations above the exposure limit they must use NIOSH approved respirators.
Hygienic Practices	:	Avoid contact with skin, eyes and clothing. Ensure adequate ventilation, especially in confined areas. Wash hands before breaks and at the end of workday. When using, do not eat, drink or smoke. Handle in accordance with good industrial hygiene and safety practice.
Engineering Controls	:	Local exhaust ventilation can be necessary to control any air contaminants to within their TLVs during the use of this product.

9. PHYSICAL AND CHEMICAL PROPERTIES

Color	:	gray
Physical State	:	paste
Odor	:	none
pH	:	not applicable
Odor Threshold	:	no data available
Vapor Pressure	:	no data available
Vapor Density	:	no data available
Boiling point/range	:	>302 °F (150 °C)
Freeze Point	:	no data available
Water solubility	:	insoluble
Specific Gravity	:	1.45
Viscosity	:	no data available
Evaporation rate	:	Slower than Butyl acetate
Partition coefficient (n-octanol/water)	:	no data available
VOC Concentration as applied (less water and exempt	:	< 41 g/l Note: VOC concentration expressed as applied when all components

solvents)	are mixed and applied per manufacturer's instructions.

10. STABILITY AND REACTIVITY

Stability	:	Stable under recommended storage conditions.
Conditions to avoid	:	Prolonged exposure to high temperatures
Materials to avoid	:	strong acids
Hazardous decomposition products	:	Oxides of carbon
Hazardous polymerization	:	Will not occur under normal conditions.

11. TOXICOLOGICAL INFORMATION

Acute inhalation toxicity

	Type	Value	Species	Exposure time
Product	LC50	no data available		
Component				
POLY(DIMETHYLSILOXANE)	LC50	no data available		

Acute oral toxicity

	Type	Value	Species
Product	LD50 (Oral)	no data available	
Component			
POLY(DIMETHYLSILOXANE)	LD50 (Oral)	no data available	

Acute dermal toxicity

	Type	Value	Species
Product	LD50 (Dermal)	no data available	
Component			
POLY(DIMETHYLSILOXANE)	LD50 (Dermal)	no data available	

12. ECOLOGICAL INFORMATION

Ecotoxicological Information	:	There is no data available for this product.

13. DISPOSAL CONSIDERATIONS

Recommendations: Use excess product in an alternate beneficial application. Handle disposal of waste material in manner which complies with local, state, province and federal regulation.

14. TRANSPORT INFORMATION

DOT	:	Proper shipping name	Not regulated
IATA	:	Proper shipping name	Not regulated

15. REGULATORY INFORMATION

SARA 311/312 (RTK)
This product has been reviewed according to the EPA 'Hazard Categories' promulgated under Sections 311 and 312 of the Superfund Amendments and Reauthorization Act of 1986 (SARA Title III) and is considered, under applicable definitions, to meet the following categories:

not applicable

SARA 313
This product contains the following substances subject to the reporting requirements of Section 313 of Title III of the Superfund Amendments and Reauthorization Act of 1986 and 40 CFR Part 372:

Weight %	CAS No.	Chemical Name

This product contains no chemicals subject to the SARA 313 supplier notification requirements.

CERCLA
CERCLA section 103(a) specifically requires the person in charge of a vessel or facility to report immediately to the National Response Center (NRC) a release of a hazardous substance whose amount equals or exceeds the assigned RQ. The following hazardous substances are contained in this product.

RQ	CAS No.	Chemical Name

No CERCLA chemicals exist in this product above reportable concentrations.

TSCA Section 12(b) Export Notification
This product contains the following chemical substances subject to the reporting requirements of TSCA 12(b) if exported from the United States:

CAS No.	Chemical Name

There are no TSCA 12(b) Chemicals in this product.

California Proposition 65
The chemical(s) noted below and contained in this product, are known to the state of California to cause cancer, birth defects or other reproductive harm. Unless otherwise specified in Section 2 of this MSDS, these chemicals are present at < 0.1%:

CAS No.	Chemical Name
1333-86-4	CARBON BLACK

16. OTHER INFORMATION

WABO® SILICONE SEALANT, PART B

Legend	:	N.E. - Not Established TLV - Threshold Limit Value STEL - Short Term Exposure Limit PEL - Permissible Exposure Limit CEIL - Ceiling
Prepared By	:	Environment, Health and Safety Department

End of MSDS.

Material Safety Data Sheet

WABO® SILICONE SEALANT, PART A

Version 1.2 06/30/2006

1. PRODUCT AND COMPANY INFORMATION

Company	:	**Watson Bowman Acme Corporation** 95 Pineview Drive Amherst, NY 14228
Telephone	:	716-691-7566
Emergency telephone number	:	(800) 424-9300 (703) 527-3887 (Outside Continental US)
Product name	:	WABO® SILICONE SEALANT, PART A
MSDS ID No.	:	10520
TSCA Inventory	:	All components of this product are included, or are exempt from inclusion, in the EPA Toxic Substances Control Act (TSCA) Chemical Substance Inventory.
Canadian DSL	:	This product contains material not included on the Canadian Domestic Substance List (DSL).
Product Use Description	:	Sealant

2. HAZARDOUS INGREDIENTS

Chemical	CAS No.	TLV	STEL	PEL	CEIL	Weight %
POLY(DIMETHYLSILOXANE), HYDROXY TERMINATED	70131-67-8		N.E.	N.E.	N.E.	30.00 - 60.00 %
POLY(DIMETHYLSILOXANE)	63148-62-9	N.E.	N.E.	N.E.	N.E.	10.00 - 30.00 %
OXIMINO SILANES	Proprietary	N.E.	N.E.	N.E.	N.E.	5.00 - 10.00 %
TOLUENE	108-88-3	50 ppm	150 ppm	N.E.	300 ppm	1.00 - 5.00 %

3. HAZARDS IDENTIFICATION

	HEALTH	FLAMMABILITY	PHYSICAL HAZARD
HMIS® Rating	2	1	1

WHMIS Class	:	D2B
Primary Routes of Entry	:	Ingestion Inhalation Eye contact Skin contact Skin absorbtion

Effects of Overexposure

Inhalation	:	Inhalation of high vapor concentrations may cause symptoms like headache, dizziness, tiredness, nausea and vomiting. Inhalation of high vapor concentrations can cause CNS-depression and narcosis. Prolonged inhalation can be harmful.
Skin	:	Prolonged skin contact may defat the skin and produce dermatitis. Prolonged or repeated exposure can cause skin irritation and redness. Repeated or prolonged skin contact may cause allergic reactions with susceptible persons. May cause sensitization

by skin contact. Components of the product may be absorbed into the body through the skin. When this product is exposed to moisture, Methyl Ethyl Ketoxime may be formed. Methyl Ethyl Ketoxime may be absorbed through the skin reducing the blood's ability to transport oxygen (methemoglobinemia and anemia).

Eyes : Can cause moderate to severe irritation, tearing and blurred vision. Prolonged exposure can result in more sever irritation and possible corneal injury.

Ingestion : Intake can cause gastrointestinal irritation, nausea, vomiting, diarrhea, headache, and drowsiness. Can cause moderate to severe irritation.

Chronic exposure : Existing respiratory or skin ailments may be aggravated by exposure. This product contains solvents. Reports associate repeated and prolonged occupational overexposure to solvents with permanent brain and nervous system damage. Reports also indicate that solvents cause liver damage, kidney damage, and mucous membrane irritation. Be warned that intentional misuse by deliberately inhaling the vapors and/or the product contents (a process often called "sniffing") can be harmful or fatal.

Carcinogenicity

	ACGIH	IARC	NTP	OSHA
POLY(DIMETHYLSILOXANE), HYDROXY TERMINATED	N.E.	N.E.	N.E.	N.E.
POLY(DIMETHYLSILOXANE)	N.E.	N.E.	N.E.	N.E.
OXIMINO SILANES	N.E.	N.E.	N.E.	N.E.
TOLUENE	Not classifiable as a human carcinogen.	Inadequate data.	N.E.	N.E.

4. FIRST AID MEASURES

Eye contact : Flush eyes with water, lifting upper and lower lids occasionally for 15 minutes. Seek medical attention.

Skin contact : Remove contaminated clothing. Wash thoroughly with soap and water. If irritation persists seek medical attention. Wash contaminated clothing before reuse.

Ingestion : Do not induce vomiting without medical advice. If conscious, drink plenty of water. If a person feels unwell or symptoms of skin irritation appear, consult a physician. If a person vomits, place him/her in the recovery position. Never give anything by mouth to an unconscious person.

Inhalation : Remove victim from exposure. If difficulty with breathing, administer oxygen. If breathing has stopped administer artificial respiration, preferably mouth-to-mouth. Seek immediate medical attention.

5. FIRE-FIGHTING MEASURES

Flash point : > 200.01 °F (93.34 °C)

Autoignition temperature : no data available

Lower explosion limit : no data available

Upper explosion limit : no data available

Suitable extinguishing media : dry chemical
carbon dioxide (CO_2)
water fog
foam

Fire and Explosion Hazards	:	Containers can build up pressure if exposed to heat (fire). Cool closed containers exposed to fire with water spray. Solid stream of water or foam can cause frothing. Fire may produce irritating or poisonous fumes.
Special Fire-fighting Procedures	:	As in any fire, wear pressure demand self-contained breathing apparatus (NIOSH approved or equivalent) and full protective gear.

6. ACCIDENTAL RELEASE MEASURES

Methods for cleaning up	:	Wear appropriate protective equipment (refer to section 8). Take action to eliminate source of leak; prevent from entry into open streams or sewers; contain spill by diking; vacuum up liquid or use absorbent media; remove to storage for disposal and rinse residual stain with water.

7. HANDLING AND STORAGE

Handling	:	Use only in area provided with appropriate ventilation. Keep out of reach of children. For personal protection see section 8.
Storage	:	Keep tightly closed in a dry and cool place.

8. EXPOSURE CONTROLS / PERSONAL PROTECTION

Eye protection	:	Wear as appropriate: safety glasses with side-shields goggles face-shield
Hand protection	:	Wear as appropriate: impervious gloves
Body Protection	:	Wear as appropriate: impervious clothing preventive skin protection
Respiratory protection	:	In case of insufficient ventilation wear suitable respiratory equipment. When workers are facing concentrations above the exposure limit they must use NIOSH approved respirators.
Hygienic Practices	:	Avoid contact with skin, eyes and clothing. Ensure adequate ventilation, especially in confined areas. Wash hands before breaks and at the end of workday. When using, do not eat, drink or smoke. Handle in accordance with good industrial hygiene and safety practice.
Engineering Controls	:	Local exhaust ventilation can be necessary to control any air contaminants to within their TLVs during the use of this product.

9. PHYSICAL AND CHEMICAL PROPERTIES

Color	:	white
Physical State	:	liquid

Odor	:	slight aromatic
pH	:	not applicable
Odor Threshold	:	no data available
Vapor Pressure	:	no data available
Vapor Density	:	no data available
Boiling point/range	:	>230 °F (110 °C)
Freeze Point	:	no data available
Water solubility	:	insoluble
Specific Gravity	:	1.08
Viscosity	:	no data available
Evaporation rate	:	no data available
Partition coefficient (n-octanol/water)	:	no data available
VOC Concentration as applied (less water and exempt solvents)	:	< 41 g/l Note: VOC concentration expressed as applied when all components are mixed and applied per manufacturer's instructions.

10. STABILITY AND REACTIVITY

Stability	:	Stable under recommended storage conditions.
Conditions to avoid	:	Prolonged exposure to high temperatures
Materials to avoid	:	Water acids oxidizing agents metals
Hazardous decomposition products	:	Oxides of carbon nitrogen oxides (NOx) Reaction with water will release Methyl Ethyl Ketoxime.
Hazardous polymerization	:	May occur. Avoid exposure to water, strong acids, and heat treatment, especially in the presence of iron.

11. TOXICOLOGICAL INFORMATION

Acute inhalation toxicity

	Type	Value	Species	Exposure time
Product	LC50	no data available		

Component

POLY(DIMETHYLSILOXANE), HYDROXY TERMINATED	LC50	no data available
POLY(DIMETHYLSILOXANE)	LC50	no data available
OXIMINO SILANES	LC50	no data available
TOLUENE	LC50	no data available

Acute oral toxicity

	Type	Value	Species
Product	LD50 (Oral)	no data available	
Component			
POLY(DIMETHYLSILOXANE), HYDROXY TERMINATED	LD50 (Oral)	no data available	
POLY(DIMETHYLSILOXANE)	LD50 (Oral)	no data available	
OXIMINO SILANES	LD50 (Oral)	no data available	
TOLUENE	LD50 (Oral)	636 mg/kg	

Acute dermal toxicity

	Type	Value	Species
Product	LD50 (Dermal)	no data available	
Component			
POLY(DIMETHYLSILOXANE), HYDROXY TERMINATED	LD50 (Dermal)	no data available	
POLY(DIMETHYLSILOXANE)	LD50 (Dermal)	no data available	
OXIMINO SILANES	LD50 (Dermal)	no data available	
TOLUENE	LD50 (Dermal)	20 mg/kg	

12. ECOLOGICAL INFORMATION

Ecotoxicological Information : There is no data available for this product.

13. DISPOSAL CONSIDERATIONS

Recommendations: Use excess product in an alternate beneficial application. Handle disposal of waste material in manner which complies with local, state, province and federal regulation.

14. TRANSPORT INFORMATION

DOT	:	Proper shipping name	Not regulated
IATA	:	Proper shipping name	Not regulated

15. REGULATORY INFORMATION

SARA 311/312 (RTK)
This product has been reviewed according to the EPA 'Hazard Categories' promulgated under Sections 311 and 312 of the Superfund Amendments and Reauthorization Act of 1986 (SARA Title III) and is considered, under applicable definitions, to meet the following categories:

IMMEDIATE (ACUTE) HEALTH HAZARD

SARA 313
This product contains the following substances subject to the reporting requirements of Section 313 of Title III of the Superfund Amendments and Reauthorization Act of 1986 and 40 CFR Part 372:

Weight %	CAS No.	Chemical Name
1.00 - 5.00 %	108-88-3	TOLUENE

CERCLA
CERCLA section 103(a) specifically requires the person in charge of a vessel or facility to report immediately to the National Response Center (NRC) a release of a hazardous substance whose amount equals or exceeds the assigned RQ. The following hazardous substances are contained in this product.

RQ	CAS No.	Chemical Name
1,000 lbs	108-88-3	TOLUENE

TSCA Section 12(b) Export Notification
This product contains the following chemical substances subject to the reporting requirements of TSCA 12(b) if exported from the United States:

CAS No.	Chemical Name

There are no TSCA 12(b) Chemicals in this product.

California Proposition 65
The chemical(s) noted below and contained in this product, are known to the state of California to cause cancer, birth defects or other reproductive harm. Unless otherwise specified in Section 2 of this MSDS, these chemicals are present at < 0.1%:

CAS No.	Chemical Name
108-88-3	TOLUENE

16. OTHER INFORMATION

Legend : N.E. - Not Established
TLV - Threshold Limit Value
STEL - Short Term Exposure Limit
PEL - Permissible Exposure Limit
CEIL - Ceiling

Prepared By : Environment, Health and Safety Department

This information is furnished without warranty, representation, or license of any kind, except that this information is accurate to the best of the manufacturer's knowledge, or is obtained from sources believed by the manufacturer to be accurate and is not intended to be all inclusive. No warranty is expressed or implied regarding the accuracy of this information or the results to be obtained from its use thereof. The manufacturer assumes no responsibility for injuries proximately caused by use of the Material if reasonable safety procedures are not followed as stipulated in this Data Sheet. Additionally, the manufacturer assumes no responsibility for injuries proximately caused by abnormal use of the Material even if reasonable safety procedures are followed. Buyer assumes the risk in its use of the Material.

End of MSDS.

Appendix C – Lab Testing Results

C1

Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 12/15/08

Name: Jen

Number of specimens: 1

Temperature:

Humidity:

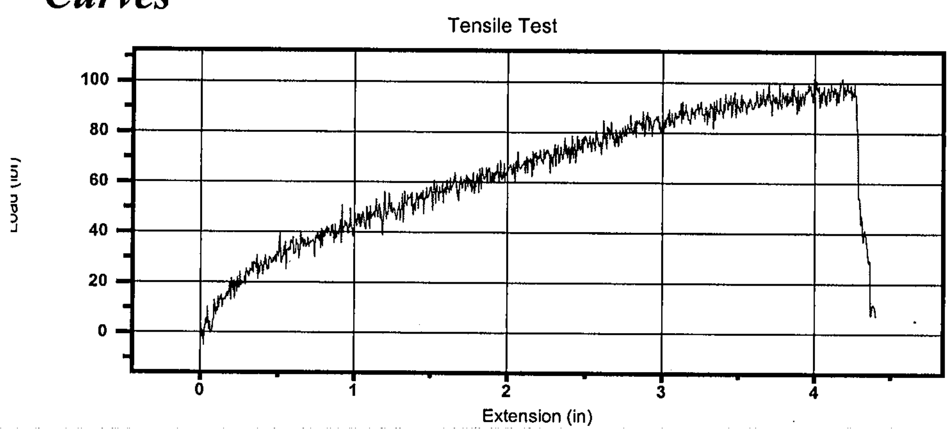
Note 1:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	101.78	4.18
Mean	101.78	4.18
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	101.78	4.18
Maximum	101.78	4.18
Range	0.00	0.00

Curves



C2

Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 12/16/08

Note 1:

Name: C2

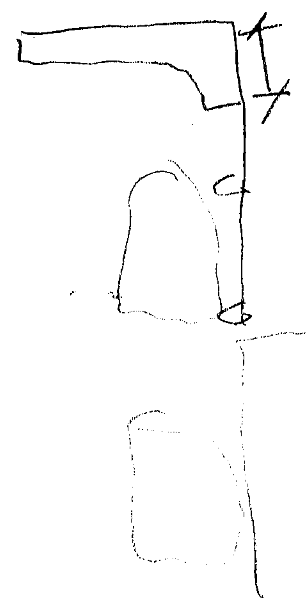
Number of specimens: 1

Temperature:

Humidity:

Speed 1: 0.30 in/min

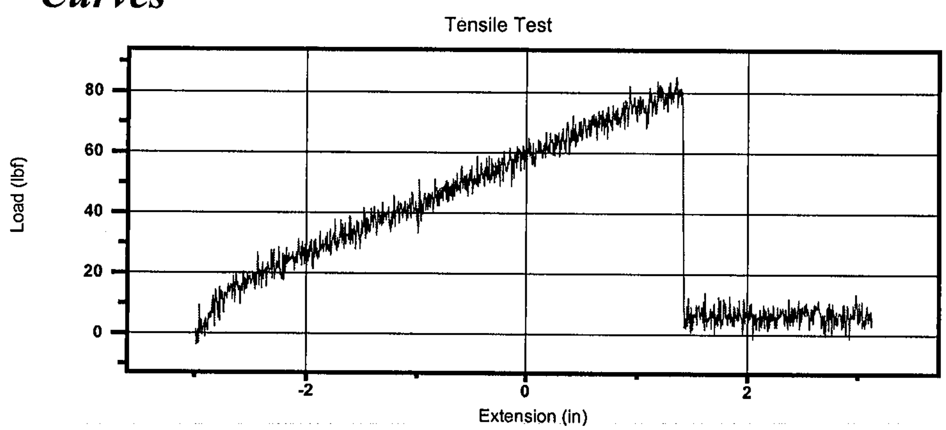
1' - 8 1/2"



Results

	Maximum Load (lbf)	Extension (in)
1	85.14	1.35
Mean	85.14	1.35
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	85.14	1.35
Maximum	85.14	1.35
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/6/09

Name: C-3

Number of specimens: 1

Temperature:

Humidity:

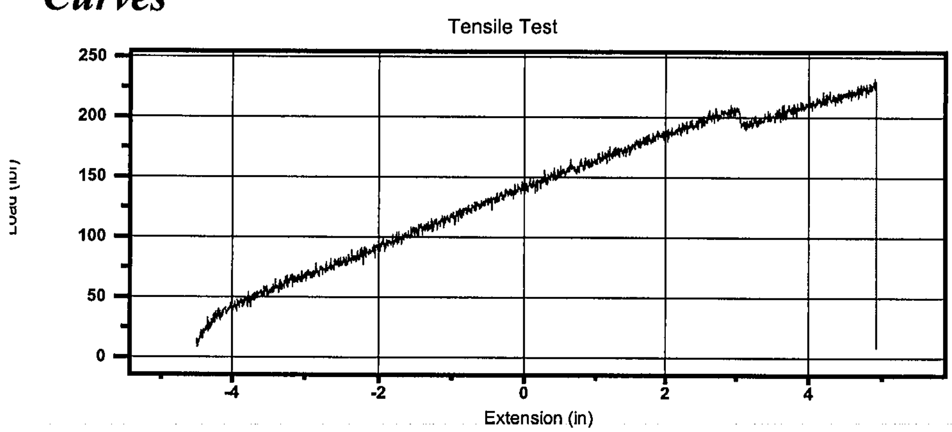
Note 1:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	232.11	4.91
Mean	232.11	4.91
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	232.11	4.91
Maximum	232.11	4.91
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/7/09

Note 1:

Name: C-4-09

Number of specimens: 1

Temperature:

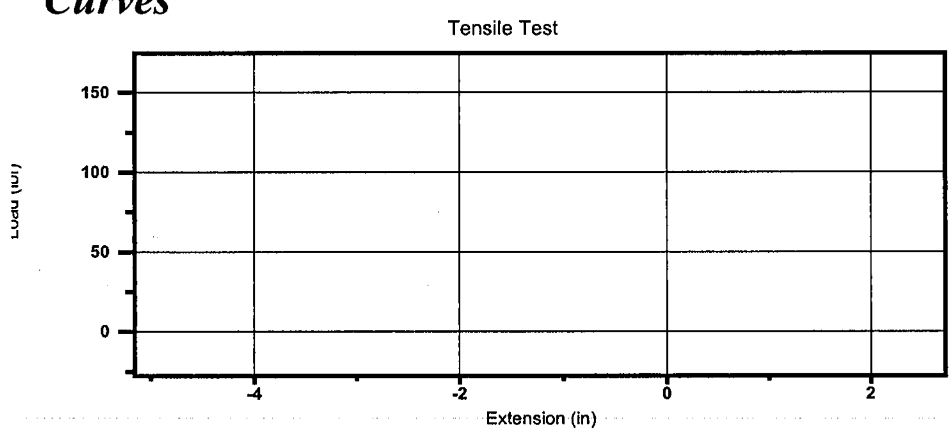
Humidity:

Speed 1: 0.30 in/min

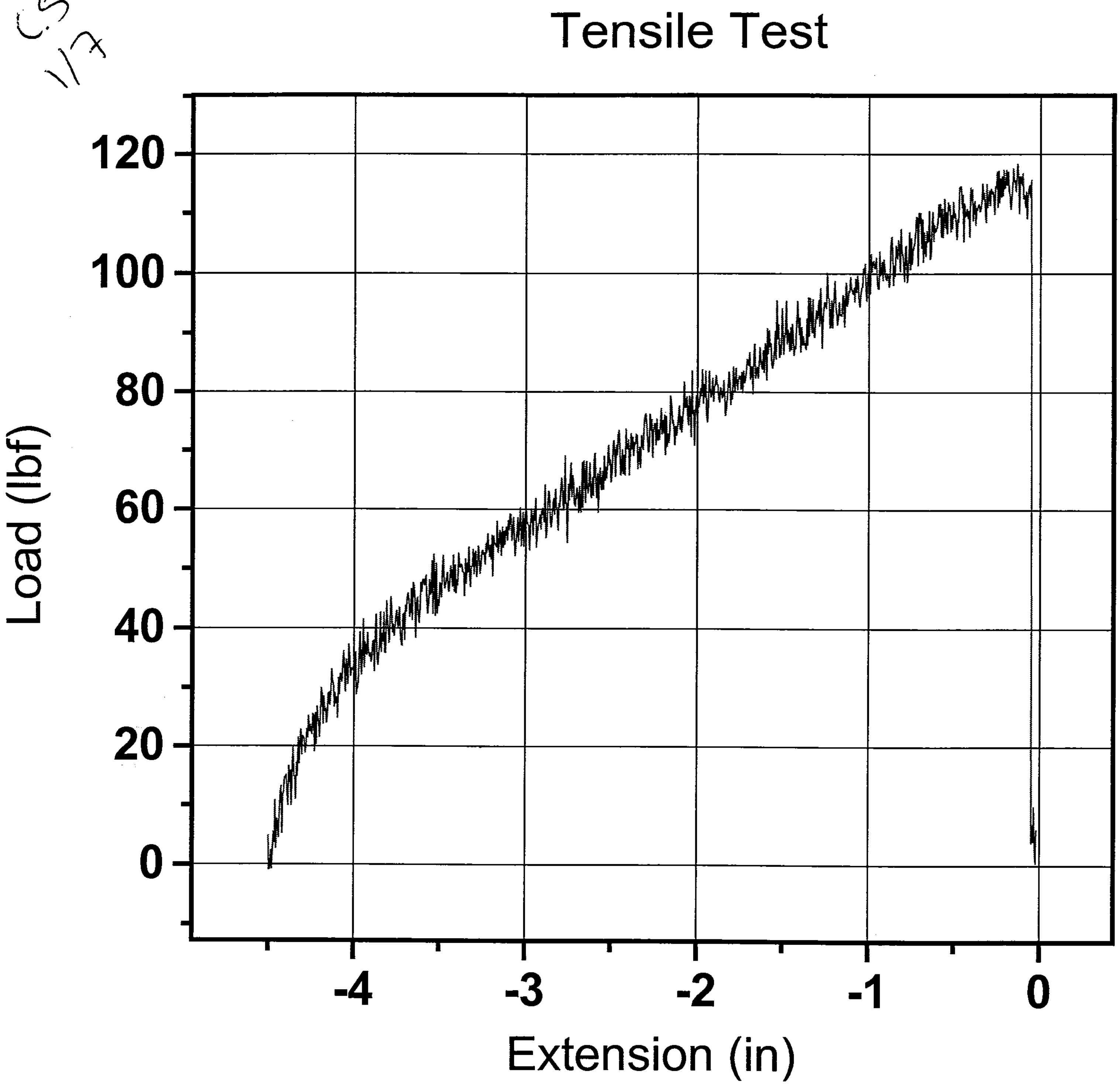
Results

	Maximum Load (lbf)	Extension (in)
1	157.71	2.02
Mean	157.71	2.02
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	157.71	2.02
Maximum	157.71	2.02
Range	0.00	0.00

Curves



C.S
1/7



1/7/05

	Maximum Load (lbf)	Extension (in)
1	-	-
2	118.28	-0.13
Mean	118.28	-0.13
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	118.28	-0.13
Maximum	118.28	-0.13
Range	0.00	0.00

Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/27/09

Note 1:

Name: C-6

Number of specimens: 1

Temperature:

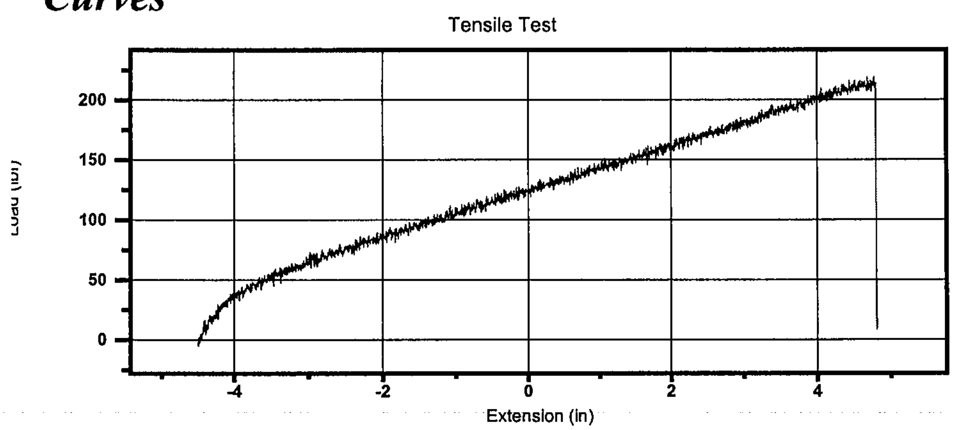
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	219.22	4.77
Mean	219.22	4.77
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	219.22	4.77
Maximum	219.22	4.77
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/6/09

Name: SS-1

Number of specimens: 1

Temperature:

Humidity:

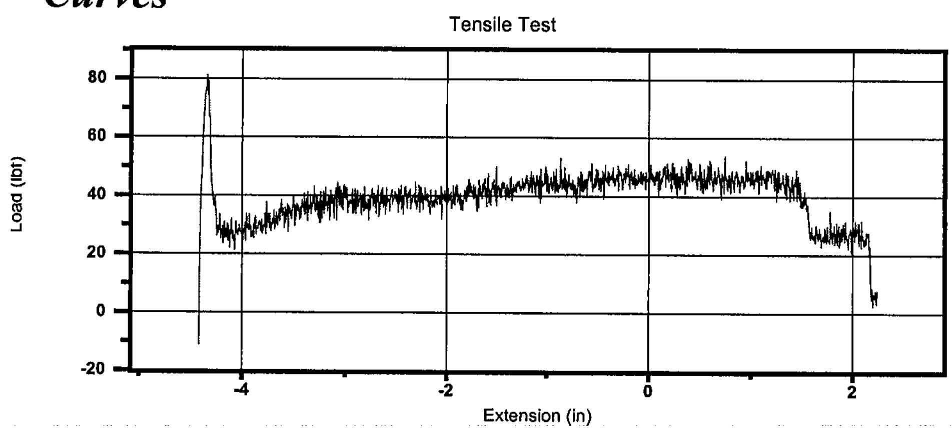
Note 1:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	81.07	-4.34
Mean	81.07	-4.34
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	81.07	-4.34
Maximum	81.07	-4.34
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/27/09

Name: SS-2

Number of specimens: 1

Temperature:

Humidity:

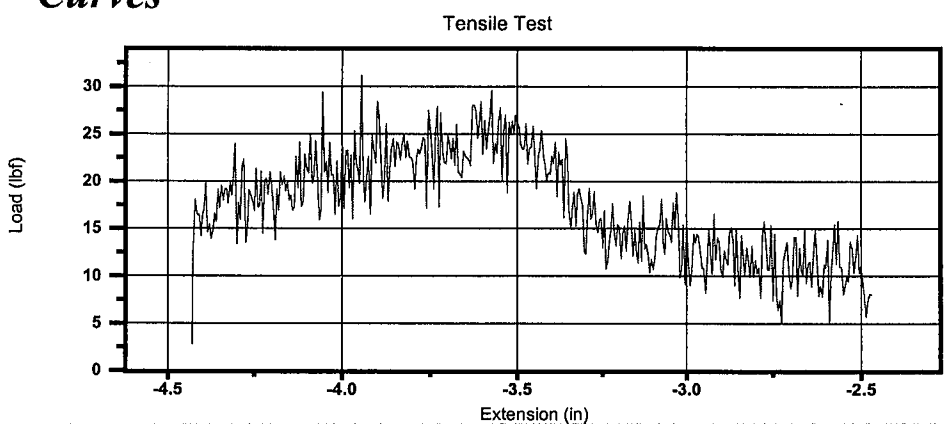
Note 1:

Speed 1: 0.30 in/min

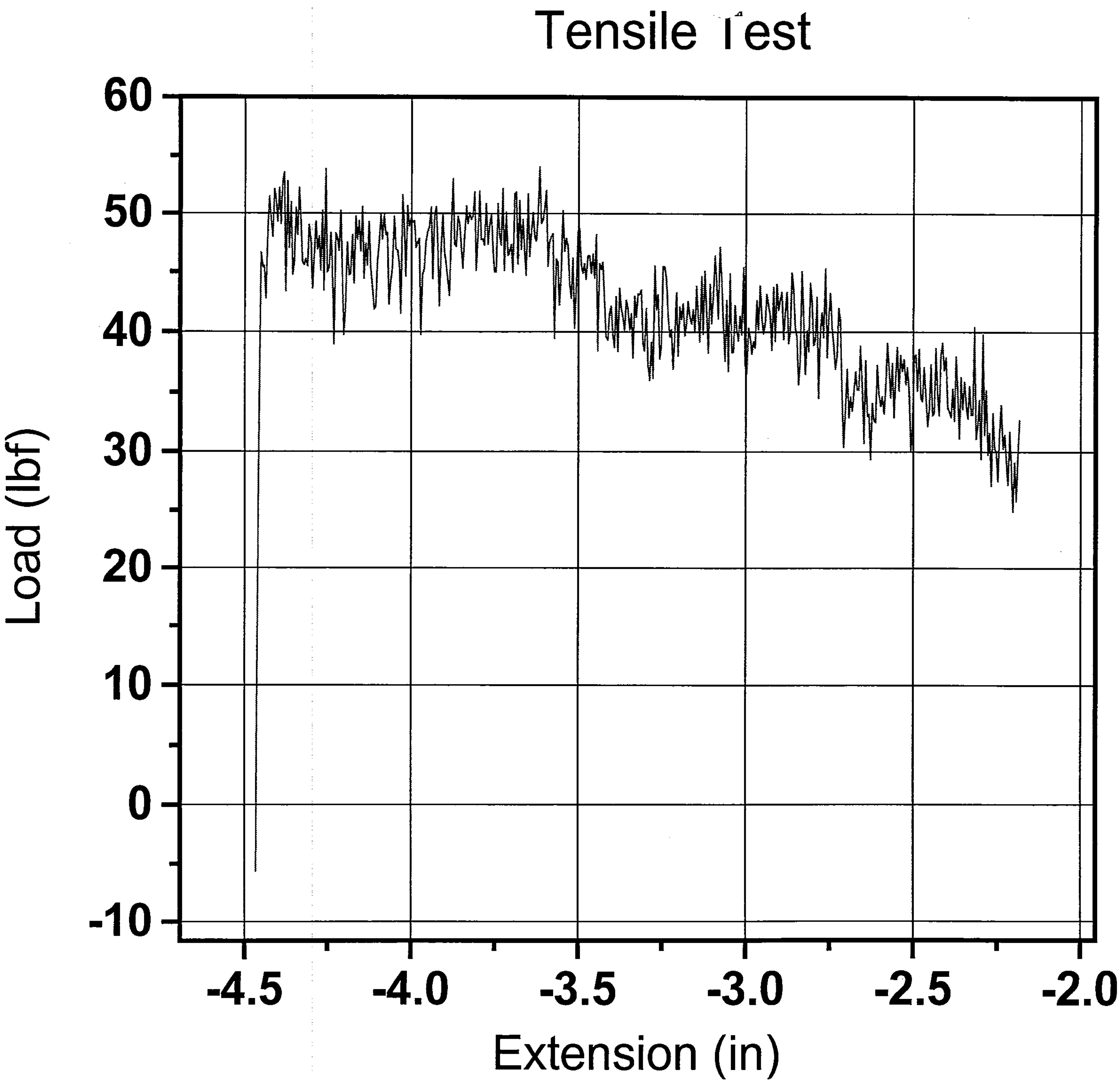
Results

	Maximum Load (lbf)	Extension (in)
1	31.19	-3.94
Mean	31.19	-3.94
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	31.19	-3.94
Maximum	31.19	-3.94
Range	0.00	0.00

Curves



SS3


Tensile Test
60
50
40
30
20
10
0
-10
Load (lbf)
-4.5
-4.0
-3.5
-3.0
-2.5
-2.0
Extension (in)

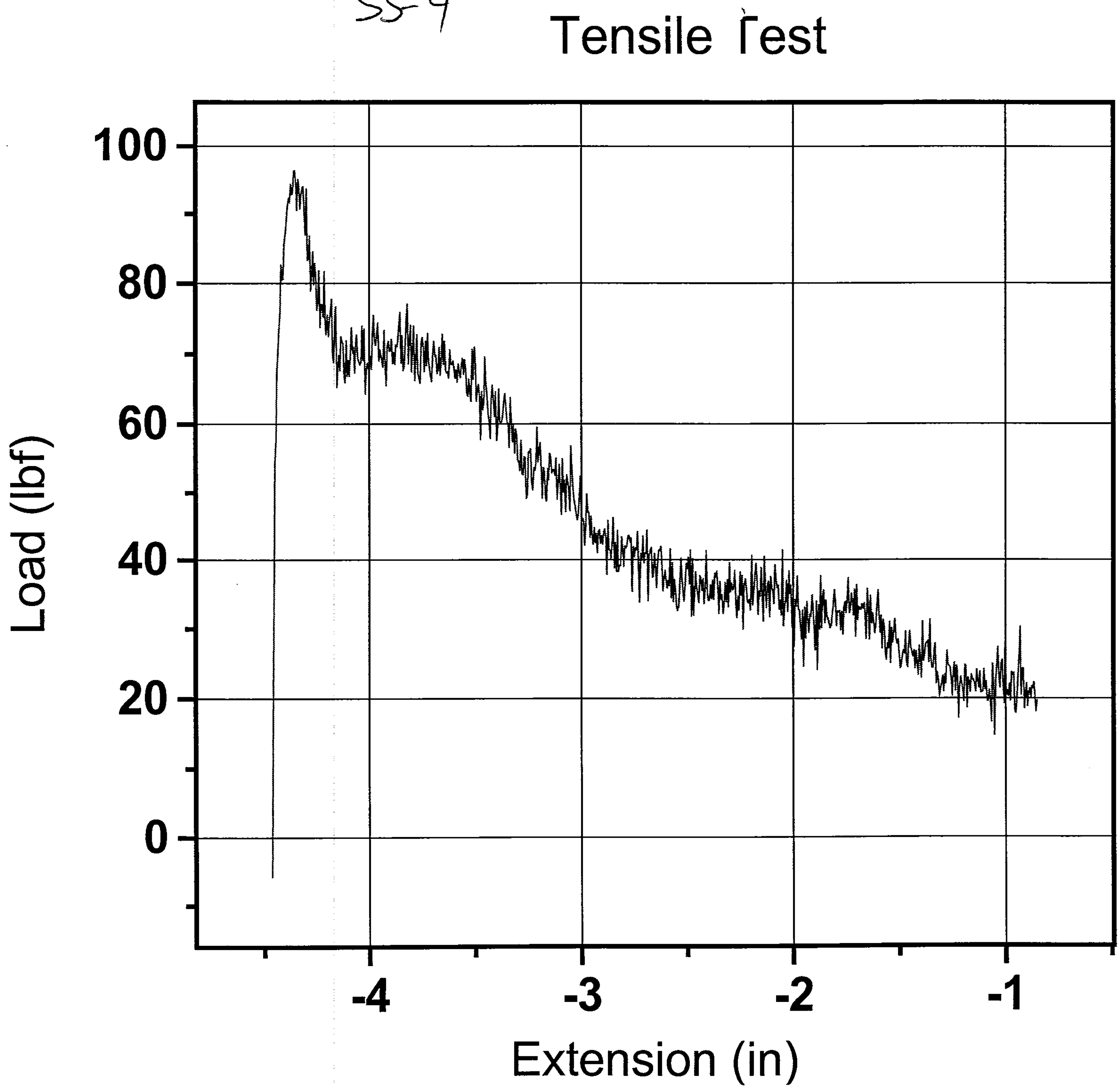
SS3

	Maximum Load (lbf)	Extension (in)
1	54.05	-3.62
Mean	54.05	-3.62
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	54.05	-3.62
Maximum	54.05	-3.62
Range	0.00	0.00

SS-4

	Maximum Load (lbf)	Extension (in)
1	96.33	-4.36
Mean	96.33	-4.36
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	96.33	-4.36
Maximum	96.33	-4.36
Range	0.00	0.00

SS-4



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/20/09

Note 1:

Name: SS-5

Number of specimens: 1

Temperature:

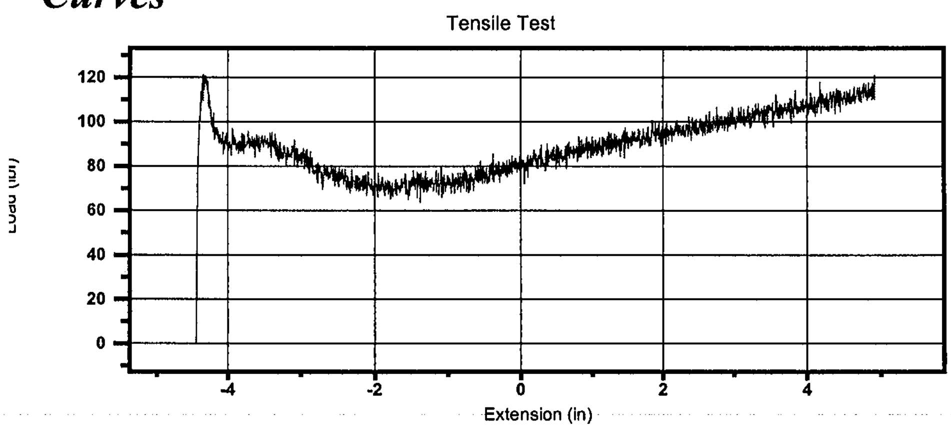
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	121.00	-4.35
Mean	121.00	-4.35
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	121.00	-4.35
Maximum	121.00	-4.35
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/27/09

Note 1:

Name: SS-6

Number of specimens: 1

Temperature:

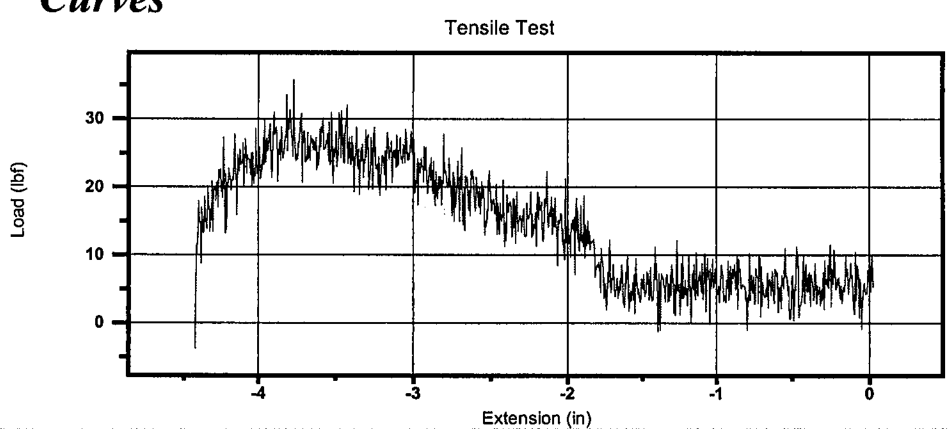
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	35.79	-3.77
Mean	35.79	-3.77
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	35.79	-3.77
Maximum	35.79	-3.77
Range	0.00	0.00

Curves



S-1

Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 1/6/09

Note 1:

Name: S-1

Number of specimens: 1

Temperature:

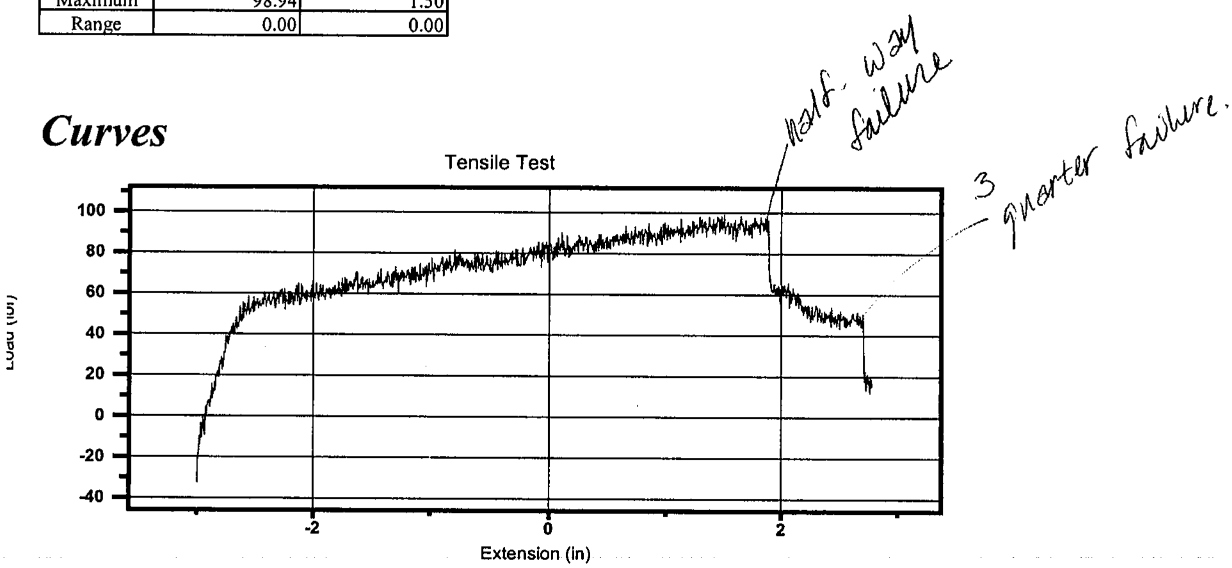
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	98.94	1.50 +3
Mean	98.94	1.50
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	98.94	1.50
Maximum	98.94	1.50
Range	0.00	0.00

Curves



Instron Application Laboratory

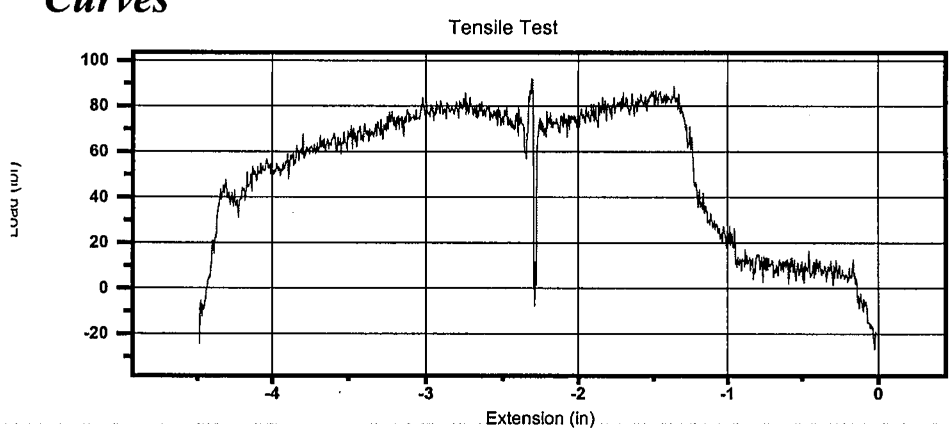
S-2

Company:		Name:	MA
Lab name:	WPI Structures Lab	Number of specimens:	1
Operator ID:		Temperature:	
Test date:	2/2/09	Humidity:	
Note 1:		Speed 1:	0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	91.73	-2.31
Mean	91.73	-2.31
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	91.73	-2.31
Maximum	91.73	-2.31
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/2/09

Note 1:

Name: S-3

Number of specimens: 1

Temperature:

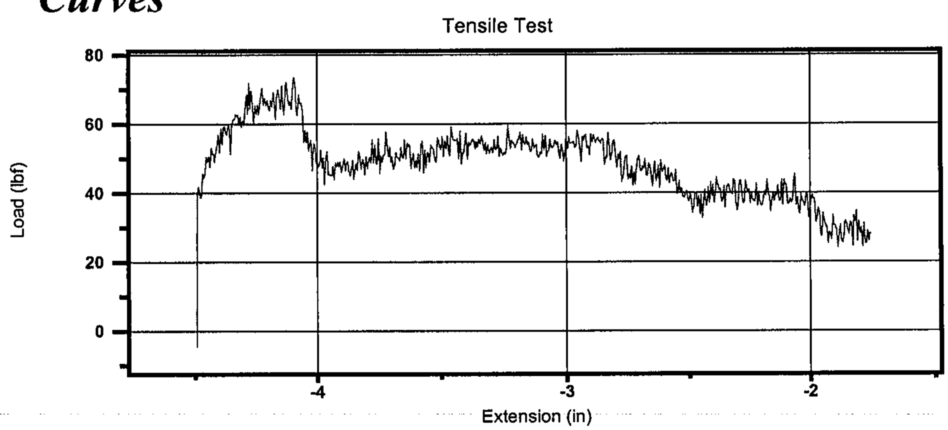
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	73.36	-4.09
Mean	73.36	-4.09
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	73.36	-4.09
Maximum	73.36	-4.09
Range	0.00	0.00

Curves



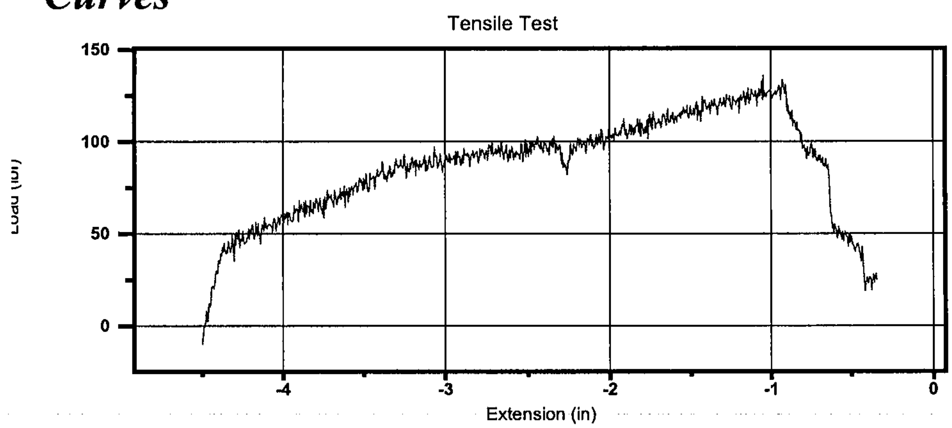
Instron Application Laboratory

Company:		Name: S-4	
Lab name:	WPI Structures Lab	Number of specimens:	1
Operator ID:		Temperature:	
Test date:	2/2/09	Humidity:	
Note 1:		Speed 1:	0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	136.01	-1.05
Mean	136.01	-1.05
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	136.01	-1.05
Maximum	136.01	-1.05
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/2/09

Note 1:

Name: S-6

Number of specimens: 1

Temperature:

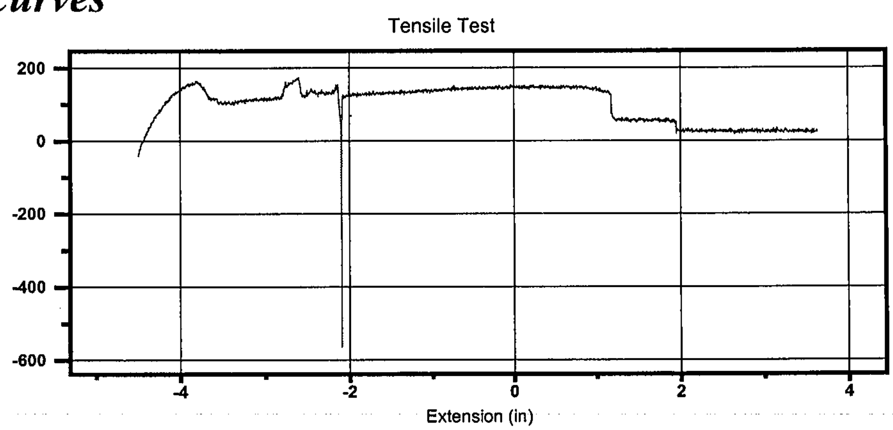
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	173.10	-2.61
Mean	173.10	-2.61
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	173.10	-2.61
Maximum	173.10	-2.61
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/13/09

Note 1:

Name: F1-2 13

Number of specimens: 1

Temperature:

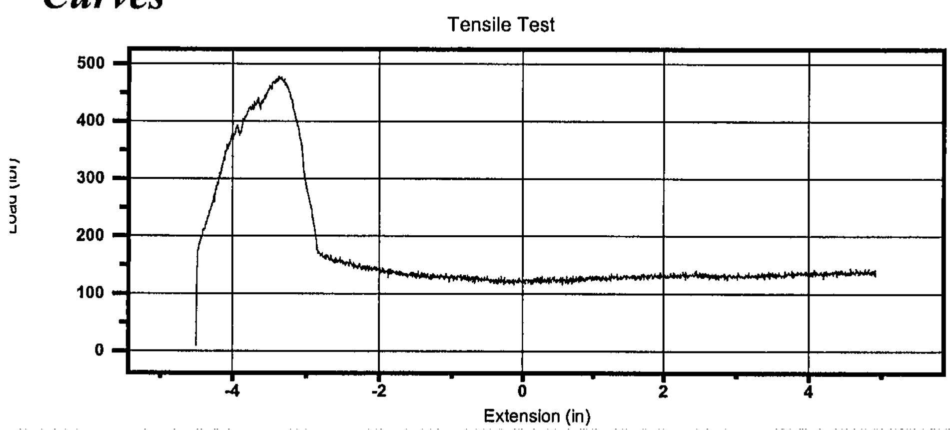
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	478.85	-3.37
Mean	478.85	-3.37
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	478.85	-3.37
Maximum	478.85	-3.37
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/13/09

Note 1:

Name: F2-2 13

Number of specimens: 1

Temperature:

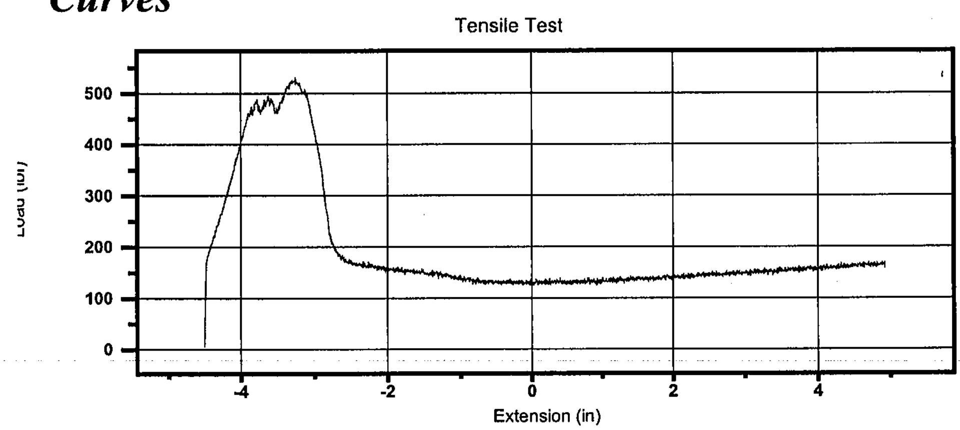
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	-	-
2	-	-
3	530.43	-3.25
Mean	530.43	-3.25
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	530.43	-3.25
Maximum	530.43	-3.25
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/13/09

Note 1:

Name: F3-2 13

Number of specimens: 1

Temperature:

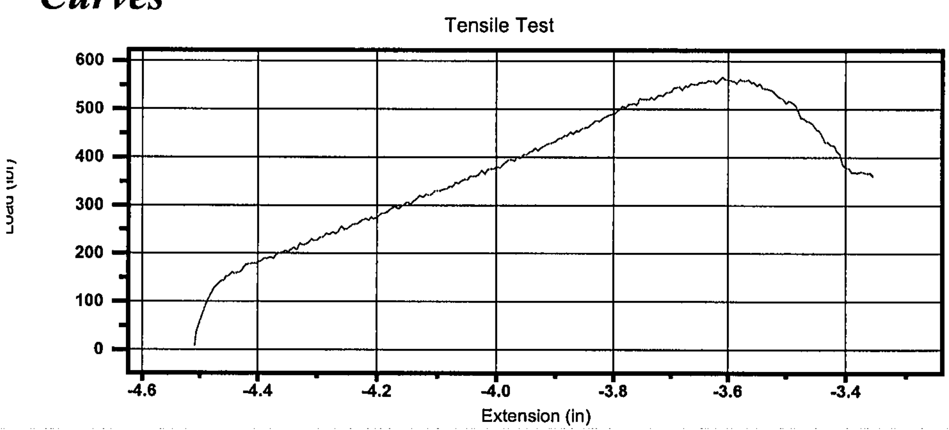
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	566.73	-3.61
Mean	566.73	-3.61
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	566.73	-3.61
Maximum	566.73	-3.61
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/13/09

Note 1:

Name: F3-2 13

Number of specimens: 1

Temperature:

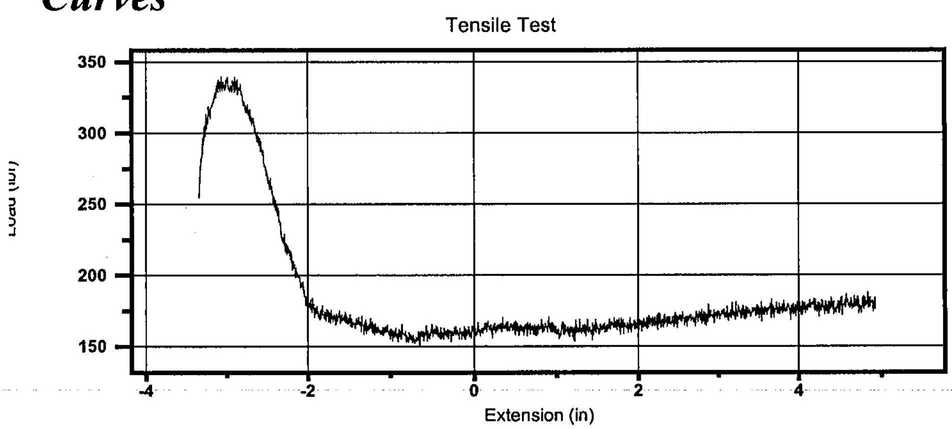
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	566.73	-3.61
2	339.32	-3.07
Mean	453.02	-3.34
S.D.	160.80	0.38
C.V.	35.49	-11.44
Minimum	339.32	-3.61
Maximum	566.73	-3.07
Range	227.41	0.54

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/13/09

Note 1:

Name: F4-2 13

Number of specimens: 1

Temperature:

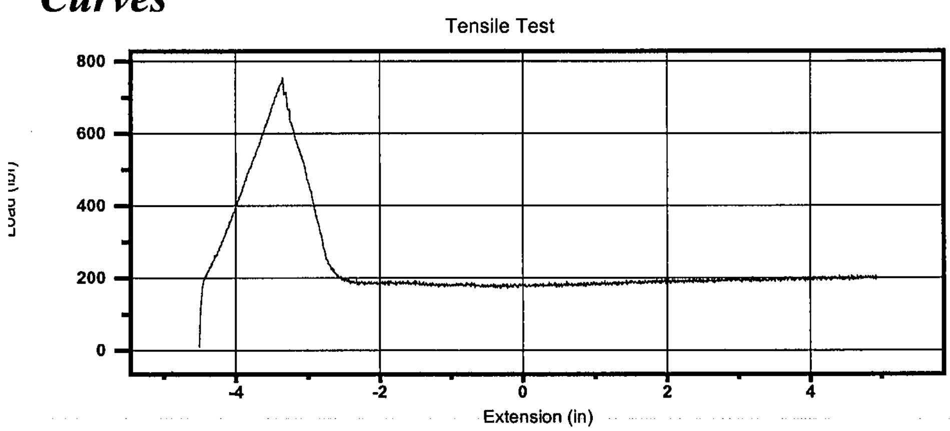
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	753.88	-3.35
Mean	753.88	-3.35
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	753.88	-3.35
Maximum	753.88	-3.35
Range	0.00	0.00

Curves



Instron Application Laboratory

Company:

Lab name: WPI Structures Lab

Operator ID:

Test date: 2/13/09

Note 1:

Name: F5-2 13

Number of specimens: 1

Temperature:

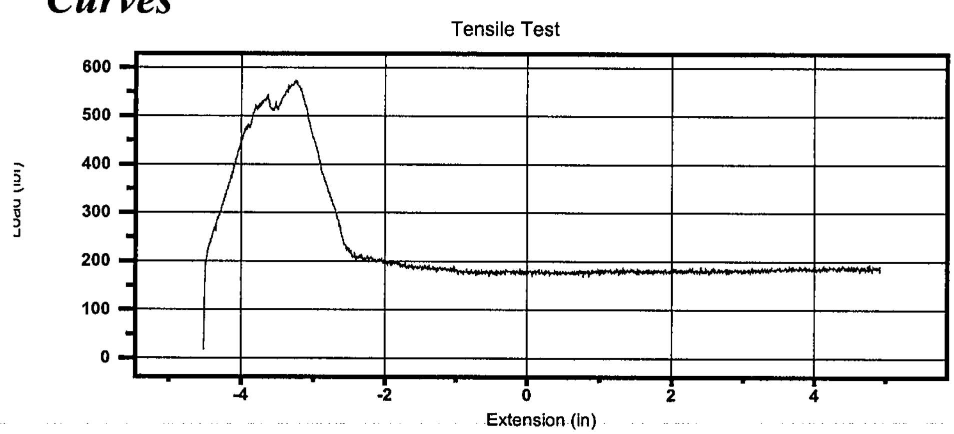
Humidity:

Speed 1: 0.30 in/min

Results

	Maximum Load (lbf)	Extension (in)
1	573.57	-3.25
Mean	573.57	-3.25
S.D.	0.00	0.00
C.V.	0.00	0.00
Minimum	573.57	-3.25
Maximum	573.57	-3.25
Range	0.00	0.00

Curves



Appendix D – Calculations

Loading and Moments

Dead Load

Deck

Unit Weight Concrete x Width x Thickness

$0.145\ ^{k}/_{ft^3} \times 59.39\ ft \times 0.75\ ft$

$w_{deck} = 6.459\ ^{k}/_{ft}$

Sidewalk

Unit Weight Concrete x Area ← 6in x 2m (5.900ft)

$0.145\ ^{k}/_{ft^3} \times 2.9530\ ft^2$

$w_{side} = 0.428\ ^{k}/_{ft}$

Barrier

Unit Weight Concrete x Area

$0.145\ ^{k}/_{ft^3} \times 4.3294\ ft^2$

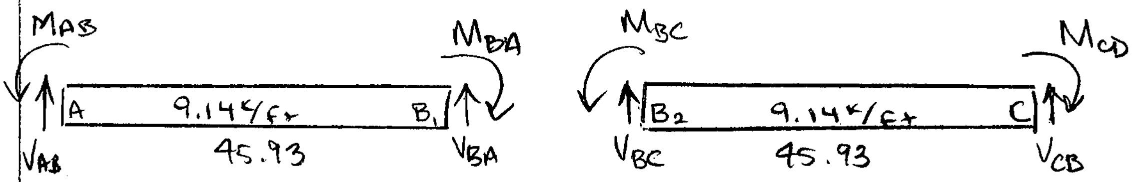
$w_{Bar} = 0.628\ ^{k}/_{ft}$

Asphalt

Unit Weight Bituminous Wearing Surface x Width of Pavement x thickness

$0.140\ ^{k}/_{ft^3} \times 48.23\ ft \times \frac{1}{12}\ ft$ ← assumes 1" thick asphalt layer over deck.

$= 0.563\ ^{k}/_{ft}$



$M_{AB} = 0$

$M_{BC} = -2409$

$M_{BA} = 2409$

$M_{CB} = 0$

$\curvearrowright + \Sigma M_A = 0 = 2409 - 45.93 V_{BA} + \left(\frac{45.93}{2}\right)(9.14)(45.93)$

$V_{BA} = 262$ kips

$\curvearrowright + \Sigma M_{B_1} = 0 = 2409 + 45.93 V_{AB} - \left(\frac{45.93}{2}\right)(9.14)(45.93)$

$V_{AB} = 157$ kips

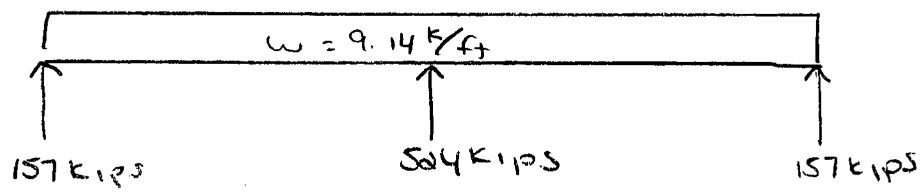
$\curvearrowright + \Sigma M_{B_2} = 0 = -2409 - 45.93 V_{CB} + \left(\frac{45.93}{2}\right)(9.14)(45.93)$

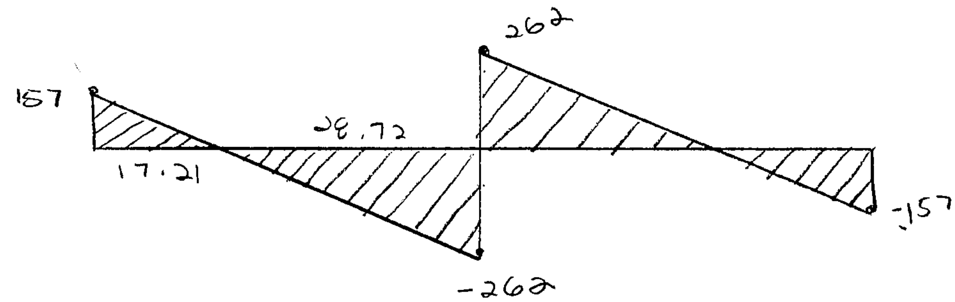
$V_{CD} = 157$ kips

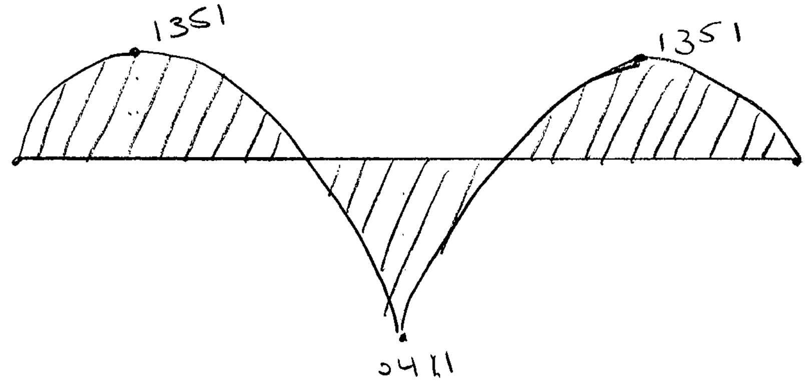
$\curvearrowright + \Sigma M_C = 0 = -2409 + 45.93 V_{BC} - \left(\frac{45.93}{2}\right)(9.14)(45.93)$

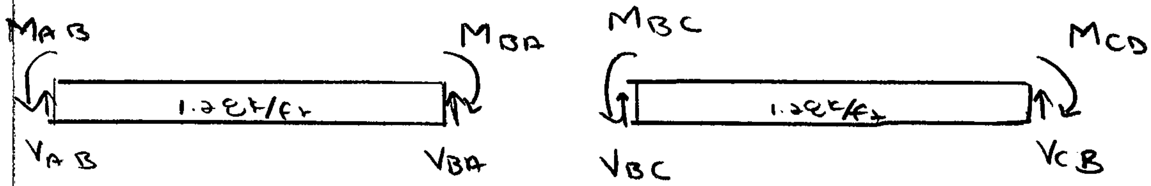
$V_{BC} = 262$ kips

Dead Load - No Girders









$M_{AB} = 0$ $\qquad$ $M_{BC} = -338$

$M_{BA} = 338$ $\qquad$ $M_{CB} = 0$

$$\overset{\curvearrowright}{+} \Sigma M_A = 0 = 338 - 45.93 V_{BA} + \frac{45.93}{2}(1.28)(45.93)$$

$V_{BA} = 37 \text{ kips}$

$$\overset{\curvearrowright}{+} \Sigma M_{B1} = 0 = 338 + 45.93 V_{AB} - \frac{45.93}{2}(1.28)(45.93)$$

$V_{AB} = 22 \text{ kips}$

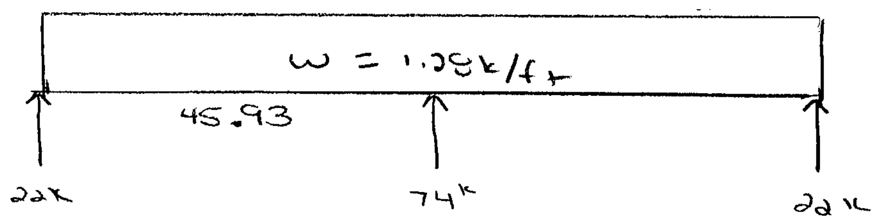
$$\overset{\curvearrowright}{+} \Sigma M_{B2} = 0 = -338 - 45.93 V_{CB} + \frac{45.93}{2}(1.28)(45.93)$$

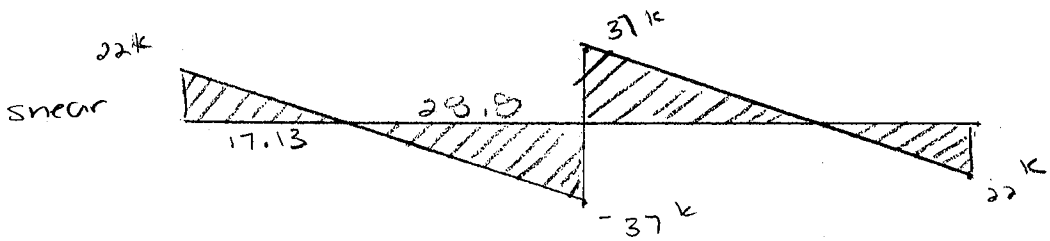
$V_{CB} = 22 \text{ kips}$

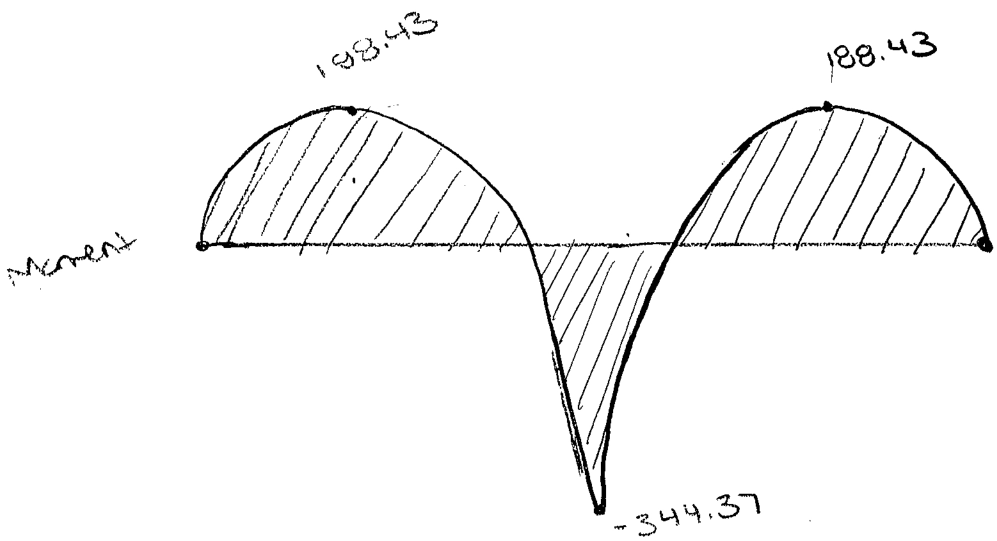
$$\overset{\curvearrowright}{+} \Sigma M_C = 0 = -338 + 45.93 V_{BC} - \frac{45.93}{2}(1.28)(45.93)$$

$V_{BC} = 37 \text{ kips}$

Live Load Entire

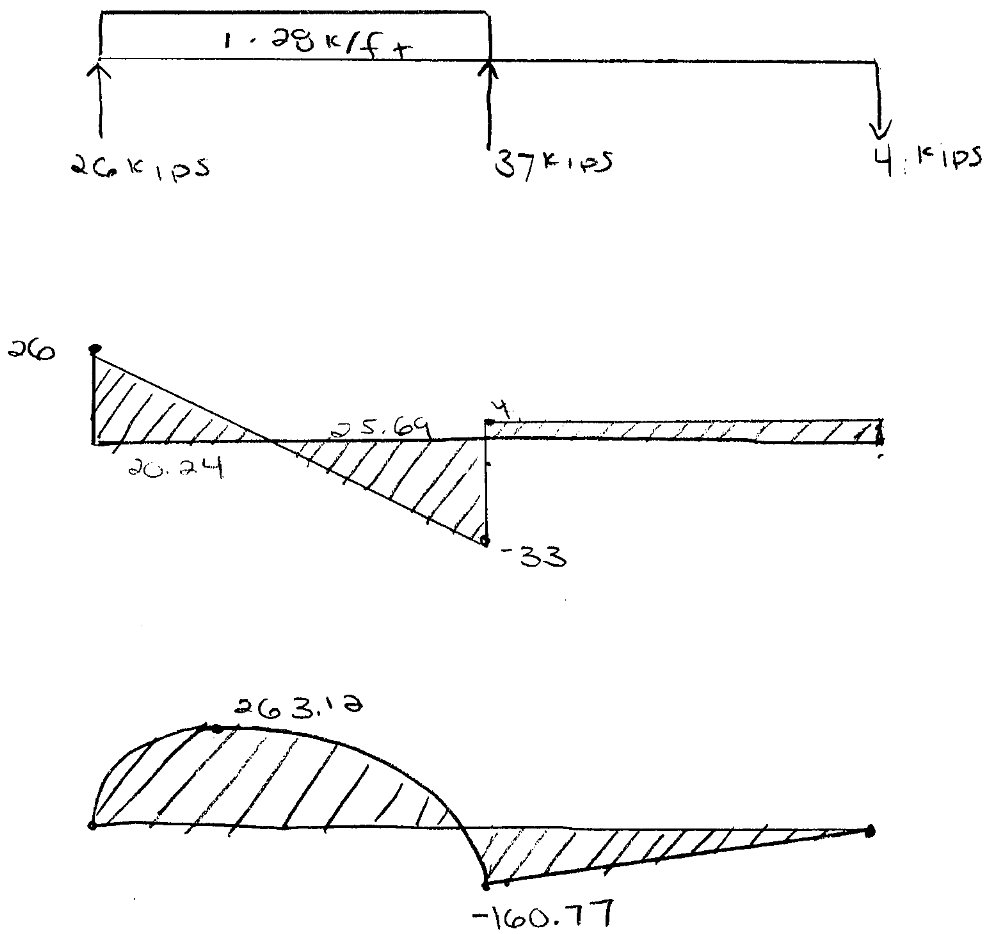






Live Load Half

M 5



$M_{AB} = 0$ $\qquad$ $M_{BC} = -169$

$M_{BA} = 169$ $\qquad$ $M_{CB} = 0$

$\circlearrowright + \sum M_A = 0 = 169 - 45.93 V_{BA} + \frac{45.93}{2}(1.28)(45.93)$

$V_{BA} = 33 \text{ kips}$

$\circlearrowright + \sum M_{B1} = 0 = 169 + 45.93 V_{AB} - \frac{45.93}{2}(1.28)(45.93)$

$V_{AB} = 26 \text{ kips}$

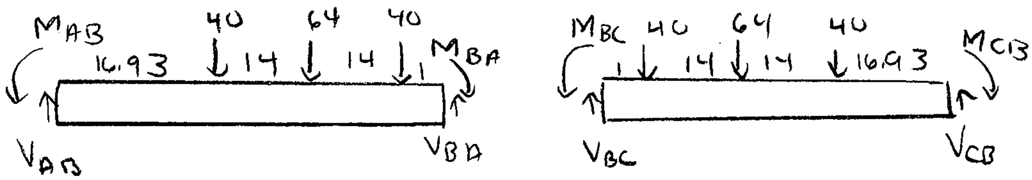
$\circlearrowright + \sum M_{B2} = 0 = -169 - 45.93 V_{CD}$

$V_{CD} = -4 \text{ kips}$

$\circlearrowright + \sum M_C = 0 = -169 + 45.93 V_{BC}$

$V_{BC} = 4 \text{ kips}$

Truck Max Shear



$M_{AB} = 0$ $\quad$ $M_{BC} = -872$

$M_{BA} = 872$ $\quad$ $M_{CB} = 0$

$$(+\circlearrowright)\ \Sigma M_A = 0 = 872 + 40(16.93) + 64(30.93) + 40(44.93) - V_{BA}(45.93)$$

$$V_{BA} = 116 \text{ kips}$$

$$(+\circlearrowright)\ \Sigma M_{B1} = 0 = 872 + 45.93 V_{AB} - 40(1) - 64(15) - 40(29)$$

$$V_{AB} = 28 \text{ kips}$$

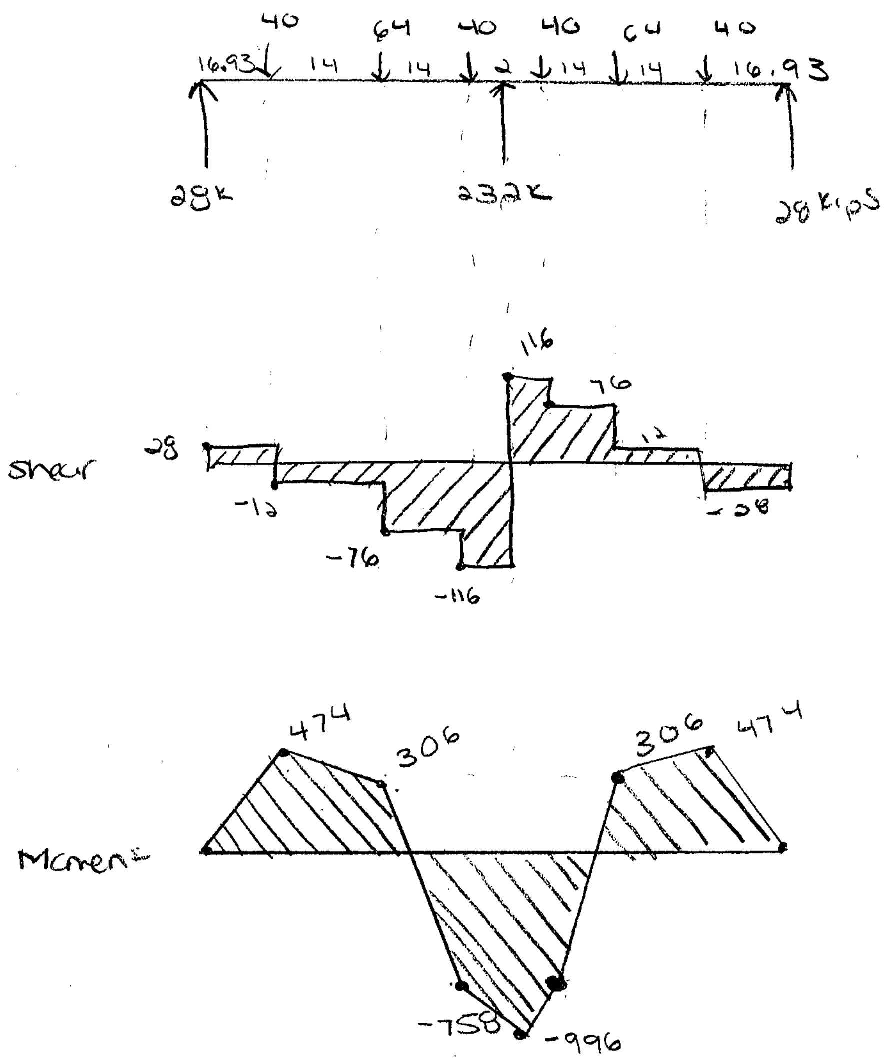
$$(+\circlearrowright)\ \Sigma M_{B2} = 0 = -872 + 40(1) + 64(15) + 40(29) - 45.93 V_{CB}$$

$$V_{CB} = 28 \text{ kips}$$

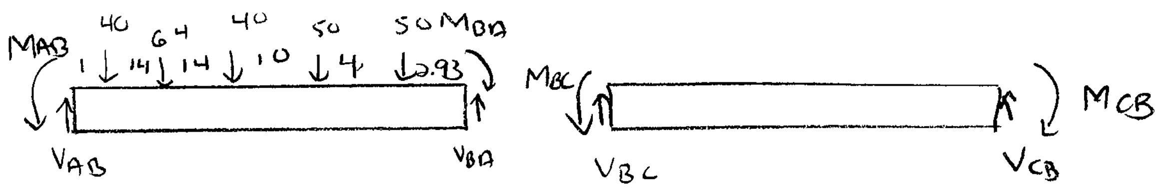
$$(+\circlearrowright)\ \Sigma M_C = 0 = -872 + 45.93 V_{BC} - 40(16.93) - 64(30.93) - 40(44.93)$$

$$V_{BC} = 116 \text{ kips}$$

Truck Max Shear



Truck Max + Moment



$M_{AB} = 0$ $\qquad$ $M_{BC} = -601$

$M_{BA} = 601$ $\qquad$ $M_{CB} = 0$

$$\circlearrowright^{+} \Sigma M_A = 0 = 601 + 40(1) + 64(15) + 40(29) + 50(39)$$
$$+ 50(43) - 45.93 V_{BA}$$

$V_{BA} = 149$ KIPS

$$\circlearrowright^{+} \Sigma M_{B1} = 0 = 601 - 50(2.93) - 50(6.93) - 40(16.93)$$
$$- 64(30.93) - 40(44.93) + 45.93 V_{AB}$$

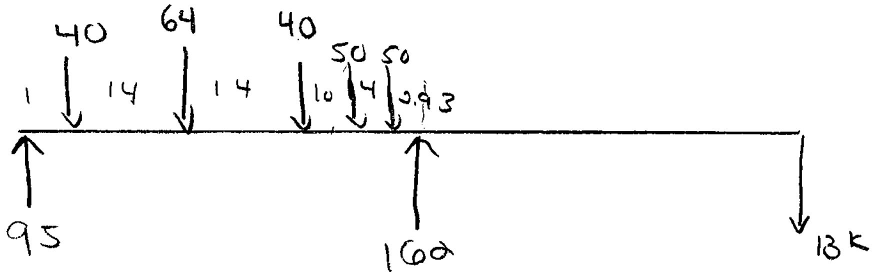
$V_{AB} = 95$ kips

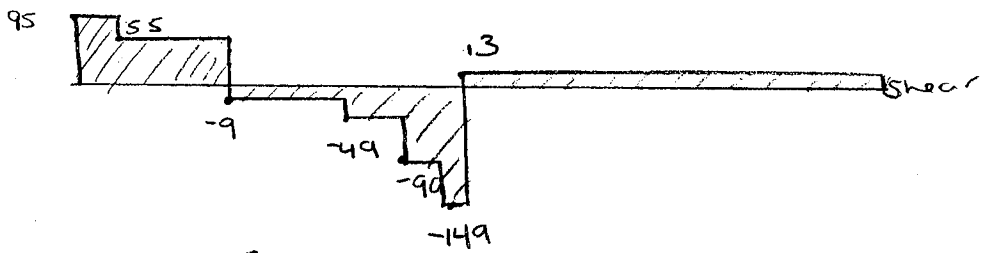
$$\circlearrowright^{+} \Sigma M_{B2} = 0 = -601 - 45.93 V_{CB}$$

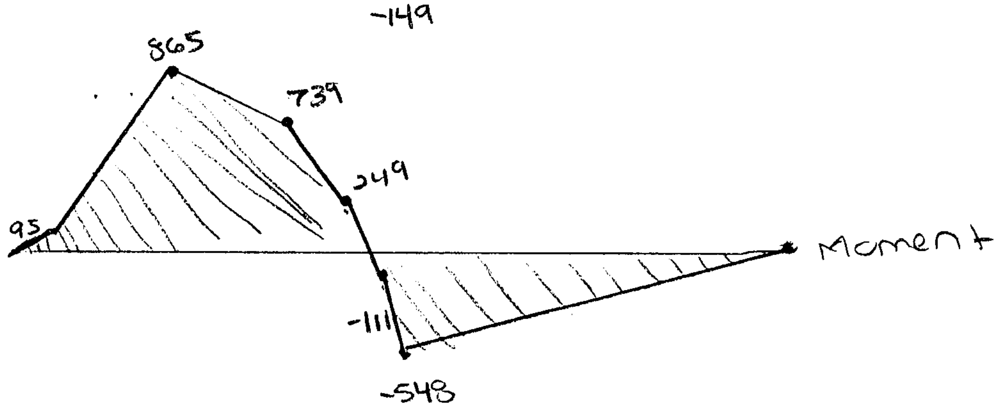
$V_{CB} = -13$ kips

$$\circlearrowright^{+} \Sigma M_C = 0 = -601 + 45.93 V_{BC}$$

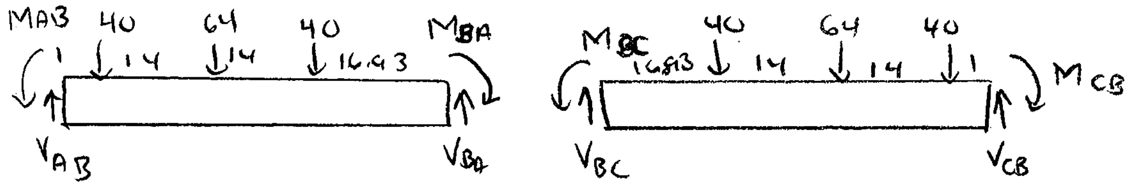
$V_{BC} = 13$ kips


40
64
40
50
50
1
14
14
10
4
0.9
3
95
160
13 k


95
55
13
Shear
-9
-49
-99
-149


865
739
249
95
Moment
-111
-548

Truck Max - Moment



$M_{AB} = 0$

$M_{BA} = 798$

$M_{BC} = -798$

$M_{CB} = 0$

$$\circlearrowright^{+} \Sigma M_A = 0 = -798 + 40(1) + 64(15) + 40(29) - 45.93\, V_{BA}$$

$V_{BA} = 64^k$

$$\circlearrowright^{+} \Sigma M_{B1} = 0 = 798 + 45.93\, V_{AB} - 40(16.93) - 64(30.93) - 40(44.93)$$

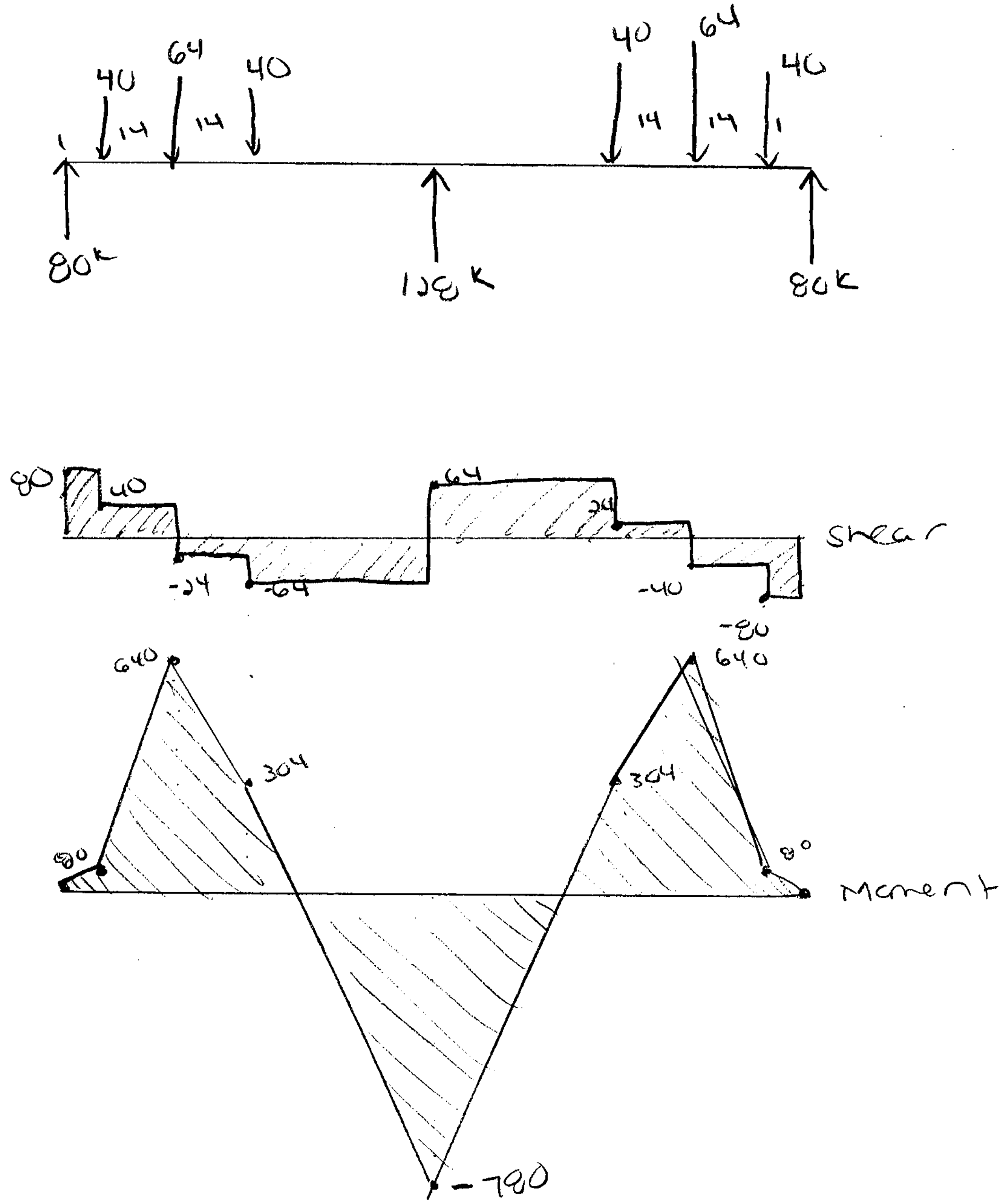
$V_{AB} = 80^k$

$$\circlearrowright^{+} \Sigma M_{B2} = 0 = -798 + 40(16.93) + 64(30.93) + 40(44.93) - 45.93\, V_{CB}$$

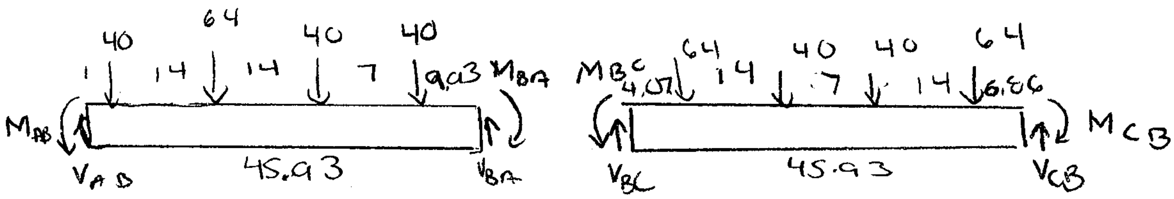
$V_{CB} = 80^k$

$$\circlearrowright^{+} \Sigma M_C = 0 = -798 + 45.93\, V_{BC} - 40(1) - 64(15) - 40(29)$$

$V_{BC} = 64^k$


40
64
40
14
14
1
40
64
40
14
14
1
80k
128k
80k
80
40
64
24
-24
-64
-40
-80
Shear
640
304
80
640
304
80
-780
Moment

Truck Fully Loaded



$M_{AB} = 0$

$M_{BC} = -1100$

$M_{BA} = 1100$

$M_{CB} = 0$

$\circlearrowleft+ \; \Sigma M_A = 0 = 1100 + 40(1) + 64(15) + 40(29) + 40(36) - 45.93 V_{BA}$

$V_{BA} = 102$ kips

$\circlearrowleft+ \; \Sigma M_{B1} = 0 = 1100 + 45.93 V_{AB} - 40(9.93) - 40(16.93) - 64(30.93) - 40(44.93)$

$V_{AB} = 82$ kips

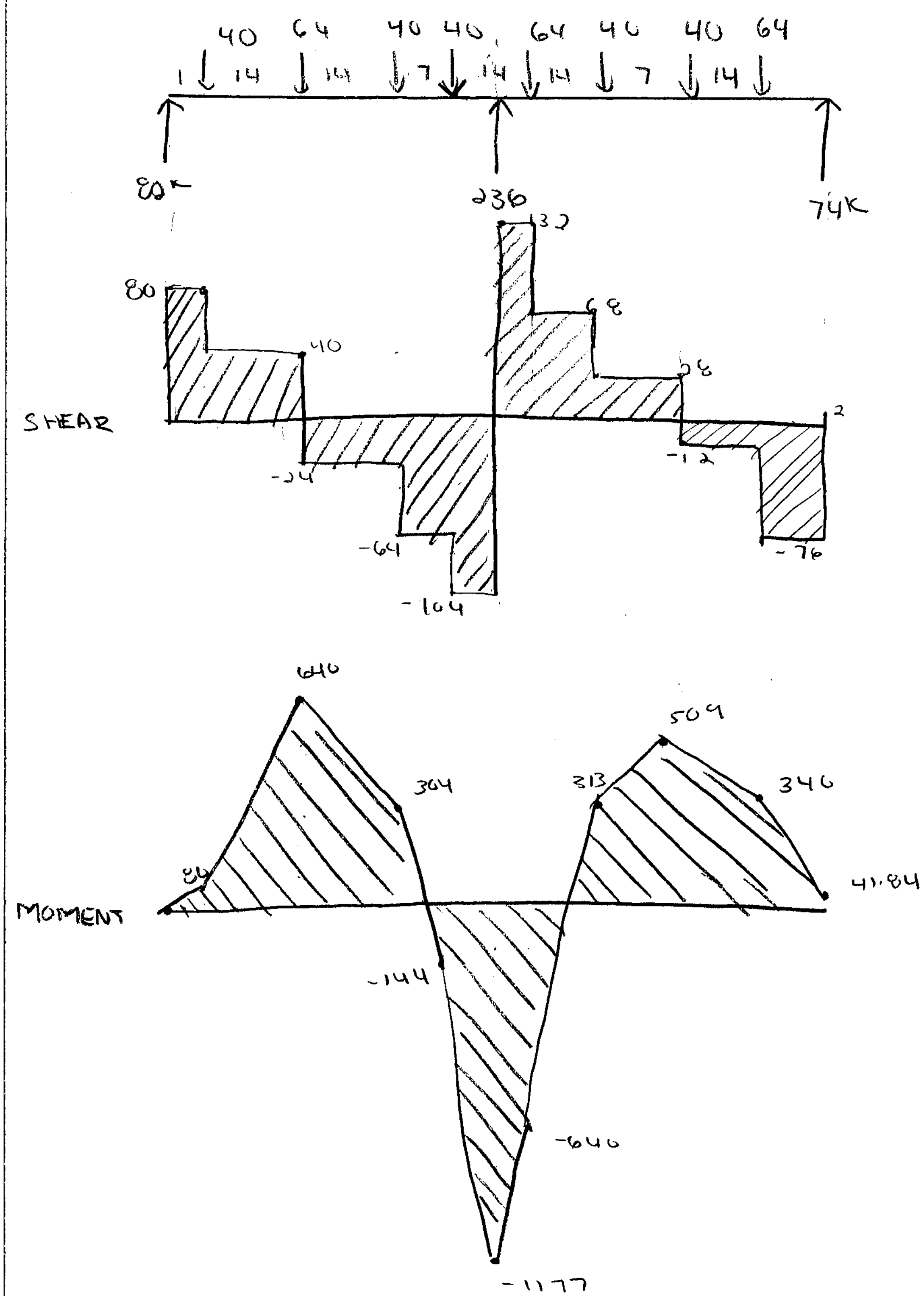
$\circlearrowleft+ \; \Sigma M_{B2} = 0 = -1100 + 64(4.07) + 40(18.07) + 40(25.07) + 64(39.07) - 45.93 V_{CB}$

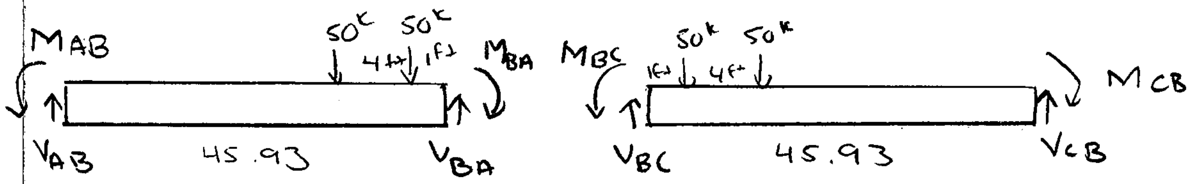
$V_{CB} = 74$ kips

$\circlearrowleft+ \; \Sigma M_C = 0 = -1100 + 45.93 V_{BC} - 64(6.86) - 40(20.86) - 40(27.86) - 64(41.86)$

$V_{BC} = 134$ kips

Truck Fully Loaded





$M_{AB} = 0$

$M_{BA} = 259$

$M_{BC} = -259$

$M_{CB} = 0$

$\circlearrowleft + \Sigma M_A = 0 = 259 + 50(44.93) + 50(40.93) - 45.93 V_{BA}$

$V_{BA} = 99$ kips

$\circlearrowleft + \Sigma M_{B1} = 0 = 259 + 45.93 V_{AB} - 50(1) - 50(5)$

$V_{AB} = 1$ kip

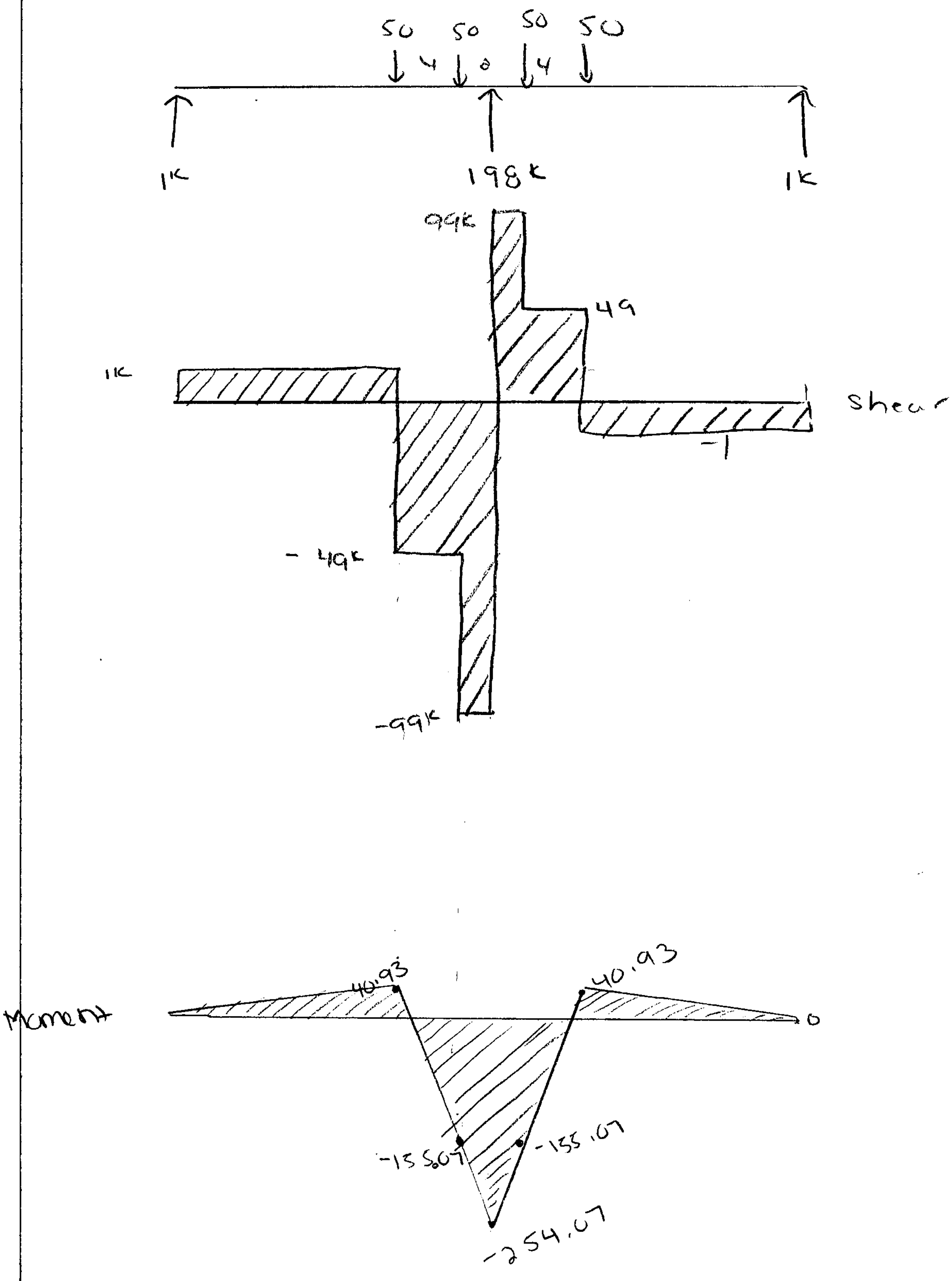
$\circlearrowleft + \Sigma M_{B2} = 0 = -259 + 1(50) + (5)(50) - 45.93 V_{CB}$

$V_{CB} = 1$ kip

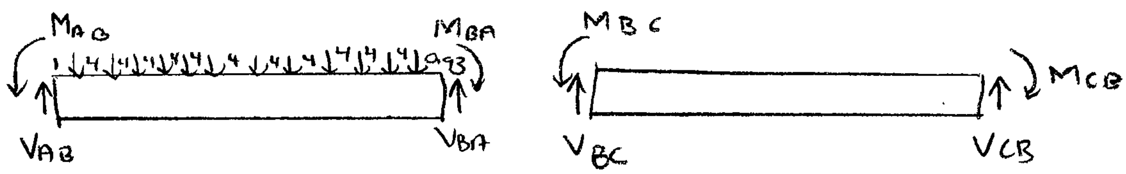
$\circlearrowleft + \Sigma M_C = 0 = -259 - 44.93(50) - 40.93(50) + 45.93 V_{BC}$

$V_{BC} = 99$ kips

Car Max Shear



Car Max + Moment



$M_{AB} = 0$

$M_{BA} = 1649$

$M_{BC} = -1649$

$M_{CB} = 0$

$$\circlearrowleft + \Sigma M_A = 0 = 1649 + 50(1) + 50(5) + 50(9) + 50(13) + 50(17) + 50(21) + 50(25) + 50(29) + 50(33) + 50(37) + 50(41) + 50(45) - 45.93 V_{BA}$$

$V_{BA} = 337^k$

$$\circlearrowleft + \Sigma M_{B_1} = 0 = 1649 - 50(0.93) - 50(4.93) - 50(8.93) - 50(12.93) - 50(16.93) - 50(20.93) - 50(24.93) - 50(28.93) - 50(32.93) - 50(36.93) - 50(40.93) - 50(44.93) + 45.93 V_{AB}$$

$V_{AB} = 264^k$

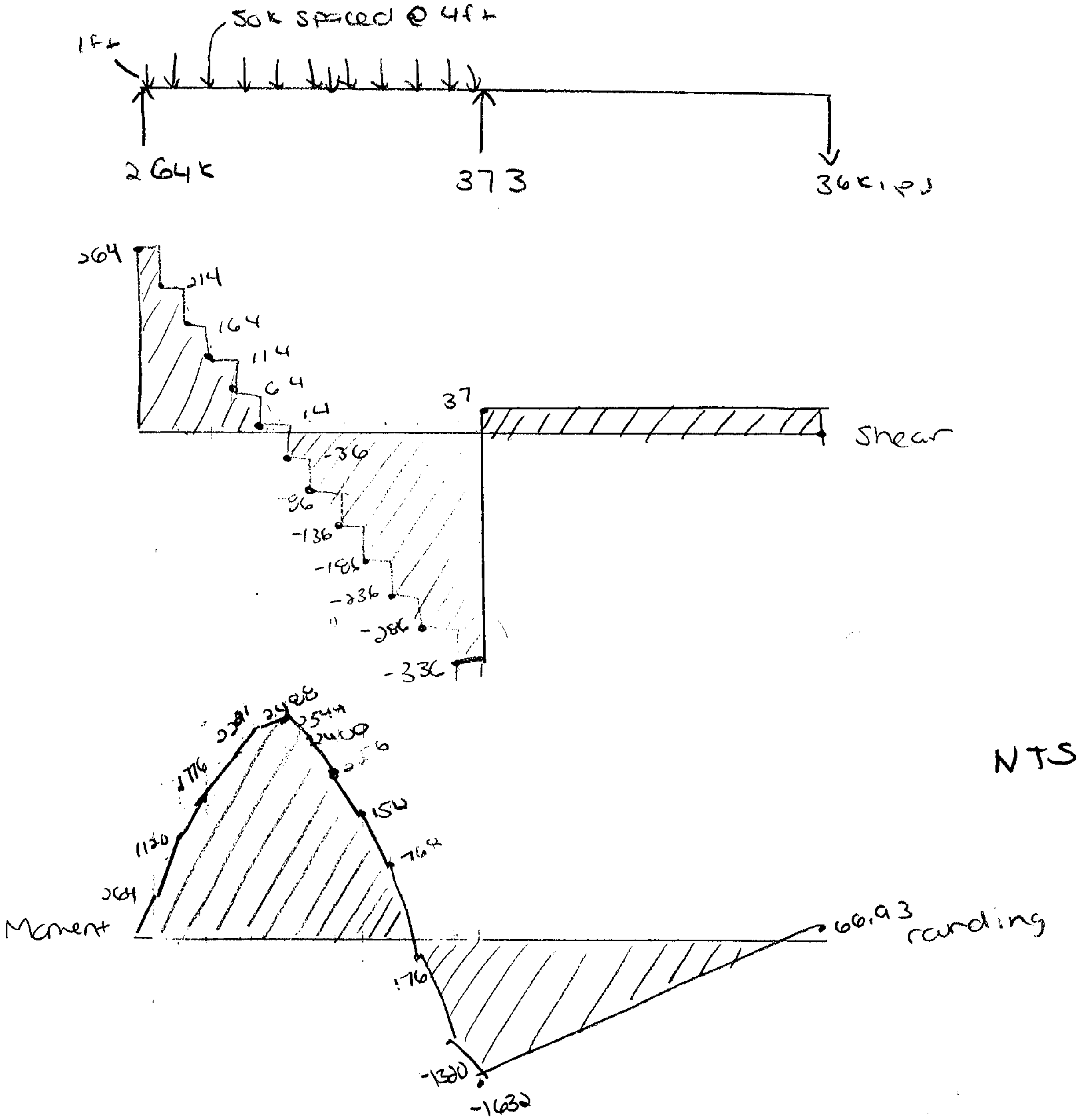
$$\circlearrowleft + \Sigma M_{B_2} = 0 = -1649 - 45.93 V_{CB}$$

$V_{CB} = -36$ kips

$$\circlearrowleft + \Sigma M_C = 0 = -1649 + 45.93 V_{BC}$$

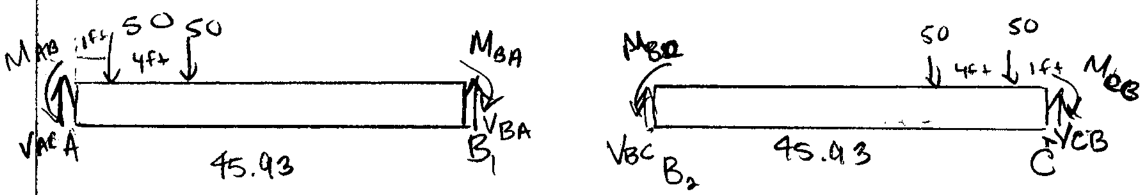
$V_{BC} = 36$ kips

Car Max (+) Moment



2544 = Max + Moment

Car Max (-) Moment



$M_{AB} = 0$ $\qquad$ $M_{BC} = -149$

$M_{BA} = 149$ $\qquad$ $M_{CB} = 0$

$\curvearrowright + \Sigma M_A = 0 = 149 + 50 + 5(50) - 45.93 V_{BA}$

$V_{BA} = 10$ kips

$\curvearrowright + \Sigma M_{B_1} = 0 = 149 + 45.93 V_{AB} - 40.93(50) - 44.93(50)$

$V_{AB} = 90$ kips

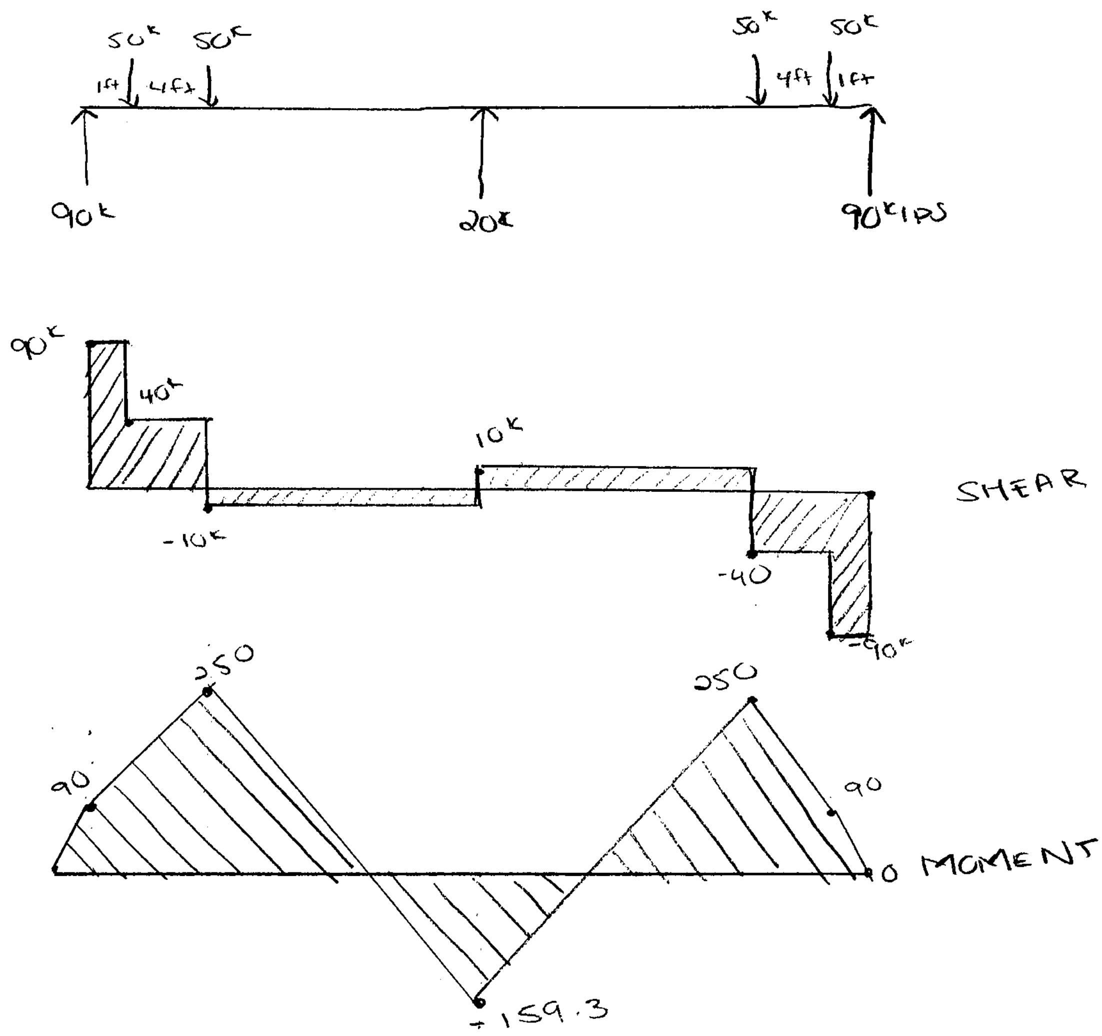
$\curvearrowright + \Sigma M_{B_2} = 0 = -149 + 40.93(50) + 44.93(50) - 45.93 V_{CB}$

$V_{CB} = 90$ kips

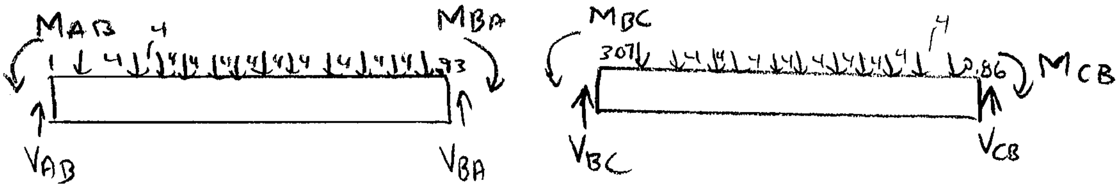
$\curvearrowright + \Sigma M_D = 0 = -149 - (1)(50) - (5)(50) + 45.93 V_{BC}$

$V_{BC} = 10$ kips

Calc Max (-) Moment



Car Fully Loaded



$M_{AB} = 0$

$M_{BA} = 3299$

$M_{BC} = -3299$

$M_{CB} = 0$

$$\circlearrowleft\!+\ \Sigma M_A = 0 = 3299 + 50(1) + 50(5) + 50(9) + 50(13) + 50(17) + 50(21) + 50(25) + 50(29) + 50(33) + 50(37) + 50(41) + 50(45) - V_{BA}(45.93)$$

$V_{BA} = 372^k$

$$\circlearrowleft\!+\ \Sigma M_{B_1} = 0 = 3299 - 50(0.93) - 50(4.93) - 50(8.93) - 50(12.93) - 50(16.93) - 50(20.93) - 50(24.93) - 50(28.93) - 50(32.93) - 50(36.93) - 50(40.93) - 50(44.9 + 45.93\,V_{AB}$$

$V_{AB} = 228^k$

$$\circlearrowleft\!+\ \Sigma M_{B_2} = 0 = -3299 + 50(3.07) + 50(7.07) + 50(11.07) + 50(15.07) + 50(19.07) + 50(23.07) + 50(27.07) + 50(31.07) + 50(35.07) + 50(39.0 + 50(43.07) - 45.93\,V_{CB}$$

$V_{CB} = 204$ kips

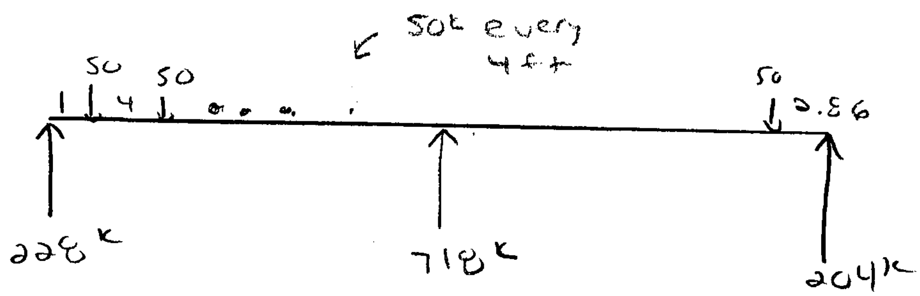
M
22

Car Fully Loaded

$$\circlearrowleft + \quad \Sigma M_C = 0 = -3299 - 50(2.86) - 50(6.86) - 50(10.86)$$
$$- 50(14.86) - 50(18.86) - 50(22.86) - 50(26.86)$$
$$- 50(30.86) - 50(34.86) - 50(38.86) - 50(42.86)$$
$$+ 45.93 V_{BC}$$

$$V_{BC} = 346^{kips}$$

Car Fully Loaded



since so close together use table

Position	Shear	Moment
0	228	0
1	178	712
4	128	512
5	79	316
9	28	112
13	-22	-88
17	-72	-288
21	-122	-488
25	-172	-688
29	-222	-888
33	-272	-1088
37	-322	-1288
41	-372	-1488
44.93	346	1384
45	296	1184
49	246	984
53	196	784
57	146	584

Position	Shear	Moment
61	96	384
64	46	184
67	-4	-16
71	-54	-216
75	-104	-416
79	-154	-616
83	-204	-583.4
91.86	0	0

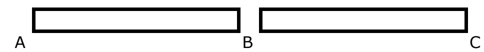
Factors

Stiffness Factors
Kab=Kba=Kbc=Kcb
K=4EI/L 0.087089049 EI

Distribution Factors

DF (AB)	1
DF (BA)	0.5
DF (BC)	0.5
DF (CB)	1

Truck Fully Loaded Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

d

Truck Fully Loaded Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
40	1	44.93	38.27718	0.85193
64	15	30.93	435.3496	211.13
40	29	16.93	157.6083	269.973
40	36	9.93	67.30822	244.018

FEM (AB) -698.5433063 FEM (BA) 725.972

Truck Fully Loaded Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
64	4.07	41.86	216.3615	21.0366
40	18.07	27.86	265.9423	172.49
40	25.07	20.86	206.8474	248.594
64	39.07	6.86	55.78003	317.686

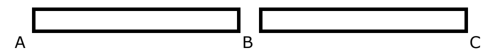
FEM (BC) -744.9312296 FEM (CB) 759.806

Truck Fully Loaded Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-698.543	725.972	-744.931	759.806
Dist.	698.543	9.479	9.479	-759.806
CO	4.740	349.272	-379.903	4.740
Dist.	-4.740	15.316	15.316	-4.740
CO	7.658	-2.370	-2.370	7.658
Dist.	-7.658	2.370	2.370	-7.658
CO	1.185	-3.829	-3.829	1.185
Dist.	-1.185	3.829	3.829	-1.185
CO	1.914	-0.592	-0.592	1.914
Dist.	-1.914	0.592	0.592	-1.914
CO	0.296	-0.957	-0.957	0.296
Dist.	-0.296	0.957	0.957	-0.296
CO	0.479	-0.148	-0.148	0.479
Dist.	-0.479	0.148	0.148	-0.479
CO	0.074	-0.239	-0.239	0.074
Dist.	-0.074	0.239	0.239	-0.074
CO	0.120	-0.037	-0.037	0.120
Dist.	-0.120	0.037	0.037	-0.120
CO	0.019	-0.060	-0.060	0.019
Dist.	-0.019	0.060	0.060	-0.019
CO	0.030	-0.009	-0.009	0.030
Dist.	-0.030	0.009	0.009	-0.030
CO	0.005	-0.015	-0.015	0.005
Dist.	-0.005	0.015	0.015	-0.005
CO	0.007	-0.002	-0.002	0.007
Dist.	-0.007	0.002	0.002	-0.007
CO	0.001	-0.004	-0.004	0.001
Dist.	-0.001	0.004	0.004	-0.001
CO	0.002	-0.001	-0.001	0.002
Dist.	-0.002	0.001	0.001	-0.002
CO	0.000	-0.001	-0.001	0.000
Dist.	0.000	0.001	0.001	0.000
ΣM	0	1100	-1100	0

Truck Max Shear Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

Truck Max Shear Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
40	16.93	29	269.9728	157.608
64	30.93	15	211.1298	435.35
40	44.93	1	0.851929	38.2772

FEM (AB) -481.954549 FEM (BA) 631.235

Truck Max Shear Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
40	1	44.93	38.27718	0.85193
64	15	30.93	435.3496	211.13
40	29	16.93	157.6083	269.973

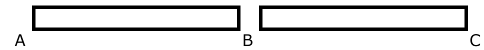
FEM (BC) -631.2350874 FEM (CB) 481.955

Truck Max Shear Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-481.955	631.235	-631.235	481.955
Dist.	481.955	0.000	0.000	-481.955
CO	0.000	240.977	-240.977	0.000
Dist.	0.000	0.000	0.000	0.000
ΣM	0	872	-872	0

Truck Max - Moment Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

Truck Max - Moment Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
40	1	44.93	38.27718	0.85193
64	15	30.93	435.3496	211.13
40	29	16.93	157.6083	269.973

FEM (AB) -631.2350874

FEM (BA) 481.955

Truck Max - Moment Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
40	16.93	29	269.9728	157.608
64	30.93	15	211.1298	435.35
40	44.93	1	0.851929	38.2772

FEM (BC) -481.954549

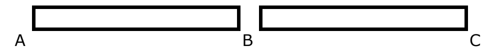
FEM (CB) 631.235

Truck Max - Moment Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-631.235	481.955	-481.955	631.235
Dist.	631.235	0.000	0.000	-631.235
CO	0.000	315.618	-315.618	0.000
Dist.	0.000	0.000	0.000	0.000
ΣM	0	798	-798	0

Truck Max + Moment Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

Truck Max + Moment Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
40	1	44.93	38.27718	0.85193
64	15	30.93	435.3496	211.13
40	29	16.93	157.6083	269.973
50	39	6.93	44.39236	249.827
50	43	2.93	8.749451	128.405

FEM (AB) -684.376894

FEM (BA) 860.187

Truck Max + Moment Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
0	0	0	0	0

FEM (BC) 0

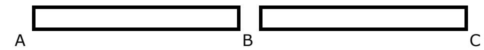
FEM (CB) 0

Truck Max + Moment Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-684.377	860.187	0.000	0.000
Dist.	684.377	-430.093	-430.093	0.000
CO	-215.047	342.188	0.000	-215.047
Dist.	215.047	-171.094	-171.094	215.047
CO	-85.547	107.523	107.523	-85.547
Dist.	85.547	-107.523	-107.523	85.547
CO	-53.762	42.774	42.774	-53.762
Dist.	53.762	-42.774	-42.774	53.762
CO	-21.387	26.881	26.881	-21.387
Dist.	21.387	-26.881	-26.881	21.387
CO	-13.440	10.693	10.693	-13.440
Dist.	13.440	-10.693	-10.693	13.440
CO	-5.347	6.720	6.720	-5.347
Dist.	5.347	-6.720	-6.720	5.347
CO	-3.360	2.673	2.673	-3.360
Dist.	3.360	-2.673	-2.673	3.360
CO	-1.337	1.680	1.680	-1.337
Dist.	1.337	-1.680	-1.680	1.337
CO	-0.840	0.668	0.668	-0.840
Dist.	0.840	-0.668	-0.668	0.840
CO	-0.334	0.420	0.420	-0.334
Dist.	0.334	-0.420	-0.420	0.334
CO	-0.210	0.167	0.167	-0.210
Dist.	0.210	-0.167	-0.167	0.210
CO	-0.084	0.105	0.105	-0.084
Dist.	0.084	-0.105	-0.105	0.084
CO	-0.053	0.042	0.042	-0.053
Dist.	0.053	-0.042	-0.042	0.053
CO	-0.021	0.026	0.026	-0.021
Dist.	0.021	-0.026	-0.026	0.021
CO	-0.013	0.010	0.010	-0.013
Dist.	0.013	-0.010	-0.010	0.013
ΣM	0	601	-601	0

Car Fully Loaded Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

Car Fully Loaded Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
50	1	44.93	47.84648	1.06491
50	5	40.93	198.532	24.2526
50	9	36.93	290.9231	70.8992
50	13	32.93	334.1211	131.903
50	17	28.93	337.2274	198.163
50	21	24.93	309.3435	260.578
50	25	20.93	259.5706	310.046
50	29	16.93	197.0103	337.466
50	33	12.93	130.764	333.736
50	37	8.93	69.93293	289.756
50	41	4.93	23.61864	196.423
50	45	0.93	0.922477	44.636

228	228
178	712
128	512
79	316
28	112
-22	-88
-72	-288
-122	-488
-172	-688
-222	-888
-272	-1088
-322	-1288
-372	-1488

FEM (AB) -2199.812682 FEM (BA) 2198.92

Car Fully Loaded Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
50	3.07	42.86	133.6657	9.57428
50	7.07	38.86	253.0475	46.0382
50	11.07	34.86	318.8449	101.251
50	15.07	30.86	340.1592	166.111
50	19.07	26.86	326.0918	231.518
50	23.07	22.86	285.744	288.369
50	27.07	18.86	228.2174	327.563
50	31.07	14.86	162.6133	340
50	35.07	10.86	98.03305	316.576
50	39.07	6.86	43.57815	248.192
50	43.07	2.86	8.349953	125.746

346	1384
296	1184
246	984
196	784
146	584
96	384
46	184
-4	-16
-54	-216
-104	-416
-154	-616
-204	-583.44

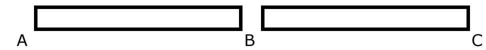
FEM (BC) -2198.344907 FEM (CB) 2200.94

Car Fully Loaded Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-2199.813	2198.925	-2198.345	2200.939
Dist.	2199.813	-0.290	-0.290	-2200.939
CO	-0.145	1099.906	-1100.470	-0.145
Dist.	0.145	0.282	0.282	0.145
ΣM	0	3299	-3299	0

Car Max - Moment Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

Car Max - Moment Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
50	1	44.93	47.84648	1.06491
50	5	40.93	198.532	24.2526

FEM (AB) -246.3785163

FEM (BA) 25.3175

Car Max - Moment Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
50	44.93	1	1.064912	47.8465
50	40.93	5	24.25263	198.532

FEM (BC) -25.31754297

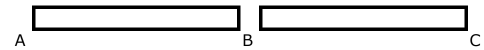
FEM (CB) 246.379

Car Max - Moment Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-246.379	25.318	-25.318	246.379
Dist.	246.379	0.000	0.000	-246.379
CO	0.000	123.189	-123.189	0.000
Dist.	0.000	0.000	0.000	0.000
ΣM	0	149	-149	0

Car Max Shear Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

Car Max Shear Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
50	44.93	1	1.064912	47.8465
50	40.93	5	24.25263	198.532

FEM (AB) -25.31754297 FEM (BA) 246.379

Car Max Shear Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
50	1	44.93	47.84648	1.06491
50	5	40.93	198.532	24.2526

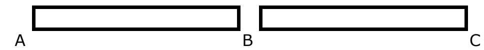
FEM (BC) -246.3785163 FEM (CB) 25.3175

Car Max Shear Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-25.318	246.379	-246.379	25.318
Dist.	25.318	0.000	0.000	-25.318
CO	0.000	12.659	-12.659	0.000
Dist.	0.000	0.000	0.000	0.000
ΣM	0	259	-259	0

Car Max + Moment Case



Distributed Loads

Load	Length	Moment
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00

Car Max + Moment Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
50	1	44.93	47.84648	1.06491
50	5	40.93	198.532	24.2526
50	9	36.93	290.9231	70.8992
50	13	32.93	334.1211	131.903
50	17	28.93	337.2274	198.163
50	21	24.93	309.3435	260.578
50	25	20.93	259.5706	310.046
50	29	16.93	197.0103	337.466
50	33	12.93	130.764	333.736
50	37	8.93	69.93293	289.756
50	41	4.93	23.61864	196.423
50	45	0.93	0.922477	44.636

FEM (AB) -2199.812682

FEM (BA) 2198.92

Car Max + Moment Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
0	0	0	0	0

FEM (BC) 0

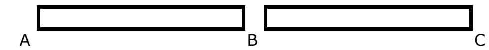
FEM (CB) 0

Car Max + Moment Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-2199.813	2198.925	0.000	0.000
Dist.	2199.813	-1099.462	-1099.462	0.000
CO	-549.731	1099.906	0.000	-549.731
Dist.	549.731	-549.953	-549.953	549.731
CO	-274.977	274.866	274.866	-274.977
Dist.	274.977	-274.866	-274.866	274.977
CO	-137.433	137.488	137.488	-137.433
Dist.	137.433	-137.488	-137.488	137.433
CO	-68.744	68.716	68.716	-68.744
Dist.	68.744	-68.716	-68.716	68.744
CO	-34.358	34.372	34.372	-34.358
Dist.	34.358	-34.372	-34.372	34.358
CO	-17.186	17.179	17.179	-17.186
Dist.	17.186	-17.179	-17.179	17.186
CO	-8.590	8.593	8.593	-8.590
Dist.	8.590	-8.593	-8.593	8.590
CO	-4.297	4.295	4.295	-4.297
Dist.	4.297	-4.295	-4.295	4.297
CO	-2.147	2.148	2.148	-2.147
Dist.	2.147	-2.148	-2.148	2.147
CO	-1.074	1.074	1.074	-1.074
Dist.	1.074	-1.074	-1.074	1.074
CO	-0.537	0.537	0.537	-0.537
Dist.	0.537	-0.537	-0.537	0.537
CO	-0.269	0.268	0.268	-0.269
Dist.	0.269	-0.268	-0.268	0.269
CO	-0.134	0.134	0.134	-0.134
Dist.	0.134	-0.134	-0.134	0.134
CO	-0.067	0.067	0.067	-0.067
Dist.	0.067	-0.067	-0.067	0.067
CO	-0.034	0.034	0.034	-0.034
Dist.	0.034	-0.034	-0.034	0.034
ΣM	0	1649	-1649	0

Dead Load - No Girders Case



Distributed Loads

Load	Length	Moment	Load	Length	Moment	
6.459	45.93	1135.47	6.459	45.93	1135.47	Deck
0.856	45.93	150.48	0.856	45.93	150.48	Sidewalk (2)
1.256	45.93	220.80	1.256	45.93	220.80	Barrier (2)
0.563	45.93	98.97	0.563	45.93	98.97	Asphalt

Dead Load - No Girders Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
0	0	0	0	0

FEM (AB) -1605.730483

FEM (BA) 1605.73

Dead Load - No Girders Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
0	0	0	0	0

FEM (BC) -1605.730483

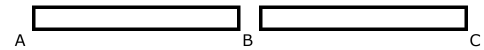
FEM (CB) 1605.73

Dead Load - No Girders Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-1605.730	1605.730	-1605.730	1605.730
Dist.	1605.730	0.000	0.000	-1605.730
CO	0.000	802.865	-802.865	0.000
Dist.	0.000	0.000	0.000	0.000
ΣM	0	2409	-2409	0

Live Load - Both Spans Case



Distributed Loads

Load	Length	Moment
1.28	45.93	225.02
0	45.93	0.00
0	45.93	0.00
0	45.93	0.00

Load	Length	Moment
1.28	45.93	225.02
0	45.93	0.00
0	45.93	0.00
0	45.93	0.00

Live Load - Both Spans Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
0	0	0	0	0

FEM (AB) -225.020256 FEM (BA) 225.02

Live Load - Both Spans Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
0	0	0	0	0

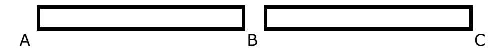
FEM (BC) -225.020256 FEM (CB) 225.02

Live Load - Both Spans Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-225.020	225.020	-225.020	225.020
Dist.	225.020	0.000	0.000	-225.020
CO	0.000	112.510	-112.510	0.000
Dist.	0.000	0.000	0.000	0.000
ΣM	0	338	-338	0

Live Load - 1 Span Case



Distributed Loads

Load	Length	Moment
1.28	45.93	225.02
0	45.93	0.00
0	45.93	0.00
0	45.93	0.00

Load	Length	Moment
0	45.93	0.00
0	45.93	0.00
0	45.93	0.00
0	45.93	0.00

Live Load - 1 Span Case

Span AB

Point Load	Distance from A	Distance from B	FEM (AB)	FEB (BA)
0	0	0	0	0

FEM (AB) -225.020256 FEM (BA) 225.02

Live Load - 1 Span Case

Span BC

Point Load	Distance from B	Distance from C	FEM (BC)	FEB (CB)
0	0	0	0	0

FEM (BC) 0 FEM (CB) 0

Live Load - 1 Span Case

Moment Distribution

Joint	A	B		C
Member	AB	BA	BC	CB
DF	1	0.5	0.5	1
FEM	-225.020	225.020	0.000	0.000
Dist.	225.020	-112.510	-112.510	0.000
CO	-56.255	112.510	0.000	-56.255
Dist.	56.255	-56.255	-56.255	56.255
CO	-28.128	28.128	28.128	-28.128
Dist.	28.128	-28.128	-28.128	28.128
CO	-14.064	14.064	14.064	-14.064
Dist.	14.064	-14.064	-14.064	14.064
CO	-7.032	7.032	7.032	-7.032
Dist.	7.032	-7.032	-7.032	7.032
CO	-3.516	3.516	3.516	-3.516
Dist.	3.516	-3.516	-3.516	3.516
CO	-1.758	1.758	1.758	-1.758
Dist.	1.758	-1.758	-1.758	1.758
CO	-0.879	0.879	0.879	-0.879
Dist.	0.879	-0.879	-0.879	0.879
CO	-0.439	0.439	0.439	-0.439
Dist.	0.439	-0.439	-0.439	0.439
CO	-0.220	0.220	0.220	-0.220
Dist.	0.220	-0.220	-0.220	0.220
CO	-0.110	0.110	0.110	-0.110
Dist.	0.110	-0.110	-0.110	0.110
CO	-0.055	0.055	0.055	-0.055
Dist.	0.055	-0.055	-0.055	0.055
CO	-0.027	0.027	0.027	-0.027
Dist.	0.027	-0.027	-0.027	0.027
CO	-0.014	0.014	0.014	-0.014
Dist.	0.014	-0.014	-0.014	0.014
CO	-0.007	0.007	0.007	-0.007
Dist.	0.007	-0.007	-0.007	0.007
CO	-0.003	0.003	0.003	-0.003
Dist.	0.003	-0.003	-0.003	0.003
ΣM	0	169	-169	0

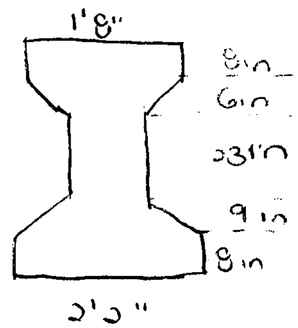
Combined Moments

Location (ft)	*Moment (ft-kips)*	*Notes/Combined Effects*
	LL Entire	
17.13	188.43	
45.93	-344.37	
74.73	188.43	
	LL Half	
20.24	263.12	
45.93	-160.77	
	Truck Max Shear	
16.93	474	Close enough to say max + moment = 662.43
45.93	-996	Max - Moment = -1340.37
	Truck Max + Moment	
15	865	
20	820	Max + Moment =1083.12
	Truck Max - Moment	
45.93	-780	-1124.37
	Truck Fully Loaded	
45.93	-1177	-1521.37
	Car Max + Moment	
21	2488	Close enough to say max + moment =2751.12
45.93	-1632	-1792.77
	Car Fully Loaded	
45.93	-1488	-1832.37

Prestressed Concrete - Girder

Prestressed Concrete Girder Design

AASHTO Type IV



9 girders spaced 7ft

Step 1: Determine DF

IMPACT FACTOR

$I = 50/(L+125) = 50/(91.86+125) = 0.23$

use $I = 1.23$

Tbl 3-6 (Bridge Eng)

$DF = S/5.5 = 7/5.5 = 1.27$

Step 2: Calculate the Moment of Inertia of Composite Section

Using Tbl 3-24

bottom face to COG slab

Element	$A(in^2)$	$Y(in)$	$Ay(in^3)$	$Ay^2(in^4)$	$I_o(in^4)$
Slab	576	59	33984	2005056	3072
Girder	789	24.73	19511.97	482531	260730
Totals	1365		53496	2487587	263802

$b_e = 1/4 L = 1/4(91.86ft) = 22.97ft = 275.58in$

$2[S/2 + 6t_s) = 2(7/2 + 6(8/12)) = 15ft = 180in$

$2(S/2 - 6/12) = 2(3.5 - 6/12) = 6ft = 72in$

$I_0 = 1/12\,bh^3$ for slab

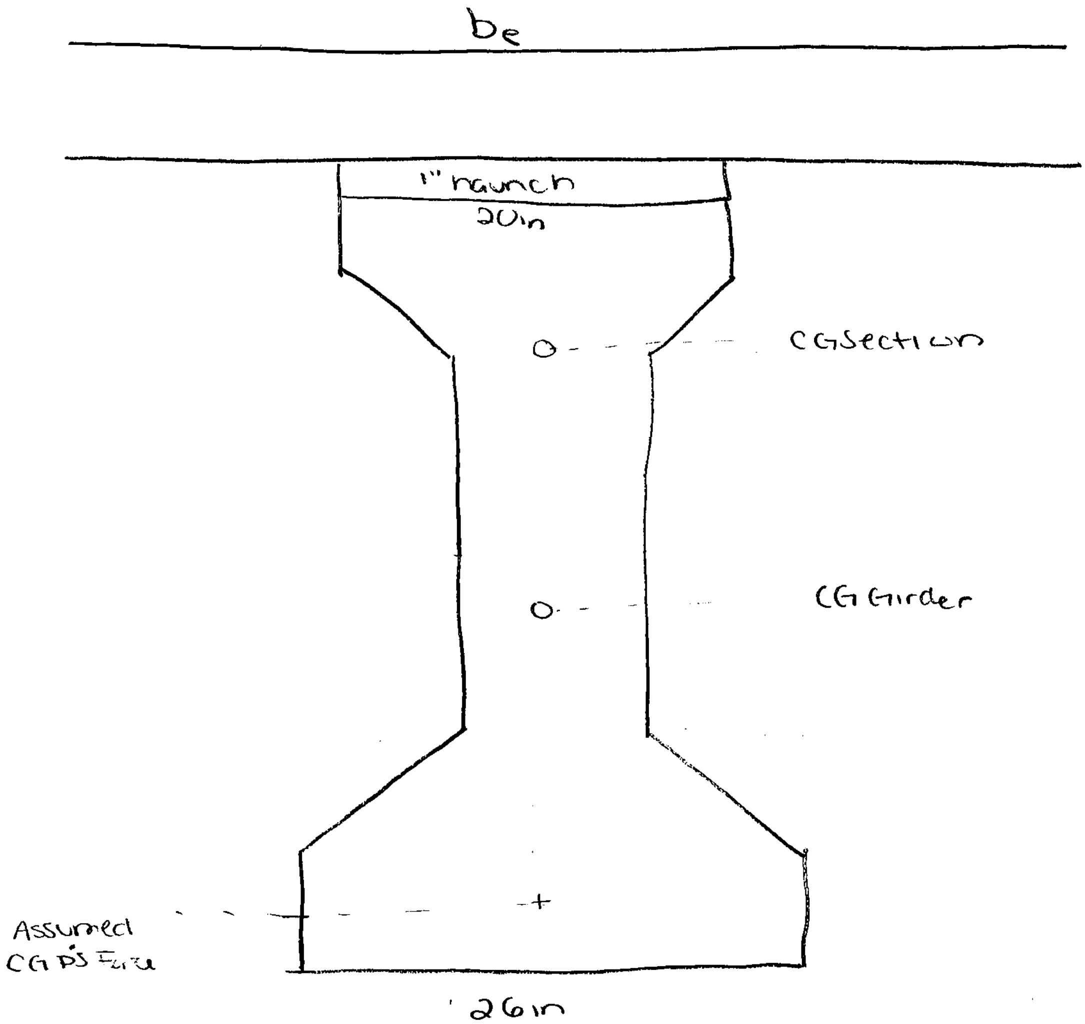
$A_{slab} = b_e t_s = 72in(8in) = 576in$

$I_z = \Sigma I_0 + \Sigma AY^2 - 2751389$

$Y' = \frac{\Sigma AY}{\Sigma A} = 53496/1365 = 39.2$

$I = I_z - \Sigma A(\Sigma Y')^2 = 2751389 - (1365)(39.2)^2$

$= 653875 in^4$


be
1" haunch
20in
CGSection
CG Girder
Assumed
CG PS Force
26in

Step3: Calculate DL on Prestressed Girder

$DL_{Slab} = S\, t_s\, w_c$

$= 7\,(8/12)\,(0.150) = 0.7\ k/ft$

$DL_{Haunch} = w_h\, t_h\, w_c$

$= 20/12\,(1/12)(0.150) = 0.0208\ k/ft$

$DL_{Girder} = A_g W_c$

$= \left(\frac{789}{144}\right)(0.150) = 0.822\ k/ft$

$DL_{barrier} = \frac{2\, A_b w_c}{\#\ Girders}$

$= \frac{2\,(479.19/144)(0.150)}{9} = 0.111\ k/ft$

$DL_{wearing} = \frac{t_p\, w_p\, w_p}{\#\ Girders}$

$= \frac{1.5/12\,(47.39)(0.140)}{9} = 0.092\ k/ft$

$DL_{Sidewalk} = \frac{t_s\, w_s\, w_c}{\#\ Girders}$

$= \frac{6/12\,(6)\,(0.150)}{9} = 0.050\ k/ft$

Total DL
$= 1.796\ k/ft$

Step4: Compute DL Moments

$M = wL^2/8$

$M_{Slab} = \frac{0.7\ k/ft\,(91.86 ft)^2}{8} = 738.35\ ft\ kips$

$M_{Haunch} = \frac{0.021\,(91.86)^2}{8} = 22.15\ ft\ kips$

$$M_{girder} = \frac{0.822(91.86)^2}{8} = 867.03 \text{ ft kips}$$

$$M_{barrier} = \frac{0.111(91.86)^2}{8} = 117.08 \text{ ft kips}$$

$$M_{wearing} = \frac{0.092(91.86)^2}{8} = 97.04 \text{ ft kips}$$

$$M_{sidewalk} = \frac{0.050(91.86)^2}{8} = 52.74 \text{ ft kips}$$

Total Moment = 1894.39 ft kips

Total Factored Moment $= 1.2(M_T) = 1.2(1894.39)$

$= 2273.27$ ft kips

≈ 2273 ft kips

Step 5: Calculate LL + Impact Moment

- Max LL Moment computed in moment calcs

$M_{LL} = 2751 \text{ ft-kips } (1.6)^{\leftarrow \text{Factor}} = 4402 \text{ ft kips}$

$M_{LL+I} = M_{LL}(DF)I$

$= 4402(1.23)(1.27) = 6876 \text{ ft kips}$

Step 6: Calculate Stresses at Top Fiber of Girder

	Noncomposite Type IV Girder	Composite 8" Slab & Type IV Girder
I	260730	653875
y_t	29.27	$(54-39.2)=14.8$
y_b	24.73	39.2

Stresses at the top fiber of the girder is calculated using $f = Mc/I$

	Element	Equation	Top Fiber
Non composite	Slab	$\frac{738.35(12)(29.27)}{260730}$	0.995 ksi
	Type IV Girder	$\frac{861.03(12)(29.27)}{260730}$	1.168 ksi
Composite	LL+I	$\frac{6876(12)(14.8)}{653875}$	1.868 ksi
	Barrier	$\frac{117.08(12)(14.8)}{653875}$	0.032 ksi
	Wear Course	$\frac{97.04(12)(14.8)}{653875}$	0.026 ksi
	Sidewalk	$\frac{52.74(12)(14.8)}{653875}$	0.014 ksi

$f_{top} = 4.103$ ksi

Step 7: Calculate Stresses at Bottom Fiber of Girder

	Element	Equation	Bottom Fiber
noncomposite	Slab	$\frac{738.35(12)(24.73)}{260730}$	0.840 ksi
	Type IV Girder	$\frac{867.03(12)(24.73)}{260730}$	0.987 ksi
composite	LL + I	$\frac{6876(12)(39.2)}{653875}$	4.947 ksi
	Barrier	$\frac{117.08(12)(39.2)}{653875}$	0.084 ksi
	Wear Course	$\frac{97.04(12)(39.2)}{653875}$	0.070 ksi
	Sidewalk	$\frac{52.74(12)(39.2)}{653875}$	0.038 ksi

$f_{bot} = 6.966\,ksi$

Step 8: Calculate Initial Prestressing Force

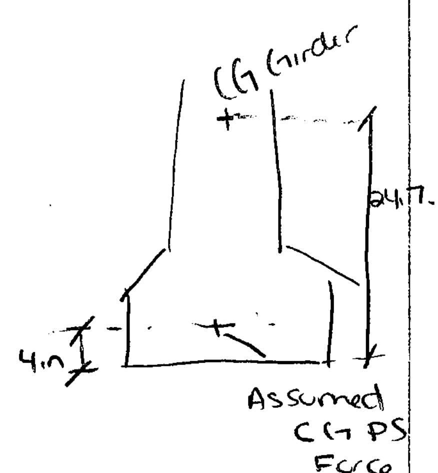
$e = 20.73\,in$

$r^2 = \frac{I}{A}$

$= \frac{260730}{789} = 330.46\,in^2$

Calculate Effective prestressing force

$$C = P_f = \frac{f_{bot} A}{1 + \frac{e y_b}{r^2}}$$



$$= \frac{6.966(789)}{1 + \frac{(20.73)(24.73)}{(330.46)}} = 2161.66\,K$$

Calculate effective Stress

f_e = Allowable Initial Stress − Assumed Losses

AASHTO 9.15.1 $= 0.75 f'_s = 0.75\,(270\text{ ksi}) = 202.50\text{ ksi}$

Table 3.25 (Bridge Engineering)

Assumed Loss = 35 ksi (pretensioned)

$$f_e = 202.50 - 35 = 167.50\text{ ksi}$$

Area steel

$$A_s = P_f / f_e = 2161.66 / 167.50 = 12.9\text{ in}^2$$

Assume losses due to elastic shortening

Losses After Transfer = 35 ksi − 13 ksi = 22 ksi

$$P_i = P_f + (\text{Losses After Transfer})(A_s)$$

$$P_i = 2161.66 + (22)(12.9) = 2445.46^k$$

Step 9: Calculate Fiber Stresses in Beam

Top fiber

$$1 + \frac{e y_t}{r^2}$$

$$1 + \frac{(20.73)(-29.27)}{330.46} = -0.84$$

$$f_{top} = -0.84\, P/A \pm f_{time}$$

Bottom Fiber

$$1 + \frac{e y_b}{r^2}$$

$$1 + \frac{(20.73)(24.73)}{330.46} = 2.55$$

$$f_{bot} = 2.55\, P/A \pm f_{time}$$

	Time of Stress	f_{time}	Equation	Stress
Top Fiber	At time of prestressing	Girder	$(-0.84)\frac{-2445.46}{789} - 1.68$	0.923 ksi (T)
	At time slab is placed	Girder + Slab	$(-0.84) - \frac{2162}{789} - 2.136$	0.166 ksi (T)
	At design load	ALL	$(-0.84)\frac{-2162}{789} - 4.103$	−1.801 ksi (C)
Bottom Fiber	At time of prestressing	Girder	$(2.55)\frac{-2445.46}{789} + 0.987$	−6.917 ksi (C)
	At time slab is placed	Girder + Slab	$(2.55)\frac{-2162}{789} + 1.827$	−5.160 ksi (C)
	At design load	ALL	$(2.55)\frac{-2162}{789} + 6.966$	−0.021 ksi (C)

Step 10: Determine & Check Required Concrete Strength

$0.60 f'_c$ $\qquad \frac{\text{Compressive Stress}}{0.60} = f'_c = \frac{-6.917\text{ ksi}}{0.60}$

$= 11.5$ ksi

Minimum Strength Concrete = 12 ksi

AASHTO 9.15.2.1 → Allowable temporary tensile stress for pretensioned members prior to creep shrinkage is 200 psi

or $\quad 3\sqrt{f'_c} = 3\sqrt{12000} = 329$ psi so use 200 as max

0.2 ksi < 0.923 ksi   Not okay!

In excel use different # of girders to make this work

Bonded Reinforcement may allow

$7.5\sqrt{12{,}000} = 0.821$

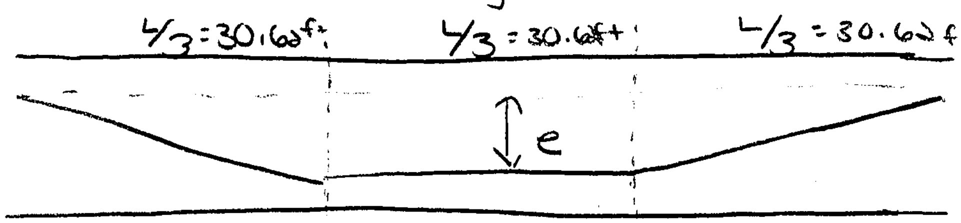
still < 0.923ksi

AASHTO 9.15.2.1 the allowable service load tensile stress for pretensioned members after losses have occurred:

$6\sqrt{f'c} = 6\sqrt{12000} = 657\text{ psi}$

does not apply because no tension at service loading

Step 11: Define Draping of Tendons



Dead Load Ratios

$M_x = \frac{wx}{2}(L-x)$ $x = L/3$

$$M_x = \frac{w\,L/3}{2}\left(L - L/3\right) = wL^2/9$$

$M_{mid} = \frac{wL^2}{8}$ $\text{ratio} = \frac{wL^2/9}{wL^2/8} = 8/9$

Assume similiar LL ratio

use 0.89 as Multiplier

Step 12: Fiber stresses at Third Points of Beam

Top Fiber

At time of prestressing	$0.84\left(2446/789\right) - 0.89(1.69)$	1.109 ksi (T)
At time slab is placed	$0.84\left(2162/789\right) - 0.89(2.136)$	0.401 ksi (T)
At design load	$0.84\left(2162/789\right) - 0.89(4.103)$	-1.350 ksi (C)

Bottom Fiber

At time of prestressing	$-2.55\left(2446/789\right) + 0.89(0.987)$	-7.027 (C)
At time slab is placed	$-2.55\left(2162/789\right) + 0.89(1.827)$	-5.361 (C)
At design load	$-2.55\left(2162/789\right) + 0.89(6.966)$	-0.788 (C)

Step 13: Check Req. Concrete Strength

$$f'_c = \frac{\text{Comp stress}}{0.60} = \frac{7.027}{0.60} = 11.7 \text{ ksi} < 12 \text{ ksi} \checkmark$$

AASHTO 9.15.2.1 temp Tensile

0.200 ksi < 1.109 ksi

bonded reinforcement $7.5\sqrt{f'_c} = 7.5\sqrt{12{,}000}$

0.822 ksi

still < 1.109 ksi

again excel changes to spacing & stuff

Determine # of conventional reinforcing bars required

$$\text{Distance to N.A} = \frac{54\,in\,(1109\,psi)}{1109\,psi + 7027\,psi} = 7.36\,in$$

$$\text{Tensile force} = 0.5\,f_t\,(A_c)$$
$$= 0.5(1109)(7.36)(20) = 81.6^k$$

For 60 ksi steel

$$A = \frac{81.6^k}{60\,ksi} = 1.36\,in^2$$

Use 5 #5 bars $\quad A = 1.55\,in^2$

Step 14: Check Flexural Strength

Compute prestressing steel ratio defined as

$$p = \frac{A_s}{b\,d} = \frac{12.9\,in^2}{72(59)} = 0.00304$$

Compute average stress in prestressing steel at ultimate load for bonded members

$$f_{su} = f_s' \left(1 - \frac{\lambda}{\beta_1}\left(\frac{p\,f_s'}{f'c}\right)\right)$$

$$= 270\left[1 - \frac{0.28 \leftarrow \text{constant}}{0.85 - 0.05(12-4)}\left(\frac{0.00304\,(270)}{12}\right)\right]$$

$$= 246\,ksi$$

$$a = \frac{A_s f_{su}}{0.85 f'_c b} = \frac{12.9(246)}{0.85(12)(72)} = 4.32\,in$$

Thickness of Slab = 8 in $> a = 4.32$ in

NA is located in the flange meaning that we design for a rectangular section

For rect. section check that

$$\frac{p f_{su}}{f'_c} = \frac{0.00304(246)}{12} = 0.062$$

$$0.36 B_1 = 0.36(0.85 - 0.05(12-4)) = 0.162 > 0.062 \checkmark$$

$$\phi M_n = \phi A_s f_{su}(d - a/2) = 0.9(12.9)(246)\left(59 - \frac{4.32}{2}\right) \times 1/12$$

$$\phi M_n = 13528.2 \text{ ft kips}$$

$$M = \gamma(B_{DL} M_{DL} + B_{LL+I} M_{LL+I})$$

$$= 1.3(2273 + 6876)$$

$$= 11893.7 \text{ ft kips}$$

$$\phi M_n > M \checkmark$$

Prestressed Concrete Girder
12

Girder Height	54	inches
Span	45.93	feet
Spacing C to C	5	ft
Slab Thickness	8	in
Haunch Width / Top Flange	20	in

Number of Girders	12

Step 1: Determine Factors

Impact Factor	1.292517405	
DF	1.27	Table 3-6 Bridge Engineering

Step 2: Calculate the Moment of Interat of Composite Section

Calculate be Values	137.79	156	48
Use be	48		

	A	Y	AY	AY^2	Io
Slab	384	58	22272	1291776	2048
Girder	789	24.73	19512	482531	260730
TOTALS	1173	82.73	41784	1774307	262778

Iz	2037085.018
Y'	35.6214578
I	548679.094

Step 3: Calculate DL on Prestressed Girder

Slab	0.5	k/ft
Haunch	0.020833333	k/ft
Girder	0.821875	k/ft
Barrier	0.083192708	k/ft
Wearing	0.069110417	k/ft
Sidewalk	0.0375	k/ft
TOTAL	1.532511458	k/ft

Step 4: Compute DL Moments

Slab	131.8478063	ft-kips
Haunch	5.493658594	ft-kips
Girder	216.7248315	ft-kips
Barrier	21.93755218	ft-kips
Wearing	18.22411365	ft-kips
Sidewalk	9.888585469	ft-kips
TOTAL	404.1165477	ft-kips
Factored	484.9398572	ft-kips

Step 5: Calcualted LL + Impact Moment

MLL	590	ft-kips
Factored	944	ft-kips
M LL+I	1549.573266	ft-kips

Assume Max LL Moment is from 2 trucks in middle of span (see supplemental hand calcs)

Step 6 and 7: Calculate Stresses at Top and Bottom Fibers of Girder

	Non Composit	Compoiste
I	260730	548679.094
Yt	29.27	18.3785422
Yb	24.73	35.6214578

Top Fibers

Non Composite		
Slab	0.177617549	ksi
Girder	0.291958846	ksi

Composite

LL + I	0.622853642	ksi
Barrier	0.008817837	ksi
Wearing	0.007325214	ksi
Sidewalk	0.003974734	ksi

TOTAL	1.112547821	ksi

Bottom Fibers

Non Composite		
Slab	0.150067714	ksi
Girder	0.246673804	ksi

Composite

LL + I	1.207220599	ksi
Barrier	0.017090812	ksi
Wearing	0.014197796	ksi
Sidewalk	0.007703866	ksi

TOTAL	1.64295459	ksi

Step 8: Calculate Initial Prestressing Force

e	20.73	in	change only if girder changes
r^2	330.4562738	in^2	
C	508.0807607	kips	
fe	167.5	ksi	
As	3.033317974	in^2	
Pi	574.8137561	kips	

Step 9: Calculate Fiber Stresses in Beam

Top Fiber	-0.83614943
Bottom Fiber	2.551348668

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.317204902	-1.61207184
At time slab is places	0.068866496	-1.24621307
At design load	-0.57410493	0

Step 10: Determine and Check Required Concrete Strength

f'c	2.686786395	ksi
Minimum Strength Requirement	3	ksi
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	317.2049019	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

Step 11: Define Draping Tendons (1/3 L and 2/3L)

Dead Load Ratios	0.888888889

Step 12: Fiber Stress at Third Points of Beam

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.349644774	-1.63948004
At time slab is places	0.121041651	-1.29029546
At design load	-0.45048851	-0.18255051

Step 13: Check Required Concrete Strength

f'c	2.732466729	ksi
Strength	3	ksi
Check	TRUE	
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	349.6447736	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

Determine Number of Conventional Reinforceing Bars Required			
Distance to NA	9.492022658	inches	
Area of Concrete	189.8404532	in^2	(if NA is in the top flange)
Tensile Force	33.18836114	kips	
As	0.553139352	in^2	
USE		bars	

Step 14: Check Flexural Strength

p	0.001089554	
fsu	261.7629727	ksi
a	6.487012502	
Check if a in Slab	TRUE	
pfsu/f'c	0.095068287	
Check	TRUE	
ϕMn	39844.10881	ft-kips
M	2644.86706	
Check	TRUE	

Prestressed Concrete Girder
10

Girder Height	54	inches
Span	45.93	feet
Spacing C to C	6	ft
Slab Thickness	8	in
Haunch Width / Top Flange	20	in

# of Girders	10

Step 1: Determine Factors

Impact Factor	1.292517405	
DF	1.27	Table 3-6 Bridge Engineering

Step 2: Calculate the Moment of Inertia of Composite Section

Calculate be Values	137.79	168	60
Use be	60		

	A	Y	AY	AY^2	Io
Slab	480	58	27840	1614720	2560
Girder	789	24.73	19512	482531	260730
TOTALS	1269	82.73	47352	2097251	263290

Iz	2360541.018
Y'	37.31439716
I	593630.8031

Step 3: Calculate DL on Prestressed Girder

Slab	0.6	k/ft
Haunch	0.020833333	k/ft
Girder	0.821875	k/ft
Barrier	0.09983125	k/ft
Wearing	0.0829325	k/ft
Sidewalk	0.0703125	k/ft
TOTAL	1.695784583	k/ft

Step 4: Compute DL Moments

Slab	158.2173675	ft-kips
Haunch	5.493658594	ft-kips
Girder	216.7248315	ft-kips
Barrier	26.32506262	ft-kips
Wearing	21.86893638	ft-kips
Sidewalk	18.54109775	ft-kips
TOTAL	447.1709544	ft-kips
Factored	536.6051452	ft-kips

Step 5: Calcualted LL + Impact Moment

MLL	590	ft-kips
Factored	944	ft-kips
M LL+I	1549.573266	ft-kips

Max LL Moment is from 2 trucks in middle of span (see supplemental hand calcs)

Step 6 and 7: Calculate Stresses at Top and Bottom Fibers of Girder

	Non Composit	Compoiste
I	260730	593630.8031
Yt	29.27	16.68560284
Yb	24.73	37.31439716

Top Fibers

Non Composite		
Slab	0.213141058	ksi
Girder	0.291958846	ksi

Composite

LL + I	0.522659484	ksi
Barrier	0.008879247	ksi
Wearing	0.007376229	ksi
Sidewalk	0.006253774	ksi

TOTAL	1.050268637	ksi

Bottom Fibers

Non Composite		
Slab	0.180081256	ksi
Girder	0.246673804	ksi

Composite

LL + I	1.168835417	ksi
Barrier	0.019856864	ksi
Wearing	0.01649563	ksi
Sidewalk	0.013985458	ksi

TOTAL	1.64592843	ksi

Step 8: Calculate Initial Prestressing Force

e	20.73	in
r^2	330.4562738	in^2
C	509.0004153	kips
fe	167.5	ksi
As	3.038808449	in^2
Pi	575.8542012	kips

Step 9: Calculate Fiber Stresses in Beam

Top Fiber	-0.83614943
Bottom Fiber	2.551348668

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.318307522	-1.61543627
At time slab is places	0.034317598	-1.21917337
At design load	-0.51085114	0

Step 10: Determine and Check Required Concrete Strength

f'c	2.692393784	ksi
Minimum Strength Requirement	3	ksi
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	318.3075223	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

Step 11: Define Draping Tendons (1/3 L and 2/3L)

Dead Load Ratios	0.888888889

Step 12: Fiber Stress at Third Points of Beam

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.350747394	-1.64284447
At time slab is places	0.09043981	-1.2665906
At design load	-0.39415462	-0.18288094

Step 13: Check Required Concrete Strength

f'c	2.738074118	ksi
Strength	3	ksi
Check	TRUE	
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	350.7473941	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

Determine Number of Conventional Reinforceing Bars Required		
Distance to NA	9.500620269	inches
Area of Concrete	190.0124054	in^2
Tensile Force	33.32317801	kips
As	0.5553863	in^2
USE	2 - #5	bars

Step 14: Check Flexural Strength

p	0.000873221	
fsu	263.3984506	ksi
a	5.231486518	
Check if a in Slab	TRUE	
pfsu/f'c	0.076668337	
Check	TRUE	
ϕMn	40617.84806	ft-kips
M	2712.031935	
Check	TRUE	

Prestressed Concrete - Deck

Deck Design: Approximate Method

Girder spacing = 6 ft

# of Girders = 10

Top Cover = 2.5 in

Girder Height = 54 in

Bottom Cover = 1 in

f_y steel = 60 ksi

f'_c conc = 4 ksi

ρ conc = 150 pcf

$\rho_{\text{future wearing}}$ = 30 psf

Deck Thickness

$$\frac{t_D}{L} = \frac{8''}{91.86 \times 12} = 0.00726$$

$$1/20 = 0.05 \qquad 0.00726 < 0.05$$

slab should be prestressed in direction of span

Overhang Thickness

8'' deck + 1'' extra (typical)
= 9''

Parapet

Type F

Mass/unit length = 650 lb/ft

Lc = 235.2 in

W_b = 1 ft - 8½ in

$R_W = 137.22^k$

M_c/l = 17.83 k-ft/ft

H = 42 in

Load Factors

Deck
Girders
Sidewalt
$\Big\}$ 1.25

Asphalt → 1.5

$M = wl^2/c$ → Unfactored Moments

Deck

$w = 8(150)/12 = 100\text{psf}$

$$M = \frac{\frac{100\text{psf}}{1000\text{ lb/kip}}(6\text{ft})^2}{10} = 0.36\text{ k-ft/ft}$$

Asphalt/FWS

$w = 30\text{ psf}$

$$M = \frac{(30\text{psf}/1000)(6\text{ft})^2}{10} = 0.11\text{ k-ft/ft}$$

Dist. From Center Girder to design sect. for neg. moment

$W_{TF} = 20\text{in}$

$1/3\, W_{TF} = 6.67\text{ in} < 15\text{ in}$ ✓

Live Load Effects

Min dist b/t wheels of 2 adjacent trucks = 4 ft

Dynamic Allowance = 33% 3.6.2.1

LF = 1.75 3.4.1

Mult. Pres. Factor = 1.00 3.6.1.1.2

φ = 0.9 (strength) 5.5.4.2
1.0 (extreme) 1.3.2.1

Design for (+) Moment in deck

Factored Loads

Live Load

Table A.4.1

unfactored LL(+) moment / unit width

$= 4.71\ ^{kft}/ft$

Max Factored (+) Moment / unit width

$= 1.75\,(4.71\ kft/ft)$

$= 8.24\ kft/ft$

Dead Load

Deck

$1.25\,(0.36\ k\text{-}ft/ft) \quad = 0.45\ k\text{-}ft/ft$

FWS

$1.5\,(0.11\ k\text{-}ft/ft) = 0.17\ k\text{-}ft/ft$

$DL + LL = 8.24 + 0.45 + 0.17 = 8.85\ ^{kft}/ft$

Positive Moment in Deck Design

$d_e = t - c_b - \tfrac{1}{2} d_b - FWS$

$= 8 - 1 - \tfrac{1}{2}(0.625) - 0.5$

$= 6.19\,in$

$k' = M_u / \phi b d^2 = \dfrac{8.85}{0.9(1)(6.19)^2} = 0.2568$

$\rho = 0.85\left(f'c / f_y\right)\left(1 - \sqrt{1 - \dfrac{2k'}{0.85 f'c}}\right)$

$= 0.0045$

$A_{sreq} = \rho d_e = 0.0045\,(6.19\,in) = 0.028\,in^2/in$

Req. #5 bar spacing w/ $A_b = 0.31\,in^2$

$$0.31\,in^2 / 0.028\,in^2/in = 11.2\,in$$ ← Max spacing
From trial & error
know smaller
spacing needed

Use #5 bars @ 7 in spacing

Check Reinforcement

$$T = 0.31(60) = 18.6^k$$

$$a = 18.6 / 0.85(4)(7) = 0.7815$$

$$\beta_1 = 0.85 \text{ for } f'c = 4\,ksi$$

$$c = 0.7815 / 0.85 = 0.9194$$

Not over reinforced!

$$c/d_e = 0.9194 / 6.19 = 0.149 < 0.42 \checkmark$$
(5.7.3.3.1)

Cracking

$$f_{sA} = Z / (d_c A)^{1/3} \le 0.6 f_y = 36\,ksi$$

$$d_c = C_{bot} + 1/2\, d_b = 1\,in + 0.31\,in$$
$$= 1.31\,in < 2 + 1/2\, d_b$$
$$= 2.16\,in$$

$$A = 2(1.31\,in)(7\,in) = 18.34\,in^2$$

$$z = 130\,k/in$$

$$f_{sA} = 130 / (1.31(18.34))^{1/3} = 45.05\,ksi$$

45.05 ksi > 36 ksi use 36 ksi !

Service Load Stresses

$n = 8$

$DL = 0.61$ k-ft/ft

$LL = 8.24$ k-ft/ft

Total = 8.85 k-ft/ft

Section width = 7 in

Transformed As = As n = 0.31(8) = 2.48 in²

$$2.48(6.19 - y) = 7y(y/2)$$

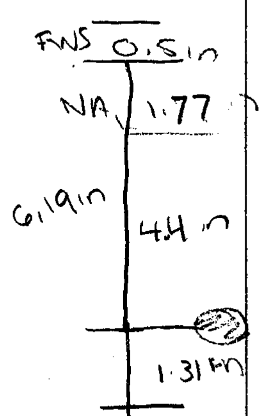
$$y = 1.77 \text{ in}$$

$$I_{transformed} = 2.48(6.19 - 1.77)^2 + 7(1.77^3)/3$$

$$= 61.4 \text{ in}^4$$

$$f_s = \frac{Mc}{I} n = \left(8.85\left(\tfrac{12}{12}\right)(7)(4.4)/61.4\right)8$$

$$= 35.7 \text{ ksi}$$



$f_{sa} = 36 > 35.7$ ✓ ok

Design for Negative Moment at Interior Girders

Live Load

Table A4.1 (interpolation)

$= 3.5 + (2.88 - 3.5)(0.67/3)$

$= 3.36$ k-ft/ft

Max Factored negative moment per unit width

$= 1.75(3.36$ k-ft/ft$)$

$= 5.88$ k-ft/ft

Dead Load

$= 0.45 + 0.162$

$= 0.612$ k-ft/ft

Total $= 5.88$ k-ft/ft $+ 0.612$ k-ft/ft

$= 6.492$ k-ft/ft

Assume # 5 bars

$d_e = 5.19$ in $(8 - 2.5 - 0.5 d_b)$

Req. $A_s = \rho\, d_e$

$K' = M_u / \phi b d^2 = 6.492 / 0.9(1.0)(5.19)^2$

$= 0.2678$

$\rho = 0.85\left(f'c/f_y\right)\left(1 - \sqrt{1 - 2K'/0.85f'c}\right)$

$= 0.85(4/60)\left(1 - \sqrt{1 - 2(0.2678)/0.85(4)}\right)$

$= 0.0047$

$A_s = \rho d_e = 0.0045\,(5.19)$

$= 0.024\ in^2/in$

Req. spacing $= 0.31/0.024 = 12.8\ in$

Use #5 at 12 in spacing

Cracking

$f_{sA} = \dfrac{Z}{(d_c A)^{1/3}} \leq 0.6 f_y = 36\ ksi$

$d_c = 2.31\ in$ (same as before)

$A = 2\,(2.31)(5) = 23.1\ in^2$

$Z = 130^k/in$

$f_{sA} = \dfrac{130}{(2.31\,(23.1))^{1/3}} = 34.53\ ksi$
$< 36\ ksi$ ✓

DL service moment at design section for neg. moment near the middle

$= 0.612\ k\text{-}ft/ft$

LL service moment at design section

$= 4.71\ k\text{-}ft/ft$

$n = 8$

$A_{trans} = 0.31(8) = 2.48\ in^2$ assume 7" section

$2.48\,(5.19 - y) = 7\,y\,(y/2)$

$y = 1.6\ in$

$$I_{transfer} = 2.48(5.19 - 1.6)^2 + \frac{5(1.6)^3}{3}$$

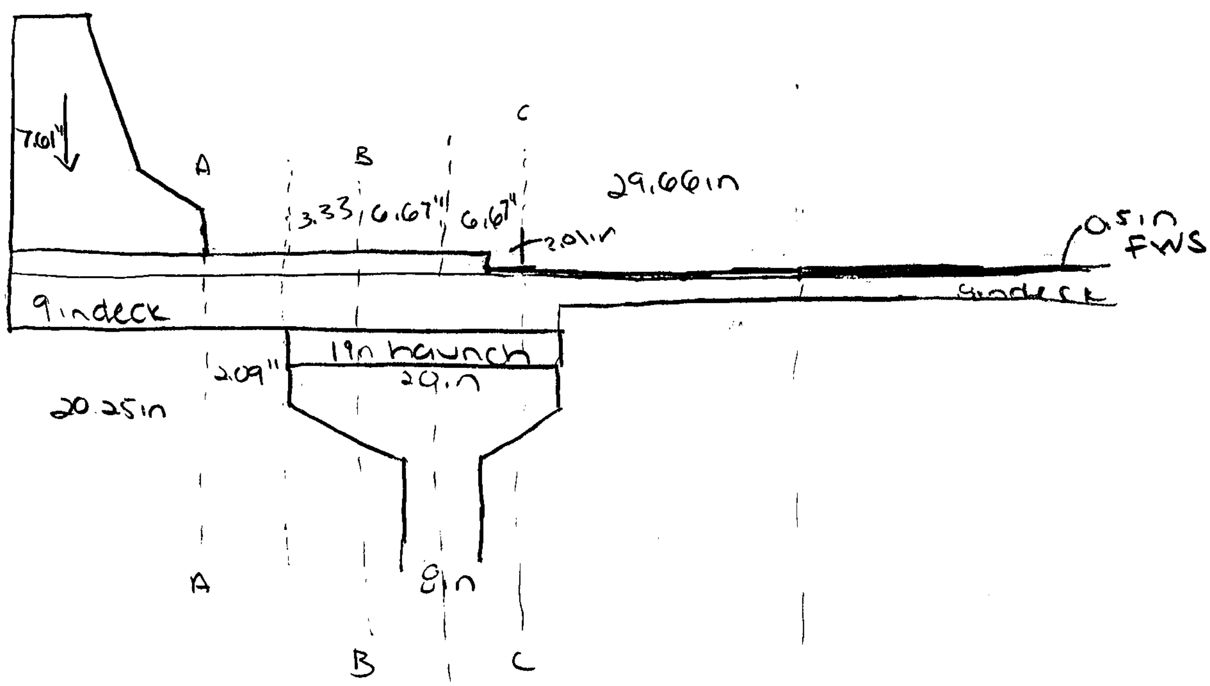
$$= 48.29 \text{ in}^4$$

Total DL + LL Service Load Stresses

$$= \frac{5.322(7)(3.59)}{48.29}(8)$$

$$= 22.18 \text{ ksi} < f_{sa} = 25.79 \text{ ksi} \checkmark$$

Design of the Overhang



Self weight of slab = $\frac{9}{12} \times 150 = 112.5 \text{ lb/ft}^2$

Parapet = 650 lb/ft

FWS = 30 lb/ft^2

Sidewalk = $\frac{6}{12} \times 150 = 75 \text{ lb/ft}^2$

At inside face of parapet (A-A)

$M_C = 17.83 \text{ k-ft/ft}$ @ base of parapet

$$M_{DL} \text{ slab} = -\frac{0.1125\left(\frac{20.25}{12}\right)^2}{2}$$

$$= 0.16 \text{ k-ft/ft}$$

$$M_{DL} \text{ Parapet} = 0.65\left(\frac{20.25-7.61}{12}\right)$$

$$= -0.68 \text{ k-ft/ft}$$

$$M_{DL} \text{ sidewalk} = \frac{0.075\left(\frac{20.25}{12}\right)^2}{2}$$

$$= -0.11 \text{ k-ft/ft}$$

Design Factored Moment

$$= -17.83 - 1.25(0.16 \text{ k-ft/ft} + 0.68 \text{ k-ft/ft} + 0.11 \text{ kft/ft})$$

$$= -19.02 \text{ kft/ft}$$

Design Axial Tensile Force

$$\frac{R_W}{L_C} + 2H = \frac{137.22}{(235.2 + 2(54))}\Big/12$$

$$= 4.8 \text{ k/ft}$$

Assume As req
$= 0.7 \text{ in}^2/\text{ft}$

$$M_n = T(d - a/2) - P(h/2 - a/2)$$

$$T = 0.7(60) = 42 \text{ k/ft}$$

$$C = 42 - 4.8 = 37.2 \text{ k/ft}$$

Reduced cover to 2"

$$a = \frac{C}{12(0.85)(4)} = 0.91 \text{ in}$$

$$M_n = \left[42.0\left(6.69 - \frac{0.91}{2}\right) - 4.8\left(\frac{6.69}{2} - \frac{0.91}{2}\right)\right]\frac{1}{12}$$

$M_n = 20.66 k\text{-}ft/ft = M_r > 19.02$

$$c/d_e = \frac{\left(0.91/0.85\right)}{6.69} = 0.16 < 0.42 \checkmark$$

steel yields before concrete crushes

At design section (B-B)

Collision Moment at Design Section

$$= \frac{M_c L_c}{L_c + 2(0.577)x}$$

$$= \frac{-17.83(235.2)}{235.2 + 2(0.577)(5.42)}$$

$$= -17.72 \text{ k ft/ft}$$

$$M_{DL\,slab} = \frac{0.1125\left[\frac{(22.34 + 3.33)}{12}\right]^2}{2} = 0.26 \text{ k ft/ft}$$

$$M_{DL\,parapet} = \frac{0.65\,(22.34 + 3.33 - 7.61)}{12} = 0.98 \text{ k-ft/ft}$$

$$M_{DL\,sidewalk} = \frac{0.075\left[\frac{22.34 + 3.33}{12}\right]^2}{2} = 0.17 \text{ k ft/ft}$$

Factored Design Moment

$$M = -17.72 - 1.25\,(0.26 + 0.98 + 0.17) = -19.48^k$$

Design tensile force

$$= \frac{R_N}{L_c + 2H + 2(0.577)x}$$

$$= \frac{137.22}{\left[235.2 + 2(54) + 2(0.577)(5.42)\right]\frac{1}{12}}$$

$$= 4.712 \ ^{k}/_{ft}$$

$h = 9 \text{ in}$

$d = 9\text{in} - 2.5\text{ in} - 0.5(0.625) = 6.19$

Assumed As req $= 0.7 \text{ in}^2/\text{ft}$

$T = 0.7(60) = 42 \text{ k/ft}$

$C = 42 - 4.712 = 37.3 \text{ k/ft}$

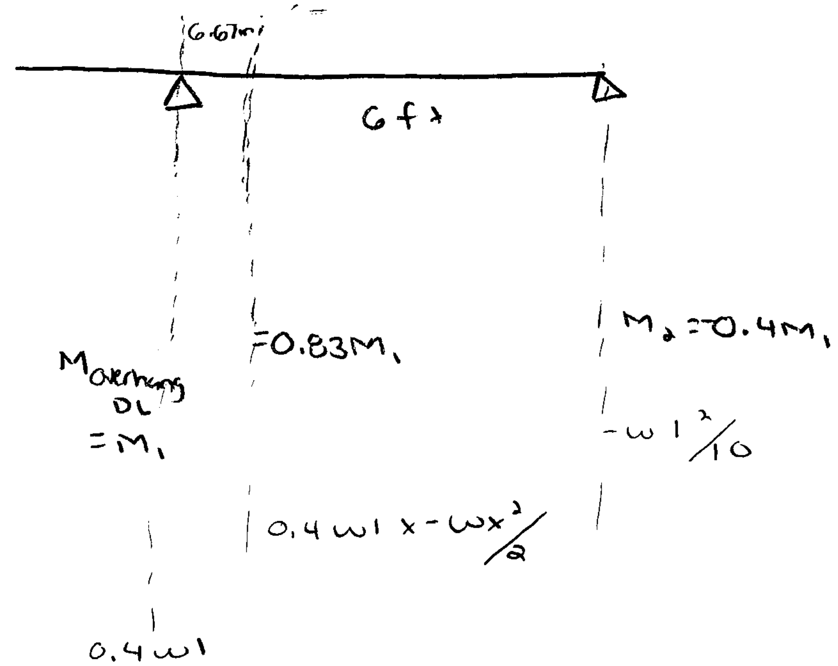
$a = \frac{37.3}{12(0.85)(4)} = 0.91 \text{ in}$

$$M_n = \left[42\left(6.19 - \left(\frac{0.91}{2}\right)\right) - 4.712\left(\frac{6.19}{2} - \frac{0.91}{2}\right)\right]\frac{1}{12}$$

$= 19.022 \text{ k-ft/ft} = M_r > 17.72$ ✓

$$\frac{c}{d_e} = \frac{\left(\frac{0.91}{0.85}\right)}{6.19} = 0.17 < 0.42 \ ✓$$

Check DL + Collision Moment at C-C



DL on 1st span

$M_1 = -17.83$ k-ft/ft

$M_2 = -7.13$ k-ft/ft

Total Collision Moment

$$= -17.83 + 6.67\left(\frac{17.83 + 7.13}{72}\right) = -15.52 \text{ kft/ft}$$

Design Collision Moment

$$= -15.52(235.2) \Big/ (235.2 + 2(0.577)(10 + 2.09 + 6.67))$$

$$= -14.2 \text{ kft/ft}$$

Dead Load Moment at the centerline of the exterior girder

$$M_{DL\,slab} = -0.1125\left(32.34/12\right)^2/2 = -0.41 \text{ k-ft/ft}$$

$$M_{DL\ Parapet} = -0.65(32.34 - 7.61)/12 = -1.75 \text{ kft/ft}$$

M_{DL} sidewalk $= wL^2/2$

Distribute sidewalk ..

$$\frac{0.075(36)}{39.01} = 0.069$$

$$M_{DL} \text{ sidewalk} = \frac{0.069\left(\frac{39.01}{12}\right)^2}{2} = -0.37 \text{ k-ft/ft}$$

$$M_{DL} \text{ FWS} = \frac{0.03\left(3.01/12\right)^2}{2} = -0.0009 \text{ kft/ft}$$

$$M_{FDL} = 1.25(-0.41 - 1.75 - 0.37)$$
$$+ 1.5(-0.0009) = -3.072 \text{ k-ft/ft}$$

$$M_{FDL,O} = 0.83(-3.072) = -2.55 \text{ k-ft/ft}$$

Design Factored Moment due to DL on 1st deck span

$$= 1.25\left(0.1125\left[0.4(6)\left(6.67/12\right) - \left(6.67/12\right)^2/2\right]\right]$$

$$+ 1.5\left[0.03\left[0.4(6\left(6.67/12\right) - \left(6.67/12\right)^2/2\right]\right]$$

$$= 0.219 \text{ k-ft/ft}$$

Total DL + C

$M_{DL+C} = -14.2 - 2.6 + 0.22$

$= -16.53$ k-ft/ft

$d = 5.19$ in

$k' = M / \phi b d^2 = 16.5 / 1.0(1)(5.19)^2 = 0.61$

$\rho = 0.85 \left(4/60\right)\left(1 - \sqrt{1 - 2(0.61)/0.85(4)}\right) = 0.0114$

$A_s = \rho d = 0.0114(5.19) = 0.059$ in²/ft

Detailing of Overhang Reinforcement

Largest $A_s = 0.7$ in²/ft

Provided top reinforcement $= 0.31\left(12/12\right) = 0.31$ in²/ft

bundle #5 bar to each + 1 extra to every other

$= 0.31 \times 5\left(12/24\right) = 0.775$ in²/f > 0.7 in²/ft ✓

Check depth of compression block

$T = 60(0.775) = 46.5$

$a = 46.5 / 0.85(4)(12) = 1.15$ in

$B_1 = 0.85$

$c = 1.15/0.85 = 1.36$

d_e at least slab thickness = 5.19 in

Max $c/d_e = 1.36/5.19 = 0.26 < 0.42$ ✓

Cutoff Length Requirement

$= 15 d_b = 15(0.625) = 9.375$ in

Req. Length past CL of girder $= 25 + 9.375 = 34.375$ in

Development Length

$$\frac{1.25 A_b f_y}{\sqrt{f'_c}} = 11.625$$

$0.4 d_b f_y = 0.4(0.625)(60) = 15$ in

12 in

spacing > 6 in

$L_{Dev} = 0.8(15) = 12$ in

Reqd Length of additional bars past CL ext. girder

$= 6.67 + 12 = 18.67 < 34.375$ ✓

Longitudinal Reinforcement

Bottom Dist. Reinforcement

$$\text{Percent} = 220/\sqrt{S} = 220/\sqrt{6(12)-6.67} = 27.2\%$$

$$\leq 67\% \checkmark$$

Transverse Reinforcement

$$= \#5 \ @ \ 6 \text{ in} = 0.62 \text{ in}^2/\text{ft}$$

Long.

$$= 0.27\,(0.62) = 0.17 \text{ in}^2/\text{ft}$$

Use #5 bars

$$\text{Req spacing} = 0.31/0.17 = 22 \text{ in}$$

Use #5 bars @ 22 in spacing

Top Long. Reinforce

#4 bars @ 12 in spacing

Check Shrinkage & Temp Reinforcement:

$$As_{req} = 0.11\,Ag/F_y = 0.11\,(90)/60 = 0.165/2 \text{ surfaces}$$

$$= 0.0825 < 0.31\left(12/22\right)$$

$$= 0.169 \checkmark$$

Prestressed Concrete Girder
12

Girder Height	54	inches
Span	45.93	feet
Spacing C to C	5	ft
Slab Thickness	8	in
Haunch Width / Top Flange	20	in

Number of Girders	12

Step 1: Determine Factors

Impact Factor	1.292517405	
DF	1.27	Table 3-6 Bridge Engineering

Step 2: Calculate the Moment of Interat of Composite Section

Calculate be Values	137.79	156	48
Use be	48		

	A	Y	AY	AY^2	Io
Slab	384	58	22272	1291776	2048
Girder	789	24.73	19512	482531	260730
TOTALS	1173	82.73	41784	1774307	262778

Iz	2037085.018
Y'	35.6214578
I	548679.094

Step 3: Calculate DL on Prestressed Girder

Slab	0.5	k/ft
Haunch	0.020833333	k/ft
Girder	0.821875	k/ft
Barrier	0.083192708	k/ft
Wearing	0.069110417	k/ft
Sidewalk	0.0375	k/ft
TOTAL	1.532511458	k/ft

Step 4: Compute DL Moments

Slab	131.8478063	ft-kips
Haunch	5.493658594	ft-kips
Girder	216.7248315	ft-kips
Barrier	21.93755218	ft-kips
Wearing	18.22411365	ft-kips
Sidewalk	9.888585469	ft-kips
TOTAL	404.1165477	ft-kips
Factored	484.9398572	ft-kips

Step 5: Calcualted LL + Impact Moment

MLL	590	ft-kips
Factored	944	ft-kips
M LL+I	1549.573266	ft-kips

Assume Max LL Moment is from 2 trucks in middle of span (see supplemental hand calcs)

Step 6 and 7: Calculate Stresses at Top and Bottom Fibers of Girder

	Non Composit	Composiste
I	260730	548679.094
Yt	29.27	18.3785422
Yb	24.73	35.6214578

Top Fibers

Non Composite		
Slab	0.177617549	ksi
Girder	0.291958846	ksi

Composite

LL + I	0.622853642	ksi
Barrier	0.008817837	ksi
Wearing	0.007325214	ksi
Sidewalk	0.003974734	ksi

TOTAL	1.112547821	ksi

Bottom Fibers

Non Composite		
Slab	0.150067714	ksi
Girder	0.246673804	ksi

Composite

LL + I	1.207220599	ksi
Barrier	0.017090812	ksi
Wearing	0.014197796	ksi
Sidewalk	0.007703866	ksi

TOTAL	1.64295459	ksi

Step 8: Calculate Initial Prestressing Force

e	20.73	in	change only if girder changes
r^2	330.4562738	in^2	
C	508.0807607	kips	
fe	167.5	ksi	
As	3.033317974	in^2	
Pi	574.8137561	kips	

Step 9: Calculate Fiber Stresses in Beam

Top Fiber	-0.83614943
Bottom Fiber	2.551348668

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.317204902	-1.61207184
At time slab is places	0.068866496	-1.24621307
At design load	-0.57410493	0

Step 10: Determine and Check Required Concrete Strength

f'c	2.686786395	ksi
Minimum Strength Requirement	3	ksi
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	317.2049019	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

Step 11: Define Draping Tendons (1/3 L and 2/3L)

Dead Load Ratios	0.888888889

Step 12: Fiber Stress at Third Points of Beam

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.349644774	-1.63948004
At time slab is places	0.121041651	-1.29029546
At design load	-0.45048851	-0.18255051

Step 13: Check Required Concrete Strength

f'c	2.732466729	ksi
Strength	3	ksi
Check	TRUE	
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	349.6447736	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

Determine Number of Conventional Reinforceing Bars Required			
Distance to NA	9.492022658	inches	
Area of Concrete	189.8404532	in^2	(if NA is in the top flange)
Tensile Force	33.18836114	kips	
As	0.553139352	in^2	
USE		bars	

Step 14: Check Flexural Strength

p	0.001089554	
fsu	261.7629727	ksi
a	6.487012502	
Check if a in Slab	TRUE	
pfsu/f'c	0.095068287	
Check	TRUE	
ϕMn	39844.10881	ft-kips
M	2644.86706	
Check	TRUE	

Prestressed Concrete Girder
10

Girder Height	54	inches		
Span	45.93	feet		
Spacing C to C	6	ft	# of Girders	10
Slab Thickness	8	in		
Haunch Width / Top Flange	20	in		

Step 1: Determine Factors

Impact Factor	1.292517405	
DF	1.27	Table 3-6 Bridge Engineering

Step 2: Calculate the Moment of Inertia of Composite Section

Calculate be Values	137.79	168	60
Use be	60		

	A	Y	AY	AY^2	Io
Slab	480	58	27840	1614720	2560
Girder	789	24.73	19512	482531	260730
TOTALS	1269	82.73	47352	2097251	263290

Iz	2360541.018
Y'	37.31439716
I	593630.8031

Step 3: Calculate DL on Prestressed Girder

Slab	0.6	k/ft
Haunch	0.020833333	k/ft
Girder	0.821875	k/ft
Barrier	0.09983125	k/ft
Wearing	0.0829325	k/ft
Sidewalk	0.0703125	k/ft
TOTAL	1.695784583	k/ft

Step 4: Compute DL Moments

Slab	158.2173675	ft-kips
Haunch	5.493658594	ft-kips
Girder	216.7248315	ft-kips
Barrier	26.32506262	ft-kips
Wearing	21.86893638	ft-kips
Sidewalk	18.54109775	ft-kips
TOTAL	447.1709544	ft-kips
Factored	536.6051452	ft-kips

Step 5: Calcualted LL + Impact Moment

MLL	590	ft-kips	Max LL Moment is from 2 trucks in middle of span (see supplemental hand calcs)
Factored	944	ft-kips	
M LL+I	1549.573266	ft-kips	

Step 6 and 7: Calculate Stresses at Top and Bottom Fibers of Girder

	Non Composit	Compoiste
I	260730	593630.8031
Yt	29.27	16.68560284
Yb	24.73	37.31439716

Top Fibers

Non Composite		
Slab	0.213141058	ksi
Girder	0.291958846	ksi

Composite

LL + I	0.522659484	ksi
Barrier	0.008879247	ksi
Wearing	0.007376229	ksi
Sidewalk	0.006253774	ksi

TOTAL	1.050268637	ksi

Bottom Fibers

Non Composite		
Slab	0.180081256	ksi
Girder	0.246673804	ksi

Composite

LL + I	1.168835417	ksi
Barrier	0.019856864	ksi
Wearing	0.01649563	ksi
Sidewalk	0.013985458	ksi

TOTAL	1.64592843	ksi

Step 8: Calculate Initial Prestressing Force

e	20.73	in
r^2	330.4562738	in^2
C	509.0004153	kips
fe	167.5	ksi
As	3.038808449	in^2
Pi	575.8542012	kips

Step 9: Calculate Fiber Stresses in Beam

Top Fiber	-0.83614943
Bottom Fiber	2.551348668

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.318307522	-1.61543627
At time slab is places	0.034317598	-1.21917337
At design load	-0.51085114	0

Step 10: Determine and Check Required Concrete Strength

f'c	2.692393784	ksi
Minimum Strength Requirement	3	ksi
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	318.3075223	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

Step 11: Define Draping Tendons (1/3 L and 2/3L)

Dead Load Ratios	0.888888889

Step 12: Fiber Stress at Third Points of Beam

Time Of Stress	Top Fiber	Bottom Fiber
At time of prestressing	0.350747394	-1.64284447
At time slab is places	0.09043981	-1.2665906
At design load	-0.39415462	-0.18288094

Step 13: Check Required Concrete Strength

f'c	2.738074118	ksi
Strength	3	ksi
Check	TRUE	
Allowable Tensile Strength	164.3167673	psi
Max	200	psi
USE	164.3167673	psi
Max Tensile	350.7473941	
Check	FALSE	
Try Bonded Reinforcement	410.7919181	
Check	TRUE	
Allowable Service Load Tensile Strength	328.6335345	
Actual Tensile at deisgn load	0	
Check	TRUE	

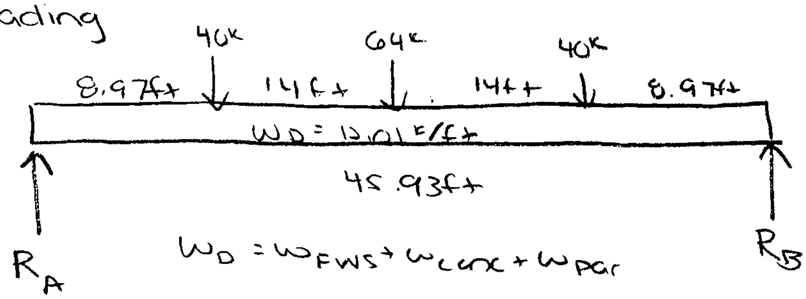
Determine Number of Conventional Reinforceing Bars Required		
Distance to NA	9.500620269	inches
Area of Concrete	190.0124054	in^2
Tensile Force	33.32317801	kips
As	0.5553863	in^2
USE	2 - #5	bars

Step 14: Check Flexural Strength

p	0.000873221	
fsu	263.3984506	ksi
a	5.231486518	
Check if a in Slab	TRUE	
pfsu/f'c	0.076668337	
Check	TRUE	
ϕMn	40617.84806	ft-kips
M	2712.031935	
Check	TRUE	

Prestressed Concrete - Bearings

Loading



$W_D = w_{FWS} + w_{conc} + w_{par} + w_{s'ide}$

$\Sigma F_y = 0 = -12.01^k/ft\,(45.93ft) - 40^k - 64^k - 40^k + R_A + R_B$

$\circlearrowleft + \Sigma M_A = 0 = \dfrac{12.01^k/ft\,(45.93ft)^2}{2} + 40^k(8.97ft) + 64^k(22.97ft)$

$+ 40^k(36.97ft) - R_B(45.93ft)$

$R_B = 347.9^k \quad (1/10) = \text{each girder} = 34.79^k$

$R_A = 347.9^k \quad (1/10) = \text{each girder} = 34.79^k$

$+ \dfrac{\text{weight girder}}{2} = 18.9$

Elastomeric Pad

For 120°F: $6.5 \times 10^{-6} \times 120 \times 45.93\,(12 in/ft)$

$= 0.397\,in = \Delta t$

$\Delta t \leq T/0.5$

$T \leq 0.794$ inches

Try a 1 inch pad w/ two laminations (5/8 in each)

$A_{req} = \dfrac{46.5}{0.5} = 93\ in^2$ DL

$A_{req} = \dfrac{53.7}{0.8} = 67.1\ in^2$ DL+LL

Bottom Flange = 26 inches

Try 4 x 26 A = 104 in²

$f_{DL} = \frac{46.5}{104} = 447$ psi < 500 psi > 200 psi ✓

$f_{DL+LL} = \frac{53.7}{104} = 516$ psi < 800 psi

$S = \frac{4(26)}{2(0.5)(4+26)} = 3.5$

End Compressive strain = from Figure 10.10 Design of Modern Concrete Highway Bridges

prestress $\theta_A = 0.0074$

girder$_{DL}$ $\theta_A = \frac{wL^3}{24EI} = \frac{0.82 \text{ k/ft} (45.93\text{ft})^3}{24(10544)(26.46)} = -0.0119$

slab$_{DL}$ $\theta_A = \frac{wL^3}{24EI} = \frac{0.59(45.93\text{ft})^3}{24(11619)(26.46)} = -0.0077$

Net $\theta_A = 0.0074 - 0.0119 - 0.0077$

$= -0.0122$

offset $= 10(-0.0122) = -0.122$ in

Allowable $= 0.06(1\text{in}) = 0.06$ in

Therefore provided beveled sole plate

Design Shear Force from Pad

$$\text{shear force} = \frac{\text{modulus} \times \text{area} \times \text{movement}}{\text{pad thickness}}$$

60 hardness

0°F temp

Fig 10.13 modulus = 180

Movement = 0.000006 × 120° × 45.93

= 0.033 ft

Creep + shrinkage = 0.03 ft

Total = 0.063 ft

area = 4(26) = 104 in^2

T = 1 in

$$\text{Shear force} = \frac{180(104)(0.063 \times 12)}{1} = 14.2^k$$

Max force before slip = 20% DL

$0.20(46.5) = 9.3^k$

Design abutment for 14.2^k shear force per pad

Bearing pad

1" × 4" × 26" two laminations beveled sole plate

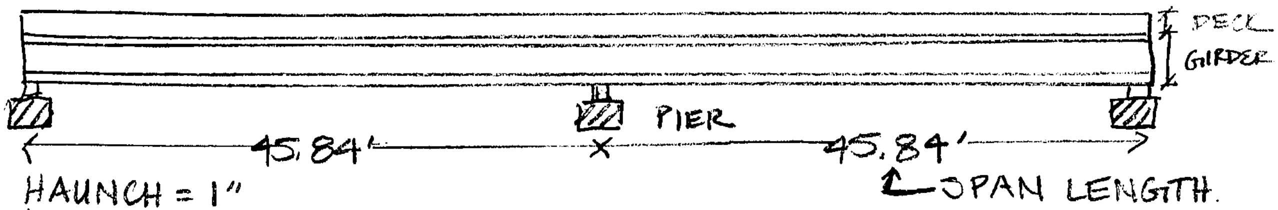
Steel - Girder

GIVEN FROM DECK DESIGN:

DL max(−) = −2411 ft·k
max(+) = 1351 ft·k

LL max(−) = −1832 ft k ← CHECK #
max(+) = 2,751 ft·k

} ALL GREATER THAN MOMENTS CALCULATED IN FOLLOWING DESIGN.



HAUNCH = 1"
ADTT = 251
SLAB = 8" w_c = 0.150 kcf ⇒ 100 psf
wearing surface = 0.5" w_{fws} = 0.140 kcf ⇒ 5.83 psf.
Steel strength = 50 ksi = f_y
concrete " = 6.5 ksi = f'_c

DEAD LOADS: (NON COMPOSITE SECTION)
SLAB = (8/12) (6FT SPACING) (0.150 k/cft) = 600 plf
HAUNCH = (1/12) (1.5in) (0.150) = 18.75 plf
STEEL GIRDER (ASSUME) = = 200 plf
DECK FORMS = 0.015 (6) = 90 plf

908.75 plf (DC 1)

DL ON COMPOSITE SECTION
EACH CONCRETE BARRIER. 0.50 k/ft.
(0.50 k/ft) (2/10 GIRDERS) = 100 plf. (DC 2)

FWS DL
5.83 psf (59.39 FT ROAD WIDTH) * (1/10 GIRDERS) = 34.62 plf (D

LL → uniform distributed loading of 640 plf.
ADJACENT 18 K LOAD CREATES MAX NEG. MOMENT

LIVE LOAD DISTRIBUTION FACTORS:

ASSUME GIRDER:

TOP FLANGE: 1" x 12"
WEB : 1/2 x 40"
BOTTOM FLANGE: 1" x 12"

	A (in^2)	Y(in)	AY (in^3)	AY^2 (in^4)	I_o (in^4)
TOP	12	41.5	498	20667	1
WEB	20	21.0	420	8820	2667
BOTTOM	12	0.5	6	3	1
TOTAL	44		924	29,490	2,669

$$I_z = \Sigma I_o + \Sigma AY^2 = 2669 + 29{,}490 = 32{,}159 \text{ in}^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{924}{44} = 21.00 \text{ in.}$$

$$I = I_z - (\Sigma A)(Y')^2 = 32{,}159 \text{ in}^4 - 44 \text{ in}^2 (21.0 \text{ in})^2 = 12{,}755 \text{ in}^4$$

40×183
$I_x = 13{,}200$

$$e_g = \frac{6.0}{2} + 1.5 + (\underbrace{42}_{\text{DEPTH}} - 21) = 25.5 \text{ in}$$

$$n = \frac{E_s}{E_c} = \frac{29{,}000 \text{ ksi}}{1820\sqrt{f'c} = 6.5} = 6.25 \rightarrow 7$$

$$K_g = n(I + Ae_g^2) = 7(12{,}755 \text{ in}^4 + 44(25.5)^2) =$$

$$289{,}562 \text{ in}^4$$

LL DISTRIBUTION FACTOR FOR POSITIVE BENDING
2 LOADED LANES:

$$DF = 0.075 + \left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0 L t_s^3}\right)^{0.1}$$

$$= 0.075 + \left(\frac{6}{9.5}\right)^{0.6}\left(\frac{6}{54.84}\right)^{0.2}\left(\frac{289{,}562 \text{ in}^4}{12.0(54.84)(8)^3}\right)^{0.1}$$

$$= 0.555$$

ONE LOADED LANE:

$$DF = 0.06 + \left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12 L t_s^3}\right)^{0.1}$$

$$= 0.06 + \left(\frac{6}{14}\right)^{0.4}\left(\frac{6}{54.84}\right)^{0.3}\left(\frac{289{,}562}{12(54.84)(8)^3}\right)^{0.1}$$

$$= 0.421$$

LL DF FOR SHEAR, 2 LOADED LANES:

$$DF = 0.2 + \left(\frac{S}{12}\right) - \left(\frac{S}{35}\right)^2 = 0.2 + \frac{6}{12} - \left(\frac{6}{35}\right)^2 = 0.671$$

ONE LOADED LANE:

$$DF = 0.36 + \frac{S}{25} = 0.36 + \frac{6}{25} = 0.6$$

SUMMARY OF LL DF

POSITIVE BENDING	(−) BENDING	SHEAR
0.555	0.555	0.671
0.377	0.389	0.6 ← FATIGUE LIM

③ COMPUTE EFFECTIVE FLANGE WIDTH:

- $1/4\,(L) = 1/4\,(54.84) = 13.71$ FT
  $13.71\,(.72 \leftarrow$ simple span$) =$ 9.9 FT
- Span = 6 FT
- 12 × min slab t + 1/2 FLANGE W. =
  $(12)(8/12) + (1/2)(1) =$ 8.5 FT

MINIMUM = 6 FT.

b_e = 6 FT.

④ DYNAMIC LOAD ALLOWANCE

FOR STRENGTH & SERVICE LIMIT STATES, LL IMPACT

IM = 0.33

FOR FATIGUE LIMIT STATE, LL IMPACT

IM = 0.15

5) UNFACTORED MOMENTS & SHEARS (USING AISC Design Tables)

DC1 = 0.90875 k/ft.

AISC Table 2.0: HS20-44 LOADING

a) max + moment = $(0.90875)(\underline{0.0703})(45.84\text{ FT})^2 = 134.24$ kft
b) max − moment = $(0.90875)(-\underline{0.1250})(45.84\text{ FT})^2 = -238.70$ kft
c) max V @ ABUTMENT = $(0.90875)(\underline{0.3750})(45.84\text{ FT}) = 15.62$ k
d) max V @ PIER = $(0.90875)(\underline{0.6250})(45.84') = 26.04$ k
e) max Rxn @ ABUTMENT = $(0.90875)(0.3750)(45.84') = 15.62$ k
f) max Rxn @ PIER = $(0.90875)(1.25)(45.84') = 52.07$ k

DC2 = 100 plf = 0.1 k/ft

a) 14.77 ft-k
b) −26.27 ft-k
c) 1.72 k
d) 2.87 k
e) 1.72 k
f) 5.73 k

DW case

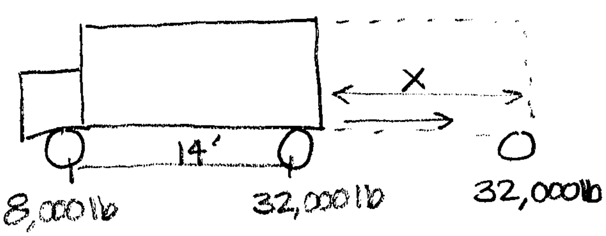
34.62 plf = 0.03462 k/f.

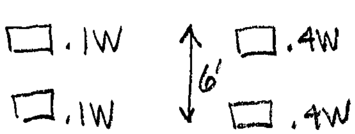
a) 5.11 ft-k
b) -9.09 ft-k
c) 0.60 k
d) 0.99 k
e) 0.60 k
f) 1.98 k

HL-93 LOADING - AASHTO Specs, uses HS 20
HS20 LL, Truck or Tandem load. + 0.64 k/f
U.D.L
unfactored D.L.

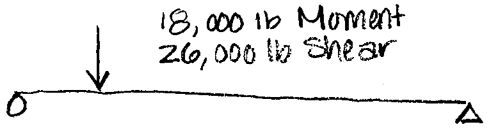
DESIGN TRUCK
HS20 for 1' < L < 140
↳ TRUCK GOVERNS BENDING M
& END SHEAR / RXN.

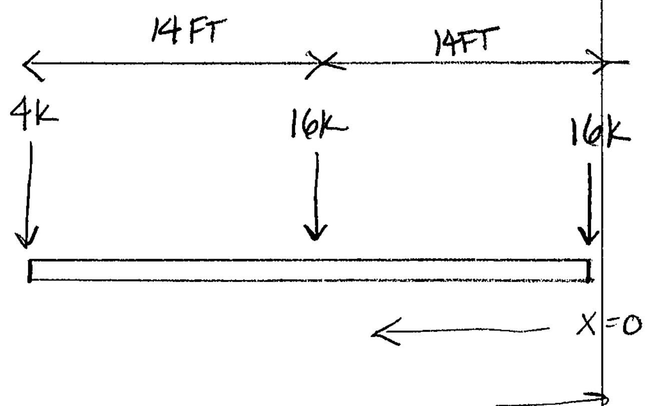
D Lane Load = $\boxed{0.64\ k/ft}$
(AISC Table A2.0)
a) 127.76 ft-k
b) -168.10 ft-k
c) 12.84 k
d) 18.34 k
e) 12.84 k
f) 36.67 k
LOOKING @ AREA ± ROWS





18,000 lb Moment
26,000 lb Shear



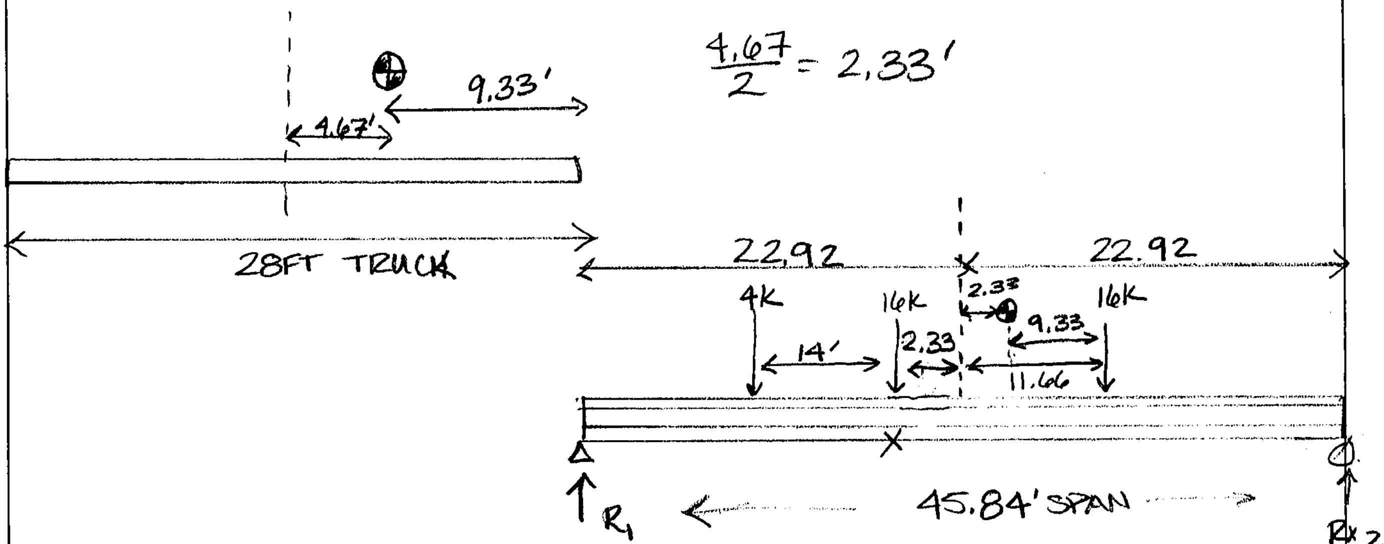


c.g. loads

P	X	PX
16k	0	0
16k	14	224
4k	28	112
36k		336

$X' = \frac{336 ft \cdot k}{36 K} = 9.33 ft$ $X = 14 - X' = 14 - 9.33$
$X = 4.67 ft = 4' - 8''$

$\frac{4.67}{2} = 2.33'$



X = TYP. POINT OF MAX MOMENT

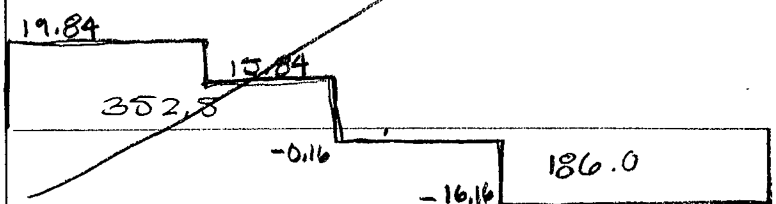
FROM AASHTO. STANDARD SPECS FOR HWGY BRIDGES

APPENDIX A: 405.7 k-ft = max M
40.7 K = max V / Rxn

ESTIMATED SIMPLE SPAN
$R_1 + R_2 - 36 = 0$

$\Sigma M_{R_1} = 4(6.59) + 16(20.59) + 16(34.59) - R_2(45.84)$

$R_2 = 19.84 K$
$R_1 = 16.165 K$



FROM TABLE 2.0 MOMENTS, SHEARS, REACTIONS (AISC)

a) 429.1
b) -317.9
c) ~~54~~.1
d) 60.6
e) ~~54~~.1
f) 67.8

ASSUME L = 90'
SHORT SPAN = 45'

$0.0703\, w (45.84^2) = 429.1$
$w = 2.90$

⑥ POSITIVE FLEXURAL DESIGN.

MOMENT @ STRENGTH I

$M_u = 1.25\,DC + 1.5\,DW + 1.75\,DF\,(LL + IM)$

$1.25(134.24 + 14.77) + 1.5(5.11) + 1.75(.555)(1.33 * 429.1 + 127.76)$

$M_u = \boxed{872.31\ ft\text{-}k}$

MOMENT @ STRENGTH IV.

$M_u = 1.5 * (DC + DW)$

$1.5 * (134.24 + 14.77 + 5.11)$

$M_u = 231.18\,ft\text{-}k$

FOR COMPOSITE SECTION, COMPRESSION BLOCK HEIGHT.

$a = \dfrac{A\,F_y}{0.85 f'_c\, b_{ef}} = \dfrac{44(50)}{0.85(6.5)(6\,FT \times 12)} = 5.53\,in < 8''$ OK!

$d = \overset{ts}{8} + 1.5 + 42 - \overset{\text{c.g. from bottom}}{21} = 30.5\ in$ center of gravity to top deck.

$M_p = A_sF_y\,(d - a/2) = 44(50)(30.5 - 5.53/2)(\frac{1}{12}) = 5084\ k\text{-}ft.$

Since $D_p = a = 5.53 in. > 1/10\,D_t = 1/10 * \overbrace{(8 + 1.5 + 42)}^{D_t = 51.5} = 5.15 in$

$M_n = M_p\,(1.07 - 0.7\,(D_p/D_t)) = [1.07 - 0.07\ (5.53/51.5)](5084)$
$= 5401.67\ k\text{-}ft.$

CONSIDERED A COMPACT SECTION FOR POSITIVE BENDING.

$\phi_f M_n = 1 * 5401.67\,kft = 5401.7\,kft > M_u = 872.3$ OK!

Satisfies strength / limit state requirements OK!

DUCTILITY REQMT: $\dfrac{D_p}{D_t} = \dfrac{5.53}{51.5} = 0.107 < 0.42$ OK!

CHECK FOR SERVICE LIMIT STATE:

$n = 7$

$$b_f = \frac{b_{eff}}{n} = \frac{72''}{7} = 10.29 \text{ in}$$

$$I_{deck} = \frac{b_f * t_s^3}{12} = \frac{10.3 \text{ in} * 8^3 \text{ in}^3}{12} = 438.9 \text{ in}^4$$

$$\left(\frac{bh^3}{12}\right)$$

$$Y_{conc} = [(8) + 1.5 + 42] - 8/2 = 47.5$$

ELEMENT	A (in²)	Y (in)	AY in³	AY² (in⁴)	I_o (in⁴)
STEEL SECTION	44	21	924	19,404	12,755 in⁴
CONCRETE SLAB	$t_s(b_f)$ = 82.32	47.5	3,910.2	185,734.5	438.9 in⁴
TOTALS	126.32		4834.2	205,138.5	13,193.9

$$I_z = \Sigma I_o + \Sigma AY^2 = 13,193.9 + (19,404 + 185,734.5)$$
$$= 218,332.4 \text{ in}^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = \frac{(4834.2)}{126.32} = 38.27 \text{ in}$$

$$I = I_z - \Sigma A (Y')^2 = 218,332.4 - 126.32(38.27)^2 = 33,325.02 \text{ in}^4$$

$$S_b = \frac{I}{Y'} = \frac{(33,325)}{(38.27)} = 870.8 \text{ in}^3$$

COMPOSITE SECTION FOR SUPERIMPOSED D.L. $3n = 21$

$$b_f = \frac{b_{eff}}{3n} = \frac{6*12}{21} = 3.43 \text{ in}$$

$$I_{deck} = \frac{3.43 \text{ in} * 8.0^3}{12} = 146.35 \text{ in}^4$$

ELEMENT	A (in²)	Y (in)	AY (in³)	AY² (in⁴)	I_o (in⁴)
STEEL	44	21	924	19,404	12,755
CONCRETE	27.44	47.5	1303.4	61,911.5	146.35
TOTALS	71.44		2,227.4	81,315.5	12901.35

$$I_z = \Sigma I_o + \Sigma AY^2 = 94,216.85 \text{ in}^4$$

$$Y' = \frac{\Sigma AY}{\Sigma A} = 31.18 \text{ in}$$

$$I = I_z - \Sigma A (Y')^2 = 24,763.42 \text{ in}^4$$

$$S_b = I/Y' = 794.21 \text{ in}^3$$

FOR NONCOMPOSITE SECTION

$I = 12,755$
$Y' = 21 \text{ in}$
$$S_b = \frac{12,755}{21} = 607.4 \text{ in}^3$$

DL STRESS

$$f_{DL} = \frac{M_{DL1}}{S} = \frac{134.24 * 12}{607.4} = \underline{2.65 \text{ ksi}}$$

SUPERIMPOSED DL STRESS

$$f_{SDL} = \frac{M_{DL2} + M_{DW}}{S_b} = \frac{(14.77+5.11)*12}{794.21} = \underline{0.300 \text{ ksi}}$$

LL STRESS:

FACTOR FOR LIMIT STATE 2 = 1.3

$M_u = 1.3(0.555) * (1.33 * \underset{\uparrow\ max\ LL}{\underline{429.1}} + \underset{\uparrow\ 0.640\ U.D.L.}{127.76}) = \boxed{503.94\ ft\cdot k}$

↑ + bending state factor

$f_{LL} = \frac{M_{LL}}{S} = \frac{503.9 \cdot (12)}{870.8} \; 6.94 \text{ ksi}$

TOTAL FACTORED STRESS @ BOTTOM FLANGE @ SERVICE STATE II

$f_{bot} = f_{DL} + f_{SDL} + f_{LL} = 2.65 + 0.3 + 6.94 = 9.89 \text{ ksi} < 0.95 f_y$ OK!

(7) NEGATIVE FLEXURAL DESIGN

FACTORED MOMENT MOMENT @ STRENGTH LIMIT STATE 1:

$M_u = 1.25\,DC + 1.5\,DW + 1.75\,DF\,(LL + IM)$
$1.25(238.7 + 26.27) + 1.5(9.09) + 1.75(0.555)(\underline{317.9} * 1.33 + 168.1)$
$= \boxed{918.77}\ ft\cdot k$

FACTORED MOMENT AT STRENGTH LIMIT STATE 4:

$M_u = 1.5(DC + DW)$
$1.5(238.7 + 26.27 + 9.09)$
$= 411.09\ ft\text{-}k$

ELASTIC COMPOSITE SECTION PROPERTY:

$A_s = 10.37 \text{ in}^2$ (ASSUMED) 5.93" FROM TOP OF STEEL GIRDER.

ELEMENT	A(in²)	y(in)	Ay (in³)	Ay² (in⁴)	Io (in⁴)
STEEL GIRDER	44	21	924	19 404	12,755
REINF. STEEL	10.37	47.5	492.58	23397.3	0
TOTAL	54.37		1416.58	42,801.3	12,755

$I_z = 12,755 + (42801.8) = 55,556.3\ in^4$

$y' = \frac{1416.58}{54.37} = 26.05\ in$

$I = 55,556.3 - 54.37(26.05)^2 = 18,660.38$

$S_{bot} = 18,660.38 / 26.05 = 716.33\ in^3$

$S_{top} = 18,660.38/(42 - 26.05) = 1169.9\ in^3$

STRESS IN COMPRESSION FLANGE

$\lambda_{rw} = 5.7\sqrt{\frac{E = 29000}{F_y = 50}} = 137.3$

$\frac{2D_c}{t_w} = \frac{2(\overset{26.05}{y} - 1.5)}{0.5} = 98.2 < \lambda_{rw}$ $\therefore R_b = 1.0$

LOCAL BUCKLING CHECK:

$\frac{b_{fc}}{2t_{fc}} = \frac{\overset{\text{flange}}{12''}}{2(1.0)} = 6 < 0.38\sqrt{\frac{E}{F_y}} = 9.15$

$\therefore$ compression flange is compact & no allowable stress reduction is necessary.

CHECK FOR LATERAL TORSIONAL BUCKLING:

$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1 D_c t_w}{3 b_{fc} t_{fc}}\right)}} = \frac{12}{\sqrt{12\left(1 + \frac{1}{3}\frac{(26.05 - 1.5)(1)}{3(12)(1)}\right)}} = 3.127$

$L_p = 1.0 r_t \sqrt{\frac{E}{F_y}} = 1.0(3.127)\sqrt{\frac{29,000}{50}} = 75.31\ in.$

$L_r = \pi L_p = 236.59\ in$

ASSUME COMPRESSION FLANGE UNBRACED LENGTH $L_b = 20 = 240''$

$L_p < L_b \leq L_r$

$f_2 = \frac{M_u}{S_{bot}} = \frac{(918.77) * 12}{716.33\ in^3} = 15.39\ ksi$

AT MIDDLE OF BRACED POINT. (0.1L FROM PIER) → 4.58 FT TABLE A2.0 coeff = -0.0675

.909 K/F $M_{DC1} = 0.909\ (-0.0675)(45.84\ FT^2) = 128.9\ k\text{-}f$
.1 K/F $M_{DC2} = 0.1\ (\ \text{"}\ )\ \text{"} = 14.2\ k\text{-}f$
.035 K/F $M_{DW} = 0.035\ (\ \text{"}\ )\ \text{"} = 4.96\ k\text{-}f$
2.90 $M_{truck} = 2.90\ (\ \text{"}\ )\ \text{"} = 411.3\ k\text{-}f$
.64 K/F $M_{LANE} = 0.64\ (-0.0675)(45.84^2) = 90.8\ k\text{-}f$

$M_u = 1.25DC + 1.5DW + 1.75\ DF\ (LL+IM)$
$1.25(128.9+14.2) + 1.5(4.96) + 1.75(0.555)(1.33 * 411.3 + 90.8)$

$= \boxed{805.8\ k\text{-}ft.}$

ASSUME 0.9L, SECTION IS SAME AS OVER PIER.

$$f_{mid} = \frac{M_u}{S_{bot}} = \frac{(805.8) * 12}{716.33\ in^3} = 13.50\ ksi$$

AT BRACE POINT / 9.17FT FROM PIER ; 0.2L — TABLE A2.0 — coeff = -0.0200

$M_{DC1} = 0.909\ (-0.02)(45.84^2) = 38.2\ k\text{-}ft.$
$M_{DC2} = 0.1 = 4.20\ k\text{-}ft$
$M_{DW} = 0.035 = 1.47\ k\text{-}ft$
$M_{truck} = 2.90 = 121.9\ k\text{-}ft$
$M_{lane} = 0.64 = 26.9\ k\text{-}ft.$

$M_u = 1.25DC + 1.5DW + 1.75\ DF\ (LL+IM)$
$1.25(38.2+4.2) + 1.5(1.47) + 1.75(0.555)(1.33 * 121.9 + 26.9)$
$= \boxed{238.80\ k\text{-}ft}$

ASSUME AS 0.8L, SECTION IS SAME AS THAT OVER PIER.

$$f_0 = \frac{Mu}{S_{bot}} = \frac{(238.8)*12}{716.33 in^3} = 4.00 \text{ ksi}$$

$$f_1 = 2f_{mid} - f_2 > f_0$$

$$2f_{mid} - f_2 = 2*(13.5) - (15.39) = 11.61 > f_0 = 4.00\text{ksi} \text{ OK!}$$

if $f_0 > f_1$ then f_1 becomes f_0

$$C_b = 1.75 - 1.05(f_1/f_2) + 0.3(f_1/f_2)^2 \le 2.3$$
$$= 1.75 - 1.05(11.61/15.39) + 0.3()^2 = 1.13 \le 2.3$$

$$F_{nc} = C_b\left(1 - 0.3\frac{L_b - L_p}{L_r - L_p}\right)R_b F_y \le R_b F_y = 50\text{ksi}$$

$$1.13\left[1 - 0.3\left(\frac{240 - 75.31}{236.6 - 75.31}\right)\right]50 = 39.19 \le 50\text{ksi} \quad \text{OK!}$$

IF $F_{NC} > 50$ksi, $F_{NC} = 50$ksi.

MAXIMUM STRESS IN COMPRESSION FLANGE:

from neg. flex design.

$$f_c = \frac{Mu}{S_{bot}} = \frac{(918.77)*12}{716.33 in^3} = \underline{15.39 \text{ ksi}} < F_{NC} \quad \underline{\text{OK!}}$$

CHECK ALLOWABLE STRESS IN TENSION FLANGE:

$$f_t = \frac{Mu}{S_{top}} = \frac{(918.77)*12}{1169.9 in^3} = \underline{9.42 \text{ ksi}} < F_{NC} \quad \underline{\text{OK!}}$$

8 SHEAR @ ABUTMENT.

FACTORED SHEAR AT STRENGTH I:

$$V_u = 1.25\,DC + 1.5\,DW + 1.75\,DF\,(LL + IM)$$
$$= 1.25(15.62 + 1.72) + 1.5(0.60K) + 1.75(0.671)(1.33 * 54.1 + 12.84)$$
$$= \boxed{122.14\,K}$$

FACTORED SHEAR AT STRENGTH IV

$$V_u = 1.5\,(DC + DW)$$
$$= 1.5\,(15.62 + 1.72 + 0.60)$$
$$= 26.91$$

$$\frac{D}{t_w} = \frac{40}{0.5} = 80.0$$

ASSUME K = 5 (no transverse stiffiners)

$$1.4\sqrt{\frac{EK}{F_y}} = 1.4\sqrt{\frac{29000 * 5}{50}} = 75.4 < \frac{D}{t_w} = 80, \text{ therefore}$$

$$C = \frac{1.57}{80.0^2}\left(\frac{29{,}000 * 5}{50}\right)$$ PG 186 "BRIDGE ENG." OTHERWISE

$$= 0.711$$

$$V_n = CV_p = C(0.58\,F_y\,D\,t_w) = 0.711\,(0.58 * 50 * 40 * 0.5)$$
$$= 412.38\,K$$

$$\Phi V_n = 1.0 * 412.38K > V_u = 122.14K$$ OK!

∴ NO SHEAR STIFFENERS REQ'D @ ABUTMENT WHEN USING ASSUMED SECTIONS.

(9) SHEAR @ PIER

FACTORED SHEAR @ STRENGTH I.

$$V_u = 1.25DC + 1.5DW + 1.75DF(LL+IM)$$
$$= 1.25(26.04+2.87) + 1.5(0.99) + 1.75(0.671)(1.33*60.6+18.34)$$
$$= \boxed{153.8\,k}$$

FACTORED SHEAR @ STRENGTH IV

$$V_u = 1.5(DC+DW)$$
$$= 1.5(26.04 + 2.87 + 0.99)$$
$$= 44.85\,k$$

FROM STEP 8, ALREADY CALCULATED: (BOTH WEBS ARE SAME)

$\phi_v V_n = 412.4k > V_u = 341.6k$ ∴ NO SHEAR STIFFENERS REQ'D NEAR PIER.

(10) CHECK FATIGUE LIMIT STATE

DETAIL CATEGORY C   pg 208 TAB 3.13 "BRIDGE ENG"

$A = 44 * 10^8\ ksi^3$ ← FOR 2-LANES

$ADTT = P(ADTT) = 0.85(251) = 214$ trucks/day

$$N = 365 * 75n(ADTT)_{SL} = 365*75(1.5)(214) = 8{,}787{,}375$$

$$(\Delta F)_n = \left(\frac{A}{N}\right)^{1/3} = \left(\frac{44*10^8}{8787375}\right)^{1/3} = 7.94\ ksi$$

$$\tfrac{1}{2}(\Delta F_n)_{TH} = \tfrac{1}{2}(10) = 5$$

↑ FROM TABLE 3.13 (CATEGORY C)

∴ $(\Delta F)_N = \underline{7.94\ ksi}$ governs

MAX. FACTORED MOMENT OVER PIER @ FATIGUE LIMIT STATE.

$$M_{max} = 0.75\,DF(LL+IM)$$ ← max neg. moment
$$= 0.75(0.389)(1.15 * 317.9) = \mathbf{106.66\ k\text{-}ft}$$

$$M_{min} = \phi$$

$$\Delta f_{top} = \frac{\Delta M}{S_{top}} = \frac{(106.66-0)*12}{1169.9\,in^3} = 1.09\ ksi < (\Delta F_n) = 7.94$$

OK!

WELDS @ BOTTOM FLANGE w/ STIFFENER PLATES (ASSUMED AT MAX. POSITIVE MOMENT LOCATION)

$\left(\frac{A}{N}\right)^{1/3} = 7.94\,ksi$ CATEGORY C'

$\frac{1}{2}(\Delta F)_{TH} = \frac{1}{2} * 12 = 6.0\,ksi$

$(\Delta F)_N = 7.94\,ksi$

MAX FACTORED (+) MOMENT (@ FATIGUE LIMIT STATE):

$M_{max} = 0.75\ DF\ (LL + IM)$

$0.75(0.377)(1.15 * 429.1) = 139.5\ k\text{-}ft.$

$M_{min} = 0.75\ DF\ (LL+IM)$

$0.75(0.377)(1.15)(\underline{-63.56}) = -20.67\ k\text{-}ft.$

need to obtain @0.4L

$w(-.1250)(45.84^2) = -317.9$

$w(-0.0250)(\text{''}) = 63.56$

$\leftarrow 1,2)$

$\Delta f_{bot} = \frac{\Delta M}{S_{bot}} = \frac{(139.5 + 20.67)(12)}{716.33\,in^3} = 2.67\,ksi < (\Delta F_n) = 7.94\,ksi$

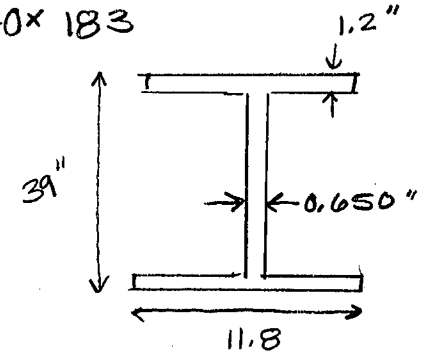
OK!

(11) SHEAR CONNECTORS

SHAPE / GIRDER PROPERTIES: W40× 183

$I_x = 13,200$ $I_y = 331$

$A = 53.3$



ΣQ_n TABLE 3-19 AISC MANUAL

↳ 2670 kip.

$Q_n = 0.5(A_{sc})\sqrt{f'_c(E_c)} \le A_{sc}\,F_u$

assume 3/4" studs, $A_s = 0.4418\ in^2$

$E_c = w_c^{1.5}\sqrt{f'_c}$

$(150)^{1.5}\sqrt{6.5\,ksi}$

$= 4683.7\ ksi$

$Q_n = 0.5(0.4418\,in^2)\sqrt{(6.5\,k/in^2)(4683.7\,k/in^2)}$

$= 38.54 \le 0.4418(65)$ X

$= 28.72^k/stud$

USE $Q_n = 28.72\ ^k/stud.$

studs: $\frac{\Sigma Q_n}{Q_n} = 92.97$ STUDS $93 \times 2 = 186$ STUDS TOTAL

STUD SPACING: $S = \frac{L = 45.84 \times 2}{93} = \boxed{11.8\text{'' SPACING}}$

On each girder

$$\text{Slab DL} = \frac{8}{12}(6ft)(0.150\ k/ft^3) = 600\ lb/ft$$

$$\text{haunch} = \frac{12}{12}(1.5)(0.150) = .225\ lb/ft$$

$$\text{Steel Girder} = 200\ lb/ft$$

$$\text{Deckforms} = 90\ lb/ft$$

$$\text{Concrete barrier} = 130\ lb/ft$$

$$\text{sidewalk} = \underset{\uparrow}{5}\left(\frac{6}{12}\right)(0.150) = 375\ lb/ft$$

overhang + 3ft

FWS

$$90\ lb/ft^2\,(59.39 - 10ft)\,\frac{1}{10} = 445\ lb/ft$$

$$LL = 640\ lb/ft$$

$$U = 1.2D + 1.6L$$

$$U = 1.2(1865\ lb/ft) + 1.6(640\ lb/ft)$$

$$= 3262\ lb/ft$$

$$M_U = \frac{wL^2}{8} = \frac{3262(91.86ft)^2}{8} \times \frac{1}{1000}$$

$$= 3441\ ft\ kips$$

$$\phi M_p = \phi Z_x F_y$$

$$Z_x = \frac{3441(12)}{0.9(50)} = 917.6\ in^3$$

Tbl 3 - 2

W 40 x 215

FLB & WLB met

$\phi M_p = 3620$ ft kips

$Z_x = 964 \text{ in}^3$

update M_u

$M_u = 3713$ ft kips

revise section

W 36 x 247 $\quad \phi M_p = 3860$ ft kips

upade M_u

3753 ft kips ✓

FLB WLB

both met ✓

LTB

$$KL/r_x = \frac{2\left(91.86/2\right)\times 12}{14.9} = 73.98$$

$$4.71\sqrt{E/F_y} = 113.43$$

$$F_{cr} = 0.658^{F_y/F_e} F_y \qquad F_e = \pi^2 E/(KL/r)^2 = 53.3$$

$$= 0.658^{\left(50/53.3\right)} 50$$

$$= 35.5 \text{ ksi}$$

$$\phi P_n = 0.9\,(35.5)(75.4)$$

$$= 2274^{k}$$

Not > Mu

Try W 33 x 263

$$M_u = \frac{\left(1.2(1865 + 63) + 1.6\,(640)\right)(91.86)^2}{8}$$

$$M_u = 3520 \text{ ft kips}$$

FLB WLB

$$b_f/2t_f = 5.03 < 9.2 \checkmark$$

$$h/t_w = 34.3 < 90.5 \checkmark$$

LTB

$$KL/r = \frac{2\left(91.86/2\right) \times 12}{14.3} = 77.1$$

$$F_{cr} = 0.658^{F_y/F_e}\, F_y \qquad F_e = \pi^2 E/(KL/r)^2$$

$$= 0.658^{(50/48.2)}\, F_y \qquad 48.2$$

$$= 32.4 \text{ ksi}$$

$$\phi P_u = 0.9(32.4)(77.5)$$

$$\phi P_u = 2259.9 \text{ kips}$$

go to Excel spreadsheets

Steel Girder

Slab	600	lb/ft	
Haunch	225	lb/ft	
Steel Girder	200	lb/ft	
Deck Forms	90	lb/ft	
Concrete Barrier	130	lb/ft	
Sidewalk	375	lb/ft	
FWS	445	lb/ft	
LL	640	lb/ft	
Additional Weight Girder	231		
w	3779.2	lb/ft	
Mu	3986.233835	ft-kips	
FLB	3.44		TRUE
WLB	25.2		TRUE
rx	16.6		
KL/rx	66.40481928		

Area of Girder	127

KL/rx <113.43	TRUE		
Fe	64.84225292		
Fcr	36.20792207		
Pu	4138.565493	kips	**TRUE**

KL/rx > 113.43	FALSE		
Fcr	56.86665581		
Pu	6499.85876	kips	**TRUE**

Steel - Deck

Steel Girder
Deck

Overall Information/Specifications

Girder Spacing	6	ft	Number of Girders	10
Top Cover	2.5	in	Girder Height	41.3
Bottom Cover	2.5	in		
Yield Strength of Steel	60	ksi		
Compressive Strength of Concrete	4	ksi		
Density of Concrete	150	pcf		
Density of Future Driving Surface	90	psf		

Deck Thickness

Deck Thickness	8	in	
Deck Thickness/Span Length	0.0072574		**TRUE**
Overhang Thickness	9	in	

Dead Load Effects

Unfactored Moments

Deck	0.36	k-ft/ft
Asphalt	0.324	k-ft/ft

Load Factors

Deck, Girders, Sidewalk	1.25
Asphalt	1.5

Dead Load Effects

Deck	0.45	k-ft/ft
Future Wearing Surface	0.486	k-ft/ft

Live Load Effects

Min d wheels to parapet	1	ft	
Min d b/t wheels	4	ft	
Dynamic Load Allowance	33	%	
Load Factor	1.75	(Strength I)	
Multiple Presence Factor	1		
ψ Strength Limit State	0.9		
ψ Extreme Event Limit State	1		
Unfactored LL + Moment	4.71	k-ft/ft	Table A 4.1
Max Factored Moment	8.2425	k-ft/ft	

Total Load Moment

Moment (DL+LL)	9.1785	k-ft/ft

Positive Moment in Deck Design

de	5.19	for 8 inch deck	Assuming #5 bars	
k'	0.3786121			
ρ	0.0067071			
As	0.03481			
Bar Area	**0.31**		**db**	**0.625**
Spacing	8.9054807	inches		

Steel Girder
Deck

Spacing Used!	**4**		

Check Reinforcement

T	18.6		
a	1.3676471		
β1	0.85	for f'c = 4	
c	1.6089965		
c/de	0.3100186	**TRUE**	<- Checking Over-Reinforcement

Cracking

Z	130	k/in		
dc	2.31	in	for 8 in deck	**TRUE**
A	18.48	in2		
fsa	37.196782	ksi	**TRUE**	
fsa final	36	ksi		

Service Load Stresses

n	8		
DL	0.936	k-ft/ft	
LL	8.2425	k-ft/ft	
Transformed As	2.48	in2	
Neutral Axis (Quadratic)			
a	2		
b	2.48		
c	12.8712		
y	1.991513	in	
Transformed I	35.902641	in4	
fs	26.166153		**TRUE**

Distance from Center of Girder to the Design Section for Negative Moment

Girder Top Flange Width	16.2	in	
1/3 W	5.4		**TRUE**

Negative Moment at Interior Girders Design

Unfactored LL - Moment	3.96	ft-kips	Table A 4.1	
LL (-) Moment	6.93	ft-kips		
DL + LL (-) Moment	7.866	ft-kips		
d	5.19	for 8 inch deck	Assuming #5 bars	
k'	0.3244716			
ρ	0.0056939			
As	0.0295515	in^2		
Bar Area	**0.31**	**in^2**	**db**	**0.625**
Spacing	10.490169	inches		
Spacing Used!	**10**			

Cracking Under Service Limit State

Z	130	k/in
dc	2.31	in
A	46.2	in2
fsa	27.406823	ksi

Steel Girder
Deck

Service Load Stresses

n	8		
DL	0.936	k-ft/ft	
LL	4.71	k-ft/ft	
Transformed As	2.48	in2	
Neutral Axis (Quadratic)			
a	3.5	7" section	
b	2.48		
c	12.8712		
y	1.5958435	in	
Transformed I	45.583747	in4	
fs	24.929632		TRUE

Design of the Overhang

Overhang	24.24	in
Self Weight	112.5	lb/ft^2
Parapet	650	lb/ft
Parapet Width	20.25	in
Girder Width	12	in
Sidewalk in overhang area	24.24	in
Sidewalk height	6	in
Sidewalk Weight in Overhang area	75	lb/ft^2

Case 1: Horizontal Vehicular Collision Load

a. At inside face of parapet

Mc at base of Parapet	-17.83	k-ft/ft		
MDL Slab	-0.160181	k-ft/ft		
MDL Parapet	-0.684667	k-ft/ft		
MDL Sidewalk	-0.106787	k-ft/ft		
Design Factored Moment	-19.01954	kips		
Design Axial Tensile Force	5.1813719	k/ft		A13.4.2
h	9	in		
d	5.19	in		Assuming #5 Bar
Assume As req	0.9	in2/ft		
T	54	k/ft		
C	48.818628	k/ft		
a	1.196535	in		
Mn	19.800645	k-ft/ft		
Mr	19.800645		TRUE	
c/de	0.2712309		TRUE	

b. At design section in the overhang

Collision Moment at Design Section	-17.94357	k-ft/ft
MDL Slab	0.2835014	k-ft/ft
MDL Parapet	1.0470417	k-ft/ft
MDL Sidewalk	0.1890009	k-ft/ft
Design Factored Moment	-19.843	kips
Design Tensile Force	5.058487	k/ft
h	9	in
d	6.1875	in

Assume As req	0.7	in2/ft	
T	42	k/ft	
C	36.941513	k/ft	
a	0.9054292	in	
Mn	18.958445	k-ft/ft	
Mr	18.958445		**TRUE**
c/de	0.1721553		**TRUE**

c. Check DL and Collision Moments at Design section C-C

M1	-17.83	k-ft/ft	
M2	-7.132	k-ft/ft	
Total Collision Moment	-15.95785	k-ft/ft	
Design Collision Moment	-14.82485	k-ft/ft	
MDL Slab	-0.35721	k-ft/ft	
MDL Parapet	-1.638	k-ft/ft	
Distributed Sidewalk	0.0757576		
MDL Sidewalk	-0.334125	k-ft/ft	
MDL FWS	-0.000946		
MFDL	-2.818391	k-ft/ft	
MFDL, O	-2.339265	k-ft/ft	
DL Design Factored Moment due to DL in 1st Span	0.269768	k-ft/ft	
MDL+C	-16.89435	k-ft/ft	
d	5.19	in	Assuming Slab Thickness =8in with 2.5 in cover
k'	0.6272009		
ρ	0.0116511		
As	0.0604694	in2/ft	

Detailing of Overhang Reinforcement

Largest As	0.9	
Top Reinforcement	#5 @ 12 inches	
Provided Top Reinforcement	0.31	in2/ft
Additional Reinforcement needed	0.59	Bundle 2 #5 to every one
T	55.8	kips
a	1.3839286	in
B1	0.85	
c	1.6281513	in
c/de	0.3137093	TRUE
Cutoff Length Requirement	9.375	inches
Required Length Past Centerline of exterior Girder	34.375	inches

Development Length

Basic Development length is the larger of the three values	11.625	Assumed #5 bars
	15	
	12	

Steel Girder
Deck

Development Length	12	Correction Factor for spacing > 6 inches
Required length of additional bars past the centerline of the exterior girder	17.4	TRUE

Longitudinal Reinforcement

Bottom Distribution Reinforcement		
Percentage	26.957869	TRUE
Bottom Transverse Reinforcement	0.62	# 5 at 6in
Require Long. Reinforcement	0.1671388	
Reqd Spacing	22.256952	in
USE	#5 bars @ 12 in spacing	

Top Longitudinal Reinforcement	Use # 4 bars at 12 in spacing

Check Shrinkage and Temperature Reinforcement

Ag	90	in2/ft	
As req	0.165	in2/ft	
As req per surface	0.0825		
As provided (long rein least)	0.169	in2/ft	TRUE

Steel - Bearings

PART II

BEARING DESIGN

(1) REVISIT LOADING

DL Shear = 17.94 K } NO ADDED FACTORS
LL Shear = 54.1 K } @ ABUTMENT

TOTAL BEAM LENGTH = 91.68 + 1 = 92.68 FT

SHEAR AT ABUTMENT V_u = 122.14^k FROM PART I, (8) & (9)
" " PIER V_u = 153.8^k

(2) TOTAL EXPANSION & CONTRACTION: ELASTOMERIC PAD BEARING

FOR 120°F: $6.5 * 10^{-6} * 120 * L = 92.68FT (12")$
$= 0.867$ in

MIN. THICKNESS (total) of elastomer: ERT = 2(0.867 in)
= 1.735 in.

5 LAYERS @ 3/8" T = 1.875 in > 1.735 in

PLATE THICKNESS: 14 GAUGE (0.075 in or THICKER)

BOTTOM PLY: 1/2 (3/8) = 3/16"

MAX. AREA OF PAD: $\frac{DL@ABUT}{0.2} = \frac{(15.62+1.72+0.60k)}{0.2 ksi}$ (17.94) $= \underline{90}$ in^2

MIN. AREA OF PAD: $\frac{DL@ABUT}{0.5 ksi}$ OR $\frac{TOTAL LOAD@ABUT}{0.8 ksi}$; $\frac{DL}{0.5} = \underline{35.88}$ in2

→ 106.1

FLANGE WIDTH OF BEAM = 11.8 in

- THICK LOAD PLATE WELDED TO BOTTOM OF BEAM TO DISTRIBUTE RXN t = 1 1/4"

PLATE CAN BE WIDER THAN FLANGE

TRY ~~W = 10"~~ ~~L = 7 1/2"~~ ~~t = 3/8"~~ TRY W = 12" L = 7 1/2" t = 3/8"

(3) DETERMINE SHAPE FACTOR:

$$SF = \frac{L \times W}{2t(L+W)}$$

$$= \frac{7.5(10)}{2(0.375)(7.5+10)} = 5.7$$

REVISED SF = $\underline{6.15}$

COMPRESSIVE STRESS:

$\frac{\text{TOTAL LOAD}}{\text{PLATE AREA}}$ ksi = 0.96, too large

TRY $A = 12'' \times 7.5'' = 90 in^2 \rightarrow 72.04^k / 90 in^2 = 0.8$

VERTICAL STRAIN: ~4.75% ≤ 7% OK!

TOT. PAD THICKNESS: SEE BELOW FIG

$4.8(\Delta)^2 < DL \Rightarrow 4.8(0.867 in)^2 = 4.16 in^2$

FIG 10.1 "DESIGN OF MODERN HWY BRIDGES"

4 LONGITUDINAL FORCE:

HS-20 LOADING

$\frac{(0.64)(L+1')(2) + (18)(2)}{10 \text{ GIRDERS}}(0.05) = 0.773 K$

(92.68' ; # lanes (traffic) ; 5%)

F < DL / # GIRDERS ? (FOR NO SLIPPAGE)

$0.773^k < \frac{17.94}{10} = 1.794^k$

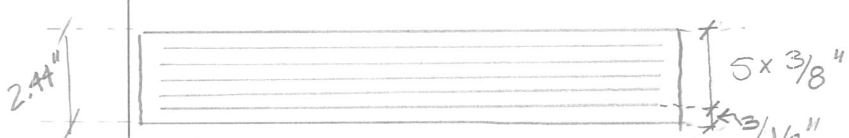
T. 10.1 "DESIGN OF MODERN HGWY BRIDGES"

$F = \frac{\text{SHEAR MODULUS } (L \times W)\, \Delta}{\text{ERT}}$

ERT = EFFECTIVE RUBBER THICKNESS → 3/8 * 5 + 3/16

ASSUME 50 HARDNESS

$= \frac{[1.9(110)]\ (7.5 * 12)(.867 in)}{2.06 in} = 7916.6 lb.$



14-GAUGE STEEL PLATES.

BEARING PAD.

3/8 * 5 + 3/16 + 0.075" x 5

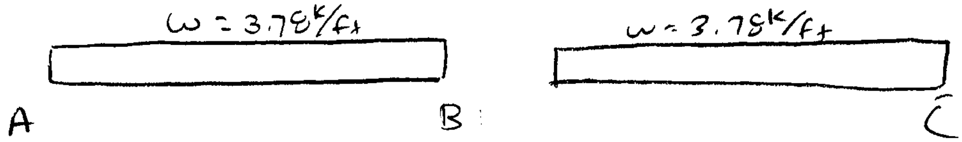
= 2.44 in

5 SUMMARIZE

- SUBSTRUCTURE MUST BE DESIGNED FOR TOTAL LONG. F = 7.92 + .773 = 8.69 K
- SIDE JACKET = 1/8 ALL SIDES
- BEARING DIM: 7 3/4" x 12 1/4"
- LOAD PLATE THICKNESS: 1 1/4"
- SIZE: 1/2" > than bearing all around.

MIN. LOAD PLATE SIZE:

8 3/4" x 13 1/4" x 1 1/4"



$$\Sigma M_A = 0$$

$$0 = \frac{wL^2}{2} - R_b(L)$$

$$\frac{wL}{2} = R_b = \frac{3.78\left(91.86/2\right)}{2} = 87^k$$

$$\Sigma M_B = 997^k$$

$$997^k = -\frac{3.78\ ^k/_{ft}\left(91.86/2\right)^2}{2} + R_A\left(91.86/2\right)$$

$$R_A = 109^k$$

Steel Bearing Design

① Shear at Abutment = 108.5^k DL = 102.6^k

LL = 5.9^k

② Total Expansion & Contraction: Elastometric Pad Bearing

For 120°F: $6.5\times10^{-6} \times 120 \times 92.86 \times 12$

$= 0.869 in$

Min thickness (total) of elastomer ERT = 2(0.869)

= 1.738 in

5 layers @ 3/8" T = 1.875 in > 1.738 in

Plate Thickness: 14 Gauge (0.075 in or thicker)

Bottom ply; 1/2(3/8) = 3/16"

Max area of Pad

$$\frac{102.6}{0.2} = 513 in^2$$

Min area of pad

$$\frac{102.6}{0.5} = 205.2$$

Flange width of Beam = 16.2 in

Thick load plate welded to bottom of beam to distribute R x N t = 1 1/4"

Try w = 20

L = 12

t = 3/8

③ Determine Shape Factor

$$SF = \frac{L \times W}{2t\,(L+W)}$$

$$= \frac{12 \times 20}{(\frac{3}{8})(2)\,(12+20)} = 10$$

Compressive Stress

$$108.5/(20*12) = 0.45\ ksi \ < 0.8 \quad OK!$$

VERTICAL STRAIN: $\approx 1.75\%$ FIG 10.1 'DESIGN OF MOD. HWY BRIDGES'

TOTAL PAD THICKNESS - SEE BELOW FIGURE

$$4.8(\Delta)^2 < DL = 4.8(0.867 in)^2 = 4.16\ K < 102.6K \ \checkmark OK!$$

④ LONGITUDINAL FORCE

HS-20 LOADING

$$\frac{0.64(92.68')(\overset{\text{\# lanes traffic)}}{2}) + (18)(2)}{10\ GIRDERS}(\overset{5\%}{0.05}) = 0.773k$$

$$F < DL/\#\,GIRDER. \Rightarrow 0.773k < \frac{102.6}{10} = 10.4\ k \ \checkmark$$

$$F = \underset{\text{FROM TAB 10.1}}{\text{SHEAR MODULUS}}\,(L \times W)\,\Delta\,/ERT$$ FOR NO SLIPPAGE.

$$\frac{1.9(110)(12 \times 20)(0.867\,in)}{2.06} = 21{,}111\ lb = 21.1\ K$$

21.1K + 0.773k = 21.8k design force

MIN LOAD PLATE SIZE

$13\frac{1}{4} \times 21\frac{1}{4} \times 1\frac{1}{4}''$

Costs

Concrete

#5@4in

$$\frac{91.86 \times 12}{4} = 275 \quad \frac{\times 59.39 \times 0.31}{144} = 35.2$$

#5@12in

$$\frac{59.39 \times 12}{12} = \frac{59 \times 91.86 \times 0.31}{144} = 11.7$$

#4@12

$$\frac{59.39 \times 12}{12} = \frac{59 \times 91.86 \times 0.26}{144} = 7.5$$

#5@12in

$$\frac{91.86 \times 12}{12} = \frac{91 \times 59.39 \times 0.31}{144} = 11.6$$

Overhang

#5@12in

$$2 \times \left(\frac{91.86 \times 12}{12}\right) = \frac{2(91)(36 + 22.34 + 10) \times 0.31}{12^3} = 2.2$$

total Volume = 68.2 ft^3

steel

only last 2 change

$$\frac{91.86 \times 12}{10} = \frac{110 \times 59.39 \times 0.31}{144} = 14.1$$

Overhang

$$2\left(\frac{91.86 \times 12}{10}\right) = \frac{2(110)(36 + 24.24 + 8.1) \times 0.31}{12^3} = 2.7$$

Total Steel = 71.2 ft^3

Miscelaneous Items

Description	Unit	Materials	Labor	Equipment	Total/unit	Quantity	Total
Parapet	CY	170	241	21.5	432.5	29.45916	$12,741.09
Sidewalk Concrete 3 ksi CIP 6/6 - W1.4 x W1.4 mesh broomed finish no base 6" thick	SF	2.1	1.47		3.57	861.1875	$3,074.44
Curbing Granite 6" x 18"	LF	12.4	3.49	1.12	17.01	183.72	$3,125.08
Asphaltic Concrete Pavement, Highways, Wearing Course 1.5" thick	SY	2.83	0.37	0.28	3.48	558.3302	$1,942.99
Painted Traffic Lines and Markings Acrylic waterborne white or yellow 4" wide	LF	0.15	0.06	0.02	0.23	367.44	$84.51
						TOTAL	$20,968.10

Steel Girders

Description	Unit	Materials	Labor	Equipment	Total/unit	Quantity	Total
Structural Steel, rolled beams	Ton	1750	365	165	2280	197.9583	$451,344.92
Expansion seams, steel, double 8" x 6" angles only, 1-3/4" compression seal	LF	223	45.5	2.21	270.71	118.78	$32,154.93
						TOTAL	$483,499.86

Deck-Steel Girder

Description	Unit	Materials	Labor	Equipment	Total/unit	Quantity	Total
Reinforced Steel (Rough estimate under 10 ton job #3 to #7)	Ton	790	550		1340	17.444	$23,374.96
Deck, including finish and cure 8" thick, shored forms	SF	4.48	1.59	0.33	6.4	5455.565	$34,915.62
						TOTAL	$58,290.58

Total Cost - Steel

Bridge Component	*Total Cost*
Steel Girders	$483,499.86
Deck	$58,290.58
Miscellaneous Surface Components	$20,968.10
TOTAL COST OF STEEL BRIDGE	**$541,790.44**

Deck-Concrete Girder

Description	Unit	Materials	Labor	Equipment	Total/unit	Quantity	Total
Reinforced Steel (Rough estimate under 10 ton job #3 to #7)	Ton	790	550		1340	16.709	$22,390.06
Deck, including finish and cure 8" thick, shored forms	SF	4.48	1.59	0.33	6.4	5455.565	$34,915.62
						TOTAL	$57,305.68

Prestressed Concrete Girders

Description	Unit	Materials	Labor	Equipment	Total/unit	Quantity	Total
Conventional Reinforceing Steel #3 to # 7	Ton	800	760		1560	0.968995	$1,511.63
Precast, prestressed concrete I beams 60' to 80' span (smallest available)	each				8775	20	$175,500.00
Expansion seams, steel, double 8" x 6" angles only, 1-3/4" compression seal	LF	223	45.5	2.21	270.71	178.17	$48,232.40
						TOTAL	$225,244.03

Total Cost - Concrete

Bridge Component	*Total Cost*
Concrete Girders	$225,244.03
Deck	$57,305.68
Miscellaneous Surface Components	$20,968.10
TOTAL COST OF PRESTRESSED CONCRETE BRIDGE	**$282,549.71**