Project # LDA-1304

HIGHWAY BRIDGE REDESIGN

A Major Qualifying Project Report:

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Abstract
The goal of this MQP was to analyze different possible replacement strategies for a specific highway bridge to design the most easily implementable and efficient replacement. Design of the bridge elements was completed in accordance with the *AASHTO LRFD Bridge Specification* and was aided by the use of software. The final presentation included a structural design of the selected superstructure and substructure with accompanying construction and traffic planning, as well as a cost estimate of direct and indirect costs.

Acknowledgements

This project was only possible with the support and guidance of much of the engineering community at Worcester Polytechnic Institute. Countless work hours were spent in campus computer labs, at the library, and occasionally in random departments (Did you know WPI Facilities have a full size plotter?). The campus culture is invaluable in completing a complete and massive undertaking such as a Major Qualifying Project.

Specifically there are a few people we would like to thank specifically for their help with the project, starting with our Advisor Professor Albano. We always received quick responses to questions and good engineering guidance when the project stalled. It was a journey, but the final project has been shaped by Professor Albano's advice since the very beginning. None of the group members had any experience with bridge design before the project, and we made it to the finish, which says a lot for our advising quality.

We would especially like to thank Professor Okumus for her guidance throughout the course of the project. Her extensive knowledge in the subject and guidance in navigating the extensive AASHTO code were invaluable.

Finally, we would like to thank the engineers at the State of New Hampshire Department of Transportation, Construction Bureau. Personally, I (Joshua Nitso) had the opportunity to intern in the construction bureau working on a similar project last summer (2012), and it was one of the best experiences a young engineer can have. The ability to construct such important structures in our daily lives cannot be overstated, and I have a great respect for NHDOT engineers. Without their help in acquiring plans and information for this project, it would have been more difficult if not impossible to accomplish our goal.

Thank you,

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Capstone Design Statement

In this Major Qualifying Project, the project team was responsible for planning an Interstate Bridge replacement project. The subject bridge is located in Salem, New Hampshire and carries Interstate-93 over Pelham Road (NH 97). The replacement bridge had to be durable, functional, cost effective, and designed in a manner to be easily constructed with minimum impact on the community. The project group investigated several bridge designs and created a construction and traffic diversion plan. The design incorporated the concept of Accelerated Bridge Construction in order to minimize the project impacts to the economy and environment of Salem, and to the safety of the community. By attempting to meet the requirements for a successful replacement span, the MQP satisfies the requirements necessary for Capstone Design which includes the following realistic constraints: manufacturability, health and safety, and environmental implications.

Economic

- Cost estimates for the preliminary designs and final designs were completed for the total cost of the materials, equipment, and labor required.
- A Work Zone Road User Cost was developed for the final design to compare the financial impacts of Accelerated Bridge Construction compared to traditional construction methods.
- The proposed design was created to be as cost-effective as possible by attempting to minimize time onsite, traffic related costs, and by selecting an appropriate superstructure for the length of the design span.

Constructability

- Standard construction materials and practice were utilized when considering construction design scenarios. Materials and members easily obtainable were used whenever possible.
- Several superstructure designs for both steel and precast concrete were developed and analyzed to provide the most practical construction options.
- The structure was designed to be erected as quickly and efficiently as possible.

Environmental

- The project was designed to minimize the amount of construction time on-site which in turn limits the time the ground is left susceptible to erosion.
- The project attempted to limit detours and the construction of temporary roads and bridges by reducing the necessary traffic diversion.
- Appropriate *Best Management Practices* were utilized to limit the amount of stormwater runoff from the construction site.

Safety and Ethics

- The respective sections of the design were developed using the *AASHTO LRFD Bridge Specification*
- The construction was designed to be completed as quickly as possible with as much work done off-site as possible to reduce the danger to both workers and commuters.

Social and Political

- The project was designed to provide a safer, more functional bridge for public use. The bridge was designed to improve on the appearance of the existing structure.
- The construction and traffic plan were completed to minimize the impact on the public.

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1. Introduction

1.1 Introduction to Highway Bridges in the United States

According to the American Society of Civil Engineers, there are just north of six hundred thousand bridges in the United States (ASCE). This number may catch the attention of the average American, because that works out to one bridge for roughly every five hundred citizens. Many are aware of the majesty of the Golden Gate in San Francisco, and the bottleneck of the George Washington Bridge over the Hudson River. Not as many people are aware of the abundance of bridges required to maintain grade separation for interstate highways.

Let us look for instance, at a theoretical trip from Londonderry NH, entering at exit 5 on I-93, and finishing at the start of the Zakim Bridge in Boston. Many people who live in southern NH work in the Boston area, making their commute by this route every day. A Google maps trip search deems the drive a 46 mile and (very optimistically) a 51 minute excursion. To maintain the grade separation, which in turn allows the higher speed flow of more traffic, no fewer than forty-six separate bridges must be crossed (Google Maps). That is just for a southbound, one-way trip, and does not include the additional twenty-two overpasses carrying other roads above the highway. A return trip northbound would nearly double the number of separate bridges crossed. Most of these would be similar to the respective southbound structures, but the number of crossings required for a roughly 92-mile round trip is staggering.

The typical bridge may pass under the average citizen's radar, but they do not pass the years unmarked by corrosion, fatigue, and traffic damage. The Eisenhower Interstate Highway System was signed into law by the Federal Aid-Highway Act in 1956, and changed American culture and transportation forever (Eisenhower Interstate Highway System Web Site). The corridor of the new Interstate -93, stretching from Manchester, was one of the earlier sections completed, with the NH sections finished by the early 1960s. The new highway systems represented unprecedented speed and scale in terms of roadway and bridge construction. Now, more than fifty years later, they present a quandary for state highway departments nationwide. Due to the great growth and bridge building periods of the early and mid-twentieth century, the average age of a bridge in the United States is 43 years old (ASCE). As of 2008, ASCE classified more than 12% of bridges as structurally deficient and over 14% of bridges functionally obsolete. The report describes these two conditions as such:

"A structurally deficient bridge may be closed or restrict traffic in accordance with weight limits because of limited structural capacity. These bridges are not unsafe, but must post limits for speed and weight. A functionally obsolete bridge has older design features and geometrics, and though not unsafe, cannot accommodate current traffic volumes, vehicle sizes, and weights. These restrictions not only contribute to traffic congestion, they also cause such major inconveniences as forcing emergency vehicles to take lengthy detours and lengthening the routes of school buses."-ASCE 2009 Infrastructure Fact Sheet

As the definition makes clear, most of the structurally deficient or functionally obsolete bridges are not in immediate danger of collapsing or failing. They exact their price on society through delays to commuters and emergency responders, in heavy maintenance costs and in unnecessary suspension damage to the average automobile. Even worse, through the difficulty of predicting the failure of half century old spans, some do fail. These failures include not just the catastrophic failures like that of the Interstate-35 Bridge in Minnesota, but many less publicized but still scary incidents. In 2010, just north of Boston, a deck failure on Interstate-93 sent concrete crashing onto the road below, and caused lane closures for several days (Fast 14: I-93 Rapid Bridge Replacement Project*)*. In 2007, in Boston, trucks were banned and commuters sent scrambling when the Tobin Bridge was revealed to have cracks in a main support beam, just months after debris from the bridge had fallen on boaters below (Taurasi). On September 1st 2012 a section of the Arborway in Jamaica Plain had to be closed for emergency repairs as concrete fell onto traffic below (*wcvb.com)*. The list of minor, but scary failures goes on and on.

It is clear, as stated in the ASCE report plan, that a coordinated and determined effort to maintain and replace bridges, especially highway bridges, is crucial to America's continued quality of transportation. It is an urgent situation, and the sheer scale in terms of number of spans is staggering, and so is the fact that nearly every American is and will be affected by the success or failure of current and future engineers in improving the system. Replacing these bridges, or repairing them, in a cost efficient way with minimum traffic delays and danger is ultimately important. Every option to increase construction speed, lower cost and eliminate traffic delays must be exhaustively evaluated.

1.2 Problem Statement

The bridge on Interstate-93 Southbound going over NH-97 (Pelham Road) has been put on the New Hampshire Red List for bridges that are structurally deficient or functionally obsolete. The bridge was noted to be in serious condition and is scheduled to be replaced in 2012. It scored very poorly on the last published inspection completed November 2009. The deck condition was rated as serious, scoring 3 out of 9 possible marks. The superstructure condition was rated as satisfactory with a mark of 6, and the substructure was rated as poor with a mark of 4. The structural appraisal of the bridge was "Basically intolerable, requiring high priority of replacement." With this evaluation the bridge is structurally deficient and must be replaced. The design and construction of this bridge is especially important given the fact that it supports a major artery in the Northeast (I-93), and averages 14,500 daily commuters as of 2006.

The replacement of this bridge must be done as soon as possible to insure the safety of the commuters, and as quickly as possible to minimize traffic congestion. The fact that this bridge is a vital part of a highway used by so many people only makes it more important to be replaced as quickly as possible. The bridge currently lacks the capacity to effectively handle the traffic flow with only two lanes to carry southbound traffic. It is important to widen the new bridge to four lanes to handle the traffic. A construction plan must be carefully devised to avoid traffic congestion by allowing traffic to pass over the bridge whenever possible and minimizing the duration of bridge closures. The solution must also be economical since the State of New Hampshire has so many highway bridges to maintain.

The construction began during August of 2012, but the majority of the work is to be done from late 2012-2013.

1.3 Objective

The purpose of this project is to design and evaluate alternative bridge designs while mitigating the impact to the surrounding community and restricting traffic as little as possible. The designed bridge must be structurally adequate to handle the traffic and loads related to construction or weather. The design of the bridge must also be able to accommodate current and future traffic volumes. The construction of the bridge must be done using Accelerated Bridge Construction in order to ensure that traffic congestion and bridge closings are kept to a minimum. The design must also be an economical and practical solution. A successful solution will show measurable improvement in delays or cost as compared to traditional construction.

1.4 Scope

The purpose of the project is to identify the most economical bridge replacement plan for I-93 SB over Pelham Road. Background research on different bridge types was conducted to narrow down some practical design considerations. Research on different materials, planning and design processes and construction methods was considered during the project. The two most practical designs were further developed into preliminary bridge designs for evaluation. A specific solution was determined based on the economic feasibility and design features from the preliminary designs.

A full design of the chosen bridge solution was completed, including the substructure and superstructure. This was accomplished while taking into consideration construction methods. Once the design was completed a construction plan and traffic diversion plan were developed. The completed design was then compared to the State’s actual plan for the Pelham Road replacement bridge.

2. Background

The principle concept of the interstate highway system is grade separation. It is what prevents there from being any intersections or traffic lights on the interstates. It would be impossible to maintain highway speeds with constant intersections with surface level roads. Therefore, the high speed road must be kept from intersection with local roads, which creates an interesting engineering challenge. Roads must inevitably cross, so the solution is bridging of either the highway over the local roadways or the local roadways over the highway. This bridging must happen every time a local street and an interstate cross paths, so the sheer volume of required bridges becomes immense. Add the selection of different material and superstructure types, and it becomes clear the dizzying variety of engineering challenges related to grade separation.

2.1 Interstate 93 SB over Pelham Road

One of the forty-six crossings Interstate-93 crossings between Londonderry and Boston is an overpass over NH Route 97, or Pelham road. It is a twin to a very similar span carrying northbound traffic over the same road. This bridge is designated Salem 068/078 by the state, and most people barely regard it as they pass on their way to Boston. The existing span currently carries just two southbound lanes of traffic, with minimal shoulders and no breakdown lanes. One of the lanes is larger than standard to allow for crossover merging for the cloverleaf style exit system surrounding the bridge.

This relatively standard bridge was designed in November of 1959, by the Clarkson Engineering Company of Boston Massachusetts. It was designed for a modified Military loading of a 16 kip wheel load on the deck slab, according to the AASHO 1957 specification, and the NH DPW H. 1954 with 1958 supplement(1959, Clarkson Engineering Company Incorporated). The bridge was constructed in 1961 and has been in service continuously since then. It currently is used by around fifty thousand vehicles a day (NHDOT Construction Plans). The bridge, as seen in an engineering rendering from the west, is pictured below in Figure 1.

Design wise, the bridge contains three spans. These spans consist of rolled wide flange steel girders. End to end, the first span stretches from a mass abutment, to a hammer head pier supported by three circular columns, the second span stretches from one hammer head to pier to another, and the final, third span stretches from the second pier to a second mass abutment.

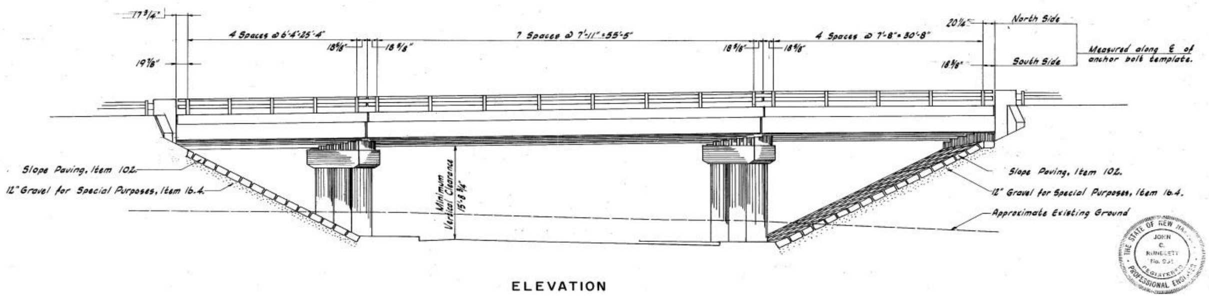


Figure 1: A rendering of the bridge from the original 1959 plans. (Clarkson Engineering Company Incorporated)

The bridge is located in a commercial area. Most of the daily travel on week days is commuter traffic, and a significant number of people work in the area immediately surrounding the exit interchange. On the weekends there is less commuter traffic, but during the summer, there is significant draw to Canobie Lake Park, a local amusement park. Specifically, heading eastbound on Pelham road, there are several industrial complexes, a park and ride station and the amusement park. Heading west, there are many office and light industrial complexes. An aerial view of the bridge and I-93 /Pelham Road is shown in Figure 2. There currently is a significant area of undeveloped land near the bridges due to the old cloverleaf style exit setup.

Figure 2: A satellite picture of I-93 SB over Pelham Rd. (Google Images)

The bridge is currently in poor condition, unsurprising for a fifty plus year old span. There has been significant damage to the concrete cover in the substructure. Water and chloride-related spalling has removed much of the concrete on the top of the hammer head piers and at the bridge seats of the mass abutments. There is also obvious damage to the concrete piers themselves, mostly cracking.

Figures 3 and 4 below show the damage to the bridge seats. Figure 5 shows severe damage to one of the piers supporting the twin northbound span of identical age and design.

Figure 3: Large number of repaired and current concrete failure locations on the southern pier of the SB Bridge. Note the very evident water staining and continued cracking to the bridge seats.

Figure 4: Clear evidence of rust related spalling, reinforcing cage fully exposed in places. This damage is located on the bottom of hammerhead on northern pier of SB Bridge.

Figure 5: Severe spalling, cracking and separation of concrete cover on a pier on I-93 on the Northbound bridge.

The roadway structural deck and coping are also in poor condition. The coping is severely decayed, with reinforcing cage completely exposed in places on the northbound bridge. The deck is shedding the bottom cover concrete, and has been repaired extensively, with a very large repair using "leave in place" forms near the southern abutment. The entirety of the span over traffic is covered with debris netting to reduce the chance of debris harming passersby. The netting can be seen in Figure 6 below and some of the forms in Figure 3 above. The roadway surface has been recoated recently and is in very good condition, so it offers little insight as to the condition of the concrete below. Unfortunately the concrete sections of the bridge are in need of immediate repair or replacement (NHDOT Red list).

In general, upon observation the steel superstructure is in good condition. The paint is beginning to fail in select sections, but significant loss of section in the stringers has not occurred. The safety rail at the top of the bridge is visibly rusted. The actual effectiveness of the bridge is unknown without further examination, which is made difficult by the lack of shoulders and the heavy traffic on the bridge. Some of the damaged paint and decayed coping on the northbound bridge is visible in Figure 7.

Figure 7: Debris netting under the SB Bridge

Figure 6: Paint damage and decaying roadway coping. Picture shows the northern expansion joint on the eastern side of NB Bridge.

2.2 Design of Highway Bridges

As early as the mid 1800's, engineers and designers used standardized methods to increase the design strength of structures in order to reasonably decrease the probability of the structure failures (LRFD Design vs. ASD and LFD). The first standardized method used was called Allowable Stress Design (ASD), which designated certain factors of safety to structures to improve structural reliability despite a variety of uncertainties. This is a very simple approach and can be overly conservative in certain aspects of design but at the same time it does not take into account potential worst case scenarios. A more accurate measure of resistance besides stress had not been developed and the factors of safety in ASD can be fairly subjective numbers.

Load Factor Design (LFD), also known as Ultimate Strength Design, assigns load and resistance factors for different types of loading including dead loads, live loads and impact loads. In this approach load factors are applied to each load combination and different types of loads can have different levels of uncertainty. Unfortunately, LFD still does not assess certain risks to the structure and is more complex and time consuming than ASD.

Load and Resistance Factor Design (LRFD) is probabilistic approach to structural safety able to account for a wide variety of loading conditions, even referring to low likelihood but high impact events. It is also more complicated and time consuming than ASD. Load and Resistance Factor Design also provides acceptable safety parameters while not being over conservative and overdesigning for very unlikely and extreme scenarios. This method is capable of providing a uniform level of safety based on risks and probabilities of worst case scenarios happening concurrently. After year 2007 all bridges in the United States are to be designed using AASHTO LRFD Specifications in order to qualify for federal funding (Tonias & Zhao). The LRFD Specifications are used for design during this project.

2.2.1 AASHTO LRFD and New Hampshire State Bridge Design Specifications

As mentioned above, the design standard for interstate bridge in the United States is currently the *American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specification*. The specification ensures that bridges are "designed to specific limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspect ability, economy and aesthetics." Specifically AASHTO prescribes that every bridge should be designed to meet the force effects due to the defined design loads multiplied by the load modifier η_i and the load factors γ_i. It specifies that a bridge should meet these force effects in each of five strength states, two extreme event states, four service states and two fatigue states. In all of the conditions, any force that has a decreasing effect on another shall be taken at minimum value. For instance, the designer cannot allow wind uplift to reduce the loading forces applied through supports, because it is unreasonable to assume the structure will always be assisted by wind uplift or similar effects. The load combinations are given in Table 3.3.1-1 of AASHTO LRFD Bridge 2012, seen as Figure 8 below.

Table 3.4.1-1—Load Combinations and Load Factors

Load Combination Limit State	*DC* *DD* *DW* *EH* *EV* *ES* *EL* *PS* *CR* *SH*	*LL* *IM* *CE* *BR* *PL* *LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*
Strength I (unless noted)	γ_p	1.75	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength II	γ_p	1.35	1.00	—	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength III	γ_p	—	1.00	1.40	—	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Strength IV	γ_p	—	1.00	—	—	1.00	0.50/1.20	—	—	—	—	—	—	—
Strength V	γ_p	1.35	1.00	0.40	1.0	1.00	0.50/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Extreme Event I	γ_p	γEQ	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II	γ_p	0.50	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.0	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service II	1.00	1.30	1.00	—	—	1.00	1.00/1.20	—	—	—	—	—	—	—
Service III	1.00	0.80	1.00	—	—	1.00	1.00/1.20	γ_{TG}	γ_{SE}	—	—	—	—	—
Service IV	1.00	—	1.00	0.70	—	1.00	1.00/1.20	—	1.0	—	—	—	—	—
Fatigue I—*LL*, *IM* & *CE* only	—	1.50	—	—	—	—	—	—	—	—	—	—	—	—
Fatigue I II—*LL*, *IM* & *CE* only	—	0.75	—	—	—	—	—	—	—	—	—	—	—	—

Figure 8: AASHTO Load Combinations (AASHTO 2012)

The first AASHTO state, Strength I, is a basic load combination, which accounts for normal vehicle live load on the bridge without wind. Strength II is a load combination that accounts for user specific or permit vehicles or any combination of both. Strength III is a load combination relating to an over 55 mph wind acting on the bridge, but with no live loads because AASHTO states it would likely be unsafe for vehicles to be travelling in such high wind. Strength IV is a load case that accounts for very high dead loads in comparison to live loads. It is roughly applicable when the force effect of dead vs. live loads is about seven to one. This load combination can often control during construction. Strength V accounts for normal vehicle live loading combined with a 55 mph wind.

The extreme event cases attempt to provide design protection and consideration to disastrous, infrequent events. Extreme Event I is a load combination that includes earthquake loads. The specific live load earthquake factor must be determined. Typically, a value for this factor of 0.5 is valid for a wide range of truck traffic volumes as specified in the *AASHTO LRFD Bridge Specifications 2012*. Extreme event II is a load combination that considers a variety of disastrous happenings, including floods, vehicle and ship collisions and ice loads. It generally is significant for bridges spanning over water, because shipping collisions have a history of causing significant bridge collapses (Arsava).

The service conditions are designed to relate to the normal operational use of a bridge, with wind loadings. Service I is a load combination relating to the normal operation of a bridge and a 55 mph wind, with all loads taken at nominal values. This case is important for the evaluation of compression in prestressed components. Service II is intended to control the yielding of steel bridge structures and slip critical connections, and is valid only for steel bridges. Service III is a load combination used to analyze tension in prestressed concrete superstructures, and control cracking. Service IV is a load case relating only to control of prestressed concrete columns.

The final two load combinations are used to control fatigue. Fatigue I is a "fatigue and fracture load combination related to infinite load-induced fatigue" (AASHTO 2012). It is used to design a structure for infinite fatigue life, implying that fatigue will not be the limiting factor in the useable lifespan of the bridge. Fatigue II is a load case that allows for fatigue design for a finite number of loads.

Taking under account all of these load cases, it is the responsibility of the designer to decide which cases are relevant to a specific bridge project, according to the wishes of the owner. They must combine the factors and investigate the positive and negative extremes of each load case.

Specifically for roadways, bridges designed under the *AASHTO 2012 LRFD Bridge Specifications* are designed to support the HL-93 live load. This HL-93 live load consists of either a design truck or a design tandem, and a design lane load (AASHTO 2012). Every design lane is viewed as being occupied by either a design truck or a design tandem and a lane load; so a four-lane bridge would have four design trucks and four design lane loads. The governing load is the combination that produces the highest positive moment and shear. This situation is termed the "extreme case" by AASHTO.

For the design of continuous spans and, therefore, design for negative moment, AASHTO specifies an additional design case. This case is two design trucks, with a minimum spacing of 50 feet between them. The spacing between axles in each truck is set at 14 feet. The loads are taken as 90% of the normal lane loads and 90% of the truck loads.

The New Hampshire DOT also provides a highway design code. It is called the *Standard Specifications for Road and Bridge Construction* and contains additional design requirements specific to New Hampshire.

2.2.2 Bridge Superstructure

There are a wide range of structural possibilities when designing a replacement bridge so it is important to be able to recognize a few practical designs to complete an efficient design and analysis. Function is one of the first aspects of the bridge to consider when planning a design. The function of the bridge will determine the design loads and provide an idea of how much support the bridge will require. The project bridge is being redesigned is an interstate bridge so it will need to carry a substantial load. Another important aspect of the bridge is the span length. The Pelham Road overpass spans 150 feet which falls on the larger side of a medium span. Medium span bridges have typically been constructed with prestressed concrete and steel girders. As the materials and design concepts used improve, the bridge spans are more capable of extending to longer distances. The longer the bridge can span, the less substructure is needed for support. This can reduce the total cost of the bridge and reduce the time and effort spent during construction. The fewer members needed to create a bridge, the quicker it can be erected which leads to less traffic congestion. Unfortunately it is also important to consider the increasing material costs, weights and depths involved with longer spans.

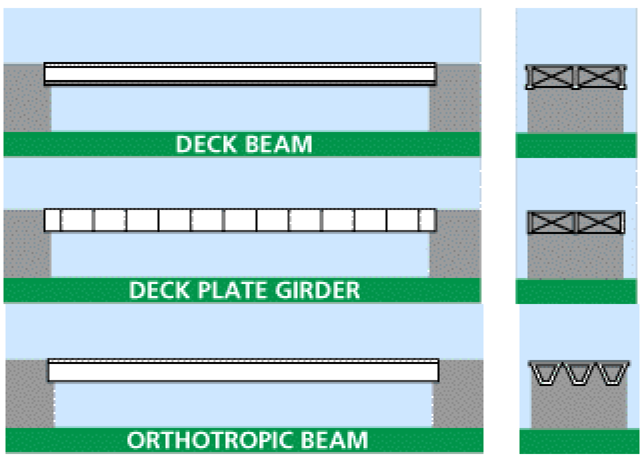


Figure 9: Commonly used beams taken from "Bridge Basics".

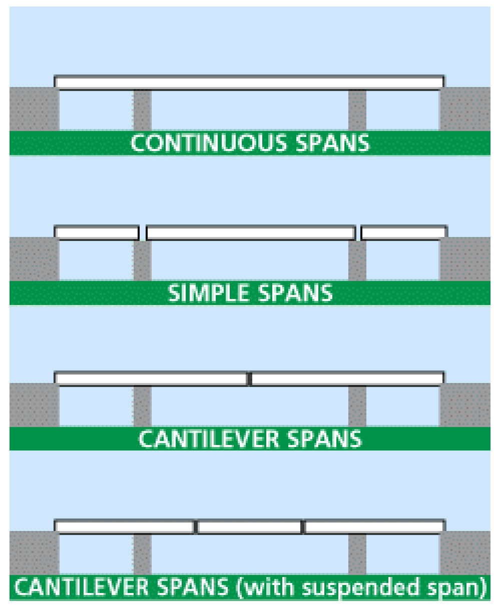


Figure 10: Four types of spans taken from "Bridge Basics".

The structural type of a bridge defines the structural framing system and the type of superstructure. There are four major types of structural framing systems as seen in Figure 10. One of the structural framing systems is a simple span. This consists of a superstructure span containing just one unrestrained bearing at each end. The supports allow rotation as the span flexes under the load. There must be at least one support to keep the span from moving longitudinally. Another type of structural framing system is a continuous span. This consists of one continuous span crossing at least three supports. The member's rotation is restricted in the area next to the pier since it is continuous. The third structural framing system is a cantilever and suspended span. The span is continuous over the pier and ends shortly after creating a cantilever. This cantilever is typically used to support the end of an adjacent span. In this type of system, there is a "suspended" span which relies on the adjacent cantilever spans for support. Sometimes the suspended span rests on an ordinary simple support. The final structural framing system is a rigid frame. Rigid frames are generally used as transverse supports, but can also be used as longitudinal spans. They are called rigid frames because the method of fabrication does not allow relative rotation between members at joints.

There are many different types of superstructures for a bridge. Arch bridges, cable-stayed, and suspension bridges are all bridge types that would typically be large spans, and they are too involved and costly for a medium span interstate bridge. Such complicated construction techniques are simply unnecessary for a typical span. A truss is not a good choice either, and not just because it is historical form of construction. Steel trusses were originally economical because they saved on material, and required more but smaller pieces. These same features are why truss bridges are no longer economical for even large spans, because material is relatively inexpensive today but the enormous amounts of labor in truss fabrication, construction and maintenance is not. Truss bridges as a rule are also less safe, because they tend to be non-redundant, have a large number of connections that can fatigue, and have smaller members that are easily damaged by rust or traffic. Labor costs can also quickly eliminate stone or cast-in-place reinforced concrete bridges. Having a hundred laborers onsite is not going to produce profit in 2012, with the emphasis on speed and consistency.

Therefore, despite the great variety of bridge choices, the pressure of economics generally limits medium span highway bridges to a choice between or combination of two options for superstructure material. The first is material is precast, pretensioned concrete beams or other structural shapes, made offsite at a factory. The second material is the venerable classic, steel I-shaped beams. The vast majority of new bridges of similar spans to the project bridge utilize one of these two materials. There is also a great deal of experience and literature in the construction industry involving these two types of bridge structures.

2.2.2.1 Precast Pretensioned Bridges

Precast concrete has a long history of use in transportation applications. The histories of precast concrete and prestressed concrete for use in bridges are thoroughly intertwined. Prestressed concrete takes advantage of increased strengths by using tension elements to create compressive stress in members. This compression is induced by either pre or post tensioning, and allows the members to support much higher loads. This in turn, allows for much smaller cross sections than regular cast-in-place concrete to support the same loads. This is important for bridges with their long spans and persistent clearance issues.

Cast-in-place concrete can be prestressed onsite, but the extra tensioning equipment, in addition to the typical forms and false work would quickly make for a costly operation (PCI Manual). It would be for all purposes impossible to do this for a replacement bridge. Therefore, precast is married to prestressed in bridging applications, with members arriving fully cured and ready for installation. The first American bridge constructed using prestressed concrete was in 1949 in Philadelphia, Pennsylvania, and now the precast-prestressed concrete industry accounts for roughly 50% of new bridges in the United States (PCI Manual).

Structurally, precast varies by bridge span length. For very short spans, precast slabs, or voided slabs are used, sometimes as part of a rigid (three sided) bridge system. These techniques can be used up to spans of forty feet. For mid-span bridges, there are a variety of concrete shapes available such as a multi stem, double stem or channel setup, with economical spans to around sixty feet or so. For the longest spans, those stretching to a hundred feet or more, there are typically three options. First is single stem, featuring a very large beam shape that tapers to the edges of the deck. The second is a box beam style; a wide, deep box shaped element that supports the deck directly above it. The final type of long span precast and one of the most often used for highway projects is the bulb tee or I beam-shaped member. Often known as the “New England Bulb Tee” these are concrete members that look similar to rolled steel members in cross sectional shape.

States choose to use precast for a variety of reasons. One can often be cost. Depending on the region, precast bridges are competitive on initial cost, with other types of bridge structures. Once life cycle costs are considered, precast can sometimes become even more attractive. Precast concrete tends to have better resistance to damage than CIP concrete and does not need to be painted as does traditional steel. Precast tends to have very low net stresses due to the prestressed nature of the structural members, as well as a very massive cross section. This produces savings as bridge decks and mounting components wear out more quickly on more flexible steel bridges (PCI Manual). Precast bridges also come in a variety of aesthetic shapes and colors, and are more easily adapted to assume very unusual shapes. It can simulate a wide variety of finishes at low cost.

Precast is often more easily manufactured than structural steel, and there are more local producers of the precast structural elements than those for steel (PCI Manual). This is because the amount of specialized equipment to create precast is usually less than that to create structural steel. For example, an August 2007 issue of *Modern Steel Design* details the modern steel casting process. The article focuses on a steel plant in Arkansas that sprawls over 860 acres and has 850 employees that “tend exploding furnaces as big as brownstones”. The electric arc furnace shown in Figure 11 is one such large, dangerous machine. Production of precast is easier in that the effect of economies of scale tend to force steel production to occur in widely spaced, gigantic factories. For precast production, only a mold shown in Figure 12, tensioning, batching, moving and curing equipment is required all of which is often portable. In fact, many of the largest precast projects actually establish temporary plants onsite to save on transportation costs, something which will never occur with structural steel. Precast orders, especially those taking advantage of standard or common shapes such as Bulb Tees can often have a fraction of the lead time of structural steel. This can be of great use in ABC or in emergency situations.

Figure 11: Electric Arc furnace used in the production of structural steel taken from http://www.aisc.org/content.aspx?id=3786

Figure 12: Prestressed I-beam forms taken from http://www.aisc.org/content.aspx?id=3786

2.2.2.2 Steel Stringer Bridges

Structural steel has been an important material in the construction of bridges for well over 100 years. In fact the first time steel was used in bridges was the Kymijoki railway bridge in Finland. It was the first 3-span steel truss bridge and was built in 1870 (*History of Iron and Steel Bridges*).

A multitude of designs are now used for various bridge spans. Some common steel bridge types include plate girder, box girder, or rolled beam bridges. The plate girder is comprised of a vertical plate, which has top and bottom flanges welded to it to create an I-shaped beam. They are generally optimized for bridges with spans of 125 feet or more if the girder spacing is between 11 and 14 feet. Box girders include welding a top plate to a C-shape beam to create an enclosed box and can be used under similar circumstances. Rolled beam bridges are generally used for short spans up to 100 feet and continuous spans up to 120 ft. The span lengths are restricted due to limitations of the steel mill. Issues with transportation of beams also govern the longest continuous beams available.

Steel bridges can prove to be very desirable due to their ability to provide high capacities with low material weights. In fact weight optimization is one of the most important aspects of efficient steel bridge design. Designing optimized members can improve efficiency if employed correctly. Often times the lightest weight or most optimal shaped member is not a standard size provided by common steel mills. Therefore it is important to devise an efficient design while also keeping in mind the most common member sizes.

The main disadvantage of using structural steel for bridge design is the increase in cost from that of reinforced concrete or pre-cast concrete. The unit pricing for structural steel as specified by the NHDOT unit pricing is $1.90. Life span and required maintenance can also be factors in the total cost of the project. Various coatings and paints or weathering steel can be used to greatly increase life span and reduce the overall cost of repairs (McCormac). Structural steel bridges can be very effective in various situations though the scarcity of steel mills in close proximity for some regions, and the span restrictions of bridge elements can make steel construction more expensive than alternative methods.

2.2.2.3 Composite Beam Design

For years steel beams were used to support concrete slabs without taking advantage of composite effects. The benefits of using the concrete slab and steel beams together in resisting loads compositely have been shown in recent decades. Together steel beams and concrete slabs can support 33 to 50 percent or more load than steel beams alone in non-composite action (McCormac). Composite construction for highways bridges was accepted and adopted by AASHTO in 1944, and the use of composite bridge floors has rapidly increased since 1950 (McCormac).

The slab in a composite section acts as the compression portion of the combined beam. The composite section makes use of the high compressive strength of the concrete slab and the high tensile strength of the steel. Overall, composite beams are more efficient at resisting loads than simple steel sections. Composite sections have greater stiffness than non-composite sections and also have deflections from 70 to 80 percent less (McCormac). Less steel is needed which can make the beam lighter and less expensive.

Steel shear anchors must be installed in order to connect the concrete slab with the steel beams. The shear anchors are designed to resist all shear forces between bridge slabs and beams. Several types of connectors are used, but the most economic have been round studs welded to the top flanges of the beams (McCormac). The studs are rounded steel bars with a rounded head on top to help keep the concrete and steel together and force the composite section to act as a unit. Other steel anchors which can be used include spiral anchors and channel anchors; these are displayed in Figure 14.

Figure 13: Prefabricated deck sections being placed over supports, image taken from FHWA website.

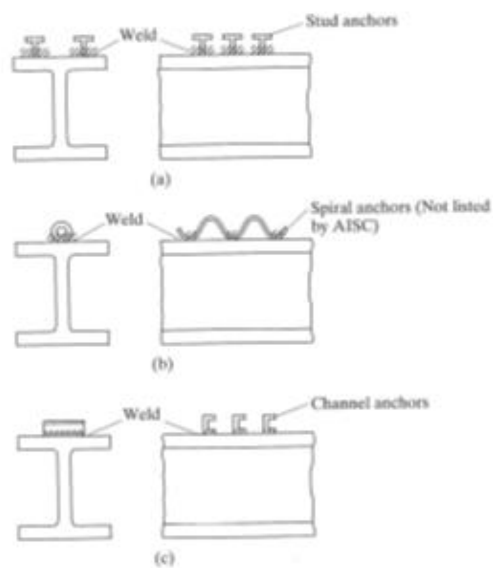


Figure 14: Displays different type of shear anchors used in composite beams, image taken from "Structural Steel Design" (McCormac).

In order for the steel anchors to bond with the concrete to create the composite beams, the wet concrete must be placed over the anchors and allowed to cure. This can cause constructability issues for projects which must be completed in a short period of time, and the wet concrete adds to the weight which the beams must support during construction. In order to minimize these issues, innovative techniques have been used to limit the construction time and increase the constructability of the bridge. Prefabricated sections of concrete slab with holes to make room for steel anchors can be transported to the site and intermittently placed along sections of the bridge as shown in Figure 13 from FHWA. High strength concrete can then be placed in between sections and over the steel anchors to create the bond between the steel and concrete slab. This is a relatively new technique used to quickly and efficiently erect a bridge.

Another more generally practiced technique is to prefabricate portions of the span with the slab already cured on the beams. In order to limit the amount of the structure necessary to lift and transport, sections generally consist of two beams connected by a portion of the cured section on top. These sections are then installed and connected with high strength concrete. This method allows the beam-and-slab system to provide full composite action during construction in addition to adding to the constructability of the bridge.

2.2.3 Foundations

The substructure is one of two main parts of a bridge. The purpose of the substructure is to transfer the loads of the bridge to the supporting ground below. The loads from the superstructure are transferred to the bearing plates which then transfer the load to the foundations, which are supported by the ground. The types of foundations used are greatly dependent on the site geometry and soil strength. Large spread footings are preferred because they are shallow foundations which do not require much excavation, and they distribute the loads over a larger area. In more extreme conditions it may be beneficial to have some deep foundations which have a smaller footprint and penetrate to deeper levels of the earth, offering more stability and support.

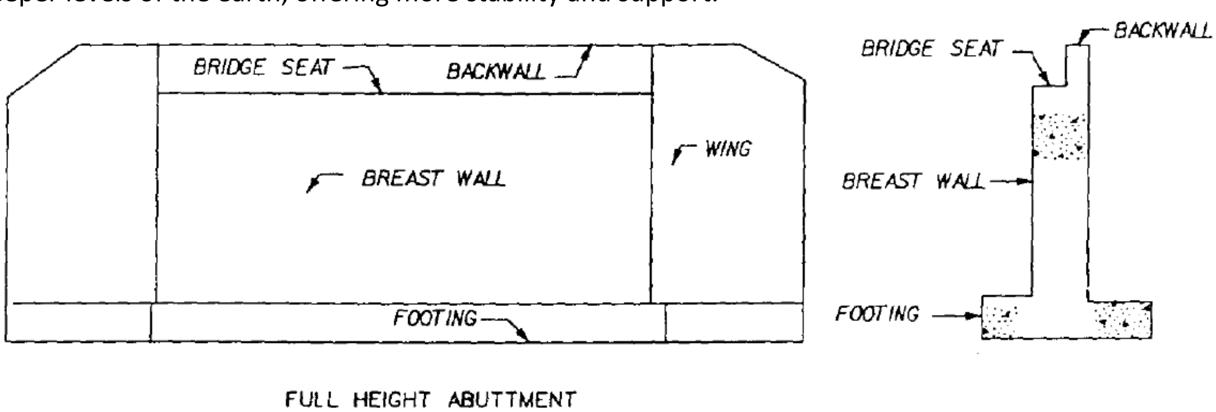


Figure 15: Standard abutment (Bridge Inspection, Maintenance, and Repair)

The common foundation types for bridges are abutments, piers, and bents. The abutments are the structures which offer support at each end of the bridge. Abutments are either constructed with plain concrete, reinforced concrete, stone masonry or a combination of concrete and stone masonry (*Bridge Inspection, Maintenance, and Repair*). They will be the only foundations for a single span bridge and consist of five parts: a footing, a backwall, a stern or breast wall, a bridge seat and a wing wall as shown in Figure 15. The backwall is a small retaining wall which extends up from the abutment. Its purpose is to prevent the embankment soil from spilling on to the bridge seat and they must provide the necessary clearance between the ends of the bridge span and the face of the backwall to allow the bridge to expand and contract (*Backwalls*). Wing walls are the retaining walls adjacent to the abutment responsible for keeping the embankment soil around the abutments from spilling into the waterway or roadway being spanned by the bridge (Childs). The bridge seat is an indent on the top of the abutment which contains the bearings on which the superstructure sites. This is the area where the end of the span will be supported and connected to the abutment. The breast wall is the retaining wall directly under the span. It is used to prevent the embankment soil from spilling over and maintain the structural integrity of the embankment soil by providing horizontal stability. The footing is what anchors the abutment in the ground and is typically a very wide shallow foundation.

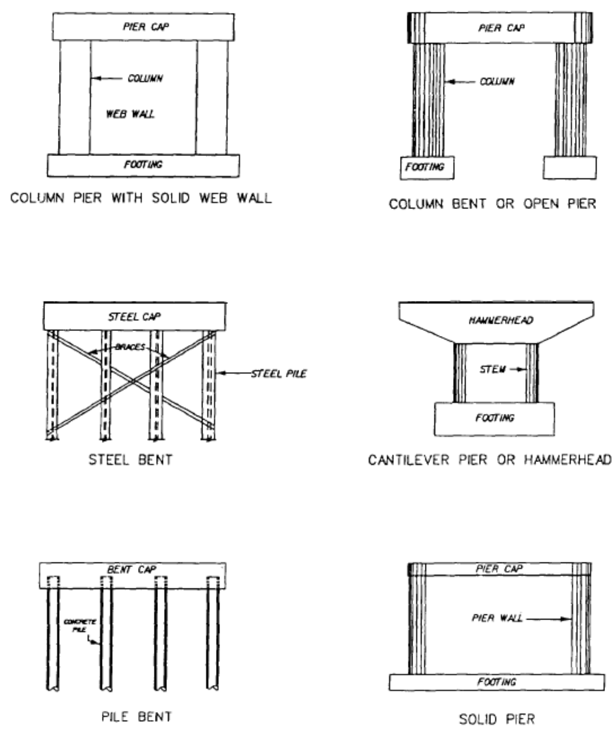


Figure 16: Typical piers and bents (Bridge Inspection, Maintenance, and Repair)

Piers are used as supports between the two abutments and generally consist of footings, columns or stems, and caps as shown in Figure 16. A highway overpass can use a variety of foundations. Some designs consist of a single span only supported by abutments, while others require one or more other types of foundations between the abutments. The footings are generally slabs at the bottom of each column or stem which transmit the load into the deeper layers of soil or rock. The footings must be designed to resist the vertical load and moment from the superstructure, if the slabs cannot adequately resist the load additional deep foundations are required. Some of these deep foundations may be piles, caissons, or drilled shafts which penetrate deeper into the soil and possibly bedrock to secure the footing. The columns or stems transmit the applied loads and moments to the footings and must meet strict size and clearance requirements. The cap of a pier takes on the loads from the superstructure and transmits the loads to the columns or stem. The pile cap will bind the columns together and create a rigid structure if the pier consists of multiple columns. Bents are piers which penetrate the ground but don't have footings. Piers and bents are typically made of steel, concrete, stone, or a combination of the materials.

2.2.4 Pavement

Pavement is generally one of two materials used for the actual road surface in the United States. Pavement usually consists of either aggregate with asphalt binder, or some form of concrete. An asphalt paved road varies in thickness depending on traffic and soil conditions, but is generally between six and eight inches thick for newly placed highways (Mallick, El-Korchi). Concrete pavement is generally thicker than asphalt and can be reinforced in one of three ways: unreinforced, reinforced at panel joints or continuously reinforced through its whole length. Due to the vast number of freeze-thaw cycles and the heavy application of road salts, northern areas tend to use more asphalt paving.

An asphalt road consists of various layers with different grades of asphalt as shown in Figure 17. Generally the sub base, or underlying ground is compacted or replaced, then a base layer of structural stone or sand is layered and graded. On top of this coat comes a first thick layer of base course, asphalt with large aggregate and lower asphalt content. Following this comes at least a layer or two of medium asphalt, with a final wearing course at the top of about 3" of the highest quality asphalt (Mallick, El-Korchi).

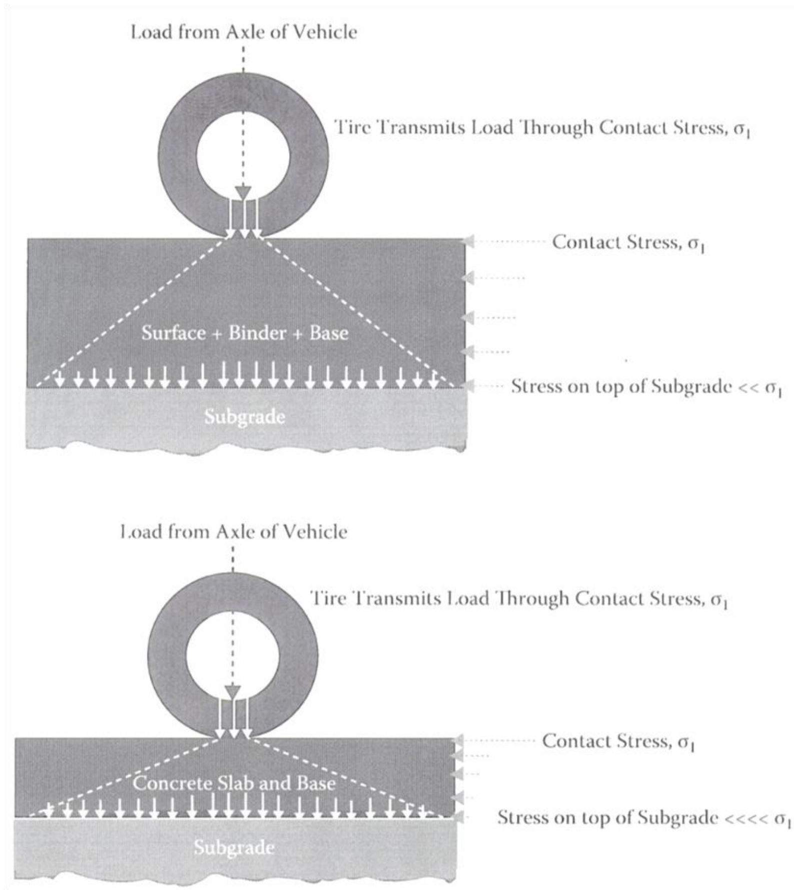


Figure 17: Illustration of wheel loadings on asphalt and concrete paving taken from *Pavement Engineering Principles and Practice.*

The topmost layers are the best quality because pavement is designed according to stress cones from wheel loads. Stress is most concentrated in the top and dissipates as it moves towards the bottom, allowing the use of lower cost materials in lower layers. Software and equations enable engineers to model the stress at different points beneath the roadway. They can then pick the least expensive material that meets the stress criteria. The top wearing course must also meet friction requirements to allow braking and maneuvering, and be able to withstand weather and UV damage.

An asphalt based bridge paving consists of several parts. First the concrete deck must be in-place on the bridge and cured enough to support the weight of construction equipment and the eventual wearing course. Then the surface is cleaned and dried to a high standard of uniformity, to preserve bonding for the water sealant. The water sealant is a heat welded heavy duty tar-fabric, similar to the material used in flat roofing. This is crucial to prolong the structural life of the concrete deck by protecting it from water and de-icing chemicals. Within a few days, or immediately if possible, the deck is paved by the HMA bridge pavement. This is applied either by the normal transfer vessel, paver, and roller combination. The bridge pavement itself is typical wearing course grade pavement. This means it must meet the same requirement as a typical top layer pavement, and also that it applied with the most precision, and therefore typically at a much slower rate than base type asphalt courses.

This bridge pavement serves several purposes. It provides a level of water proofing protection similar to how a normal roadway pavement prevents water saturation of the sub base beneath it. The pavement protects the waterproofing membrane from both traffic and UV based degradation, increasing the time it provides protection to the deck. Finally, the bridge pavement, as compared to an unpaved concrete wearing surface, prevents traffic wear of the deck concrete directly.

2.2.5 Safety

The bridge design must include bridge railings classified as longitudinal barriers. The bridge railings must meet full-scale crash-test criteria and the Federal Highway Administration (FHWA) codes and regulations. Typically, barriers can only be selected from a variety of crash worthy shapes. Crash worthy barriers are barrier types with defined specifications that have already been crash tested successfully by the Federal Highway Administration. These barriers have been used traditionally for one of several purposes. First, barriers are generally used, if necessary, to prevent cars from exiting the roadway into: steep slopes or vertical drops, active construction sites, or other traveled ways. Barrier types range from aluminum rails, to steel wire, to concrete “Jersey” barriers.

Barriers are also often used to protect important infrastructure components from vehicle collision, and to protect vehicles themselves from collision with infrastructure components. If piers are used on an interstate highway project, they will generally be surrounded by Jersey Barriers both for the safety of the driver and to protect the structural integrity of the foundation. Barriers can also be used to protect cranes, temporary shoring, or permanent but crash damageable bridge components, such as low hanging beams or truss members.

Additionally, for roadway safety, care is taken in any bridge design to follow typical state and federal highway codes. These codes include among many other things, rules governing sight distance as it relates to stopping distance, and limits on curves, speeds, grades, and lane specifications. These conditions are also directed towards the design speed of the roadway. The purpose of the specifications is to provide some assurance that even the most unsafe, typical car with an average driver can safely traverse the new bridge and roadway, both during construction and during the service of the bridge. In the extreme, the codes prevent sudden lane drops, sharp curves or dangerous merge areas on highways. These limits can often complicate temporary traffic routing (NHDOT).

2.2.6 Design Aids

2.2.6.1 PG-SUPER

PG-Super is prestressed bridge design software. It is the most widely used precast, prestressed bridge software based on the AASHTO LRFD specifications in the world (http://www.pgsuper.com/). It was initially developed as a combined effort of the Texas Department of Transportation and the Washington Department of Transportation to aid in the design of prestressed bridges. It has a vast library of AASHTO and state standard concrete sections. These sections include box girders, concrete I-girders, a variety of Bulb Tee designs, and others.

A user of *PG-Super* must specify design strengths for the concrete and for the steel strands, bridge geometry, as well as the design methods and assumptions. The software can accommodate multiple spans, and skewing of the entire bridge or a specific span. Finally the user will specify the type of girder from a library file, the desired girder spacing, and also the load cases, service conditions, and specific AASHTO loads for design evaluation. At this point, a user can design one girder at a time or all of the bridge girders simultaneously. If the strand inputs are left blank, the program output will define the number of strands and harped strands required in each girder. It will also render the bridge according to the specified spacing, geometries and slopes.

In addition to designing the strands for precast members, the program also conducts the necessary load analysis. From the user defined load cases, *PG-Super* calculates the maximum moment (known in AASHTO as the extreme case) for a variety of possible situations, and creates a report indicating coordinates and pass/fail for each individual case. The output also provides design moments acting on each girder in the bridge individually, and charts of the various moments due to different load cases on that member. For a user designed member, *PG-Super* will calculate the design moment capacity and compare it to the calculated moment due to loads.

In summation, *PG-Super* is an excellent engineering aid for the design of precast, prestressed bridges. It can be used to develop a comprehensive design, complete with deck and beam reinforcing steel quantities. Results can also be used as a base component for design details, or the program can be used to check hand calculated beam and bridge designs quickly against the full range of AASHTO loads and load cases.

2.2.6.2 ESpan 140

ESpan 140 is a free web based design tool developed by the Short Span Steel Alliance (eSpan 140). *ESpan* can be used to create a quick design for a steel solution for a simply supported bridge with a span of no more than 140 feet. The design is created based on various inputs such as number of traffic lanes, roadway width, average daily traffic, and design speed. The outputs that are obtained from *ESpan 140* include designs for rolled beams, plate girders, corrugated structural plate, and corrugated steel pipe. The program also presents durability solutions for the bridge design.

ESpan 140 can be used to obtain a complete bridge superstructure, or as a preliminary design estimate for various bridge aspects such as beam size and spacing.

2.2.6.3 RISA-2D

The demonstration version of *RISA-2D* is a simple structural analysis program that assisted in the static analysis of the structure (RISA-2D). The structure may be analyzed as a simply supported or continuous beam in the program with the appropriate loads applied as specified by the *AASHTO LRFD Design Specifications*. The static analysis is then run, and *RISA* generates shear and moment diagrams displaying maximum values. The shear and moment diagrams may also be viewed interactively by selecting a point along the horizontal axis of the diagram and obtaining the shear and moment at that specific point on the beam. In addition to the benefits offered by the software, *RISA* was selected for its ease of use and convenience of revising previously analyzed structures. Loads are easily adjusted for different loading scenarios and coefficients can be assigned to specific loads to create multiple load cases. The multiple load cases are particularly helpful when evaluating the different Strength conditions specified by AASHTO.

2.2.6.4 NY DOT Math Cad LRF D Cantilever

The NYSDOT cantilever abutment analysis tool is a free file provided by the New York State Department of Transportation. The analysis worksheets are based in Mathcad, which is a calculation organization and data analysis program (PTC Mathcad). The NYSDOT website offers a file that "contains Mathcad version 14 worksheets that analyze and design cast-in-place abutments and wing walls in accordance with the *AASHTO LRFD Design Specifications"* (NYSDOT). The worksheet produces a full AASTHO limit state abutment and wing wall design based on the input of various values such as material strengths, dimensions, soil properties, loading conditions, and load magnitudes.

2.2.6.4 Other Design Tools

The library of tools a bridge designer has at his disposal is extremely relevant to the accuracy and promptness of his bridge design. Other, non-software based resources project play a crucial role in the development of designs. *The Prestressed Construction Institute Precast Bridge Manual* (*PCI Bridge Manual*) provides assistance in preliminary precast design with examples and charts. The industry standard estimation book, *R.S. Means*, provides cost estimation values and methods. The AISC Steel manual provides design information and specifications for rolled steel members. Two textbooks, *Bridge Engineering* by Tonias and *Structural Steel Design* contain very useful design examples and explanations. *Microsoft Excel*, as spreadsheet software, provides a means of easy and adjustable calculation of design parameters.

2.3 Construction Plan

2.3.1 Traditional Construction Plan

In the United States, an owner or agency proposes a bridge for construction. An external design firm or the state bridge design division reviews the job and advertises it to the public for contractors to bid on it. The owner, in the vast majority of cases a state or the federal government, accepts the least costly eligible bid.

The first issue that needs to be resolved before any work can begin is that of funding. Next, the proper standards and references are chosen based on the location of the project and the type. Once all the preliminary requirements are met the design can begin. For existing bridge sites, inspections must take place to observe the present condition. The site can then be inspected for various features such as drainage conditions, vertical clearance, soil condition, horizontal and vertical alignment constraints, underpass or channel constraints, and potential utility impacts.

The design of any bridge involves implementing various substructure and superstructure elements. The different components such as the abutments, footings, piers, beams and girders are designed using AASHTO's *Standard Specifications for Highway Bridges*. Design loads are implemented to determine the correct capacities for the bridge elements.

Traditional construction plans allow fewer restrictions to the building materials that can be used during construction. Materials such as cast-in-place concrete can be used more effectively with conventional construction due to the more relaxed timelines and use of detouring and lane closures.

During construction various safety precautions have to be enacted to protect motorists and workers. This can include using detours, implementing lane closures, or even constructing the bridge in a new location and then altering the existing roadway alignment to incorporate the new bridge.

Once the construction is complete, and the bridge is in service, it is general practice to setup maintenance schedules to help promote the durability of the structure. This can ensure that the structure has a long life span and will not require costly repairs in the future.

2.3.2 Accelerated Bridge Construction Plan

Over the past few decades Accelerated Bridge Construction (ABC) has emerged as an effective alternative to conventional bridge design and construction. ABC involves employing efficient planning and design, along with innovative materials and construction methods to substantially reduce the total construction time of new or replacement bridges. This can be very desirable when considering the replacement of bridges that are vital to the flow of the traffic system.

By implementing ABC in a given project, substantial improvements to various consequences of construction can be observed. For instance time delays due to traffic impacts and weather can be lowered. Additionally, by applying ABC, impacts to traffic flow can be greatly reduced. This is because ABC involves minimal to no traffic detouring, no temporary bridge structures, and no change to the existing roadway alignment. Also, due to the efficiency of the process, ABC can greatly reduce the environmental impact of new or existing bridge construction.

An important aspect of ABC is the use of Prefabricated Bridge Elements and Systems (PBES) (Culmo, Michael P.). As the name suggests, PBES are structural elements that are manufactured off site under controlled settings. The elements are generally pre-cast concrete, composite pre-cast and steel, or steel systems, and can be used for all structural components including decks, beams, piers, and abutments. PBES have their advantages and disadvantages. An advantage is the higher quality of the materials when using PBES which is due mainly to the environment in which the elements are made. The facilities are generally set up so that temperature, humidity, and weather conditions are controlled. As a result of the higher quality of the materials, PBES can provide improvements in safety, quality, and long-term durability.

The main disadvantage of ABC with PBES is the increase in cost of construction. Companies who have completed initial ABC projects with PBES have seen increases from 10% to 30% in construction costs (Culmo, Michael P). The main factors that affect the increase in cost include the size of the project and the tight construction time limits. Obviously a bigger project will lead to higher costs due to the greater amount of PBES needed. A tight construction time limit also increases costs because more work needs to be done in less time. This could involve including more engineers for the design, more workers to ensure the structure is erected in a timely manner and more effort by the contractor to create a very efficient construction schedule.

There are many factors that can affect the need for the use of ABC. An effective way of deciding if ABC is an appropriate alternative to conventional bridge construction is to look at how much the mobility impact time will be improved. The *ABC Manual* defines mobility impact time as any period of time the traffic flow of the transportation network is reduced due to onsite construction activities. The list below demonstrates the different tiers of mobility impact time.

Tier 1: Traffic Impacts within 1 to 24 hours

Tier 2: Traffic Impacts within 3 days

Tier 3: Traffic Impacts within 2 weeks

Tier 4: Traffic Impacts within 3 months

Tier 5: Overall project schedule is significantly reduced by months to years

If the use of ABC lowers the tier number associated with a proposed project it is useful to conduct an analysis to determine if the use of ABC is appropriate.

Accelerated Bridge Construction has proven to be an important asset over the years. As was seen in the “Fast 14” bridge replacement project, ABC can greatly increase the efficiency of a proposed job. The Fast 14 involved replacing 14 deteriorating bridges on I-93, north of Boston. The use of Accelerated Bridge construction made it possible for the job, which could have taken as long as 4 years under traditional methods, to be completed in just 10 weekends. To keep the project on schedule, strict timelines were enacted to ensure maximum productivity and minimal traffic delays. For instance the *Engineering News-Record* states if the four lanes of I-93 did not open by 5 am on any given weekend the joint venture would have lost $3.23 million for the first minute alone but if the bridges were completed in the predicted 10 weeks, $7 million of incentives would be given. This technique of offering incentives and disincentives proved to be very effective in the Fast 14 project and can be used with future projects to increase efficiency.

2.3.3 Cost of Delays

Many of today's construction projects consist of rebuilding structures in urbanized areas or performing maintenance on structures that many people depend on to accomplish everyday tasks, such as commuting to work. Closing down high volume, frequently used structures and roadways can have a serious effect on the commuter, surrounding businesses, and the community.

Vehicle miles travelled (VMT) on U.S. highways have doubled in the past three decades, while the miles of highway has only increased by 5% during the same period of time (Mallella and Sadasivam). The cost congestion incurred by the commuters for travel delay and extra vehicle fuel has risen from an estimated $24 billion in 1982 to $115 billion in 2009 (Mallella and Sadasivam). To keep up with the pace of growing congestion, the funding for highways has increased by 300% through Federal, state, or local sources in the past three decades (Mallella and Sadasivam). As a result of this increased funding, there have been many new construction projects to repair, improve and widen existing roadways. These construction projects have been encouraged to use innovative construction methods which limit traffic and safety implications and use the time spent obstructing traffic flow as efficiently as possible.

One method of limiting traffic congestion is making use of "A+B" bidding. "A+B" cost estimation and bidding gives the construction company incentives to maximize efficiency and minimize the time a roadway is shut down. The "A" component represents the base cost of the project if it were to be completed using a traditional construction method. The "B" component represents the cost of the impact to the public. The projects are encouraged to schedule operations to maximize efficiency. Some ways to maximize efficiency are to decrease idling time during construction, have multiple activities occurring at once if they don't have to be done sequentially, have all the equipment ready for when it is needed, have as much of the material as possible prefabricated to expedite the construction process, and have major construction done at hours with the least amount of traffic flow. Some incentives given are paying workers extra to work overtime or double shifts and bonuses for completing a project ahead of schedule.

The Work Zone Road User Cost (WZ RUC) is the additional costs borne by the motorists and the community as a result of the work zone activity (Mallella and Sadasivam). The WZ RUC consists of the monetized components of the implications of the work zone such as the user delay costs, vehicle operating costs (VOC), crash costs, and emission costs. Other off-site components like noise and impacts to the business and local community can also be taken into consideration. Many of the impacts are displayed below in Figure 18. These components, especially the off-site components can be extremely difficult to monetize because requires establishing utility functions to put a quantitative value on something qualitative like noise. The steps for computing the road user cost are as follows: (Mallella and Sadasivam)

1. Gather data for work zone impact assessment
2. Estimate work zone impacts
3. Compute unit cost for each impact type
4. Estimate WZ RUC components

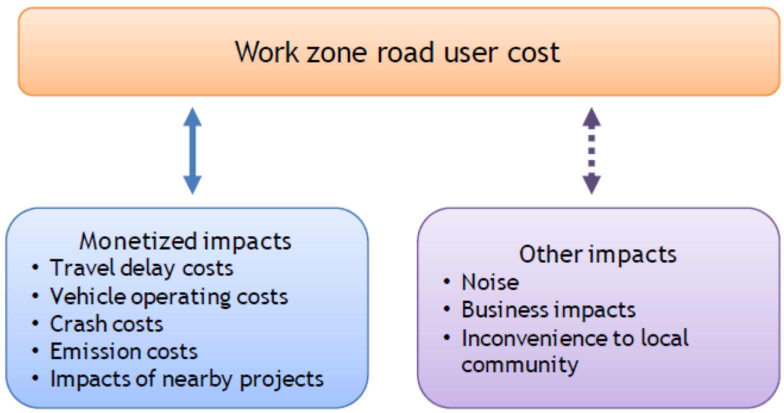


Figure 18: Illustration of different aspects of the WZ RUC (Mallella and Sadasivam)

The monetized impacts are calculated by multiplying the estimated delay time by the unit cost of travel time. The delay time is quantified by the additional travel time needed to pass through the work zone or detour around it. The unit cost of travel is quantified as an hourly rate taking into consideration the value of personal travel, business travel, truck travel, time related depreciation of the vehicle, and the value of freight inventory for loaded trucks. Once the unit cost data is determined, the number of vehicles must be determined. The number of vehicles multiplied by the time delay and unit cost data is used to obtain the WZ RUC.

2.4 Traffic Plan

There is no general equation that allows a civil engineer to “plug and chug” his way to a complete traffic diversion plan. Each site and situation has too many unique combinations of variables and concerns to make a single approach universal. The Federal Government outlines the regulations for roadway signage and traffic diversion in the *Standard Highway Signs and Markings* publication it produces and updates from time to time. This standard includes many theoretical situations to assist engineers, and lays out the signage, merge and other traffic requirements. It also contains pictures of typical roadway signs and plan view diagrams of signs placement for various examples. The general goal of traffic diversion and signage is to protect the safety and health of both construction workers and roadway users, with the secondary goal of maintaining as much of traffic flow as possible.

An engineer has many examples available to choose from. In general for a major operation such as a bridge replacement, one of two methods will be followed. The first possibility involves the temporary re-routing of traffic. This would be implemented for a longer construction time, and include temporary roads and or bridges. A strategy like this is often used to allow the contractor to build a bridge more slowly in place. This strategy could also include closing some of the lanes in the construction area without adding a diversion, although that can have drastic consequences during peak travel times. Negatives of diverting or narrowing traffic include increased costs and traffic flow issues in general. Constructing a temporary bridge or detour lanes may not even be possible if there is limited space near to a construction area.

A second method of road construction would involve complete or partial shutdown, on a temporary basis. These shutdowns are typically short, and last from a few minutes to a weekend. Shutdowns are often used with Accelerated Bridge Construction, where a night or weekend of furious construction can replace years of slight delays. Obviously traffic must be detoured, so shutdowns must occur with advance notice at non-peak times. Traffic would be detoured to other existing roads without any new construction of temporary infrastructure.

2.5 Stormwater Management

Stormwater runoff is produced by precipitation and it picks up pollutants from ground surfaces. These pollutants are then carried into lakes, streams, and other surface waters and impact the surrounding environment. This is a major concern regarding the construction of the bridge I-93 over Pelham Road because Porcupine Brook, a designated Salem Prime Wetland passes through the construction site (Pelham Road Mitigation Site: Site #31). Construction sites are also exceptionally vulnerable to releasing pollutants during rainfalls since the soil is loose from excavation and the site lacks vegetation to absorb and stabilize pollutants. Some of the pollutants most commonly found near highways and construction sites are nutrients such as nitrogen and phosphorous, sediment, pesticides and fertilizers, petroleum and other chemicals ("What is Nonpoint Source Pollution?").

Sites below 10 acres are not monitored for nonpoint pollution, but must follow Best Management Practices (BMPs) specified by the Environmental Protection Agency (EPA) if the site is greater than 1 acre (Construction Site Stormwater Runoff Control). Towns are required to monitor and record specific information pertaining to stormwater runoff and report this information to the state and EPA. Some of the BMPs include properly storing materials such as oil, paint and gasoline to limit the repercussions caused by spilled chemicals. Other solid and liquid waste also must be contained and disposed of properly. A stormwater management plan must be devised and storm drain inlets must be installed before construction. It is strongly encouraged to use some type of storm drain inlet protection. One option would be to surround or cover storm drains with a material that would allow the sediment to be filtered out before the water leaves the construction site. Some commonly used types of storm drain inlet protection are surrounding the inlets with silt fences, sand bags or gravel. It is also important to stabilize the entrances and exits of the construction site. Sediment and chemicals are often tracked into and out of the construction site on the tires of cars and trucks. This pollution can be reduced by having the entrance and exit ways made of rock or gravel. All loose soil must be covered to reduce the amount of sediment washed away during storms.

Accelerated Bridge Construction (ABC) can help reduce the amount of nonpoint pollution by reducing the time of construction, therefore limiting the amount of time loose soil is left exposed and vulnerable to being washed away. The shortened construction time will also decrease the chances of rain storms occurring during construction.

3. Methodology

The design of highway bridges is an immensely involved and complicated task. An engineer must consider many factors during any design process, but this is especially the case for bridges. Bridges can be constructed of many different types of materials, in different framing styles, and in vastly varying sizes and lengths. Bridges must withstand some of the worst environmental and load conditions of any commonly built structure. Bridges must also handle great volumes of vehicular and pedestrian traffic without failure.

Fortunately, a state of the art design practice will take advantage of the styles and tools outlined in the background sections above. The AASHTO bridge code represents a significant and nearly comprehensive design specification for highway bridges. There are respective state and federal codes dictating nearly every aspect of a bridge design. A successful process will meet these constraints while also using both electronic and information-based design aids to maximize personnel efficiency. The codes and design aides allow an engineer to select and design a bridge as one of or as a combination of the many styles discussed in the background.

The design of the I-93/Pelham Road bridge is then by definition also complex, with many phases of development. In each phase, from choosing a strategic bridge design to pursue, to the final deliverables, this project attempted to use appropriate, reasonable methods of design that align with standard engineering practice. For structural calculations, the overlying concept was Load Resistance Factor Design (LRFD). The structural and highway geometry design was compliant with both State and AASHTO specifications for highway bridges. The bridge was designed by modern, standard methods.

The initial phase consisted of design studies for alternatives considering both structural steel and precast concrete superstructures, to attempt to identify reasonable spans, spacings and beam configurations. The second phase consisted of further developing the initial steel and precast superstructure designs, and comparing them to each other in order to determine the best bridge option based on a variety of factors. The third phase consisted of the development of the selected preliminary design into a final and complete bridge design. The final phase of the project took the completed design, developed a set of construction plans, a final A+ B project estimate and an evaluation of the success of the project as compared to a traditionally constructed bridge. The proposed solution was benchmarked against the actual I-93/Pelham Road replacement bridge,

3.1 Procurement of Site and Background Information

State plans for the ongoing replacement of the I-93 SB bridge over Pelham road were acquired (NHDOT Construction Plans). These plans not only contain the State's design for a replacement bridge but also a significant quantity of site, soil, traffic, and roadway profile data. Most of the site data comes from these plans, as they have very recently been assembled, and represent a trusted source. Information about the condition of the current bridge has been acquired from an onsite visit, and many pictures were retained for future consultation. Information on removal and original design conditions came from a scan of the original archived bridge plans for the bridge constructed in 1961. Information regarding design specifications and state-specific requirements was obtained from the NH DOT website, while the *NH Bridge Manual* is only a guidance tool used to supplement the code.

General site information was collected using satellite imaging from Google maps. A site visit provided additional reference as to the nature of the development along Pelham Road and Interstate I-93. Observation also provided qualitative assessments of the condition of the current overpass system, the nature of traffic volumes, and the failures of the current roadway and bridge geometry.

3.2 Selection of Superstructure Types to Pursue for Preliminary Design

3.2.1 Structural Steel Bridge

For initial design of the steel bridge, the two most common forms of steel girder were considered for two different bridge geometries: a single span and a two-span continuous. Although different steel shapes could be used for the various spans, only I-shaped, rolled or built up members were considered in order to reduce the total number of options. To acquire a rough estimate of the beam sizing and spacing required for the bridge superstructure, the *eSpan 140* software was used. Appendix 2.2 shows the *eSpan 140* results for the two different spans. The most important value gathered from *eSpan 140* was the beam spacing. The software filled a role in selecting beam spacing that may be filled by engineering experience or proprietary charts at a design office. The program gave a starting point for the design of the superstructure of the bridge.

3.2.2 Precast Bridge

Availability of precast concrete sections and time constraints dictated that pre-design research was limited to typical precast shapes. The deck bulb tees, box beams, and AASHTO I Beams were all evaluated. Precast splicing is generally a complicated undertaking which, for the purposes of simplicity, limited the length of a precast span in this project to what can be produced and shipped in one length. The general maximum varies by beam type, but according to the Executive Director of PCI northeast on a recent campus visit, the largest beams ever delivered in the northeast topped out at 140 feet (Seraderian). This means that most of the precast bridge superstructures evaluated had to have at least two spans to meet the required 150 foot span. Two design options did explore the possibility that design constraints could be lifted if there was significant economy in single span construction. However, the majority of the designs studied were directed at two-span precast bridges.

Using the preliminary design tables in the *PCI Bridge Design Manual*, each of the three types of beams was analyzed as to which sections specifically could meet load requirements. The *PCI Bridge Design Manual* charts were designed for the selection of members and strands using the HS-25 truck loading, a loading not equal to HL-93 but a significant approximation that preliminary design could still benefit from the charts. A chart similar to that shown in Figure 19 below was used to select the subsets of a beam type worthy of consideration. For example, in Figure 19, it shows that a 65” deck bulb tee has sufficient capacity to bridge a 150’ span at a beam spacing of about six feet center to center.

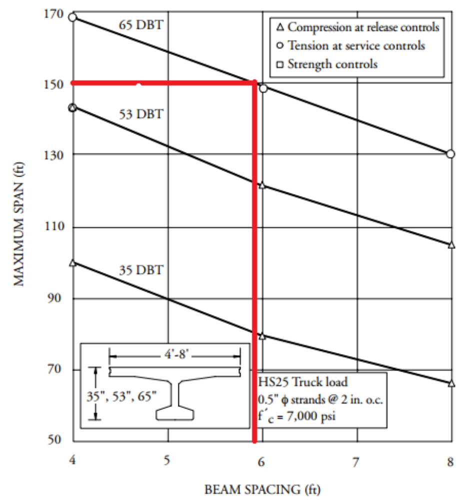


Figure 19: Table from PCI Bridge Manual relating the capacity of DBT to Span and Beam Spacing

Sample beams were sized using additional *PCI Bridge Design Manual* charts for each of the 12 specific beam sizes that could work between the three evaluated types. The 12 specific beam sizes represent the beam sizes where the span fell within the curve, without a smaller section also being sufficient at that same spacing. For example if the span had been only 100 ft. in Figure 19 above, the size option selected for the DBT would have be the 55” DBT, because the 65” DBT would be too conservative and the 35” DBT would not have enough capacity. This selection process for the various precast configurations produced the 12 specific candidate beam sizes. The typical type of chart used to construct preliminary strand requirements is shown below in Figure 20.

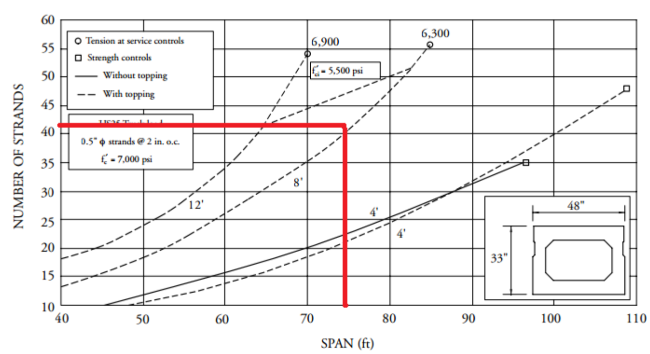


Figure 20: Table from PCI Bridge Manual relating the capacity of an ASHTO 48” Box Beam to the estimated number of strands required according to beam spacing and spanned length

The 12 beams were then evaluated for expected cost based on unit costs and clearance requirements. A spreadsheet was constructed of unit areas, strand counts, and required decking, bearings and additional supports between abutments (piers). The final cost for each beam type consisted of the price of beams, based on installed unit price of structural concrete; the cost of the required bearings by the cost per bearing; the pier cost by number of piers required; and the cost of any additional concrete deck required besides that which is provided by the structural members themselves. The calculations of a typical pier cost and of the decking and bearing cost considerations are explained below in the section titled Development of the Design Considerations for Preliminary Design.

The unit costs came as both prices per unit of material for the members, and as values for fully installed work which were taken from a similar highway bridge project (NHDOT 13933N). The material costs were simply used for reference and only the estimated total bid costs were used for comparison. Different unit prices were used for the concrete in the superstructure members, the concrete deck and the concrete piers (when necessary). Of the 12 precast beam sections considered, the best beam type and span alignment was the one that not only met all requirements economically, but also passed engineering judgment tests as to practicality and delivery and erection feasibility.

3.3 Development of the Structural Steel Preliminary Design

The structural steel members designed were rolled steel W sections and plate girders. Spreadsheets, including simple calculations, were made in order to design composite and non-composite members for simple spans. The members were designed, according to LRFD specifications, to meet the flexure and shear requirements set forth in chapters F and G of the *AISC Specification*. The deck thickness was constant, and was initially chosen based on the industry standard. This value was later adjusted to obtain suitable composite member capacities. The total length spanned by the bridge was 150 feet, and the girder spacing was set to be uniform in order to simplify calculations and improve constructability. The combination of AASHTO and AISC specifications was implemented to expedite preliminary design process. The *AASHTO LRFD Bridge Specification* is an extremely complex document that is not often used in undergraduate classrooms, whereas many students are familiar with parts of the AISC specification. Specifically AASHTO loads were used for the development of all designs, but AISC resistance concepts and LRFD dead and live load factors were used to develop lateral and vertical shear preliminary steel designs. For the final design of winning superstructure type, the only specification considered will be the *AASHTO LRFD Bridge Specification*.

3.3.1 Distribution Factors

Section four of the *AASHTO LRFD Bridge Design Specification* was the principle reference for the loads and load distribution for structural analysis and evaluation of the preliminary bridge designs. An important tool that was obtained from *AASHTO* section 4.6.2.2.2 was the distribution factor method for moment and shear analysis of simply supported bridge decks. In accordance with *AASHTO* Table 4.6.2.2.1-1, shown in Figure 18, a typical cross-section was chosen. Figure 21 shows a portion of Table 4.6.2.2.1-1 which categorizes the bridge design to determine which distribution factors to use.

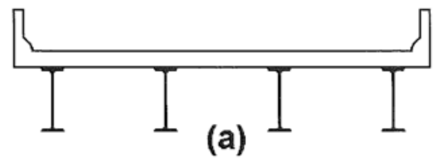
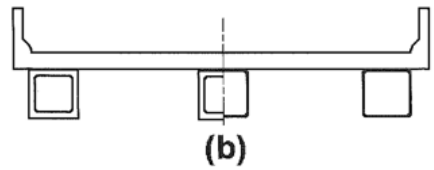
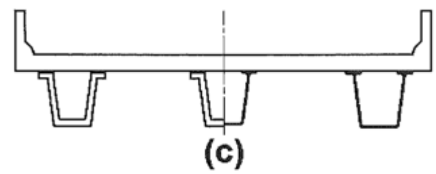
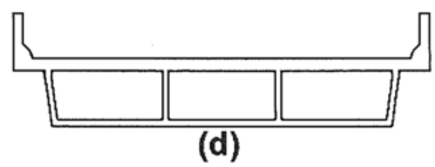
Supporting Components	Type Of Deck	Typical Cross-Section
Steel Beam	Cast-in-place concrete slab, precast concrete slab, steel grid, glued/spiked panels, stressed wood	
Closed Steel or Precast Concrete Boxes	Cast-in-place concrete slab	
Open Steel or Precast Concrete Boxes	Cast-in-place concrete slab, precast concrete deck slab	
Cast-in-Place Concrete Multicell Box	Monolithic concrete	

Figure 21: Common Deck Superstructures taken from *AASHTO LRFD Bridge Specifications* 2012

3.3.2 Moment Distribution Factors

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\, g_{interior}$ $e = 0.77 + \frac{d_e}{9.1}$	$-1.0 \le d_e \le 5.5$
			use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

Figure 22: Distribution of Live Loads for Moment in Interior Beams taken from *AASHTO LRFD Bridge Specifications 2012*

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	Distribution Factors	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	One Design Lane Loaded: $0.06+\left(\frac{S}{14}\right)^{0.4}\left(\frac{S}{L}\right)^{0.3}\left(\frac{K_g}{12.0Lt_s^3}\right)^{0.1}$ Two or More Design Lanes Loaded: $0.075+\left(\frac{S}{9.5}\right)^{0.6}\left(\frac{S}{L}\right)^{0.2}\left(\frac{K_g}{12.0\,Lt_s^3}\right)^{0.1}$	$3.5 \le S \le 16.0$ $4.5 \le t_s \le 12.0$ $20 \le L \le 240$ $N_b \ge 4$ $10{,}000 \le K_g \le 7{,}000{,}000$
		use lesser of the values obtained from the equation above with $N_b = 3$ or the lever rule	$N_b = 3$

Figure 23: Distribution of Live Loads for Moment in Exterior Beams taken from *AASHTO LRFD Bridge Specifications 2012*

The categorized superstructure type was then used in conjunction with *AASHTO* Tables 4.6.2.2.2b-1 for interior beams and 4.6.2.2.2d-1 for exterior beams. The equations provided by these tables are shown in Figures 22 and 23 for interior and exterior beams respectively. The factors obtained by the equations were then multiplied by the maximum LRFD design moments produced from the governing vehicle and lane loads; these products determined the design moments for the interior and exterior girders.

3.3.3 Shear Distribution Factors

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck on Wood Beams	l	Lever Rule	Lever Rule	N/A
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T-and Double T-Sections	a, e, k and also i, j if sufficiently connected to act as a unit	$0.36+\frac{S}{25.0}$	$0.2+\frac{S}{12}-\left(\frac{S}{35}\right)^{2.0}$	$3.5 \le S \le 16.0$ $20 \le L \le 240$ $4.5 \le t_s \le 12.0$ $N_b \ge 4$
		Lever Rule	Lever Rule	$N_b = 3$

Figure 24: Table 4.6.2.2.3a-1 of *AASHTO LRFD Bridge Design Specifications*

Type of Superstructure	Applicable Cross-Section from Table 4.6.2.2.1-1	One Design Lane Loaded	Two or More Design Lanes Loaded	Range of Applicability
Concrete Deck, Filled Grid, Partially Filled Grid, or Unfilled Grid Deck Composite with Reinforced Concrete Slab on Steel or Concrete Beams; Concrete T-Beams, T- and Double T-Beams	a, e, k and also i, j if sufficiently connected to act as a unit	Lever Rule	$g = e\, g_{interior}$ $e = 0.6 + \frac{d_e}{10}$	$-1.0 \le d_e \le 5.5$
			Lever Rule	$N_b = 3$

Figure 25: Table 4.6.2.2.3b-1 of *AASHTO LRFD Bridge Design Specification*

Obtaining shear distribution factors was a similar process to that for obtaining moment distribution factors. Tables 4.6.2.2.3a-1 and 4.6.2.2.3b-1 of the *AASHTO LRFD Bridge Design Specification*, shown in Figures 24 and 25 respectively, provided the equations for the determination of the shear distribution factors for interior and exterior beams. The factors were then multiplied by the maximum LRFD design shear values produced from the governing vehicle and lane loads to determine the amount of design shear that the interior and exterior beams must be sized to carry.

3.3.4 The Lever Rule

The lever rule was used to determine the moment and shear distribution factors for the exterior girders of the bridge superstructure. The deck was assumed to be a rigid structure in order to determine the portion of the load that the exterior girder carried. A static analysis was set up using the transverse cross section of the superstructure. Figure 26 shows a view of the factors used for the lever rule during design. The portion of the cross section was taken from the edge of the slab to the second adjacent beam. The exterior beam was assumed to be a fixed support while the interior beam was treated as a hinge for the purposes of calculating the distribution factor.

Based on the geometry of the bridge superstructure, the outermost wheel load was placed 2 feet in from the exterior beam. The other half of the axle load was placed 6 feet away from the outermost wheel load as specified by AASHTO. The moment was taken about the interior beam/hinge to obtain the distribution factor multiplied by the weight of the design vehicle.

Figure 26: Conceptual view of components used for the Lever Rule

3.3.5 Preliminary Non-Composite Beam Design

The designs of the non-composite girder-and-deck systems for the one and two-span bridges were completed in accordance with chapters F and G of the *AISC Specification (2011)*. All non-composite members were designed with compact webs and flanges to simplify the check for lateral-torsional buckling. The LRFD load factor method was used in the preliminary steel design for simplicity. For the two-span design, the girder experienced positive and negative moments on different portions of the span. Because of this, the continuous girder was split up into three beam lengths based on the nature of the moment. The two end girders were designed as simply supported members under positive moment, as was the girder for the single span bridge. The center girder extended between two inflection points and was designed for negative moment.

All non-composite members were designed as built-up girders to ensure the most efficient design for the ultimate shear and moment values. For each member, two designs were completed: an interior and exterior girder design. The larger member size from the two designs governed the overall bridge design. A spreadsheet was created to aid in the design of the members for non-composite action. The girders were sized by changing the design parameters input into the spreadsheet. All the equations were linked in the spreadsheet to ensure that the output values constantly updated with any changes to the inputs. An example of input and output information for the interior design portion of the two-span, built-up girders under negative moment is shown in Table 1.

Table 1: Input and output information for non-composite beam design.

Non-Composite Beam Description			
Shear Design			
Input		Output	
bf	19 in	Ec	5072.24 ksi
tf	1.13 in	As	51.62 in^2
hw	26.7 in	Steel Weight	0.176 k/ft.
tw	0.325 in	Slab Weight	1.093 k/ft.
Es	29000 ksi	Wearing Surface Weight	0.340 k/ft.
f'c	7 ksi	Wu (DL Factored)	1.930 k/ft.
fy	60 ksi	h/tw	82.15
kv	5	Minimum tw	0.323 in
Span Length	75 ft.	Cv	0.54068
Effective Length	40 ft.	n	5.717
Tributary Width	9.71425 ft.	Ix	520.08 in^4
Slab Thickness	9 in	eg	18.98 in
Wearing Surface Thickness	3 in	Kg	109286.77
Exterior Factor	0.485	Interior Factors	0.74857
Multiple Presence Factor	1.2		0.93249
LL Moment (service)	1248.2 k-ft	Vu	149.16 k
LL Shear (service)	74.1 k	ΦVn	152.01 k

3.3.6 Preliminary Composite Bridge Design

As part of the preliminary steel design, continuous and single span composite girder-and-deck systems were completed. Each design was prepared using LRFD specifications with only factored dead loads and live loads for preliminary design purposes. The continuous span contained a pier located at the middle of the span (75 ft.) and three cantilever beams spliced together. The span is represented in Figure 27 with three fixed supports and splices joining the beams.

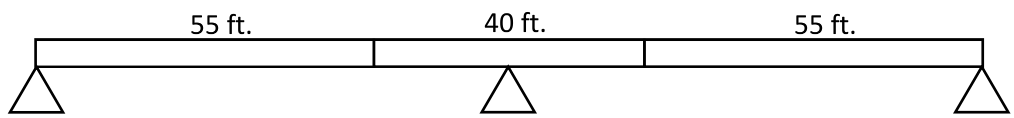


Figure 27: Continuous composite bridge span with three fixed supports and two splices.

The locations of the splices were determined in order to keep the two end beams in the positive moment region; these splices were designed to transfer moment. The two end beams were then designed as composite beams for positive moment, and the center beam was designed as a girder for negative moment. An advantage of this design is the bridge can be split up into sections and a composite design is not used in the negative moment region where a composite beam would not be effective. Another advantage to the design is the shorter beams are easier to transport. The outer beams can extend 55 feet from the end supports before experiencing negative moment, and the center beam can extend approximately 40 feet before experiencing positive moment which respectively determined the lengths of the beams.

The two end beams of the bridge were designed as exterior and interior composite beams using their corresponding distribution factors. The more conservative of the two was then used for the design. These beams were designed as standard rolled sections and also as plate girders in a spreadsheet so values could change as the design of the beams changed. The beams were designed to achieve full composite action in order to design the most efficient cross section; however, some of the larger sections needed to be evaluated as partially composite because the slab could not balance the full tensile capacity of the steel girders' cross-sectional area. The location of the plastic neutral axis (PNA) was determined for these beams in the spreadsheet and the structural adequacy was determined. The steel girders were sized based of dimensions of rolled sections, using a guess-and-check method in the spreadsheet. As the cross sections of the beams changed, the moment capacity and other values in the spreadsheet updated themselves. An example of input and output information for the rolled section design of the interior beams is shown in Table 2 below.

Table 2: Input and output information to design composite sections.

Composite Beam Description			
Moment Design			
Input		**Output**	
b_f	9 in	β_1	0.7
t_f	0.68 in	b_e	58.2855 in
d_w	22.54 in	Distance to edge	36.0015 in
t_w	0.44 in	a	3.229539 in
A_s	22.4 in^2	c	4.6136271 in
I_x	2850 in^4	E_c	5072.2406 psi
Beam spacing	116.571 in	W_{beam}	0.0762222 k/ft
Slab thickness	9 in	W_{slab} (interior)	1.0928531 k/ft
N_b	8	$W_{Wearing}$ (interior)	0.3521416 k/ft
f_c'	7000 psi	DL (interior)	1.5212169 k/ft
f_y	50,000 psi	M_{DL} (interior)	575.2101 k-ft
Span length	55 ft	Interior factors	0.470312
Bridge length	150 ft		0.6474116
Span width	74 ft	e_g	16.45 in
E_s	29,000 ksi	n	5.7173944 in
$W_{barrier}$	0.425 k/ft	ϕM_n	1624.159 k-ft
Exterior factor	0.485	M_u	1615.688 k-ft
Multiple presence	1.2		
M_{LL} (service)	893.4 k-ft		

3.3.6.1 Standard Section Design for Moment Capacity

The cross sectional information of the rolled sections was taken from Edition 14 of *AISC Steel Manual*. The beams were listed in the spreadsheet and organized by the plastic section modulus about the major axis (Z_x) in order to quicken the convergence of the guess-and-check method. The W-shapes that fulfilled the moment and shear requirements were then noted, and the steel section requiring the least amount of steel was selected for the preliminary design.

Once all the beam and span information was entered into the spreadsheet, the effective flange width (b_e) was determined based on span length, beam spacing, and distance from the edge of the slab for exterior beams. The dead loads were then determined for exterior and interior girders through consideration of the dead loads of the barrier, beam weight and tributary area of the slab and wearing surface. The dead load determined was then applied to the girder as a uniformly distributed load to determine the moment. Since the placement of the design vehicle live loads involves a series of concentrated forces, the maximum moments caused by the dead and live loads typically occurred at different locations along the spans. An accurate approach to obtain maximum moment would investigate the resultant moment diagrams obtained from dead loads plus the various placements of the live loads. A more conservative approach was taken by adding the maximum moment from the dead load with the maximum moment from the live load. This was deemed acceptable for the preliminary design since the maximum moment caused by the live load was relatively close to the center of the beam. A check was done to determine the difference between the maximum moment caused by the dead load at the center of the span for the design of the standard sections and at the point where the live load maximum moment is located. The moments differed by approximately 1%.

The multiple presence factor and distribution factors were then obtained in accordance with the *AASHTO Specifications*. The exterior beam distribution factor was determined using the Lever Rule and the multiple presence factor was obtained from AASHTO for girders with one lane of loading for the lever rule. Both of the interior distribution factors were calculated and the larger of the two was used. The larger factor between the interior distribution factor and the exterior distribution factor multiplied by the multiple presence factor was determined to be the governing distribution factor. With the dead and live load moments and the factors established, the ultimate moment (M_u) was then calculated and compared to the moment capacity (ϕM_n). Steel sections were designed for vertical and horizontal shear in addition to moment.

3.3.6.2 Plate Girder Design for Moment Capacity

The built-up girders were designed with the aid of a spreadsheet using the same techniques as the rolled sections. The input information only differed by the fact that the cross sectional dimensions were adjusted accordingly. The dimensions were calculated in order to adequately resist moment and shear while trying to use as little steel as possible. The spreadsheet then checked the dimensions of the plate girder in accordance with the *AASHTO Specifications* for web and flange dimensions per sections 6.10.2.1 and 6.10.2.2 in order to maintain standard proportions.

3.3.6.3 Shear Anchor Design for Horizontal Shear

Steel headed studs or anchors were selected for their strength and economical attributes for all of the composite designs. The horizontal shear was first calculated as equivalent to the compressive force in the slab of the fully composite beam. The design process consisted of selecting a cross sectional area of the shank of the anchors (A_{sa}) as an input to the anchor's shear strength. The AISC equation for the normal shear strength of an anchor is shown below.

$$Q_n = 0.5A_{sa}\sqrt{f'_c E_c} \leq R_g R_p A_{sa} F_u \qquad \text{(AISC Equation 18-1)}$$

The R values were taken from a chart in *AISC Specification* 18.2a and then the anchors were designed to see how many were necessary to obtain adequate strength. Once the number of anchors required was determined, the necessary spacing was calculated. The calculations were completed in a spreadsheet; Table 3 displays the results of this spreadsheet for the continuous composite beam design. Only the two end portions of the continuous span were designed for composite action so no shear anchors were designed for negative moment in the center portion.

Table 3: Input and output information for design of shear anchors.

Horizontal Shear Design			
Input		**Output**	
A_{sa}	10.5 in^2	V_u	2240 k
R_g	1	Studs	144
R_p	0.75	$\sum A_{sa}$	1512 in^2
F_u	60 ksi	Spacing	4.583 in

3.3.7 Vertical Shear Design

The steel members in this report were all designed for vertical shear in accordance with Chapter G of the *AISC Specification*. The nominal shear equation used was:

$$\Phi V_n = 0.6 * F_y * A_w * C_v \qquad \text{(G2-1)}$$

In the above equation the variables were F_y (yield stress of steel), A_w (area of the web), and C_v (web shear coefficient). The web shear coefficient was dependent on the ratio of the web height to thickness (h/t_w). For doubly and singly symmetric shapes, C_v was determined by adhering to the following limits:

- For webs of rolled I-shaped members with $h/t_w \leq 2.24\sqrt{(E/F_y)}$
  - $\Phi_v = 1.0$ (LRFD)
  - $C_v = 1.0$ (G2-2)
- When $h/t_w \leq 1.10\sqrt{(k_v*E/F_y)}$
  - $C_v = 1.0$ (G2-3)
- When $1.10\sqrt{(k_v*E/F_y)} < h/t_w \leq 1.37\sqrt{(k_v*E/F_y)}$
  - $C_v = [1.10\sqrt{(k_v*E/F_y)}]/(h/t_w)$ (G2-4)
- When $h/t_w > 1.37\sqrt{(k_v*E/F_y)}$
  - $C_v = 1.51*k_v*E/[(h/t_w)^2]*F_y$ (G2-5)

The shear-buckling coefficient (k_v) was needed in order to solve for the web shear coefficient. The first step was to determine if the web needed stiffeners. The web did not need stiffeners if $h/t_w \leq 2.46\sqrt{(E/F_y)}$, or if the available shear strength for unstiffened webs was greater than the required shear strength. For the purpose of simplifying the preliminary design, all beams were designed to perform adequately without stiffeners. The *AISC Steel Construction Manual* specified that:

For webs without transverse stiffeners and with $h/t_w < 260$

$k_v = 5$

With all the variables in equation G2-1 defined, the nominal shear strength (ΦV_n) was determined. The nominal shear strength was then compared with the ultimate shear in the steel member from the LRFD load study. The design of the member was checked, by confirming that the nominal shear capacity was sufficient for the ultimate shear produced from the load study.

3.4 Development of the Precast Concrete Preliminary Design

The precast concrete design began with the best concrete design option determined from the precast design study, the Deck Bulb Tee. The key insight provided by the design study was a particular type of shape, and the relative total depth that would be reasonable and possibly capable of carrying the loads calculated during a full analysis. The prestressed design program *PG-Super* was used to perform the preliminary design of the precast system with the Deck Bulb Tee.

Upon opening the program, the user is confronted by a selection of various typical members available in the default library. The W35DG shape was selected, the Washington Department of Transportation standard file for a 35" deep deck bulb tee. While the Washington Department of Transportation has little jurisdiction in New Hampshire, their deck bulb tee specifications represent a relative standard size. The selection of this file is show in Figure 28 below.

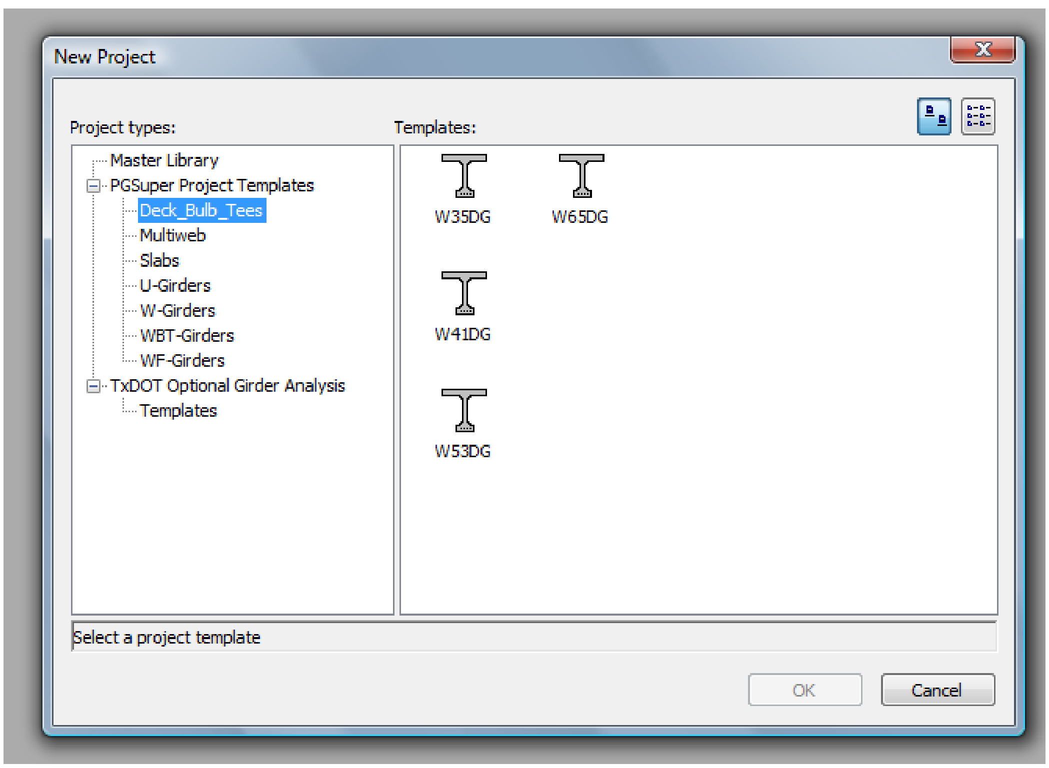


Figure 28: Prestressed Shape Library, PG-Super

Once the initial girder type was selected, various conditions relating to the bridge were input into the program. This included information on widths, beam spacing, spans, piers, alignments, coordinates and desired connection types at supports. A sample information entry window is shown below in Figure 29. Data was also entered into the software as to the load types and trucks to be considered, as well as the load cases to be considered, and to define the methods of analysis.

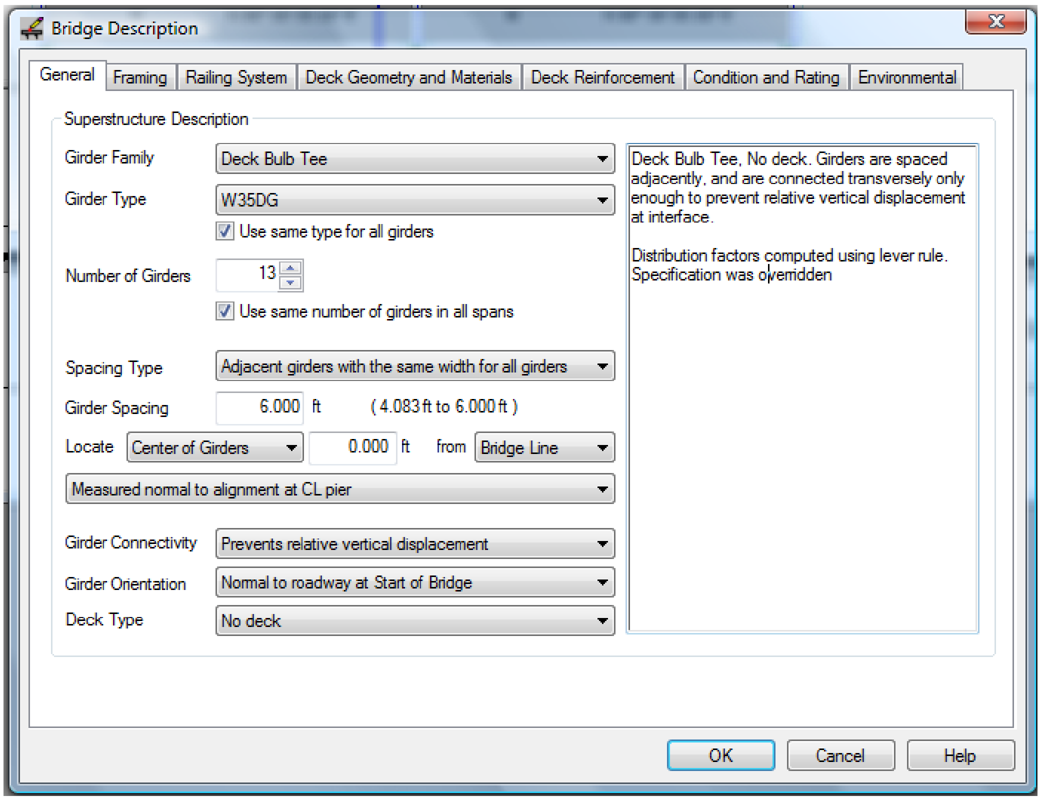


Figure 29: Information Entry window in PG-Super.

From the information entered, the program designed the amount of reinforcement and the locations of the reinforcement within the specified girder type. Using the design girder command, all 13 girders in span 1 and all 13 girders in span 2 were designed for all applicable AASHTO load cases, with strength and service being of primary importance for preliminary design. The software determined the number of strands required to meet the calculated strength requirements, and checked the moment capacity of the designed members to meet the loads. The output presented a variety of moment charts for each load case for each girder, as well as an interactive chart containing the nominal girder strength for comparison. A sample of the bridge cross section results from the program is shown in Figure 32. A sample of the program strand design within a girder is shown in Figure 31. Finally, a sample chart of moment capacity vs. the loading from various load states is shown in Figure 30.

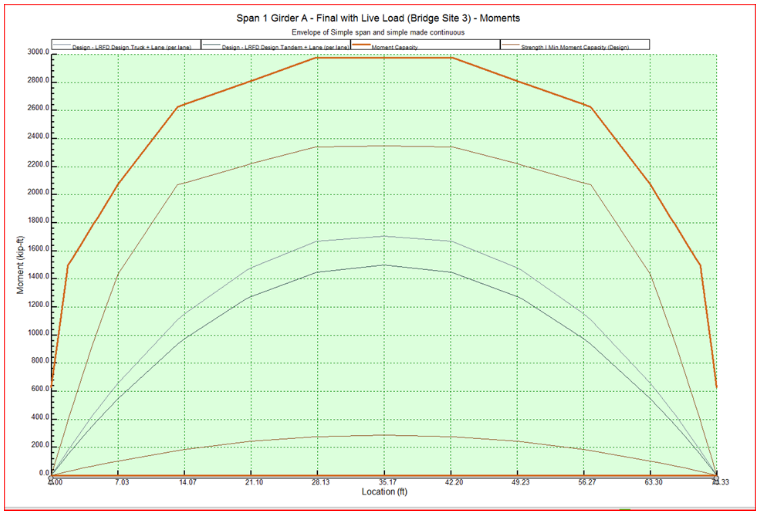


Figure 30: Chart of nominal moment capacity vs. various design moments.

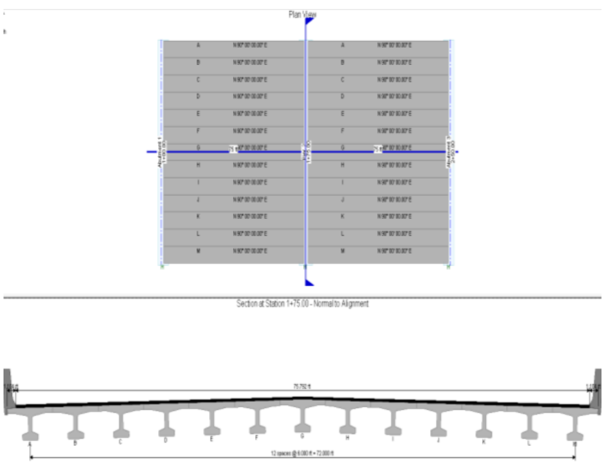


Figure 32: Bridge cross section sample.

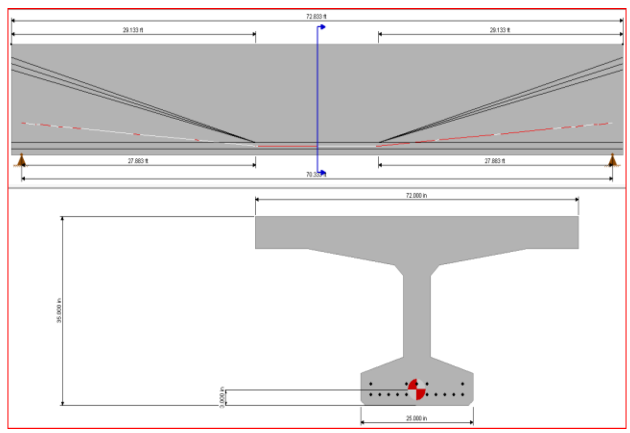


Figure 31: Cross sectional girder view with strands.

The final design approved by *PG-Super* and constructed from input data from the *AASHTO LRFD Bridge*, the *NH Bridge Manual*, and from the precast design study represented the final preliminary precast design. This preliminary design is complete only for the purposes of comparison with other designs for selection as a final design types. The output from *PG-Super* was checked using appropriate manual calculations according to the *AASHTO LRFD Specifications* to confirm the final design of a bridge. The results were not trusted for final design without this in-depth check.

Design quantities for estimation of the cost of the precast concrete design were taken from the geometry outputs of the program and combined with unit price values for installed members, similar to the process during the design study. This pricing and quantity information combined with the design details directly from the software formed the full precast preliminary design and material estimate.

3.5 Development of Design Considerations for Preliminary Design

The preliminary design process was limited to the design aspects most likely to change due to a change in superstructure type. The scope was limited to these areas to reduce time spent on preliminary design work that contributes little variation in cost across competing alternatives. Specifically, the cost of structural steel and steel reinforcing, the cost of structural concrete, the cost of the required deck by design, the cost of bearing assemblies, and the cost of the additional pier were considered. The preliminary design did not contain actual structural designs for any of the bridge substructure, the highway wearing surfaces or the roadway geometries. To aid the in the preliminary design, because design of the substructure had yet to be completed, certain generic values were calculated. The preliminary design did not heavily consider site geometry, limiting bridge input to a rectangular, no skew bridge, with no slope along the direction of the roadway. This was primarily to limit complexity when defining and evaluating many competing preliminary designs.

To develop a consistent factor that would at least partially describe the cost of having more, yet smaller members, the cost for bearings was included. These costs, sourced from the elastomeric bearings entry in a very similar bridge projects item list (NHDOT 2009), reflects the cost of a single bearing multiplied by two to reflect both ends of a span, and multiplied by the total number of girders in the entire bridge. It was included to weight the cost of having more or less total girders.

To show the cost of additional supports between abutments, an estimated generic pier cost was also established. This was developed by taking the minimum specified pier wall thickness from the *NH Bridge Manual*, and multiplying it by the estimated height of the new bridge and its expected width. Added to this cost was an estimated strip footing two feet thick, extending five feet beyond both faces of the pier wall. While a rough estimate, the conservative nature of this calculation for concrete volume accounts for some of the excavation and unexpected costs that cannot be considered during preliminary design. The volume from this approximate calculation was multiplied by the average bid price for footing concrete from the NH DOT 13933N contract quantities list, to create a per pier estimate to adjust the cost of two span bridges for fair comparison to single span bridges.

All other quantities used in preliminary design were estimated from typical values from the *AASHTO LRFD Bridge 2012 Specifications* or the *NH Bridge Manual*. For example, dead load estimates due to the wearing course or barriers were calculated according to the values found in *AASHTO LRFD Bridge Specifications 2012* Table 3.5.1-1, Figure 33 below, or from similar resources.

Table 3.5.1-1—Unit Weights

Material		Unit Weight (kcf)
Aluminum Alloys		0.175
Bituminous Wearing Surfaces		0.140
Cast Iron		0.450
Cinder Filling		0.060
Compacted Sand, Silt, or Clay		0.120
Concrete	Lightweight	0.110
	Sand-Lightweight	0.120
	Normal Weight with $f'_c \leq 5.0$ ksi	0.145
	Normal Weight with $5.0 < f'_c \leq 15.0$ ksi	$0.140 + 0.001 f'_c$
Loose Sand, Silt, or Gravel		0.100
Soft Clay		0.100
Rolled Gravel, Macadam, or Ballast		0.140
Steel		0.490
Stone Masonry		0.170
Wood	Hard	0.060
	Soft	0.050
Water	Fresh	0.0624
	Salt	0.0640
Item		Weight per Unit Length (klf)
Transit Rails, Ties, and Fastening per Track		0.200

Figure 33: Unit weights from AASHTO LRFD Bridge 2012 Specification

3.6 Development of the Construction Time and Labor Estimates for Preliminary Designs

Similar to the previous discussion of preliminary design of the superstructure, the development of initial construction time and labor estimates was limited to those design elements that introduce significant differences between designs. Therefore, the construction estimates primarily focused on erection of the superstructure members, the casting of the deck, and any associated activities that must occur in the field. The construction estimates therefore did not represent the total cost of the whole project or of any single phase. This is again to facilitate the comparison of separate beam designs without spending time on elements that offered little variation.

To estimate erection times, *RS-Means 2009 Heavy Construction* was used. This resource provided suggested sample crews based on girder sizes. Each crew then had a corresponding labor, equipment and crane cost, as well as a typical output per day. Estimating attempted to match the design beams sizes selected in preliminary design with as close of a beam as was contained within *RS-Means* to determine the necessary crew for members of that size. The production rate of the crew combined with the number of beams in a design was used to determine the time required to erect each type of superstructure, including the labor and crane time.

These crew values were then analyzed and summed for each design. Application of each design consisted of the shortest possible time that could be reasonably accomplished. This included engineering judgment as to the feasibility of simultaneous tasks. Finally, this construction time and crew requirement schedule was used to estimate the unique additional construction factor for a given design. The construction estimate is an estimation of the personnel days and crane time onsite, as well as the amount of time obstructing the roadway and disturbing the open ground, that are unique to a specific design. The construction labor and time estimates were meant to be used to aid in the selection of a final bridge design type.

3.7 Development of the Cost Estimates for the Preliminary Design Final Estimates

The cost estimates were prepared taking into account both the financial aspects and time required. Therefore, preliminary design consisted of a cost based on unit installed prices for its design quantities, a total machinery time requirement by class, and a total labor requirement by class.

Similar to the Design Considerations calculations, the cost estimates were calculated from total bid price unit costs. These costs represented the total of the contractors profit, labor expenses, machinery expenses, and material expenses per unit, and encompass what the State would pay in full to build the bridge. In the case of the precast elements for example, the unit used was a CY cost for structural grade concrete, and a weight dependent cost was used for the structural steel. Below, in Figure 34 is an excerpt from an average unit cost sheet published by the NHDOT.

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
534.3	WATER REPELLENT (SILANE/ SILOXANE)	GAL	2519.4	$76.56	A	17	$100.00	$55.00
			252	$82.30	C	3	$85.00	$80.00
			936	$85.00	E	2	$105.00	$65.00
538.2	BARRIER MEMBRANE, PEEL AN STICK - VERTICAL SURFACES (	SY	1252	$52.43	A	11	$63.00	$40.80
			208	$42.50	E	2	$60.00	$25.00
538.5	BARRIER MEMBRANE, HEAT WELDED (F)	SY	9238	$19.04	A	8	$25.00	$17.00
			470	$52.50	C	2	$55.00	$50.00
			144	$45.80	E	2	$46.60	$45.00
541.1	PVC WATERSTOPS, NH TYPE 1 (F)	LF	821	$5.03	A	3	$5.15	$5.00
541.2	PVC WATERSTOPS, NH TYPE 2 (F)	LF	608	$7.25	A	9	$15.00	$5.00
541.3	PVC WATERSTOPS, NH TYPE 3 (F)	LF	288	$7.24	A	5	$10.00	$5.00
541.4	PVC WATERSTOPS, NH TYPE 4 (F)	LF	290	$10.14	A	3	$10.30	$10.00
541.5	PVC WATERSTOPS, NH TYPE 5 (F)	LF	2060	$10.32	A	13	$15.00	$8.00
			88	$10.50	C	2	$11.00	$10.00
			624	$9.00	E	2	$10.00	$8.00
544.	REINFORCING STEEL (F)	LB	82443	$1.43	A	8	$2.50	$1.00
			876	$4.05	E	2	$4.10	$4.00
544.1	REINFORCING STEEL (ROADWAY)	LB	388650	$1.01	A	12	$2.00	$0.85
			1120	$2.73	B	2	$4.20	$1.25
544.2	REINFORCING STEEL, EPOXY COATED (F)	LB	524416	$1.48	A	7	$2.25	$1.20
			158285	$1.23	C	3	$1.35	$1.20
544.21	REINFORCING STEEL, EPOXY COATED,MECHANICAL CONNEC	LB	17232	$3.01	A	4	$8.00	$2.45
			526	$6.25	C	2	$8.00	$4.50
547.	SHEAR CONNECTORS (F)	EA	46880	$5.13	A	9	$6.00	$4.00
548.21	ELASTOMERIC BEARING ASSEMBLIES (F)	EA	84	$1,873.57	A	6	$2,750.00	$930.00
550.1	STRUCTURAL STEEL (F)	LB	1988414	$1.71	A	2	$1.75	$1.66
			1700	$9.65	C	2	$10.00	$9.30

Figure 34: A screenshot of the Current Average Unit Price sheet published by NH DOT. Taken fromhttp://www.nh.gov/dot/business/contractors.htm

The unit cost data on this project came from a single state contract, because it was for a very similar bridge less than 10 miles away (NHDOT, 2009.) The cost of each bridge type was calculated as the sum of all the included components: the cost of structural steel, the cost of structural concrete, the cost of the required deck concrete and reinforcing by design, the cost of bearing assemblies, and the cost of the additional pier as needed. Every type of girder was assumed to use the same type of bearing assembly for simplicity, so the bearing cost is simply a function of the number of girders.

The completed estimates contain for the preliminary precast and preliminary steel designs contain an estimated installed cost and an estimated required labor and machinery schedule.

3.8 Selection of a Final Design Type

The selection of the final design was the better alternative for meeting the following conditions:

- Cost, as expressed in the Estimate
- Labor and Machinery Time, also as expressed in the Estimate
- Aesthetics, expressed as the engineers judgment as to the aesthetic value of the structure
- General logic, expressed as engineering judgment of a design that seems reasonable to construct given factors such as shipment costs and availability.

3.9 Development of the Proposed Design

3.9.1 Loading for Prestressed Deck Bulb Tees

The loads that were applied to the superstructure of the selected bridge for the flexural design of the beams were determined in accordance with Section 3: Loads and Load Factors, of the *AASHTO LRFD Bridge Specification*. All of the load combinations outlined in Table 3.4.1-1 (Figure 8) were considered when choosing the governing moments and shears that the member was required to carry. Due to simple support conditions, various loads could be eliminated for the design of the superstructure. Because the bridge was designed to be free to translate longitudinally at the abutments, loads such as force effects due to uniform temperature, earthquake load, and horizontal wind pressure were omitted.

The loads used for the shear and moment calculations included DC (dead load of structural components and nonstructural attachments), DW (dead load of wearing surfaces and utilities), and LL (vehicular live load). A dynamic load allowance was applied to the vehicular live load in accordance with section 3.6.2 of the *AASHTO LRFD Bridge Specification*. Distribution factors were also applied to the vehicular live load in accordance with Table 4.6.2.2.2b-1 (Distribution of Live Loads for Moment in Interior Beams), Table 4.6.2.2.2d-1 (Distribution of Live Loads for Moment in Exterior Longitudinal Beams), Table 4.6.2.2.3a-1 (Distribution of Live Load for Shear in Interior Beams), and Table 4.6.2.2.3b-1 (Distribution of Live Load for Shear in Exterior Beams) of the *AASHTO LRFD Bridge Specification*.

The braking force was added as additional moment acting on the bridge superstructure as specified in section 3.6.4 of the *AASHTO LRFD Bridge Specification*. Because the braking force creates a horizontal shear on the superstructure it is not used in the vertical shear design of the beams and can therefore be omitted from this portion of the design evaluation.

3.9.2 Flexural Design for Prestressed Deck Bulb Tees

The design of the prestressed deck bulb tees was made much simpler by the use of the free software, *PG-Super*, developed by the coordinated efforts of the Washington and Texas Departments of Transportation. This program was used to provide the preliminary prestressed design. For final design, the software was also invaluable, as it confirmed the reasonable nature of the selected prestressed member and also provided an excellent starting point. Instead of following traditional design steps to establish preliminary member sizes, the program outputs provided the basis for design development. Important details such as concrete strength, strand location, and number of strands were all originally provided by the PG-Super analysis outputs. Other design input came from the Deck Bulb Tee standard section sheets published by the Washington Department of Transportation, one of which is shown in Figure 36 below.

The design development process was therefore the manual checking of the *PG-Super* results for adequacy. For simplicity of construction, and to insure that *AASHTO* equations for factors such as Kg adequately described the proposal bridge, all girders were designed to be identical. Therefore, the design calculations were only tabulated for the girder with the overall highest loading, no matter the design state, because all other sections would therefore be slightly over designed.

To design the most highly loaded girder, much of the calculations followed a publically available design example published by the Prestressed Concrete Institute (PCI). The particular design example that was of most help was the design of prestressed concrete box beam. This design example was useful because of the lack of Deck Bulb Tee examples, and the proliferation of composite design for other prestressed beam types. The example presents the design of bridge using abutting AASHTO box beams and does not feature a composite topping or any other additional structural deck. A drawing of AASHTO Box Beams is provided below in Figure 35.

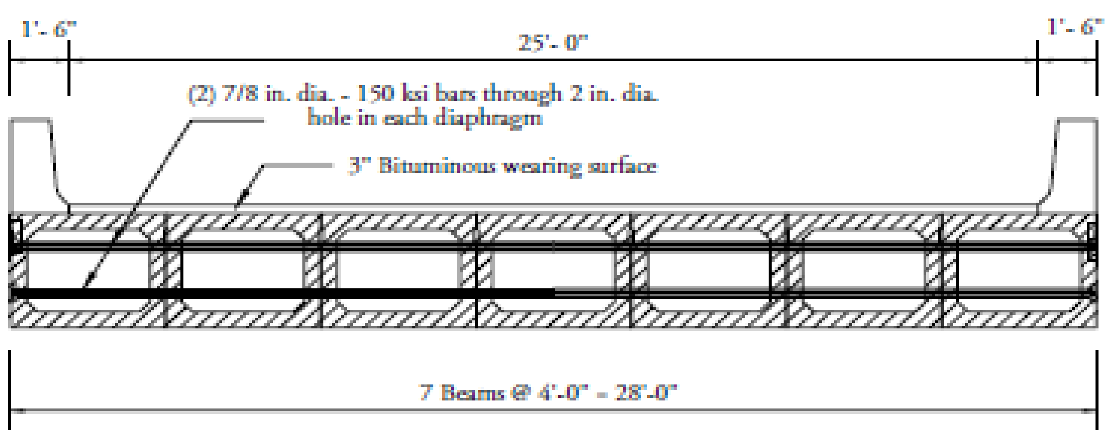


Figure 35: Excerpt from PCI Bridge Design Manual Example 9.2, showing cross section for a bridge constructed from prestressed box beams.

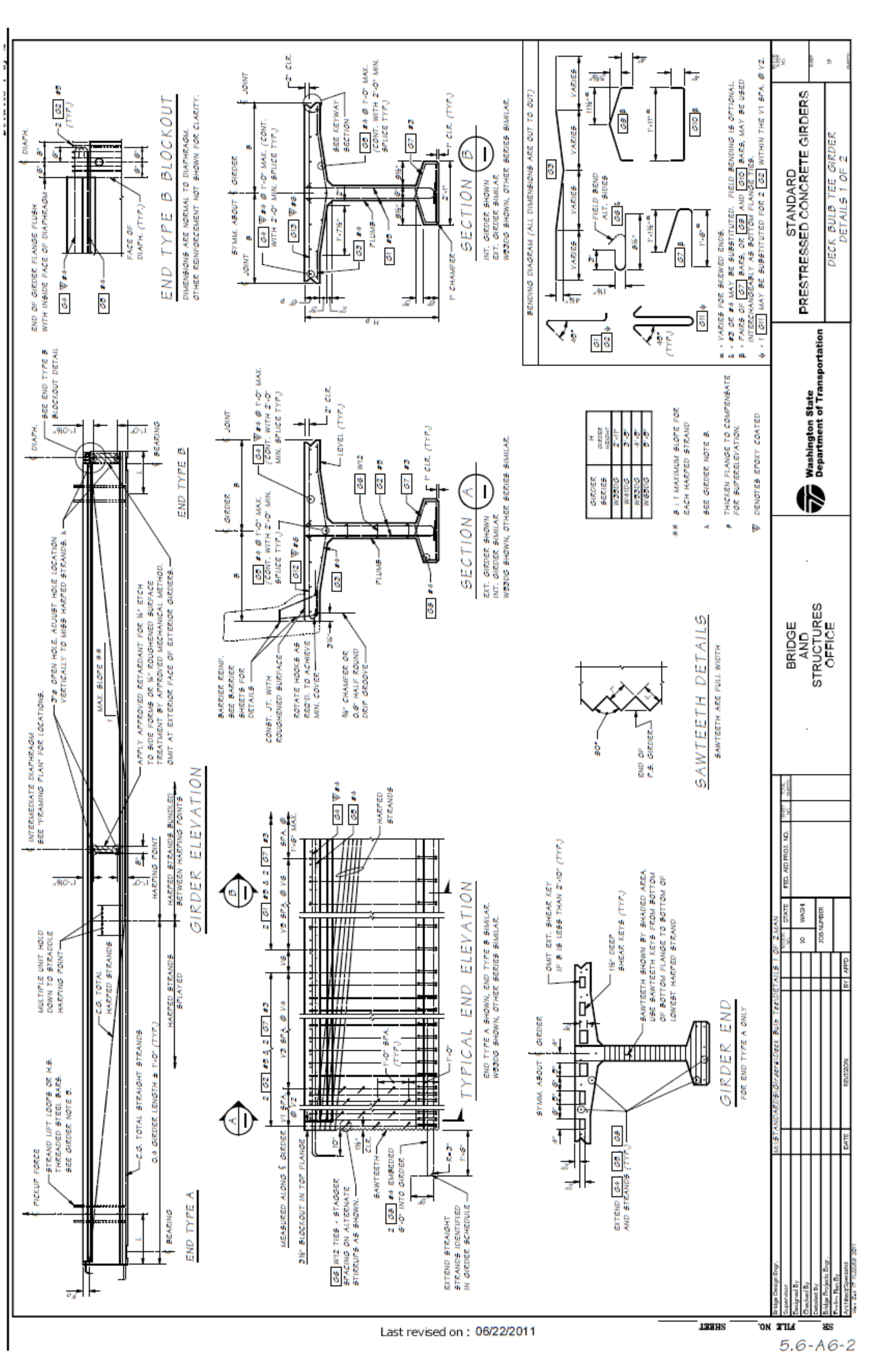


Figure 36: Part of the Washington Department of Transportation Standard Deck Bulb Tee Drawings

The design example follows well established LRFD prestressed design steps. It designs for Strength I, which was also the governing load case in the project beam design. The project flexural calculations followed the same general format. First, key section properties such as the depth of the compression block and the depth of the center of gravity of the prestressing steel were calculated. From these the final moment capacity of the beam according to assumed prestressing forces was calculated. Once the design had progressed to this point, the properties of the beam were used to calculate the actual prestressing loss values. The losses were calculated for both service and transfer conditions.

The stresses in the concrete were then checked according to *AASHTO* limit states for compression and tension. This involved six calculations, one for both tension and compression in the transfer load case, the service case without live loading, and the final service case with live loading. The girder properties were adjusted as necessary according to design calculations, such as compression stresses at transfer necessitating a higher f'ci, and therefore a higher f'c. Similar force calculations for the ends of the beams at transfer also served to check the adequacy of the harping design provided by *PG-Super*. Once the design concrete stresses were safely under the cracking limits in all cases, the girder was reevaluated for capacity, and would have been adjusted further if ΦMr had been lower than the Strength I Mu, which as mentioned before, governed flexural beam design for the project bridge.

3.9.3 Vertical Shear Design for Prestressed Deck Bulb Tees

The final designs of the prestressed concrete girders were completed in accordance with section 5.3 of the *PCI Design Handbook*.

The ultimate shears to be designed for were derived from the governing *AASHTO* limit state. The girder was designed for shear at 10% increments of the beam its length under Strength I loading conditions.

The shear capacity of the concrete with prestressing was taken as the lesser of the flexural shear strength (V_{ci}) and the web shear strength (V_{cw}). The *PCI Design Handbook* provides the following equations:

$\mathbf{\Phi V_{ci}=0.6*\sqrt{(f'c)}*b_w*d_p+V_d+(V_i*M_{cre})/M_{max}}$ (*PCI Design Handbook* Eq. 5-21)

and

$\mathbf{\Phi V_{cw}=(3.5*\sqrt{(f'c)}+0.3f_{pc})*b_w*d_p+V_p}$ (*PCI Design Handbook* Eq. 5-23)

In equation 5-21, M_{cre} was determined using the following equation:

$\mathbf{M_{cre}=(I/y)*(6*\lambda*\sqrt{(f'c)}+f_{pe}-f_d)}$ (*PCI Design Handbook* Eq. 5-22)

The lesser of the two shear values calculated from *PCI Design Handbook* equations 5-21 and 5-23 was designated as ΦV_c in the nominal beam shear capacity equation:

$\mathbf{\Phi V_n=\Phi V_c+\Phi V_s}$

The excess shear that was not covered by the concrete and prestressing strands was required to be carried by mild shear reinforcement. The shear reinforcement was designed in accordance with section 5.3.4 of the *PCI Design Handbook*. There were two limiting equations for the minimum area of shear reinforcement presented in the *PCI Design Handbook*. These equations are shown below:

$$(50*b_w*s)/f_y \le A_{v,min}=(0.75*\sqrt{f'c}*b_w*s)/f_y \qquad (\textit{PCI Design Handbook}\text{ Eq. 5-24})$$

Trial stirrup areas were used in equation 5-24 of the *PCI Design Handbook* to determine two possible values for maximum stirrup spacing. The maximum spacing value was determined by taking the minimum spacing found in equation 5-24 of the *PCI Design Handbook*, 0.75 times the height of the cross section, or 24”.

The shear resistance of the stirrups was calculated using the equation below:

$$\Phi V_s=A_v*f_y*(d/s) \qquad (\textit{ACI})$$

The final step was to ensure that the nominal shear capacity of the beam was girder to withstand the ultimate shear at every tenth span location along the beam.

3.9.4 Pier Design

The pier was designed as an intermediate support in accordance with *AASHTO LRFD Specifications*. The design process for the pier was done parallel to the design process illustrated by the *FHWA* design example (FHWA). The pier cap was designed as a flexural member, the bents were designed as slender columns, and the foundations were designed as spread footings. It is important to note that the foundations were only designed taking bearing capacity and settlement into account. The reinforcement and eccentricities caused by the connections in the footing were not evaluated.

The loads on the pier were determined using the method specified by *AASHTO*. The critical moments, shearing forces and reactions were determined from a structural analysis that considered three *AASHTO* load combinations deemed to govern in the pier design (FDOT)

- $Strength\ I = 1.25 \cdot DC + 1.50 \cdot DW + 1.75 \cdot LL + 1.75 \cdot BR + 0.50 \cdot (TU + CR + SH)$
- $Strength\ V = 1.25 \cdot DC + 1.50 \cdot DW + 1.35 \cdot LL + 1.35 \cdot BR + 1.3 \cdot WS + 1.0 \cdot WL + 0.5(TU + CR + SH)$
- $Service\ I = 1.0 \cdot DC + 1.0 \cdot DW + 1.0 \cdot LL + 1.0 \cdot BR + 1.0 \cdot WS + 1.0 \cdot WL + 1.0(TU + CR + SH)$

AASHTO requires the live load (LL) to be placed along the superstructure transversely so the worst possible loading scenario can occur. The live load contains multiple design trucks and lane loads spaced two feet from the edge of the bridge and each other. A trial-and-error approach was used to determine where the live loads should be placed using two different *RISA 2-D* models. The maximum axial load was determined by loading as many lanes as possible, and the minimum axial load was determined by using the least amount of live load possible as specified by AASHTO. The minimum axial load became relevant when designing the columns as seen in section 5.2.3. The maximum moment in the pier cap was obtained by creating the highest reactions in the girders located between columns. These portions of the pier cap were then only loaded on every other span in order to produce maximum positive and negative moment.   One of the models was used to determine the reactions in the girders with the corresponding live load placements. These reactions were then superimposed on the model containing the pier cap and columns in order to determine the effects the live load placement had on the substructure. Other loads calculated include the dead load of the utilities and future wearing surface (DW), dead load of the structural members (DC), braking force caused by the vehicles (BR), wind acting on the structure (WS), and wind acting on the live load (WL). All calculations regarding wind were taken as acting at angles 0, 15, 30, 45, and 60 degrees in both the transverse and longitudinal axes as specified by *AASHTO* and the most conservative loading was taken. Temperature and shrinkage forces were not developed at the intermediate pier due to the symmetry of the structure.

In lieu of using three-dimensional analysis software, *RISA* models were made for both the transverse and longitudinal direction. This was important for the design of the pier columns in order to take the biaxial bending into consideration. The moments were magnified in accordance with Section 4.5.3.2 of *AASHTO* to determine the ultimate moment. The maximum and minimum axial load on the column was then coupled with the maximum moment to determine capacity of the structure.

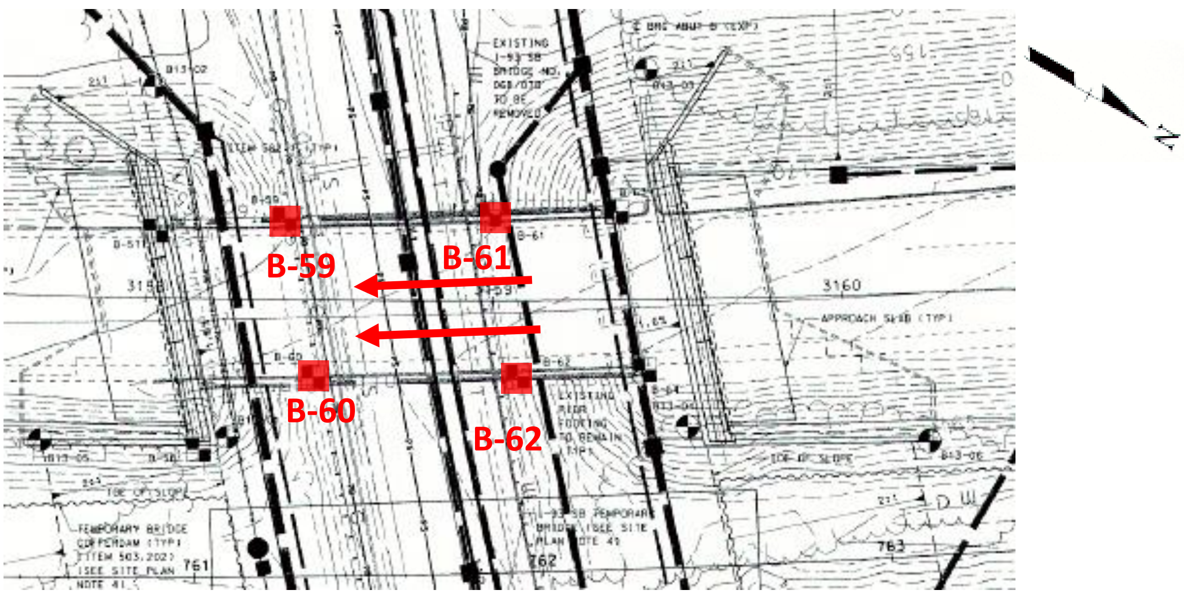


Figure 37: Site and boring log layout of bridge, image taken from NHDOT state plans (NHDOT, 2012)

The foundations were designed to meet bearing capacity and settlement limits as specified in Coduto's text (Coduto). Highway bridges were determined to have a total allowable settlement of 2 inches and a factor of safety of 3.5 (Coduto). A site investigation was completed using the information provided by the State. Figure 37 above is an image taken from the State plans displaying the layout and locations of the boring tests relative to the current bridge; the red arrows denote the direction of traffic. The four relevant boring logs from SPT tests are displayed below. They were analyzed and averaged in order to develop a representative soil profile used for the design of the foundation for the intermediate pier. Appropriate adjustment factors were used to correct the values found in the boring logs. These adjusted $(N_1)_{60}$ values were then used to obtain properties of the soil layer such as unit weight and other properties displayed in Figure 38 below. Reasonable unit weights were assumed at first in order to determine the vertical effective stress at any given point which was necessary in order to correct the data obtained from the SPT tests. This data was then refined using a guess-and-check method in order to reasonably fit the values within the ranges specified in Figure 39.

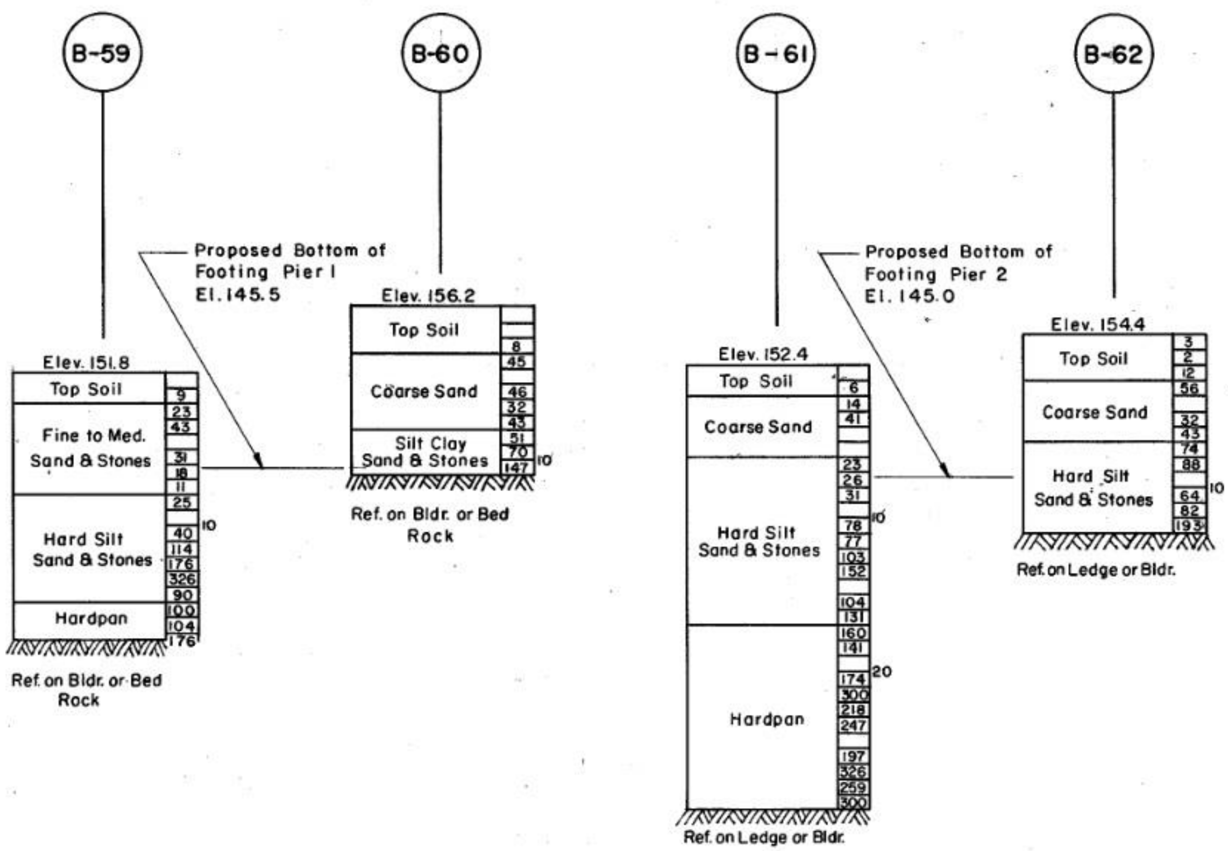


Figure 38: Boring logs used for foundation design of intermediate pier, taken from state plans (NHDOT, 1959).

TABLE B5.2 Correlation of N, γ, D_r, and ϕ' for Coarse-Grained Soils

N	Description	γ (kN/m^3)	D_r (%)	ϕ' (degrees)
0 – 4	Very loose	11 – 13	0 – 15	26 – 28
4 – 10	Loose	14 – 16	16 – 35	29 – 34
10 – 30	Medium	17 – 19	36 – 65	35 – 40*
30 – 50	Dense	20 – 21	66 – 85	38 – 45*
> 50	Very dense	> 21	> 86	> 45*

*These values correspond to ϕ'_p.

Figure 39: Soil properties in respect to data from SPT test, taken from (Tao, CE 3044 Lecture Material)

When calculating for settlement, certain consolidation and compressibility values are necessary. Since no samples were taken and no other data on the site was available, the values were interpreted from Figures 40 and 41 shown below.

TABLE 3.6 TYPICAL RANGES OF OVERCONSOLIDATION MARGINS

Overconsolidation Margin, σ_m'		
(kPa)	(lb/ft^2)	Classification
0	0	Normally consolidated
0–100	0–2000	Slightly overconsolidated
100–400	2000–8000	Moderately overconsolidated
> 400	> 8000	Heavily overconsolidated

Figure 40: Typical ranges of overconsolidation, image taken from (Coduto, 2001)

TABLE 3.5 CLASSIFICATION OF SOIL COMPRESSIBILITY

$\frac{C_c}{1+e_0}$ or $\frac{C_r}{1+e_0}$	Classification
0–0.05	Very slightly compressible
0.05–0.10	Slightly compressible
0.10–0.20	Moderately compressible
0.20–0.35	Highly compressible
> 0.35	Very highly compressible

Figure 41: Classification of soil compressibility, image taken from (Coduto, 2001)

3.9.5 Abutment and Wingwall Design

The final abutment and wingwall designs were completed in accordance with section 11.6 of the *AASHTO LRFD Bridge Specification*. In order to design the abutments and wingwalls an investigation of available site data was completed. Figure 42 below displays the site layout with the locations of the boring logs taken by State Engineers. These engineers also created a soil profile for each boring log. These profiles were combined to create a representative soil profile for the North and South abutments and wingwalls.

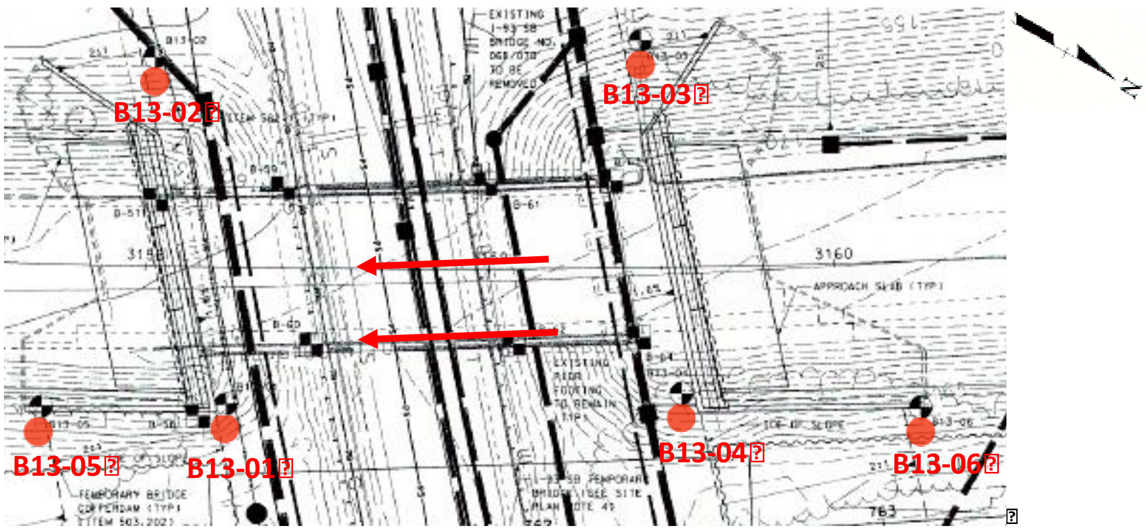


Figure 42: Location of test borings, and current direction of travel on I-93 SB.

The cantilever abutment and wingwall analysis worksheets provided by the New York State Department of Transportation were used in the final design of the abutments and wingwalls. Information about the conditions surrounding the abutment and wingwall was entered into the worksheets. Material properties were selected in order to best suit the project and geometric properties of the abutments and wingwalls were influenced by clearance requirements and roadway restraints. Necessary input categories for the abutment and wingwall designs are shown in Tables 4 and 5 respectively.

Table 4: Mathcad Inputs for Abutment Design

Inputs for Abutment Analysis Mathcad Worksheet	
Yield strength of reinforcing bars	60 ksi
Compressive strength of concrete	4 ksi
Clear cover of reinforcement in backwall and stem	2.5”
Clear cover of reinforcement in footing	3”
Exposure factor	1
Service bearing resistance of soil	6 ksf
Coefficient of sliding friction	0.8
Resistance factor for sliding	0.8
Internal angel of friction for backfill	35°
Unit weight of backfill	125 pcf
Internal angle of friction for soil under the footing	30°
Unit weight of soil under the footing	125 pcf
Resistance factor for bearing	0.45
Berm width	0’
Berm slope	0°
Elevation of water table	147.9’
Spacing of girder	5.75’
Beam skew	0°
Unfactored girder reaction due to DL and SDL	47.8 kips
Unfactored girder reaction due to DW	7.8 kips
Maximum unfactored girder reaction due to LL	120 kips
Distance from center line of bearings to front face of abutment stem	9”
Axial girder load due to temperature fall	0 kips
Number of design lanes	6
Extra dead load 1	6.4 k/ft
Distance of extra load 1 from the front face of abutment stem	-35”
Extra dead load 2	0 k/ft
Distance of extra load 2 from the front face of abutment stem	0”
Extra live load	2.69 k/ft
Distance of extra live load from the front face of abutment stem	-35”
Percentage of live load surcharge	50%
Elevation of top of backwall	174.55’
Elevation of bridge seat	171.63’
Elevation at bottom of footing	147.63’
Fill height over toe	1’
Thickness of footing	36”
Heel width	10’
Toe width	4’
Thickness of backwall	18”
Thickness of stem	62”
Length of abutment	75’

Table 5: Mathcad Inputs for Wingwall Design

Inputs for Wingwall Analysis Mathcad Worksheet	
Yield strength of reinforcing bars	60 ksi
Compressive strength of concrete	3 ksi
Clear cover of reinforcement in wall	2.5”
Clear cover of reinforcement in footing	3”
Exposure factor	1
Service bearing resistance of soil	6 ksf
Coefficient of sliding friction	0.8
Resistance factor for sliding	0.8
Internal angel of friction for backfill	35°
Unit weight of backfill	125 pcf
Internal angle of friction for soil under the footing	30°
Unit weight of soil under the footing	125 pcf
Resistance factor for bearing	0.45
Berm width	0’
Berm slope	0°
Elevation of water table	147.9’
Elevation at bottom of footing	147.63'
Distance of traffic edge	4'
Extra dead load 1	0 k/ft
Distance of extra load 1 from the front face of abutment stem	0”
Extra dead load 2	0 k/ft
Distance of extra load 2 from the front face of abutment stem	0”
Thickness of footing	24"
Heel width	8'
Toe width	3'
Fill height over heel	24'
Height of reveal	0"
Slope of backfill over heel	0°
Distance of slope of backfill over heel	0'
Thickness of wall at top	18"
Thickness of step	24"
Height of step	16'
Fill height over toe	1'
Length of wall	58'

3.10 Development of the Traffic Plan

Traffic planning was a key component of the bridge project. Often bridge replacement projects feature lengthy detours, lane closures or drastically reduced travel speeds. The methodology used to create the project traffic plan was simple: to avoid these common pitfalls as much as possible. However, the project traffic plan was more the methodology of the traffic diversion than a full dimensioned set of plans to commence construction tomorrow.

The main weapon during this project in terms of traffic diversion was time. This project was setup to construct the bridge as fast as possible and to avoid delays to normal traffic. The accelerated construction allowed the traffic plan to succeed within very strict parameters. The first was no daytime closures of any lanes on Interstate-93. There were to be no daytime closures because the I-93 corridor is one of the most congested roadways in the U.S. during prime hours, and therefore delays and road user cost penalties can mount extremely quickly. NH 97 is also a heavy commuter route and currently backups severely onto surrounding surface streets during peak hours. Daytime closures also serve to confuse motorists with new traffic plans, merges, and in general are not positive public relations for a project.

The second tenet of the traffic plan methodology was similar to the first, and it achieved design speed limits at all times on Interstate 93. All construction would be completed offline if possible to avoid having workers/temporary barriers near the current interstate mainline. From experience, roadway users often ignore reduced speed signs, so the safest possible option was to avoid work on or directly abutting the active mainline barrel. The primary exception to this was during final phase of segmental construction, as traffic would necessarily be quite close and on the same plane to the workers completing the second portion of a bridge.

The third tenet of the traffic plan was to avoid detours whenever possible. Detours were undesirable for many reasons. Detours often place high volume traffic on secondary streets that are not structurally equivalent to mainline highways or major arterial streets, with resulting pavement damage and lifespan reductions. Detour streets could also suffer from a variety of geometry, design speed, and light/traffic control issues, as few were designed to suddenly carry much greater traffic volume than would be usually expected. Finally, detours could also put both drivers and pedestrians at risk by forcing drivers in large volumes onto streets they may be unfamiliar with. Detours could also be undesirable economically if they require extensive signage or law enforcement manpower to function effectively.

The fourth and final tenet of the traffic plan was to avoid the creation of temporary roadways or bridges. This was an extremely important focus for the plan. In the actual contractor bids for the replacement of the project bridge, all three contractors carried line items of more than half a million dollars for a temporary bridge, and some carried hundreds of thousands of dollars' worth of temporary roadway paving (NHDOT). This represented a place where creative traffic diversion could have extremely desirable economic effects.

The components of the traffic plan are not unique individually. For example, three exits north on the same highway, on a very similar bridge project, the contractor opted to use segmental construction to avoid creation of a temporary bridge structure. One exit north, the State chose to use offline construction to drastically reduce traffic control costs and improve site safety. In the northern section of the State, in Laconia, a total precast bridge was constructed of the same total span length as the project bridge, in only four weeks, avoiding months of traffic diversion, slowing and road user cost. The traffic plan methodology is not to develop radical new techniques, but to use these common methods to the highest advantage. Accelerated construction, with precast components allows a great flexibility to create economical and delay free traffic plans

3.11 Development of the Cost and Time Estimates for the Proposed Solution

For the proposed solution, pricing was based primarily on takeoffs from design. Estimates of units, such as footing concrete, or area of grubbing required were calculated from site conditions and the final design geometry. Footings were estimated by CY as was excavation and fill, while other quantities such as protective steel sheet piles, and bridge membrane were estimated in square footages.

Once quantities were determined for every construction task, the tasks were priced using *RS MEANS Heavy Construction 2011. RS MEANS* lists both square foot costs and assembly cost data for various heavy construction activities. The proposal tasks were matched to the best matches in RS means and priced according to the price given. The prices given are a national average for material, labor and subcontractor profit. Each of these unit prices were multiplied by a correction factor for two years of inflation, and a factor that represented the cost of construction in Manchester NH versus the national average prices given in charts. The adjusted numbers were then multiplied by the task quantities calculated earlier to obtain the task total cost. All of the task costs were summed to estimate the total construction cost of the proposed bridge.

Cost estimation for the State design was tabulated in the same fashion. First, task quantities were calculated from the construction plans. These tasks were then priced by the unit prices given in the actual bid information for the project. The source of this unit price information is a text document that is publically released for every NH project that goes out to bid. The unit prices were not adjusted for the state design, because they were already provided in the present and were presented by companies bidding work in NH. The unit prices were again multiplied by task quantities, and then the totals of all tasks were summed to create the total estimated cost of just the bridge portion of the state design. The bid prices could not be used as is because the bid that included the actual bridge project included many other tasks.

Time estimation for the project was combined from several sources. Each task as shown in Appendix 4.2, was timed using one of three ways. First, a crew was selected from *RS MEANS* and the given output for that crew (or several crews of that size) in addition to other factors such as curing time for CIP was calculated. The longest task in each step was therefore the governing value for the length of each step. The steps were then scheduled in precedence order in a CPM program to determine the minimum real world project duration assuming regular workdays, without work on holidays, weekends or second/third shift.

The time estimating information for the state design is provided by the Actual project CPM schedule. The total schedule contains a section for just the southbound bridge, which provides step durations in addition to float and start and finish information. The start and finish information was used to estimated total construction time.

3.12 Determining the Work Zone Road User Cost

Additional costs borne by the motorists can count as incentives for the State to use Accelerated Bridge Construction methods. These costs were calculated by determining the Work Zone Road User Costs for the bridge designed for the project and the bridge designed for the State. The first step in this process was to determine which tasks would restrict traffic flow and their durations. These tasks and their durations for the State plans were determined by obtaining the actual Critical Path Method Schedule from a source at the NHDOT (NHDOT). The tasks and durations for the project were taken from the traffic plan which was completed earlier in the project. Once the durations for all the tasks which would have an impact to the traffic flow were determined, the unit costs per hour for delays of both cars and trucks were determined from the AASHTO publication *A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements* (AASHTO) which is also known as the *AASHTO Redbook*. Since the values found in this publication were from 1977, the unit cost of the cars was updated to the values assumed for 2012 by using the Consumer Price Index (CPI) and the Producer Price Index (PPI) for trucks. The relevant additional costs for the project were the cost of rerouting the traffic and reducing the speed limit. The cost for rerouting the traffic was determined by calculating the change in distance of the detour from the original route and using relevant traffic data, the unit cost of car and truck delays, and speed limits to calculate the total Work Zone Road User Cost for using the detour. The formula for determining the Work Zone Road User Cost for speed limit changes is shown in the formula *below, represented as* C_{zi} (Chen, Jiang, Li).

$$C_{zi} = L\left(\frac{1}{v_z} - \frac{1}{v_f}\right) \cdot F_{ai} \cdot (P_c \cdot U_c + P_t \cdot U_t)$$

L = Work Zone Distance
v_z = Work Zone Speed Limit
v_f = Normal Speed Limit
F_{ai} = Hourly Traffic Flow
P_c = Percentage of Cars
P_t = Percentage of Trucks
U_c = Unit Cost of Time for Cars
U_t = Unit Cost of Time for Trucks

All relevant traffic data was obtained from the NHDOT website under the Traffic Volumes section (NHDOT). This information provided all the data needed to determine the traffic flow for given locations affected by the bridge during different hours of the week. These documents taken from the NHDOT website can be seen in Appendix 4.3. The percentage of trucks was determined from the data taken from *Computation of User Costs at Freeway Work Zones Using Weigh-in-Motion Traffic Data* (Chen, Jiang, Li). Figure 43 shows the data as it is seen in the text. Percent ADT refers to the percent of average daily traffic that passes within the specified hour.

Time	State roads		US roads		Interstate	
	% ADT	% Trucks	% ADT	% Trucks	% ADT	% Trucks
0:00–1:00	1.08	11.12	1.10	34.83	1.53	35.01
1:00–2:00	0.62	15.14	0.83	41.41	1.19	39.95
2:00–3:00	0.46	19.17	0.75	45.01	1.06	43.48
3:00–4:00	0.54	20.42	0.82	45.83	1.12	43.26
4:00–5:00	0.97	17.66	1.31	37.96	1.46	38.18
5:00–6:00	2.21	13.42	2.62	28.25	2.42	29.61
6:00–7:00	4.46	11.31	4.12	24.60	3.89	23.50
7:00–8:00	5.92	10.70	5.07	24.22	4.88	21.45
8:00–9:00	5.21	13.26	5.14	26.68	4.93	22.61
9:00–10:00	4.84	14.52	5.39	26.72	5.07	23.30
10:00–11:00	5.12	14.20	5.74	26.59	5.40	23.21
11:00–12:00	5.50	13.35	6.12	25.92	5.67	22.90
12:00–13:00	5.81	12.61	6.25	25.70	5.85	22.64
13:00–14:00	6.00	12.57	6.29	25.48	6.06	22.17
14:00–15:00	6.54	12.00	6.54	24.19	6.40	21.30
15:00–16:00	7.62	10.40	7.17	21.75	6.89	19.96
16:00–17:00	8.09	8.96	7.39	20.00	7.15	18.83
17:00–18:00	7.85	7.76	7.14	18.29	6.78	18.51
18:00–19:00	6.15	7.61	5.70	18.95	5.63	19.88
19:00–20:00	4.53	7.81	4.25	21.22	4.59	21.76
20:00–21:00	3.58	7.97	3.53	21.81	3.90	23.38
21:00–22:00	2.94	7.96	2.96	22.71	3.36	24.84
22:00–23:00	2.27	8.41	2.23	24.86	2.73	27.27
23:00–0:00	1.70	9.19	1.52	27.88	2.05	30.31

Figure 43: Study representing the percentage of trucks on the road in an hourly breakdown along with the percentage of the Average Daily Traffic hourly breakdown for different types of roads in the United States. Image taken from http://www.tandfonline.com/doi/pdf/10.1080/15578770903152823.

3.13 Comparison between Project Design and State Plans

The projects were compared based on a grading system which rated each design on a scale from 1-10 in the categories of cost, constructability, aesthetics, maintenance and incentives. Each category was weighted with respect to its importance in the project, and the grades were summed together to determine the final grade for each project.

4. Preliminary Results and Evaluation

The preliminary investigation produced both steel and prestressed design studies. These served to determine the member type and framing arrangement selected for preliminary design. A load study was conducted for *AASHTO LRFD* lane, HL-93 truck and tandem loads. The results from this load study were used to create a preliminary steel bridge design. At the same time a preliminary prestressed design was developed using *PG-Super.* Cost estimates were created for both designs were created from preliminary design quantitates. The design girder sizes were also used to estimate girder erection times

The cost estimates were combined with the girder erection times and aesthetic values assigned to the bridge types to score the preliminary designs. Each factor was given a weight, and a zero to ten score was given to each design in each category. The selected design had the highest aggregate score, as represented by the sum of the category scores times their weights.

4.1 Load Results

The results from the LRFD load study are shown in Table 6. The vehicle loads and load factors and combinations used for the study are defined in Section 3.4 of the *AASHTO LRFD Bridge Design Specification*. AASHTO provides various strength limit states to be considered when designing a bridge, but for simplicity of the preliminary design, standard LRFD load factors were used (1.2DL +1.6LL). The shear and moment values obtained from the studies were maximized by placing the axel loads at various locations within the span, according to points of theoretical maximum moment. The vehicle loads were placed as close to these critical moment locations as allowed under the *AASHTO LRFD Bridge Design Specification* using a trial-and-error approach until the maximum moments were observed. Figures 44 and 45 show how the load placement maximizes the moment on the beams for the two span, continuous and one span designs. The shapes of the moment diagrams were the same for both loading within a framing style, meaning both the truck and tandem moment diagrams had similar shapes.

Table 6: Results from LRFD load study

HS20-44 (Service)			
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4201.3	
Continuous	74.1	893.4	-1248.2
Tandem (Service)			
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.2	-1040.2

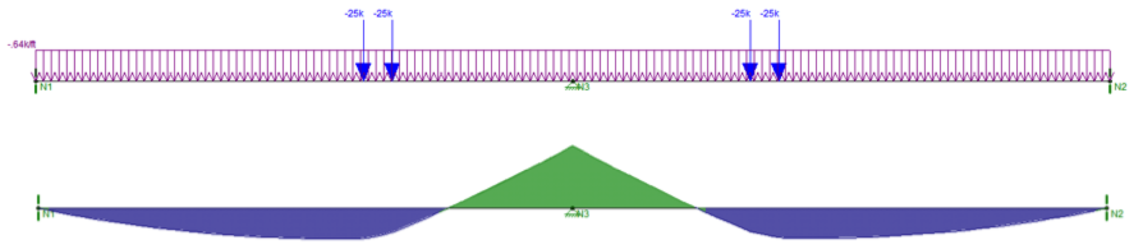


Figure 44: Tandem load placement and moment diagram for 2-span continuous design

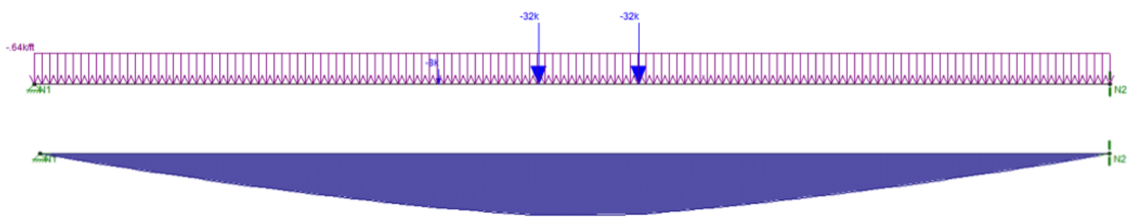


Figure 45: Truck load placement and moment diagram for 1-span design

4.2 Design Studies

4.2.1 Steel Design Study

The initial steel design study was completed using the computer-based design aid, *eSpan 140*. Based on the geometric and travel usage inputs into the program, *eSpan 140* provided an estimate of the size of the members, beam spacing, and number of beams needed to support the superstructure for a 75 and 140-foot simple span bridge. The information gathered from *eSpan 140* was useful in providing a starting point for design development. The full *eSpan 140* results for both the 75 and 140-foot simple spans can be referenced in Appendix 2.2. Emphasis can be placed on the beam spacing values obtained from the *eSpan 140* report as they were the most important results obtained for the preliminary design. The spacing was used as a starting point for the design of the steel superstructure.

4.2.2 Precast Design Study

The precast design study shows that of the three considered shapes of beam (Deck Bulb Tees, AASHTO I Beams, and AASHTO Box beams); there were 12 total possible standard beams that could be sufficient. Of these, 10 beams were for a 75 foot simple span and two for a 150 foot simple span. The two beams that were shown to have the ability to span 150 feet were a 65” Deck Bulb Tee and a Type

VII AASHTO Beam. The beams that could reasonably span the 75 foot span in the two-span bridge included four sizes of 48-inch wide AASHTO Box, four sizes of 36 inch wide AASHTO Box Beam, a 35” Deck Bulb Tee and a Type III AASHTO I BEAM.

The total cost estimates for each of these beam options all fell in the range of $513,000 to $395,000. The single span, Type VII AASHTO Beam, came in as the highest value by nearly $50,000 and was eliminated, because it would also have high shipping and erection costs, and the added difficulty of quickly placing a CIP deck. The second highest was the two-span Type III AASHTO Beam, and again it was eliminated due to the lack of cost savings to make up for the inconvenience of placing an accelerated CIP deck.

Of the remaining designs, the cost range was only $395,000 to $446,000. The least expensive design was the two span 35” Deck Bulb Tee. The Deck Bulb Tee design had the advantage of not requiring any additional deck panels, unlike the other two beam classes. All the sizes and depths of AASHTO Box Beams required the use of additional decking panels between beams, which, while accounted for in cost, would add hidden penalties to erection, design and fabrication times. Therefore those designs could not be chosen over the 35” without significant financial gain, which did not exist. Under the same line of reasoning, not only did the 35” Deck Bulb have a lower estimated cost, it also fell within normal shipping and handling constraints (McCormac), whereas the large 65” Deck Bulb Tee, with its 150 foot span certainly did not. Table 7 displays the structural elements considered and their estimated cost.

Table 7: Design study results precast concrete shapes.

Case	Structural Element	Spans	Cross Sectional Area (In^2)	Estimated Strands Required	Total Bridge Structure Estimated Cost (Stringers, Deck + Pier)
1	AASHTO BOX BEAM 48" X 27"	2	692.5	40	$ 440,118.50
2	AASHTO BOX BEAM 48" X 33"	2	752.5	34	$ 422,914.27
3	AASHTO BOX BEAM 48" X 39"	2	812.5	38	$ 412,896.41
4	AASHTO BOX BEAM 48" X 42"	2	842.5	42	$ 406,030.58
5	AASHTO BOX BEAM 36" X 27"	2	560.5	41	$ 446,424.16
6	AASHTO BOX BEAM 36" X 33"	2	620.5	36	$ 431,498.76
7	AASHTO BOX BEAM 36" X 39"	2	680.5	38	$ 422,915.87
8	AASHTO BOX BEAM 36" X 42"	2	710.5	50	$ 405,671.17
9	DECK BULB TEE 65"	1	1003	57	$ 400,093.01
10	DECK BULB TEE 35"	2	850	35	$ 394,594.18
11	TYPE IV AASHTO I BEAM	1	1085	47	$ 513,042.89
12	TYPE III AASHTO I BEAM	2	560	28	$ 452,761.70

Therefore, by process of elimination, weighted by cost and the factors mentioned above, a two-span alignment using 35” Deck Bulb Tee structural members was selected as the most promising option to pursue for preliminary design. This design accounts for a beam spacing of roughly six feet, standard concrete and steel strengths, and standard relative humidity during bridge service conditions.

4.3 Preliminary Designs

4.3.1 Steel Preliminary Designs

The steel preliminary design results evaluated the designs of a continuous span and a single span using both composite and non-composite beam and slab systems. Each design considered the use of standard sections and plate girders. The designs were evaluated for clearance over Pelham Road and all the beams designed allowed for more than the required clearance of 16'-6" by the *AASHTO LRFD Bridge Design Specification*.

4.3.1.2 Composite Beam-and-Slab System

Continuous Span

Table 8: Capacities and governing loads

End Beams-W24X76				Center Beams-Girder			
ϕM_n	1624.159 k-ft	M_u	1615.688 k-ft	ϕM_n	2949.45 k-ft	M_u	2905.01 k-ft
ϕV_n	267.7752 k	V_u	160.7558 k	ϕV_n	152.01 k	V_u	149.16 k
$\sum Q_n$	2252.3753 k	Q_u	2240 k				

The continuous span using composite beams was designed using three beam segments along the length of the bridge, and the beams were spliced at two connections. A total of eight beams were used across the cross-section at a spacing of 9'-8.571", with a three-foot distance from inter-beam to the edge of the bridge. The transverse cross-section of the end and center beams can be seen in Figures 46 and 47 respectively. The cross-sections of both beams are shown in Figures 48 and 49, the longitudinal view of the continuous steel design can be seen in Figure 50.

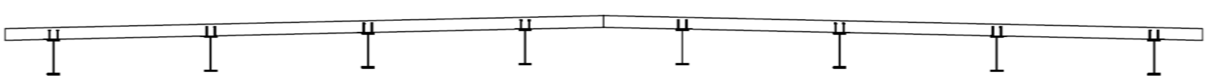


Figure 46: A cross-section view of the end beams for continuous composite design.

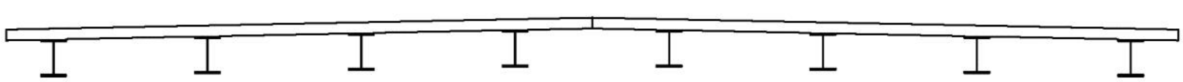


Figure 47: A cross-section view of the center beams for continuous composite design

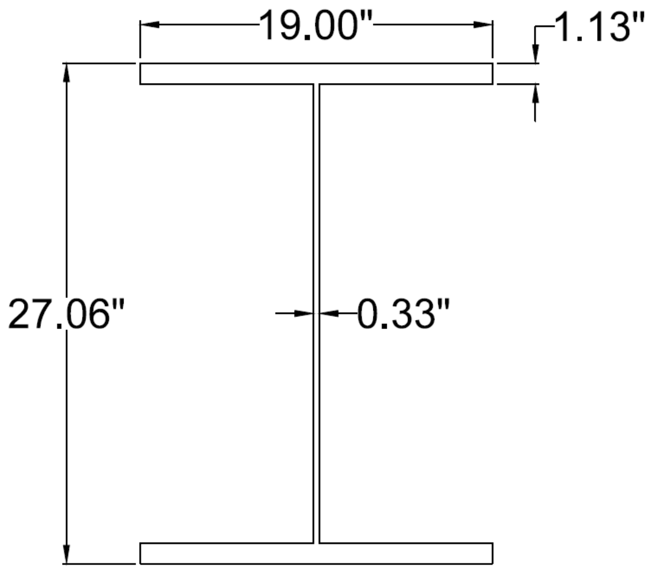


Figure 48: Cross-section of middle steel girder.

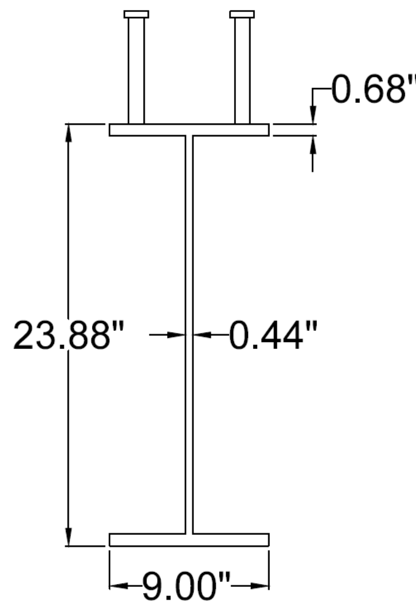


Figure 49: Cross-section of end composite beams

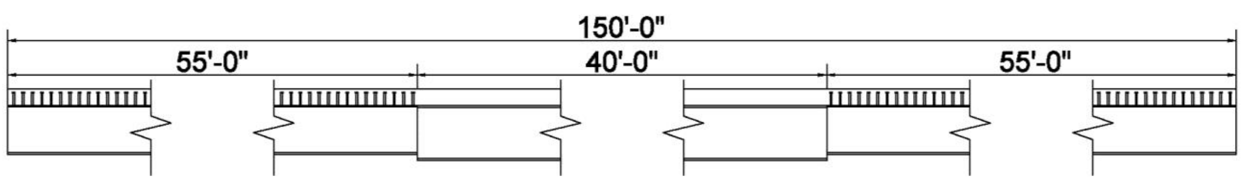


Figure 50: Longitudinal view of continuous composite design

The two end beams were designed as fully composite beams to resist the governing moment, vertical shear and horizontal shear. The steel anchors were designed to be 7/8-inch diameter with a 6-inch tall shaft and 2 anchors per row spaced 3” from the center of the stud to each side of the beam centerline. A total of 144 rows of steel anchors were required per beam at a 4.58” spacing to resist the horizontal shear. The steel anchors may seem relatively large compared to the thickness of the slab, but if this bridge were to be selected for a final design, haunches would be added. The most economic standard section which passed all the design checks was the W24X76 beam. A girder design was also considered for the design of the end beams; however the standard shape proved to be sufficient at resisting loads to make the economic advantages of the girder obsolete.

The beam segments used at the center of the span were designed as non-composite beams in order to effectively resist the negative moment. Steel studs were not designed for this beam section because it was designed as a non-composite section. The capacities with their governing loads are displayed above in Table 8, all other specific information on the bridge is displayed in Tables 9 and 10.

Table 9: Bridge information for steel continuous span using composite sections.

Bridge Information		
Beam Spacing	9'-8.571"	
Number of Spans	2	
Bridge Width	74	ft
Total Bridge Length	150	ft
Number of Individual Beams	24	
Steel Anchors	3904	
Anchor Spacing	9.167	in
Number Of Individual Bearings	24	
Number of Piers	1	
Total Concrete Used in Super Structure	308.33	CY
Total Steel Used in Super Structure	61.642	tons

Table 10: Beam descriptions for steel continuous span using composite sections.

Beam Information		
End Beams-W24X76		
Height Top of Deck to Bottom of Beam	32.9	in
Deck Width	74	ft
Flange Thickness	0.68	in
Flange Width	9	in
Web Thickness	0.44	in
Cross Sectional Area	22.4	in^2
Number of Sections	16	
Length of Section	55	ft
Steel Per Beam	8.556	ft^3
Center Beams-Plate Girders		
Height Top of Deck to Bottom of Beam	28.96	in
Deck Width	74	ft
Flange Thickness	1.13	in
Flange Width	19	in
Web Thickness	0.325	in
Cross Sectional Area	51.6175	in^2
Number of Sections	8	
Length of Section	40	ft
Steel Per Section	14.338	ft^3

Single Span

The composite single span design was designed using one beam section spanning the entire 150 foot span without any pier. Since the maximum length section that can be reasonably shipped is 120 feet, the span would consist of 8 beams, 120 feet in length and spliced with another 8 beams 30 feet in length with the same spacing. The beams would be spaced at 9'-8.571". The design moment for the single span was much more than the continuous span, thus greatly increasing the amount of steel required in the girders. This made it necessary to design the sections for partial composite action. The beam was designed using steel anchors with 7/8-inch diameter heads with a 6-inch tall shaft and 2 anchors per row spaced 3" from the center of the stud to each side of the beam centerline. The bridge used 596 rows of steel anchors spaced at 3.02" apart. All of the rows must be welded to the steel flanges; Figure 51 shows a typical section with studs. Standard sections were considered for this design, but it was decided they were not economically feasible to design. The capacities with their governing loads are displayed in Table 11, all other specific information on the bridge is displayed in Tables 12 and 13.

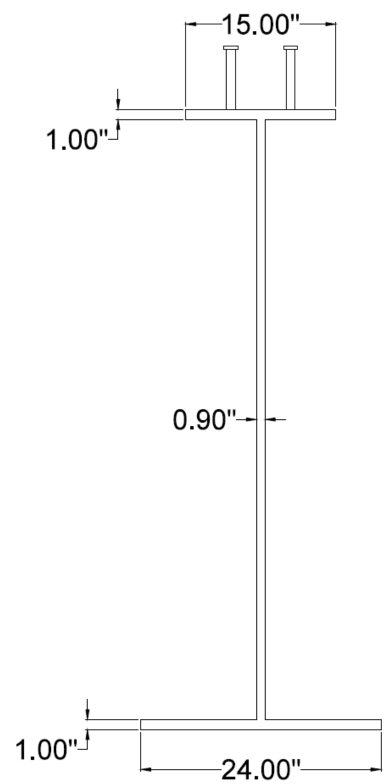


Figure 51: Single Span Composite cross-section.

Table 11: Governing loads and capacities for the steel single span design using composite sections.

ϕM_n	12835.2694 k-ft	M_u	10359.26 k-ft
ϕV_n	1295.509 k	V_u	283.857 k
$\sum Q_n$	9322.331 k	Q_u	9300 k

Table 12: Bridge information for steel single span using composite sections.

Bridge Information		
Beam Spacing	9'-8.571"	
Number of Spans	1	
Bridge Width	74	ft
Total Bridge Length	150	ft
Number of Individual Beams	16	
Number Of Individual Bearings	16	
Number of Piers	0	
Total Concrete Used in Super Structure	308.33	CY
Total Steel Used in Super Structure	189.875	tons

Table 13: Beam descriptions for steel single span using composite sections.

Beam Information		
Girder		
Height Top of Deck to Bottom of Beam	71	in
Deck Width	74	ft
Top Flange Thickness	1	in
Top Flange Width	15	in
Web Thickness	0.9	in
Web Depth	60	in
Bottom Flange Thickness	1	in
Top Flange Width	24	in
Cross Sectional Area	93	in^2

4.3.1.3 Non-Composite

The two scenarios that were considered for non-composite steel design were single span and a 2-span continuous orientation. The single span design was disregarded due to economic feasibility and inefficiency at resisting loads. The capacities and the governing loads of the continuous span are displayed below in Table 14. Tables 15 and 16 display all other pertinent information.

Table 14: Governing loads and capacities for the steel continuous design using non-composite sections.

ϕM_n	2949.45 k-ft	M_u	2905.01 k-ft
ϕV_n	152.01 k	V_u	149.16 k

Table 15: Bridge information for steel continuous span using non-composite sections.

Bridge Information		
Beam Spacing	9'-8.571"	
Number of Spans	2	
Bridge Width	74	ft
Total Bridge Length	150	ft
Number of Individual Beams	24	
Number Of Individual Bearings	24	
Number of Piers	1	
Total Concrete Used in Super Structure	308.33	CY
Total Steel Used in Super Structure	105.389	tons

Table 16: Beam descriptions for steel continuous span using non-composite sections.

Beam Information		
End Girder		
Height Top of Deck to Bottom of Beam	37.6	in
Deck Width	74	ft
Top Flange Thickness	0.95	in
Top Flange Width	16	in
Web Thickness	0.325	in
Web Depth	26.7	in
Bottom Flange Thickness	0.95	in
Top Flange Width	16	in
Cross Sectional Area	39.078	in^2
Center Girder		
Height Top of Deck to Bottom of Beam	28.96	in
Deck Width	74	ft
Top Flange Thickness	1.13	in
Top Flange Width	19	in
Web Thickness	.325	in
Web Depth	26.7	in
Bottom Flange Thickness	1.13	in
Top Flange Width	19	in
Cross Sectional Area	51.618	in^2

4.3.2 Precast Preliminary Design

Using the bridge type selected during the initial design studies; the 35” deep Deck Bulb Tee in a two-span configuration with a central pier, a preliminary design was developed in *PG-Super*. The beam characteristics are presented in Table 17 below, and the bridge cross section is shown in Figure 52.

Table 17: Precast beam design information.

Beam Information		
Type	W35DG	
Height Top of Deck to Bottom of Beam	35	in
Deck Width	72	ft
Top (Deck) Flange Thickness	6	in
Flange Width	25	in
Web Thickness	6	in
Cross Sectional Area	850	in^2
Number of Straight Strands	14	
Number of Harped Strands	6	
Total Strands	20	
Length of Beam	75	
Concrete Per Beam	16.3966	CY
Steel Strand Weight Per Beam	884	lbs.

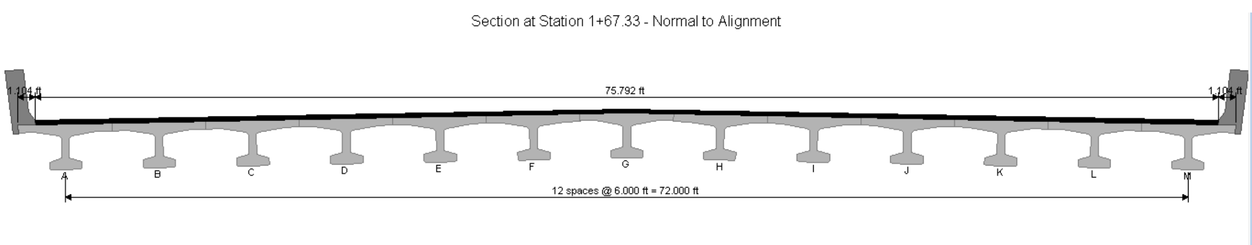


Figure 52: Bridge Cross Section, From PG-Super

The design developed consisted of two roughly 75 foot spans. Each span consisted of 13 beams, for a total of 26 beams. There was a single pier located in the center of the bridge. The program had automatically included a 2% slope from the centerline of the bridge to allow for drainage. For the purposes of design, a standard F Type Barrier had been added to each side of the bridge to allow for load calculations (the weight of a typical jersey barrier was included in the steel calculations, weighing about 400plf). A summary of the precast bridge design can be found in Tables 17 and 18. Each beam weighed roughly 32 tons, so it could be transported on a standard truck and erected by a single crane (McCormac).

Table 18: Preliminary Precast Bridge Information

Bridge Information		
Beam Spacing	6	ft
Number of Spans	2	
Individual Span Width	78	ft
Individual Span Length	75	ft
Total Bridge Length	150	ft
Number of beams per span	13	
Number of Individual Beams	26	
Number Of Individual Bearings	52	
Number of Piers	1	
Total Concrete Used in Super Structure	426.312	CY
Total Steel Used in Super Structure	11.492	Tons

4.4 Evaluation of Alternatives

4.4.1 Construction Time and Labor Estimates

Using *R.S. Means Heavy Construction 2009*, and the method described in the Development of the Construction Time and Labor Estimate section (3.6) above, a complete set of expected accelerated construction erection times was completed for each of the three preliminary bridge types. These erection times, as well as expected crane size and manpower needs are outlined below in Table 19. The Work hours to finish bridge column is the estimated total girder erection time.

Table 19: Table of the erection requirements of the three preliminary bridge types.

Superstructure Construction Requirements According to RS-Means									
Bridge Design Type	Crane Required	Total Man Hours	Work Hours Required to Finish Bridge	Total Number of Workers Required	Steel Foreman	Steel Workers	Crane Operators	Welders	Misc. Workers
Simple Two Span Precast	Truck Mounted Hydraulic Crane--40 Ton Capacity	134.1	14.9	9	1	6	1	0	1
Two Span Steel Beam	Truck Mounted Hydraulic Crane--40 Ton Capacity	49	4.9	10	2	5	1	1	1
Single Span Steel	Truck Mounted Hydraulic Crane--100 Ton Capacity	110	11.0	10	2	5	1	1	1

4.4.2 Cost Estimates

Cost estimates for the three alternatives are shown below in Table 20. The unit costs used to develop this chart are contained in Table 20.

Table 20: Preliminary Design Costs for Screening Super Structure Type

Superstructure Type	Total Reinforced Concrete For Structural Member (CY)	Total Structural Steel For Structural Members (Tons)	Total Reinforced Concrete for Deck not included in members	Total Pier Walls Required	Total Bearings Required	Total Cost
Simple Two Span Precast	426.31	0	0	1	52	$ 500,500
Continuous Two Span Steel	0	61.64	308.33	1	24	$ 611,000
Single Span Steel	0	189.875	308.33	0	16	$ 928,500

Table 21: Installed Unit Costs for Design Estimate

Unit	Prices per unit According to NH DOT Contract 13933N
CY of Superstructure Concrete	$ 600.00
Pound of Structural Steel	$ 1.72
CY of Pier Concrete	$ 350.00
CY of Deck Concrete	$ 750.00
Each Elastomeric Bearing	$ 2,750.00

4.5 Final Bridge Type Choice

The choice of the final bridge type ended up being relatively straightforward. In the price category, as can be seen in Table 19, the Two-Span Precast Bridge estimate is over $100,000 less than the estimate for the Two-Span Continuous Steel Bridge and more than $400,000 less than the Single Span Steel Bridge. It was true that the calculations did not necessarily represent the exact costs of a single span vs. two span alignment, but they represented a sufficient estimate to select a design that had a nearly $100,000 price advantage. The costs would likely skew more to favor precast construction if the cost of the far larger crane needed for the Single Span Steel Bridge was used.

The second factor of construction time did not favor the Two Span Precast Bridge, but did not seriously hamper it. The construction times calculated from *RS-Means Heavy Construction 2011* for each type of bridge are shown in Table 18. The Precast Bridge actually had the longest construction time, at nearly fifteen working hours. However, 15 working hours was not considered a concern because it could be accomplished through night erection or through short closure. The precast bridge had the lowest crew cost per working hour as well, and used the same reasonably sized crane as the Two Span Continuous Steel bridge.

Reflecting actual conditions, erection times were rounded up to the nearest day. This eroded much of the price advantage of the steel bridging option. The steel option could be erected significantly faster, but, still would, be necessity take one day. The estimated crew cost for the steel erection from RS Means Heavy Construction 2009 is $4300, and $4600 when adjusted up for four years of inflation, and discounted by the factor for construction in New Hampshire. For the prestressed concrete bulb tees, the required crew is estimated at $3900 and $4100 respectively. This leaves the difference between cost of the erecting crews to be $8200-$4600=$3600. This is a significant amount of money, but not for a bridge project. Stated differently, the cost savings of having the steel bridge crew for a day instead of the prestressed bridge crew for two days is just over .5% of the total bid price of either option.

Therefore, the erection cost/time to erect difference was not significant enough to impair the chances of any of the options. The purpose of the time estimate was to eliminate options that could not be erected in a few nights or over the course of the weekend. All of the preliminary designs met the criteria to be viable Accelerated Bridge Construction options.

As far as aesthetic value, the comparison of the large, weathering steel plate girders with the precast deck bulb tees is about even. Some people prefer the long, sweeping nature and consistent maroon color of the single-span weathering steel girder. Others prefer the flowing shape and consistent light color of the Deck Bulb Tees, which if combined with a Hammerhead pier in final design could present an impressive bridge. The only design with significantly negative visual appeal is the Two-Span Continuous Bridge. The splicing of two smaller end beam types to a much larger central plate girder would look obtuse, and would continuously confuse onlookers. Traditional bridge design avoids the splicing of beams of greatly different sizes both for this reason and the difficulty and expense in designing and constructing such a splice. The clear loser in terms of aesthetic value was therefore the Two-Span Continuous Bridge.

The choice of the final design, considering the above considerations, was precast construction: it costs less than either of the other two choices, and was significantly more visually appealing than the continuous two span steel design.

5. Design Development

The development of the final design was a multi-step process. First, using spread sheet software, a program was created to determine the governing AASHTO load cases, along with corresponding ultimate moment and shear values for our specific span and framing geometry. These values, expressed as the forces on either an exterior girder were then used to design the girders. The prestressed Deck Bulb Tee girders were originally sized using PG-Super, and prestressing volume, forces, and locations were checked and refined using the loads calculated in the spreadsheet and the AASHTO LRFD Bridge Specification. The members were also designed for vertical shear using AASHTO loads but according to PCI guidelines. Cantilever abutments were also designed using input data from the load combination spreadsheet and software providing by the New York Department of Transportation website. Separately, a pier structure was developed to support the bridge superstructure. This involved a separate load analysis using demonstration RISA 2D, and an extensive design of the pier, pier cap and pier footing according to the AASHTO LRFD Bridge Specification. Supporting hardware such as diaphragms and bearings are specified generally, but are not designed in the scope of the project.

Using the expected design a comprehensive construction plan, including all detailed and undetailed but crucial (bearings, approach slabs, diaphragms) construction tasks was developed. This schedule includes the precedents and expected durations of tasks. A traffic plan was developed to minimize impact on the community during construction, and to meet all of the other factors outlined in the traffic plan methodology. The traffic plan is directly constructed on the expected construction progress from the construction plan. Finally, a total cost was developed considering both the final design and the manor of its construction and implementation. This estimate includes two parts, the “A” portion, or the direct labor and material cost of construction, and the “B” component, the costs inflected on road users and the community by construction activities. The same style of cost estimate was prepared for the actual State design using quantity takeoffs from the state drawings. This is to be used for comparison.

5.1 Superstructure Design

5.1.1 Loads

The data obtained from the AASHTO moment analysis was used to design the flexural reinforcement of the bridge beams. The governing results from the flexural load study can be seen in Table 22.

Table 22: Moments for AASHTO loading conditions on interior girders.

	Interior Beams Moment, k-ft	
	Maximum	Minimum
Strength I	**3472.73**	3129.32
Strength II	2938.12	2594.71
Strength III	999.69	656.28
Strength IV	1155.33	656.28
Strength V	2938.12	2594.71
Extreme Event I	1717.62	1374.22
Extreme Event II	1717.62	1374.22
Service I	2206.09	
Service II	2636.85	
Service III	1918.92	
Service IV	770.22	
Fatigue I	2049.49	
Fatigue II	1024.74	

As the table shows, the moment on the interior beams, due to Strength I load factors, was the greatest and therefore governed the flexural design of the beam. The full results from the AASHTO limit state moment study can be seen in Appendix 3.1.

The data obtained from the AASHTO shear analysis was used to design the shear reinforcement of the bridge beams. The governing results from the AASHTO shear load study can be seen in Table 23.

Table 23: Shear results for AASHTO Loading Conditions for Interior Beams

	Interior Beams Shear, kips			
	One Design Lane Loaded		Two or More Design Lanes Loaded	
	Maximum	Minimum	Maximum	Minimum
Strength I	172.84	154.53	**185.44**	167.12
Strength II	145.52	127.21	155.24	136.92
Strength III	53.32	35.00	53.32	35.00
Strength IV	61.62	35.00	61.62	35.00
Strength V	145.52	127.21	155.24	136.92
Extreme Event I	87.47	69.15	91.07	72.75
Extreme Event II	87.47	69.15	91.07	72.75
Service I	109.38		116.58	
Service II	129.87		139.23	
Service III	95.72		101.48	
Service IV	41.08		41.08	
Fatigue I	102.45		113.25	
Fatigue II	51.23		56.62	

As Table 23 shows, the shear on the interior beams due to Strength I load factors, and with two or more lanes loaded, was the greatest and therefore governed the shear design of the beam. In order to design the shear reinforcement, a spreadsheet was developed to calculate the ultimate shear due to

Strength I load factors at 10 points along the length of the beam. The full results from the AASHTO limit state shear study can be seen in Appendix 3.1.

5.1.2 Flexural Design

The girders on the bridge have been designed as 35" Deck Bulb Tees. The depth was measured from the top of the "Deck" portion of the beam to the bottom of the web. The deck bulb tee shape used in design was directly based on the Washington Department of Transportation shape named DBT W35DG. The concrete is required by design to have an initial strength of 6800 psi and 28-day strength of 8000 psi. This is primarily to prevent crushing of the concrete at the bottom of the beam at transfer prior to Superimposed or Live loading. Deck Bulb tees are T-shaped bridge members used without any additional structural deck cast over the members. Both the depth of the flange and the width of the web were six inches.

The design required a total of 26 prestressing strands. Strands were low-relaxation and were formed from seven steel wires. 18 strands were strait and 8 strands were harped, with a harping length of 27.836 feet. No longitudinal steel is required for flexural capacity. The cross section of the bridge member with prestressing strand locations can be seen below in Figure 53. A length wise view of the beam, showing harping locations, angle and strand eccentricity can be seen in Figure 54. Figure 55 shows the plan view of the girder system.

Individual beam bearings would be necessary for each girder line, and it is also necessary to construct sufficient transverse reinforcing diaphragms between beams to prevent relative vertical displacement. Although the need for these elements is noted, the design of the bearings and the bridge diaphragms was beyond the scope of this project.

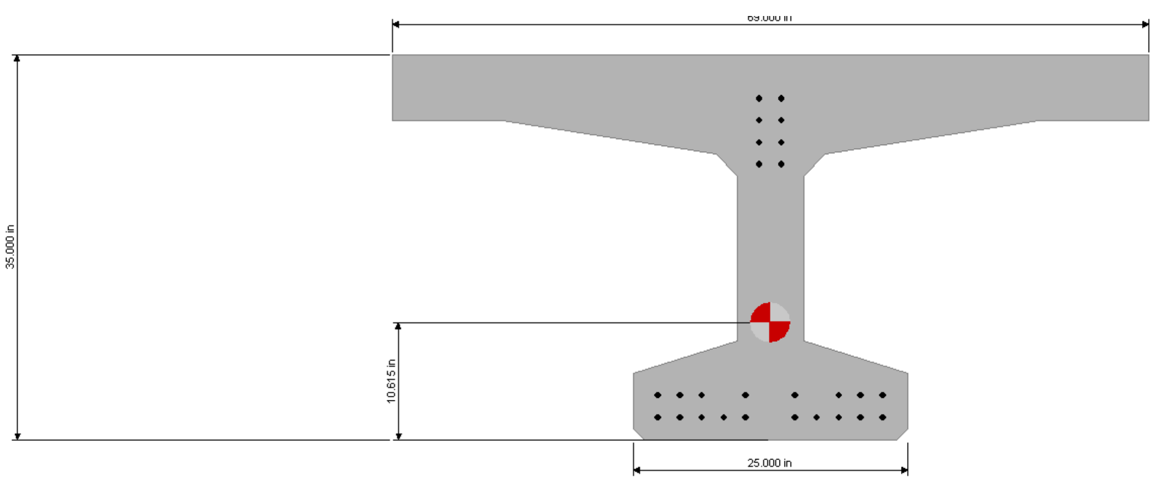


Figure 53: Prestressed Deck Bulb Tee Cross Section

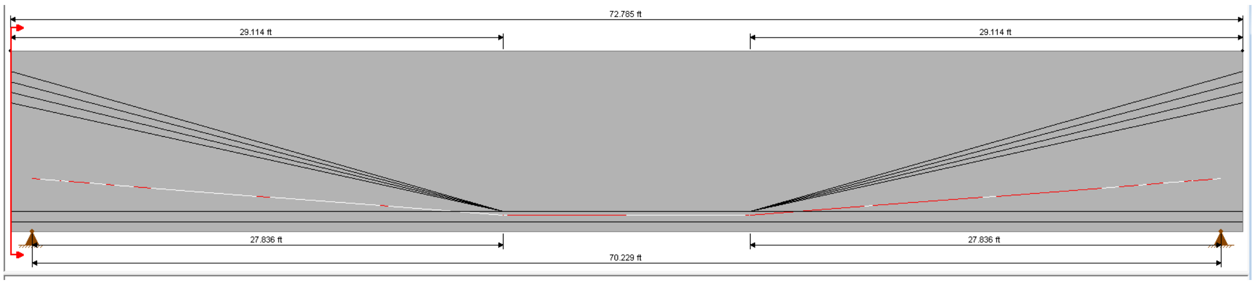


Figure 54: Prestressed Deck Bulb Tee Elevation View

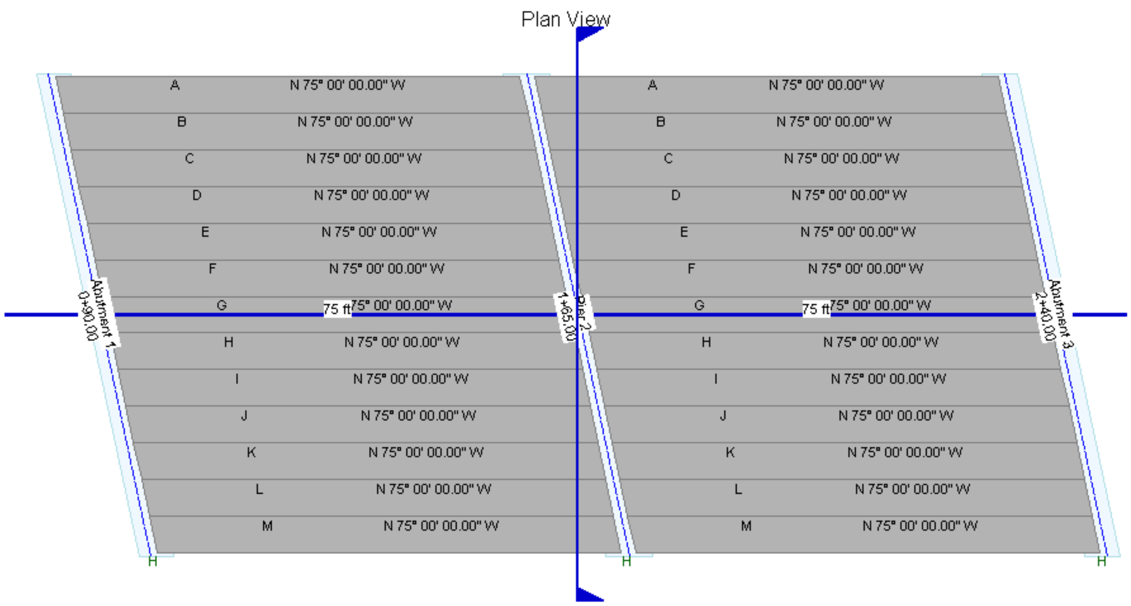


Figure 55: Plan view girder system consisting of twenty-six Prestressed Deck Bulb Tees

5.1.3 Vertical Shear Design

The data obtained from the AASHTO limit state shear analysis was used to design the shear reinforcement of the bridge beams. Figure 56 below shows the relationship between the ultimate and nominal shear capacity of the prestressed bridge beam. Shear was calculated at increments equal to 10% of the span length and at the critical section, h/2, from each end. Two stirrup designs were developed for, the ends (0-40% and 60-100% of the span length), and the center of the beam (40-60% of the length). The end design consisted of 2 legs of #4 stirrups spaced 14" apart. The center design consisted of 2 legs of #4 stirrups spaced 24" apart. Figures 57 and 58 show a lateral view of the prestressed bridge beam with shear reinforcement.

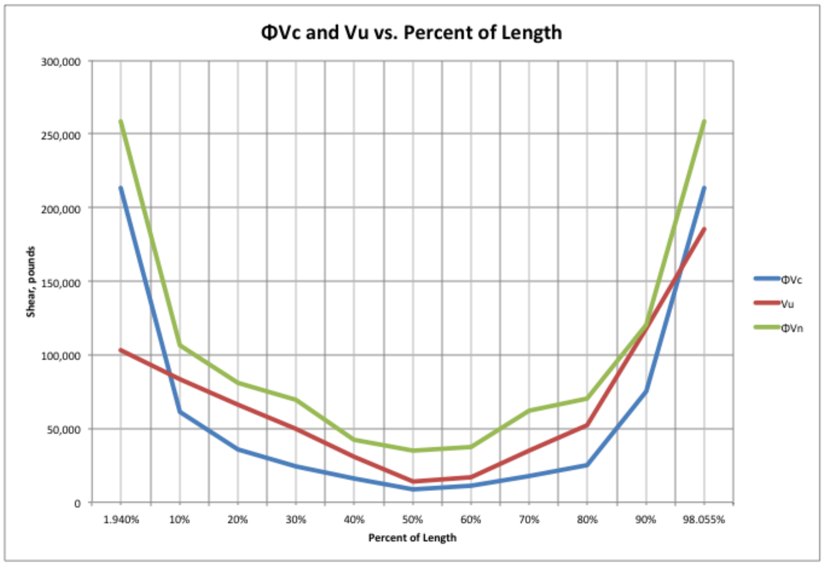


Figure 56: Chart of Design Shear (Vu) vs. Factored Shear Capacity (ϕVc) along the Percentage of Span Length.

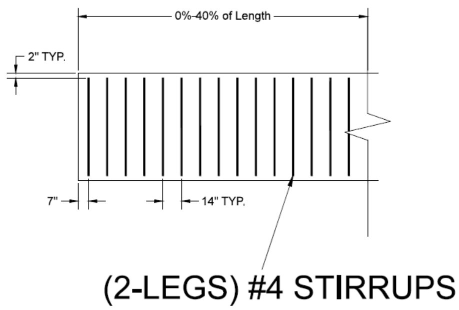


Figure 57: Shear stirrup spacing for ends of girder spans

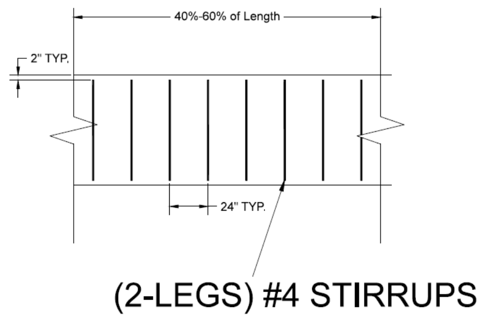


Figure 58: Shear stirrup spacing in center of girder spans

5.2 Pier Design

Out of the many different types of intermediate supports, the three that were considered for this project were a pier wall, hammerhead pier, and a row of columns supported by a single pile cap. The pier wall was considered in order to take up the minimum amount of space, the hammerhead pier was taken into consideration for its aesthetics, and the pile bent was considered for its constructability and structural efficiency. The pier wall was ruled out quickly for aesthetic reasons and the pile bent was selected for the final design. Two adjacent hammerhead piers would have been used but they would not have been cost effective due to the fact that they would have to be short and wide. The foundations were designed as precast and the pier cap and columns were designed as cast-in-place structures in order to avoid problematic connections between precast components.

The dimensions provided by the *FHWA* Intermediate Pier Design Example (FHWA) were used as an initial approximation. The example pier was scaled to fit the geometry of the superstructure and an extra column was added. The design was analyzed and revised from this point.

5.2.1 Loads

The loads placed on the pier structure included the dead load due to the future wearing surface (DW), dead load of structural members (DC), live load (LL), braking force (BR), wind load on the structure (WS), and wind load on the live load (WS). *RISA* models were created in the transverse section for the design of the pier cap and columns in order to maximize moment, axial load, and shear and minimize the axial load in the columns. The loads applied to the transverse model of the substructure can be seen in Figures 59 through 64 below.

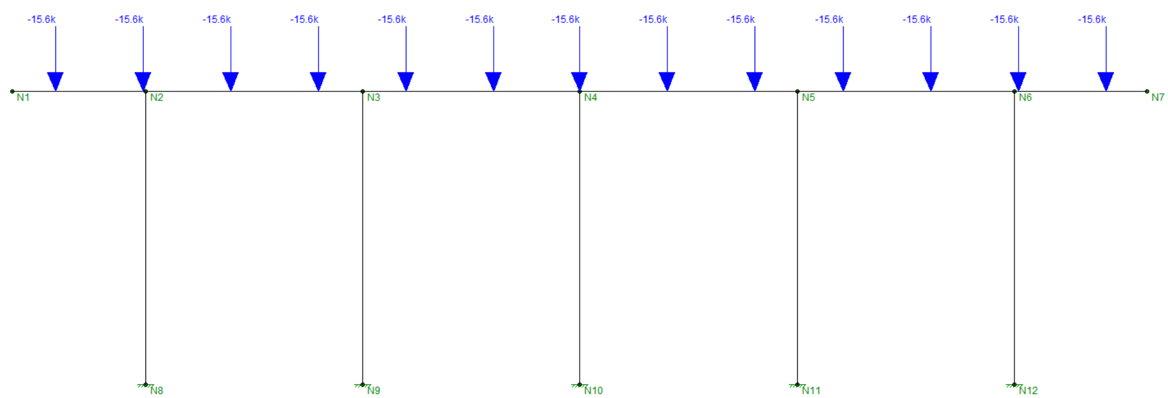


Figure 59: Dead load of future wearing surface (DW) applied to the transverse pier section.

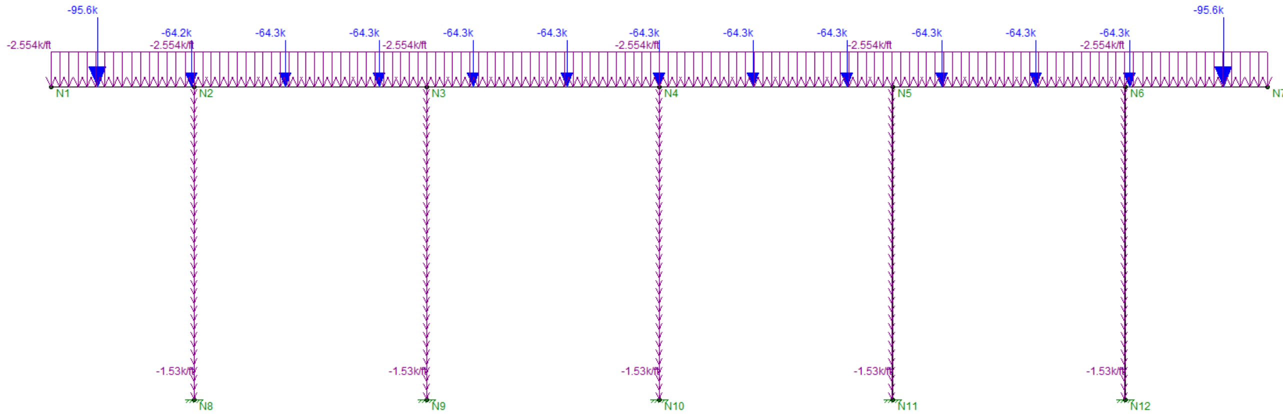


Figure 60: Dead load due to structural members (DC) applied to the transverse pier section.

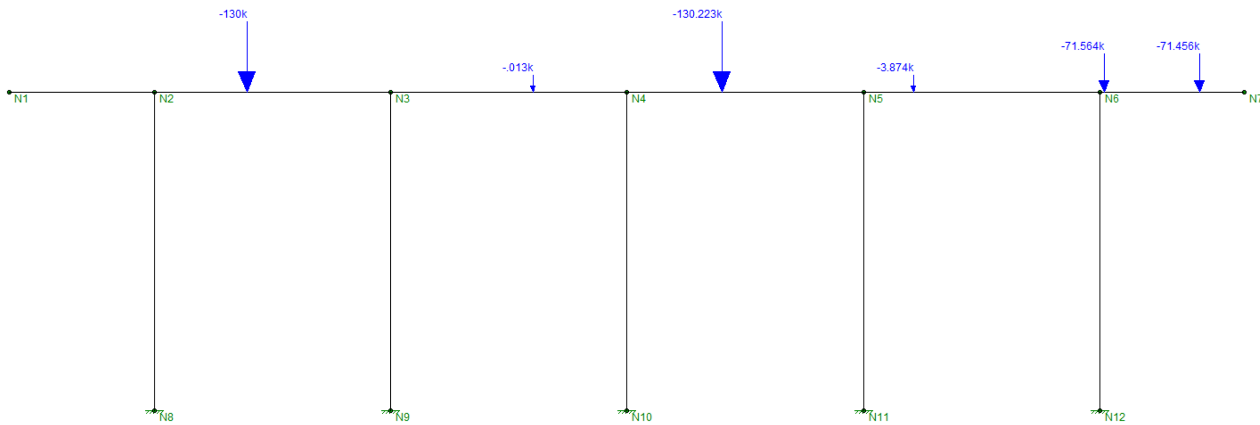


Figure 61: Live load causing maximum moment and shear (LL) applied to the transverse pier section.

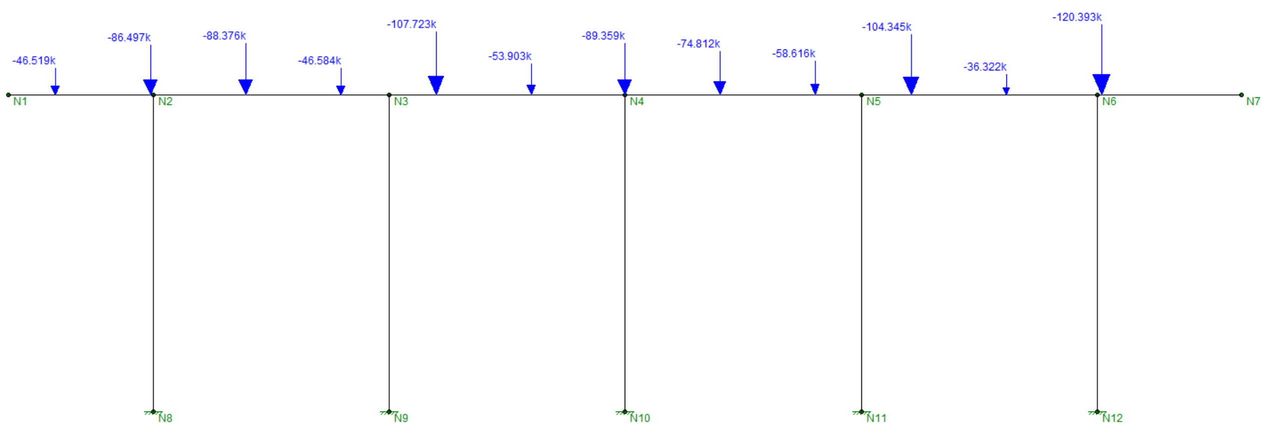


Figure 62: Live load causing maximum axial load (LL) applied to the transverse pier section.

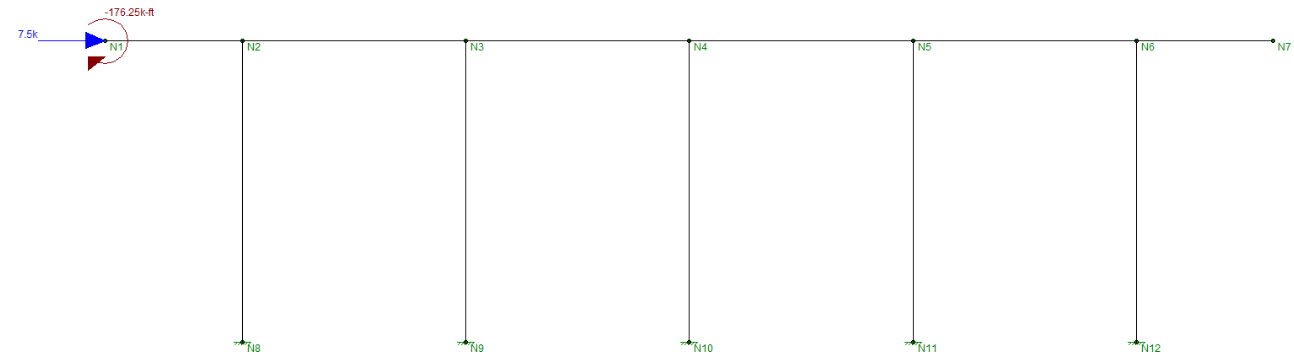


Figure 63: Wind effect on live load (WL) applied to the transverse pier section.

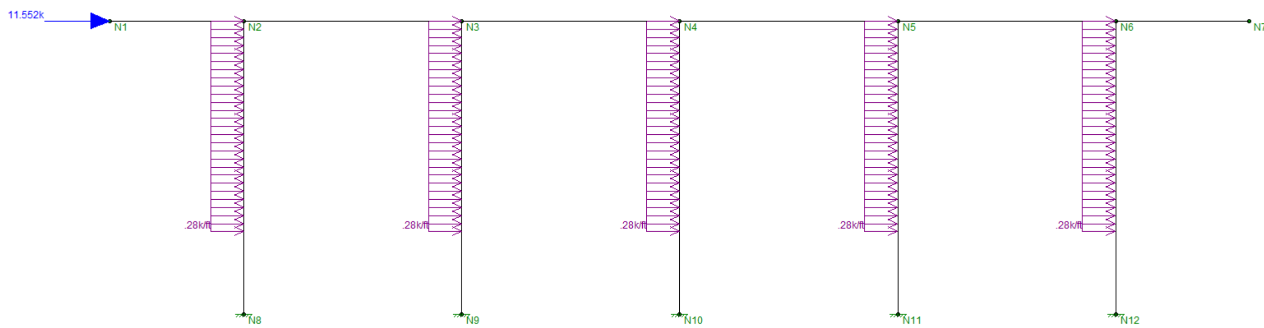


Figure 64: Wind force on structure (WS) applied to the transverse pier section.

Vehicle braking forces and other effects of the wind on the longitudinal section were also determined for the design of the columns. The critical values for design loads are displayed in the Tables 24 and 25 below. Strength Case I was the governing load combination for the design of the pier, and Strength Case V was the governing combination for the columns. The minimum column reaction was 197 kips, this was important because columns need to fall within the capacity of the column interaction diagram as shown in Figure 68.

Table 24: Critical values obtained from the transverse model used to design the pier cap.

Reactions Caused by Maximum Axial Load Case			
	Pier Cap $+M_{max}$ (k-ft)	Pier Cap $-M_{max}$ (k-ft)	Pier Cap V_{max} (k)
Strength I	435.5	1478.5	485.3
Strength V	-443.2	1279.8	477.5
Service I	309.2	873.4	333.9
Reactions Caused by Maximum Moment Load Case			
	Pier Cap $+M_{max}$ (k-ft)	Pier Cap $-M_{max}$ (k-ft)	Pier Cap V_{max} (k)
Strength I	**849.1**	**1814.9**	**524.9**
Strength V	722.9	1698	467.7
Service I	-545.5	1302.5	356.5

Table 25: Critical values obtained from transverse and longitudinal model used to design columns.

	P_u (k)	V_T (k)	V_L (k)	M_T (k-ft)	M_L (k-ft)
Strength I	**824.3**	0	16.4	7.7	575.6
Strength V	789.3	9.5	38.4	83	**1056.7**

5.2.2 Pier Cap

The pier cap was designed as a 74'-9" long rectangular concrete member with mild reinforcing steel. The concrete was reinforced with steel on the bottom and top of the cross-section to resist the negative and positive moments. The concrete used for the cap had a 28-day compressive strength of 3, ksi and the steel had a yield strength of 60 ksi. The transverse view of the pier can be seen in Figure 65 and both the longitudinal and transverse cross-sections can be seen in Figure 66 and 67. A total of six #8 bars were used for the bottom, reinforcement and thirteen #8 bars were used for the top. Four #7 bars were used per face spaced at 18 inches to provide resistance from temperature and shrinkage, and four #7 bars were also used per face for the skin reinforcement required by AASHTO. Steel stirrups consisting of 4 legs of #5 bars were placed at a minimum spacing of 8 inches over the pier cap. There will be 112 stirrups spaced 8 inches apart across the transverse section due to the variety of possible load combinations.

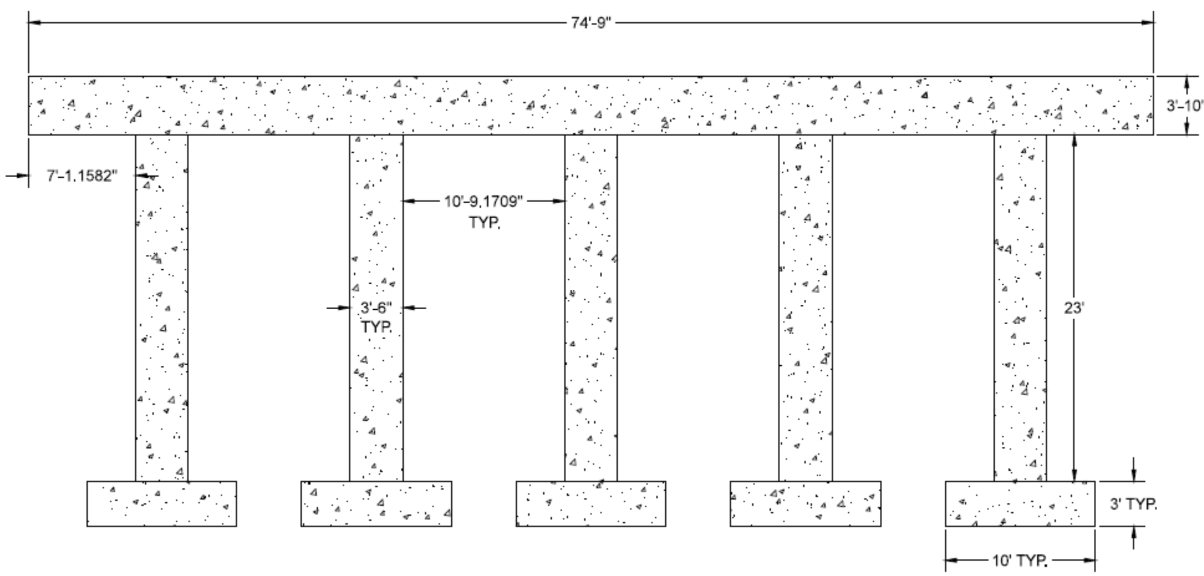


Figure 65: Overall Elevation View of Pier

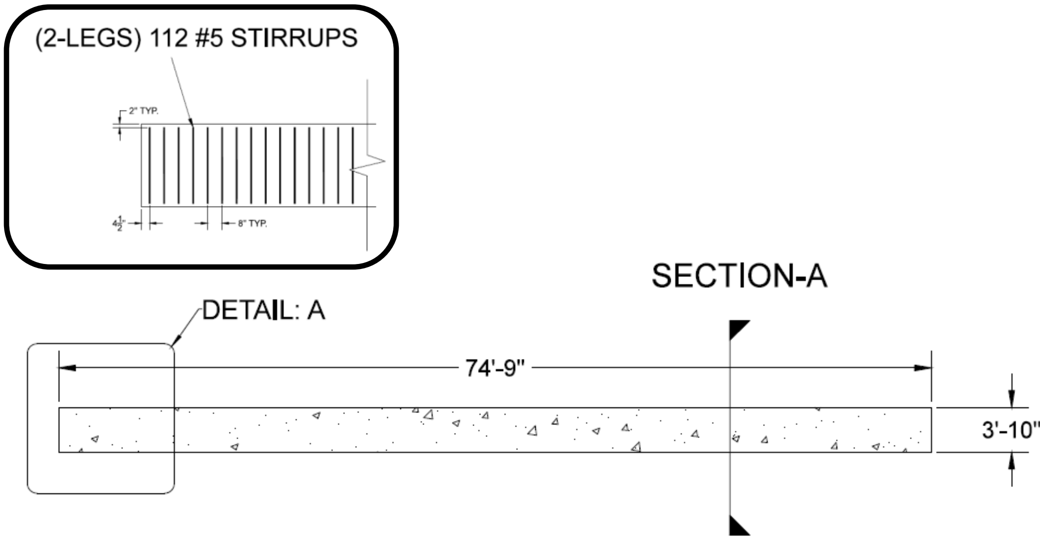


Figure 66: Longitudinal View of Pier Cap, with detail of shear stirrup spacing.

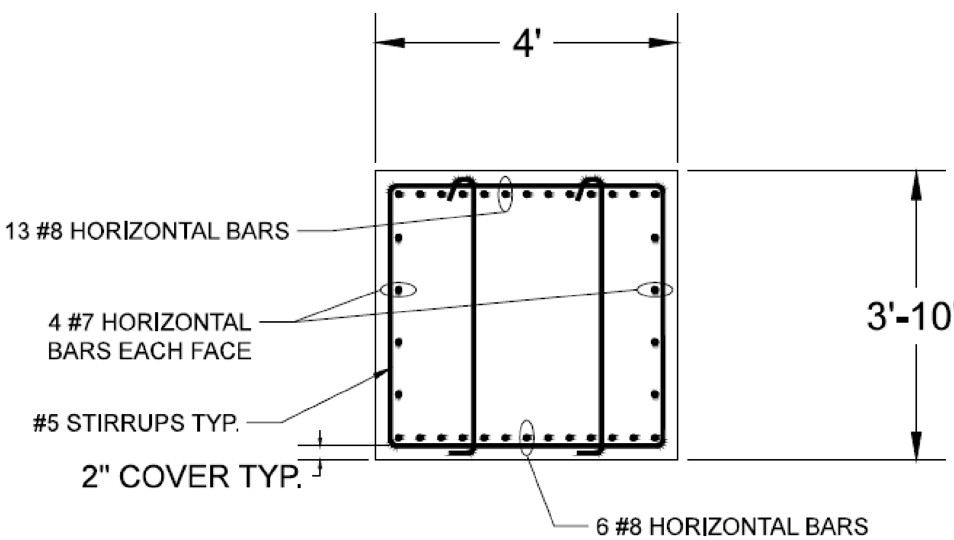


Figure 67: Cross-sectional view of Pier Cap, showing both longitudinal reinforcing and shear reinforcing.

5.2.3 Pier Columns

A total of 5 columns were designed using concrete with a compressive strength of 4 ksi and steel with a yield strength of 60 ksi. The dimensions and reinforcement were initially taken from a design example found provided by the FHWA (FHWA) and the proportions were adjusted. The columns selected had circular cross-sections with a diameter of 3.5 feet. The columns were reinforced with sixteen # 8 bars and tie reinforcement instead of spiral reinforcement for convenience during installation. Requirements for the spacing of ties are stated to be, “The spacing of ties along the longitudinal axis of the compression member shall not exceed the least dimension of the compression member or 12 in” (AASHTO, 2012). Since the column is rather large, the governing tie spacing is 12 inches center-to-center. The longitudinal and transverse moments taken from the *RISA* models were magnified in accordance with Section 4.5.3.2.2b in the *AASHTO LRFD Specifications* by combining the longitudinal and transverse values to create an ultimate moment of 1103.83 k-ft. This moment was combined with the minimum and maximum axial load and plotted on the column interaction diagram (Figure 68) shown below to determine if the column was structurally adequate. The columns were capable of resisting over twice the required capacity and could be redesigned using less material. Due to the relative proximity to moving vehicles and the standardization of columns, the conservative design was kept. Figure 69 shows the cross-section of the pier column.

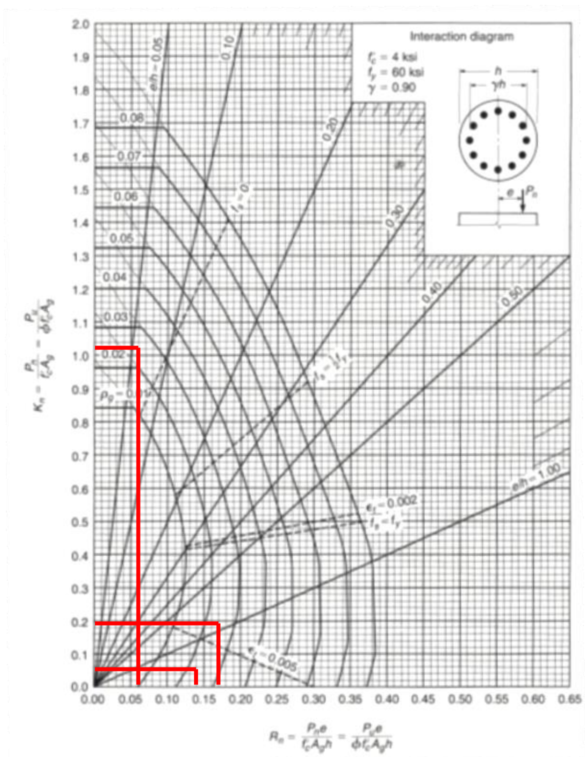


Figure 68: Interaction diagram for columns used in pier, taken from *Design of Concrete Structures* (Nilson, Darwin, Dolan)

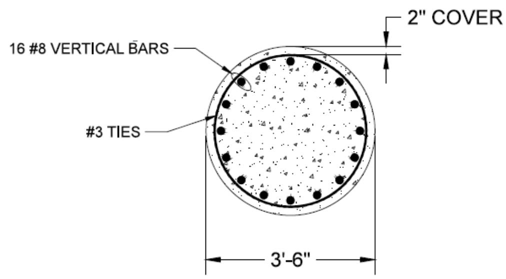


Figure 69: Cross sectional view of typical Pier Column.

5.2.4 Foundations

The site investigation was completed using the information obtained from the SPT tests in the boring logs provided by the State plans from 1959 (NHDOT). The representative soil profile at the point where the pier would be placed is shown in Figure 70 below. This soil profile was necessary for estimating the properties of the soil which were used in order to design footings properly secured in the ground. The ground-water table was determined to be below bedrock, and it did not impact the calculations. Appendix 3.2 contains spreadsheets (Coduto) used to determine the bearing capacity and settlement. The inputs to these spreadsheets consisted of the soil properties and loads.

The foundations were designed to provide adequate bearing capacity for maximum axial loads transferred through the columns and for settlement due to sustained loads from the columns. The foundations were chosen to be precast square spread footings for each column in order to increase the ease of production and construction while minimizing the cost. A 4.5’x4.5’ spread footing was required in order to meet the bearing capacity requirements which were significantly smaller than the dimensions required to meet minimum settlement requirements. Desirable numbers for compressibility were used for the settlement analysis since hard silt with sand and stones would be difficult to compress. A 10’x10’ shallow foundation was required in order to meet settlement requirements which proved to be the governing condition.

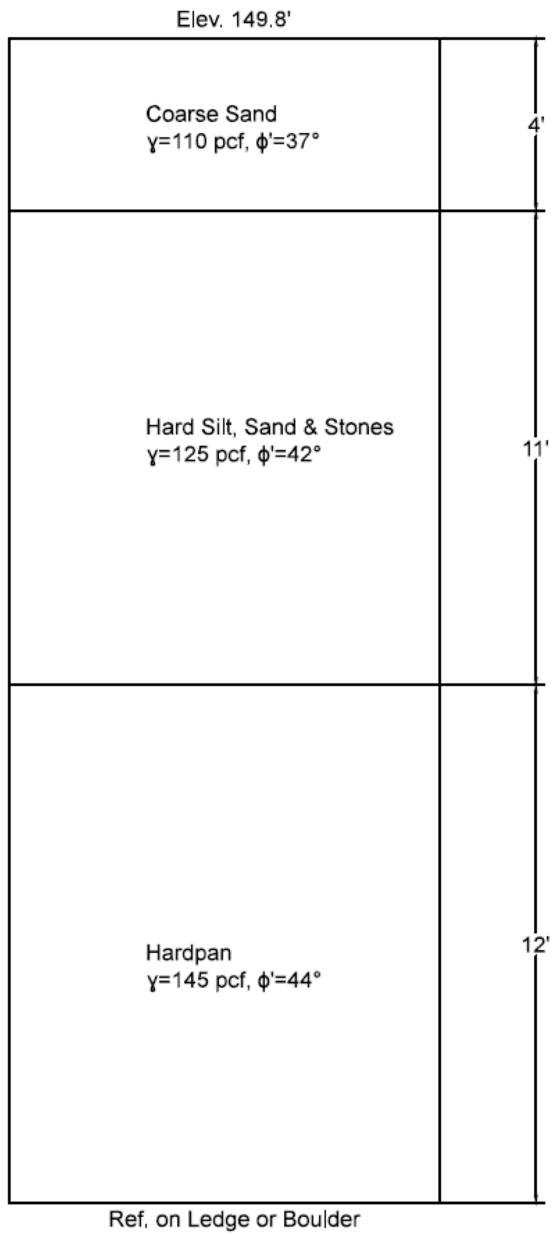


Figure 70: Representative Soil Profile at location of central pier.

5.3 Abutment Design

The abutment was designed using reinforcing bars with a yield strength of 60 ksi, concrete with a compressive strength of 4 ksi, 2.5 inches of clear cover for the reinforcement in the backwall and stem, and 3 inches of clear cover in the footing as required by section 5.12.3 of AASHTO. Figure 71 shows a longitudinal section of the abutment with dimensions.

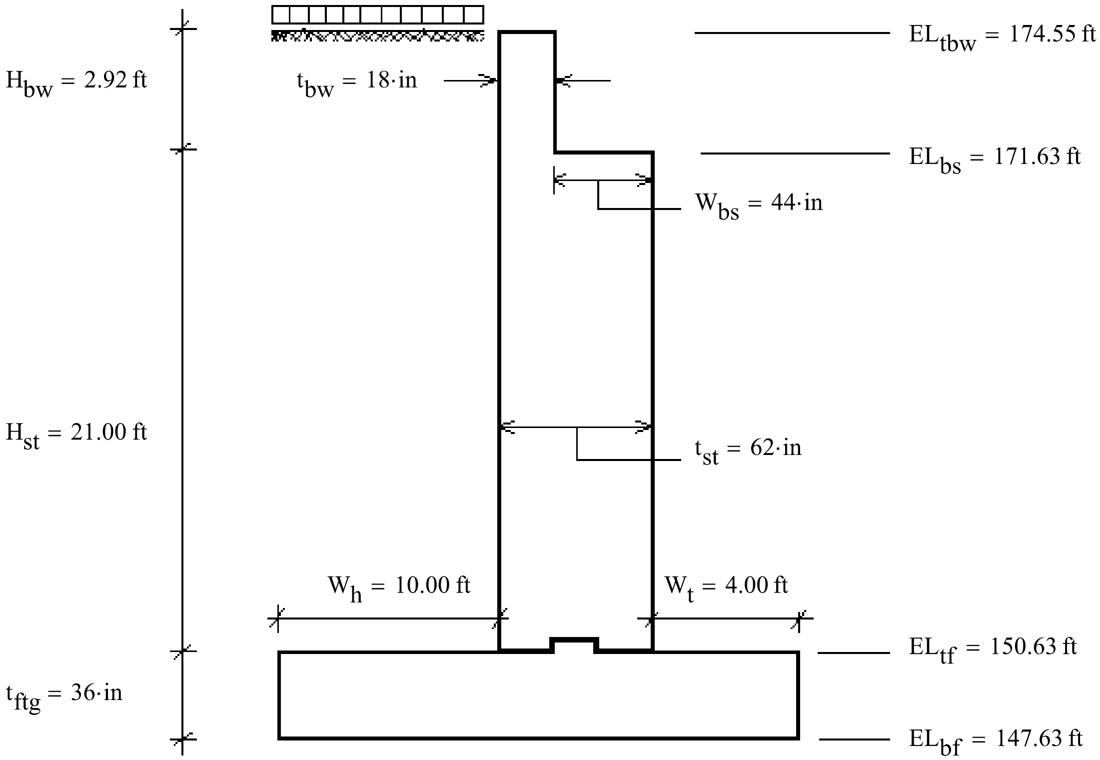


Figure 71: Longitudinal dimensioned section of cantilever abutment.

The first step in the abutment design was to determine the proper AASHTO load factors to use. After these were found, the NYDOT MathCAD program analyzed and combined the force effects based on the *AASHTO LRFD Bridge Specification*. The loads were combined in a similar manner as described for the pier design, but were treated as loads per linear foot. Live load effects were determined by maximizing the force on the structure and multiplied by their respective multiple presence factors, similar to the girder design of the superstructure.

In order to design the multiple components of the abutment, force effects were calculated for the abutment stem, backwall, and footing in order to design each component. The abutment was first broken down into components for the design. The backwall and stem were designed for shear and flexure while taking the dead load of the approach slab and worst-case-scenario live load conditions into consideration. Due to the fact that the approach slab sits on the abutment, section 3.11.6.5 of *AASHTO* specifies the surcharge on the backwall may be reduced. A surcharge reduction of 50% was selected since it is the value selected in the design example. The stem was designed using the same loads as the backwall in addition to the forces applied by the superstructure. Temperature was also taken into account in the design.

The footing was designed for bearing, shear, flexure, overturning, and sliding while taking into consideration the forces applied to the structure above the footing and the geotechnical information. Figure 72 shows the relevant reinforcement. The site investigation was completed as described in the methodology and yielded representative profiles for the North and South abutment locations as seen in Figures 74 and 75. Since the abutments were so massive and heavy, it was deemed that the foundations did not have to be below the frost level. The abutments were designed as cast-in-place since the rate of their construction would not greatly impact the community and it would also help to avoid issues with the connections. The site of the abutments had to be properly excavated, and once the structure has been erected, backfill must to be placed behind the abutments. The backfill was selected to have a unit weight of 125 pcf and an internal friction angle of 35 degrees which were very common soil properties for the area. The backfill must also be compacted before use.

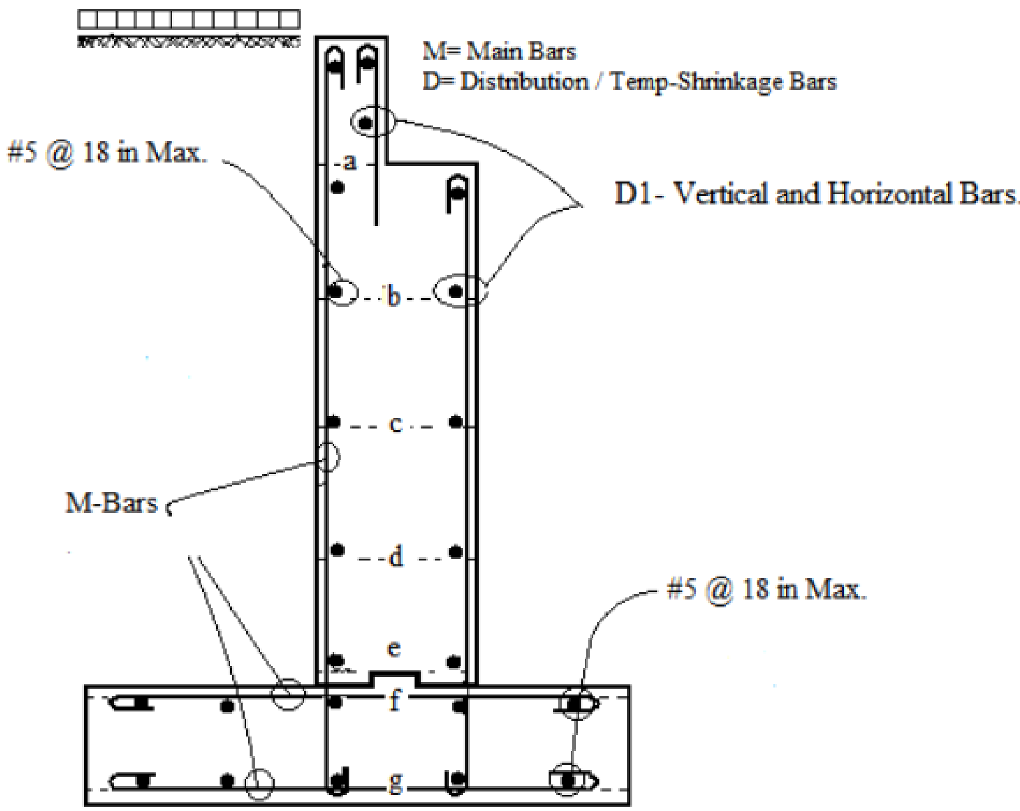


Figure 72: Abutment reinforcing bars.

The final abutment design is outlined in Table 26. In order to keep rebar sizes as consistent as possible, and for ease of construction, the vertical bars in the back wall were chosen as #6 bars. For the vertical bars, #6 bars at 6” spacing were chosen for the section from the bottom of the abutment to 13.42 feet from the top. For the section from the top of the abutment to 13.42 feet from the top, #6 bars at 12” spacing were chosen. Proper splicing was assumed to ensure the continuity of the rebar.

Table 26: Abutment Reinforcement Descriptions.

Bar Designation	Location	Bar Number	Bar Spacing
Vertical Moment Bars in Abutment Backwall	24'-13.42' from top of Abutment (each face)	#6	6"
Vertical Moment Bars in Abutment Backwall	13.42'-0' from top of Abutment (each face)	#6	12"
Horizontal Moment Bars in Abutment Footing	Bottom Bars	#5	6"
Horizontal Moment Bars in Abutment Footing	Top Bars	#6	6"

The horizontal reinforcing for the abutment footing was chosen as follows. For the horizontal bottom bars, #5 bars at 6" spacing were chosen. For the horizontal top bars, #6 bars at 6" spacing were chosen. The complete abutment design worksheet can be seen in Appendix 3.3.

Proper drainage was necessary to avoid excess pore water pressure on the retaining walls. Excess pore water pressure could potentially result in flexural and overturning moments. The *AASHTO LRFD Bridge Specification* states in section 11.6.6, "Backfills behind abutments and retaining walls shall be drained or, if drainage cannot be provided, the abutment or wall shall be designed for loads due to earth pressure, plus full hydrostatic pressure due to water in the backfill."

A cantilever design was used for both the North and South abutments in order to maximize the area for traffic flow on Pelham Road. Each abutment would have an elevation of 147.63 feet above sea level underneath the footing and an elevation of 171.63 feet above sea level where the beams sit. The bridge was designed to be level in order to avoid the excess loads of a bridge with a skew.

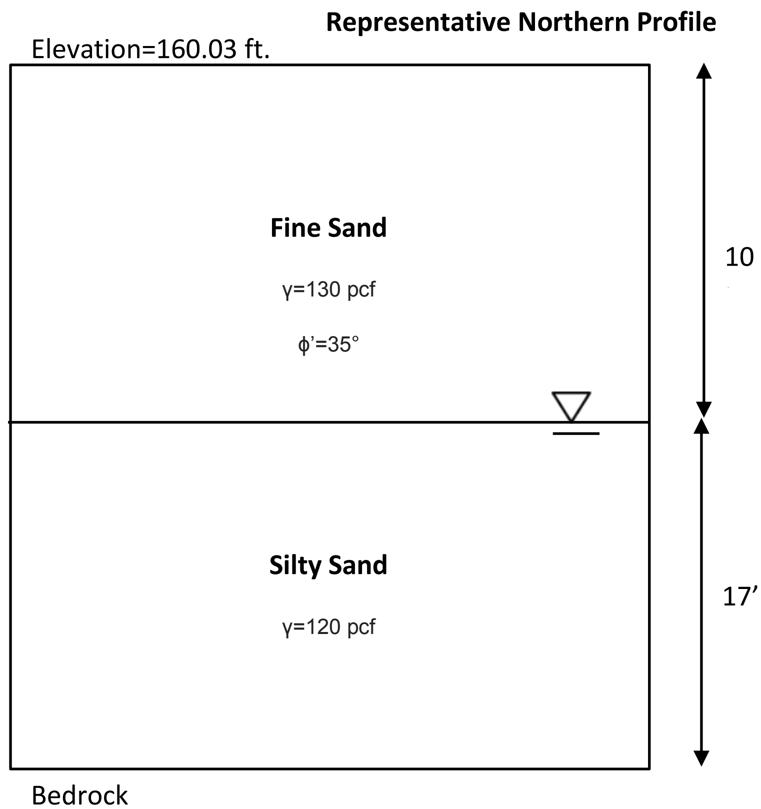


Figure 74: Representative soil profile for Northern abutment site.

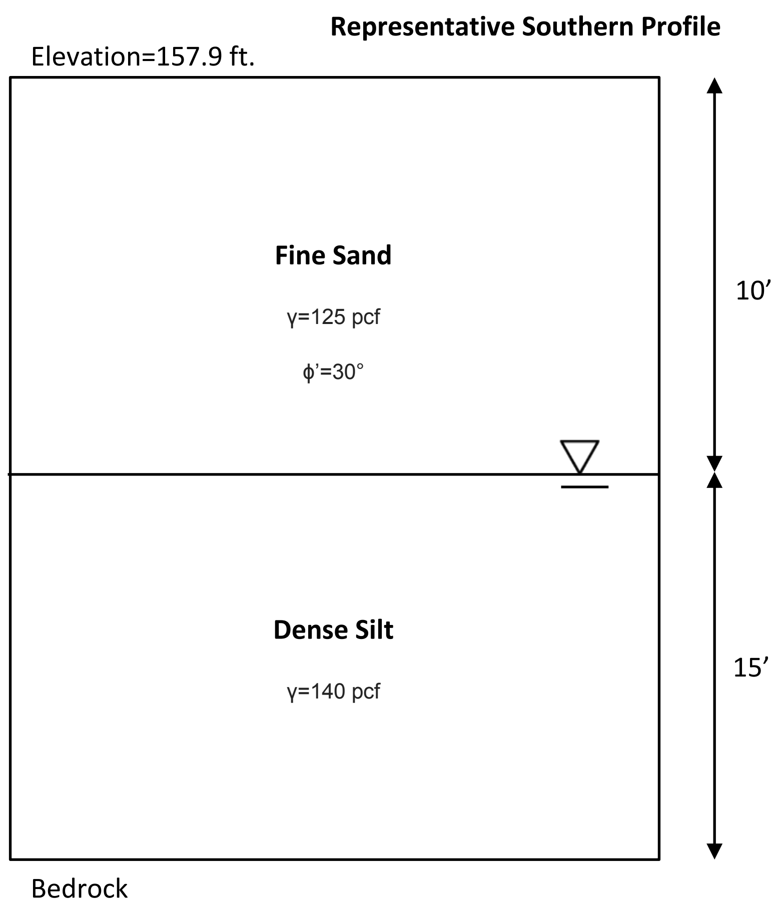


Figure 73: Representative soil profile for Southern abutment site.

5.4 Wingwall Design

The wingwall was designed using reinforcing bars with a yield strength of 60 ksi, concrete with a compressive strength of 3 ksi, 2.5 inches of clear cover for the reinforcement in the backwall and stem, and 3 inches of clear cover in the footing as required by section 5.12.3 of AASHTO. Figure 75 shows a section of the wingwall with dimensions.

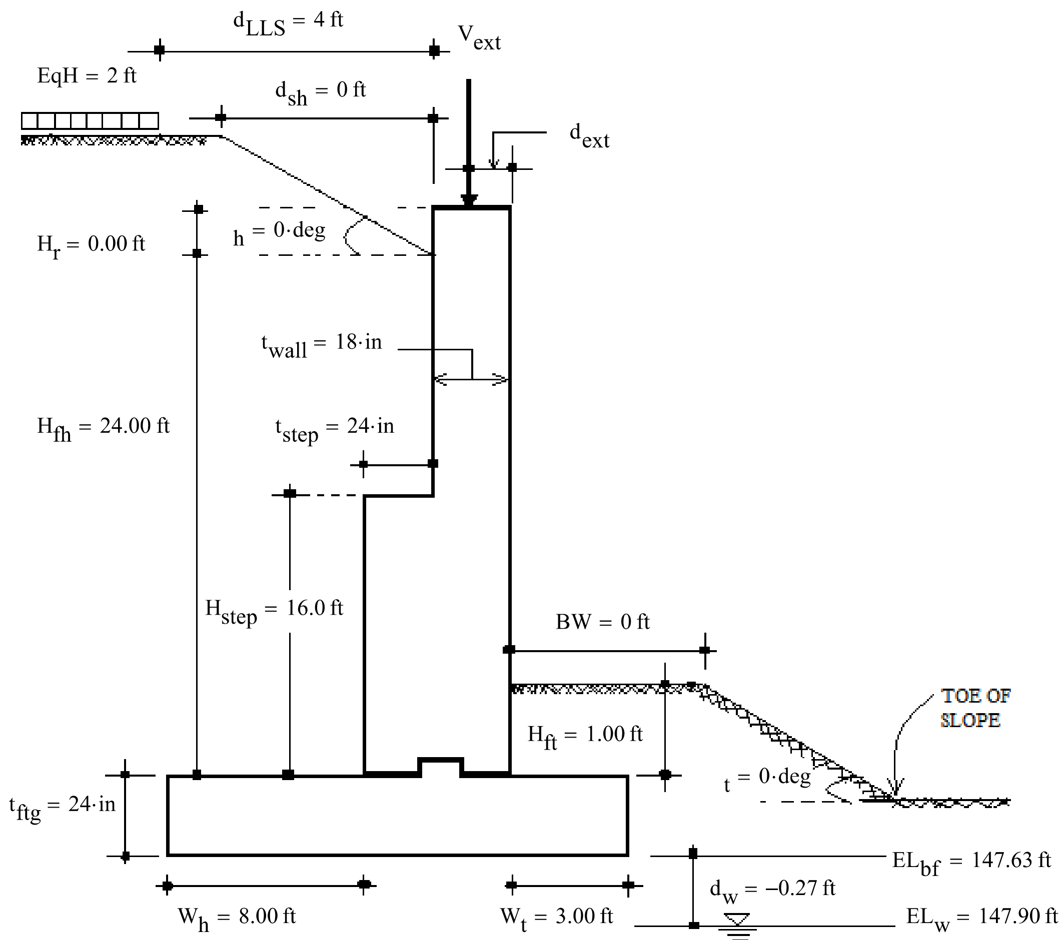


Figure 75: Cross-sectional view of wingwall with dimensions.

The first step in the wingwall design was to determine the proper AASHTO load factors to use for the effects of the soil surcharge on the cantilever wingwall. After these were found, the NYDOT MathCAD program analyzed and combined the force effects based on the *AASHTO LRFD Bridge Specification*.

The final design of the wingwall can be seen in Figure 76. In order to design the multiple components of the wingwall, force effects were calculated for the wall, and footing. The wall was designed for shear and flexure due to the surcharge caused by the adjacent soil. Temperature was also taken into account in the design.

The footing was designed for bearing, shear, flexure, overturning, and sliding while taking into consideration the forces applied to the structure above the footing and the geotechnical information.

The wingwalls were designed as cast-in-place since the rate of their construction would not greatly impact the community and it would also help to avoid issues with the connections. The site of the wingwalls had to be properly excavated, and once the structure has been erected, backfill must be placed behind the wingwalls. The backfill was selected to have a unit weight of 125 pcf and an internal friction angle of 35 degrees which were very common soil properties for the area. The backfill must also be compacted before use.

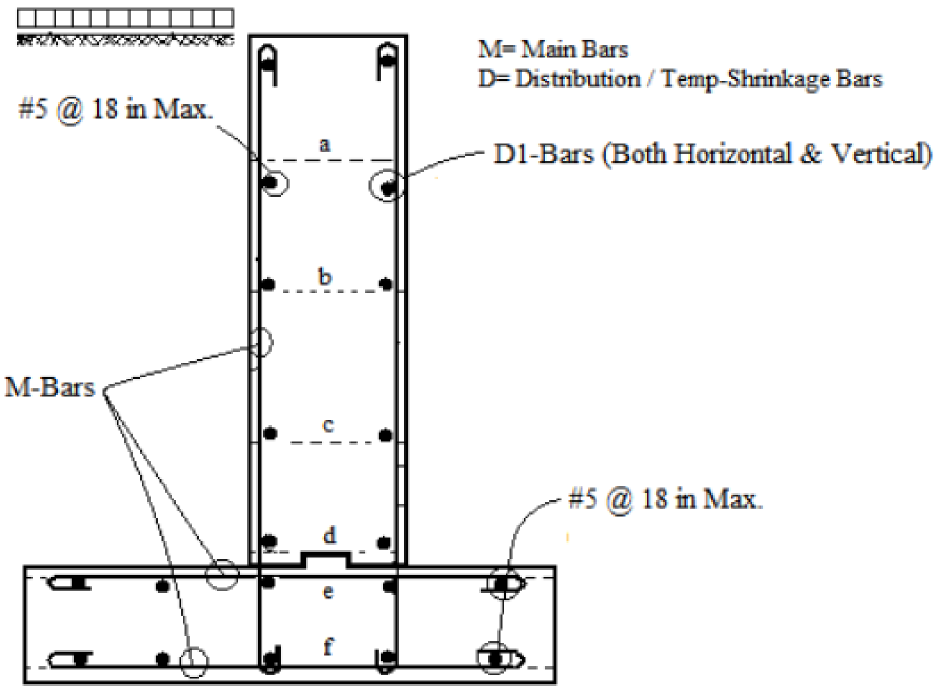


Figure 76: Location of reinforcement in wing wall.

Table 27: Description of wingwall reinforcement.

Bar Designation	Location	Bar Number	Bar Spacing
Vertical Moment Bars in Wingwall	24’-12’ from top of Wingwall (each face)	#7	6”
Vertical Moment Bars in Wingwall	12’-0’ from top of Wingwall (each face)	#5	18”
Horizontal Moment Bars in Wingwall Footing	Bottom Bars	#5	9”
Horizontal Moment Bars in Wingwall Footing	Top Bars	#7	6”

The final wingwall design is outlined in Table 27. The vertical bars in the back wall were chosen as #5 and #7 bars. For the vertical bars, #7 bars at 6” spacing were chosen for the section from 24 feet from the top to 12 feet from the top of the wingwall. For the section from the top of the wingwall to 12 feet from the top, #5 bars at 18” spacing were chosen. Proper splicing was assumed to ensure the continuity of the rebar.

The horizontal reinforcing for the wingwall footing was chosen as follows. For the horizontal bottom bars, #5 bars at 9” spacing were chosen. For the horizontal top bars, #7 bars at 6” spacing were chosen. The complete wingwall design worksheet can be seen in Appendix 3.3.

A single wingwall design was used for both the North and South abutments in order to eliminate the need to design two wingwalls for differences that were negligible. Figures 73 and 74 display the soil profiles used to determine the necessary information and inputs for the wingwall design.

5.5 Final Time and Cost Estimate

The state bridge was priced for comparison using information from the state plans (NHDOT). The plans were used to estimate quantity takeoff for various tasks. The unit prices, task prices and the total price are given in Table 28 below. The pricing data comes from the actual bid results for the state design. Each bidder provided unit price and total price for each line item within the published bid results. It was not always possible to use the quantities provided in the bid, because the project also contains a twin bridge structure, and several miles of highway widening and associated improvement. The state data does not need to be amended for time or construction locations, because the bid numbers are the actual values attached to the state project in real life. The estimate for the state bridge design, as calculated below, is $2,045,000.00.

Table 28: Price Estimate for State Design, using pricing obtained from NH Bid Results

Task	Unit	Quantity	Price Per Unit	2013 NH
Demolish Existing SB Bridge	each	1	$ 137,900.00	$ 137,900.00
Construct North Abutment MSE Wall of New SB Bridge	each	1	$ 307,335.00	$ 307,400.00
Construct South Abutment MSE Wall for New SB Bridge	each	1	$ 307,335.00	$ 307,400.00
Form/Rebar/Pour/Strip North Abutment Footing for New SB Bridge	CY	45	$ 275.00	$ 12,500.00
Form/Rebar/Pour/Strip South Abutment Footing for New SB Bridge	CY	45	$ 275.00	$ 12,500.00
Form/Rebar/Pour/Strip North Stub Abutment for New SB Bridge	CY	42	$ 670.00	$ 28,000.00
Form/Rebar/Pour/Strip South Stub Abutment for New SB Bridge	CY	41.7	$ 670.00	$ 28,000.00
Form/Rebar/Pour/Strip North Abutment Backwall for New SB Bridge	CY	33	$ 1,090.00	$ 36,400.00
Form/Rebar/Pour/Strip South Abutment Backwall for New SB Bridge	CY	33	$ 1,090.00	$ 36,400.00
Set Structural Steel & Set Blocking Grades for New SB Bridge	each	8	$ 90,500.00	$ 724,000.00
Set Precast Deck Panels for New SB Bridge	CY	122	$ 795.00	$ 96,700.00
Form/Rebar/Pour/Strip Deck Overpour, Overhang, and Ends for New SB Bridge	CY	156	$ 862.00	$ 134,700.00
Form/Rebar/Pour/Strip North Abutment Approach Slab for New SB Bridge	CY	69	$ 259.00	$ 18,000.00
Form/Rebar/Pour/Strip South Abutment Approach Slab for New SB Bridge	CY	69	$ 259.00	$ 18,000.00
Install Bridge Rail for New SB Bridge	LF	300	$ 142.00	$ 42,600.00
Install Barrier Membrane On SB Bridge Deck	Sq Ft.	1250	$ 24.75	$ 31,000.00
Install Bridge Joint for SB Bridge	each	1.0	$ 58,548.00	$ 58,600.00
Pave Base & Temporary Course for SB Bridge Deck Sta.	Tons	102	$ 160.00	$ 16,400.00
Prices sourced from actual bid information				
			Total	$ 2,045,000.00

Project Bridge

The project bridge was priced using *RS Means Heavy Construction 2011.* Quantities were takeoffs from design information, or for elements such as approach slabs that were not designed in detail, were based on an estimated volume. *RS Means* provides pricing that includes installed labor and material as well as operations and profit by either unit or assembly cost data. Assembly cost data is pricing information by area or linear footage for a unit, such as a footing or slab-on-grade or typical column. The *RS Means* data was adjusted for two years of cost inflation, and indexed to reflect the lower cost of construction in NH as compared to the index numbers in the general section. Table 29 below contains unit costs, quantity and total cost data. The specific page sources within RS Means and the adjustment factors can be found in Appendix 4.1.

Table 29: Cost estimate for Project Bridge by task.

Task	Unit	Quantity	Price Per Unit	2013 NH
Mobilization	each	1	$ 20,000.00	$ 20,000.00
Clearing & grubbing	acre	2	$ 10,000.00	$ 20,000.00
Sheet Piles	sq ft.	4722	$ 26.50	$ 124,700.00
Excavation	CY	3303	$ 2.02	$ 6,700.00
Abutment wall footings	CY	117	$ 259.00	$ 30,200.00
pier footing	CY	33	$ 395.00	$ 13,200.00
Wing wall footing	CY	94.5	$ 259.00	$ 24,400.00
Abutment wall	CY	313.6	$ 325.00	$ 101,600.00
Wing walls	CY	189	$ 325.00	$ 61,100.00
Pier columns	CY	49.2	$ 835.00	$ 41,000.00
Backfill	CY	1055	$ 22.93	$ 24,200.00
Pier cap	CY	27	$ 1,100.00	$ 29,300.00
DBTs (14)	each	14	$ 18,029.00	$ 251,600.00
Bearings	each	28	$ 1,200.00	$ 33,500.00
Approach slabs	CY	69	$ 259.00	$ 18,000.00
Expansion joints	each	1	$ 50,250.00	$ 50,100.00
Bridge membrane	sq yd.	625	$ 22.50	$ 14,100.00
Bridge pavement	tons	105.5	$ 97.50	$ 10,300.00
Precast Barrier Perm.	lf	150	$ 465.00	$ 69,600.00
Bridge Approach Rail	lf	40	$ 145.00	$ 5,800.00
Temporary Barriers	lf	190	$ 39.50	$ 7,500.00
Removal of Existing Bridge	each	1	$ 150,000.00	$ 149,500.00
Sheet Piles	sq ft.	4722	$ 26.50	$ 124,700.00
Excavation	CY	3303	$ 2.02	$ 6,700.00
Abutment wall footings	CY	117	$ 259.00	$ 30,200.00
pier footing	CY	22	$ 395.00	$ 8,800.00
Wing wall footing	CY	94.5	$ 259.00	$ 24,400.00
Abutment wall	CY	314	$ 325.00	$ 101,600.00
Wing walls	CY	189	$ 325.00	$ 61,100.00
Pier columns	CY	33	$ 835.00	$ 27,300.00
Backfill	CY	1055	$ 39.50	$ 41,600.00
Pier cap	CY	18	$ 1,100.00	$ 19,500.00
DBTs (12)	each	12	$ 18,029.00	$ 215,600.00
Bearings	each	24	$ 1,200.00	$ 28,700.00
Approach slabs	CY	69	$ 259.00	$ 18,000.00
Expansion joints	each	1	$ 50,250.00	$ 50,100.00
Bridge membrane	sq yrd.	625	$ 22.50	$ 14,100.00
Bridge pavement	tons	105.5	$ 97.50	$ 10,300.00
Precast Barrier Perm.	lf	150	$ 465.00	$ 69,600.00
Bridge Approach Rail	lf	40	$ 145.00	$ 5,800.00
Temporary Barriers Removal	lf	190	$ 23.50	$ 4,500.00
Concrete Sealant	sq ft.	9648	$ 0.06	$ 600.00
Loam & Seeding	acre	1072	28	$ 30,000.00
Information sourced from RS Means and actual bid information.				
			Total	**$ 1,997,000.00**

The tasks within the project bridge were scheduled to reduce work time as much as reasonably possible. The tasks had be grouped into steps that contained everything that would be constructed in roughly the same time period. Once these “steps” were compiled their scheduling length was determined by selecting that task that would govern by having the longest estimated time. This governing time was combined with any additional time constraints such as CIP cure time. Times were estimated using project expirience, and scheduling information from both the Laconia totally precast bridge and the actual state project. RS Means crew outputs were also used to estimate some tasks. Calculations are available in Appendix 4.2. Table 30 provides lenghts for construction steps for the project bridge. Table 31 provides scheduling information for the project bridge. This project runs 90 work days with 130 total days elasped, using total weekday work time efficiency.

Table 30: Estimated durations for tasks for Project Bridge.

Task Estimated Durations for Project Bridge	
Task	Working Days
Mobilization, clearing and grubbing of first half of abutment locations.	2
Installation of temporary steel sheet piles, Excavation of western half of abutment and pier locations.	6
First half of construction of Precast footings, CIP abutment retaining walls, and central pier, and precast abutment footings and bridge seats.	20
Placement of western 7 of DBT girder lines and accompanying apparatus	1
Placement of approach slabs. Construction of western half of bridge joints, bridge membrane, bridge pavement and western half of highway structural box, adjustment of ramp paths. Placement of western face precast bridge barrier.	10
Final paving of first half of roadway box, line painting, placement of temporary barriers and traffic diversion onto new span	7
Closure of existing bridge, removal of asphalt pavement, precutting of concrete deck, beam removal and transport off site.	1
Removal of aboveground pier cap and columns, existing bridge abutments, and slope paving	2
Removal to below new footings existing pier and abutment footings.	2
Installation of sheet piles and structural piles, excavation of remaining half of bridge abutments and pier footing	6
Second half of construction of CIP footings, abutment retaining walls, and central pier, and precast abutment footings and bridge seats.	20
Placement of remaining 6 DBTs and accompanying apparatus	1
Placement of approach slabs. Construction of remaining half of bridge joints, bridge membrane, bridge pavement and remaining half of highway structural box, adjustment of ramp paths. Placement of eastern face precast bridge barrier	10
Final paving of remaining half of roadway box, line painting, removal of temporary barriers and traffic diversion onto new span. Concrete Sealant and final loaming and seeding of slopes	7
TOTAL	95

Table 31: Scheduling Information for Project Bridge.

Scheduling Information Project Bridge	
Project Early Start	3/25/2013
Project Late Finish	8/2/2013
Total Days Elasped	130
Percent of Weekdays Utillized	100%

State Bridge

The NH DOT project schedule contains a section on the construction of the southbound bridge. Since this schedule section details just the construction of the bridge, it is comparable to the project timeline estimate for this MQP. The State tasks are shown with associated durations in Table 32 below. However, using the start and finish dates as well as the Gantt chart provided, it can be seen that there is significant float to many of the tasks and that the actual construction duration runs much longer as is shown in Table 33. This is because the effect of float reduces the work day utilization to below 60%.

Table 32: Estimated durations for tasks for State Bridge

Task Estimated Durations for State Bridge	
Task	Working Days
Install Bridge Joint for SB Bridge	5
Demolish Existing SB Bridge	15
Construct North Abutment MSE Wall of New SB Bridge	25
Construct South Abutment MSE Wall for New SB Bridge	25
Form/Rebar/Pour/Strip North Abutment Footing for New SB Bridge	15
Form/Rebar/Pour/Strip South Abutment Footing for New SB Bridge	15
Form/Rebar/Pour/Strip North Stub Abutment for New SB Bridge	20
Form/Rebar/Pour/Strip South Stub Abutment for New SB Bridge	20
Form/Rebar/Pour/Strip North Abutment Backwall for New SB Bridge	15
Form/Rebar/Pour/Strip South Abutment Backwall for New SB Bridge	15
Set Structural Steel & Set Blocking Grades for New SB Bridge	10
Set Precast Deck Panels for New SB Bridge	10
Form/Rebar/Pour/Strip Deck Overpour, Overhang, and Ends for New SB Bridge	20
Form/Rebar/Pour/Strip North Abutment Approach Slab for New SB Bridge	5
Form/Rebar/Pour/Strip South Abutment Approach Slab for New SB Bridge	5
Install Bridge Rail for New SB Bridge	5
Install Barrier Membrane On SB Bridge Deck	3
Pave Base & Temporary Course for SB Bridge Deck Sta.	1
TOTAL	229

Table 33: Scheduling information for the State Bridge.

Scheduling Information State Bridge	
State Early Start	11/25/2013
State Late Finish	6/16/2015
Total Days Elasped	568
Percent of Weekdays Utillized	56%

5.6 Construction Steps, Tasks, and Planned Traffic Routing

The construction of the bridge is broken into three phases. The work is divided in such a way that any phase can be completed any amount of time after the preceding phase. Each phase is divided into steps, which are collections of tasks that take place at the same time. Every phase is described below, with an accompanying diagram showing the locations and scope of work, in addition to the planned routing of traffic on both roads.

Construction begins with Phase 1, the construction of the first half of the replacement bridge, and it consists of six steps. Step One initiates construction, and is shown in Figure 77. This step involves all aspects of mobilization, temporary jersey barriers along Rt.97 and the placement of any other necessary construction signage. Primarily though, this step involves clearing of the locations for both future abutments of the replacement bridge structure, as well as the site of the bridge pier footing. Clearing, utility moving and general site preparation is limited to the area directly around the future bridge. In summary, the first step is the site work necessary to start construction.

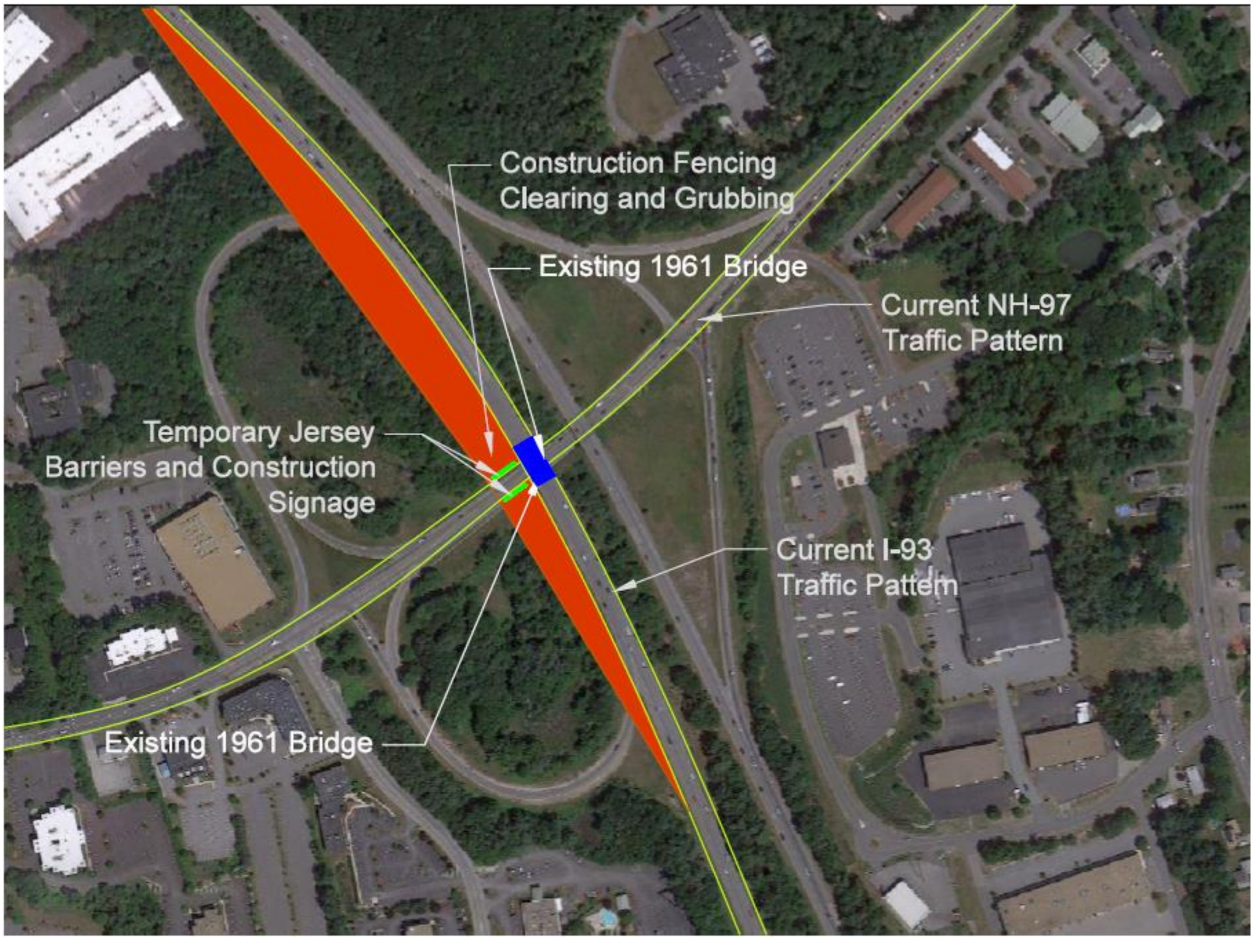


Figure 77 Phase 1 Step 1, Site Mobilization and Site work

Step Two as seen in Figure 78, is the installation of temporary sheet steel piles, and the excavation of the western half of the pier footing and both abutment boxes. Sheet piles are necessary to prevent dangerous soil collapses into the excavation and also to prevent the loss of structural support to the existing I-93 traffic corridor. Sheet piles will also be used along Rt. 97 adjacent to the pier footing excavation for the same reason. Once properly designed sheet piles are installed, excavation to final depths for the pier footing, and abutment retaining wall toe footings would proceed. Once all excavation is complete, the structural steel piles will be driven at each abutment.

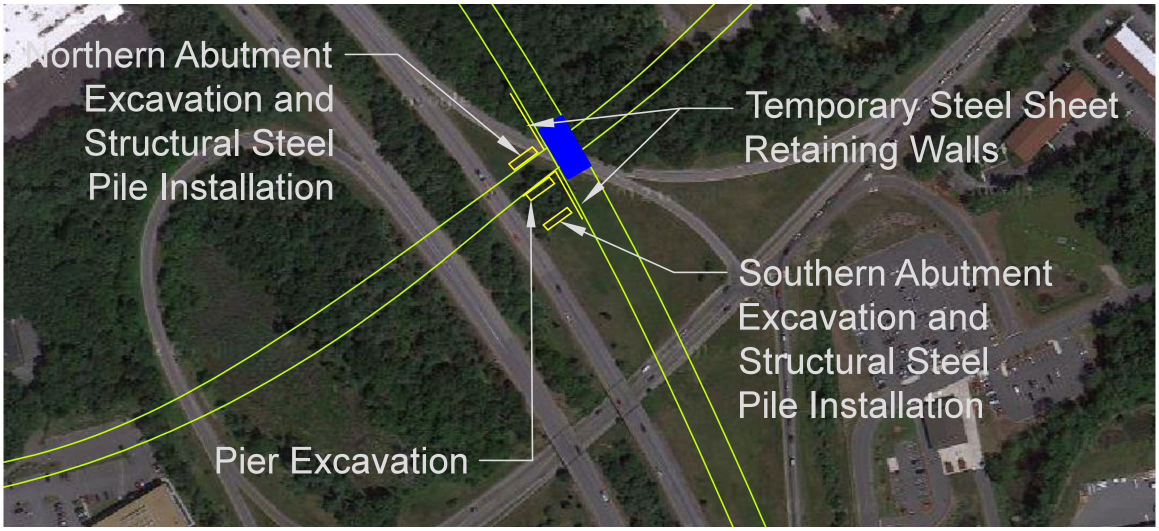


Figure 78: Phase 1 Step 2, Excavation

Step Three is the construction in place of the toe footings for the abutment wing and breast walls, and the construction of the spread footing for the pier. Once the footings have reached approved curing strengths, step three will continue with the construction of the pier columns and the abutment retaining walls. Once the forms have been removed and the curing strengths are acceptable, the abutments will be backfilled in small lifts of select granular fill, and the area pier footing will be backfilled to final surface elevation. The sheet piles near the pier will be removed once it is backfilled. Finally, the spread footings at each abutment and the pier cap will be either constructed in place or positioned in the form of grouted precast elements. Figure 79 shows the installation of footings, and Figure 80 shows the installation of the remaining substructure.

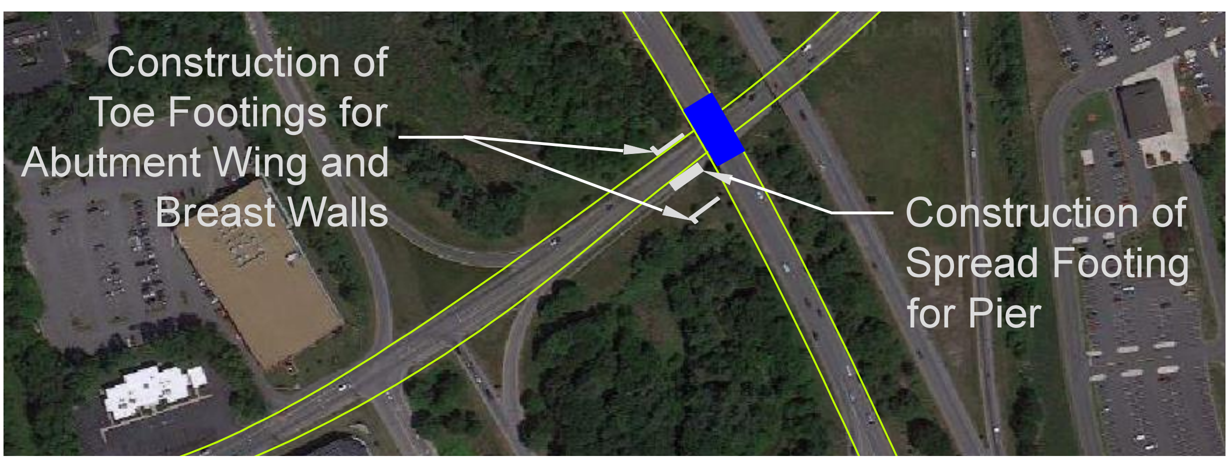


Figure 79: Phase 1 Step 3, Installation of Footings

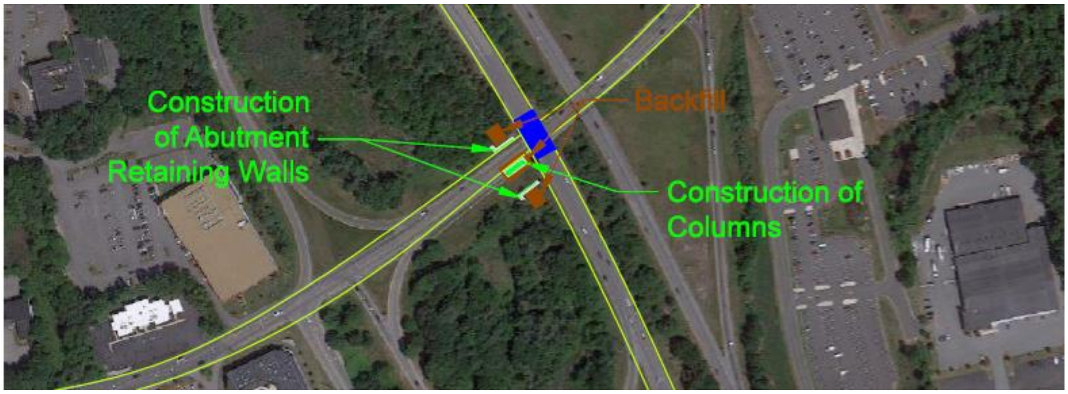


Figure 80: Phase1 Step 3 Installation of Remaining Substructure

Step Four as seen in Figure 81, is the placement of the western half of the precast prestressed deck bulb tees. The bulb tees will be placed by a single large crane, by girder line. The western half will be constructed starting with girder line seven span one, and then span two, then girder line 6 span one and so on. Installation of the accompanying bracing, bearings, and diaphragms will be coordinated with the erection of the precast beams.

Figure 81: Phase 1 Step 4 Placement of Prestressed Deck Bulb Tees

Step Five as shown in Figure 82, is the construction of the roadway expansion joints at the abutments and the joint sealant at the central pier. The precast concrete barriers on the western side of the bridge are to be placed and attached during the same time period. Temporary Jersey barrier type concrete sections are also to be placed along girder line 7 to prevent vehicles from exiting the roadway until the other half of the bridge is constructed. Once these tasks are completed, a heat-applied waterproofing membrane will be applied to the entire top surface of the deck bulb tees. This will then immediately be paved over with 3” of bridge mix asphalt pavement.

The construction of the approaches to the new bridge will begin in Step Five. Clearing of organic material from the new roadway path, excavation of unacceptable soils, and construction of the new roadway will all occur at this time. Roadway construction will consist of, as is typical, the lifted compactions of gradually better sub bases, with precision grading on the top most layer just before the beginning of step 6.

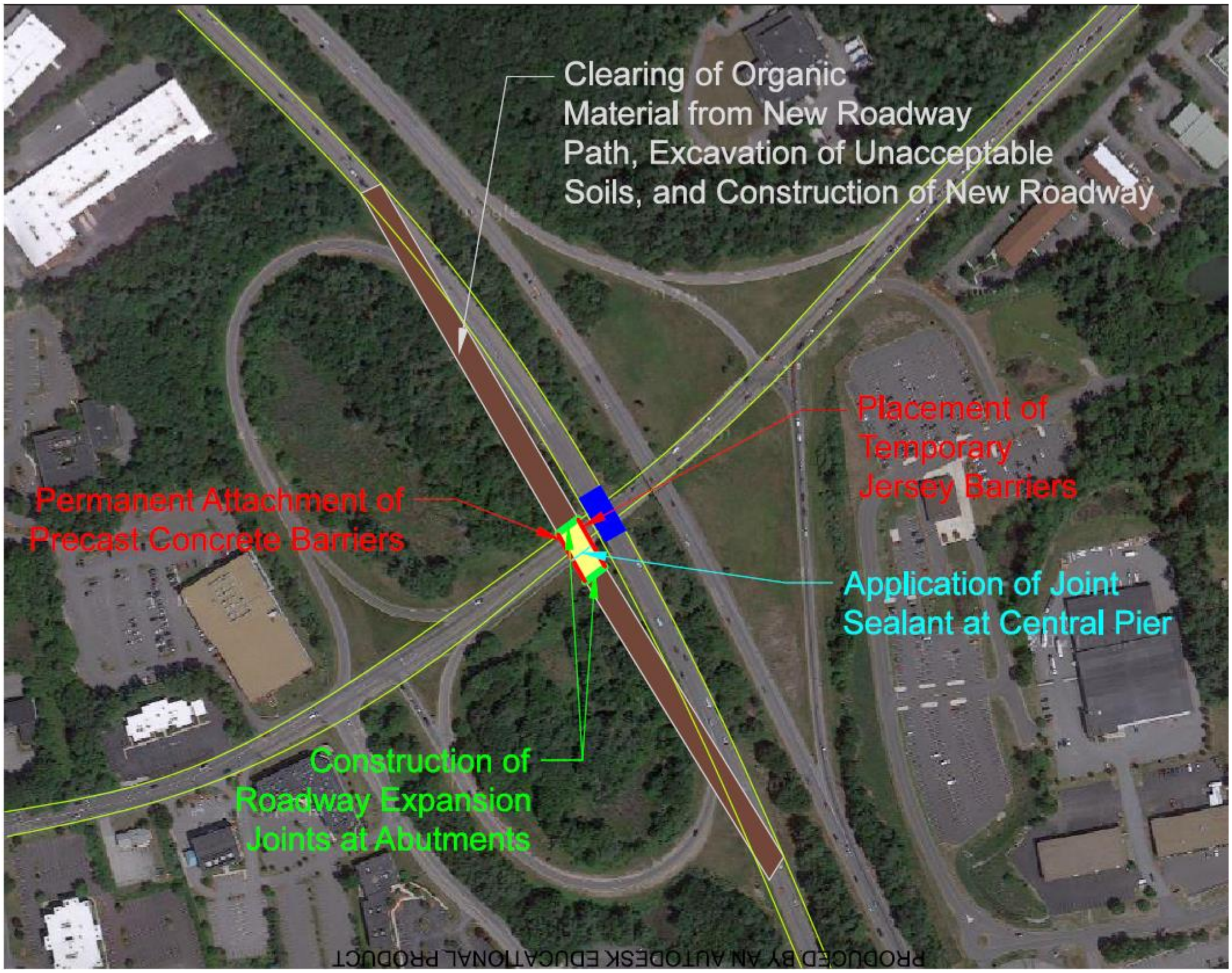


Figure 82: Phase 1 Step 5, various construction activities after beam placement.

Step Six as shown in Figure 83, is the final paving of the western half of the lanes in the replacement bridge area. This pavement will consist of several layers of base course topped by two layers of a high grade wearing course, sloped as appropriate for drainage. Installation of new signage, guard rails and slope stabilization will also occur during this time frame.

Once the majority of the top course paving is completed for the new section of highway, the final connection to the original I-93 corridor will occur. Directly before the transfer, the section connecting the roadways will be paved at night, and the lines painted as soon as possible after that. The section is the final hundred feet or so, where the new roadway intersects the old path of traffic. Once the connections are fully constructed, law enforcement units will lead traffic across the new roadway and bridge span. Temporary jersey barriers will be placed to block the old traffic pattern, and protect against confused or lost drivers in the following weeks.

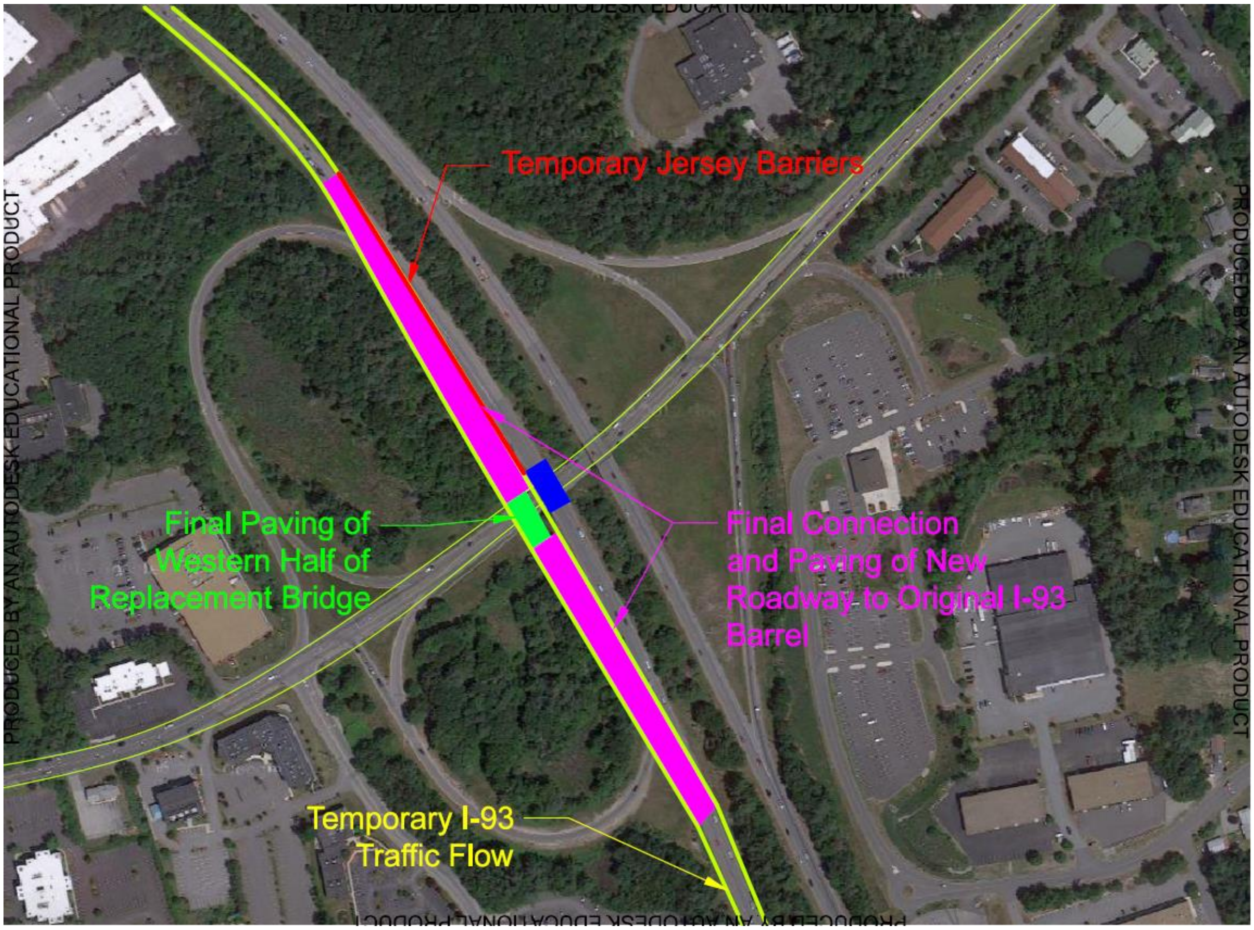


Figure 83: Phase 1 Step 6, joining of new bridge section with existing mainline highway

Phase Two consists of the demolition of the existing, and now unused bridge structure. Step 1 is displayed in Figure 84 and starts with the removal of the asphalt pavement from the surface of the existing bridge, while leaving in place the rest of the existing highway, which will help prevent sedimentary runoff from creating such a large area of exposed earth. Once the pavement is removed, the bridge deck will be precut into sections using masonry saws prior to night demolition.

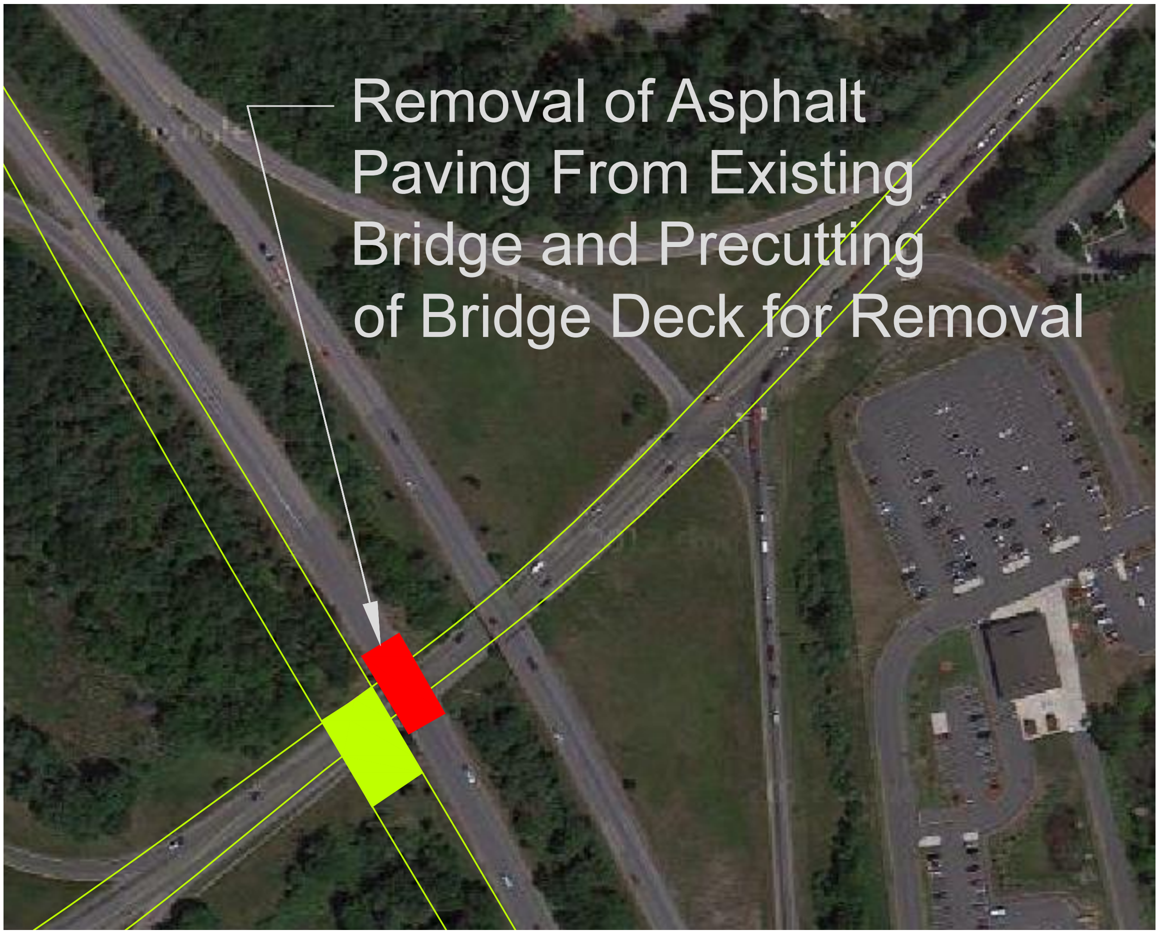


Figure 84: Phase Two Step One, removal of pavement.

Step 2 (demolition) consists of a single night's closure of NH 97. A protective layer of earth will be dumped over the roadway surface on NH 97 to prevent impact damage. The cut deck sections will be released from the steel beams using hydraulic rams, then collected into trucks and removed. The beams will be lifted down from their bearing, and placed on trucks as intact pieces for reuse on other projects/scrap metal. Any remaining debris and the protective soil layer will then be removed to allow the reopening of NH 97 next morning.

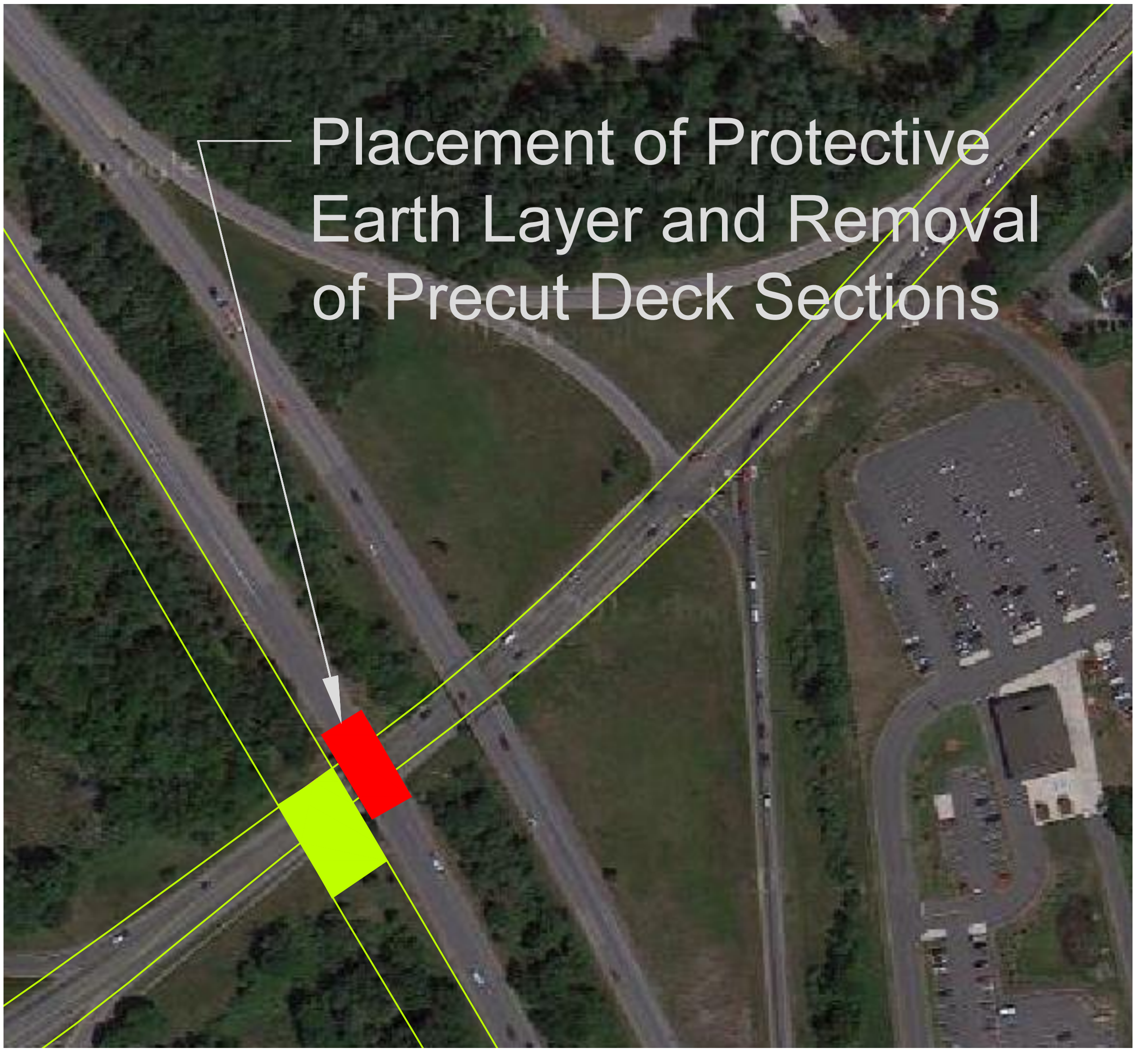


Figure 85: Phase 2 Step 2, Removal of deck and beams

Step 3 consists of the removal of the existing piers, abutments, columns and footings from the original bridge. Due to proximity to businesses and the active highway, demolition will consist of either cut and removal or hydraulic rams. The substructure will only be removed to a sufficient depth to allow the construction of the eventual NH 97 and the remainder of the new bridge pier and abutment footings.

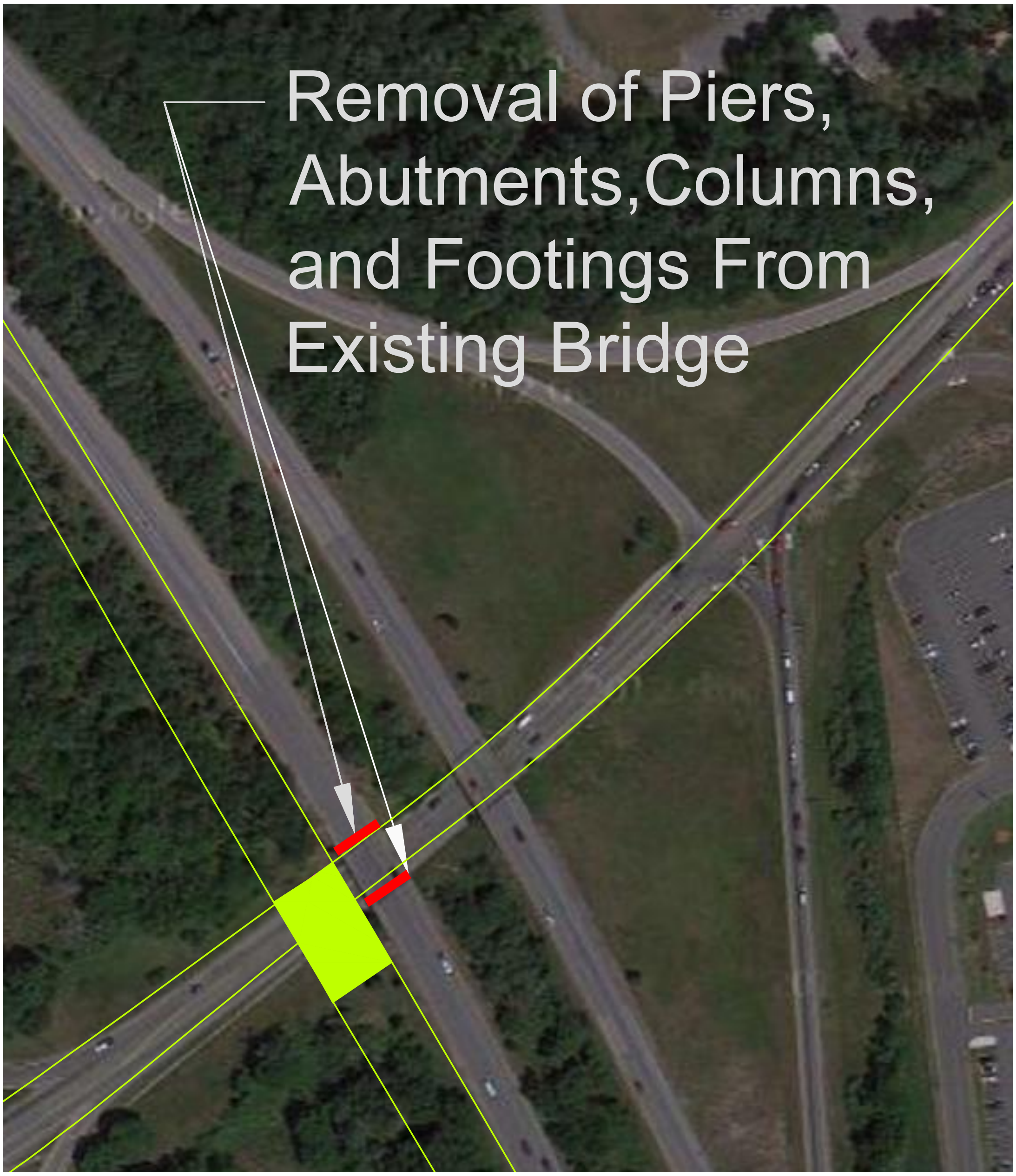


Figure 86: Phase 2 Step 3, Removal of Existing Substructure

Phase Three is the construction of the remainder of the replacement bridge. Step 1 will consist of the excavation of the locations for the rest of the bridge abutments, along with the previously Installed sheet piles. It is will also consist of the installation of new temporary sheet piles along NH 97 to allow excavation to be conducted safely. Once the sheet piles are in place, the excavation for the remainder of the pier footing will occur. The remainder of the steel piles will be driven at both abutments.

Figure 87: Phase 3 Step 1, remaining excavation

Step 2 is the construction in place of the remaining toe footings for the abutment wing and breast walls, and the construction of the spread footing for the pier. This will occur directly adjacent to the now occupied western half of the structure. Once the footings have attained adequate strength, construction will continue with the remainder of the pier columns and the abutment retaining walls. Once the forms have been removed and the curing strengths are acceptable, the abutments will be backfilled in small lifts of select granular fill, and the area pier footing will be backfilled to final surface elevation. The sheet piles near the pier will be removed once it is backfilled. Finally, the remaining spread footings at each abutment and the pier cap will be either constructed in place or positioned in the form of grouted precast elements.

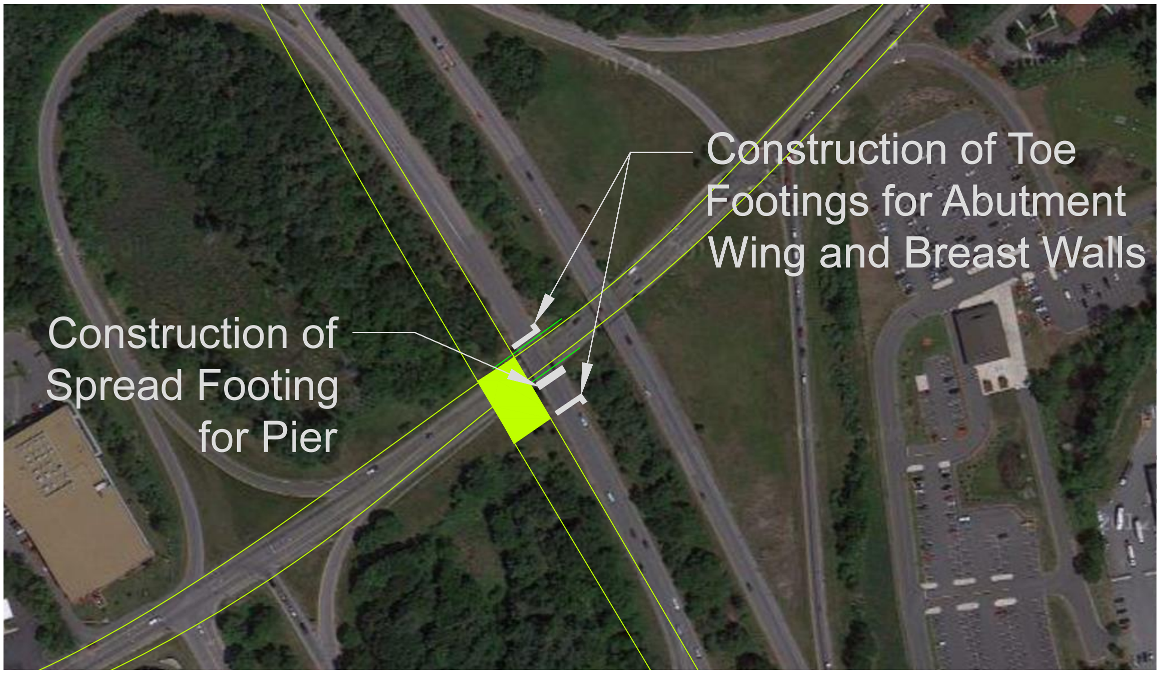


Figure 88: Phase 3 Step 2, construction of remaining footings

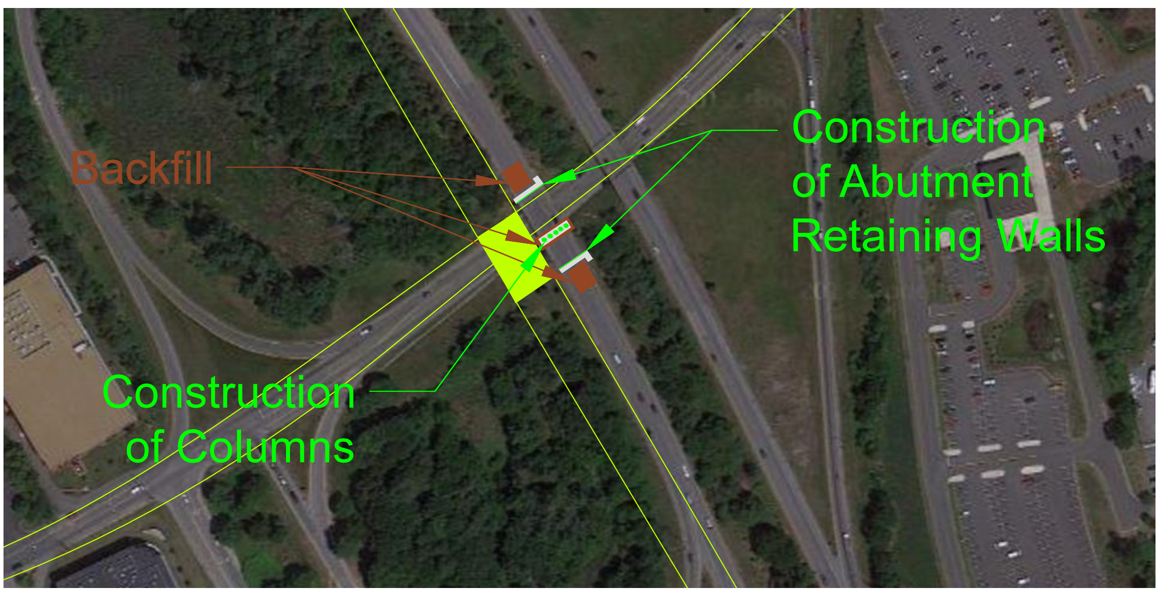


Figure 89: Phase 3 Step 2, construction of remainder of substructure

Step Three is the placement of the eastern half of the precast prestressed deck bulb tees. The bulb tees will be placed by a single large crane, by girder line. The eastern half will be constructed starting with girder line eight span one, then span two, then girder line nine span one and so on. Accompanying bracing, bearings, and diaphragms will be erected at the same times as the beams.

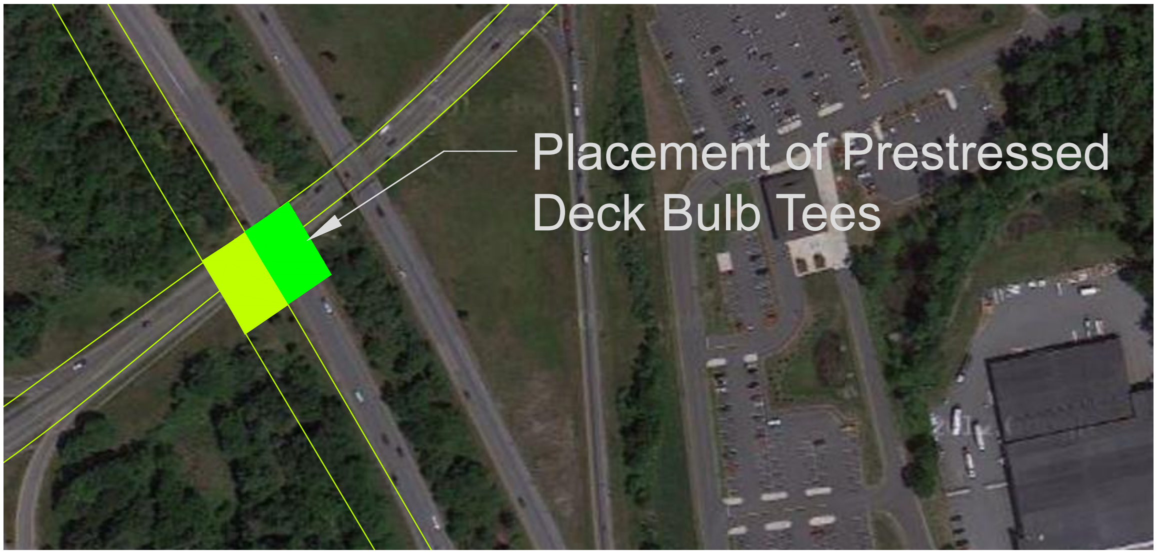


Figure 90: Phase 3 Step 3, Placement of remaining prestressed deck bulb tees.

Step Four is the construction of the roadway expansion joints at the abutments and the joint sealant at the central pier. The precast concrete barriers on the eastern side of the bridge are to be placed and attached during the same time period. Once these tasks are completed on the new half, a heat-applied waterproofing membrane will be applied to the entire top surface of the eastern deck bulb tees. This will then immediately be paved over with 3” of bridge mix asphalt pavement.

Clearing of organic material from the rest of the new roadway path, excavation of unacceptable soils, and construction of the rest of the new roadway will all occur at this time. The construction process is the same as described in Phase one, part five. Two full lanes and a breakdown lane are to be constructed, although only a single lane and breakdown lane will be paved immediately.

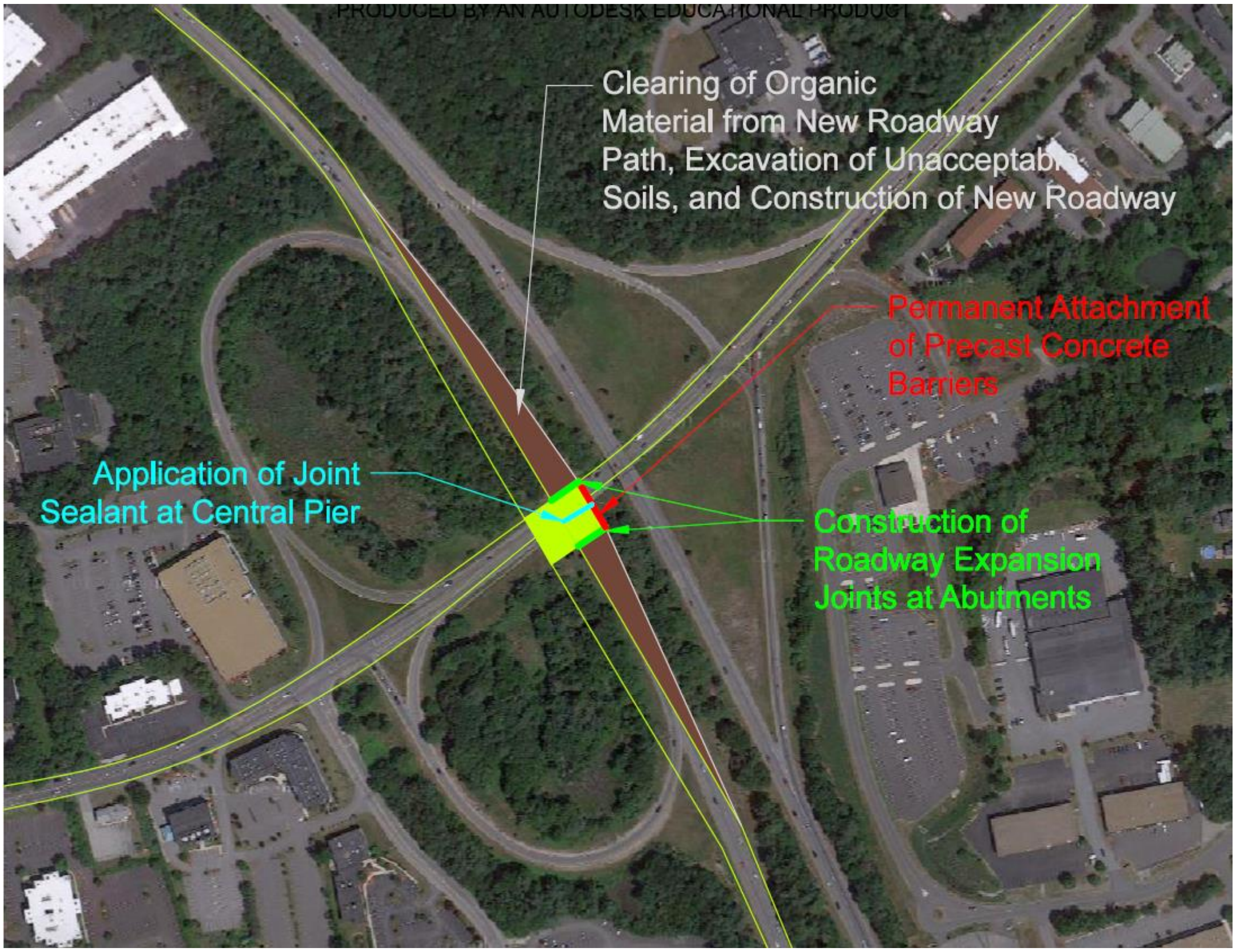
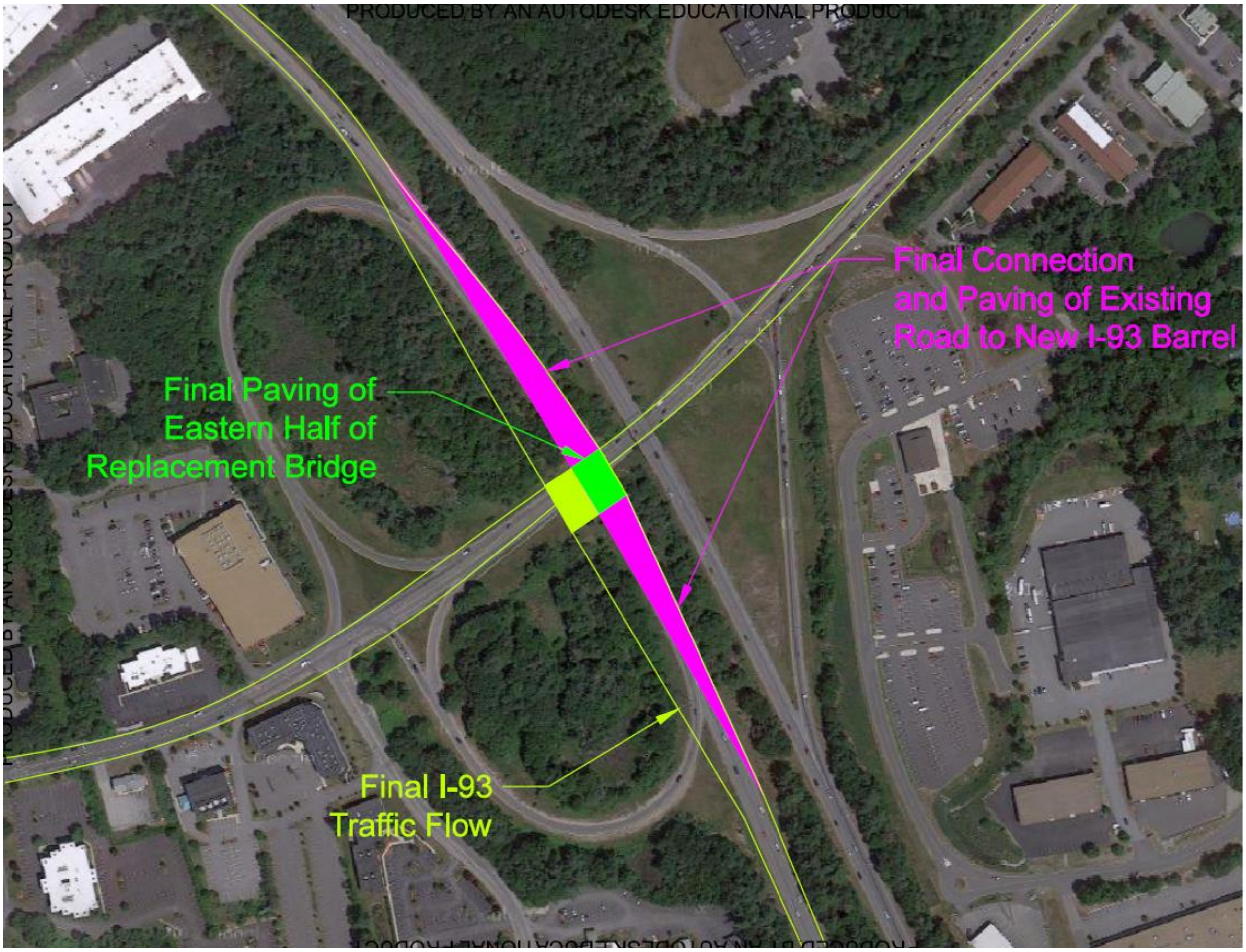


Figure 91: Phase 3 Step 4, various construction activities after beam placement.

Step Five is the final paving of the western half of the lanes in the replacement bridge area. This pavement will consist of several layers of base course topped by two layers of a high grade wearing course, sloped as appropriate for drainage. It will only comprise a single travel lane and breakdown lane. Installation of new signage, guard rails and slope stabilization will also occur during this time frame. This effort is similar to the method as the main line paving during Phase 1. Connection paving between the new paving and the existing highway will again occur at night. Once the highway is connected, new lines will be applied as soon as possible. Once all lines are in permanent order, the final three lane corridor, with a fourth additional lane of bridge and embankment, and two breakdown lanes will be open from the beginning of construction north of the bridge carrying into the ongoing widening south of the project bridge. The state plan is to re-line the entire section highway to the full four lane design capacity as it becomes necessary.



5.7 Work Zone Road User Cost

The Work Zone Road User Cost (WZ RUC) for the construction of the southbound bridge on I-93 going over Pelham Road was determined for the decreased speed limit along I-93, and the redirection of traffic along Pelham Road. The route of the detour was determined by going on Mapquest.com and selecting the shortest alternative connection that bypassed the construction site on I-93 while the beams were to be placed. This turned out to be rerouting the traffic from Pelham Road to Stiles Road, Lowell Street, S Policy Street, and back to Pelham Road as seen in Figure 93. This had a total distance of 2.55 miles with an estimated duration of 6 minutes. This differs from the original route along Pelham Road as seen in Figure 92, by adding 1.87 miles and increasing the duration by 5 minutes.

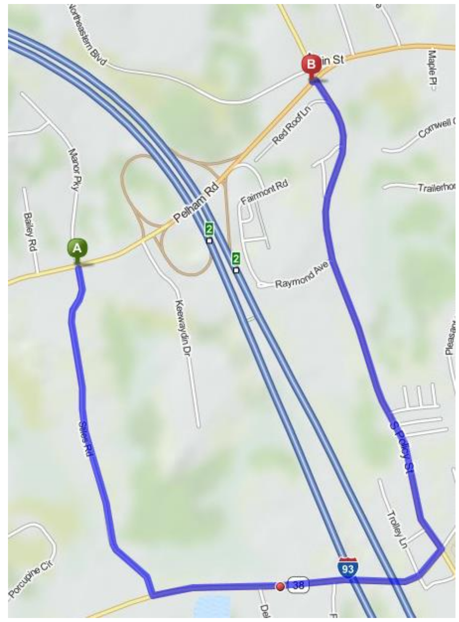


Figure 92: Pelham Road without detour.

Figure 93: Route of detour from Pelham Road.

This detour was only required while the beams were in the process of being erected which had an estimated duration of 24 hours (R.S. Means 2011). This detour was determined from the time estimates for the Project Bridge and the State CPM for the State Bridge. Saturdays and Sundays were shown to have the least amount of traffic on Pelham Road out of any other days of the week as shown by the traffic data (NHDOT). Two eight hour shifts were necessary to erect the beams were selected based on the hourly breakdown of the traffic in order to minimize the effect on the commuters. The trends shown in Figure 94 show the best times to reroute the traffic is between the hours of 11:00 PM and 7:00 AM. The calculated WZ RUC for rerouting the traffic for a weekend is estimated to have a value of $7,446.43 for both the project and the State plans.

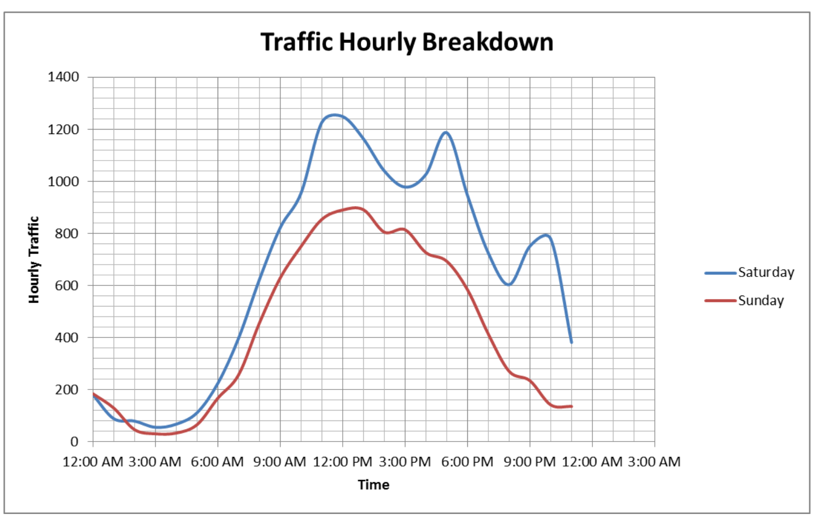


Figure 94: Hourly breakdown of commuters along Pelham Road on weekends (NHDOT).

The State plans used a temporary bridge to divert the traffic on I-93 and reduced the speed limit to 55 from 65 mph so the State project also had to take the WZ RUC for changes in the speed limit into account. The duration of change in the speed limit lasted for 568 days as determined from the CPM Schedule obtained from the NHDOT as seen in Appendix 4.2. The design for the bridge done in this project also reduced the speed limit to 55 mph for a duration of 130 days as determined in the construction plan. The traffic data was obtained from the same source as for the calculation of the WZ RUC for the detour; however the data from I-93 was used instead of from Pelham Road. The WZ RUC for the change in the speed limit was calculated as described in the methodology and determined to be $1,680,069.88 for the State plans and $384,523.04 for the project. With this information, the WZ RUC for the project was determined to be $391,969.47, while the WZ RUC for the State project was determined to be $1,687,516.31 as shown in Table 34.

Table 34: Work Zone Road User Cost Results

Work Zone Road User Cost		
	Project	State Project
Detours:	$ 7,450.00	$ 7,450.00
Speed Limit Change:	$ 384,500.00	$1,680,100.00
Total:	**$ 391,950.00**	**$1,687,550.00**

6 Comparison and Conclusion

A two span, prestressed deck bulb tee super structure, with a central pier was the design proposed. Specifically, there were thirteen girder lines, a roller bearing cantilever abutment at both ends, and central pier consisting of a pier cap, and five supporting columns which are supported by five individual spread footings. This proposal is different from the actual solution prepared by the State of New Hampshire. Every bridge designer selects his materials, geometry, and style based on a number of project factors, past experience and specific goals. It is assumed that the goal of the State design is to create a design that could be economically constructed along the entire I-93 widening corridor. This project weighed much more heavily on the specific factors that would allow quick and economical construction on just the Interstate 93 overpasses over Pelham road, specifically the SB bridge.

6.1 Introduction to the State Design

The State of New Hampshire bids entire sections of highway improvement in large packages typically totaling millions of dollars. New Hampshire contract 13933E was bid during the summer of 2012 and consists of the replacement of two highway bridges, the improvement and widening of the surrounding highway, in addition to the cost of water management and control for such a large project. The front page of the over eight hundred pages of plans is shown below in Figure 95.

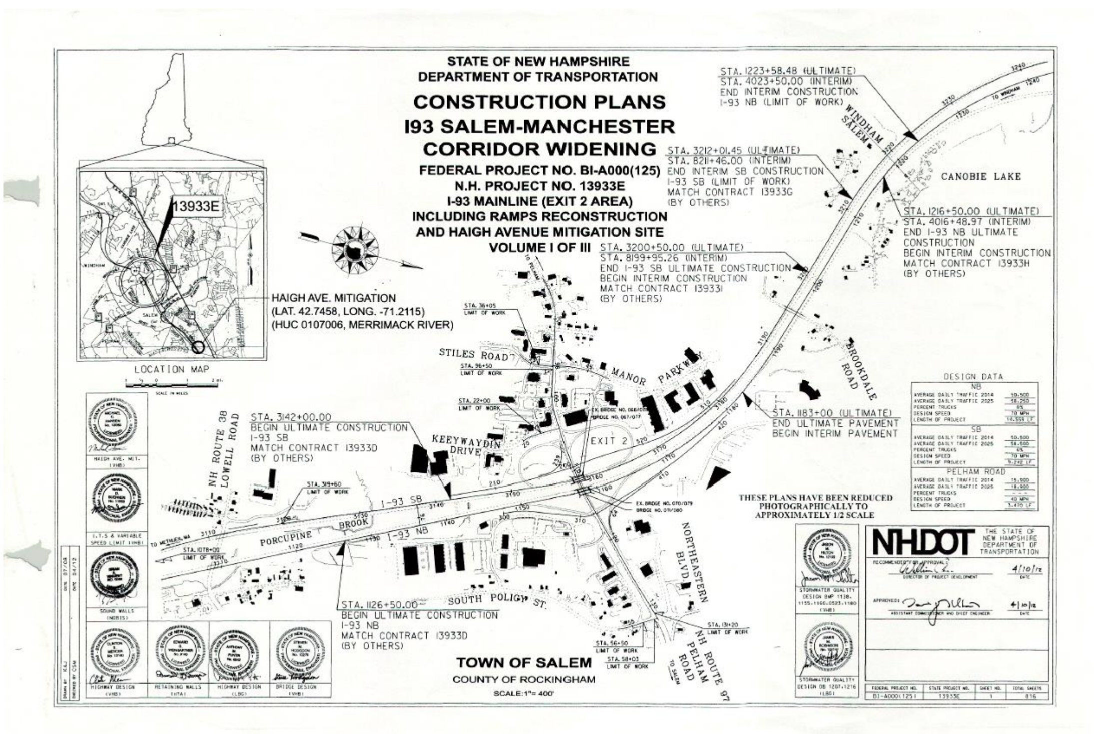


Figure 95: Front Page of 13933E Construction Plans

Therefore, the cost and the expense of replacing the highway bridges is merely a portion of a much larger project. The estimated cost of both the southbound and northbound bridges is around $12 million with a total project that is over $40 million. A construction profile of the State design for the southbound bridge is shown in Figure 96

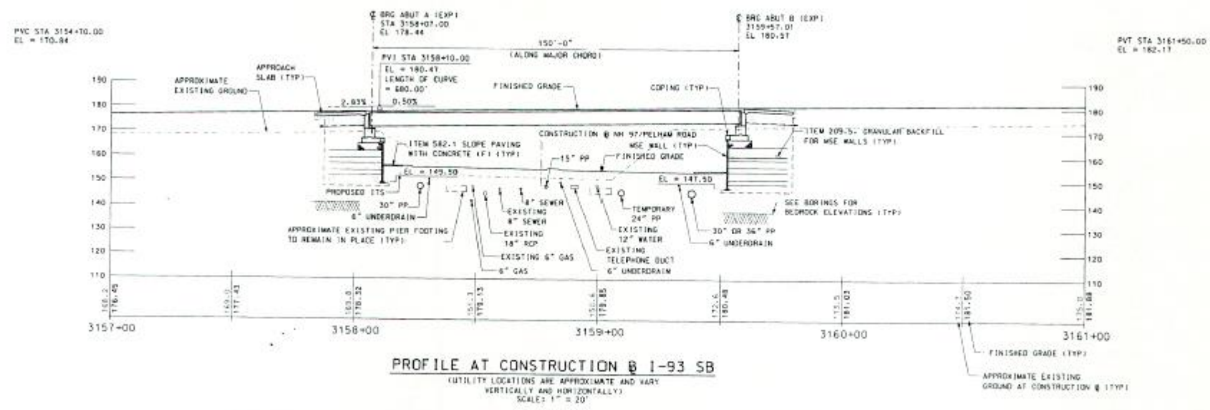


Figure 96: Profile of New Bridge according to state plans. Also shows existing ground surface.

The New Hampshire bridge design features a steel structure with composite deck. The beams are composed of weathering steel, and typically consist of a 65 inch deep web, 5/8th of an inch thick. The top flanges are 1 inch thick and 18 inches wide, and the bottom flanges are 1-1/4 inches this and 20 inches wide. The superstructure consists of a single 150 foot simple span, with parapet abutments at each end.

The parapet abutments in the State plan are similar to the one shown below in Figure 97. These abutments are constructed by placing select granular material in small lifts while constructing the wall panels and straps simultaneously as the lifts ascend. The straps are layered throughout the granular soil and serve to secure the wall panels they are attached to. This type of wall is known as a mechanically stabilized earth construction. For the State's abutments, once the wall reaches the appropriated height, a small spread footing, beam seats, and a back wall will be constructed on top of it. This cast-in-place concrete and reinforced earth combination serves as the entire structural abutments.

Figure 97: Parapet Abutment under construction. Salem NH, picture taken by NHDOT.

Each beam rests upon an elastomeric bearing assembly at each end. The beams have 452 shear studs each, separated into two rows at 8" spacing along the length. The structural concrete deck does not have metal decking and is 8 inches thick, except at beam haunches, where it can be thicker. There are eight total beams, each spliced once at 30 feet from the northern abutment.

6.2 Comparison & Analysis

The State bridge and the proposed solution in this project are clearly different takes solutions to same problem. This is most likely because the design were created with different primary goals. Specific information on the bridge designs themselves is available in Table 35. As said above, this project sought to create a bridge that promoted fast and easy onsite construction. The quest for speed leads to smaller members, which meant smaller beam spacing, and in the end, more spans.

Table 35: Comparison of State and Project Bridge Designs

	Project Bridge	State Bridge
Superstructure Material	Concrete	Steel
Superstructure Type	Deck Bulb Tee	Plate Girder
Abutment Types	Cantilever	Parapet
Depth of Beams	35"	67.25"
Number of Spans	2	1
Individual Span Width	75'	75'
Individual Span Length	75'	150'
Total Bridge Length	150'	150'
Number of beams per span	13	8
Number of Individual Beams	26	8
Pier Type	Pile Bent	Hammerhead
Total Concrete Used in Superstructure	430 yd^3	280 yd^3
Total Steel Used in Superstructure	76.4 ft^3 = 37,437 lb	696.9 ft^3 = 341,469 lb
Total Concrete Used in Substructure	1608 yd^3	400 yd^3

Another major design decision was the desire to have some form of precast deck for the project bridge. This was to avoid the costly process of creating a cast-in-place deck suspended on the bridge beams on location in Pelham. The curing time alone for a cast-in-place deck was prohibitive enough that traditional deck designs were eliminated early into the design of the project bridge. Every decision was based on the goal to have a bridge that could be open as soon as possible, with the major constraint of being relatively economic.

This project looked at bridges for cost estimate under a two part price concept. The first part, or the A component, was the traditional cost of building the bridge. This number included labor, materials, equipment, fees and any other costs that would be directly paid by the state to construct a bridge. Table 28 shows the final material and labor cost estimates for the project bridge design. Table 30 shows the same style of estimate for the State bridge design.

The two bridges both have very close cost estimates. This is a typical result, and is part of the reason steel and pre-stressed concrete continue to split the structural bridge girder market in the United States. The final cost estimate for the project bridge is roughly $2.00 million and the state bridge estimate totals to a near identical $2.04 million. The state bridge saves money on foundations and abutments, while at the same time the proposed precast design saves on the cost of the actual structural framing members significantly. The difference in cost represents only about 2.5% of the total estimate on either bridge, far within the margin of error for the estimates.

6.3 Comparison and Analysis

The project was compared with the State plans in five different categories and each project was given a grade. The categories consisting of aesthetics, constructability, cost, maintenance, and incentives were each given a grade from 1-10 and weighted based on importance for the overall comparison. The weight assigned to each category is displayed in Table 36.

Table 36: Weights of different categories.

Aesthetics	Constructability	Cost	Maintenance	Incentives
7.5%	20.0%	40.0%	12.5%	20.0%

6.3.1 Aesthetics

The aesthetic rating was done by using the qualitative information and assigning a numerical score. The completed design was deemed to be more aesthetically pleasing using concrete beams than giant maroon steel girders used in the State project. The pier in the middle of the design made the bridge less impressive than a single span. A hammerhead pier was originally selected for the intermediate pier mainly for aesthetic reasons; however it was determined to be an inefficient use of concrete after taking the geometric details into consideration and a pile bent pier was used which is less visually appealing. The abutments on the design are also very tall in order to make use of as much space as possible under the bridge which looks different than most bridges. Bridges commonly have a slope coming out from abutments which gives the bridge less of a rectangular look. The MSE walls used for the abutments in the State plans are very difficult to clean if they are vandalized. After these considerations, it was determined the design deserves a rating of 6.5 out of 10, and the State plans deserve a rating of 8 out of 10.

6.3.2 Constructability

The proposed precast design both benefitted and suffered from a constructability standpoint by the fact that it has an intermediate pier. The pier made it possible to use shorter beams along the span, which made the beams easier to transport and erect, but at the same time the pier was also an additional component that needed to be constructed. The beams used in the design were excellent from a constructability standpoint because they already had the deck panels included as part of the beam which is one less component necessary to construct. The cantilever abutments designed require much less steps to construct than the MSE walls used in the State design, however MSE walls do not require any specialized equipment. The State plans use fewer beams (8), but they were much larger and required either a very large or multiple cranes to erect and the beams also had to be spliced. For the precast system the beams were much smaller and easier to erect but there were over three times as many (26). After consideration of the crane details described above, the proposed design was rated as 8.5 out of 10 and the State plans were rated as 8 out of 10.

6.3.3 Cost

The grades for the cost were directly determined based on the direct costs of the bridges consisting of materials and labor. The grade was calculated by selecting a reasonable range of values which the cost of the bridge fell within. The selected range was determined to be from $1.5-$2.5 million and the grade was determined by inputting the cost of the project into the function $f(x)=-10x+25$ where the variable “x” indicated the cost of the project. Figure 98 shows a graphical representation of the function used to determine the cost rating. The cost of the proposed MQP bridge was very slightly lower than the price of the state bridge. The final estimated cost of the project bridge was $1,997,000 or 97.6% of the cost of the project estimate for the state design of $2,045,000, not a very significant cost savings. The project bridge saved immensely on the cost of the bridge girders, but this cost savings was almost completely offset by the cost of the central pier system required.

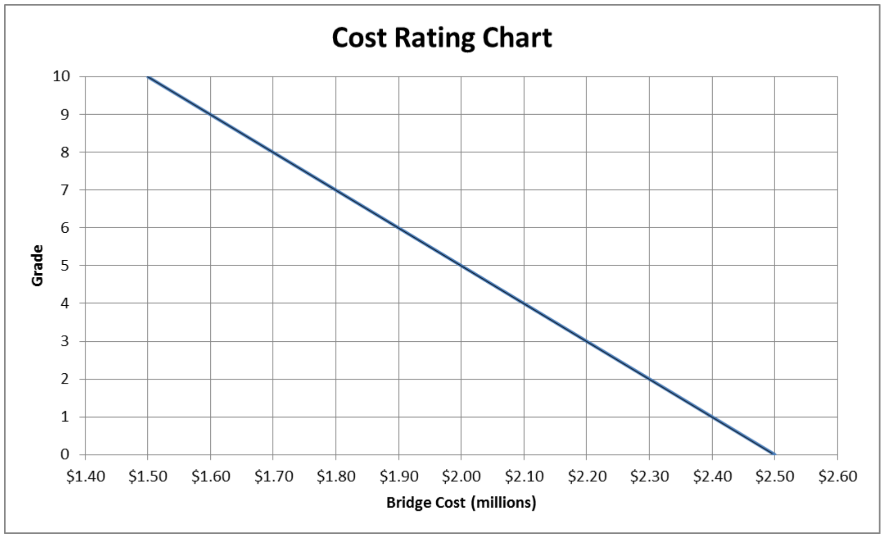


Figure 98: Determination of Cost Grade.

6.3.4 Maintenance

A life cycle cost analysis of steel bridges compared to prestressed bridges (Chen, Huang) gives data that describes galvanized steel bridges as requiring 9.34% more maintenance during the lifespan of the structure. This gives the design a distinct advantage for the material selection. Prestressed concrete may be more effective from a maintenance perspective, but the intermediate pier, larger abutments, and additional bearings add other elements to give the State plans the advantage. The proposed design was rated as 8 out of 10, and the State plans were rated as 9 out of 10.

6.3.5 Incentives

The calculated incentives consist of the Work Zone Road User Cost which put a monetary value on the delays borne by commuters. The design minimizes the impact to the public and commuters. The duration of the closure of Pelham Road was the same for both the proposed design and the State plans, but the duration of the reduced speed limit for the project was only 130 days compared to the drawn out State plans which had a reduced speed limit for 568 days. This made the design Work Zone Road User Cost 4.3 times less than the State plans. The rating was determined by selecting a reasonable range of values for the incentives which both projects fell within. The rating was determined as a range from 1-10 as a function of the Work Zone Road User Cost. This function is: f(WZ RUC)=-5x+10, Figure 99 displays the graph representing the relationship between the rating and incentives. The Work Zone Road User Cost for the design was calculated to be $0.392 million and $1.688 million for the State plans. This led to a rating of 8.040 out of 10 for the design and 1.562 out of 10 for the State plans.

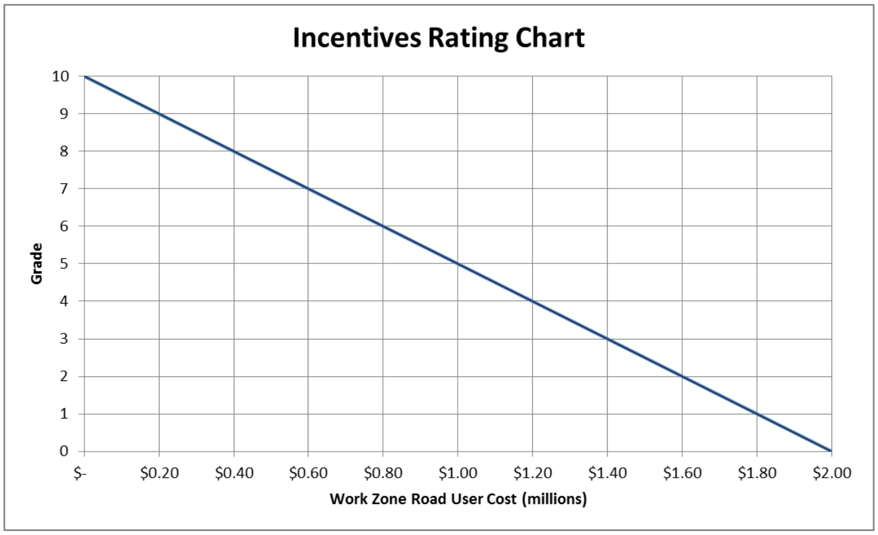


Figure 99: Determination of Incentives Grade.

6.3.6 Final Evaluation

The scores for the five categories described above were multiplied by their respective weights and summed together to determine the total grade. The grade for the incentives is what gave the project such an advantage over the State plans, without the incentives category the total grade would have been 5.20 for the project and 5.15 for the plans. The scoring breakdown and total grades is displayed below in Table 37.

Table 37: Scores of Project and State Plan

	Cost (million)	Cost Score	Constructability	Aesthetics	Maintenance	WZ RUC	Incentives	Total Grade
Project	$ 1.997	5.03	8.5	6.5	8	0.392	8.040	6.81
State Plan	$ 2.045	4.55	8	8	9	1.688	1.562	5.46

6.4 Conclusion

In conclusion, the Prestressed Deck Bulb Tee Bridge designed in this project is an effective alternative to the State design for the same bridge. As the bridge analysis proves, when all factors are considered, it can be advantageous to Accelerate Bridge Construction (ABC). The particular style of design is not the only method of accelerating construction, but it of the three options considered to reduce project duration and decrease impact on the roadway and the community. The other two options were a single-span system of composite deck with steel plate girders (similar to the State design) and a two-span continuous system of composite deck with rolled steel sections. A final centerline profile of the project bridge is shown with dimensions in Figure 100.

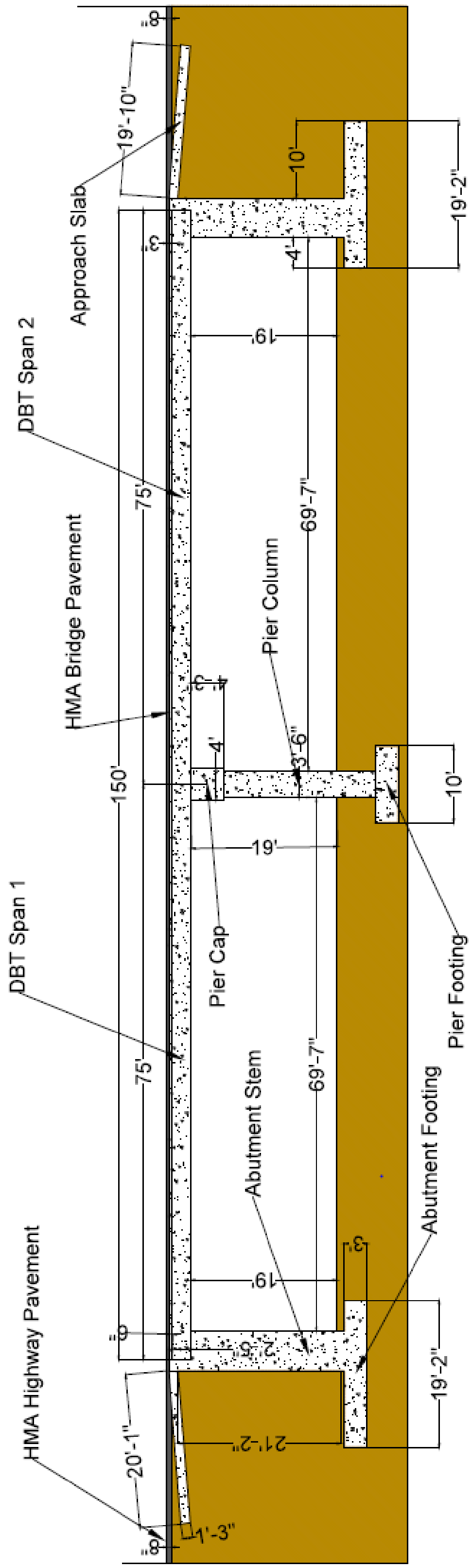


Figure 100: Construction Profile of Proposed Bridge Design Along Centerline.

6.4.1 Future Consideration

For future applications, ABC is one of the most intriguing and fastest growing segments of bridge construction. Any method of improving construction at a minor cost must be considered. Both the increasing costs to users and the ever increasing cost of labor and materials demand that projects be completed quickly. ABC construction is an important consideration for any bridge replacement with adequate space on a major highway crossing. Further projects may investigate the benefits of a more extreme, totally precast bridge or the benefits of non- steel or concrete structure, such as a composite bridge system. With the advanced age of much of the infrastructure in the US there is considerable opportunity for expansion and exploration into the region of Accelerated Bridge Construction. Another study could focus on the cost-benefit ratio of a standardized bridge network versus a specifically designed and dynamic bridge network. This is of particular interest due to the practice of generally standardizing the bridges on the southern I-93 corridor by NHDOT.

6.4.2 WPI Project Experience

This project represents the literal and figurative culmination of a civil structural course of study at Worcester Polytechnic Institute. Several classes have been of great use to this project. Obviously, this project required extensive of design courses in steel, concrete, prestressed concrete, and foundations, as well as knowledge and skills from the areas of soils, transportation, and estimating and project management. In the course of the project, subject, broad subject matter from nearly every course offered in the civil department has been of use to the design and project construction process.

The group dynamic of the project was another important step in the process. It was a unique experience to participate in and coordinate a project involving considerable depth and complexity. It was also an opportunity to work at both group management and individual discipline.

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HIGHWAY BRIDGE REDESIGN

A Proposal for a Major Qualifying Project Report:

Submitted to Faculty of

WORCESTER POLYTECHNIC INSTITUTE

In partial fulfillment of the requirements for the

Degree of Bachelor of Science

By

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Date: October 22, 2012

Approved:

Professor Leonard D. Albano, Adviser

Contents

Problem Statement

The bridge on Interstate-93 Southbound going over NH-97 (Pelham Road) has been put on the New Hampshire Red List for bridges that are structurally deficient or functionally obsolete. The bridge was noted to be in serious condition and is scheduled to be replaced in 2012. It scored very poorly on the last published inspection completed November 2009. The deck condition was rated as serious, scoring 3 out of 9 possible marks. The superstructure condition was rated as satisfactory with a mark of 6, and the substructure was rated as poor with a mark of 4. The structural appraisal of the bridge was "Basically intolerable, requiring high priority of replacement." With this evaluation the bridge is structurally deficient and must be replaced. The design and construction of this bridge is especially important given the fact that it supports a major artery in the Northeast (I-93), and averages 14,500 daily commuters as of 2006.

The replacement of this bridge must be done as soon as possible to insure the safety of the commuters, and as quickly as possible to minimize traffic congestion. The fact that this bridge is a vital part of a highway used by so many people only makes it more important to be replaced as quickly as possible. The bridge currently lacks the capacity to effectively handle the traffic flow with only two lanes. It is important to widen the new bridge to four lanes to handle the traffic. A construction plan must be carefully devised to avoid traffic congestion by allowing traffic to pass over the bridge whenever possible and minimizing the duration of bridge closures. The solution must also be economical since the state of New Hampshire has so many highway bridges to maintain. The construction began during August of 2012, but the majority of the work is to be done from late 2012-2013

Objective

The purpose of this project is to design and evaluate alternative bridge designs while impacting the surrounding community and restricting traffic as little as possible. The designed bridge must be structurally adequate to handle the traffic and loads related to construction or weather. The design of the bridge must also be able to accommodate current and future traffic volumes. The construction of the bridge must be done using Accelerated Bridge Construction in order to ensure that traffic congestion and bridge closings are kept to a minimum. The design must also be an economical and practical solution. A successful design will show measurable improvement in delays or cost as compared to traditional construction.

Scope

The purpose of the project is to identify the most economical bridge replacement plan for I-93 SB over Pelham Road. Background research on different bridge types is to be conducted to narrow down some practical design considerations. Research on different materials, planning and design processes and construction methods is to be considered during the project. The two most practical designs will be further developed into preliminary bridge designs for evaluation. A specific bridge design will be determined based on the economic feasibility and design features from the preliminary designs.

A full design of the chosen type will be completed, including the substructure, superstructure, wearing course and safety devices. This will be accomplished while taking into consideration construction methods. Once the design is complete a construction plan and traffic diversion plan will be developed. The development of an environmental plan will be devised after the structural design and construction plan are completed. The completed design will then be compared to the state's plan for the Pelham Road replacement bridge and other similar projects.

Background

According to the American Society of Civil Engineers, there are just north of six hundred thousand bridges in the United States (ASCE). This may catch the attention of the average American, because that works out to one bridge for roughly every five hundred citizens. Everyone is aware of the majesty of the Golden Gate in San Francisco, and the bottleneck of the George Washington Bridge over the Hudson River. Not as many people are aware of the abundance of bridges required to maintain grade separation for interstate highways.

Let us look for instance, at a theoretical trip from Londonderry NH, entering at exit 5 on I-93, and finishing at the start of the Zakim Bridge in Boston. Many people who live in southern NH work in the Boston area, making their commute by this route every day. A Google maps trip search deems the drive a 46 mile and (very optimistically) a 51 minute excursion. To maintain the grade separation, which in turn allows the higher speed flow of more traffic no fewer than forty-six separate bridges must be crossed(Google Maps). That is just for a southbound, one way trip, and does not include the additional twenty-two overpasses carrying other roads above the highway. A return trip northbound would nearly double the number of separate bridges crossed. Most of these would be similar to the respective southbound structures, but the number of crossings required for a roughly 92 mile round trip is staggering.

The average bridge may pass under the average citizen's radar, but they do not pass the years unmarked by corrosion, fatigue, and traffic damage. The Eisenhower Interstate Highway System was signed into law by the Federal Aid-Highway Act in 1956, and changed American culture and transportation forever (Eisenhower Interstate Highway System Web Site). The corridor of the new Interstate -93, stretching from Manchester, was one of the earlier sections completed, with the NH sections finished by the early 1960s. The new highway systems represented unprecedented speed and scale in terms of roadway and bridge construction. Now, nearly fifty years later, that presents a quandary for state highway departments nationwide. Due to the great growth and bridge building periods of the early and mid-twentieth century, the average age of a bridge in the United States is 43 years old (ASCE). Many of the currently used bridges were designed without the aid of computer modeling and using hand-drafted plans.

As of 2008, ASCE classified more than 12% of bridges as structurally deficient and over 14% of bridges functionally obsolete. The report describes these two conditions as such:

"A structurally deficient bridge may be closed or restrict traffic in accordance with weight limits because of limited structural capacity. These bridges are not unsafe, but must post limits for speed and weight. A functionally obsolete bridge has older design features and geometrics, and though not unsafe, cannot accommodate current traffic volumes, vehicle sizes, and weights. These restrictions not only contribute to traffic congestion, they also cause such major inconveniences as forcing emergency vehicles to take lengthy detours and lengthening the routes of school buses."-ASCE 2009 Infrastructure Fact Sheet

As the definition makes clear, most of the structurally deficient or functionally obsolete bridges are not in immediate danger of collapsing or failing. They exact their price on society through delays to commuters and during emergencies, in heavy maintenance costs and in unnecessary suspension damage to the average automobile. Even worse, through the difficulty of predicting the failure of half century old spans, some do fail. These failures include not just the catastrophic failures like that of the Interstate-35 bridge in Minnesota, but many less publicized but still scary incidents. In 2010, just north of Boston, a deck failure on Interstate-93 sent concrete crashing onto the road below, and caused lane closures for several days (Fast 14: I-93 Rapid Bridge Replacement Project). In 2007, in Boston, trucks were banned and commuters sent scrambling when the Tobin Bridge was revealed to have cracks in a main support beam, just months after debris from the bridge had fallen on boaters below (Taurasi). On September 1st 2012 a section of the Arborway in Jamaica Plain had to be closed for emergency repairs as concrete fell onto traffic below (*wcvb.com*). The list of minor, but scary failures goes on and on.

It is clear, as stated in the ASCE report plan, that a coordinated and determined effort to maintain and replace bridges, especially highway bridges, is crucial to America's continued quality of transportation. It is a urgent situation, and the sheer scale in terms of number of spans is staggering, and so is the fact that nearly every American is and will affected by the success or failure of current and future engineers in improving the system. Replacing these bridges, or repairing them, in a cost efficient way with minimum traffic delays and danger is ultimately important. Every option to increase construction speed, lower cost and eliminate traffic delays must be exhaustively evaluated.

Interstate 93 SB over Pelham Road

One of the forty-six crossings Interstate-93 crossings between Londonderry and Boston is an overpass over NH Route 97, or Pelham road. It is a twin to a very similar span carrying northbound traffic over the same road. This bridge is deemed Salem 068/078 by the state, and most people barely regard it as they pass on their way to Boston. The existing span currently carries just two southbound lanes of traffic, with minimal shoulders and no breakdown lanes. One of the lanes is larger than standard to allow for crossover merging for the cloverleaf style exit system surrounding the bridge.

This relatively standard bridge was designed in November of 1959, by the Clarkson Engineering Company of Boston Massachusetts. It was designed for a modified Military loading of a 16 kip wheel load on the deck slab, according to the AASHO 1957 specification, and the NH DPW H. 1954 with 1958 supplement(1959, Clarkson Engineering Company Incorporated). The bridge was constructed in 1961 and has been in service continuously since then. It currently is used by around fifty thousand vehicles a day (NHDOT Construction Plans). The bridge, as seen from the west, is pictured below in Figure 1.

Design wise, the bridge contains three spans. These spans consist of rolled wide flange steel beams. End to end, the first span stretches from a mass abutment, to a hammer head pier supported by three circular columns, the second span stretches from one hammer head to pier to another and the final, third span stretches from the second pier to a second mass abutment. Figure 1 below shows the bridge as originally envisioned

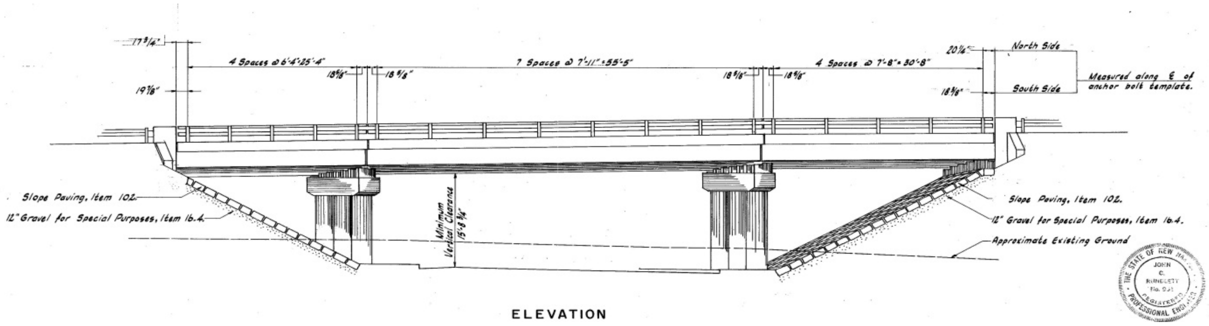

Figure 1: A rendering of the bridge from the original 1959 plans. (Clarkson Engineering Company Incorporated)

The bridge is located in a commercial area. Most of the daily travel on week days is commuter traffic, and a significant number of people work in the area immediately surrounding the exit interchange. On the weekends there is less commuter traffic, but during the summer, there is significant draw to Canobie Lake Park, a local amusement park. Specifically, heading eastbound on Pelham road, there are several industrial complexes, a park and ride station and the amusement park. Heading west, there are many office and light industrial complexes. There currently is a significant area of undeveloped land near the bridges due to the old cloverleaf style exit setup. The bridge can be seen from the air in Figure 2 below.

Figure 2: A satellite picture of I-93 SB over Pelham Rd. (Google Images)

The bridge is also currently in poor condition, unsurprising for a fifty plus year old span. There has been significant damage to the concrete cover in the substructure. Water and chloride related spalling has removed much of the concrete on the top of the hammer head piers, and at the bridge seats of the mass abutments. There is also obvious damage to the concrete piers themselves, mostly cracking. Figures 3 and 4 below show damage to the bridge seats. Figure 5 shows severe damage to one of the piers supporting the twin northbound span of identical age and design.

Figure 3: Large number of repaired and current concrete failure locations on the southern pier of the SB Bridge. Note the very evident water staining and continued cracking to the bridge seats.

Figure 4: Clear evidence of rust related spalling, reinforcing cage fully exposed in places. This damage is located on the bottom of the hammerhead on the northern pier of the SB bridge.

Figure 5: Severe spalling, cracking and separation of concrete cover on northern pier of NB Bridge.

The roadway structural deck and coping are also in poor condition. The coping is severely decayed, with reinforcing cage completely exposed in places on the NB Bridge. The deck is shedding the bottom cover concrete, and has been repaired extensively, with a very large repair using "leave in place" forms on the evident near the southern abutment. The entirety of the span over traffic is covered with debris netting to reduce the chance of debris harming passersby. The netting can be seen in Figure 6 below and some of the forms in Figure 3 above. The roadway surface has been recoated recently and is in very good condition, so it offers little insight as to the condition of the concrete below. Unfortunately the concrete sections of the bridge are in need of immediate repair or replacement (NHDOT Red list).

In general, upon our observation the steel superstructure is in good condition. The paint is beginning to fail in select sections, but significant loss of section in the stringers has not occurred. The safety rail at the top of the bridge is visibly rusted. The actual effectiveness of the bridge is unknown without further examination, which is made difficult by the lack of shoulders and the heavy traffic on the bridge. Some of the damaged paint and decayed coping on the NB Bridge is visible in Figure 7.

Figure 6: Debris netting under the SB Bridge

Figure 7: Paint damage and decaying roadway coping. Picture shows the northern expansion joint on the eastern side of NB Bridge.

Design of Highway Bridges

State/Federal Highway Design Specifications

Federal and state codes are important design requirements in the construction of commercial and state structures. The Federal Highway Code used in the United States is the *American Association of State Highway and Transportation Officials (AASHTO) LRFD Bridge Design Specification*. The specification ensures that bridges are "designed to specific limit states to achieve the objectives of constructability, safety, and serviceability, with due regard to issues of inspectability, economy and aesthetics."

The New Hampshire DOT also provides a highway code. It is called the *Standard Specifications for Road and Bridge Construction* and contains additional design requirements specific to New Hampshire.

The (AASHTO) *LRFD Bridge Design Specification* also presents limiting coefficients called limit states. The limit states serve the purpose of ensuring a safe cushion between calculated values and extreme cases. The limit states tend to lower capacities of elements to ensure a slight over design. They can also serve to raise capacity requirements, which can also result in over design.

Dead and live loads are also derived from the AASHTO Specification. Dead loads include permanent loads such as material weights and static loads. Live loads include loads that are not constant such as vehicle, wind, snow, and various possible loading conditions. Chapter 3 of the AASHTO 2012 Design Specification describes the various loads that can be applied to different bridges. Load combinations are also used with dead and live loads. The combinations add coefficients to different loads and the largest load combination governs.

The Highway Codes are important references when designing new or replacement bridges. They ensure a proper factor of safety is achieved for the structure and all governing loads and forces are accounted for in the design. This can improve the overall quality of the structure. Higher factors of safety can result in longer lasting bridges and lower maintenance costs.

Bridge Superstructure

There are a wide range of structural possibilities when designing a replacement bridge so it is important to be able to recognize a few practical designs to complete an efficient design and analysis. The function of the bridge is one of the first aspects of the bridge to consider when planning a design. The function of the bridge will yield the design loads and provide an idea of how much support the bridge will require. The project bridge is being redesigned is an interstate bridge so it will need to carry a substantial load. Another important aspect of the bridge is the span length. The Pelham Road overpass being redesigned falls on the larger side of a medium span bridge spanning 167 feet. Medium span bridges have typically been prestressed concrete and steel girders. As the materials and design concepts used improve, the bridge spans are more capable of extending to longer distances. The longer the bridge can span, the less substructure is needed for support. This can reduce the total cost of the bridge and reduce the time and effort spent during construction. The fewer members needed to create a bridge, the quicker it can be erected which leads to less traffic congestion. Unfortunately it is also important to consider the increasing material costs, weights and depths involved with increased spans.

The structural type of a bridge defines the structural framing system and the type of superstructure. There are four major types of structural framing systems. One of the structural framing systems is a simple span. This consists of a superstructure span containing just one unrestrained bearing at each end. The supports allow rotation as the span flexes under the load. There must be at least one support to keep the span from moving longitudinally. Another type of structural framing system is a continuous span. This consists of one continuous span crossing at least three supports. The member's rotation is restricted in the area next to the pier since it is continuous. The third structural framing system is a cantilever and suspended span. The span is continuous over the pier and ends shortly after creating a cantilever. This cantilever is typically used to support the end of an adjacent span. In this type of system, there is a "suspended" span which is relies on the adjacent cantilever spans for support. Sometimes the suspended span rests on an ordinary simple support. The final structural framing system is a rigid frame. Rigid frames are generally used as transverse supports, but can also be used as longitudinal spans. They are called rigid frames because the method of fabrication does not allow relative rotation between members at joints.

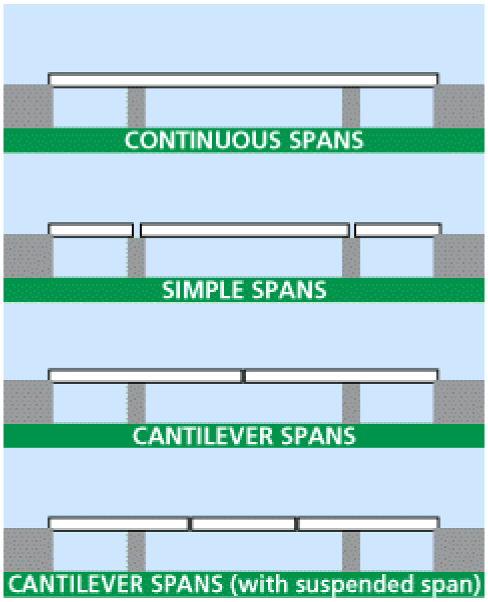

Figure 8: Four types of spans ("Bridge Basics").

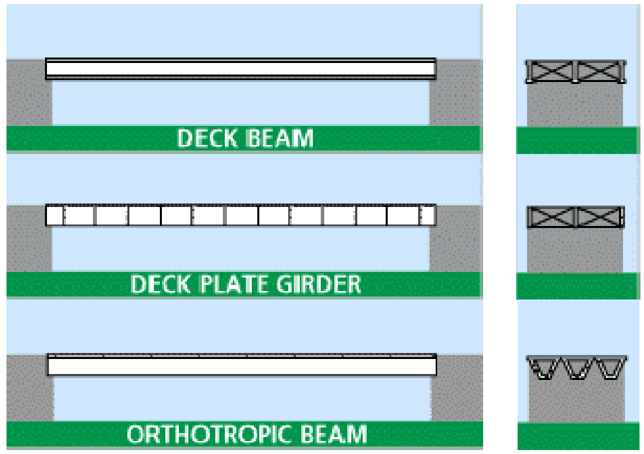

Figure 9: Commonly used beams ("Bridge Basics").

There are many different types of superstructures for a bridge. Arch bridges, cable-stayed, and suspension bridges are all bridge types that would typically be large spans and they are too involved and costly for a medium span interstate bridge. Such complicated construction techniques are simply unnecessary for a typical span. A truss is not a good choice either, and not just because it is historical form of construction. Steel trusses were originally economical because they save on material, and have more but smaller pieces. This is exactly why truss bridges are no longer economical for even large spans, because material is relatively inexpensive today but the enormous amounts of labor in truss fabrication, construction and maintenance is not. Truss bridges as a rule are also less safe, because they tend to be non-redundant, have a huge number of connections that can fatigue, and have smaller members that are easily damaged by rust or traffic. Labor costs can also quickly eliminate stone or cast-in-place reinforced concrete bridges. Having a hundred laborers onsite is not going to produce profit in 2012, with the emphasis on speed and consistency.

Therefore, despite the great variety of bridge choices, the cold pressure of economics generally limits medium span highway bridges to a choice between or combination of two options for superstructure material. The first is material is precast, pretensioned concrete beams or other structural shapes, made offsite at a factory. The second material is the venerable classic, steel I shaped beams. The vast majority of new bridges of similar spans to the project bridge utilize one of these two materials. There is also a great deal of experience and literature in the construction industry involving these two types of bridge structure.

Precast Pretensioned Bridges

Precast concrete has a long history of use in transportation applications. The history of precast concrete and prestressed concrete for use in bridges is thoroughly intertwined. Prestressed concrete takes advantage of increased strengths by using tension elements to create compressive stress in members. This compression is induced by either pre or post tensioning, and allows the members to support much higher loads. This in turn, allows for much smaller cross sections than regular cast-in-place concrete to support the same loads. This is important for bridges with their long spans and persistent clearance issues.

Cast-in-place concrete can be prestressed onsite, but the extra tensioning equipment, in addition to the typical forms and false work would quickly make for a costly operation (PCI Manual). It would be for all purposes impossible to do this for a replacement bridge. Therefore, precast is married to prestressed in bridging applications, with members arriving fully cured and ready for installation. The first bridge constructed using prestressed concrete was in 1949 in Philadelphia, Pennsylvania, and now the precast-prestressed concrete industry accounts for roughly 50% of new bridges in the United States.

Structurally, precast varies by bridge span length. For very short spans, precast slabs, or voided slabs are used, sometimes as part of a rigid (three sided) bridge system. These techniques can be used up to spans of forty feet. For mid span bridges, there are a variety of concrete shapes available such as a multi stem, double stem or channel setup, with economical spans to around sixty feet or so. For the longest spans, those stretching to a hundred feet or more, there are typically three options. First is single stem, featuring a very large beam shape that tapers to the edges of the deck. The second is a box beam style, a wide, deep box shaped element that supports the deck directly above it. The final type of long span precast and one of the most often used for highway projects is the bulb tee or I beam shaped member. Often known as the “New England Bulb Tee” these are concrete members than look similar to rolled steel members in cross sectional shape.

States choose to use precast for a variety of reasons. One can often be cost. Precast bridges are competitive on initial cost, depending on region with other types of bridge structure. Once life cycle costs are considered, precast can sometimes become even more attractive. Precast concrete tends to have better resistance to damage than CIP concrete and does not need to be painted as traditional steel does. Precast tends to have very low net stresses due to the prestressed nature of the structural members, as well as a very massive cross section. This produces savings as bridge decks and mounting components wear out more quickly on more flexible steel bridges(PCI Manual). Precast bridges also come in a variety of aesthetic shapes and colors, and are more easily adapted to architectural interference. It can simulate a wide variety of finishes at low cost.

Precast is more easily manufactured than structural steel, and there are more local producers of the precast structural elements than those for steel(PCI Manual). Precast orders, especially those taking advantage of standard or common shapes such as Bulb Tees can often have a fraction of the lead time of structural steel. This can be of great use in ABC or in emergency situations.

Steel Stringer Bridges

Structural steel has been an important material in the construction of bridges for well over 100 years. In fact the first time steel was used in bridges was the Kymijoki railway bridge in Finland. It was the first 3-span steel truss bridge and was built in 1870.

A multitude of designs are now used for various bridge spans. Some common steel bridge types include plate girder, box girder, or rolled beam bridges. The plate girder is comprised of a vertical plate, which has top and bottom flanges welded to it to create an I-shaped beam. They are generally optimized for bridges with spans of 125 feet or more if the girder spacing is between 11 and 14 feet. Box girders include welding a top plate to a C-shape beam to create an enclosed box and can be used under similar circumstances. Rolled beam bridges are generally used for short spans up to 100 feet and continuous spans up to 120 ft. The span lengths are restricted due to limitations of the steel mill. Issues with transportation of beams also govern the longest continuous beams available.

The main disadvantage of using structural steel for bridge design is the increase in cost from that of reinforced concrete or pre-cast concrete. Life span and required maintenance can also be factors in the total cost of the project. Various coatings and paints or weathering steel can be used to greatly increase life span and reduce the overall cost of repairs (McCormack).

Steel bridges can prove to be very desirable due to their ability to provide high capacities with low material weights. In fact weight optimization is one of the most important aspects of efficient steel bridge design. Designing optimized members can improve efficiency if employed correctly. Often times the lightest weight or most optimal shaped member is not a standard size provided by common steel mills. Therefore it is important to optimize the design while also keeping in mind the most common member sizes. Structural steel bridges can be very effective in various situations though the scarcity of steel mills and the span restrictions of bridge elements can make steel construction more expensive than alternative methods.

Foundations

The substructure is one of two main parts of a bridge. The purpose of the substructure is to transfer the loads of the bridge to the supporting ground below. The loads from the superstructure are transferred to the bearing plates which then transfer the load to the foundations, which are supported by the ground. The types of foundations used are greatly dependent on the site geometry and soil strength. Large spread footings are preferred because they are shallow foundations which do not require much excavation, and they distribute the loads over a larger area. In more extreme conditions it may be beneficial to have some deep foundations which have a smaller footprint and penetrate to deeper levels of the earth, offering more stability and support.

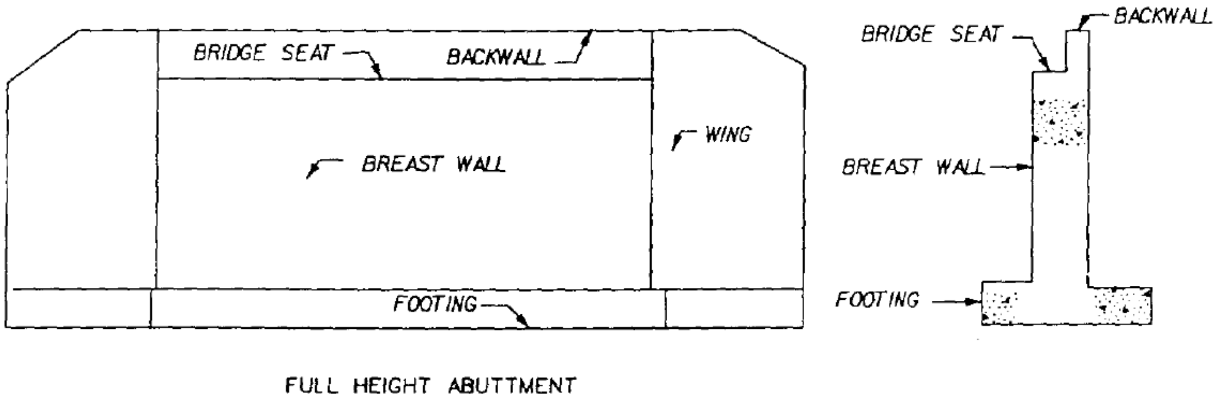

Figure 10: Standard abutment (*Bridge Inspection, Maintenance, and Repair)*

The common foundation types for bridges are abutments, piers, and bents. The abutments are the structures which offer support at each end of the bridge. Abutments are either constructed with plain concrete, reinforced concrete, stone masonry or a combination of concrete and stone masonry (*Bridge Inspection, Maintenance, and Repair*). They will be the only foundations for a single span bridge and consist of five parts; a footing, a backwall, a stern or breast wall, a bridge seat and a wing wall. The backwall is a small retaining wall which extends up from the abutment. Its purpose is to prevent the embankment soil from spilling on to the bridge seat and they must provide the necessary clearance between the ends of the bridge span and the face of the backwall to allow the bridge to expand and contract ("Backwalls"). Wing walls are the retaining walls adjacent to the abutment responsible for keeping the embankment soil around the abutments from spilling into the waterway or roadway being spanned by the bridge (Childs). The bridge seat is an indent on the top of the abutment which contains the bearings on which the superstructure sites. This is the area where the end of the span will be supported and connected to the abutment. The breast wall is the retaining wall directly under the span. It is used to prevent the embankment soil from spilling over and maintain the structural integrity of the embankment soil by providing horizontal stability. The footing is what anchors the abutment in the ground and is typically a very wide shallow foundation.

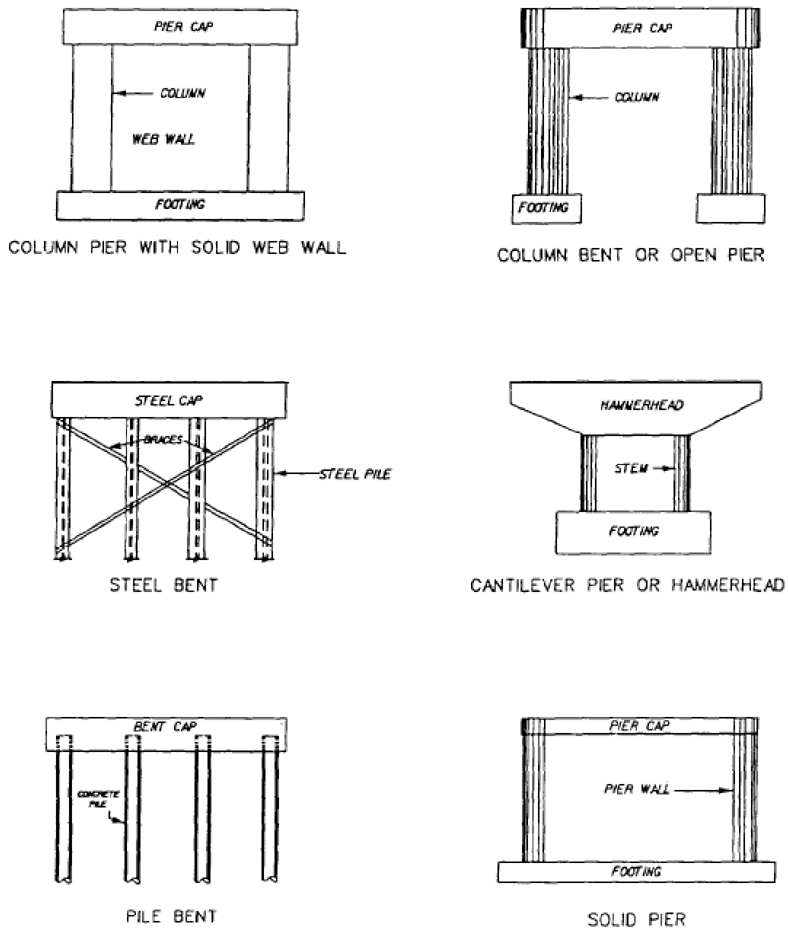

Figure 11: Typical piers and bents (*Bridge Inspection, Maintenance, and Repair*)

Piers are used as supports between the two abutments and generally consist of footings, columns or stems, and caps. A highway overpass can use a variety of foundations. Some designs consist of a single span only supported by abutments, while others require one or more other types of foundations between the abutments. The footings are generally slabs at the bottom of each column or stem which transmit the load into the deeper layers of soil or rock. The footings must be designed to resist the vertical load and moment from the superstructure, if the slabs cannot adequately resist the load additional deep foundations are required. Some of these deep foundations may be piles, caissons, or drilled shafts which penetrate deeper into the soil and possibly bedrock to secure the footing. The columns or stems transmit the applied loads and moments to the footings and must meet strict size and clearance requirements. The cap of a pier takes on the loads from the superstructure and transmits the loads to the columns or stem. The pile cap will bind the columns together and create a rigid structure if the pier consists of multiple columns. Bents are piers which penetrate the ground but don't have footings. Piers and bents are typically made of steel, concrete, stone, or a combination of the materials.

Pavement

Pavement is generally one of two materials in the United States. It is either aggregate with asphalt binder, or some form of concrete. An asphalt paved road varies in thickness depending on traffic and soil conditions, but is generally between six and eight inches thick for newly placed highway (Mallick- El-Korchi). Concrete pavement is generally thicker and can be reinforced in one of three ways. Concrete can be unreinforced, reinforced at panel joints or continuously reinforced through its whole length. Due to the vast number of freeze thaw cycles and the heavy application of road salts, northern areas tend to use more asphalt paving.

New England is an almost exclusively asphalt paved region (CE 3051). An asphalt road consists of various layers with different grades of asphalt. Generally the sub base, or underlying ground is compacted or replaced, then a base layer of structural stone or sand is layered and graded. On top of this coat comes a first thick layer of base course, asphalt with large aggregate and lower asphalt content. Following this comes at least a layer or two of medium asphalt, with a final wearing course at the top of about 3" of the highest quality asphalt.(Mallick- El-Korchi)

The highest layers are the best quality because pavement is designed according to stress cones from wheel loads. Stress is most concentrated in the top and dissipates as it moves into a substance, allowing the use of lower cost materials in lower layers. Software and equations allow engineers to model the stress at different points beneath the roadway. They can then pick the least expensive material that meets the stress criteria. The top wearing course must also meet friction requirements to allow braking and maneuvering, and be able to withstand weather and UV damage.

Typically, pavement surfaces are designed by peak flow methods. Catch basins are designed to handle maximum flow from a peak storm, and equations to eliminate pooling dictate a minimum slope based on highway width. A typical sideways slope, looking at a cross section of a four lane highway could be as follows: A peak between the second and third traffic lanes, with the traffic lanes tipping towards their respective shoulders at 2%, with the shoulders themselves sloped at a steeper 5% or so(NHDOT Construction Plans). At no point can the roadway be completely flat or concave, because the pooling during rain and ice during winter would create an immense traffic hazard (Mallick- El-Korchi).

Safety

The bridge design will include bridge railings classified as longitudinal barriers. The bridge railings will meet full-scale crash-test criteria and meet the Federal Highway Administration (FHWA) codes and regulations. If piers are used, they will be surrounded by Jersey Barriers for the safety of the driver and to protect the structural integrity of the foundation. Vehicle safety for both Pelham Road and Interstate 93 will be considered throughout the design process.

Construction Plan

Traditional Construction Plan

In the United States, an owner or agency usually proposes a bridge for construction. An external design firm reviews the job and advertises it to the public where contractors will bid on it. The owner then accepts the most favorable bid and the design phases begin.

The first issue that needs to be resolved before any work can begin is that of funding. Next, the proper standards and references are chosen based on the location of the project and the type. Once all the preliminary requirements are met the design can begin. For existing bridge sites, inspections must take place to observe the present condition. The site can then be inspected for various features such as drainage conditions, vertical clearance, soil condition, horizontal and vertical alignment constraints, underpass or channel constraints, and potential utility impacts.

The design of any bridge involves implementing various substructure and superstructure elements. The different components such as the abutments, footings, piers, beams and girders are designed using AASHTO's *Standard Specifications for Highway Bridges*. Design loads are implemented to determine the correct capacities for the bridge elements.

Unlike Accelerated Bridge Design there are fewer restrictions to the building materials that can be used during construction. Materials such as cast-in-place concrete can be used more effectively with conventional construction due to the more relaxed timelines and use of detouring and lane closures.

During construction various safety precautions have to be enacted to protect motorists and workers. This can include using detours, implementing lane closures, or even constructing the bridge in a new location and then altering the existing roadway alignment to incorporate the new bridge.

Once the construction is complete, it is general practice to setup maintenance schedules to help ensure the durability of the structure. This can ensure that the structure has a long life span and will not require costly repairs in the future.

Accelerated Bridge Construction Plan

Over the past few decades Accelerated Bridge Construction (ABC) has emerged as an effective alternative to conventional bridge design. ABC involves employing efficient planning and design, along with innovative materials and construction methods to substantially reduce the total construction time of new or replacement bridges. This can be very desirable when considering the replacement of bridges that are vital to the flow of the traffic system.

By implementing ABC to a given project, substantial improvements to various consequences of construction can be observed. For instance time delays due to traffic impacts and weather can be lowered. Additionally, by applying ABC, impacts to traffic flow can be greatly reduced. This is because ABC allows there to be minimal to no traffic detouring, no temporary bridge structures, and no change to the existing roadway alignment. Also, due to the efficiency of the process, ABC can greatly reduce the environmental impact of new or existing bridge construction.

An important aspect of ABC is the use of Prefabricated Bridge Elements and Systems (PBES). As the name suggests, PBES are structural elements that are made off site under controlled settings. The elements are generally pre-cast concrete, composite pre-cast and steel, or steel systems, and can be used for all structural components including decks, beams, piers, and abutments. PBES have their advantages and disadvantages. An advantage is the higher quality of the materials when using PBES. The increase in quality is due mainly to the environment in which the elements are made. The facilities are generally set up so that temperature, humidity, and weather conditions are controlled. As a result of the higher quality of the materials, PBES can provide improvements in safety, quality, and long-term durability.

The main disadvantage of ABC with PBES is the increase in cost of construction. As stated by the *Accelerated Bridge Construction* manual, companies who have completed initial ABC projects with PBES have seen increases from 10% to 30% in construction costs. The main factors that affect the increase in cost include the size of the project and the tight construction time limits. Obviously a bigger project will lead to higher costs due to the greater amount of PBES needed. A tight construction time limit also increases costs because more work needs to be done in less time. This could involve including more engineers for the design, more workers to ensure the structure goes up in a timely manner and more effort by the contractor to create a very efficient construction schedule.

There are many factors that can affect the need for the use of ABC. An effective way of deciding if ABC is an appropriate alternative to conventional bridge construction is to look at how much the mobility impact time will be improved. The ABC manual defines mobility impact time as any period of time the traffic flow of the transportation network is reduced due to onsite construction activities. The list below demonstrates the different tiers of mobility impact time.

Tier 1: Traffic Impacts within 1 to 24 hours

Tier 2: Traffic Impacts within 3 days

Tier 3: Traffic Impacts within 2 weeks

Tier 4: Traffic Impacts within 3 months

Tier 5: Overall project schedule is significantly reduced by months to years

If the use of ABC lowers the tier number associated with a proposed project it is useful to conduct an analysis to determine if the use of ABC is appropriate.

Accelerated Bridge Construction has proven to be an important asset over the years. As was seen in the “Fast 14” bridge replacement project, ABC can greatly increase the efficiency of a proposed job. The Fast 14 involved replacing 14 deteriorating bridges on I-93, north of Boston. The use of Accelerated Bridge construction made it possible for the job, which could have taken as long as 4 years under traditional methods, to be completed in just 10 weekends. To keep the project on schedule, strict timelines were enacted to ensure maximum productivity and minimal traffic delays. For instance the *Engineering News-Record* states if the four lanes of I-93 did not open by 5 am on any given weekend the joint venture would have lost $3.23 million for the first minute alone but if the bridges were completed in the predicted 10 weeks, $7 million of incentives would be given. This technique of offering incentives and disincentives proved to be very effective in the Fast 14 project and can be used with future projects to increase efficiency.

Cost of Delays

Many of today's construction projects consist of rebuilding structures in urbanized areas or performing maintenance on structures that many people depend on to accomplish everyday tasks, such as commuting to work. Closing down frequently used structures and roadways can have a serious effect on the commuter, surrounding businesses, and the community.

Vehicle miles travelled (VMT) on U.S. highways have doubled in the past three decades, while the miles of highway has only increased by 5% during the same period of time (Mallella and Sadasivam). The cost congestion incurred by the commuters for travel delay and extra vehicle fuel has risen from an estimated $24 billion in 1982 to $115 billion in 2009. To keep up with the pace of growing congestion, the funding for highways have increased by 300% through Federal, state, or local funding in the past three decades (Mallella and Sadasivam). As a result of this increased funding, there have been many new construction projects to repair, improve and widen existing roadways. These construction projects have been encouraged to use innovative construction methods which limit traffic and safety implications and use the time spent obstructing traffic flow as efficiently as possible.

One method of limiting traffic congestion is making use of "A+B" bidding. "A+B" cost estimation and bidding gives the construction company incentives to maximize efficiency and minimize the time a roadway is shut down. The "A" component represents the base cost of the project if it were to be completed using a traditional construction method. The "B" component represents the cost of the impact to the public. Some incentives given are paying workers extra to work overtime or double shifts and bonuses for completing a project ahead of schedule. The projects are encouraged to schedule operations to maximize efficiency. Some ways to maximize efficiency are to decrease idling time during construction, have multiple activities occurring at once if they don't have to be done sequentially, have all the equipment ready for when it is needed, have as much of the material as possible prefabricated to expedite the construction process, and have major construction done at hours with the least amount of traffic flow.

The Work Zone Road User Cost (WZ RUC) is the additional costs borne by the motorists and the community as a result of the work zone activity (Mallella and Sadasivam). The WZ RUC consists of the monetized components of the implications of the work zone such as the user delay costs, vehicle operating costs (VOC), crash costs, and emission costs. Other off-site components like noise and impacts to the business and local community can also be taken into consideration. These components, especially the off-site components can be extremely difficult to monetize because it is difficult to put a quantitative value on something qualitative like noise. The steps for computing the road user cost are as follows: (Mallella and Sadasivam)

1. Gather data for work zone impact assessment
2. Estimate work zone impacts
3. Compute unit cost for each impact type
4. Estimate WZ RUC components

The travel cost delays are calculated by multiplying the estimated delay time by the unit cost of travel time.   The delay time is quantified by the additional travel time needed to pass through the work zone or detour around it. The delay time is then multiplied by the unit cost data. This information is quantified as an hourly rate taking into consideration the value of personal travel, business travel, truck travel, time related depreciation of the vehicle, and the value of freight inventory for loaded trucks. Once the unit cost data is determined, the number of vehicles must be determined. The number of vehicles corresponding cars to the unit cost data is then obtained and multiplied by the time delay and unit cost data to obtain the WZ RUC.

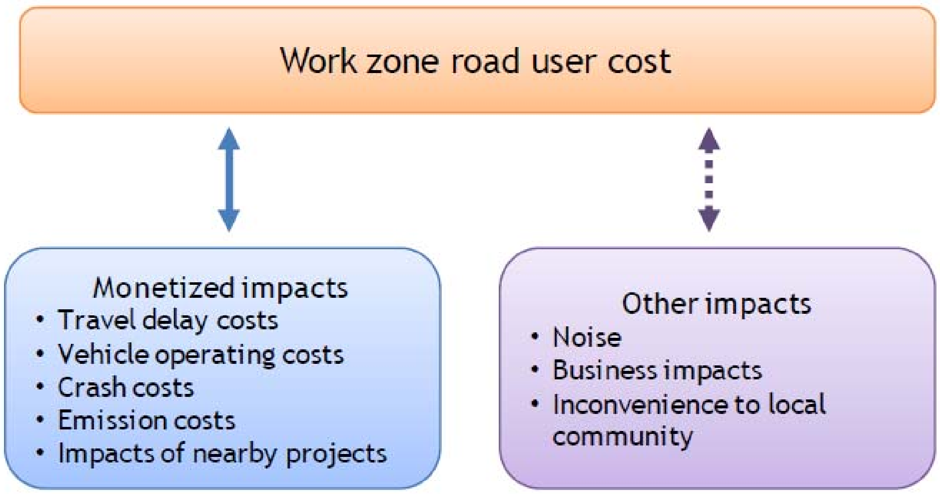

Figure 12: Illustration of different aspects of the WZ RUC (Mallella and Sadasivam)

The table below illustrates information needed to determine the WZ RUC for I-93 (SB) and Pelham Road. The information was obtained through the NHDOT plans for the construction of the replacement bridge. The table identifies the projected daily commuters, design speeds, and percent trucks which will help put a monetary value on traffic congestion.

Projected Traffic Volumes on Project

	I-93 (SB)	Pelham Rd.
Projected Daily Commuters (2014)	50,500	15,900
Projected Daily Commuters (2025)	58,500	18,900
Percent Trucks	8%	-
Design Speed	70 mph	40 mph

Traffic Plan

There is no general equation that allows a civil engineer to "plug and chug" his way to a complete traffic diversion plan. Each site and situation has too many unique combinations of variables and concerns to make a single approach universal. The Federal Government outlines the regulations for roadway signage and traffic diversion in the *Standard Highway Signs and Markings* publication it produces and updates from time to time. This standard includes many theoretical situations to assist engineers, and lays out the signage, merge and other traffic requirements. It also contains pictures of typical roadway signs and plan view diagrams of signs placement for various examples. The general goal of traffic diversion and signage is to protect the safety and health of both construction workers and roadway users, with the secondary goal of maintaining as much of traffic flow as possible.

An engineer has many examples available to choose from. In general for a major operation such as a bridge replacement, one of two methods will be followed. The first possibility involves the temporary re-routing of traffic. This would be implemented for a longer construction time, and include temporary roads and or bridges. A strategy like this is often used to allow the contractor to build a bridge more slowly in place. This strategy could also include closing some of the lanes in the construction area without adding a diversion, although that can have drastic consequences during peak times. Negatives of diverting or narrowing traffic include increased costs and traffic flow issues in general. Constructing a temporary bridge or detour lanes may not even be possible if there is limited space near to a construction area.

A second method of road construction would involve temporary complete or partial shutdown. These shutdowns are typically short, and last from a few minutes to a weekend. Shutdowns are often used with accelerated bridge construction, where a night or weekend of furious construction can replace years of slight delays. Obviously traffic must be detoured, so shutdowns must occur with advance notice at non-peak times. Traffic would be detoured to other existing roads without any new construction of temporary infrastructure.

Stormwater Management Plan

Stormwater runoff comes from precipitation that picks up pollutants from ground surfaces. These pollutants are then carried into lakes, streams, and other surface waters polluting the surrounding environment. This is a major concern regarding the construction of the bridge I-93 over Pelham Road because Porcupine Brook, a designated Salem Prime Wetland passes through the construction site. Construction sites are also exceptionally vulnerable to releasing pollutants during rainfalls since the soil is loose from excavation and the site lacks vegetation to absorb and stabilize pollutants. Some of the pollutants most commonly found near highways and construction sites are nutrients such as nitrogen and phosphorous, sediment, pesticides and fertilizers, petroleum and other chemicals (“What is Nonpoint Source Pollution?”).

Sites below 10 acres are not monitored for nonpoint pollution, but must follow Best Management Practices (BMPs) specified by the Environmental Protection Agency (EPA) if the site is greater than 1 acre (Construction Site Stormwater Runoff Control). Some of the BMPs include properly storing materials such as oil, paint and gasoline to limit the repercussions caused by spilled chemicals. Other solid and liquid waste also must be contained and disposed of properly. A stormwater management plan must be devised and storm drain inlets must be installed before construction. It is strongly encouraged to use some type of storm drain inlet protection. One option would be to surround or cover storm drains with a material that would allow the sediment to be filtered out before the water leaves the construction site. Some commonly used types of storm drain inlet protection are surrounding the inlets with silt fences, sand bags or gravel. It is also important to stabilize the entrances and exits of the construction site. Sediment and chemicals are often tracked into and out of the construction site on the tires of cars and trucks. This pollution can be reduced by having the entrance and exit ways made of rock or gravel. All loose soil must be covered to reduce the amount of sediment washed away during storms.

Accelerated Bridge Construction (ABC) can help reduce the amount of nonpoint pollution by reducing the time of construction, therefore limiting the amount of time loose soil is left exposed and vulnerable to being washed away. The shortened construction time will also decrease the chances of rain storms occurring during construction.

Capstone Design

The project design component will consist of several phases of bridge development. Preliminary Load Resistance Factor Designs will be developed in both steel and precast pretension concrete. These designs will be evaluated and the best type of superstructure chosen. A final LRFD design will then be developed for the bridge superstructure. Using the superstructure results, a design using Terzaghi 's and Schmertmann's methods for the substructure will be developed. A final pavement design will be developed according to the AASHTO Design Procedure for Flexible and Rigid Pavements. Traffic diversion and safety plans will be selected by examining past successes and picking the best available options.

There are multiple real world constraints and factors which will guide the project. One constraint is the constructability of the bridge. It is extremely important for the bridge to be constructed as quickly as possible and for a traffic plan to be developed to minimize traffic delays, but the members must be attainable and methods used must be realistic. Standard members will be used as often as possible and if specialized members are needed, they will be checked to confirm their availability. A construction plan will be developed which will display the most efficient method of construction. Finally previously successful traffic plans will set precedent for diverting the traffic and other innovative techniques will be explored.

Another constraint is the pollution and flooding caused by the construction of the bridge. A stormwater management plan will be developed by using as much of the Best Management Practices recommended by the EPA as possible for the pollution aspect. The *Standard Specifications for Road and Bridge Construction* will be followed to ensure adequate roadway drainage.

In 2009 there were almost 11 million accidents and 36 thousand deaths in the Unites States alone (U.S. Census Bureau). It is important to build a safe bridge for both the commuters and community. Bridge geometry, drainage, wear surface and the placement of traffic control devices all affect roadway safety. These factors are important in temporary traffic diversion, construction, and in the final product.

Cost is an extremely important constraint for the project. This constraint will be addressed through the material and construction costs of the alternative designs. Along with the direct cost, other costs such as the Work Zone Road User Cost and a life-cycle cost analysis of the selected will be investigated.

Economic

A very important aspect of any engineering design is the economics. Economics is often the driving force behind projects and without the proper funding a project may never have the chance to be completed. This is especially true of the large quantities of materials need for highway and highway bridge construction. The cost for materials and construction need to be taken into consideration in every project and minimized whenever needed. This project will address economic issues by conducting a life cycle cost analysis on each of our designs, taking into consideration the cost of the materials and capital. The process of selecting a design type will also weigh heavily the economic effect of traffic delays due to construction and the cost of traffic control itself.

Environmental

Every project must take the environmental impacts into consideration. It is important to evaluate the impacts at every phase of the project especially construction. Non-point pollution is the nations' leading cause of pollution and it is particularly hard to prevent (Koumbaros, Gagnon, and Abdelfattah). It is imperative to follow regulations laid out by the Environmental Protection Agency (EPA) to try and minimize pollution and adverse environmental side effects to the surrounding area. In this project, a positive benefit of accelerated construction method is decreased environmental impact. If the duration of the project is minimized, loose sediment and other excavation related pollutants will have less opportunity to wash into the environment. If the construction is shorter the chance of rain during construction is decreased, with a direct correlation to the amount of non-point pollution. A stormwater management plan will be created to insure environmental compliance.

Manufacturability (Constructability)

Manufacturability is an important aspect of highway bridge construction. Most civil projects tend to be more unique one-time projects, whereas there are hundreds of very similar bridges to ours in New England alone. It is therefore important that our process is a simple and capable of being standardized. States are looking for the fastest and most economical way to replace the multitude of highway bridges, so any process that can be easily replicated will be favored. Our bridge and other bridges similar are not signature spans, it is much more important to construct their replacements as quickly and for as little as possible.

A second concern of manufacturability for this project is the entire concept of accelerated bridge construction. The main tenet of accelerated bridge is to do most of the work out of line or in a factory far away, while not impacting traffic. The components are then assembled in place as quickly as possible, only closing or delaying traffic for days or hours. Therefore the style of bridge chosen must be easily assembled or placed in an even shorter time period. There is a definite emphasis on manufacturability in this project.

Ethical/Health and Safety

This bridge will be designed according to common practice, and according to all applicable specifications and code documents. The design will make every attempt to safeguard both workers and the general public during bridge construction, through allowances in design. Finally, to the best of abilities, and according to state and EPA guidelines, the design will safeguard the community against negative health and environmental aspects of the construction process.

Social and Political

This bridge is on a major interstate highway. It is used frequently by nearly every southern NH resident, and most citizens of New England will cross it at some point. It is for public use, and anyone with a license may drive on it. Flat out, it is part of a major artery of travel, and few would disagree that maintaining a workable bridge should be a priority (Rebuilding93). It is not a project that would disproportionately benefit one community or income group. This bridge is also not a new construction, and it would be an improvement to the existing structure. There is little need to displace new landowners or destroy environmental resources to complete this project. Finally, the bridge is not so expensive as to require political favors or special voting initiatives to construct.

Methodology

This project is complex, with many phases. In each phase, from choosing the bridge design to pursue to the final deliverables, this project will attempt to use appropriate, reasonable methods of design that align with standard engineering practice. For structural calculations, will we use Load Resistance Factor Design (LRFD). The structural and highway designs will be compliant to both state and AASHTO specifications for highway bridges. The bridge will be designed by modern, standard methods.

Acquire Site and Background Information

State plans for the ongoing replacement of the I-93 SB bridge over Pelham road have been acquired (NHDOT Construction Plans). These plans not only contain the state's design for a replacement bridge but also a significant quantity of site, soil, traffic, and profile data. Most of the site data will come from these plans, as they have very recently been assembled, and represent a trusted source. Information about the condition of the current bridge has been acquired from an onsite visit, and many pictures are retained for future consultation. Information on removal and original design conditions come from a scan of the archived original bridge plans for the span constructed finishing in 1961. Information regarding design specifications and state specific requirements is available on the NH DOT website and will be acquired as necessary.

General site information will be collected using satellite imaging from Google maps. Site visits will provide any additional data required that is not contained within the state plans. If additional technical data is needed, the NH DOT will be contacted.

Choose Bridges to Pursue for Preliminary Design

Steel Bridge

For initial design of our steel bridge, the two most common forms will be considered. These are rolled steel W sections, and steel plate girders. Using simple calculations in a spreadsheet to meet simple load requirements in the ASSHTO 2012 bridge code, a beam will be sized by LRFD to meet flexure and deflection requirements when acting compositely with a deck of constant thickness. This deck thickness will be determined by industry standard for purpose of preliminary design. Girder spacing will also be set at a uniform, common distance to reduce possible designs. The total length to be spanned will be 167 feet.

This simple beam model will be used to determine the required steel section to meet flexure and deflection with the minimum number of spans. Beams will be analyzed as simple on all supports. Three span situations will be analyzed: a single span bridge, a two span bridge with a central pier, and a three span bridge with piers at each abutment similar to the existing bridge. First, the minimum W-section that can meet the requirements for the three bridge alignments will be selected. Then, using the weights of these W sections, the required depth of plate girders for sections of two thirds and one half the weight of the W sections will be determine.

A total of nine designs will be created and they will be evaluated for material cost using common rolled and plate girder unit costs. They will be evaluated by the availability of beams of the required sizes. The designs will be evaluated for clearance, including deflection over Pelham Road. This clearance will be measured including deck thickness from the top of the current interstate to the traffic box on Pelham road. An expected concrete cost for additional supports using a unit cost for a standard pier will be added to the cost estimate. Splices will be considered to add additional cost. Finally, a degree of judgment will be exercised to assess effect of additional spans and general practicality. The winning design will be developed into a full preliminary design.

Precast Bridge

Availability and spans dictate that the most reasonable precast shapes are the bulb tee, box girder or single channel (PCI Manual). Precast generally cannot be spliced, therefore the total length of a precast span is limited to what can be produced and shipped in one length. The general maximum varies by type, but it is no more than 80'-120' for a standard project. This means that any precast bridge evaluated will have at least two spans to meet the required 167' span. Therefore, the design will look at either a two or three span precast bridges. Using the preliminary design tables in the *PCI manual*, a sample beam will be sized for each of the three types. These beams will then be evaluated for expected cost based on unit costs, and clearance requirements. An expected concrete cost for additional supports using a unit cost for a standard pier will be added to the cost estimate. The winning precast beam type and span alignment will most economically meet all requirements, and the winning design will be developed into a full preliminary design.

Develop the Steel Preliminary Design

The preliminary steel bridge design will expand on the most promising beam and span combination from the initial design studies. Exact abutment heights, skew and clearance will be determined from site data contained in the official State plans. Design loads will come from the AASHTO *2012 LRFD Bridge Code*. This data will be combined with the known bridge requirements from the state plans. Together the data provides everything needed for entry into preliminary design software.

The software that will be used to complete the preliminary bridge design by LRFD method is known as *Simon LRFD*. It is free software produced by the National Steel Bridge Alliance. Its intended purpose is the preliminary design of steel bridges. The outputs from this software will provide the basis for the preliminary steel design.

Develop the Precast Preliminary Design

The precast design will again begin with the best concrete design from the pre-preliminary calculations above. Using methods from the *PCI manual* and loadings and specifications from the *AASHTO 2012 LRFD Bridge Code*, we will develop and size a precast design. Calculations will take place in spreadsheets created by the project. Beams will be designed on a typical section basis. It is anticipated that the preliminary precast design will be more intensive than the software based steel preliminary design.

Select a Final Design

There are a number of factors that will determine the whether the final design will be a precast or steel bridge. One of the major factors will be the cost of the bridge. When determining the cost of both the steel and precast bridge, the materials must be listed out and quantified. Once the materials and their quantities are determined, the quantities will be multiplied by their respective average unit price and summed together to obtain the total construction cost. The average unit prices will be obtained from the document named “Weighted Average Unit Prices” found on the NHDOT website. The average lifespan and maintenance costs of each type of bridge will also be considered during the selection of the final design.

The construction methods associated with both steel and precast will also play a role when determining which type of superstructure to analyze in the final design. It is important to keep the duration of construction to a minimum given the importance of the bridge, so both the precast and steel bridge will be evaluated based on constructability and project duration. With the durations of an average steel and precast bridge project, a monetary value will be assigned for the construction of each type of superstructure. This method was previously displayed in the Cost of Delays section in the background. The monetary value will be the “B” component of “A+B” cost estimation and will also be called the Work Zone Road User Coast (WZ RUC). The WZ RUC will be calculated by first estimating a unit cost of depreciation in dollars per hour ($/hr) for cars and trucks, then the total work zone delay time will be determined for each construction method investigated. Once these values are estimated, the work zone delay time will be multiplied by the unit cost and number of vehicles affected to obtain the WZ RUC.

Develop a Traffic Plan

A traffic plan will be developed with the intention of maintaining full normal capacity on both the interstate and Pelham road for as much of the time as possible. This strategy is heavily dependent on the construction type and method chosen for the final design. Once the final design is selected the traffic plan will be developed. Typical federal guidelines for traffic control and signage will be followed. It is assumed that capacity would be met if there were at least two lanes capable of traveling sixty five on the interstate, and the normal three lanes operating on Pelham Rd. Previous traffic plans will be analyzed in order to gain perspective and insight into the key features of previously successful traffic plans. The traffic plans will be in accordance with the regulations set forth by the Federal Highway Association to ensure the safety of the workers and motorists.

The safety of the commuters on and below the bridge will be considered during the design and construction plan. Railings, guard rails, and Jersey Barriers will meet the criteria and regulations set forth by the Federal Highway Administration (FHWA).

Develop an Environmental Management Plan

The environmental management plan will be developed by using as many construction Best Management Practices (BMP’s) as possible. The BMP’s have been described by the EPA and are described on the EPA website. The environmental management plan will heavily rely on the construction site and the construction process chosen.

Create Deliverables

The construction plan will be supplemented by a Critical Path Diagram created in *Primavera* along with a phased *Revit* model. The *Primavera* file will outline the critical path and schedule for the project. The *Revit* model will be capable providing a realistic rendered visual for what the finished design will look like and will also be capable of displaying the project in the different phases of construction. The traffic plan document will display a plan view showing how traffic will be diverted during different phases of construction.

Evaluate Final Design

The estimated cost for the designed bridge will be developed using the types and quantities of materials specified in the design, and the weighted average unit prices of the materials for 2012 which can be found in the NHDOT document "Weighed Average Unit Prices". With the quantities and weighted averages a good estimate of what the designed bridge would cost will be calculated by the summation of the quantities times their respective weighted average prices for the year 2012. This bid price along with the projected duration of the developed design will then compared to the winning bid to the state's design.

The estimated additional costs caused by traffic delays will be estimated by determining the Work Zone Road User Cost (WZ RUC). The WZ RUC will be calculated by first estimating a unit cost of depreciation in dollars per hour ($/hr) for cars and trucks, then the total work zone delay time will be determined for each construction method investigated. Once these values are estimated, the work zone delay time will be multiplied by the unit cost and number of vehicles affected to obtain the WZ RUC.

Work Schedule

Bold Group Member is leader of task

	Milestones
<u>Acquire Site and Background Information</u> Gagnon Karolicki **Nitso**	<u>Milestones:</u> Completed

	Milestones
<u>Steel Design Study</u> **Karolicki** Gagnon	<u>Milestones:</u> Results by 10-28-12

	Milestones
<u>Precast Design Study</u> **Nitso** Gagnon	<u>Milestones:</u> Results by 10-28-12

	Milestones
<u>Steel Preliminary Design</u> **Karolicki** Gagnon	<u>Milestones</u> SIMON MODEL by 11-2-12 Final design quantities, cost estimate by 11-9-12

	Milestones
<u>Precast Preliminary Design</u> **Nitso** Gagnon	<u>Milestones</u> Find and develop MODEL by 11-2-12 Final design quantities, cost estimate by 11-9-12

	Milestones
<u>Evaluation of Preliminary Designs</u> **Gagnon** Karolicki	<u>Milestones</u> Develop full cost of construction for both types Pick a final bridge type by End of B term

Nitso

	Milestones
<u>Final Superstructure Design</u> **Karolicki or Nitso** Gagnon	<u>Milestones</u> Support Members Designed Deck Designed

	Milestones
<u>Substructure Design</u> **Gagnon**	<u>Milestones</u> Abutments and Piers Designed Foundations Designed

Pavement Design	Milestones
Nitso	Bridge pavement designed Roadway pavement designed Drainage requirements determined

Traffic Plan	Milestones
Karolicki	General type of plan determined
Nitso	Final plan with timeline/visual Determine final impact/cost of plan

Construction Plan (Primavera)	Milestones
	Primavera Schedule
Gagnon	Complete
Karolicki	Expected Start Date
Nitso	Expected Completion Date

Environmental Plan	Milestones
Gagnon	List of applicable BMPs
Karolicki	

Final Evaluation	Milestones
Gagnon	Full cost considering construction method Determine which is better option (real vs. ours)

DELIVERABLES: Collaborative
Final Report
Presentation
Phased Revit Model

Conclusion

This project will set out to develop a functional and constructible bridge design with the goal of achieving significant saving over traditional design in aggregate cost and an accelerated erection schedule. It will be successful if the final design meets these characteristics.

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Appendix 2.1: Load Results

This appendix contains the program results that were used for preliminary design.

RISA Load Results

This section contains various maximum moments calculated for various framing during initial load study.

Service Load Results

This section contains moment and shear results calculated for preliminary design according to AASHTO loading conditions, tabulated in a simplified manor.

RISA Preliminary Moment Results

One Span

Max Positive Moment:	5100	ft*kips

Two Span Simple

Max Positive Moment:	1850	ft*kips

Two Span Continous

Max Negative Moment:	1500	ft*kips
Max Positive Moment:	1150	ft*kips

HS20-44 (Service)			
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4212.4	
Continuous	74.1	893.4	1248.2
Tandem (Service)			
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.3	1040.2

* $-M_u$ calculated using 90% of loading on structure per section 3.6.1.3 of AASHTO

Appendix 2.2: Load Study

This appendix contains the calculations used to complete the prestressed and steel load studies. This load studies were conducted to determine the best member shapes, end conditions and number of spans to pursue in the preliminary designs.

Precast Design Study

This file contains calculations to determine the best preliminary design option in prestressed concrete. Using information obtained from PCI manual charts, this calculation creates an estimated superstructure cost for all of the prestressed bridge styles analyzed.

Steel Design Study eSpan 75

This file contains a proposed design for a 75 foot span steel bridge, with the project's design criteria, as produced by the eSpan program.

Steel Design Study eSpan 140

This file contains a proposed design for a 140 foot span steel bridge, with the project's design criteria, as produced by the eSpan program. It is only 140 feet instead of 150 because the extreme span limit for the program was 140.

Structural Element	Cross Sectional Area (In^2)	Estimated Strands	Total Steel Per Member (LF)
AASHTO BOX BEAM 48" X 27"	692.5	40	3000
AASHTO BOX BEAM 48" X 33"	752.5	34	2550
AASHTO BOX BEAM 48" X 39"	812.5	38	2850
AASHTO BOX BEAM 48" X 42"	842.5	42	3150
AASHTO BOX BEAM 36" X 27"	560.5	41	3075
AASHTO BOX BEAM 36" X 33"	620.5	36	2700
AASHTO BOX BEAM 36" X 39"	680.5	38	2850
AASHTO BOX BEAM 36" X 42"	710.5	50	3750
DECK BULB TEE 65"	1003	57	8550
DECK BULB TEE 35"	850	35	2625
TYPE IV AASHTO I BEAM	1085	47	7050
TYPE III AASHTO I BEAM	560	28	2100

Cost Numbers		
Concrete	$ 97.57	per CY
Steel Strands	$ 725.75	per Ton
Concrete Class AA Super Structure(Precast, Installed)	$ 600.00	per CY
Concrete Class B (Footings, Substructure), Installed Average	$ 350.00	per CY
Elastomeric Bearings	$ 2,750.00	per ea.

According to' design study, 35" deck bulb Tees are the best option

Total Concrete Per Member (CY)	Number of Members In Bridge Cross Section	Spans	Total Number of Members in Bridge	Total Steel (LF)	Total Steel Cost
13.36	13	2	26	78000	$ 14,718.18
14.52	10	2	20	51000	$ 9,623.43
15.67	8	2	16	45600	$ 8,604.48
16.25	7	2	14	44100	$ 8,321.43
10.81	14	2	28	86100	$ 16,246.61
11.97	11	2	22	59400	$ 11,208.46
13.13	9	2	18	51300	$ 9,680.04
13.71	7	2	14	52500	$ 9,906.47
38.70	14	1	14	119700	$ 22,586.75
16.40	12	2	24	63000	$ 11,887.76
41.86	10	1	10	70500	$ 13,302.97
10.80	8	2	16	33600	$ 6,340.14

Total Concrete (CY)	Total Concrete Cost	Total Material Cost for Super Structure	Requires Topping (or Precast Panel Deck)	Volume of Deck Already Contained(Box Beam CY)
347.3186728	$ 33,887.88	$ 48,606.07	Yes	192.5925926
290.316358	$ 28,326.17	$ 37,949.59	Yes	148.1481481
250.7716049	$ 24,467.79	$ 33,072.26	Yes	118.5185185
227.5270062	$ 22,199.81	$ 30,521.24	Yes	103.7037037
302.7391975	$ 29,538.26	$ 45,784.87	Yes	155.5555556
263.3294753	$ 25,693.06	$ 36,901.52	Yes	122.2222222
236.2847222	$ 23,054.30	$ 32,734.34	Yes	100
191.878858	$ 18,721.62	$ 28,628.09	Yes	77.77777778
541.7438272	$ 52,857.95	$ 75,444.69	No	0
393.5185185	$ 38,395.60	$ 50,283.36	No	0
418.595679	$ 40,842.38	$ 54,145.35	Yes	0
172.8395062	$ 16,863.95	$ 23,204.09	Yes	0

Additional Deck Cost	Number of Required Elastomeric Bearings	Total Bearing Cost	Total Estimated Cost	Piers between Abutments
$ 66,666.67	26	$48,712.82	$ 338,488.87	1
$ 100,000.00	20	$37,471.40	$ 321,284.64	1
$ 122,222.22	16	$29,977.12	$ 311,266.78	1
$ 133,333.33	14	$26,229.98	$ 304,400.95	1
$ 94,444.44	28	$52,459.96	$ 344,794.53	1
$ 119,444.44	22	$41,218.54	$ 329,869.13	1
$ 136,111.11	18	$33,724.26	$ 321,286.24	1
$ 152,777.78	14	$26,229.98	$ 304,041.54	1
$ -	28	$52,459.96	$ 400,093.01	0
$ -	24	$44,965.68	$ 292,964.55	1
$ 211,111.11	20	$37,471.40	$ 513,042.89	0
$ 211,111.11	16	$29,977.12	$ 351,132.08	1

Total Pier Cost	Total Bridge Structure Estimated Cost (Stringers, Deck + Pier)		
$ 101,629.63	$ 440,118.50		
$ 101,629.63	$ 422,914.27		
$ 101,629.63	$ 412,896.41		
$ 101,629.63	$ 406,030.58		
$ 101,629.63	$ 446,424.16		
$ 101,629.63	$ 431,498.76		
$ 101,629.63	$ 422,915.87		
$ 101,629.63	$ 405,671.17		
$ -	$ 400,093.01	Is ths Feasable?	Depth Works
$ 101,629.63	$ 394,594.18		
$ -	$ 513,042.89		
$ 101,629.63	$ 452,761.70		

Steel Bridge Solutions

I-93 over NH-97 with Speed +46 mph

Brian Karolicki

Worcester Polytechnic Institute

11/8/2012 11:10 PM

About Short Span Steel Bridge Alliance

The Short Span Steel Bridge Alliance (SSSBA) is a group of bridge and culvert industry leaders - including steel manufacturers, fabricators, service centers, coaters, researchers, and representatives of related associations and government organizations - who have joined together to provide educational information on the design and construction of short span steel bridges in installations up to 140 feet in length.

For more information about the SSSBA, please contact:

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299 Third Street
Oakland, CA 94607
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Email: dnshaw@placemakinggroup.com

Design Support

The Short Span Steel Bridge Alliance offers complimentary design support for questions relating to bridge and culvert design. Design support is offered by the following organizations (to submit an inquiry, please visit www.ShortSpanSteelBridges.org and click on the "Bridge Technology Center" link on the homepage):

Standard Design and Details of Short Span Bridges (Plate Girder & Rolled Beam Bridges)

The Bridge Technology Center is a complimentary resource available for questions specific to standard design and detail solutions of short span steel bridges (refer to the section of this Solutions Book on plate girder and rolled beam standards, if applicable). It is a resource provided by West Virginia University and the University of Wyoming.

Standard Design and Details of Corrugated Steel Pipe and Structural Plate

The National Corrugated Steel Pipe Association provides complimentary design support for questions pertaining specifically to standard design and detail solutions of corrugated steel pipe and corrugated structural plate (refer to the section of this Solutions Book on corrugated steel pipe and corrugated structural plate standards, if applicable).

Manufactured Steel Solutions

For questions pertaining to a specific manufacturer's solution (refer to section on Manufacturer's Steel Solutions of this Solutions Book), it is recommended that you directly contact the manufacturer by utilizing the contact information listed with the solution.

User Name:	Brian Karolicki
User Company:	Worcester Polytechnic Institute
User Input Date:	11/08/2012
Project Name:	I-93 over NH-97 with Speed +46 mph
City:	Salem
State/Province:	NH
Roadway:	I-93 Southbound
Span Length:	140' 0"
Number of Striped Traffic Lanes:	4
Roadway Width:	72'
Individual Parapet Width:	0'
Individual Deck Overhang Width:	2'
Pedestrian Access:	No
Number of Sidewalks:	Not provided
Total Width of Each Sidewalk:	Not provided
Skew Angle:	degrees
Average Daily Traffic (ADT):	Over 2,000
Design Speed:	46+ mph
Waterway Area:	Not provided
Height of Cover:	Not provided

Disclaimer

Notes

- Short span standards for rolled beam solutions are only available for input lengths between 40 and 100 feet and skew angles under 20 degrees.*
- Short span standards for homogeneous plate girder solutions are only available for input lengths between 60 and 140 feet and skew angles under 20 degrees.*
- Short span standards for hybrid plate girder solutions are only available for input lengths between 80 and 140 feet and skew angles under 20 degrees.*
- Design standards for rolled beam and plate girder solutions are rounded in five (5) foot increments.
- Corrugated steel pipe and structural plate standards are only available for input lengths under 85 feet.*
- Customized prefabricated manufacture solutions are available for all lengths and skew angles.

** For bridges/culverts outside of this range, standard designs will not appear in your solutions book.*

Standard Design and Details of Short Span Steel Bridge Solutions

General

These plans are intended to serve as a guide to state, county, and local highway departments in the development of suitable and economical steel bridge superstructure designs. The plans should be particularly valuable to the smaller highway departments with limited engineering staff.

Specifications

Specifications for design, materials, and construction are included in the following:

- AASHTO LRFD bridge design specifications, fifth edition with 2010 interim revisions. 2010. Adopted and published by the American Association of State Highway and Transportation Officials. Washington, DC
- AASHTO/NSBA Collaboration Standard S2.1. Steel Bridge Fabrication Guide Specifications, 2008. Developed by the AASHTO/NSBA Steel Bridge Collaboration. Washington, DC
- AASHTO/NSBA Collaboration Standard G1.4. Guidelines for design details. 2006. Developed by the AASHTO/NSBA Steel Bridge Collaboration, Washington, DC
- ASTM Standards. Published by the American Society for Testing and Materials. ASTM International, 100 Barr Harbor Drive, P.O. Box C700, West Conshohocken, PA 19428-2959 USA.

Design Loading

AASHTO HL-93 Vehicular Live Loading was used throughout.

Design Method

Load and Resistance Factor Design (LRFD) method was employed throughout. Designs were originated using 5 girders with equal spacing. However, plate sizes and beam selections are adequate for any increment of girder layout. Designs will accommodate skews up to 20° from perpendicular, and are intended to be parallel.

Three options are available for steel superstructure composite I-girders. These options are as follows:

1. Homogenous plate girders comprised of ASTM A709-50W steel. These designs are available for a span range of 60’-140’.
2. Hybrid plate girders comprised of ASTM A709-50W and A709-70W steel. A709-50W steel is utilized for the top flange and web. A709-70W steel is utilized for the bottom flange. These designs are available for a span range of 80’-140’.
3. Rolled beams comprised of ASTM A709-50W steel. These designs are available for a span range of 40’-100’.

Structural Steel

All structural steel shall conform to AASHTO M270 (ASTM A709) grade 50, 50W, or 70W, as applicable. Refer to “Design Method.”

Concrete

Concrete for deck and parapet shall have a minimum 28-day compressive strength (f’c) of 4,000 PSI.

Concrete Deck

The deck thickness employed for design was 8”. This includes a 1/4” integral wearing surface which is not considered part of the structural depth. The owner shall specify the required deck cross slope and grade.

Reinforcing Steel

Reinforcing steel shall conform to ASTM A615 grade 60.

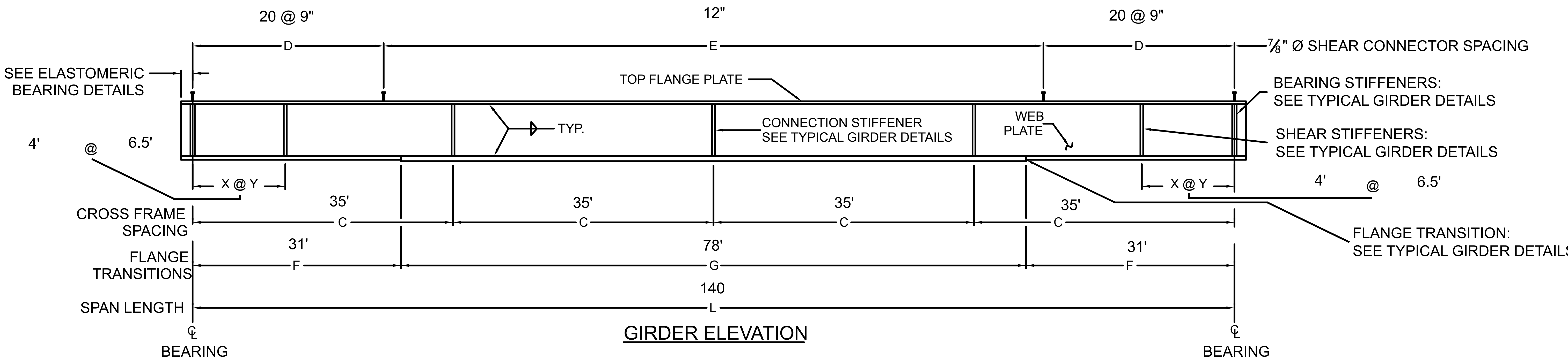
Shear Connectors

Welded stud shear connectors shall conform to the requirements of ASTM A108.

Elastomeric Bearings

See Elastomeric Bearing Details.

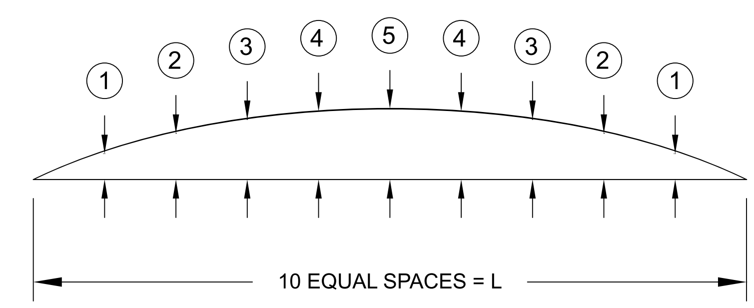
COMPOSITE PLATE GIRDER WITH PARTIALLY STIFFENED WEB - 8 GIRDERS AT 9' 8.571" GIRDER SPACING, HOMOGENEOUS



GIRDER ELEVATION

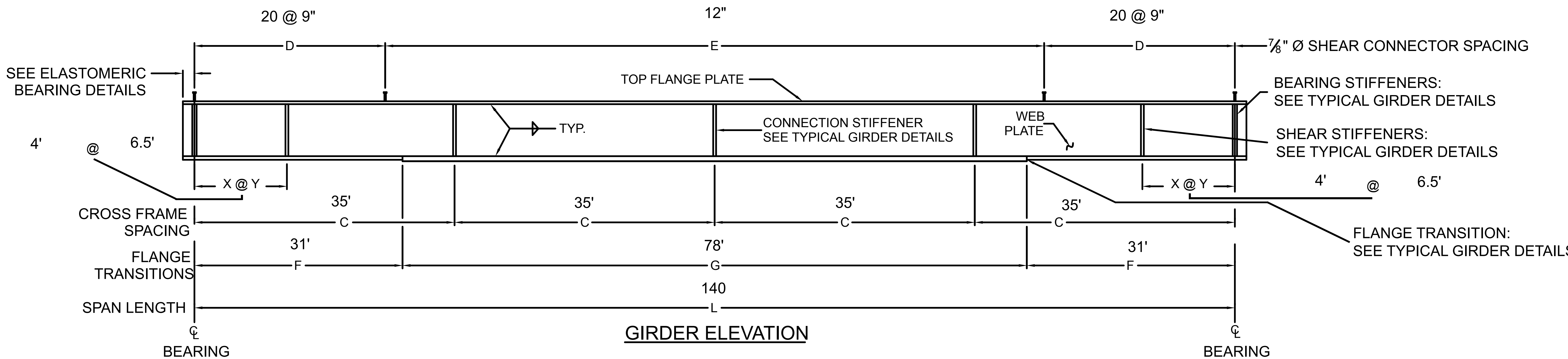
SPAN (L) - ft	PLATE GIRDER SIZE							DIAPHRAGM SPACING (C) - ft	SHEAR STIFFENERS		SHEAR CONNECTOR MAX. SPACING		INDIVIDUAL GIRDER WEIGHT
	TOP FLANGE - in	BOTTOM FLANGE (F)		BOTTOM FLANGE (G)		WEB PLATE- in							
		PLATE - in	LENGTH - Ft	PLATE - in	LENGTH - Ft				X (NO. REQ'd)	Y - ft. (SPACING)	D	E	
140	20 x 1"	20 x 1 1/2"	31'	20 x 2"	78'	54 x 1/2"		35'	4'	6.5'	20 @ 9"	12"	39,336 lbs

STEEL D.L. CAMBER - in					TOTAL D.L. CAMBER - in				
1	2	3	4	5	1	2	3	4	5
0.606"	1.141"	1.555"	1.819"	1.909"	3.428"	6.448"	8.782"	10.266"	10.774"



D.L. CAMBER

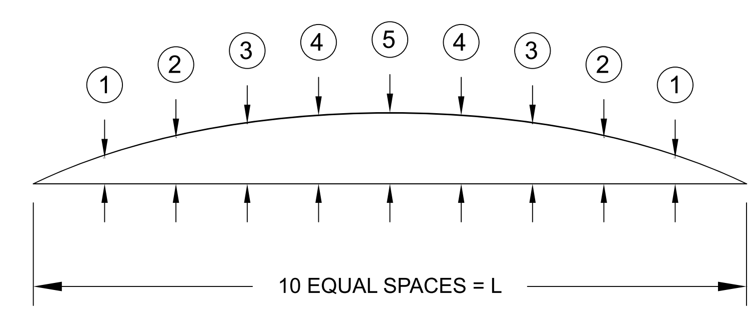
COMPOSITE PLATE GIRDER WITH PARTIALLY STIFFENED WEB - 8 GIRDERS AT 9' 8.571" GIRDER SPACING, HYBRID



GIRDER ELEVATION

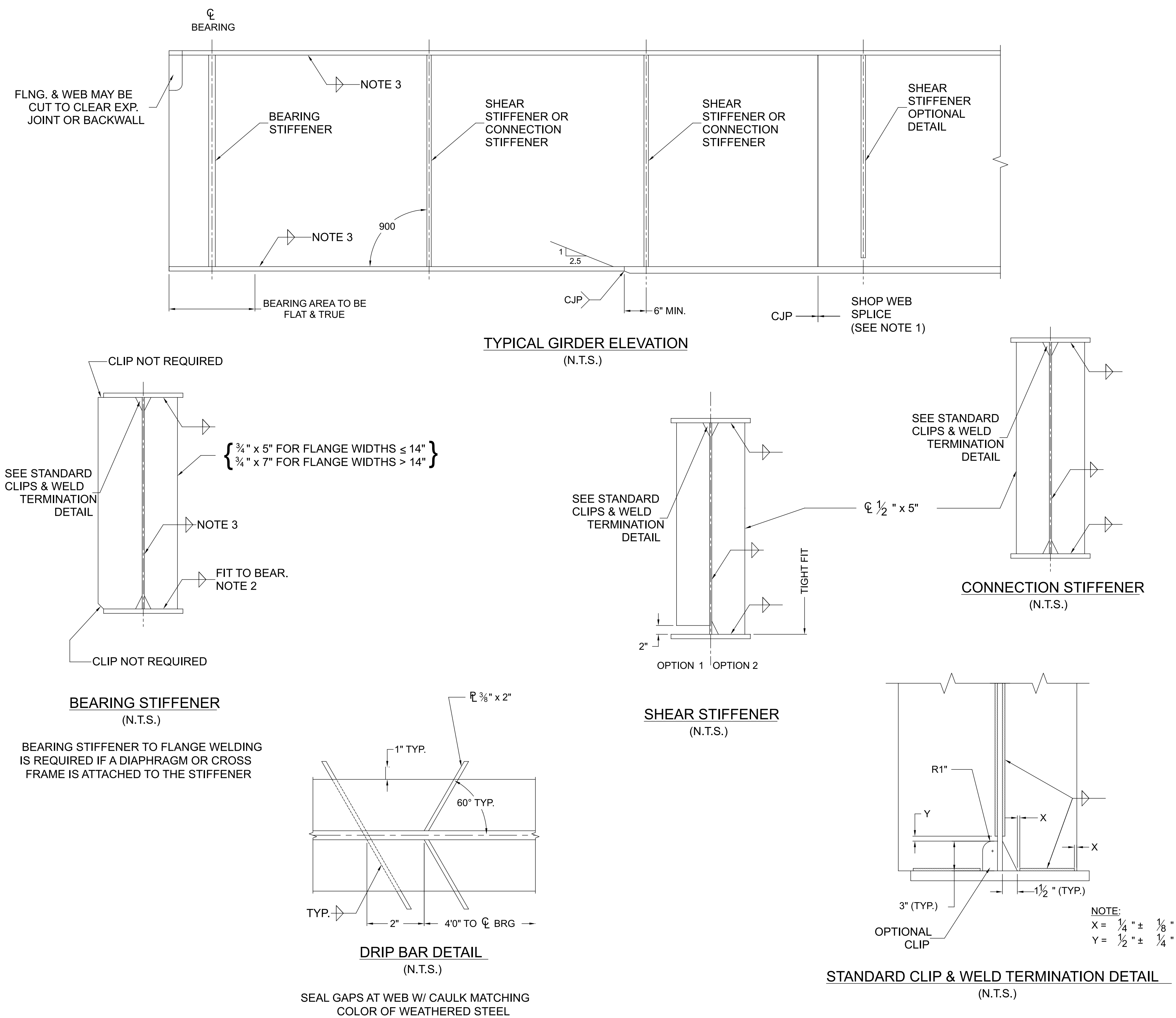
SPAN (L) - ft	PLATE GIRDER SIZE						DIAPHRAGM SPACING (C) - ft	SHEAR STIFFENERS		SHEAR CONNECTOR MAX. SPACING		INDIVIDUAL GIRDER WEIGHT
	TOP FLANGE - in	BOTTOM FLANGE (F)		BOTTOM FLANGE (G)		WEB PLATE- in		X (NO. REQ'd)	Y - ft. (SPACING)	D	E	
		PLATE - in	LENGTH - Ft	PLATE - in	LENGTH - Ft							
140	18 x 1 1/2"	20 x 1"	31'	20 x 2"	78'	54 x 1/2"	35'	4'	6.5'	20 @ 9"	12"	36,689 lbs

STEEL D.L. CAMBER - in					TOTAL D.L. CAMBER - in				
1	2	3	4	5	1	2	3	4	5
0.567"	1.059"	1.432"	1.670"	1.751"	3.136"	5.840"	7.884"	9.182"	9.627"



D.L. CAMBER

Typical Girder Details



TYPICAL GIRDER ELEVATION
(N.T.S.)

CONNECTION STIFFENER
(N.T.S.)

BEARING STIFFENER
(N.T.S.)

BEARING STIFFENER TO FLANGE WELDING IS REQUIRED IF A DIAPHRAGM OR CROSS FRAME IS ATTACHED TO THE STIFFENER

SHEAR STIFFENER
(N.T.S.)

DRIP BAR DETAIL
(N.T.S.)

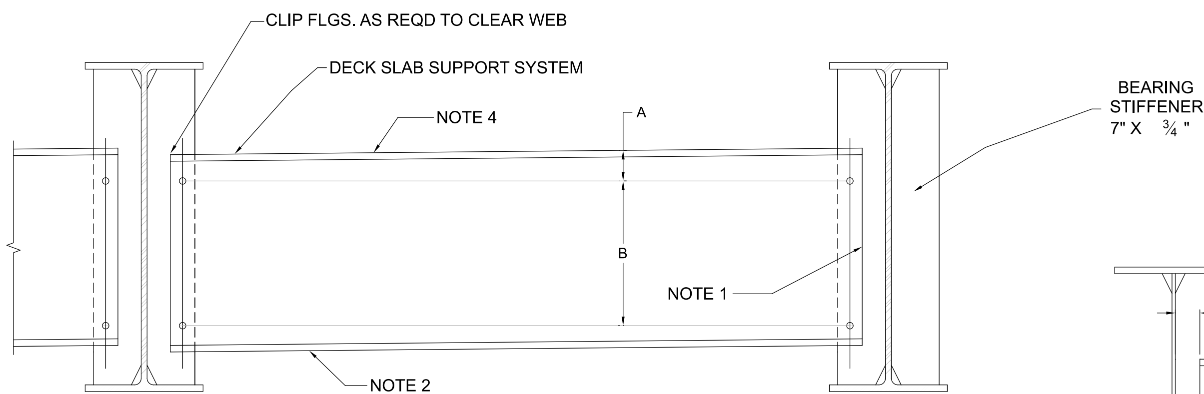
SEAL GAPS AT WEB W/ CAULK MATCHING COLOR OF WEATHERED STEEL

STANDARD CLIP & WELD TERMINATION DETAIL
(N.T.S.)

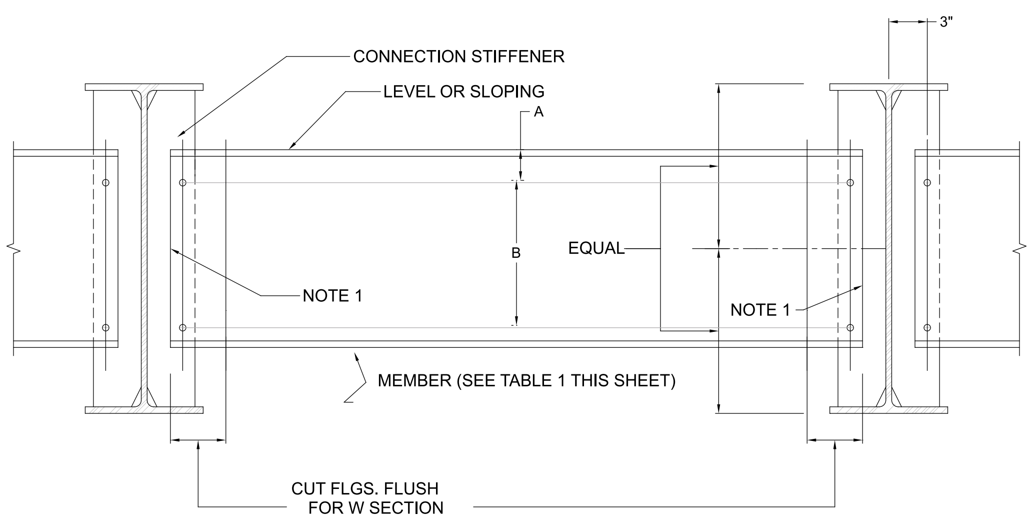
NOTES:

1. All CJP welds to be ground and tested per state specifications.
2. Fit to bearing is to be 50% in contact with flange and within 1/16” for remainder.
3. MT 1’ of every 10’ (extents of mag particle inspection for fillet welds) -OR- see state specs.

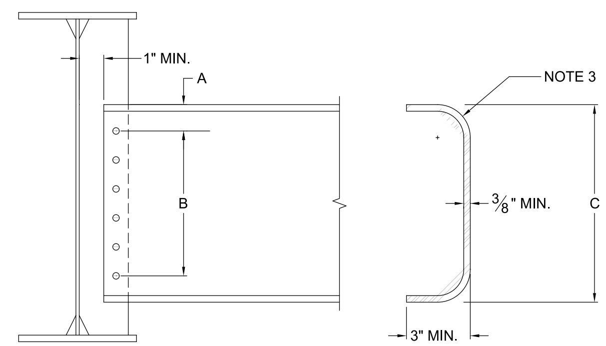
Rolled Shape and Bent Plate Diaphragm Details



END DIAPHRAGM
(N.T.S.)

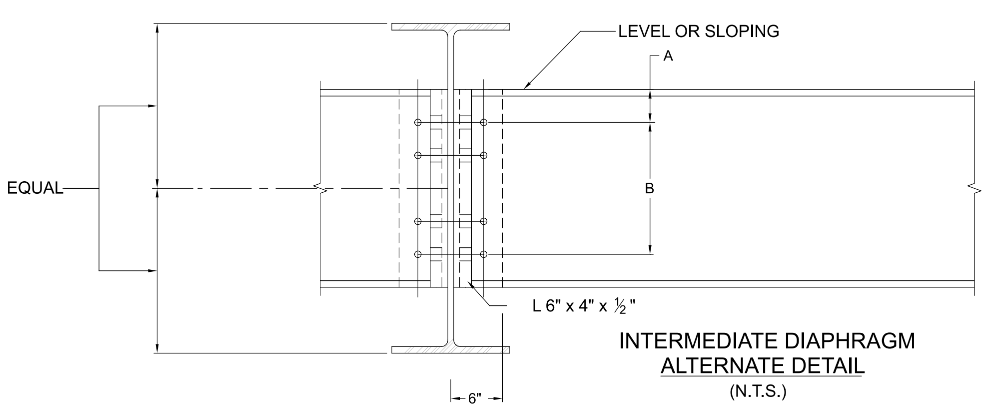


INTERMEDIATE DIAPHRAGM
(N.T.S.)



BENT PLATE DIAPHRAGM
(N.T.S.)

(CAN BE USED AS ALTERNATE TO ROLLED SHAPE DIAPHRAGM)



INTERMEDIATE DIAPHRAGM ALTERNATE DETAIL
(N.T.S.)

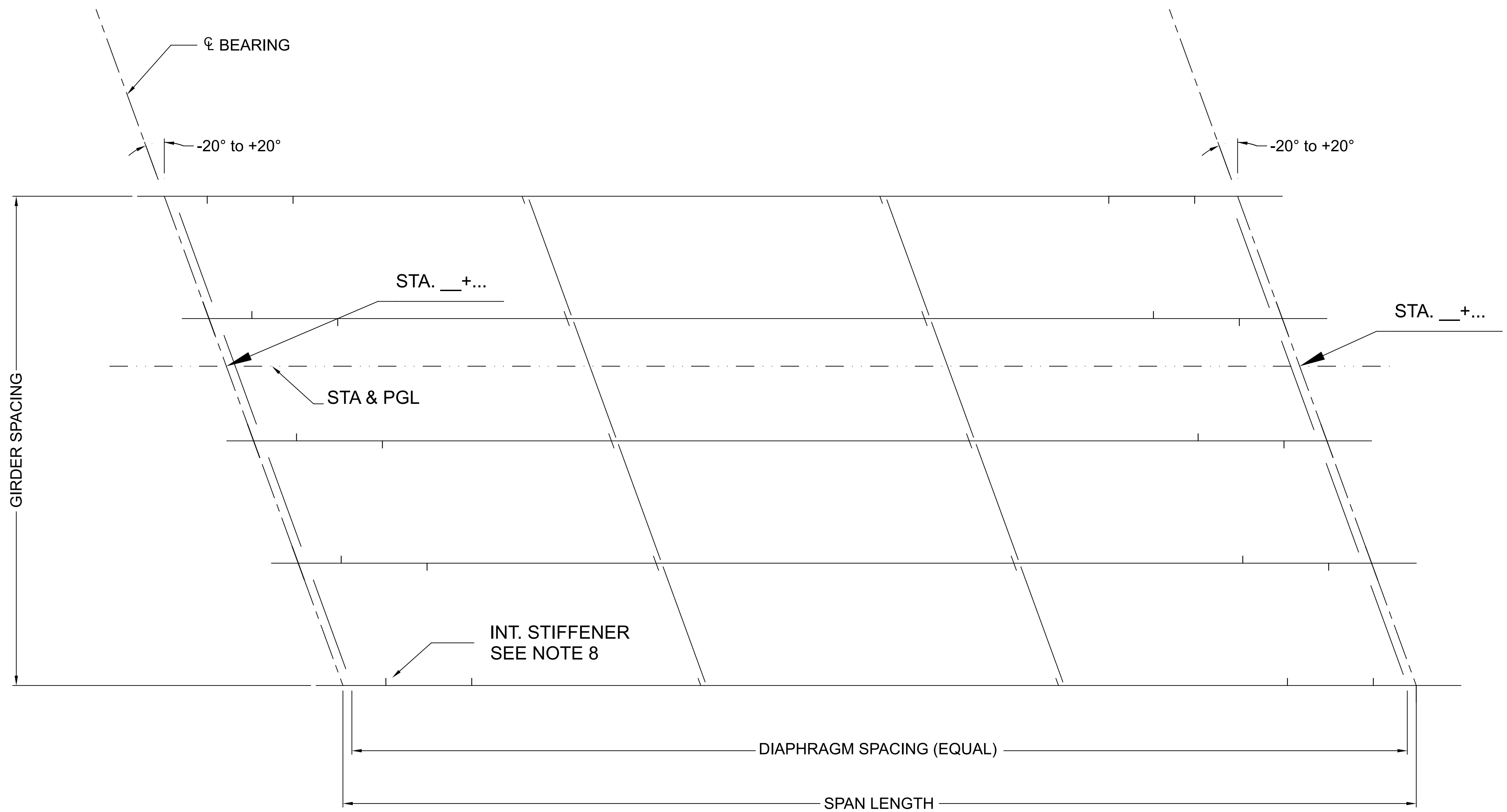
TABLE 1

DEPTH OF STRINGER OR GIRDER WEB	DIAPHRAGM SIZE	DIMENSIONS		
		A	B	C
≤ 30"	C15x33.9	3"	3 @ 3"	15"
30" < X ≤ 36"	MC18x42.7	3"	4 @ 3"	18"
> 36"	W30x90	5"	5 @ 4"	30"

NOTES:

1. Slope diaphragm and keep holes vertical in stiffener at constant dimensions (to keep all stiffeners the same) and cut ends of diaphragm square.
2. At expansion joint, orient channel flanges away from joint opening.
3. Minimum radius as per AASHTO/NSBA fabrication S2.1 table 4.3.2-1. Per section 4.3.2, if the bend is parallel to direction of rolling, multiply the minimum radii by 1.5.
4. All holes to be 15/16" ø for 7/8" ø HS bolts, ASTM A325 type 3 w/ F436-3 washers (RCT).
5. Threads excluded from shear plane.

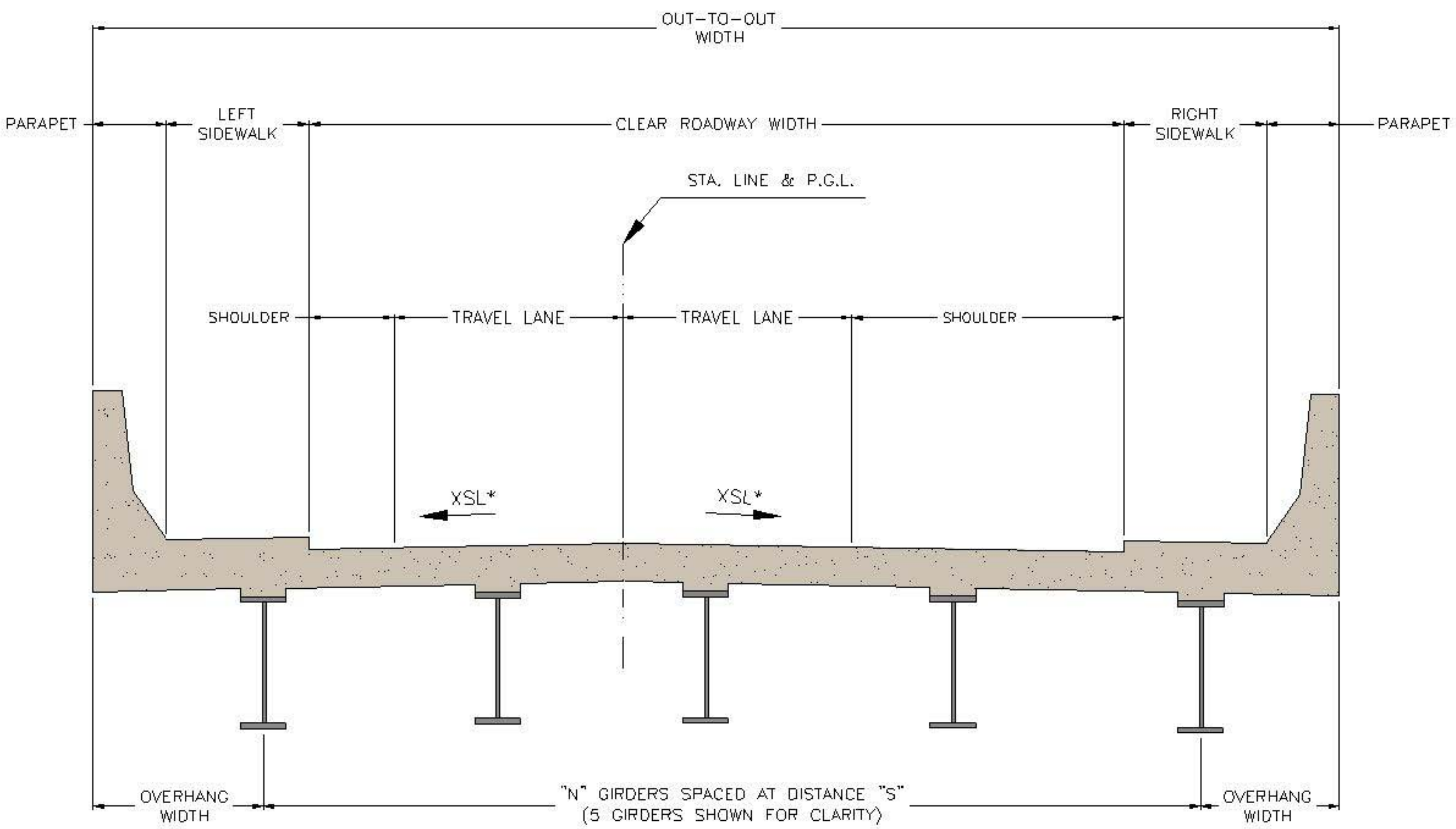
Framing Plan



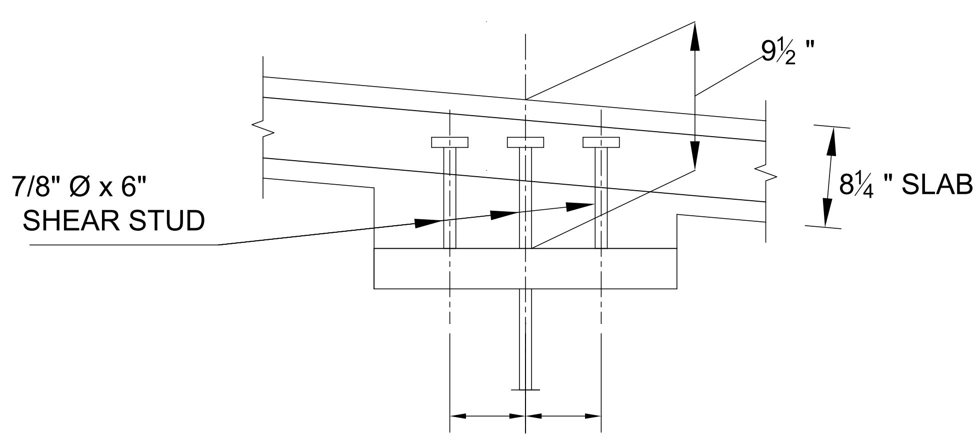
NOTES:

1. Superstructure may sit on existing bridge seats. Contractor to verify spacing in field.
2. Design will accommodate skews up to 20° from ⊥, but are intended to be parallel.
3. Station line is intended to be on a tangent alignment.
4. Max grade at bearing is ± 5%.
5. Orient toes of channel diaphragm down grade.
6. Diaphragms may be placed on either side of connection plate at the contractor's discretion.
7. Keep diaphragm lines parallel to bearing lines.
8. Int. stiffeners are required on one side of web only. On fascia girders, orient stiffeners to the inside of the girder. On interior girders, stiffeners should alternate sides. See Girder Elevations for spacing.

Typical Section



*NOTE: XSL - Cross slope can vary from -.06% to +.06%.

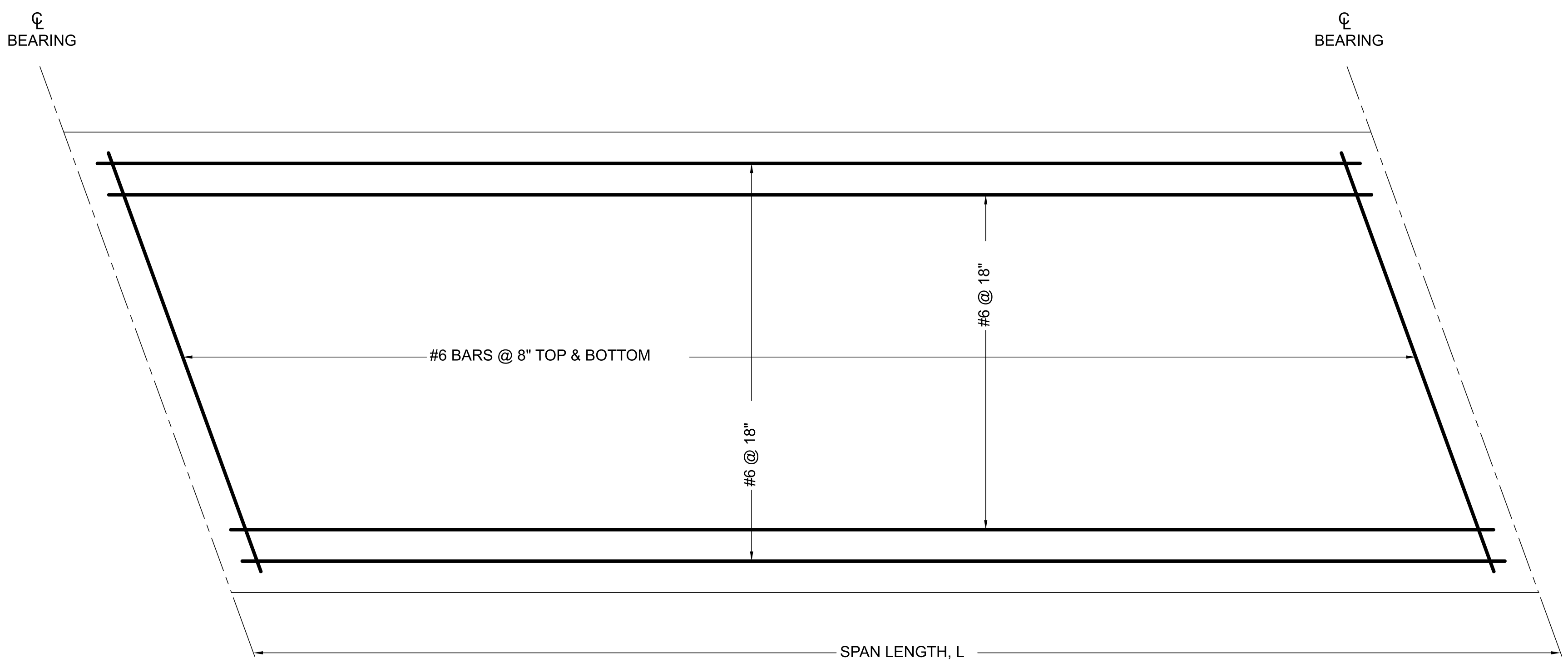


DETAIL A

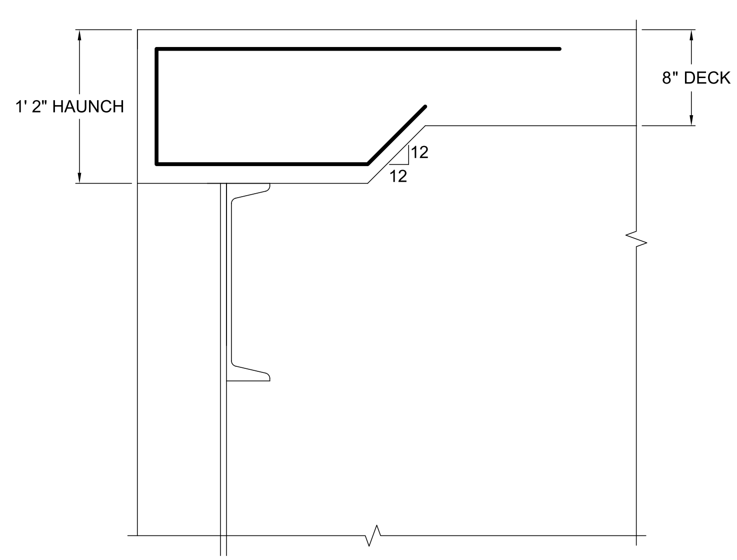
NOTES:

1. For shear stud spacing, see Girder Elevations.
2. Parapets per state DOT requirements, if cast in place, provide 2'-0" lap with transverse bars.

Deck Design



REINFORCING PLAN
(N.T.S.)

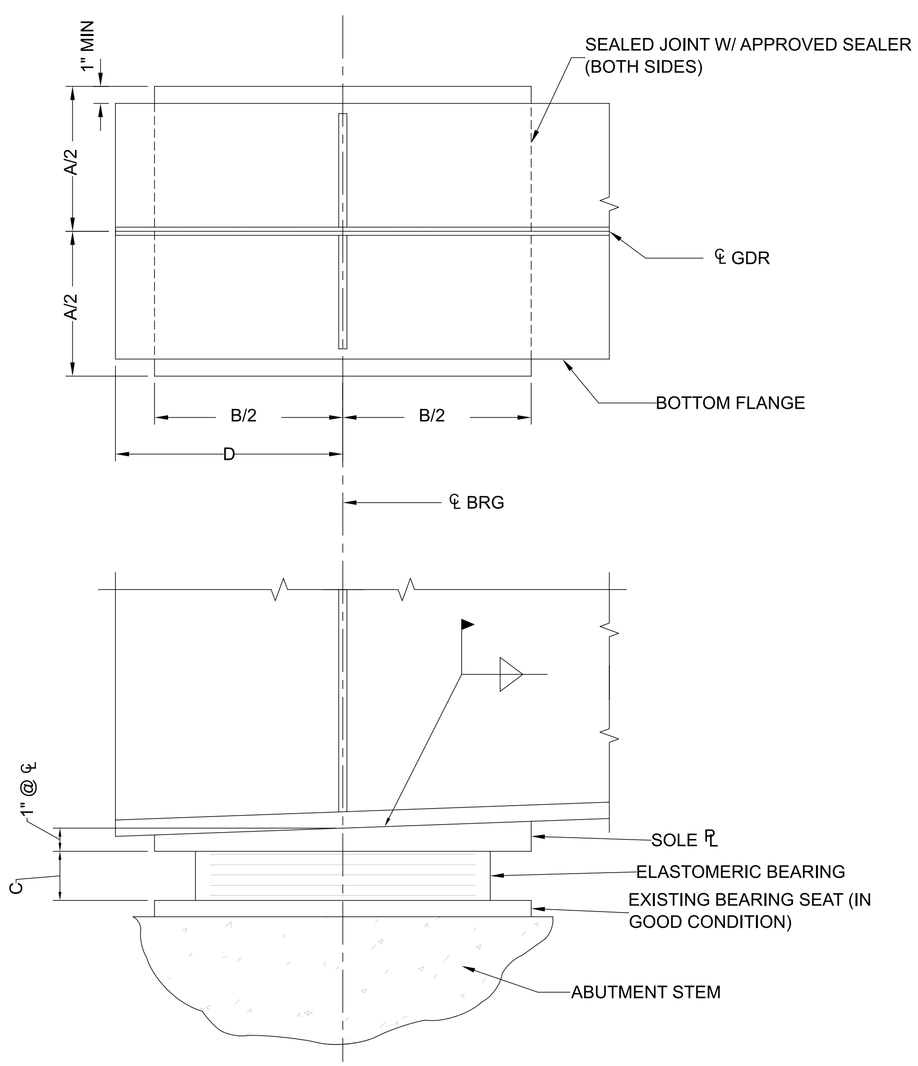


HAUNCH DETAIL
(N.T.S.)

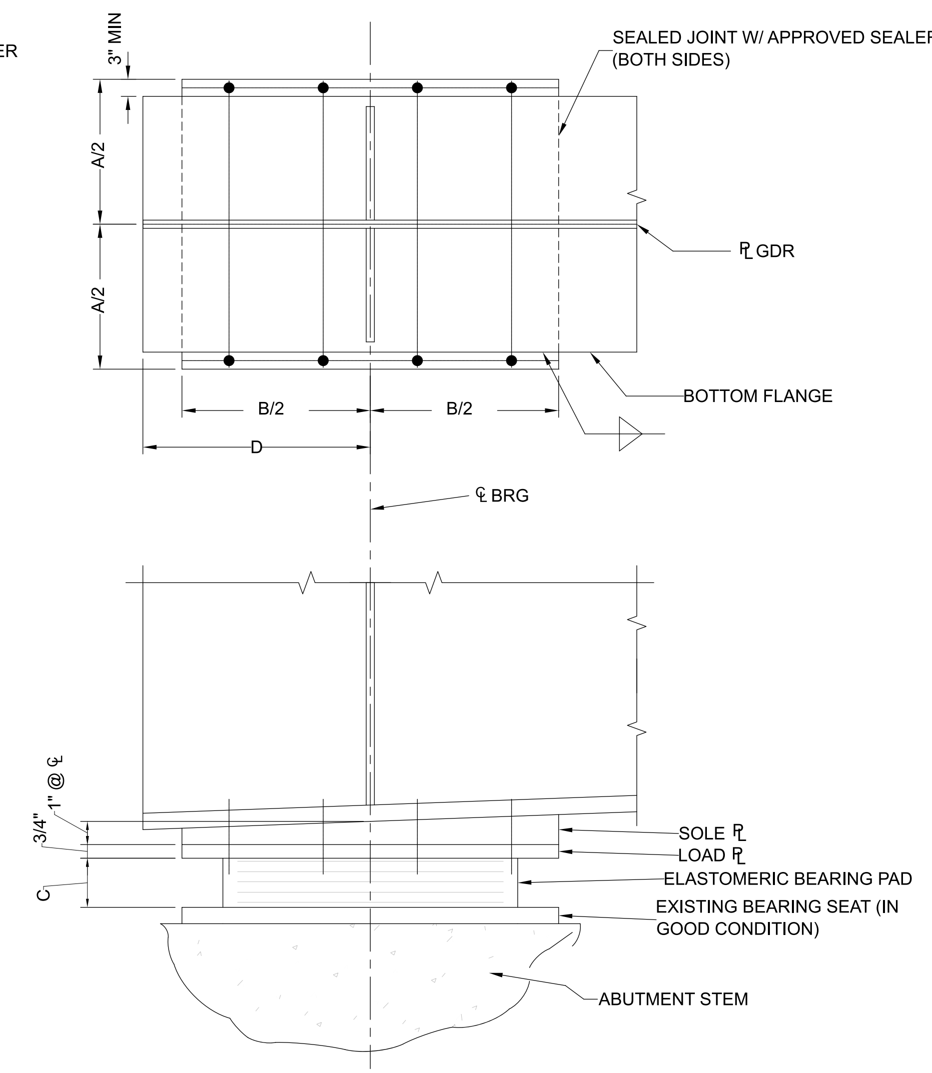
NOTES:

1. Forming brackets must extend to bottom flange.

COMPOSITE PLATE GIRDERS - 9' 8.571" GIRDER SPACING, HOMOGENEOUS



BEARING ELEVATION
OPTION "A"
(N.T.S.)



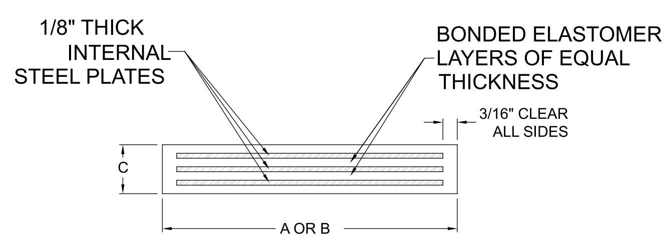
BEARING ELEVATION
OPTION "B"
(N.T.S.)

NOTES:

1. Bevel sole ℙ if grade exceeds ± 1%.
2. Max Grade is ± 5%.
3. Sole ℙ to be factory vulcanized to elastomeric bearing pad.
4. Holes to be 1 1/16" Ø in sole ℙ for 7/8" Ø bolt.
5. All elastomeric cover layers are 1/4" thick.

COMMENTARY:

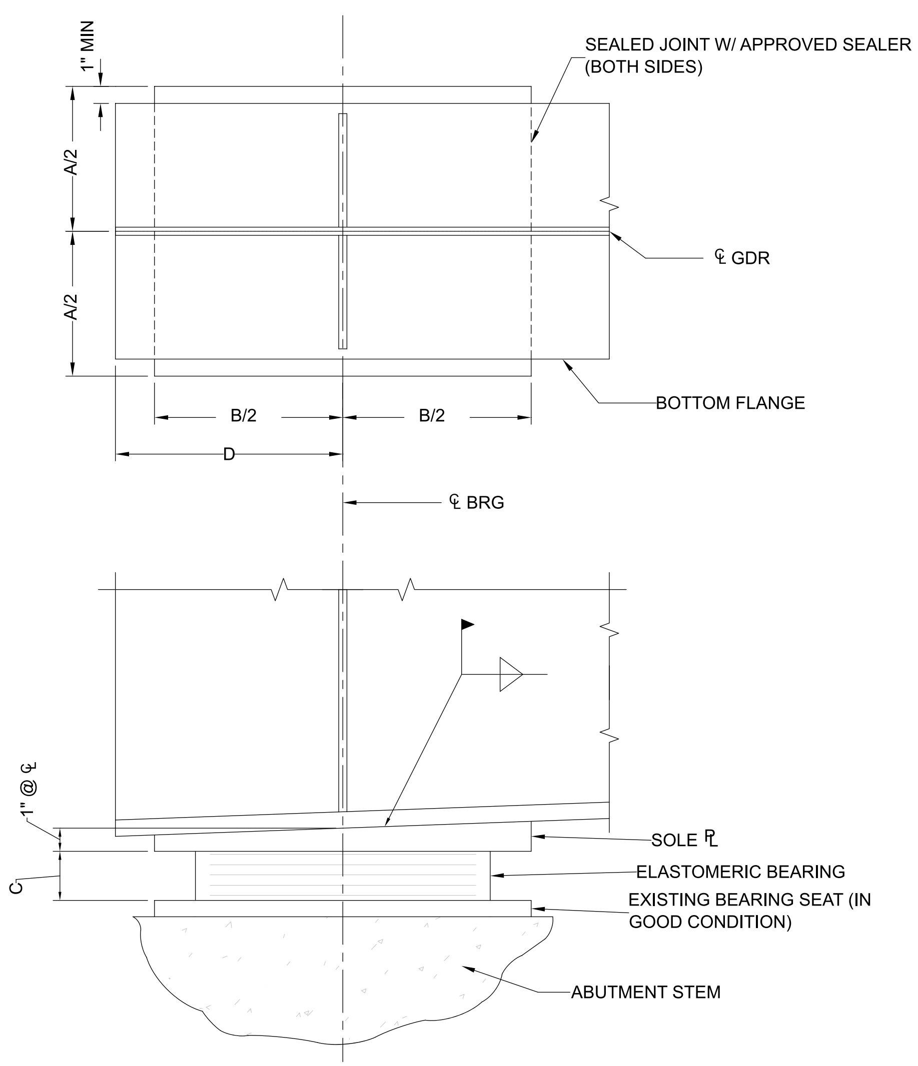
1. Care must be exercised with the field welding. The temperature of the steel adjacent to the bearing must be kept below 250°F (120°C). Temperature crayons should be used to monitor the steel temperature during welding.



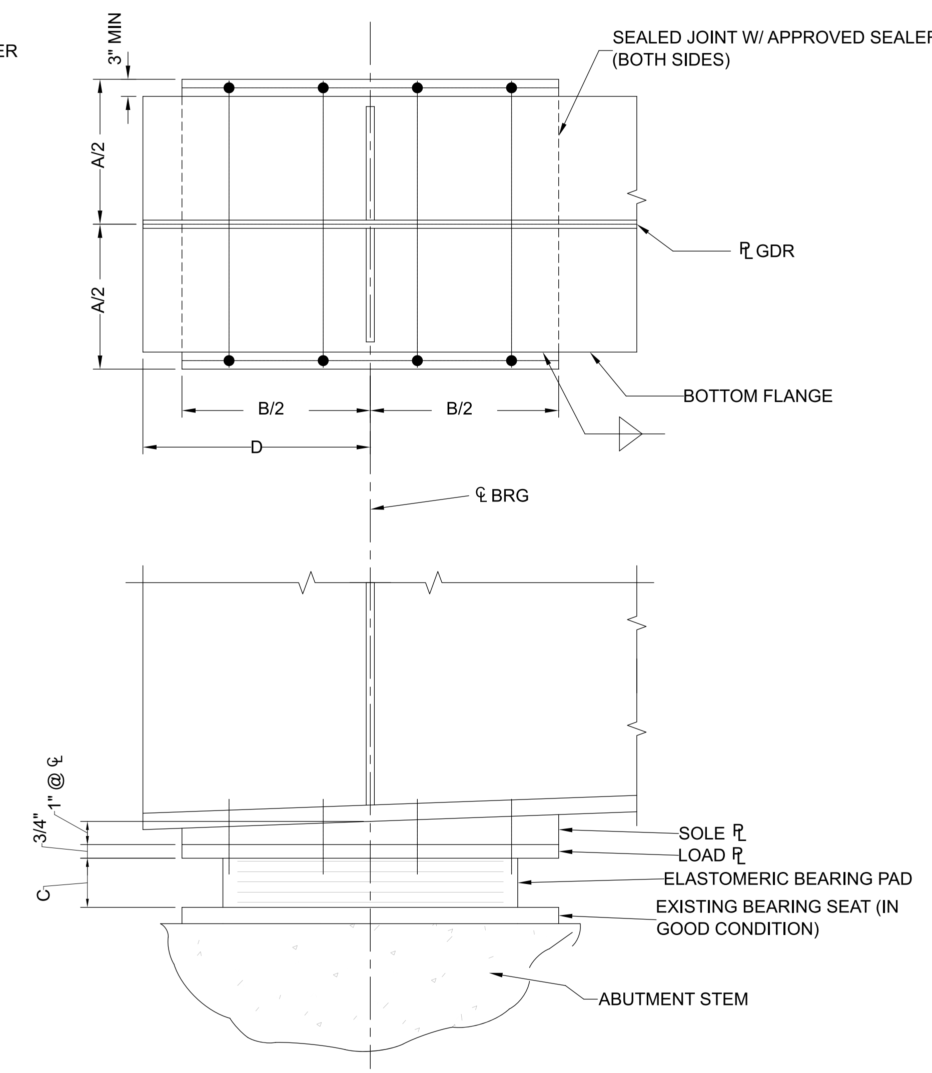
SECTION VIEW OF ELASTOMERIC BEARING
(N.T.S.)

ELASTOMETRIC BEARING DETAILS - in					
A	B	C	D	INTERNAL ELASTOMER LAYERS	
				NO. OF LAYERS	THICKNESS - in
18"	20"	5.125"	12"	6	0.625"

COMPOSITE PLATE GIRDERS - 9' 8.571" GIRDER SPACING, HYBRID



BEARING ELEVATION
OPTION "A"
(N.T.S.)



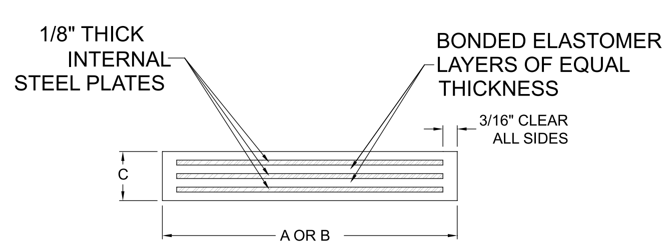
BEARING ELEVATION
OPTION "B"
(N.T.S.)

NOTES:

1. Bevel sole PL if grade exceeds ± 1%.
2. Max Grade is ± 5%.
3. Sole PL to be factory vulcanized to elastomeric bearing pad.
4. Holes to be 1 1/16” Ø in sole PL for 7/8” Ø bolt.
5. All elastomeric cover layers are 1/4” thick.

COMMENTARY:

1. Care must be exercised with the field welding. The temperature of the steel adjacent to the bearing must be kept below 250°F (120°C). Temperature crayons should be used to monitor the steel temperature during welding.



SECTION VIEW OF ELASTOMERIC BEARING
(N.T.S.)

ELASTOMETRIC BEARING DETAILS - in					
A	B	C	D	INTERNAL ELASTOMER LAYERS	
				NO. OF LAYERS	THICKNESS - in
18"	20"	5.875"	12"	7	0.625"

Manufacturer's Steel Solutions - Customized Solutions from Members of the Short Span Steel Bridge Alliance

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Big R Bridge is a world leader in developing innovative engineered solutions in Prefabricated Bridges, Structural Plate, MSE Wall Systems and Corrugated Pipe for the transportation, public works, railway, mining, forestry and development sectors. By design, our custom infrastructure solutions are easy to ship and install with minimal equipment and labor requirements, making them ideal even in remote locations. Big R's Technical Sales Representatives and Engineers are well-positioned to ensure your project's success through every phase. With product innovation, in-house engineering strength

Vehicular Truss Bridges

Vehicular Truss Bridges
Big R's Vehicular Truss Bridges offer prefabricated solutions that are ideal for counties, cities and other government agencies who desire a highly functional vehicular bridge with old style aesthetics and architecture. Vehicular Truss Bridges are engineered to be installed on an accelerated schedule when compared to site built, traditional bridge structures. Add to this the flexibility of multiple decking options, sidewalks and finishes, and you have a highly tailored solution to the most unique bridging needs.

- Spans up to 240' (most economical between 130' and 240')
- Widths up to 36'
- Decking options – poured or precast concrete, asphalt, grating, wood or gravel
- Weathering, galvanized or painted structural steel
- Curb or rail system
- Sidewalks and utility corridors can be added to enhance use

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Vehicular Truss Bridges

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TrueNorth Steel designs reliable steel Structures, Tanks, Corrugated Pipe and Bridges you can count on to always be on time and delivered to the highest quality standards.

TrueNorth Steel Bridges & Corrugated Steel Pipe

Built to AASHTO specifications TrueNorth Steel Bridge provides safe passage for pedestrians and all types of vehicles. With decades of bridge building experience, we've developed a design-build, bolt-together system that blends flexibility with standardization, so we can design a bridge for each unique application, while delivering safety, durability and easy installation. In addition to the bridges we offer pre-engineered and pre-fabricated SuperSill's and Back-Walls to simplify and reduce abutment construction and design costs.

TrueNorth Steel Corrugated Pipe has been a critical part of north America's evolving infrastructure for more than six decades. Our corrugated steel pipe offers tremendous durability and stability for casing, architecture and nearly and drainage and water flow application.

TrueNorthSteel

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TrueNorth Steel designs reliable steel Structures, Tanks, Corrugated Pipe and Bridges you can count on to always be on time and delivered to the highest quality standards.

TrueNorth Steel Bridges & Corrugated Steel Pipe

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Wheeler's Steel Fabrication Division is an extension of the experience gained by 100+ years of designing & supplying bridge materials. We have a staff of Professional Engineers & drafters who provide detailed plans specific to each project. Wheeler maintains AISC certification for Simple and Major Steel Bridges. Prefabricated bridge kits provide rapid construction for recreation & vehicular applications. The bridges are shop manufactured, detailed & shipped to site ready for installation.

Steel Stringer Vehicle Bridges Utilizing Transverse Treated Timber Deck Panels

Treated timber deck panels provide a versatile option as prefabricated bridge components. The deck panels are a good compliment to steel stringer superstructures. The combination results in a complete bridge kit. All components are shop fabricated ready for installation.
The deck panels can be designed for all loading conditions (ie. HS20, HS25, HL93, U80, U102). The panel thickness is based on loading condition and stringer spacing.
The deck panels are custom detailed to the specific application. Individual deck laminae are fabricated and pressure treated before being assembled into the panels. This enhances the long term durability of the deck system. Multiple attachment systems can be used to connect the panels to the steel stringers. As they are installed the panels are interconnected to provide load transfer improving the performance of an asphalt overlay wear surface.
Crash-tested timber railing kits attach directly to the deck panels. Pedestrian railings are available.

Advantages:
Shipped as a kit
Components are largely preassembled and sized for easy handling
Shop fabricated to control quality
Speeds installation at the site
Accepts traffic immediately after installation
Not temperature sensitive, no curing time
Ideal for remote sites
Treatment is water resistant, not susceptible to damage from road salt
Multiple wear surface options including asphalt
Compatible with crash-tested railing system

Wheeler provides complete superstructure plans for all projects supplied. All hardware is included. Foundation designs are available depending on site conditions.

Contact us for project specific pricing and application advice.

Set your project apart with a bridge from Wheeler.

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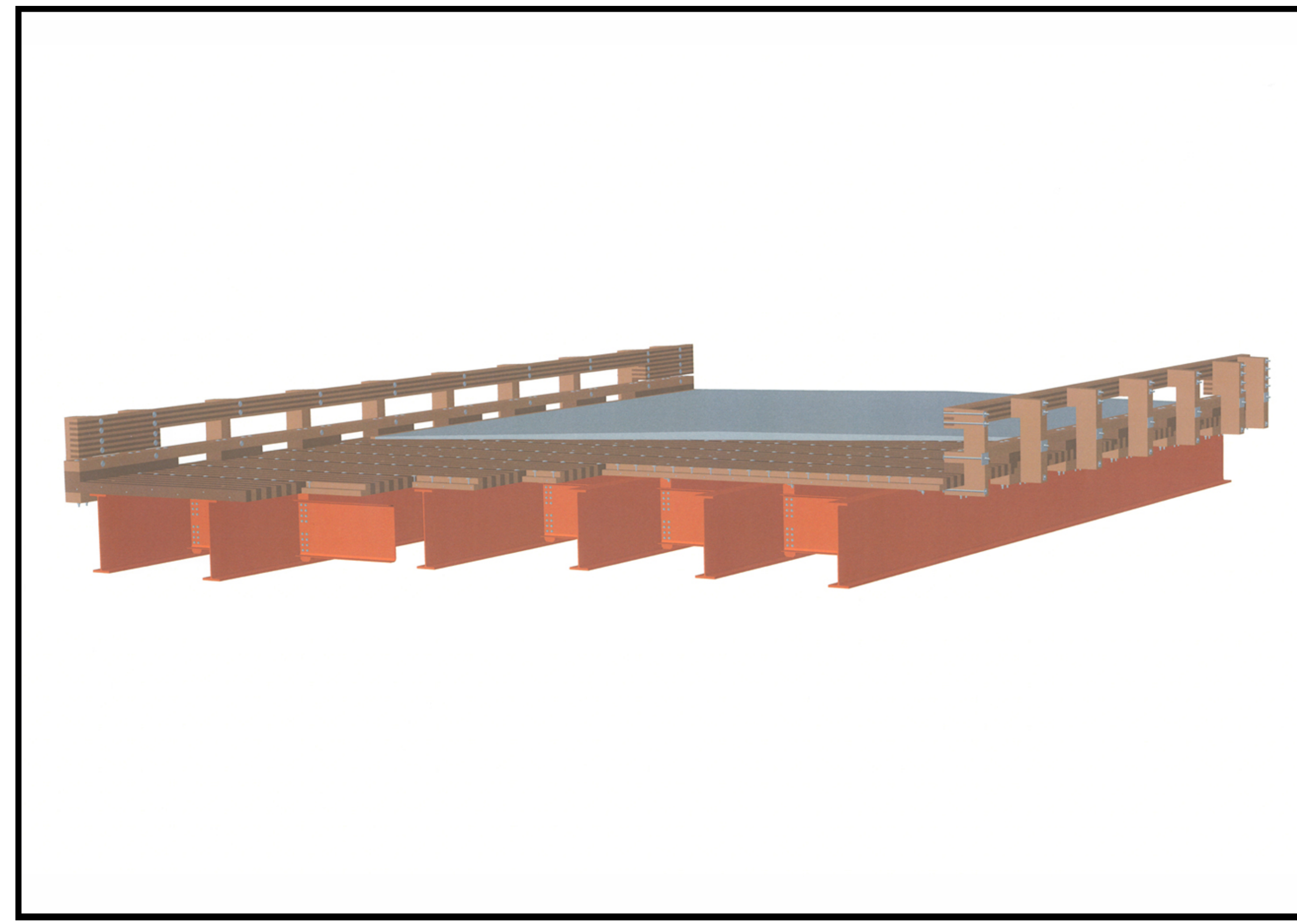

Durability Solutions

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Founded in 1935, the American Galvanizers Association (AGA) is a non-profit trade association dedicated to serving the needs of fabricators, architects, specifiers, and engineers, providing technical support on today's innovative applications and state-of-the-art technological developments in hot-dip galvanizing for corrosion control. The AGA maintains a large technical library, provides multimedia seminars, and offers a toll-free technical support hotline to assist specifiers in North America.

Hot-Dip Galvanizing

The Process

Hot-dip galvanizing (HDG) is the process whereby fabricated steel, structural steel, or small parts, including fasteners, are immersed in a kettle or vat of molten zinc, resulting in a metallurgically bonded alloy coating that protects the steel from corrosion.Galvanizing forms a metallurgical bond between the zinc and the underlying steel or iron, creating a barrier that is part of the metal itself. During galvanizing, the molten zinc reacts with the surface of the steel or iron article to form a series of zinc/iron alloy layers actually harder than the substrate steel it is protecting. The galvanizing process naturally produces coatings that are at least as thick at the corners and edges as the coating on the rest of the article. Because the galvanizing process involves total immersion of the material, all surfaces are coated.

How Hot-Dip Galvanizing Works

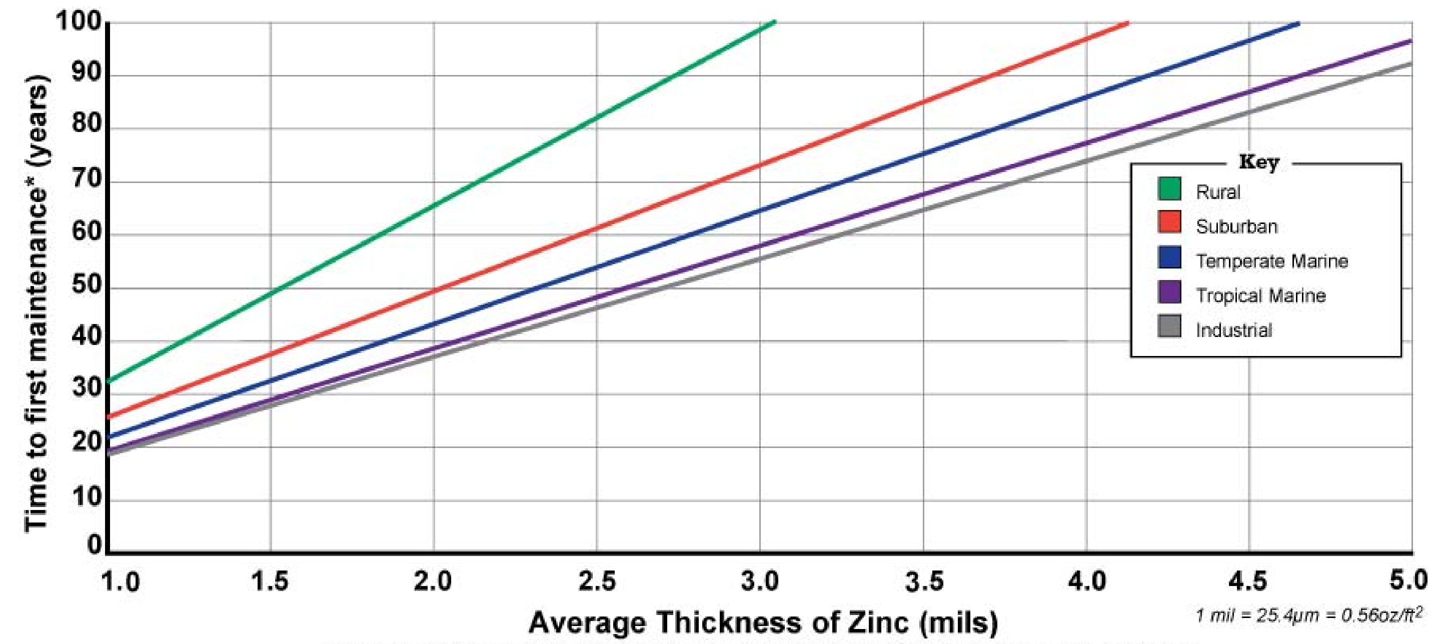
Galvanizing takes place in a factory regardless of weather or humidity conditions and is available 24/7/365 in close proximity to most new bridge locations. Freshly galvanized steel progresses through a natural weathering process to develop a corrosion resistant patina made up of zinc-oxide, zinc-hydroxide, and zinc carbonate. Typically, it takes approximately 6-12 months to fully develop. Because the corrosion rate of zinc is approximately 20 times less than that for black steel, the HDG coating has durability beyond the intended life of most steel structures. The chart below shows the typical time to first maintenance for bridges located in five different environmental exposures.

Economics and Life-cycle Cost

HDG is typically very similar and often lower in initial cost than most other corrosion protection systems considered for steel bridges and because it requires zero maintenance for 75 years or more, the life-cycle cost is typically 4 to 8 times less.

Natural and Sustainable Zinc

Zinc is found everywhere in daily life: in every cell of the human body, in the earth, in food and in products consumer products (sunblock, automobiles, cosmetics, airplanes, appliances, surgical tools, zinc lozenges). Children need zinc for growth and adults need zinc for reproduction and good health.



*Time to first maintenance is defined as the time to 5% rusting of the steel surface.

The U.S. Recommended Daily Allowance is 15 milligrams of zinc.Zinc is 100% recyclable and over 80% of the zinc available for recycling is currently recycled. For more information, click on http://www.galvanizeit.org/about-hot-dip-galvanizing/is-hdg-sustainable/

Bridge Projects

HDG is commonly used on short-span bridges, especially when the bridge will be located in relatively corrosive environments such as above rivers and streams and in humid climates. To view examples of bridges utilizing HDG steel, click on http://galvanizeit.org/project-gallery/gallery (and select "sector" and then "Bridge &Highway)

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Paint

Overview

Constructing bridges extends back thousands of years. In a relative sense steel bridge construction is in its infancy. The first iron bridge was built in 1779, while the first steel was used in a bridge in 1828. Coated bridges from the 19th century survive today.

Corrosion protection via coatings is also an interrelated subject; i.e., in order to lower longer term costs, coatings can be efficiently applied when new steel is in the fabrication shop.

Design Phase

A comprehensive plan for successful corrosion protection and mitigation is needed from the inception of the project and consists of actions which continue throughout the life of the bridge. A plan is needed which includes decisions made during the bridge design process. During this time, the site for the structure is identified. The extent of exposure to any detrimental environmental conditions should drive certain corrosion prevention design choices, such as: type of bridge, type of steel used; the details utilized in developing the shape of members, types of secondary members, and their connections are but a few. The clearance and exposure beneath the structures must be carefully planned. In this process, a corrosion protection plan which provides long-term protection is devised.

Corrosion mitigation choices may vary somewhat; however, for exposure in corrosion prone areas of the country , the use of zinc on bare steel should be considered. American Galvanizers Association (AGA) at 75 years in a severe environment with no paint topcoat. SSPC: The Society For protective Coatings (www.sspc.org) publishes and maintains standards for surface preparation of steel and for the various zinc rich coatings. The American Galvanizers Association (AGA.galvaniteit.org) has information about the uses of galvanizing.

Appropriately selected and applied layers or coats of paint over the hot dip galvanize or thermal spray applied zinc or zinc-rich paint will extend the service life of the corrosion protection. The AGA refers to these as duplex systems.

Since the first use of zinc rich primer coated steel in the late 1960's thousands of zinc-rich primer coated steel bridges have survived for almost 50 years. These bridges are distributed across the country and are examples of permanently installed corrosion protection.

Installation

Even with a properly selected system to address the most challenging exposure, a system must still be correctly installed, and the bridge must be built and maintained! SSPC has detailed information about installation practices, specification and conducts training classes.

Maintenance Completes the Process

The corrosion control effort begins with a comprehensive "corrosion review and planning" prior to and during the bridge design process. Implementation of the plan during detailing and fabrication of the steel, application of the selected coating system, shipping, erection, field painting, touch-up, and performing critical steps identified in a proper maintenance plan are the necessary items in the corrosion protection and mitigation plan. If the planning or maintenance are skipped, we are choosing to save resources in the short term, but in doing so we are consigning ourselves to pay more later on in earlier repairs.

As a reference, please see the included photo of untopcoated Zinc rich paint on the Golden Gate Bridge after 45+ years. (Photo used with permission from Golden Gate Bridge, www.goldengate.org)

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Weathering Steel

The following information is an excerpt from the National Steel Bridge Alliance's Steel Bridge Design Book. Please visit http://www.aisc.org/contentNSBA.aspx?id=20244 for the complete book.

Weathering steel is an important option for the bridge designer. The FHWA Technical Advisory T5140.22 "Uncoated Weathering Steel in Structures" - http://www.fhwa.dot.gov/bridge/t514022.cfm provides guidance to the states for development of their own policies regarding the use of weathering steel. This document contains a digest of the primary benefits and concerns regarding weathering steel and provides specific guidance on its appropriate use. Written in 1989, the document is undergoing a review and rewrite; however, the majority of its content remains useful as a starting point.

Weathering grade steels have been available for several decades. They have been produced in various proprietary chemistries; but essentially small amounts of copper, chromium, nickel and silicon are added to carbon steel to achieve an alloy with enhanced weathering properties. These steels will form a rust patina when exposed to the environment providing a barrier between the bare steel and the corrosive elements of the environment. When properly detailed and exposed to environments that include cyclic wet/dry exposures and do not introduce significant amounts of corrosive contaminants to the steel surface, this tightly adherent patina provides a weathering steel structure with its own protective coating that slows the self-corrosion rate of the steel to a very low rate.
Although highway bridges were not the first industrial application of weathering grade steels, they have been the primary market for the material since the first weathering steel bridges were built in the mid 1960's.

The primary benefit of weathering steel is the promise of long-term corrosion protection without the need for either initial or maintenance painting. The steel industry has made the point that weathering steel, when properly applied, results in a structure that provides first cost and life cycle cost savings. However, due to the assumption that all bridge expansion joints will eventually leak, current guidelines require weathering steel bridge elements to be painted at non-integral beam ends to a length of 1.5 times the girder depth. In addition, weathering steel girders are shop blasted to remove millscale so that the initial protective oxide layer is uniform. These requirements offset some of the potential cost savings associated with weathering steel versus painted or galvanized steel.

Extensive data exists regarding the corrosion performance of weathering steels. The following highlights conclusions taken from the pertinent data:

- Weathering steel requires some amount of moisture and a wet/dry weathering cycle over a period of time to develop a tightly adherent, protective oxide layer. However, excessive moisture or the presence of salt will disrupt this process and result in a structure that corrodes at an unacceptable (much higher) rate.

- Nearly all of the reported failures of weathering steel on bridges have occurred in applications where the steel is wet for a significant portion of time or the steel is exposed to salt from the ocean or deicing operations.

- Properly functioning weathering steel will corrode at a steady-state rate less than 0.3 mils per year (7.5 microns per year). Corrosion in excess of this rate indicates that weathering steel should not be used bare at that location.

Short Span Steel Bridge Alliance Member Contact Information

Short Span Steel Bridge Alliance Members

The following members of the Short Span steel Bridge Alliance are available to assist you with questions or information in regards to short span bridges.

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Regional Director
www.steelbridges.org

5620 Harding Drive
Lincoln, NE 68521
402-466-1007
schrage@steelbridges.org

National Corrugated Steel Pipe
Dwight Thompson
President
www.ncspa.org

14070 Proton Road
Suite 100
Dallas, TX 75244
877-339-4448
dwightmobile@yahoo.com

Steel Fabricators of New England
Catherine Flaherty, CAE
Executive Director
www.ssfne.org

11 Robert Toner Blvd.
234 North Attleboro
North Attleboro, MA 02763
603-766-7363
cflaherty@ssfne.org

Steel Fabricators of New England
Eric Greene
President
www.awsteel.com

PO Box 1429
Coventry, RI 02816
401-397-9155
eric@awsteel.com

Industry Associations (Cont'd)

NACE
Mark Servl
Representing NACE
www.countyengineers.org

260 North 7th Street
Barron County, WI 54812
715-637-3755
mark.servl@co.barron.wi.us

National Corrugated Steel Pipe Association
Michael McGough
Chief Engineer
www.ncspa.org

P.O. Box 4244
Falls Church, VA 22044
703-812-4701
mmcgough@ncspa.org

The following members of the Short Span steel Bridge Alliance are available to assist you with questions or information in regards to short span bridges.

Producers

Gerdau
Matthew Gomez
Natl. Sales Mgr
www.Gerdau.com
300 Ward Road
Midlothian, TX 76065
972-779-1735
matthew.gomez@gerdau.com

Arcelor Mittal
Alex Wilson
Division Manager, Customer Technical
www.arcelormittal.com
139 Modena Road
Coatesville, PA 19320
610-383-3105
alex.wilson@arcelormittal.com

Nucor-Yamato Steel Company
Michael Engestrom
Technical Marketing Director
www.nucoryamato.com
P.O. Box 1228
Blytheville, AR 72316-1228
800-289-6977
mengestrom@nucor-yamato.com

Nucor Steel - Hertford County
Phil Bischof
Plate Producer Manager
www.nucorhertford.com
1505 River Road
Cotfield, NC 27922
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phil.bischof@nucor.com

SSAB Americas-Montpelier Caster
James Barber
Regional Sales & Product Development Manager
www.ssab.com
1770 Bill Sharp Blvd.
Montpelier Street Operations
Muscatine, IA 52761
563-381-5334
jim.barber@ssab.com

Producers (Cont'd)

Nucor Fastener Division
Don West
Regional Sales Manager
www.nucor-fastener.com
P.O. Box 6100
St Joe, IN 46785
800-955-6826
don.west@nucor.com

The following members of the Short Span steel Bridge Alliance are available to assist you with questions or information in regards to short span bridges.

Service Center	
Infra-Metals Company John Lusdyk *Executive VP Commercial* www.Infra-Metals.com	580 Middletown Blvd Suite D-100 Langhorne, PA 19047 609-937-1600 johnl@infra-metals.com
Metals USA Jim Collins *Vice President Sales* www.MetalsUSA.com	2400 East Commercial Blvd Suite 905 Fort Lauderdale, FL 33308 954-202-4000 JCollins@metalsusa.com
Triple-S Steel Holdings, Inc. Kevin Dempsey *Director, Business Development* http://sss-steel.com/	6000 Jensen Drive Houston, TX 77026 (713) 697- 7105 Kevin.m.dempsey@sss-steel.com

The following members of the Short Span steel Bridge Alliance are available to assist you with questions or information in regards to short span bridges.

Universities

University of Wyoming
Michael Barker
Professor, Civil and Architectural Engineering
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1000 E University Ave
P.O. Box 3295
Laramie, WY 82071
307-766-2916
barker@uwyo.edu

West Virginia University
Karl Barth
SSSBA Consultant West Virginia University
www.cemr.wvu.edu

P.O. Box 6103
Morgantown, WV 26506
304-293-9921
kebarth@mail.wvu.edu

Steel Bridge Solutions

I-93 over NH-97 with 75 ft span

Brian Karolicki

Worcester Polytechnic Institute

11/8/2012 11:25 PM

Table of Contents

About Short Span Steel Bridge Alliance

The Short Span Steel Bridge Alliance (SSSBA) is a group of bridge and culvert industry leaders - including steel manufacturers, fabricators, service centers, coaters, researchers, and representatives of related associations and government organizations - who have joined together to provide educational information on the design and construction of short span steel bridges in installations up to 140 feet in length.

For more information about the SSSBA, please contact:

Daniel R. Snyder
Manager, Business Development
Steel Market Development Institute, a Business Unit of AISI
25 Massachusetts Ave, NW
Suite 800
Washington, DC 20001
Work Phone: (301) 367-6179
Email: dsnyder@steel.org

For media related information, please contact:

Dianne Newton-Shaw
The Placemaking Group
299 Third Street
Oakland, CA 94607
Work Phone: (510) 496-2352 ext 206
Fax: (510) 238-0589
Email: dnshaw@placemakinggroup.com

Design Support

The Short Span Steel Bridge Alliance offers complimentary design support for questions relating to bridge and culvert design. Design support is offered by the following organizations (to submit an inquiry, please visit www.ShortSpanSteelBridges.org and click on the “Bridge Technology Center” link on the homepage):

Standard Design and Details of Short Span Bridges (Plate Girder & Rolled Beam Bridges)

The Bridge Technology Center is a complimentary resource available for questions specific to standard design and detail solutions of short span steel bridges (refer to the section of this Solutions Book on plate girder and rolled beam standards, if applicable). It is a resource provided by West Virginia University and the University of Wyoming.

Standard Design and Details of Corrugated Steel Pipe and Structural Plate

The National Corrugated Steel Pipe Association provides complimentary design support for questions pertaining specifically to standard design and detail solutions of corrugated steel pipe and corrugated structural plate (refer to the section of this Solutions Book on corrugated steel pipe and corrugated structural plate standards, if applicable).

Manufactured Steel Solutions

For questions pertaining to a specific manufacturer’s solution (refer to section on Manufacturer’s Steel Solutions of this Solutions Book), it is recommended that you directly contact the manufacturer by utilizing the contact information listed with the solution.

User Name:	Brian Karolicki
User Company:	Worcester Polytechnic Institute
User Input Date:	11/08/2012
Project Name:	I-93 over NH-97 with 75 ft span
City:	Salem
State/Province:	NH
Roadway:	I-93 Southbound
Span Length:	75' 0"
Number of Striped Traffic Lanes:	4
Roadway Width:	72'
Individual Parapet Width:	0'
Individual Deck Overhang Width:	2'
Pedestrian Access:	No
Number of Sidewalks:	Not provided
Total Width of Each Sidewalk:	Not provided
Skew Angle:	degrees
Average Daily Traffic (ADT):	Over 2,000
Design Speed:	46+ mph
Waterway Area:	Not provided
Height of Cover:	Not provided

Disclaimer

Notes

- Short span standards for rolled beam solutions are only available for input lengths between 40 and 100 feet and skew angles under 20 degrees.*
- Short span standards for homogeneous plate girder solutions are only available for input lengths between 60 and 140 feet and skew angles under 20 degrees.*
- Short span standards for hybrid plate girder solutions are only available for input lengths between 80 and 140 feet and skew angles under 20 degrees.*
- Design standards for rolled beam and plate girder solutions are rounded in five (5) foot increments.
- Corrugated steel pipe and structural plate standards are only available for input lengths under 85 feet.*
- Customized prefabricated manufacture solutions are available for all lengths and skew angles.

** For bridges/culverts outside of this range, standard designs will not appear in your solutions book.*

Standard Design and Details of Short Span Steel Bridge Solutions

General

These plans are intended to serve as a guide to state, county, and local highway departments in the development of suitable and economical steel bridge superstructure designs. The plans should be particularly valuable to the smaller highway departments with limited engineering staff.

Specifications

Specifications for design, materials, and construction are included in the following:

- AASHTO LRFD bridge design specifications, fifth edition with 2010 interim revisions. 2010. Adopted and published by the American Association of State Highway and Transportation Officials. Washington, DC
- AASHTO/NSBA Collaboration Standard S2.1. Steel Bridge Fabrication Guide Specifications, 2008. Developed by the AASHTO/NSBA Steel Bridge Collaboration. Washington, DC
- AASHTO/NSBA Collaboration Standard G1.4. Guidelines for design details. 2006. Developed by the AASHTO/NSBA Steel Bridge Collaboration, Washington, DC
- ASTM Standards. Published by the American Society for Testing and Materials. ASTM International, 100 Barr Harbor Drive, P.O. Box C700, West Conshohocken, PA 19428-2959 USA.

Design Loading

AASHTO HL-93 Vehicular Live Loading was used throughout.

Design Method

Load and Resistance Factor Design (LRFD) method was employed throughout. Designs were originated using 5 girders with equal spacing. However, plate sizes and beam selections are adequate for any increment of girder layout. Designs will accommodate skews up to 20° from perpendicular, and are intended to be parallel.

Three options are available for steel superstructure composite I-girders. These options are as follows:

1. Homogenous plate girders comprised of ASTM A709-50W steel. These designs are available for a span range of 60'-140'.
2. Hybrid plate girders comprised of ASTM A709-50W and A709-70W steel. A709-50W steel is utilized for the top flange and web. A709-70W steel is utilized for the bottom flange. These designs are available for a span range of 80'-140'.
3. Rolled beams comprised of ASTM A709-50W steel. These designs are available for a span range of 40'-100'.

Structural Steel

All structural steel shall conform to AASHTO M270 (ASTM A709) grade 50, 50W, or 70W, as applicable. Refer to "Design Method."

Concrete

Concrete for deck and parapet shall have a minimum 28-day compressive strength (f'c) of 4,000 PSI.

Concrete Deck

The deck thickness employed for design was 8". This includes a 1/4" integral wearing surface which is not considered part of the structural depth. The owner shall specify the required deck cross slope and grade.

Reinforcing Steel

Reinforcing steel shall conform to ASTM A615 grade 60.

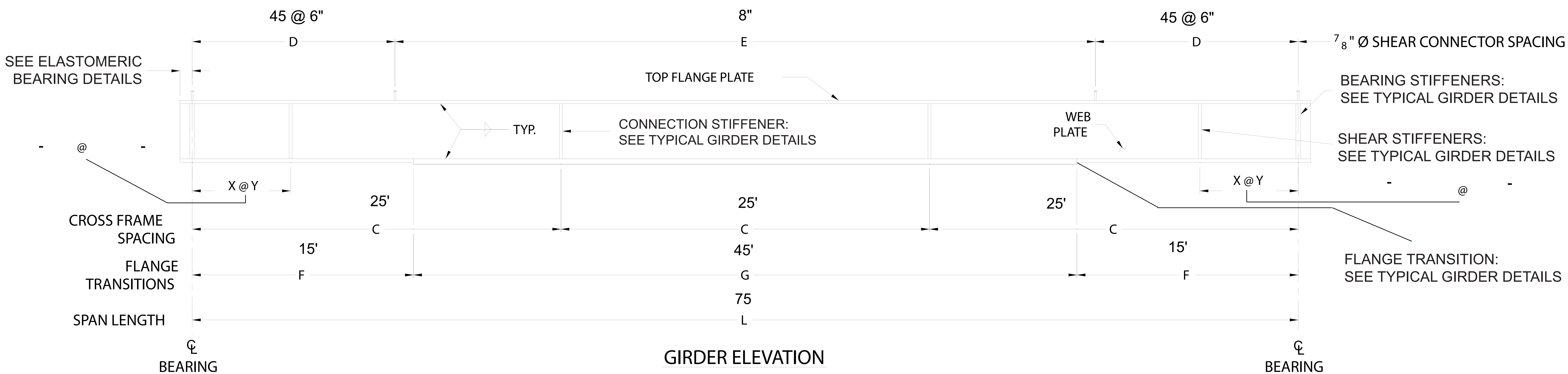
Shear Connectors

Welded stud shear connectors shall conform to the requirements of ASTM A108.

Elastomeric Bearings

See Elastomeric Bearing Details.

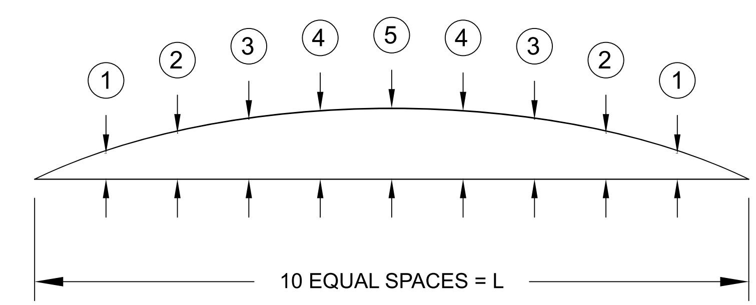
COMPOSITE PLATE GIRDER WITH PARTIALLY STIFFENED WEB - 8 GIRDERS AT 9' 8.571" GIRDER SPACING, HOMOGENEOUS



GIRDER ELEVATION

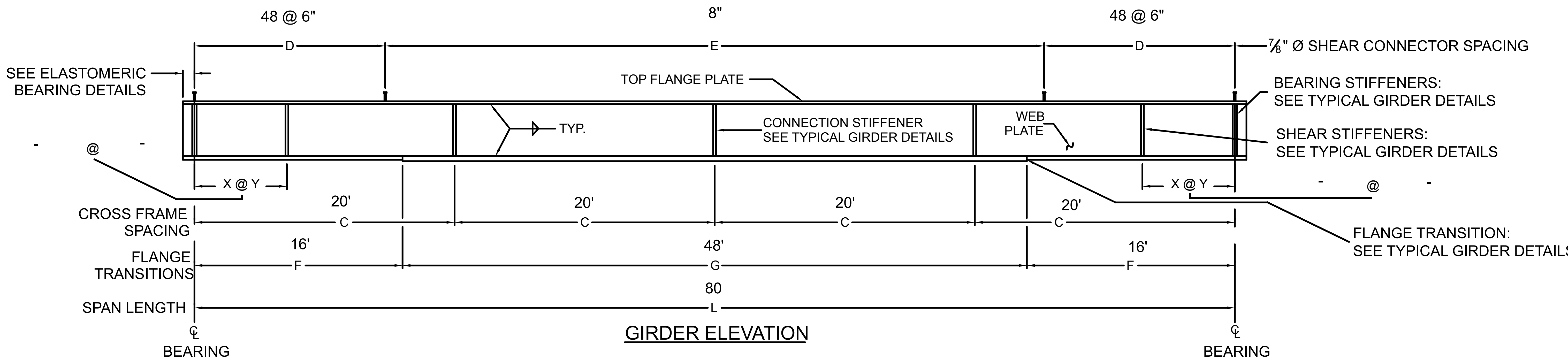
SPAN (L) - ft	PLATE GIRDER SIZE						DIAPHRAGM SPACING (C) - ft	SHEAR STIFFENERS		SHEAR CONNECTOR MAX. SPACING		INDIVIDUAL GIRDER WEIGHT
	TOP FLANGE - in	BOTTOM FLANGE (F)		BOTTOM FLANGE (G)		WEB PLATE- in		X (NO. REQ'd)	Y - ft. (SPACING)	D	E	
		PLATE - in	LENGTH - in	PLATE - in	LENGTH - in							
75	16 x 1"	16 x 1"	15'	16 x 2"	45'	24 x 1/2"	25'	-	-	45 @ 6"	8"	13,679 lbs

STEEL D.L. CAMBER - in					TOTAL D.L. CAMBER - in				
1	2	3	4	5	1	2	3	4	5
0.228"	0.425"	0.575"	0.671"	0.704"	1.735"	3.225"	4.351"	5.068"	5.312"



D.L. CAMBER

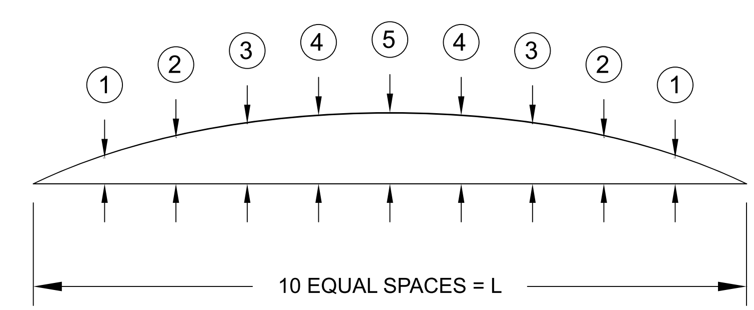
COMPOSITE PLATE GIRDER WITH PARTIALLY STIFFENED WEB - 8 GIRDERS AT 9' 8.571" GIRDER SPACING, HYBRID



GIRDER ELEVATION

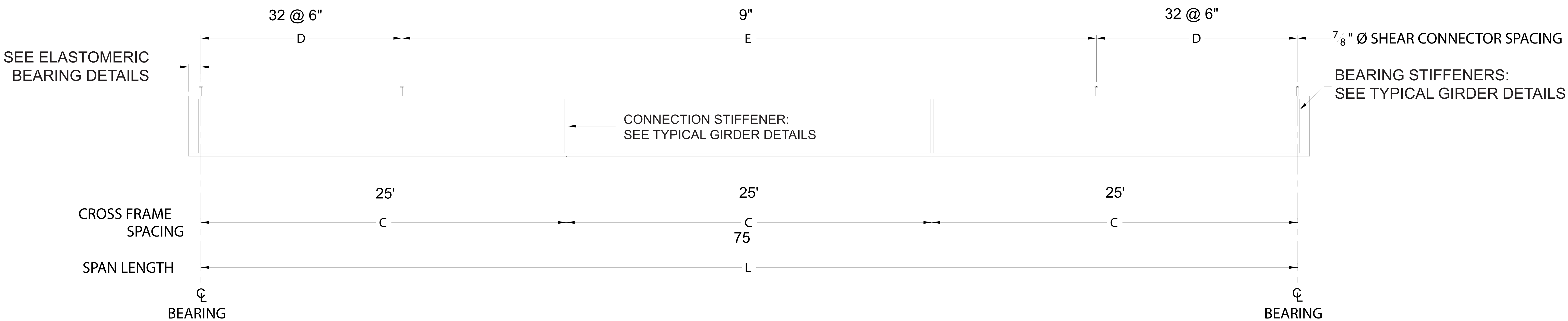
SPAN (L) - ft	PLATE GIRDER SIZE						DIAPHRAGM SPACING (C) - ft	SHEAR STIFFENERS		SHEAR CONNECTOR MAX. SPACING		INDIVIDUAL GIRDER WEIGHT
	TOP FLANGE - in	BOTTOM FLANGE (F)		BOTTOM FLANGE (G)		WEB PLATE- in						
		PLATE - in	LENGTH - Ft	PLATE - in	LENGTH - Ft			X (NO. REQ'd)	Y - ft. (SPACING)	D	E	
80	16 x 1"	16 x 1 1/2"	16'	16 x 2"	48'	24 x 1/2"	20'	-	-	48 @ 6"	8"	15,462 lbs

STEEL D.L. CAMBER - in					TOTAL D.L. CAMBER - in				
1	2	3	4	5	1	2	3	4	5
0.294"	0.553"	0.754"	0.882"	0.926"	1.807"	3.396"	4.623"	5.402"	5.670"



D.L. CAMBER

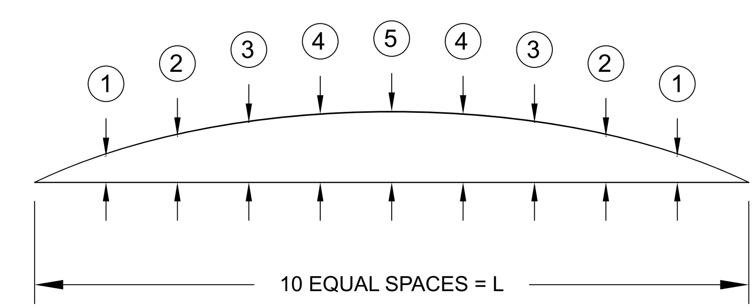
COMPOSITE ROLLED BEAM WITH PARTIALLY STIFFENED WEB - 8 GIRDERS AT 9' 8.571" GIRDER SPACING, LIGHTEST WEIGHT



BEAM ELEVATION

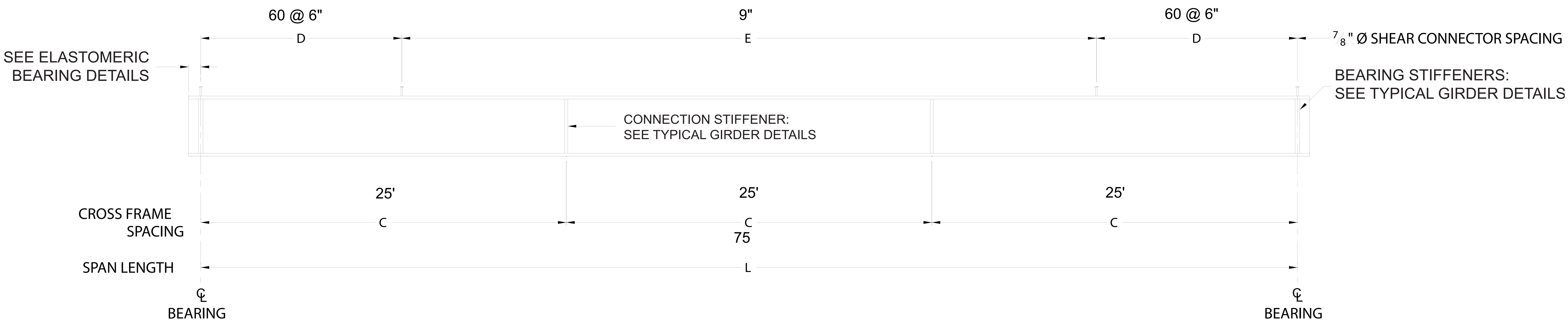
SPAN (L) - ft	SELECTED SECTIONS	DIAPHRAGM SPACING (C) - ft	SHEAR CONNECTOR MAX. SPACING		WEIGHT
			D	E	
75	W36x210	25'	32 @ 6"	9"	15,750 lbs

STEEL D.L. CAMBER - in					TOTAL D.L. CAMBER - in				
1	2	3	4	5	1	2	3	4	5
0.138"	0.261"	0.357"	0.418"	0.438"	1.033"	1.954"	2.675"	3.133"	3.289"



D.L. CAMBER

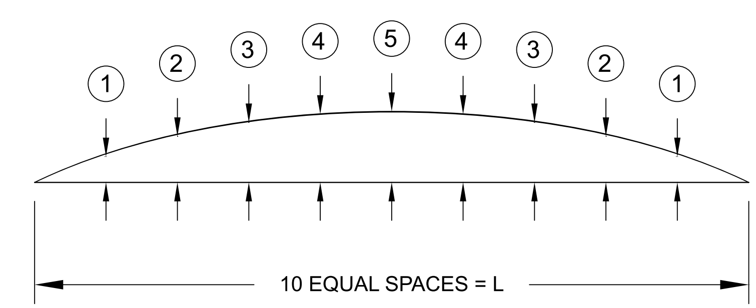
COMPOSITE ROLLED BEAM WITH PARTIALLY STIFFENED WEB - 8 GIRDERS AT 9' 8.571" GIRDER SPACING, LIMITED DEPTH



BEAM ELEVATION

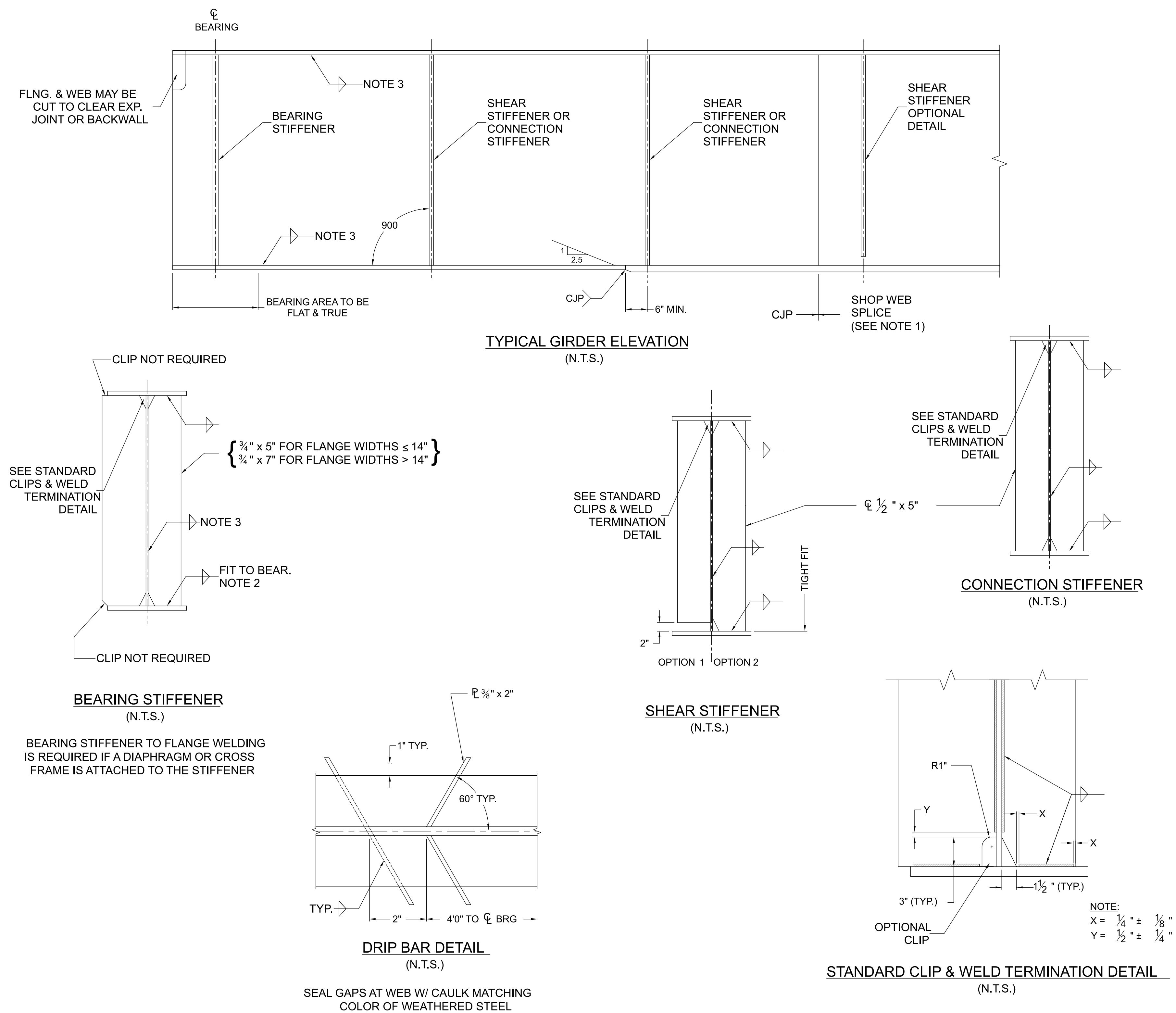
SPAN (L) - ft	SELECTED SECTIONS	DIAPHRAGM SPACING (C) - ft	SHEAR CONNECTOR MAX. SPACING		WEIGHT
			D	E	
75	W30x235	25'	60 @ 6"	9"	17,625 lbs

STEEL D.L. CAMBER - in					TOTAL D.L. CAMBER - in				
1	2	3	4	5	1	2	3	4	5
0.172"	0.325"	0.445"	0.521"	0.547"	1.116"	2.113"	2.893"	3.389"	3.558"



D.L. CAMBER

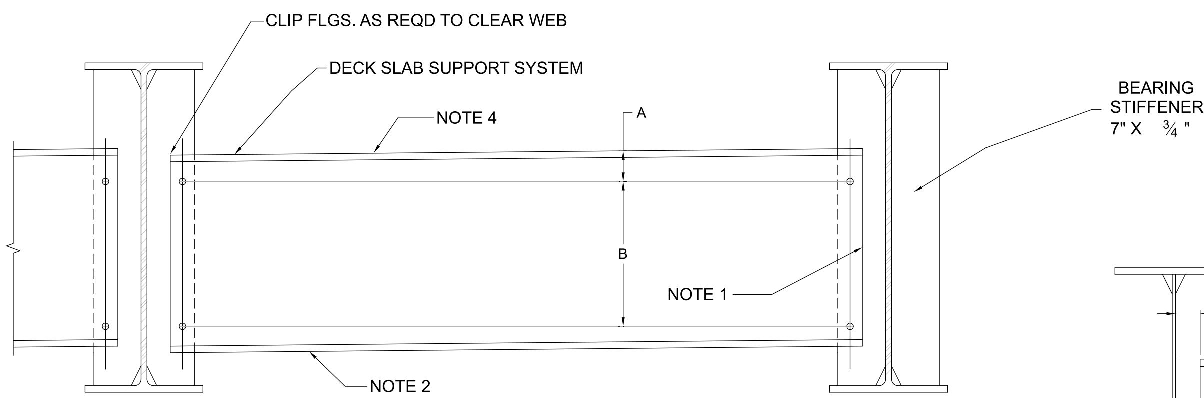
Typical Girder Details



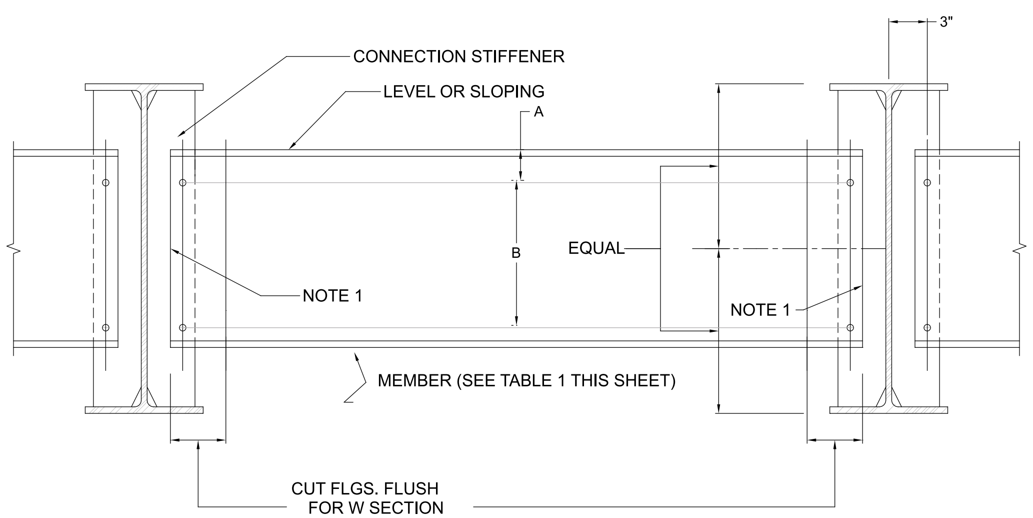
NOTES:

1. All CJP welds to be ground and tested per state specifications.
2. Fit to bearing is to be 50% in contact with flange and within 1/16” for remainder.
3. MT 1’ of every 10’ (extents of mag particle inspection for fillet welds) -OR- see state specs.

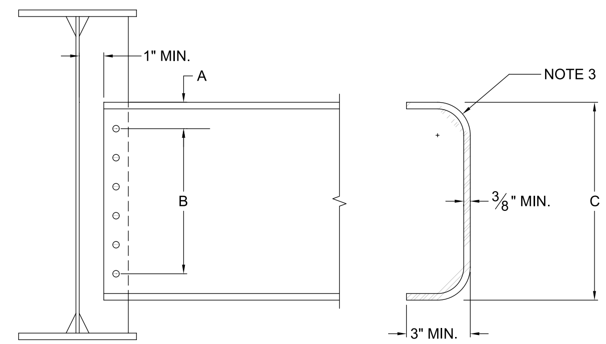
Rolled Shape and Bent Plate Diaphragm Details



END DIAPHRAGM
(N.T.S.)

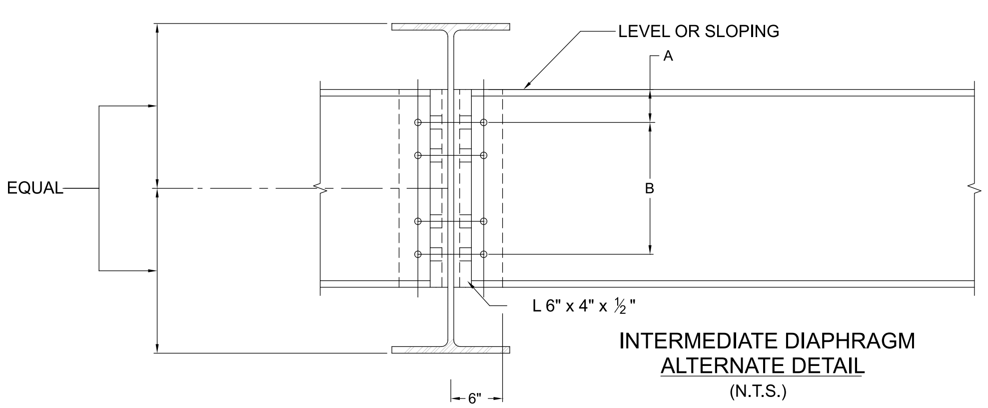


INTERMEDIATE DIAPHRAGM
(N.T.S.)



BENT PLATE DIAPHRAGM
(N.T.S.)

(CAN BE USED AS ALTERNATE TO ROLLED SHAPE DIAPHRAGM)



INTERMEDIATE DIAPHRAGM
ALTERNATE DETAIL
(N.T.S.)

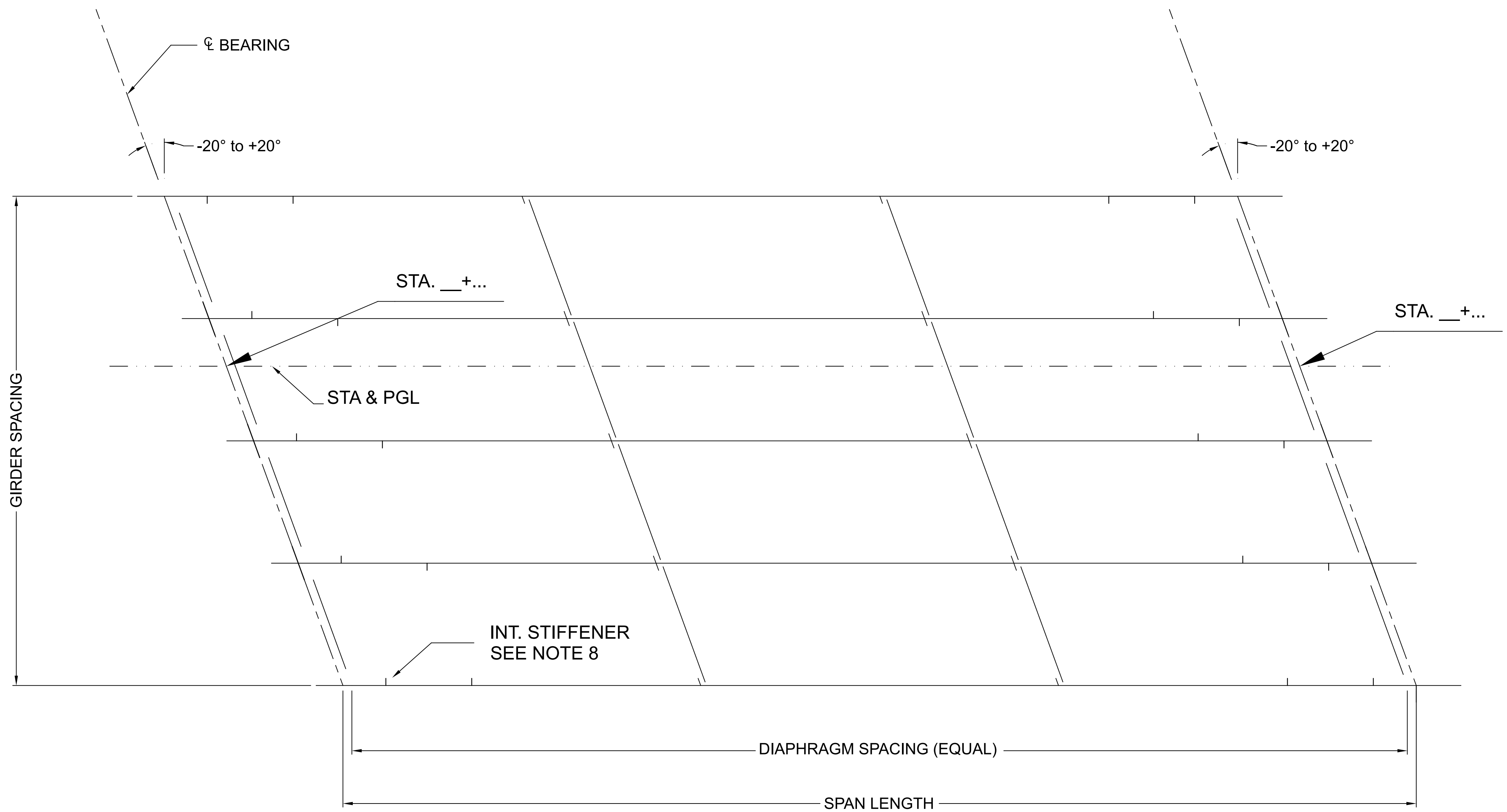
TABLE 1

DEPTH OF STRINGER OR GIRDER WEB	DIAPHRAGM SIZE	DIMENSIONS		
		A	B	C
≤ 30"	C15x33.9	3"	3 @ 3"	15"
30" < X ≤ 36"	MC18x42.7	3"	4 @ 3"	18"
> 36"	W30x90	5"	5 @ 4"	30"

NOTES:

1. Slope diaphragm and keep holes vertical in stiffener at constant dimensions (to keep all stiffeners the same) and cut ends of diaphragm square.
2. At expansion joint, orient channel flanges away from joint opening.
3. Minimum radius as per AASHTO/NSBA fabrication S2.1 table 4.3.2-1. Per section 4.3.2, if the bend is parallel to direction of rolling, multiply the minimum radii by 1.5.
4. All holes to be 15/16" ø for 7/8" ø HS bolts, ASTM A325 type 3 w/ F436-3 washers (RCT).
5. Threads excluded from shear plane.

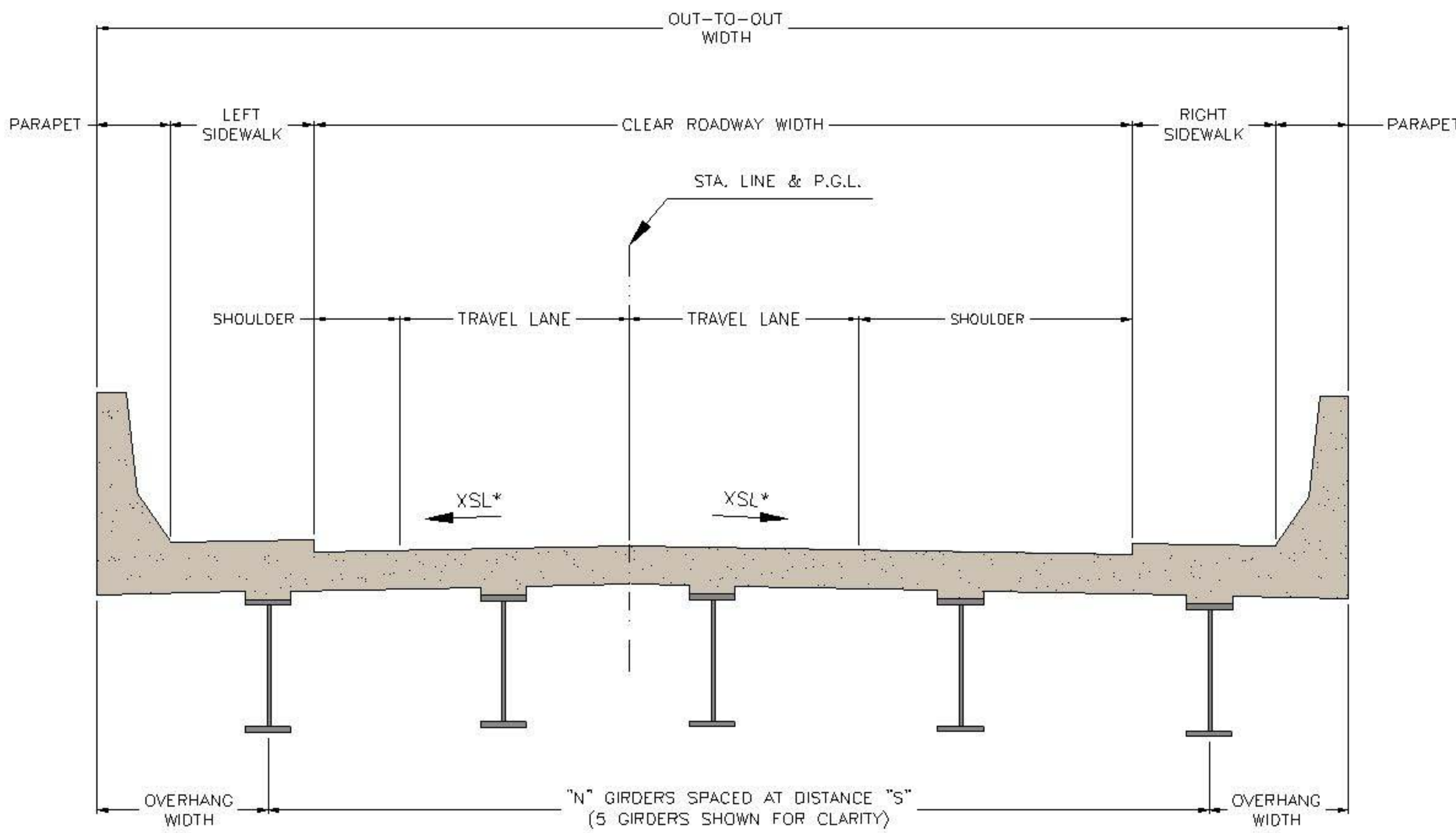
Framing Plan



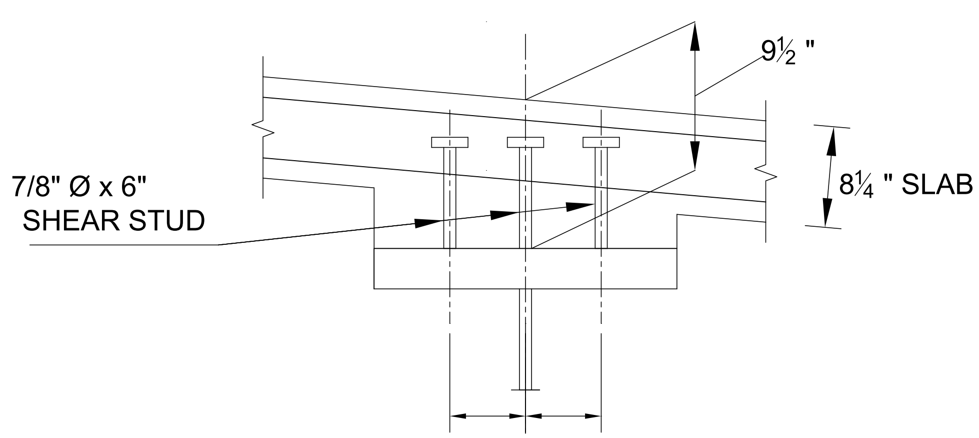
NOTES:

1. Superstructure may sit on existing bridge seats. Contractor to verify spacing in field.
2. Design will accommodate skews up to 20° from ⊥, but are intended to be parallel.
3. Station line is intended to be on a tangent alignment.
4. Max grade at bearing is ± 5%.
5. Orient toes of channel diaphragm down grade.
6. Diaphragms may be placed on either side of connection plate at the contractor's discretion.
7. Keep diaphragm lines parallel to bearing lines.
8. Int. stiffeners are required on one side of web only. On fascia girders, orient stiffeners to the inside of the girder. On interior girders, stiffeners should alternate sides. See Girder Elevations for spacing.

Typical Section



*NOTE: XSL - Cross slope can vary from -.06% to +.06%.

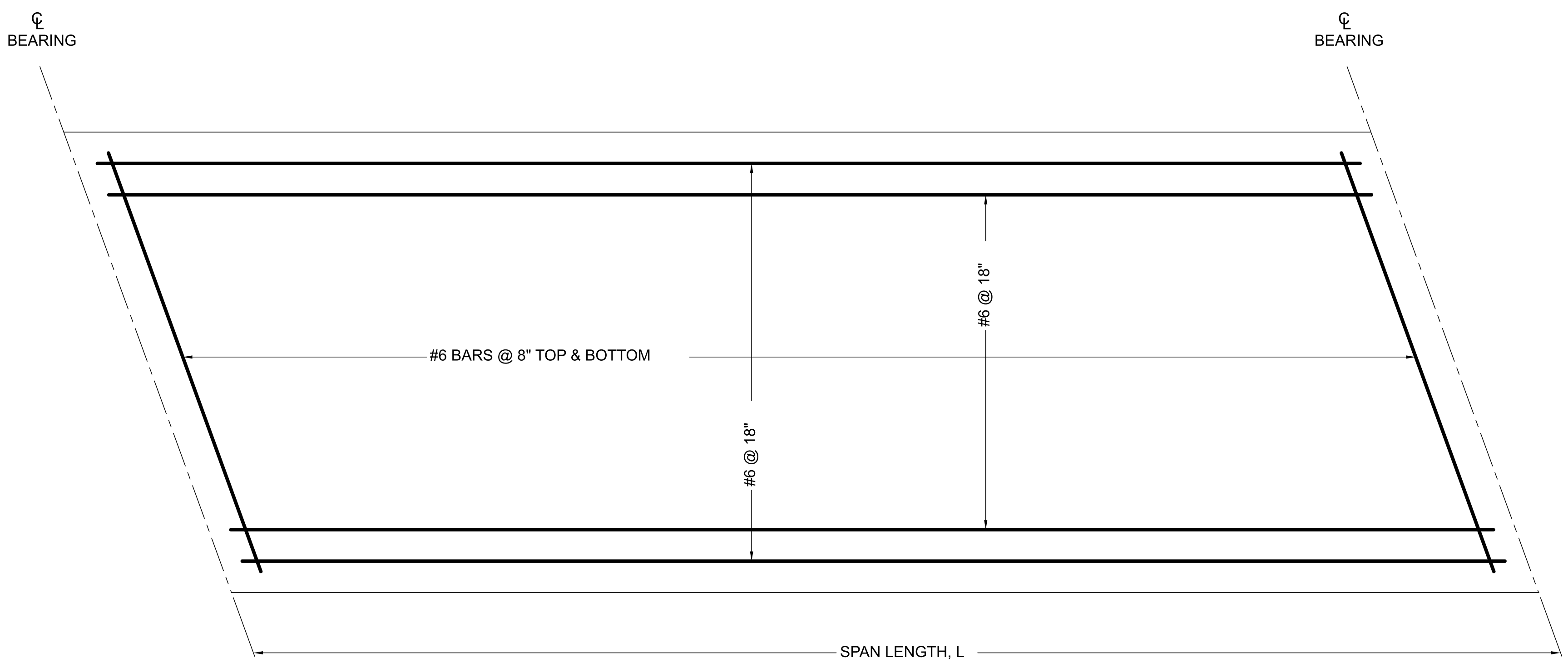


DETAIL A

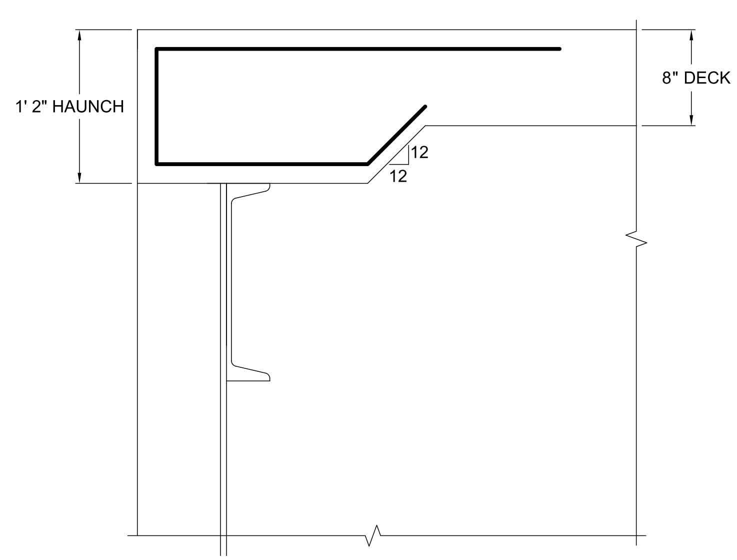
NOTES:

1. For shear stud spacing, see Girder Elevations.
2. Parapets per state DOT requirements, if cast in place, provide 2'-0" lap with transverse bars.

Deck Design



REINFORCING PLAN
(N.T.S.)

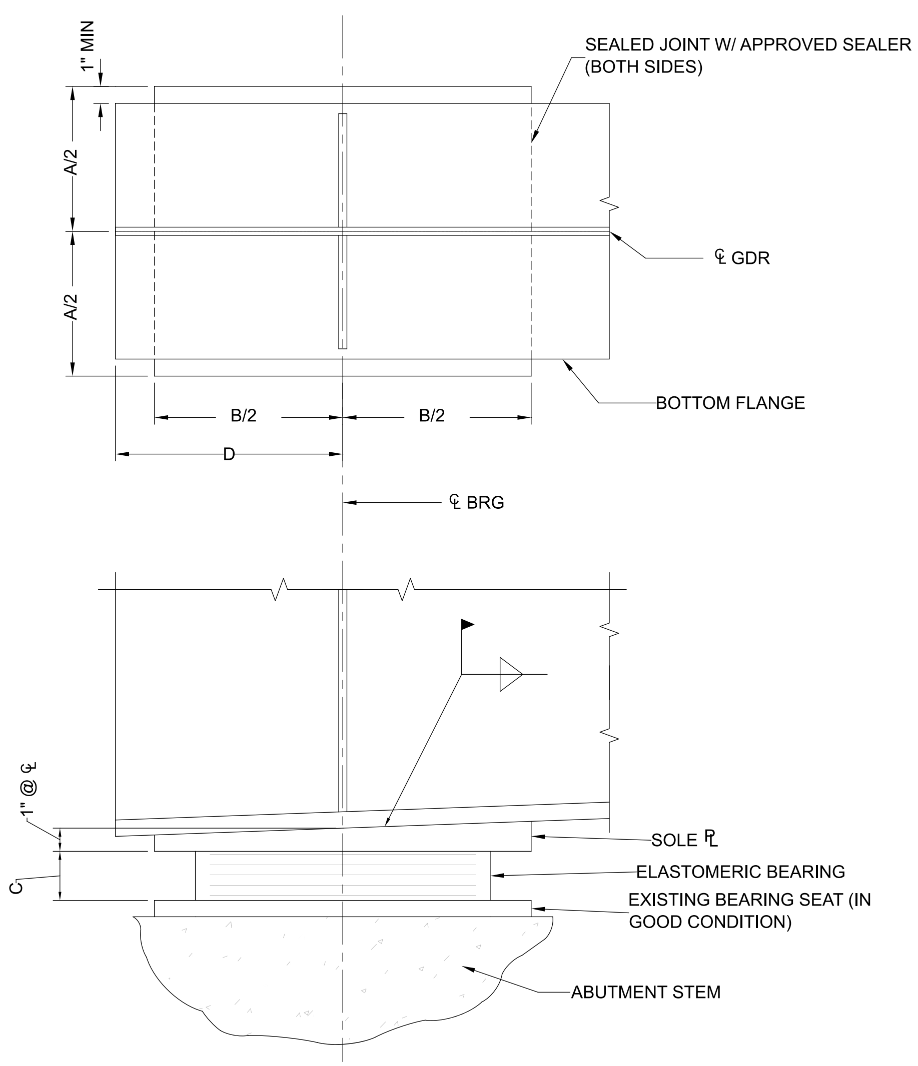


HAUNCH DETAIL
(N.T.S.)

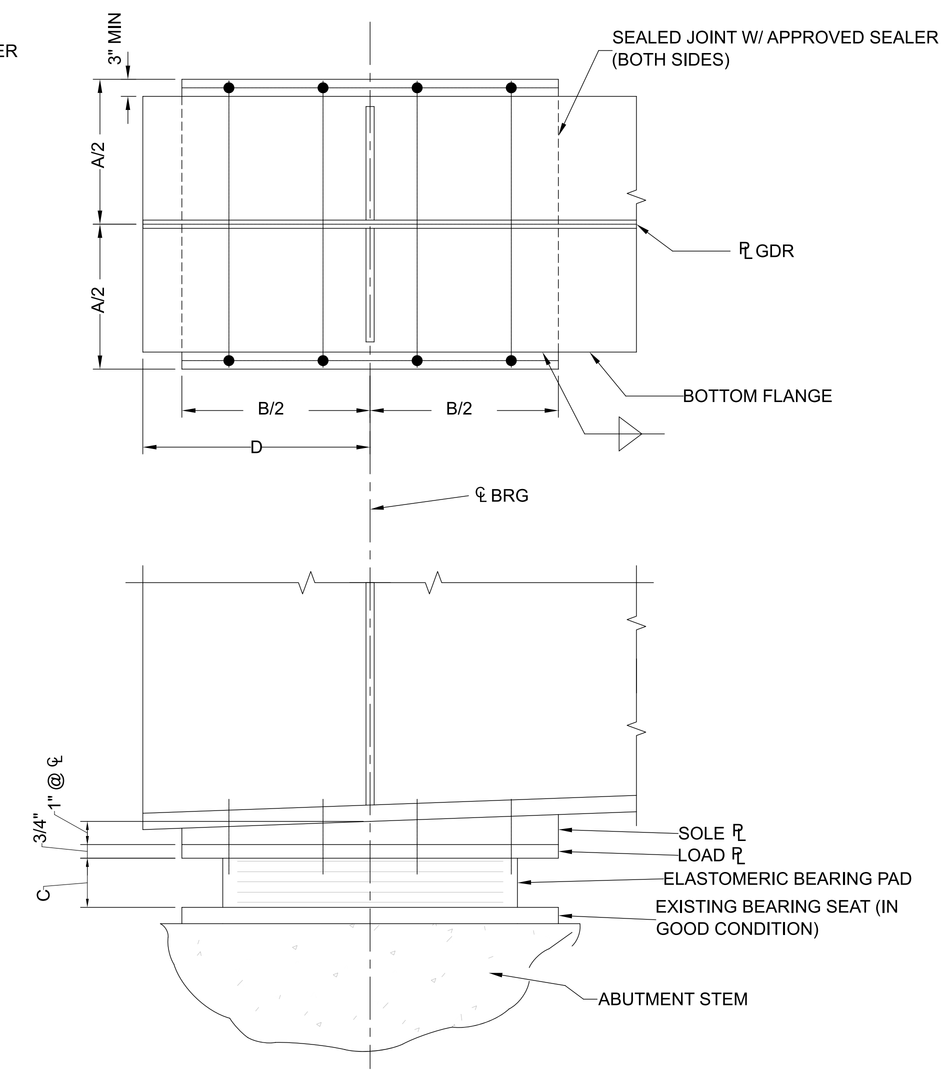
NOTES:

1. Forming brackets must extend to bottom flange.

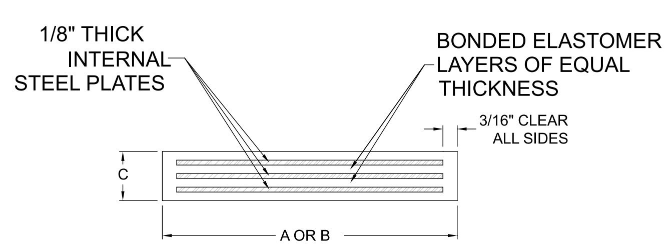
COMPOSITE PLATE GIRDERS - 9' 8.571" GIRDER SPACING, HOMOGENEOUS



BEARING ELEVATION
OPTION "A"
(N.T.S.)



BEARING ELEVATION
OPTION "B"
(N.T.S.)



SECTION VIEW OF ELASTOMERIC BEARING
(N.T.S.)

NOTES:

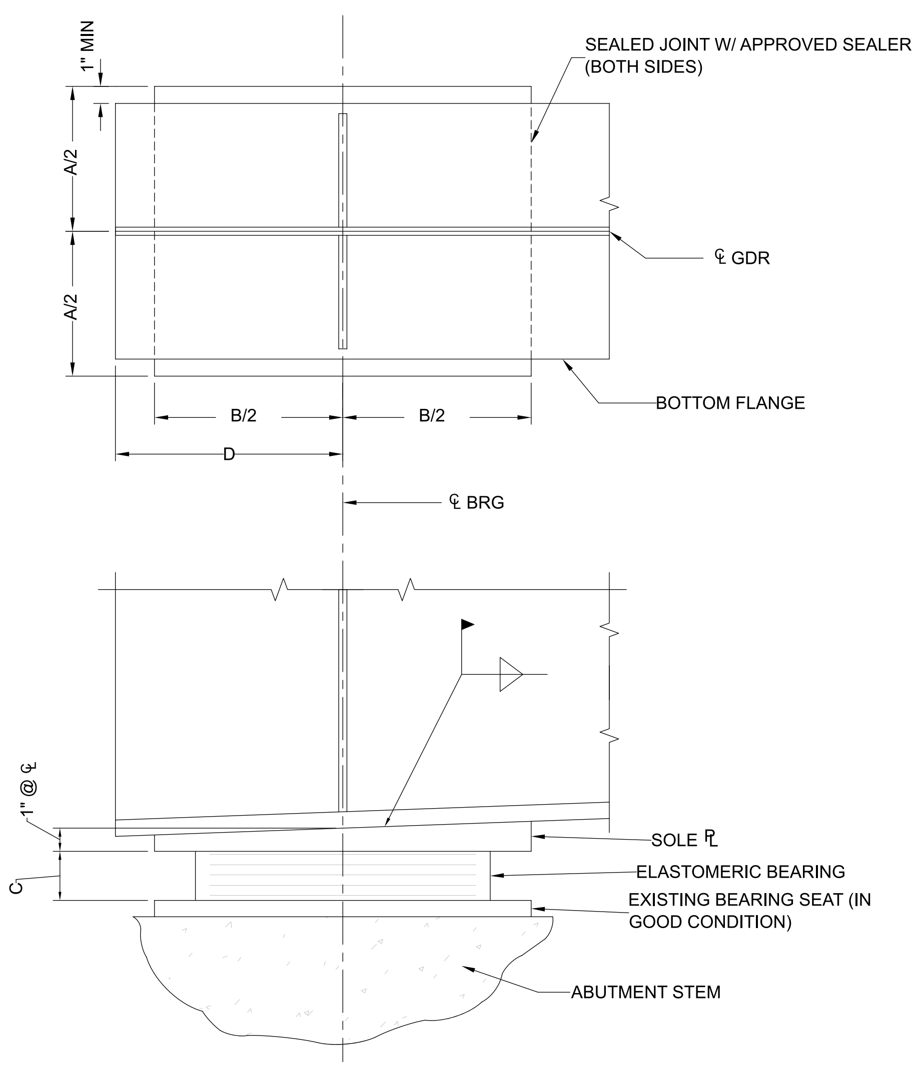
1. Bevel sole ℙ if grade exceeds ± 1%.
2. Max Grade is ± 5%.
3. Sole ℙ to be factory vulcanized to elastomeric bearing pad.
4. Holes to be 1 1/16" Ø in sole ℙ for 7/8" Ø bolt.
5. All elastomeric cover layers are 1/4" thick.

COMMENTARY:

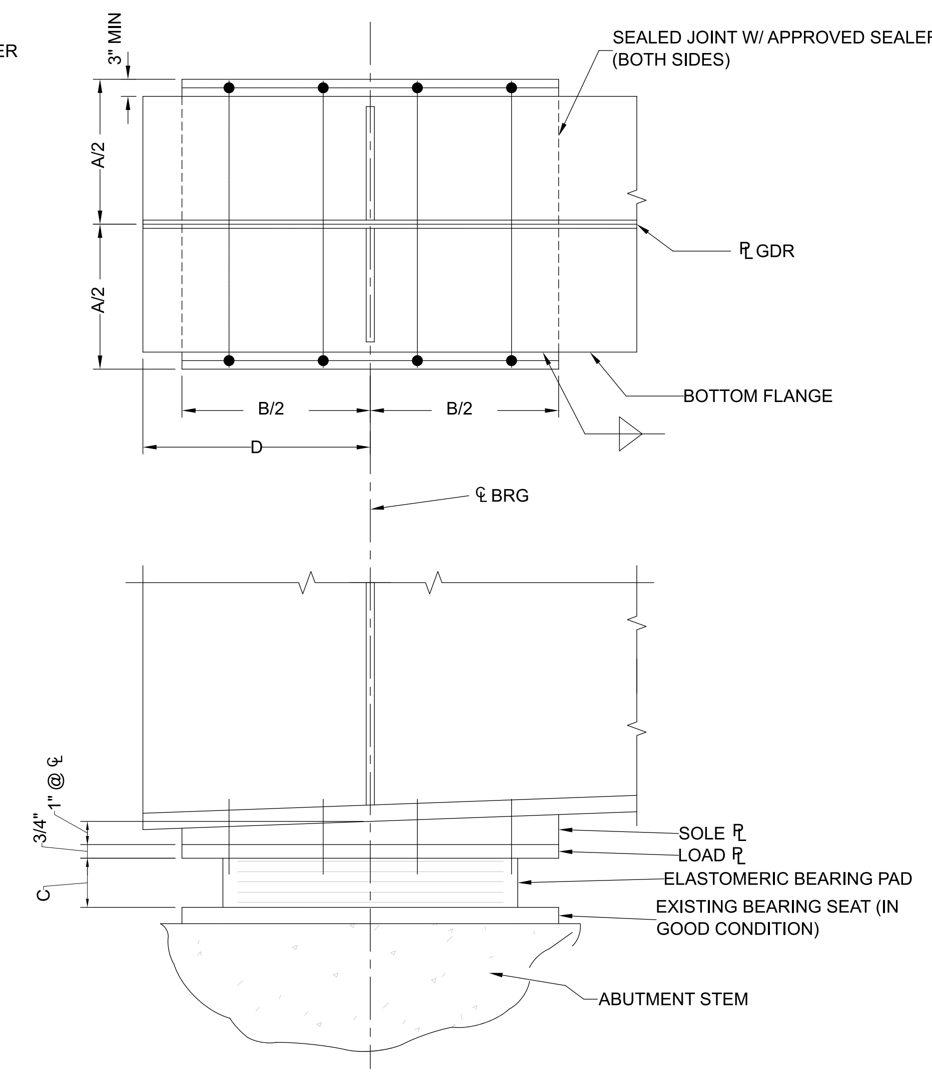
1. Care must be exercised with the field welding. The temperature of the steel adjacent to the bearing must be kept below 250°F (120°C). Temperature crayons should be used to monitor the steel temperature during welding.

ELASTOMETRIC BEARING DETAILS - in					
A	B	C	D	INTERNAL ELASTOMER LAYERS	
				NO. OF LAYERS	THICKNESS - in
16"	16"	3.75"	12"	5	0.5"

COMPOSITE PLATE GIRDERS - 9' 8.571" GIRDER SPACING, HYBRID



BEARING ELEVATION
OPTION "A"
(N.T.S.)



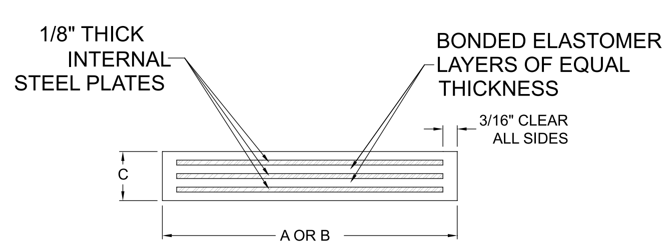
BEARING ELEVATION
OPTION "B"
(N.T.S.)

NOTES:

1. Bevel sole PL if grade exceeds ± 1%.
2. Max Grade is ± 5%.
3. Sole PL to be factory vulcanized to elastomeric bearing pad.
4. Holes to be 1 1/16” Ø in sole PL for 7/8” Ø bolt.
5. All elastomeric cover layers are 1/4” thick.

COMMENTARY:

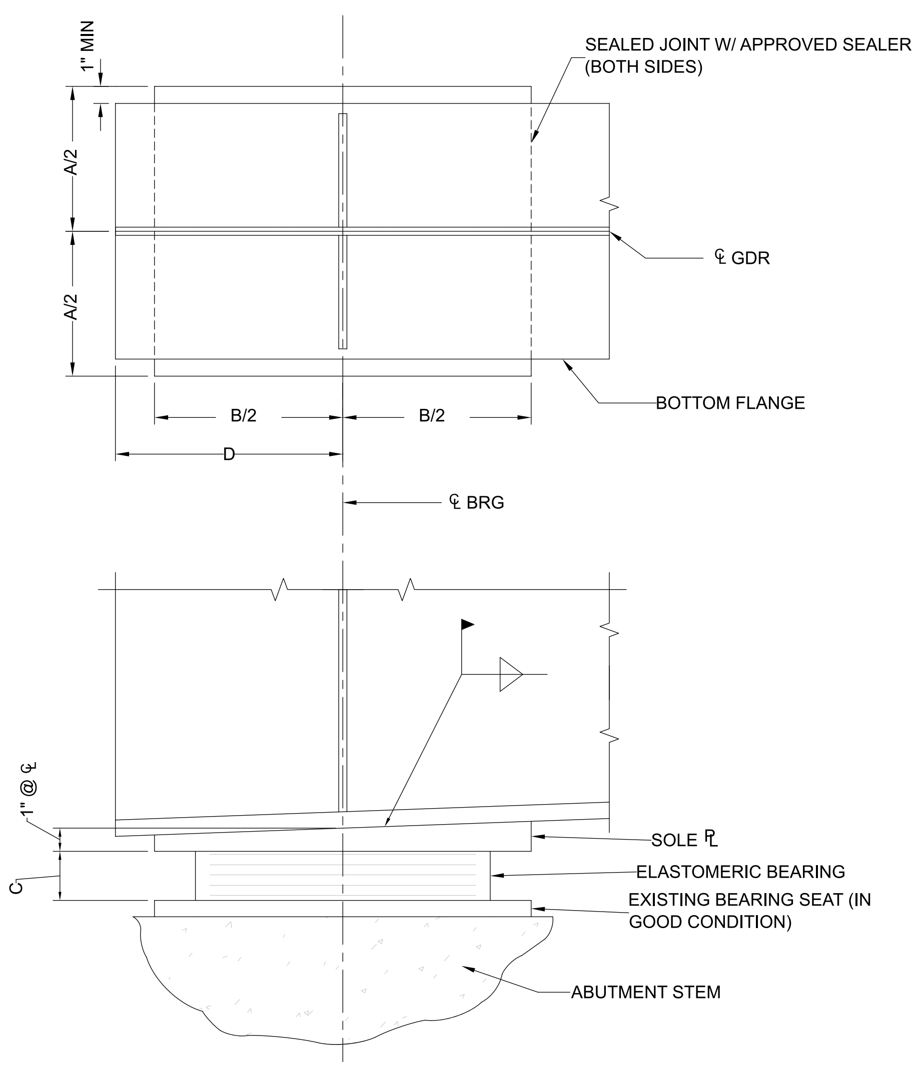
1. Care must be exercised with the field welding. The temperature of the steel adjacent to the bearing must be kept below 250°F (120°C). Temperature crayons should be used to monitor the steel temperature during welding.



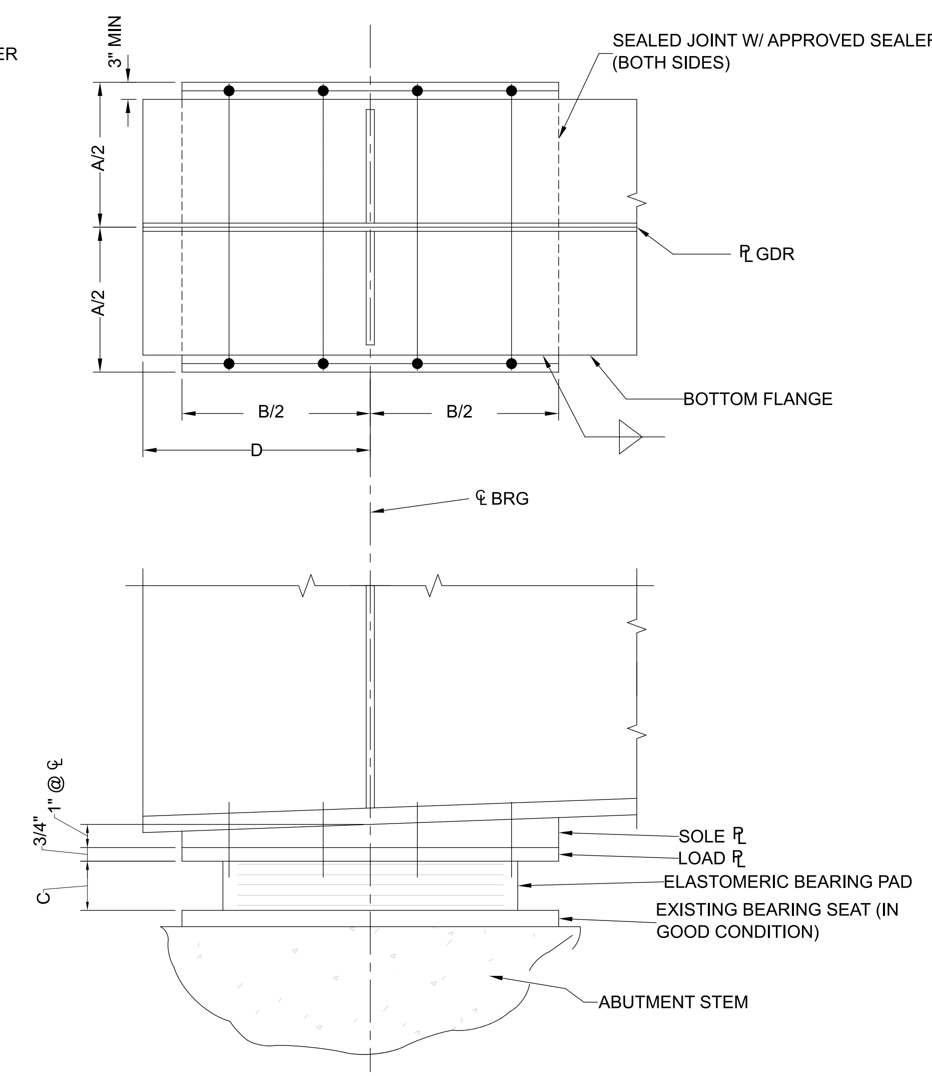
SECTION VIEW OF ELASTOMERIC BEARING
(N.T.S.)

ELASTOMETRIC BEARING DETAILS - in					
A	B	C	D	INTERNAL ELASTOMER LAYERS	
				NO. OF LAYERS	THICKNESS - in
16"	16"	3.75"	12"	5	0.5"

COMPOSITE ROLLED BEAM - 9' 8.571" GIRDER SPACING, LIGHTEST WEIGHT



BEARING ELEVATION
OPTION "A"
(N.T.S.)



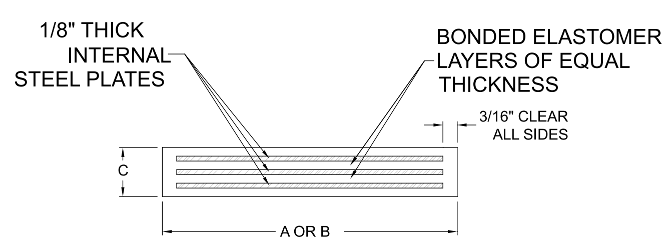
BEARING ELEVATION
OPTION "B"
(N.T.S.)

NOTES:

1. Bevel sole PL if grade exceeds ± 1%.
2. Max Grade is ± 5%.
3. Sole PL to be factory vulcanized to elastomeric bearing pad.
4. Holes to be 1 1/16" Ø in sole PL for 7/8" Ø bolt.
5. All elastomeric cover layers are 1/4" thick.

COMMENTARY:

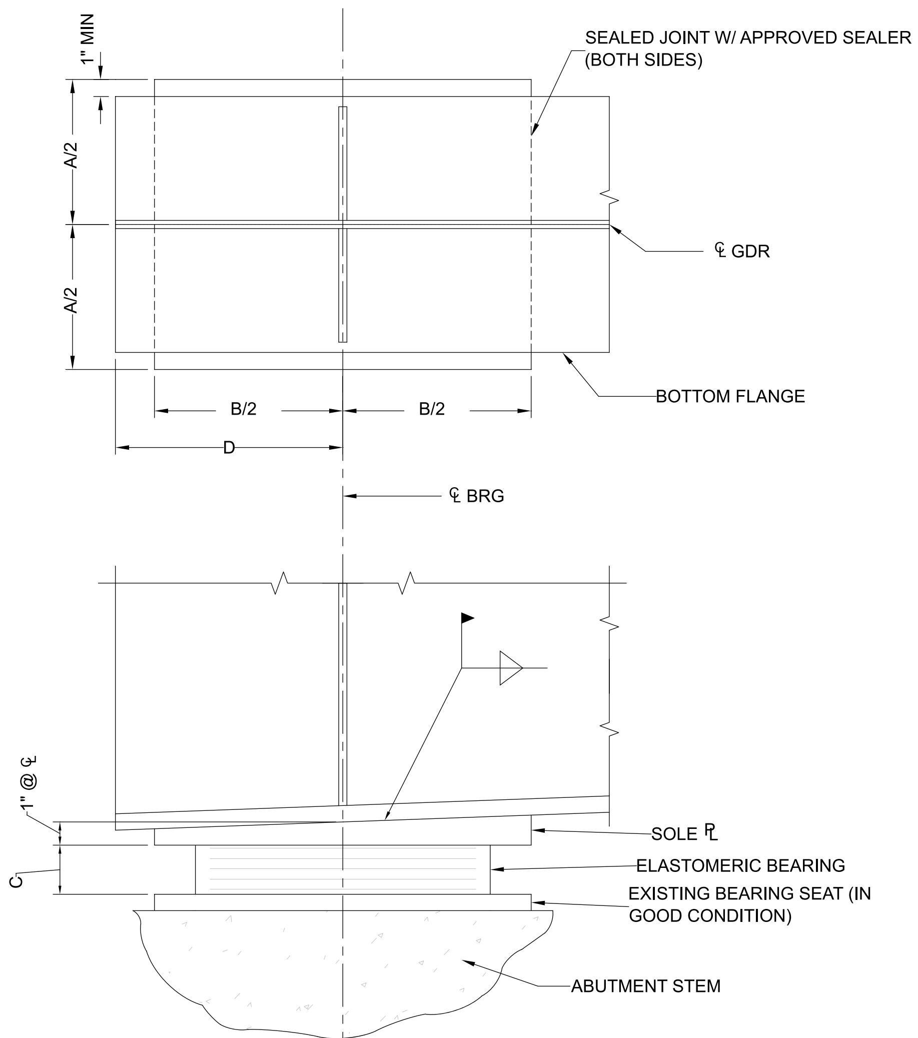
1. Care must be exercised with the field welding. The temperature of the steel adjacent to the bearing must be kept below 250°F (120°C). Temperature crayons should be used to monitor the steel temperature during welding.



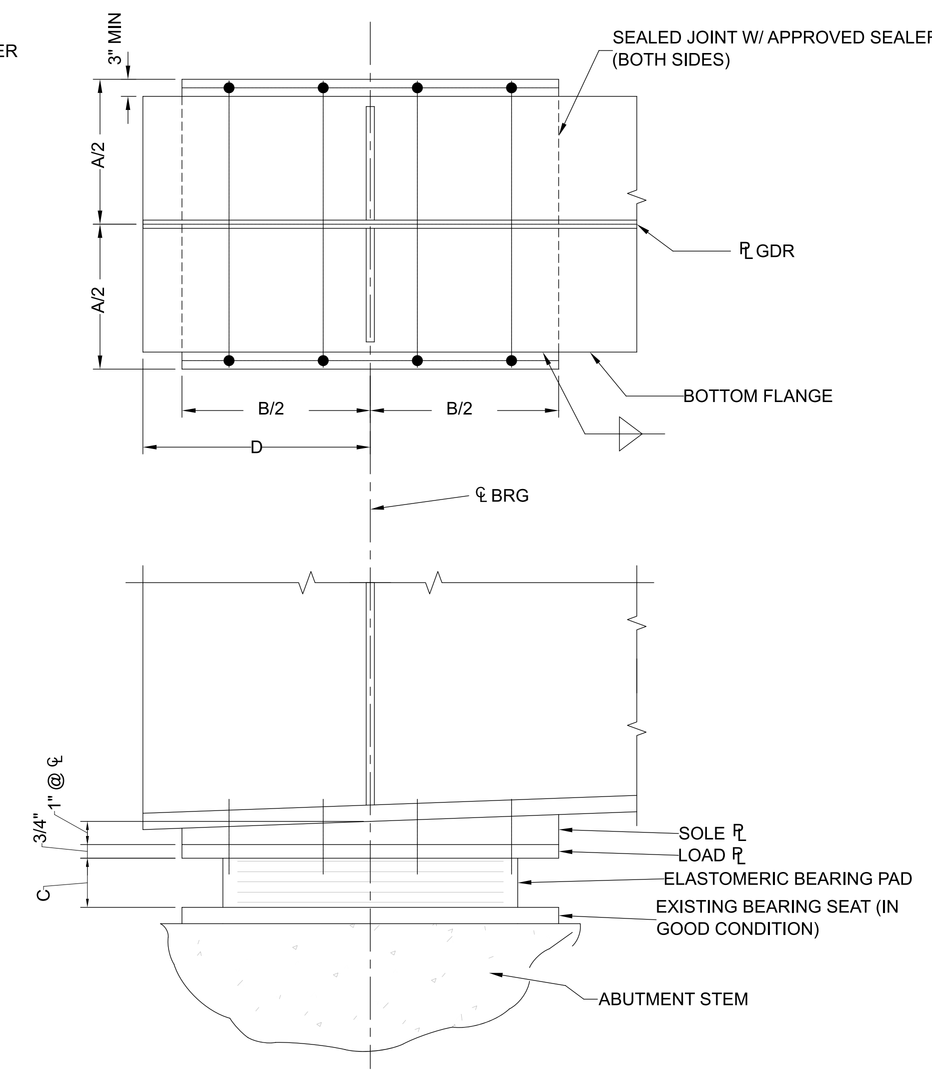
SECTION VIEW OF ELASTOMERIC BEARING
(N.T.S.)

ELASTOMETRIC BEARING DETAILS - in					
A	B	C	D	INTERNAL ELASTOMER LAYERS	
				NO. OF LAYERS	THICKNESS - in
14"	18"	3.75"	12"	5	0.5"

COMPOSITE ROLLED BEAM - 9' 8.571" GIRDER SPACING, LIMITED DEPTH



BEARING ELEVATION
OPTION "A"
(N.T.S.)



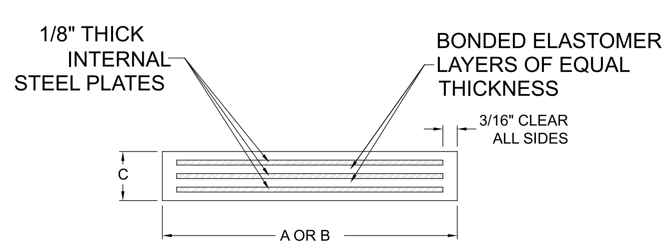
BEARING ELEVATION
OPTION "B"
(N.T.S.)

NOTES:

1. Bevel sole ℙ if grade exceeds ± 1%.
2. Max Grade is ± 5%.
3. Sole ℙ to be factory vulcanized to elastomeric bearing pad.
4. Holes to be 1 1/16” Ø in sole ℙ for 7/8” Ø bolt.
5. All elastomeric cover layers are 1/4” thick.

COMMENTARY:

1. Care must be exercised with the field welding. The temperature of the steel adjacent to the bearing must be kept below 250°F (120°C). Temperature crayons should be used to monitor the steel temperature during welding.

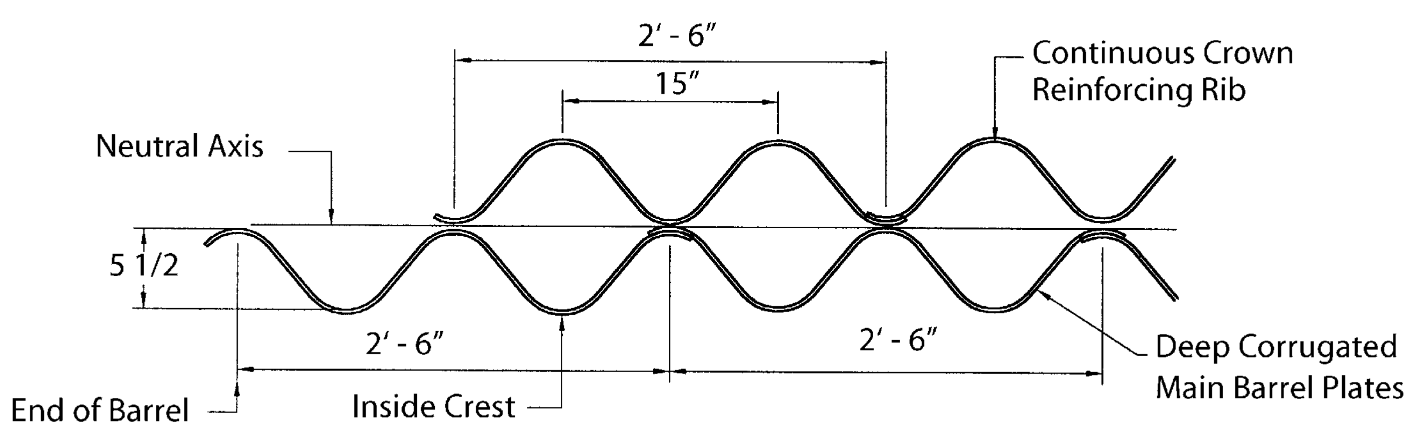


SECTION VIEW OF ELASTOMERIC BEARING
(N.T.S.)

ELASTOMETRIC BEARING DETAILS - in					
A	B	C	D	INTERNAL ELASTOMER LAYERS	
				NO. OF LAYERS	THICKNESS - in
14"	18"	3.75"	12"	5	0.5"

Standard Design and Details of Corrugated Steel Pipe and Structural Plate Solutions

Single-Radius Arch 15x5.5



SPAN - ft - in	RISE - ft - in	WATERWAY AREA - ft^2	RADIUS - in	
			Rt	Rs
78' 9"	39' 6"	2448'	39' 4"	93

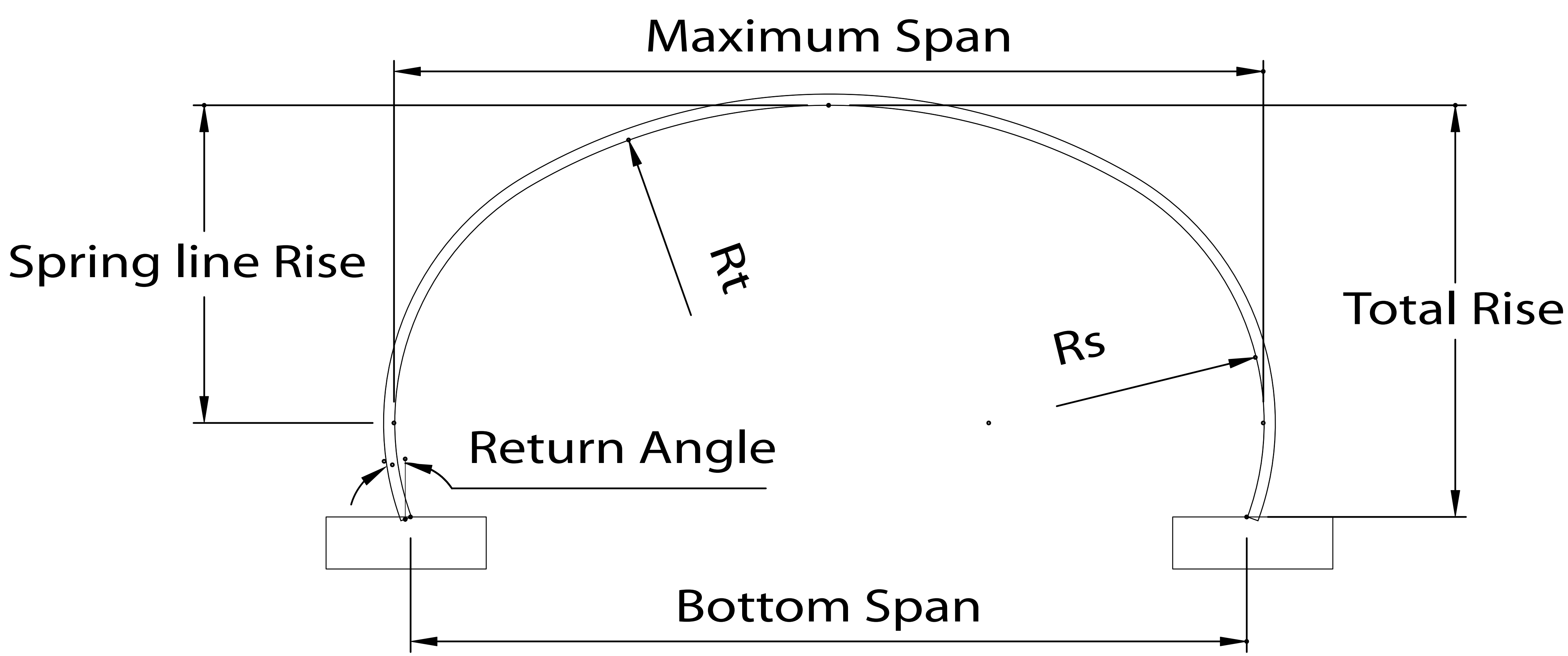
MINIMUM COVER

For specific details on minimum height of cover requirements for this gauge, profile, and shape, please contact the NCSPA.

MAXIMUM COVER

For specific details on maximum height of cover requirements for this gauge, profile, and shape, please contact the NCSPA.

Multi-Radius Arch 15x5.5 - Solution 1



MINIMUM COVER

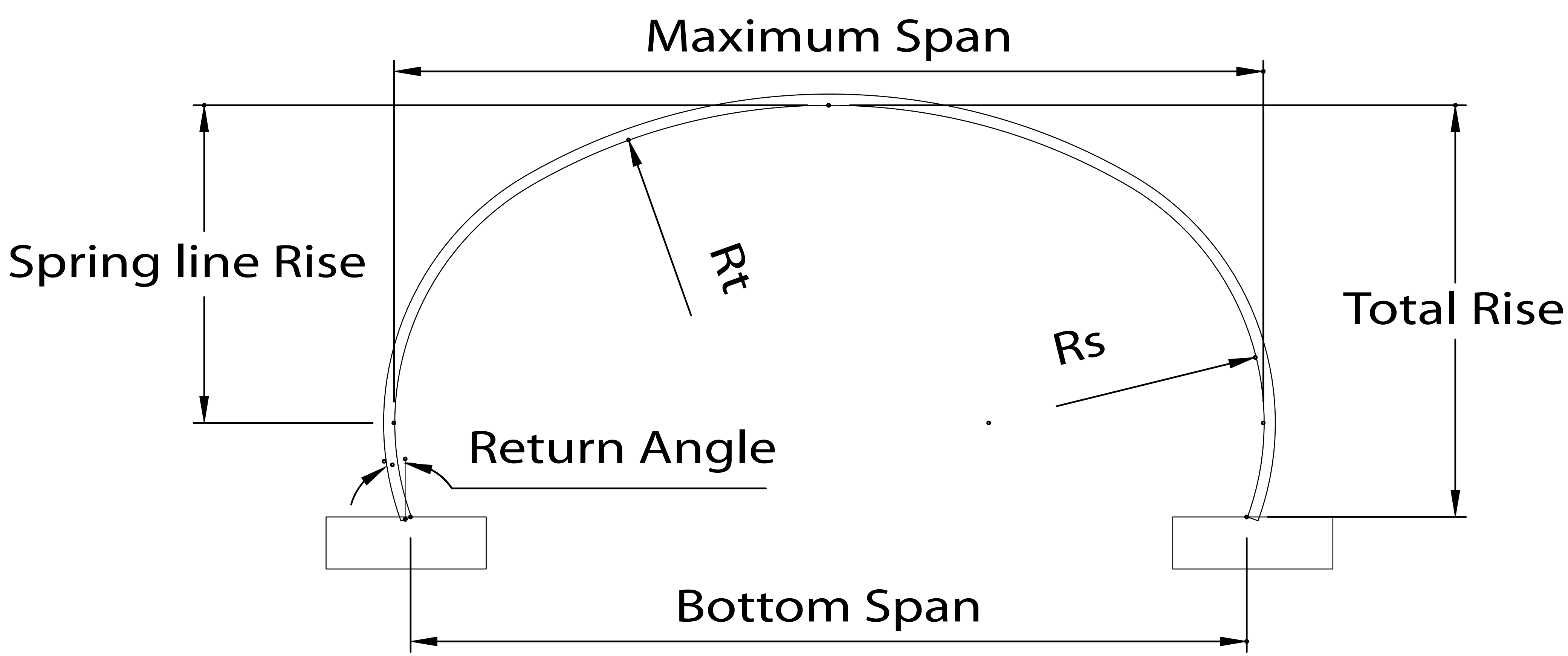
For specific details on minimum height of cover requirements for this gauge, profile, and shape, please contact the NCSPA.

MAXIMUM COVER

For specific details on maximum height of cover requirements for this gauge, profile, and shape, please contact the NCSPA.

SPAN - ft - in	RISE - ft - in	BOTTOM SPAN - ft - in	WATERWAY AREA - ft²	RADIUS - in		RETURN ANGLE
				Rt	Rc	
75' 5"	22' 10"	75' 1"	1394.2'	745"	174"	8.9

Multi-Radius Arch 15x5.5 - Solution 2



MINIMUM COVER

For specific details on minimum height of cover requirements for this gauge, profile, and shape, please contact the NCSPA.

MAXIMUM COVER

For specific details on maximum height of cover requirements for this gauge, profile, and shape, please contact the NCSPA.

SPAN - ft - in	RISE - ft - in	BOTTOM SPAN - ft - in	WATERWAY AREA - ft²	RADIUS - in		RETURN ANGLE
				Rt	Rc	
75' 5"	29' 3"	74' 11"	1837.1'	745"	273"	8.9

Manufacturer's Steel Solutions - Customized Solutions from Members of the Short Span Steel Bridge Alliance

BIG R BRIDGE

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Big R Bridge is a world leader in developing innovative engineered solutions in Prefabricated Bridges, Structural Plate, MSE Wall Systems and Corrugated Pipe for the transportation, public works, railway, mining, forestry and development sectors. By design, our custom infrastructure solutions are easy to ship and install with minimal equipment and labor requirements, making them ideal even in remote locations. Big R's Technical Sales Representatives and Engineers are well-positioned to ensure your project's success through every phase. With product innovation, in-house engineering strength

Vehicular Modular Bridges

Vehicular Modular Bridges

As the name suggests, these bridges are manufactured and shipped in modular sections that allow for rapid installation. Using equipment on hand, local crews can typically place the superstructure in one day – reducing costs and road closure time. Superstructures can be fabricated with both square and skewed ends to suit any site conditions. We also offer Portable Detour Bridges.

- Strong: able to withstand heavy-duty loading
- 8' wide modules are typical
- 4.25" corrugated steel deck (galvanized) is standard
- Decking options – poured or precast concrete, asphalt, grating, wood or gravel
- Weathering, galvanized or painted structural steel
- Curb or rail system
- Sidewalks and utility corridors can be added to enhance use

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Vehicular Modular Bridges

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TrueNorth Steel designs reliable steel Structures, Tanks, Corrugated Pipe and Bridges you can count on to always be on time and delivered to the highest quality standards.

TrueNorth Steel Bridges & Corrugated Steel Pipe

Built to AASHTO specifications TrueNorth Steel Bridge provides safe passage for pedestrians and all types of vehicles. With decades of bridge building experience, we've developed a design-build, bolt-together system that blends flexibility with standardization, so we can design a bridge for each unique application, while delivering safety, durability and easy installation. In addition to the bridges we offer pre-engineered and pre-fabricated SuperSill's and Back-Walls to simplify and reduce abutment construction and design costs.

TrueNorth Steel Corrugated Pipe has been a critical part of north America's evolving infrastructure for more than six decades. Our corrugated steel pipe offers tremendous durability and stability for casing, architecture and nearly and drainage and water flow application.

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TrueNorth Steel designs reliable steel Structures, Tanks, Corrugated Pipe and Bridges you can count on to always be on time and delivered to the highest quality standards.

TrueNorth Steel Bridges & Corrugated Steel Pipe

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Wheeler's Steel Fabrication Division is an extension of the experience gained by 100+ years of designing & supplying bridge materials. We have a staff of Professional Engineers & drafters who provide detailed plans specific to each project. Wheeler maintains AISC certification for Simple and Major Steel Bridges. Prefabricated bridge kits provide rapid construction for recreation & vehicular applications. The bridges are shop manufactured, detailed & shipped to site ready for installation.

Steel Stringer Vehicle Bridges Utilizing Transverse Treated Timber Deck Panels

Treated timber deck panels provide a versatile option as prefabricated bridge components. The deck panels are a good compliment to steel stringer superstructures. The combination results in a complete bridge kit. All components are shop fabricated ready for installation.
The deck panels can be designed for all loading conditions (ie. HS20, HS25, HL93, U80, U102). The panel thickness is based on loading condition and stringer spacing.
The deck panels are custom detailed to the specific application. Individual deck laminae are fabricated and pressure treated before being assembled into the panels. This enhances the long term durability of the deck system. Multiple attachment systems can be used to connect the panels to the steel stringers. As they are installed the panels are interconnected to provide load transfer improving the performance of an asphalt overlay wear surface.
Crash-tested timber railing kits attach directly to the deck panels. Pedestrian railings are available.

Advantages:
Shipped as a kit
Components are largely preassembled and sized for easy handling
Shop fabricated to control quality
Speeds installation at the site
Accepts traffic immediately after installation
Not temperature sensitive, no curing time
Ideal for remote sites
Treatment is water resistant, not susceptible to damage from road salt
Multiple wear surface options including asphalt
Compatible with crash-tested railing system

Wheeler provides complete superstructure plans for all projects supplied. All hardware is included. Foundation designs are available depending on site conditions.

Contact us for project specific pricing and application advice.

Set your project apart with a bridge from Wheeler.

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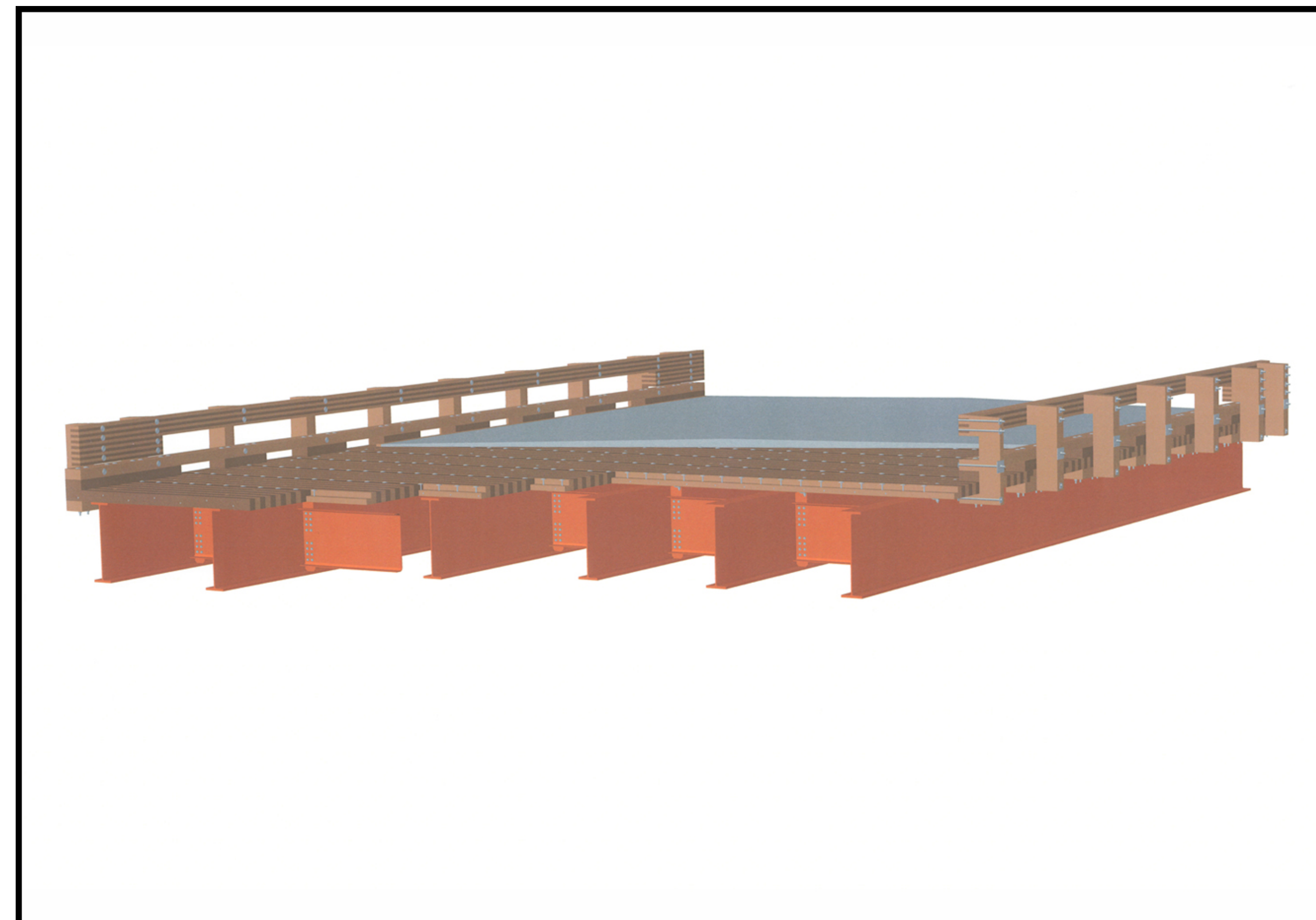

Durability Solutions

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Founded in 1935, the American Galvanizers Association (AGA) is a non-profit trade association dedicated to serving the needs of fabricators, architects, specifiers, and engineers, providing technical support on today's innovative applications and state-of-the-art technological developments in hot-dip galvanizing for corrosion control. The AGA maintains a large technical library, provides multimedia seminars, and offers a toll-free technical support hotline to assist specifiers in North America.

Hot-Dip Galvanizing

The Process

Hot-dip galvanizing (HDG) is the process whereby fabricated steel, structural steel, or small parts, including fasteners, are immersed in a kettle or vat of molten zinc, resulting in a metallurgically bonded alloy coating that protects the steel from corrosion.Galvanizing forms a metallurgical bond between the zinc and the underlying steel or iron, creating a barrier that is part of the metal itself. During galvanizing, the molten zinc reacts with the surface of the steel or iron article to form a series of zinc/iron alloy layers actually harder than the substrate steel it is protecting. The galvanizing process naturally produces coatings that are at least as thick at the corners and edges as the coating on the rest of the article. Because the galvanizing process involves total immersion of the material, all surfaces are coated.

How Hot-Dip Galvanizing Works

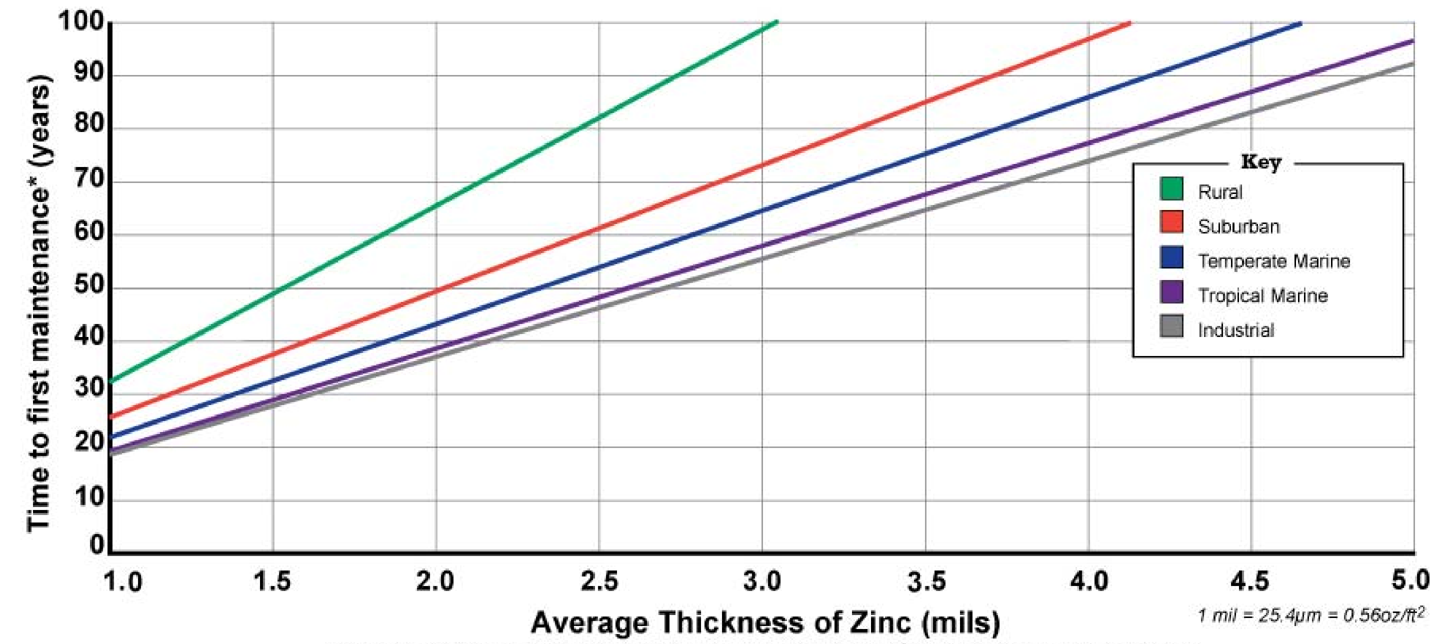
Galvanizing takes place in a factory regardless of weather or humidity conditions and is available 24/7/365 in close proximity to most new bridge locations. Freshly galvanized steel progresses through a natural weathering process to develop a corrosion resistant patina made up of zinc-oxide, zinc-hydroxide, and zinc carbonate. Typically, it takes approximately 6-12 months to fully develop. Because the corrosion rate of zinc is approximately 20 times less than that for black steel, the HDG coating has durability beyond the intended life of most steel structures. The chart below shows the typical time to first maintenance for bridges located in five different environmental exposures.

Economics and Life-cycle Cost

HDG is typically very similar and often lower in initial cost than most other corrosion protection systems considered for steel bridges and because it requires zero maintenance for 75 years or more, the life-cycle cost is typically 4 to 8 times less.

Natural and Sustainable Zinc

Zinc is found everywhere in daily life: in every cell of the human body, in the earth, in food and in products consumer products (sunblock, automobiles, cosmetics, airplanes, appliances, surgical tools, zinc lozenges). Children need zinc for growth and adults need zinc for reproduction and good health.



Time to first maintenance is defined as the time to 5% rusting of the steel surface.

The U.S. Recommended Daily Allowance is 15 milligrams of zinc.Zinc is 100% recyclable and over 80% of the zinc available for recycling is currently recycled. For more information, click on http://www.galvanizeit.org/about-hot-dip-galvanizing/is-hdg-sustainable/

Bridge Projects

HDG is commonly used on short-span bridges, especially when the bridge will be located in relatively corrosive environments such as above rivers and streams and in humid climates. To view examples of bridges utilizing HDG steel, click on http://galvanizeit.org/project-gallery/gallery (and select "sector" and then "Bridge &Highway)

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Paint

Overview

Constructing bridges extends back thousands of years. In a relative sense steel bridge construction is in its infancy. The first iron bridge was built in 1779, while the first steel was used in a bridge in 1828. Coated bridges from the 19th century survive today.

Corrosion protection via coatings is also an interrelated subject; i.e., in order to lower longer term costs, coatings can be efficiently applied when new steel is in the fabrication shop.

Design Phase

A comprehensive plan for successful corrosion protection and mitigation is needed from the inception of the project and consists of actions which continue throughout the life of the bridge. A plan is needed which includes decisions made during the bridge design process. During this time, the site for the structure is identified. The extent of exposure to any detrimental environmental conditions should drive certain corrosion prevention design choices, such as: type of bridge, type of steel used; the details utilized in developing the shape of members, types of secondary members, and their connections are but a few. The clearance and exposure beneath the structures must be carefully planned. In this process, a corrosion protection plan which provides long-term protection is devised.

Corrosion mitigation choices may vary somewhat; however, for exposure in corrosion prone areas of the country , the use of zinc on bare steel should be considered. American Galvanizers Association (AGA) at 75 years in a severe environment with no paint topcoat. SSPC: The Society For protective Coatings (www.sspc.org) publishes and maintains standards for surface preparation of steel and for the various zinc rich coatings. The American Galvanizers Association (AGA.galvaniteit.org) has information about the uses of galvanizing.

Appropriately selected and applied layers or coats of paint over the hot dip galvanize or thermal spray applied zinc or zinc-rich paint will extend the service life of the corrosion protection. The AGA refers to these as duplex systems.

Since the first use of zinc rich primer coated steel in the late 1960's thousands of zinc-rich primer coated steel bridges have survived for almost 50 years. These bridges are distributed across the country and are examples of permanently installed corrosion protection.

Installation

Even with a properly selected system to address the most challenging exposure, a system must still be correctly installed, and the bridge must be built and maintained! SSPC has detailed information about installation practices, specification and conducts training classes.

Maintenance Completes the Process

The corrosion control effort begins with a comprehensive "corrosion review and planning" prior to and during the bridge design process. Implementation of the plan during detailing and fabrication of the steel, application of the selected coating system, shipping, erection, field painting, touch-up, and performing critical steps identified in a proper maintenance plan are the necessary items in the corrosion protection and mitigation plan. If the planning or maintenance are skipped, we are choosing to save resources in the short term, but in doing so we are consigning ourselves to pay more later on in earlier repairs.

As a reference, please see the included photo of untopcoated Zinc rich paint on the Golden Gate Bridge after 45+ years. (Photo used with permission from Golden Gate Bridge, www.goldengate.org)

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Weathering Steel

The following information is an excerpt from the National Steel Bridge Alliance's Steel Bridge Design Book. Please visit http://www.aisc.org/contentNSBA.aspx?id=20244 for the complete book.

Weathering steel is an important option for the bridge designer. The FHWA Technical Advisory T5140.22 "Uncoated Weathering Steel in Structures" - http://www.fhwa.dot.gov/bridge/t514022.cfm provides guidance to the states for development of their own policies regarding the use of weathering steel. This document contains a digest of the primary benefits and concerns regarding weathering steel and provides specific guidance on its appropriate use. Written in 1989, the document is undergoing a review and rewrite; however, the majority of its content remains useful as a starting point.

Weathering grade steels have been available for several decades. They have been produced in various proprietary chemistries; but essentially small amounts of copper, chromium, nickel and silicon are added to carbon steel to achieve an alloy with enhanced weathering properties. These steels will form a rust patina when exposed to the environment providing a barrier between the bare steel and the corrosive elements of the environment. When properly detailed and exposed to environments that include cyclic wet/dry exposures and do not introduce significant amounts of corrosive contaminants to the steel surface, this tightly adherent patina provides a weathering steel structure with its own protective coating that slows the self-corrosion rate of the steel to a very low rate.
Although highway bridges were not the first industrial application of weathering grade steels, they have been the primary market for the material since the first weathering steel bridges were built in the mid 1960's.

The primary benefit of weathering steel is the promise of long-term corrosion protection without the need for either initial or maintenance painting. The steel industry has made the point that weathering steel, when properly applied, results in a structure that provides first cost and life cycle cost savings. However, due to the assumption that all bridge expansion joints will eventually leak, current guidelines require weathering steel bridge elements to be painted at non-integral beam ends to a length of 1.5 times the girder depth. In addition, weathering steel girders are shop blasted to remove millscale so that the initial protective oxide layer is uniform. These requirements offset some of the potential cost savings associated with weathering steel versus painted or galvanized steel.

Extensive data exists regarding the corrosion performance of weathering steels. The following highlights conclusions taken from the pertinent data:

- Weathering steel requires some amount of moisture and a wet/dry weathering cycle over a period of time to develop a tightly adherent, protective oxide layer. However, excessive moisture or the presence of salt will disrupt this process and result in a structure that corrodes at an unacceptable (much higher) rate.

- Nearly all of the reported failures of weathering steel on bridges have occurred in applications where the steel is wet for a significant portion of time or the steel is exposed to salt from the ocean or deicing operations.

- Properly functioning weathering steel will corrode at a steady-state rate less than 0.3 mils per year (7.5 microns per year). Corrosion in excess of this rate indicates that weathering steel should not be used bare at that location.

Short Span Steel Bridge Alliance Member Contact Information

Short Span Steel Bridge Alliance Members

The following members of the Short Span steel Bridge Alliance are available to assist you with questions or information in regards to short span bridges.

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The following members of the Short Span steel Bridge Alliance are available to assist you with questions or information in regards to short span bridges.

Service Center

Infra-Metals Company John Lusdyk *Executive VP Commercial* www.Infra-Metals.com	580 Middletown Blvd Suite D-100 Langhorne, PA 19047 609-937-1600 johnl@infra-metals.com
Metals USA Jim Collins *Vice President Sales* www.MetalsUSA.com	2400 East Commercial Blvd Suite 905 Fort Lauderdale, FL 33308 954-202-4000 JCollins@metalsusa.com
Triple-S Steel Holdings, Inc. Kevin Dempsey *Director, Business Development* http://sss-steel.com/	6000 Jensen Drive Houston, TX 77026 (713) 697- 7105 Kevin.m.dempsey@sss-steel.com

The following members of the Short Span steel Bridge Alliance are available to assist you with questions or information in regards to short span bridges.

Universities

University of Wyoming Michael Barker *Professor, Civil and Architectural Engineering* www.uwyo.edu	1000 E University Ave P.O. Box 3295 Laramie, WY 82071 307-766-2916 barker@uwyo.edu
West Virginia University Karl Barth *SSSBA Consultant West Virginia University* www.cemr.wvu.edu	P.O. Box 6103 Morgantown, WV 26506 304-293-9921 kebarth@mail.wvu.edu

Appendix 2.3: Composite Design

This appendix contains the calculations used to complete the composite steel preliminary designs. This design represented one of the two preliminary designs completed.

Composite Standard Beam and Girder Design

This file contains the standard method of calculations for the design of steel beams and girders for composite action to support the bridge. This calculation covers a standard method of designing a beam for the dynamic AASHTO loadings.

Plate Girder Design Exterior 75ft continuous span (- moment)

This section contains composite design calculations for the two-span continuous framing situations, computed for the design of an exterior beam for negative moment.

Plate Girder Design Exterior 75ft continuous span (+ moment)

This section contains composite design calculations for the two-span continuous framing situations, computed for the design of an exterior beam for positive moment.

Plate Girder Design Interior 75ft continuous span (- moment)

This section contains composite design calculations for the two-span continuous framing situations, computed for the design of an interior beam for negative moment.

Plate Girder Design Interior 75ft continuous span (+ moment)

This section contains composite design calculations for the two-span continuous framing situations, computed for the design of an interior beam for positive moment.

Plate Girder Design Exterior 150ft span final

This section contains composite design calculations for the single-span framing situations, computed for the design of an exterior beam for positive moment.

Plate Girder Design Interior 150ft span final

This section contains composite design calculations for the single-span framing situations, computed for the design of an interior beam for positive moment.

Composite Standard Beam and Girder Design

Top flange width:	15				
Top flange depth:	1	Top flange A:	15		
Web width:	0.9	Web A:	54	As:	93
Web depth:	60	Bot. flange A:	24		
Bot. flange width:	24	$\bar{y}$:	28.04839		
Bot. flange depth:	1	Span width (ft):	74		
N_b:	8				
fc':	7000	β_1:	0.7		
fy:	50000				

Eff. Flange Width

Dist. from support:	900	b_e values:	225	b_e:	58.2855 in
Beam spacing:	116.571		58.2855		
Distance:*	116.571		233.142		
Distance from edge*:	36.0015				
a:	13.40835379	PNA Location:	**PNA in Top**		
c:	19.15479113				
Slab thickness:	9				

*for end beams distance is from edge of slab to CL of beam, for interior beams distance is one half the spacing

PNA in Slab
Does Not Govern

C:	4650	k
T:	4650	k
Mn (k-ft):	12902.13	
ϕ:	0.9	
ϕMn (k-ft):	11611.91831	

PNA in Top Flange
Does Not Govern

C:	3885.594	k
T:	3885.594	k
$\bar{y}$:	1.019208	in
Mn (k-ft):	2849.272	
ϕMn (k-ft):	2564.345	

Beam weight (k/ft):	0.316458333		
Slab weight (int/ext):	1.092853125	0.883940625	k/ft
Barrier weight (k/ft):	0.425		
Wearing Surface (int/ext):	0.352141563	0.284825313	k/ft
DL (int/ext):	1.761453021	1.910224271	k/ft
M_{LL} (k-ft):	1528	(unfactored)	

Single Span

Exterior Beams

M_{DL} (k-ft):	5372.506
Mu (k-ft):	10359.26
ϕMn (k-ft):	11611.92

Beam length:	150
Bridge length:	150
E_D (ksi):	5072.240629
Es (ksi):	29000
e_g (in):	35.5
n:	5.717394367
Ix:	51672.78226
Kg:	965530.8755

Distribution Factors

Exterior factor:	0.485	*Using lever rule
Interior factor:	0.42862477	(1 lane loaded)
	0.643527687	(2 or more lanes loaded)
Multiple Presence:	1.2	*4 lanes

*Use larger lane factor

Notes

Units are assumed to be in lbs and in unless otherwise stated
Examples used from Chapter 16 of Structural Steel Design
For Continuous Spans, add dead loads obtained to RISA file to obtain Mu

66.66667 < 90.55279 Plastic stress distribution assumed

0.1 ≤

PNA in Web Governs

C:	3885.594	k
T:	3885.594	k
ȳ:	0.320128	in
Mn (k-ft):	14261.41	
ϕMn (k-ft):	12835.27	

*ȳ values determine location of PNA, if ȳ is negative PNA is moved towards slab

Interior Beams

M_{DL} (k-ft):	4954.087	*Adjust Mn used based on location of PNA
Mu (k-ft):	10270.75	
ϕMn (k-ft):	11611.92	

	HS20-44 (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4201.3	
Continuous	74.1	893.4	1248.2
	Tandem (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.2	1040.2
Applicable M:		4201.3	
Applicable V:		84	

Vertical Shear

				h/tw:		Cv:	
kv:	5	*If no stiffeners		66.6666667	≤	59.23681	1
		59.2368	<	66.6666667	≤	73.77676	0.888552
				66.6666667	>	73.77676	0.985275
		*If true no stiffeners are needed		66.6666667	<	59.24465	

				h/tw:		Cv:	
a (in):	4	*Including stiffeners		66.6666667	≤	890.5246	1
kv:	1130	890.525	<	66.6666667	≤	1109.108	13.35787
				66.6666667	>	1109.108	222.6722
		*If true no stiffeners are needed					

Horizontal Shear

A_{sa} (in^2):	10			Rg:	1
Qn (k):	29.79332	≤	450	Rp:	0.75
				Fu:	60

Exterior Beams

Cv:	0.888552
V_{DL} (k):	143.2668
ϕVn (k):	1295.509
Vu (k):	237.1042

*Account for DL of stiffeners if used

Exterior Beams

Horizontal Shear (k):	7522.189	
Number of Studs:	252.479	*from middle of beam to end
Rounded up #:	253	
Spacing (studs/in):	7.114625	

Interior Beams

V_{DL} (k):	132.109
ϕVn (k):	1295.51
Vu (k):	283.857

Interior Beams

Horizontal Shear (k):	9300
Number of Studs:	312.1505
Rounded up #:	313
Spacing (studs/in):	2.875399

I_{yc} (in^4):	281.25
I_{yt} (in^4):	1152

Dimensions Check *per AASHTO 6.10.2.1-2

Top Flange						Bottom Flange			
7.5	≤	12	Adequate			12	≤	12	Adequate
15	≥	10	Adequate			24	≥	10	Adequate
1	≥	0.99	Adequate			1	≥	0.99	Adequate
0.244141	≤	10	Adequate	0.1	≤	0.244141	≤	10	TRUE

With Longitudinal Stiffeners				Without Longitudinal Stiffeners			
66.66667	≤	150	Adequate	66.66667	≤	300	Adequate

* -M_u calculated using 90% of loading on structure per section 3.6.1.3 of AASHTO

Shear Distribution Factors

Interior:	0.932487	
Exterior:	0.485	*obtained using lever rule

Plate Girder Design 75' Continuous

Unfactored Moment (L	1248.2	k-ft	*obtained from Table 1
t(flange)	0.85	in	
b(flange)	14	in	
t(web)	0.325	in	
h(web)	26.7	in	
E(steel)	29000	ksi	
f'c	7	ksi	
Fy	60	ksi	
Kv	5		
Span Length	75	ft	
Effective Length	45	ft	
Tributary Width	8.857125	ft	
Slab Thickness	9	in	
Wearing Surface Thickn	3	in	
Jersey Barrier Weight	0.425	k/ft	
E(concrete)	5072.24	ksi	
Area of section	32.4775	in^2	
Area of Web	8.6775	in^2	
Steel Weight	0.111	k/ft	
Slab Weight	0.996	k/ft	
Wearing Surface Weigh	0.310	k/ft	
Wu (DL factored)	2.210	k/ft	
h/t(web)	82.1538462		must be < 260
Web Compact Check	82.66		must be > h/t(web)
Minimum t(web)	0.32		
Cv	0.54068		

Mu	1721.81	k-ft	choose from interior or exterior moments fro
Af	11.79	in^2	
minimum b(flange)	13.87	in	
b(flange)/2t(flange)	8.16		
Flange Compact Check	8.35		must be > b(flange)/2t(flange)
Ix	516.94	in^4	
Z(required)	382.63	in^3	
Z(calculated)	385.77	in^	must be > Z(required)

Longitudinal Stiffness Parameter

n	5.717394367	
Moment of Inertia	516.94	in^4
Area of Steel	32.4775	in^2
e_g	18.7	in
K_g	67888.33	

LL Distribution Factors (Interior Girders)

	Moment Factor	Shear Factor	
Interior Factor	0.796838369	0.714285	*1 design lane loaded
Interior Factor	1.125569256	0.874054025	*2 design lanes loaded

LL Distribution Factor (Exterior Girder)

Exterior Factor	0.485	*using lever rule
Multiple Prescence	1.2	*4 lanes

Interior Girder Checks

V_{LL}	280.7	k	*from RISA results
V_u	442.29	k	
ΦVn	152.01	k	*must be > Vu

M_{DL}	559.49	k-ft
Mu	2807.39	k-ft
ΦMn	1735.95	k-ft

Exterior Girder Checks

V_{LL}	74.1	k	
V_u	118.73	k	*must be > Vu
ΦVn	152.01	k	

M_{DL}	559.49	k-ft	
Mu	1721.81301	k-ft	
ΦMn	1735.95291	k-ft	*must be > Mu

* Dark gray cells indicate inputs
* Light gray cells indicate results

Table 1

	HS20-44 (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4212.4	
Continuous	74.1	893.4	1248.2
	Tandem (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.3	1040.2

Moment Checks

Yielding			Lateral-Torsional Buckling		
Fy	60	ksi	I_y	388.81	in^4
Zx	385.77	in^3	r_y	3.46000923	in
Mn	1928.84	k-ft	S_x	354.437071	in^3
			J	6.03735365	in^4
			h_0	27.55	in
			r_{ts}	3.88727072	
			L_p	133.879259	in
			L_r	348.527241	in
			L_b	116.571	in

*if < Lp lateral torsional buckling does not apply

Plate Girder Design Exterior Positive Moment

Unfactored Moment (LL)	1248.2	k-ft
t(flange)	0.79	in
b(flange)	13	in
t(web)	0.325	in
h(web)	26.7	in
E(steel)	29000	ksi
f'c	7	ksi
Fy	60	ksi
Kv	5	
Span Length	75	ft
Effective Length	40	ft
Tributary Width	9.71425	ft
Slab Thickness	9	in
Wearing Surface Thicknes	3	in
Jersey Barrier Weight	0	k/ft
E(concrete)	5072.24	ksi
Area of section	29.2175	in^2
Area of Web	8.6775	in^2
Steel Weight	0.099	k/ft
Slab Weight	1.093	k/ft
Wearing Surface Weight	0.340	k/ft
Wu (DL factored)	1.839	k/ft
h/t(web)	82.1538462	
Web Compact Check	82.66	
Minimum t(web)	0.323	
Cv	0.54068	

M_u	1530.07	k-ft
Af	10.26	in^2
minimum b(flange)	12.99	in
b(flange)/2t(flange)	8.22	
Flange Compact Check	8.35	
I_x	516.58	in^4
Z(required)	340.02	in^3
Z(calculated)	340.24	in^3

*obtained from Table 1

Longitudinal Stiffness Param

n	5.717394367
Moment of Inertia	516.58
Area of Steel	29.2175
e_g	18.64
K_g	60994.22

LL Distribution Factors (Interior

	Moment Factor
Interior Factor	0.837683959
Interior Factor	1.194095749

LL Distribution Factor (Exterior

Exterior Factor	0.485
Multiple Prescence	1.2

Interior Girder Checks

must be < 260

must be > h/t(web)

V_{LL}	74.1
V_u	147.33
ΦVn	152.01

choose from interior or exterior moments from checks

M_{DL}	367.75
Mu	2752.50
ΦMn	1531.10

Exterior Girder Checks

must be > b(flange)/2t(flange)

V_{LL}	74.1
V_u	105.78
ΦVn	152.01

must be > Z(required)

M_{DL}	367.75
Mu	1530.07
ΦMn	1531.10

ıeter

in^4
in^2
in

* Dark gray cells indicate inputs
* Light gray cells indicate results

Table 1

	HS20-44 (Service)	
	V_u (k)	M_u (k-ft)
Single Span	84	4212.4
Continuous	74.1	893.4
	Tandem (Service)	
	V_u (k)	M_u (k-ft)
Single Span	73	3625
Continuous	71.6	599.3

Girders)

Shear Factor	
0.74857	*1 design lane loaded
0.932486831	*2 design lanes loaded

Girder)

*using lever rule

*4 lanes

k	*from RISA results
k	
k	*must be > Vu

k-ft
k-ft
k-ft

k	
k	
k	*must be > Vu

k-ft	
k-ft	
k-ft	*must be > Mu

Moment Checks

	Yielding	
Fy	60	ksi
Zx	340.24	in^3
Mn	1701.22	k-ft

$-M_u$ (k-ft)
1248.2
$-M_u$ (k-ft)
1040.2

Lateral-Torsional Buckling

I_y	289.35	in^4
r_y	3.14694209	in
S_x	310.96866	in^3
J	4.57852498	in^4
h_0	27.49	in
r_{ts}	3.57622092	
L_p	121.76565	in
L_r	317.177908	in
L_b	116.571	in

*if < Lp lateral torsional buckling does not apply

Interior Plate Girder Design 75' Continuous Negative Moment

Unfactored Moment (LL)	1248.2	k-ft	*obtained from Table 1
t(flange)	1.13	in	
b(flange)	19	in	
t(web)	0.325	in	
h(web)	26.7	in	
E(steel)	29000	ksi	
f'c	7	ksi	
Fy	60	ksi	
Kv	5		
Span Length	75	ft	
Effective Length	40	ft	
Tributary Width	9.71425	ft	
Slab Thickness	9	in	
Wearing Surface Thickness	3	in	
Jersey Barrier Weight	0	k/ft	
E(concrete)	5072.24	ksi	
Area of section	51.6175	in^2	
Area of Web	8.6775	in^2	
Steel Weight	0.176	k/ft	
Slab Weight	1.093	k/ft	
Wearing Surface Weight	0.340	k/ft	
Wu (DL factored)	1.930	k/ft	
h/t(web)	82.1538462		must be < 260
Web Compact Check	82.66		must be > h/t(web)
Minimum t(web)	0.323		
Cv	0.54068		

M_u	2905.01	k-ft	choose from interior or exterior moments from che
Af	21.12	in^2	
minimum b(flange)	18.69	in	
b(flange)/2t(flange)	8.27		
Flange Compact Check	8.35		must be > b(flange)/2t(flange)
I_x	520.08	in^4	
Z(required)	645.56	in^3	
Z(calculated)	655.43	in^3	must be > Z(required)

Longitudinal Stiffness Parameter

n	5.717394367	
Moment of Inertia	520.08	in^4
Area of Steel	51.6175	in^2
e_g	18.98	in
K_g	109286.77	

LL Distribution Factors (Interior Girders)

	Moment Factor	Shear Factor	
Interior Factor	0.884386802	0.74857	*1 design lane loaded
Interior Factor	1.261301652	0.932486831	*2 design lanes loaded

LL Distribution Factor (Exterior Girder)

Exterior Factor	0.485	*using lever rule
Multiple Prescence	1.2	*4 lanes

Interior Girder Checks

V_{LL}	74.1	k	*from RISA results
V_u	149.16	k	
ΦVn	152.01	k	*must be > Vu

M_{DL}	386.04	k-ft
Mu	2905.01	k-ft
ΦMn	2949.45	k-ft

Exterior Girder Checks

V_{LL}		k	
V_u	38.60	k	*must be > Vu
ΦVn	152.01	k	

M_{DL}	386.04	k-ft	
Mu	1548.362582	k-ft	*must be > Mu

* Dark gray cells indicate inputs
* Light gray cells indicate results

Table 1

	HS20-44 (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4212.4	
Continuous	74.1	893.4	1248.2
	Tandem (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.3	1040.2

Moment Checks

Yielding			Lateral-Torsional Buckling		
Fy	60	ksi	I_y	1291.85	in^4
Zx	655.43	in^3	r_y	5.00274485	in
Mn	3277.16	k-ft	S_x	610.112624	in^3
			J	18.5822156	in^4
			h_0	27.83	in
			r_{ts}	5.42804497	
			L_p	193.572828	in
			L_r	517.599195	in
			L_b	116.571	in

*if < Lp lateral torsional buckling does not apply

Plate Girder Design Interior 75' Continous Positive Moment

Unfactored Moment (LL)	1248.2	k-ft	*obtained from Table 1
t(flange)	1.13	in	
b(flange)	19	in	
t(web)	0.325	in	
h(web)	26.7	in	
E(steel)	29000	ksi	
f'c	7	ksi	
Fy	60	ksi	
Kv	5		
Span Length	75	ft	
Effective Length	40	ft	
Tributary Width	9.71425	ft	
Slab Thickness	9	in	
Wearing Surface Thickness	3	in	
Jersey Barrier Weight	0	k/ft	
E(concrete)	5072.24	ksi	
Area of section	51.6175	in^2	
Area of Web	8.6775	in^2	
Steel Weight	0.176	k/ft	
Slab Weight	1.093	k/ft	
Wearing Surface Weight	0.340	k/ft	
Wu (DL factored)	1.930	k/ft	
h/t(web)	82.1538462		must be < 260
Web Compact Check	82.66		must be > h/t(web)
Minimum t(web)	0.323		
Cv	0.54068		

M_u	2905.01	k-ft	choose from interior or exterior moments
Af	21.12	in^2	
minimum b(flange)	18.69	in	
b(flange)/2t(flange)	8.27		
Flange Compact Check	8.35		must be > b(flange)/2t(flange)
I_x	520.08	in^4	
Z(required)	645.56	in^3	
Z(calculated)	655.43	in^3	must be > Z(required)

Longitudinal Stiffness Parameter

n	5.717394367	
Moment of Inertia	520.08	in^4
Area of Steel	51.6175	in^2
e_g	18.98	in
K_g	109286.77	

LL Distribution Factors (Interior Girders)

	Moment Factor	Shear Factor	
Interior Factor	0.884386802	0.74857	*1 design lane loaded
Interior Factor	1.261301652	0.932486831	*2 design lanes loaded

LL Distribution Factor (Exterior Girder)

Exterior Factor	0.485	*using lever rule
Multiple Prescence	1.2	*4 lanes

Interior Girder Checks

V_{LL}	74.1	k	*from RISA results
V_u	149.16	k	
ΦVn	152.01	k	*must be > Vu

M_{DL}	386.04	k-ft
Mu	2905.01	k-ft
ΦMn	2949.45	k-ft

Exterior Girder Checks

V_{LL}		k	
V_u	38.60	k	*must be > Vu
ΦVn	152.01	k	

M_{DL}	386.04	k-ft	
Mu	1548.362582	k-ft	*must be > Mu

* Dark gray cells indicate inputs
* Light gray cells indicate results

Table 1

	HS20-44 (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4212.4	
Continuous	74.1	893.4	1248.2
	Tandem (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.3	1040.2

Moment Checks

Yielding			Lateral-Torsional Buckling		
Fy	60	ksi	I_y	1291.85	in^4
Zx	655.43	in^3	r_y	5.00274485	in
Mn	3277.16	k-ft	S_x	610.112624	in^3
			J	18.5822156	in^4
			h_0	27.83	in
			r_{ts}	5.42804497	
			L_p	193.572828	in
			L_r	517.599195	in
			L_b	116.571	in

*if < Lp lateral torsional buckling does not apply

Plate Girder Design Exterior 150'

Unfactored Moment (LL)	4212.4	k-ft	*obtained from Table 1
t(flange)	1.85	in	
b(flange)	31	in	
t(web)	0.41	in	
h(web)	33.8	in	
E(steel)	29000	ksi	
f'c	7	ksi	
Fy	60	ksi	
Kv	5		
Span Length	150	ft	
Effective Length	150	ft	
Tributary Width	6	ft	
Slab Thickness	8	in	
Wearing Surface Thicknes	3	in	
Jersey Barrier Weight	0.425	k/ft	
E(concrete)	5072.24	ksi	
Area of section	128.558	in^2	
Area of Web	13.858	in^2	
Steel Weight	0.437	k/ft	
Slab Weight	0.600	k/ft	
Wearing Surface Weight	0.210	k/ft	
Wu (DL factored)	2.007	k/ft	
h/t(web)	82.43902		must be < 260
Web Compact Check	82.66		must be > h/t(web)
Minimum t(web)	0.41		
Cv	0.53694		

M_u	9567.12	k-ft	choose from interior or exterior moments from checks
Af	56.35	in^2	
minimum b(flange)	30.46	in	
b(flange)/2t(flange)	8.23		
Flange Compact Check	8.35		must be > b(flange)/2t(flange)
I_x	1352.04	in^4	
Z(required)	2126.03	in^3	
Z(calculated)	2161.63	in^3	must be > Z(required)

Longitudinal Stiffness Parameter

n	5.717394367	
Moment of Inertia	1352.04	in^4
Area of Steel	128.558	in^2
e_g	22.75	in
K_g	388147.28	

LL Distribution Factors (Interior Girders)

	Moment Factor	Shear Factor	
Interior Factor	0.584356994	0.6	*1 design lane loaded
Interior Factor	0.845669846	0.670612245	*2 design lanes loaded

LL Distribution Factor (Exterior Girder)

Exterior Factor	0.485	*using lever rule
Multiple Prescence	1.2	*4 lanes

Interior Girder Checks

V_{LL}	280.7	k	*from RISA results
V_u	451.71	k	
ΦVn	241.09	k	*must be > Vu

M_{DL}	5644.53	k-ft
Mu	11344.21	k-ft
ΦMn	9727.32	k-ft

Exterior Girder Checks

V_{LL}	84	k	
V_u	228.74	k	*must be > Vu
ΦVn	241.09	k	

M_{DL}	5644.53	k-ft	
Mu	9567.12	k-ft	*must be > Mu
ΦMn	9727.32	k-ft	

* Dark gray cells indicate inputs
* Light gray cells indicate results

Table 1

	HS20-44 (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4212.4	
Continuous	74.1	893.4	1248.2
	Tandem (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.3	1040.2

Moment Checks

Yielding

Fy	60	ksi
Zx	2161.63	in^3
Mn	10808.14	k-ft

Lateral-Torsional Buckling

I_y	9185.75	in^4	
r_y	8.45294056	in	
S_x	2015.77301	in^3	
J	131.630093	in^4	
h_0	35.65	in	
r_{ts}	9.01262551		
L_p	327.072369	in	
L_r	941.45602	in	
L_b	72	in	*if < Lp latera

l torsional buckling does not apply

Plate Girder Design 150' Interior span

Unfactored Moment (LL)	4212.4	k-ft	*obtained from Table 1
t(flange)	2.03	in	
b(flange)	34	in	
t(web)	0.45	in	
h(web)	37	in	
E(steel)	29000	ksi	
f'c	7	ksi	
Fy	60	ksi	
Kv	5		
Span Length	150	ft	
Effective Length	150	ft	
Tributary Width	8	ft	
Slab Thickness	8	in	
Wearing Surface Thickness	3	in	
Jersey Barrier Weight	0	k/ft	
E(concrete)	5072.24	ksi	
Area of section	154.69	in^2	
Area of Web	16.65	in^2	
Steel Weight	0.526	k/ft	
Slab Weight	0.800	k/ft	
Wearing Surface Weight	0.280	k/ft	
Wu (DL factored)	1.928	k/ft	
h/t(web)	82.2222222		must be < 260
Web Compact Check	82.66		must be > h/t(web)
Minimum t(web)	0.45		
Cv	0.53978		

M_u	12688.99	k-ft	choose from interior or exterior moments from chec
Af	68.30	in^2	
minimum b(flange)	33.65	in	
b(flange)/2t(flange)	8.29		
Flange Compact Check	8.35		must be > b(flange)/2t(flange)
I_x	1946.89	in^4	
Z(required)	2819.77	in^3	
Z(calculated)	2847.86	in^3	must be > Z(required)

Longitudinal Stiffness Parameter

n	5.717394367	
Moment of Inertia	1946.89	in^4
Area of Steel	154.69	in^2
e_g	24.53	in
K_g	543307.39	

LL Distribution Factors (Interior Girders)

	Moment Factor	Shear Factor	
Interior Factor	0.723268065	0.68	*1 design lane loaded
Interior Factor	1.078284871	0.814421769	*2 design lanes loaded

LL Distribution Factor (Exterior Girder)

Exterior Factor	0.485	*using lever rule
Multiple Prescence	1.2	*4 lanes

Interior Girder Checks

V_{LL}	84	k	*from RISA results
V_u	254.03	k	
ΦV_n	291.19	k	*must be > Vu

M_{DL}	5421.52	k-ft
M_u	12688.99	k-ft
ΦM_n	12815.38	k-ft

Exterior Girder Checks

V_{LL}		k	
V_u	144.57	k	*must be > Vu
ΦV_n	291.19	k	

M_{DL}	5421.52	k-ft	
M_u	9344.104849	k-ft	*must be > Mu

* Dark gray cells indicate inputs
* Light gray cells indicate results

Table 1

	HS20-44 (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	84	4212.4	
Continuous	74.1	893.4	1248.2
	Tandem (Service)		
	V_u (k)	M_u (k-ft)	$-M_u$ (k-ft)
Single Span	73	3625	
Continuous	71.6	599.3	1040.2

Moment Checks

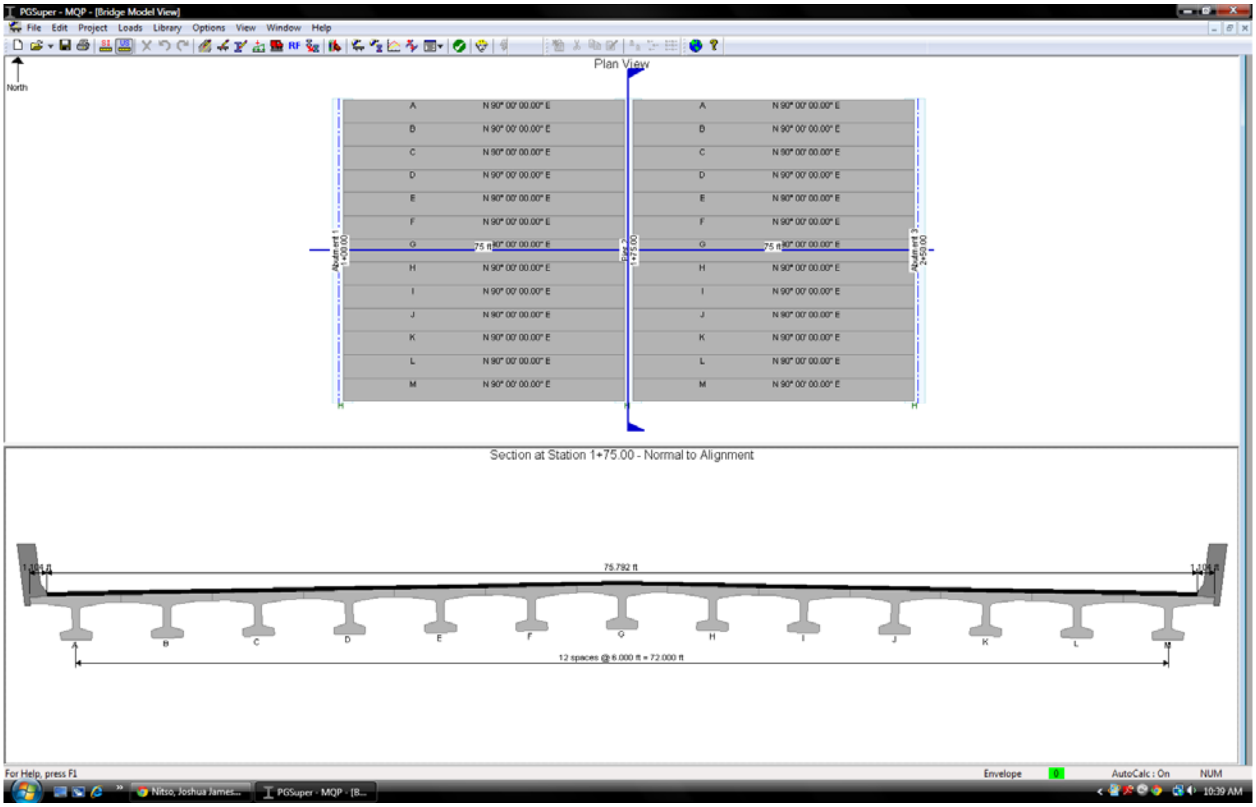
Yielding			Lateral-Torsional Buckling		
Fy	60	ksi	I_y	13298.13	in^4
Zx	2847.86	in^3	r_y	9.2718039	in
Mn	14239.32	k-ft	S_x	2655.49859	in^3
			J	190.74022	in^4
			h_0	39.03	in
			r_{ts}	9.88568112	
			L_p	358.756914	in
			L_r	1033.72051	in
			L_b	96	in

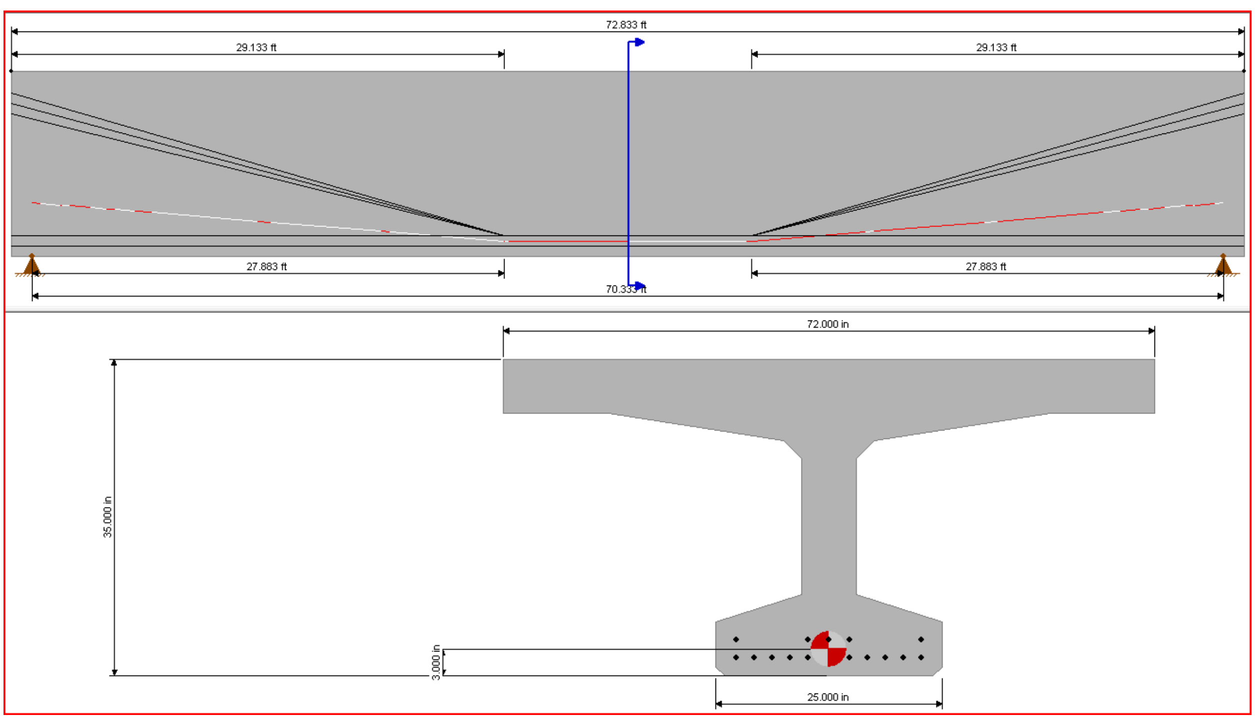
*if < Lp lateral torsional buckling does not apply

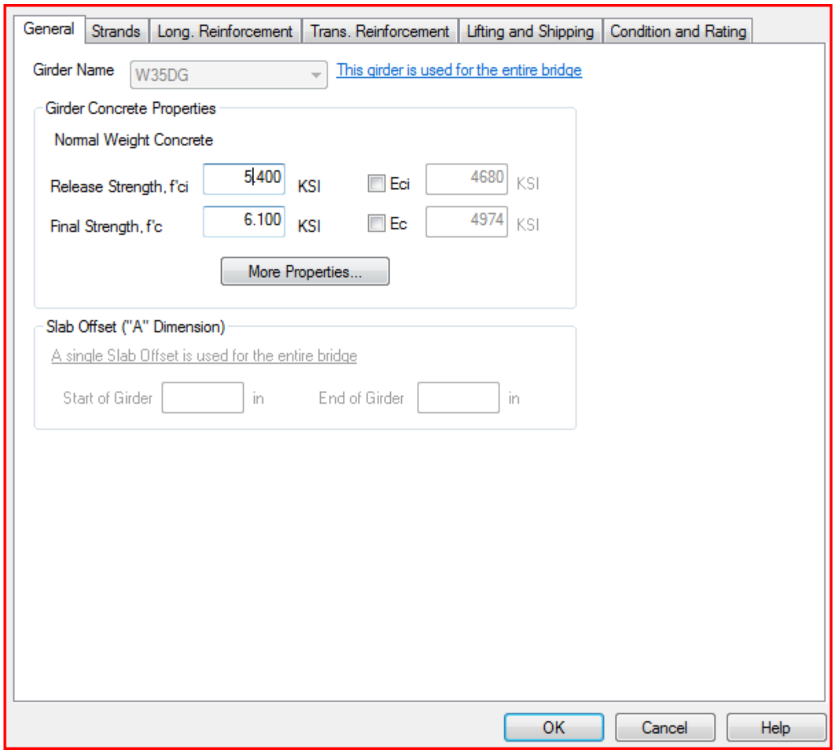
Appendix 2.3: Composite Design

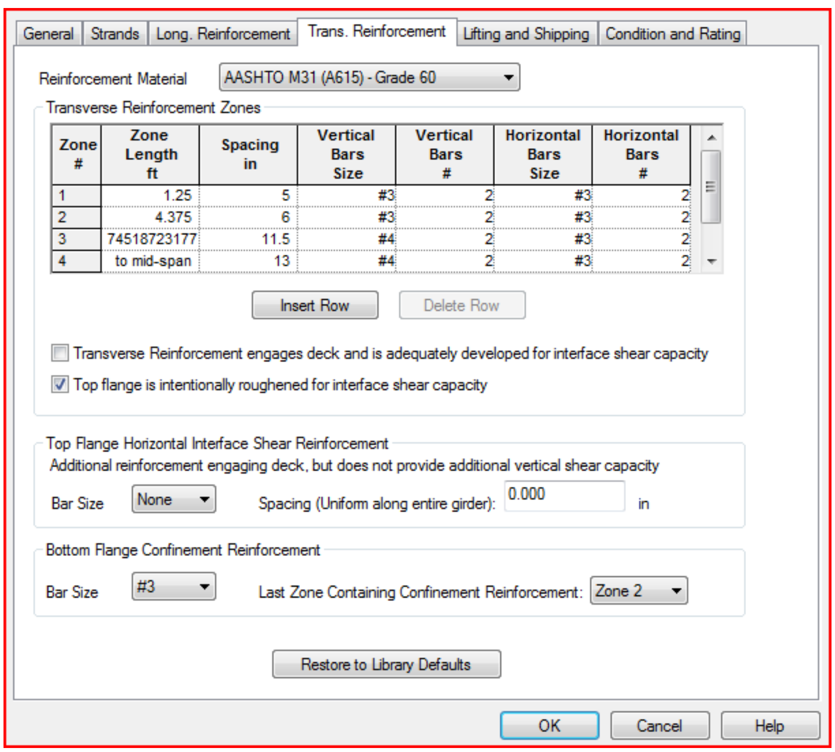
This appendix contains screen shots from the preliminary precast design. The preliminary precast design was produced in PG super. Design inputs and captured member cross sections are shown.

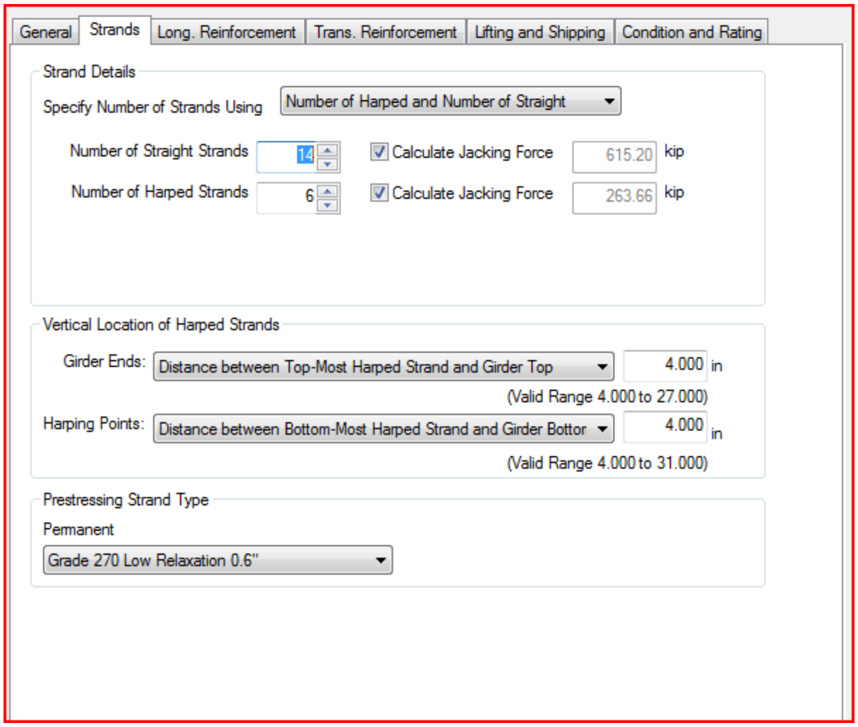
Preliminary Design PG-Super Results (Screen Shots)

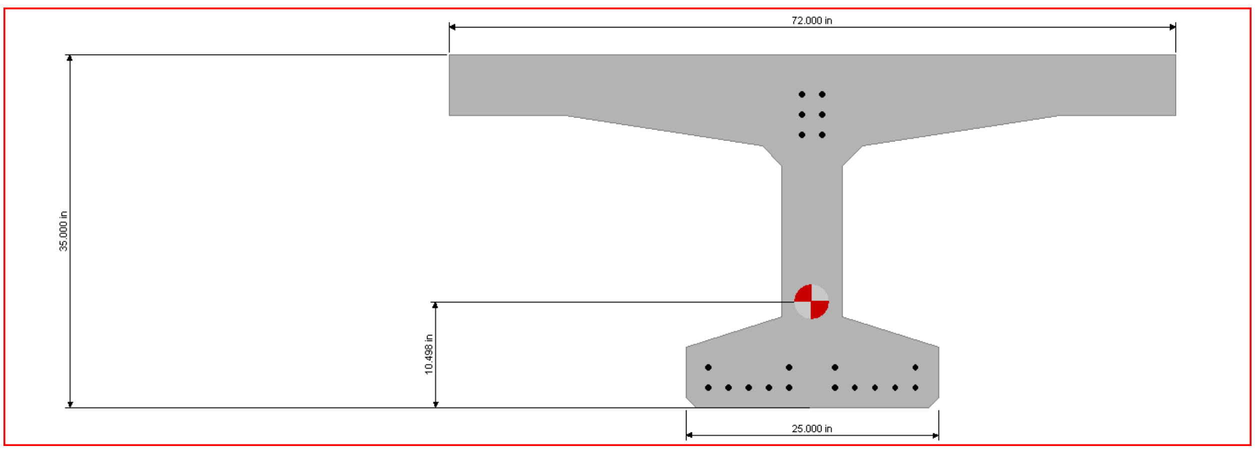


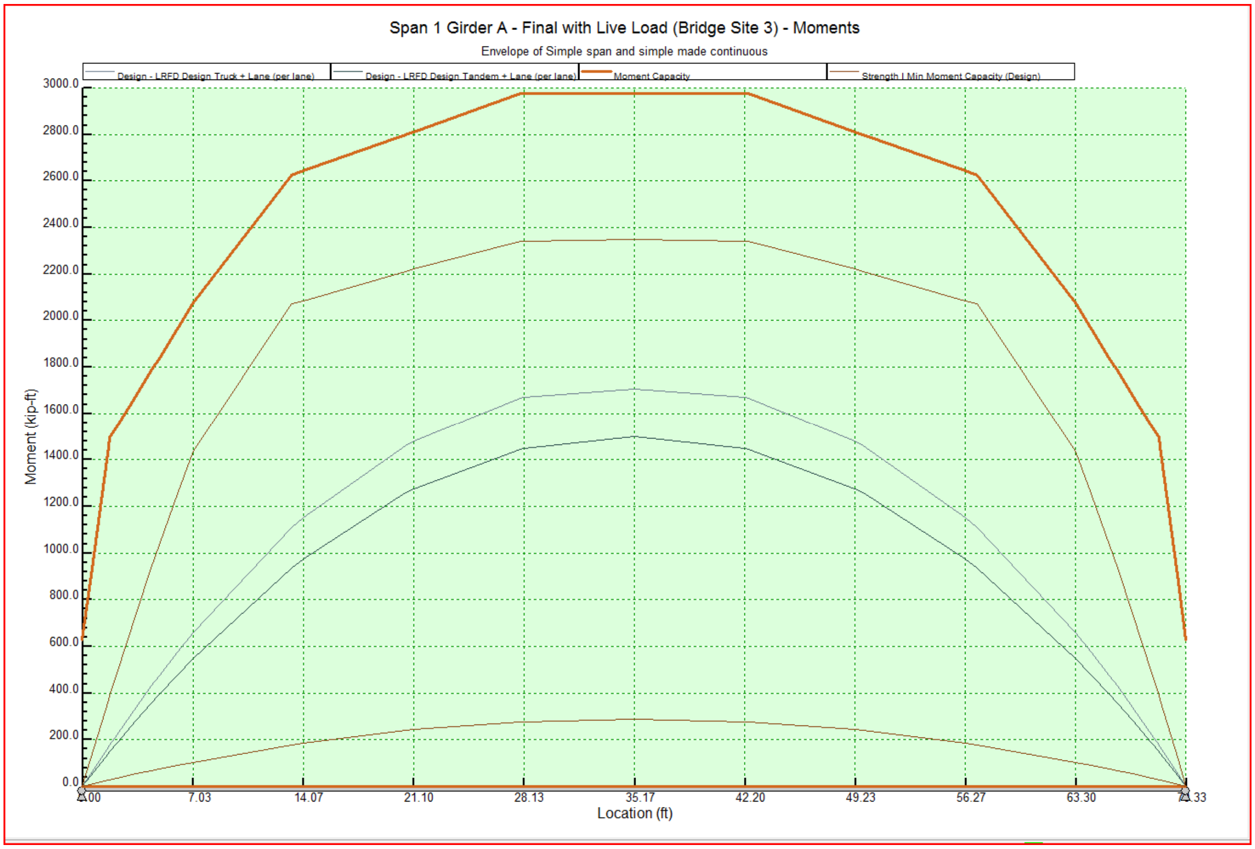



General
Strands
Long. Reinforcement
Trans. Reinforcement
Lifting and Shipping
Condition and Rating
Girder Name
W35DG
This girder is used for the entire bridge
Girder Concrete Properties
Normal Weight Concrete
Release Strength, f'ci
5.400
KSI
Eci
4680
KSI
Final Strength, f'c
6.100
KSI
Ec
4974
KSI
More Properties...
Slab Offset ("A" Dimension)
A single Slab Offset is used for the entire bridge
Start of Girder
in
End of Girder
in
OK
Cancel
Help


General
Strands
Long. Reinforcement
Trans. Reinforcement
Lifting and Shipping
Condition and Rating
Reinforcement Material
AASHTO M31 (A615) - Grade 60
Transverse Reinforcement Zones
Zone #
Zone Length ft
Spacing in
Vertical Bars Size
Vertical Bars #
Horizontal Bars Size
Horizontal Bars #
1 1.25 5 #3 2 #3 2
2 4.375 6 #3 2 #3 2
3 74518723177 11.5 #4 2 #3 2
4 to mid-span 13 #4 2 #3 2
Insert Row
Delete Row
Transverse Reinforcement engages deck and is adequately developed for interface shear capacity
Top flange is intentionally roughened for interface shear capacity
Top Flange Horizontal Interface Shear Reinforcement
Additional reinforcement engaging deck, but does not provide additional vertical shear capacity
Bar Size
None
Spacing (Uniform along entire girder):
0.000
in
Bottom Flange Confinement Reinforcement
Bar Size
#3
Last Zone Containing Confinement Reinforcement:
Zone 2
Restore to Library Defaults
OK
Cancel
Help


General
Strands
Long. Reinforcement
Trans. Reinforcement
Lifting and Shipping
Condition and Rating
Strand Details
Specify Number of Strands Using
Number of Harped and Number of Straight
Number of Straight Strands
14
Calculate Jacking Force
615.20
kip
Number of Harped Strands
6
Calculate Jacking Force
263.66
kip
Vertical Location of Harped Strands
Girder Ends:
Distance between Top-Most Harped Strand and Girder Top
4.000
in
(Valid Range 4.000 to 27.000)
Harping Points:
Distance between Bottom-Most Harped Strand and Girder Bottor
4.000
in
(Valid Range 4.000 to 31.000)
Prestressing Strand Type
Permanent
Grade 270 Low Relaxation 0.6"


72.000 in
35.000 in
10.498 in
25.000 in


Span 1 Girder A - Final with Live Load (Bridge Site 3) - Moments
Envelope of Simple span and simple made continuous
Design - LRFD Design Truck + Lane (per lane)
Design - LRFD Design Tandem + Lane (per lane)
Moment Capacity
Strength I Min Moment Capacity (Design)
Moment (kip-ft)
3000.0
2800.0
2600.0
2400.0
2200.0
2000.0
1800.0
1600.0
1400.0
1200.0
1000.0
800.0
600.0
400.0
200.0
0.0
7.03
14.07
21.10
28.13
35.17
42.20
49.23
56.27
63.30
Location (ft)

Appendix 2.5: Cost and Time Estimating

This appendix contains the calculations used to create the preliminary design cost estimates for both the steel and prestressed bridge options.

Construction Time Estimate

This sections shows calculations for girder erection times using design takeoffs and crew production out information from RS MEANS Heavy Construction.

NHDOT Weighted Average Prices, provided by NHDOT

This document is provided as a sample and was not created by the project, but contains the pricing information published by NHDOT about bid unit prices for construction tasks.

Precast Design Study Estimates

This document uses design quantities state pricing information to provide a detailed estimated cost fo the various precast shape options,.

Steel Pre-Design Study Estimates

This document contains the preliminary steel design cost estimates. This sheet uses design takeoffs from the eSpan results to produce the design cost estimates.

Preliminary Price Estimate Summary Table

This section contains a summary of the price estimate information for all the preliminary bridge designs.

Construction Time Estimate

Bridge	Beam Size (lbs/ft)	Number of splices or gaps	Governing Beam Length (ft)	Deck Weight Per Foot Interior Beam (lbs/ft)	Total Weight Per Foot (lbs/ft)	Total Weight of Heaviest member (tons)	Crew Suggested By R-S Means Heavy Construction 2009	Number of Individual Members before splicing
Simple Two Span Precast	886	1	75	Included in Beam	886	33.225	C-11+new crane	26
Continuous Two Span Steel End Beams	76	2	Does not govern					Included Below
Continuous Two Span Steel Center Beam	176	2	40	1092.85	1268.85	25.377067	E-5 +new crane	24
Single Span Steel	316.5	1	120	1115.63	1432.08	85.925	E-5 +new crane	16

* Information from RS-Means Heavy Construction 2009

Superstructure Construction Requirements According to RS-Means									
Bridge Design Type	Crane Required	Work Hours Required to Finish Bridge	Total Number of Workers Required	Steel Foreman	Steel Workers	Crane Operators	Welders	Misc. Workers
Simple Two Span Precast	Truck Mounted Hydralic Crane--40 Ton Capacity	14.9	9	1	6	1	0	1
Two Span Steel Beam	Truck Mounted Hydralic Crane--40 Ton Capacity	4.9	10	2	5	1	1	1
Single Span Steel	Truck Mounted Hydralic Crane--100 Ton Capacity	11.0	10	2	5	1	1	1

Crew	Total Workers	Total Large Equipment	Crane Specified	Welding Equipment	Steel Foreman
C-11	9	1	Truck Mounted Hydralic Crane--40 Ton Capacity	0	1
E-5 Two Span	10	2	Truck Mounted Hydralic Crane--40 Ton Capacity	1	2
E-5 Single Span	10	2	Truck Mounted Hydralic Crane--100 Ton	1	2

Steel Workers	Crane Operators	Welders	Misc. Workers	Daily Labor Cost +Equipment Cost	Daily Output
6	1	0	1	$ 3,823.90	14 beams
5	1	1	1	$ 4,331.60	1000 tons per day
5	1	1	1	$ 5,211.60	1000 tons per day

Daily Beam Output	Number of Days Required	Estimated Total Labor+ Equipment Cost
14 beams	2	$ 7,647.80
39 Beams	1	$ 4,331.60
11 Beams	2	#########

Daily Costs	
300amp Welder	$ 134.20
Steel Foreman	$ 373.50
Steel Workers	$ 357.60
Crane Operators	$ 340.40
Welders	$ 357.60
Misc. Workers	$ 294.40
Truck Mounted Hydralic Crane--40 Ton Capacity	$ 670.00
Truck Mounted Hydralic Crane--40 Ton Capacity	$ 670.00

C-11

E-5 1

E-5 2

Truck Mounted Hydralic Crane--100 Ton Capacity	$ 1,550.00

Group A: Rural Project Over $750,000
Group B: Rural Project Under $750,000
Group C: Bridge Project
Group D: Urban Poject
Group E: Special Projects
Group F: Signalization Project

Project Standard **Imperial**

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
201.1	CLEARING AND GRUBBING (F)	A	251.32	$4,721.57	A	18	$20,000.00	$2,900.00
			2	$11,000.00	B	2	$12,000.00	$10,000.00
			1.3	$15,300.00	E	2	$22,600.00	$8,000.00
201.21	REMOVING SMALL TREES	EA	295	$168.86	A	9	$300.00	$50.00
			38	$195.00	B	2	$240.00	$150.00
			2	$500.00	E	2	$800.00	$200.00
201.22	REMOVING LARGE TREES	EA	22	$402.27	A	5	$750.00	$270.00
			4	$900.00	E	2	$1,500.00	$300.00
201.4	REMOVING STUMPS	EA	26	$142.50	A	2	$150.00	$135.00
201.6	CLEARING FOR FENCE LINES (F)	A	3.96	$2,819.47	A	4	$4,225.00	$1,500.00
202.31	FILL ABANDONED PIPE	CY	3587	$156.84	A	20	$250.00	$115.96
			78	$132.50	B	2	$150.00	$115.00
202.41	REMOVAL OF EXISTING PIPE 0-24" DIAMETER	LF	55488	$7.89	A	18	$25.00	$5.00
			3980	$11.00	B	2	$12.00	$10.00
			20	$20.50	E	2	$30.00	$11.00
202.42	REMOVAL OF EXISTING PIPE OVER 24" DIAMETER	LF	2823	$11.02	A	6	$12.82	$7.00
202.43	REMOVAL OF EXIST. ASBEST. CEMENT PIPE, 0-24" DIAM.	LF	856	$29.28	A	2	$35.00	$23.55
202.5	REMOVAL OF CATCH BASINS, DROP INLETS, AND MANHOLES	EA	694	$227.15	A	15	$300.00	$175.00
			40	$175.00	B	2	$200.00	$150.00
			2	$420.00	E	2	$450.00	$390.00
202.6	CURB REMOVAL FOR STORAGE	LF	6025	$2.97	A	7	$4.20	$2.00
			6500	$2.25	E	2	$2.50	$2.00
202.7	REMOVAL OF GUARDRAIL	LF	266430	$1.46	A	31	$2.50	$0.75
			5208	$1.50	B	2	$2.00	$1.00
			1320	$1.05	C	2	$1.10	$1.00
			54560	$1.65	E	12	$5.75	$0.46
202.9	REMOVAL OF FUEL TANKS	U	4	$1,750.00	B	2	$3,000.00	$500.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
203.1	COMMON EXCAVATION	CY	2524150	$4.95	A	20	$13.00	$2.50
			45860	$7.33	B	8	$20.00	$6.50
			70	$21.45	C	2	$27.90	$15.00
			40095	$8.20	E	14	$20.00	$4.75
203.2	ROCK EXCAVATION	CY	1016640	$10.07	A	16	$50.00	$6.00
			66	$115.00	E	2	$165.00	$65.00
203.4	MUCK EXCAVATION	CY	69500	$5.95	A	5	$8.00	$4.20
203.49	WETLAND SOIL EXCAVATION	CY	5360	$6.86	A	3	$9.00	$5.00
203.52	IMPERVIOUS MATERIAL (F)	CY	38189	$11.15	A	6	$15.00	$9.00
203.6	EMBANKMENT-IN-PLACE (F)	CY	2953508	$4.42	A	22	$10.00	$2.32
			3140	$4.36	B	4	$14.00	$2.00
			5600	$10.00	E	2	$10.00	$10.00
203.7	REHANDLING SURCHARGE MATERIAL	CY	2320	$4.54	A	3	$4.80	$4.00
203.81	PRESPLITTING HOLES	LF	69000	$8.22	A	4	$9.36	$5.00
203.82	EXTRA DRILLED HOLES WITHOUT EXPLOSIVES	LF	12000	$2.72	A	2	$2.94	$2.50
206.1	COMMON STRUCTURE EXCAVATION	CY	49019	$15.22	A	25	$50.00	$7.15
			310	$15.26	B	3	$19.00	$15.00
			370	$29.74	E	4	$40.00	$18.00
206.19	COMMON STRUCTURE EXCAVATION EXPLORATORY	CY	3300	$37.75	A	23	$80.00	$20.00
			310	$28.42	B	5	$36.00	$1.00
			20	$30.00	C	2	$40.00	$20.00
			765	$25.36	E	8	$56.00	$5.00
206.2	ROCK STRUCTURE EXCAVATION	CY	53604	$40.28	A	24	$115.00	$12.25
			106	$40.00	B	2	$50.00	$30.00
			1150	$7.83	E	4	$45.00	$1.00
207.3	UNCLASSIFIED CHANNEL EXCAVATION	CY	50	$30.00	A	2	$40.00	$20.00
			720	$26.50	C	2	$43.00	$10.00
209.1	GRANULAR BACKFILL	CY	18654	$22.24	A	18	$50.00	$12.00
			60	$48.50	E	2	$67.00	$30.00
209.201	GRANULAR BACKFILL (BRIDGE) (F)	CY	9277	$33.83	A	9	$50.00	$23.50
209.4	GRANULAR BACKFILL (GRAV)	CY	1450	$25.87	A	6	$30.00	$22.00
			160	$28.00	B	2	$30.00	$26.00
			600	$32.50	E	2	$35.00	$30.00
210.1	SETTLEMENT PLATFORMS WITH CAP AND LOCK	EA	4	$1,272.50	A	2	$1,544.99	$1,000.00
214.3	FINE GRADING EARTH BERMS IN ROCKCUTS	LF	7800	$6.58	A	4	$9.00	$3.00
304.1	SAND (F)	CY	626154.2	$15.12	A	8	$20.00	$11.50
			300	$15.00	E	2	$15.00	$15.00
304.2	GRAVEL (F)	CY	7322	$21.00	A	2	$22.00	$20.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
304.2	GRAVEL (F)	CY	23500	$17.30	B	2	$24.60	$10.00
304.3	CRUSHED GRAVEL (F)	CY	9240	$22.37	A	3	$23.50	$22.00
			14	$57.00	C	2	$64.00	$50.00
304.32	CRUSHED GRAVEL FOR SHOULDER LEVELING	TON	34486	$15.69	A	25	$50.00	$7.00
			8	$48.50	C	2	$50.00	$47.00
			67652	$11.87	E	26	$50.00	$5.00
304.35	CRUSHED GRAVEL FOR DRIVES	CY	1695	$30.62	A	9	$41.00	$28.00
			24	$109.25	B	2	$180.00	$38.50
			331	$54.35	E	6	$95.00	$25.00
304.399	TEMPORARY CRUSHED GRAVEL	CY	100	$31.75	A	2	$35.00	$28.50
304.4	CRUSHED STONE (FINE GRADATION) (F)	CY	255379.1	$17.30	A	6	$20.00	$15.00
			13660	$25.00	B	2	$28.00	$22.00
			1100	$20.00	E	2	$20.00	$20.00
304.499	TEMPORARY CRUSHED STONE (FINE GRADATION)	CY	80	$42.50	A	2	$50.00	$35.00
			3250	$25.00	B	2	$28.00	$22.00
304.5	CRUSHED STONE (COARSE GRADATION) (F)	CY	206168.7	$17.27	A	6	$20.00	$15.00
			1600	$20.00	E	2	$20.00	$20.00
306.112	RECLAIMED STABILIZED BASE PROCESSED IN PLACE, 12 IN D	SY	68600	$1.85	A	2	$2.60	$1.10
			118600	$0.89	E	4	$1.00	$0.75
306.36	STONE FOR RECLAIMED STABILIZED BASE	TON	14400	$14.00	A	2	$18.00	$10.00
			6000	$12.63	E	2	$13.25	$12.00
403.11	HOT BITUMINOUS PAVEMENT, MACHINE METHOD	TON	32132	$70.93	A	14	$100.00	$56.00
			725	$93.97	B	3	$110.00	$81.50
			274	$157.50	C	2	$165.00	$150.00
			221054	$73.50	E	24	$112.00	$60.50
403.11002	HOT BITUMINOUS PAVEMENT, MACHINE METHOD (QC/QA TIER	TON	184900	$64.95	A	16	$75.00	$60.00
			6880	$71.00	B	2	$72.00	$70.00
			37925	$64.06	E	3	$65.25	$63.00
403.1109	HOT BITUMINOUS PAVEMENT (HIGH STRENGTH)	TON	23795	$77.02	A	12	$88.00	$70.00
	HOT BITUMINOUS PAVEMENT, MACHINE METHOD, HIGH STRE		2300	$81.00	B	2	$82.00	$80.00
			12000	$80.08	E	9	$84.50	$75.00
403.11091	HOT BITUMINOUS PAVEMENT MACHINE METHOD, NIGHT (QC/Q	TON	58750	$67.13	A	5	$72.00	$65.00
403.11092	HOT BITUMINOUS PAVEMENT MACHINE METHOD, NIGHT (QC/Q	TON	21930	$70.70	A	7	$75.00	$65.00
403.119	HOT BITUMINOUS PAVEMENT, MACHINE METHOD (NIGHT)	TON	11000	$76.50	A	2	$78.00	$75.00
			3900	$90.00	E	2	$100.00	$80.00
403.1199	HOT BITUMINOUS PAVEMENT, MACHINE METHOD, HIGH STRE	TON	5180	$80.48	A	4	$84.00	$77.50
			600	$112.50	E	2	$135.00	$90.00
403.12	HOT BITUMINOUS PAVEMENT, HAND METHOD	TON	20676	$105.81	A	31	$135.00	$100.00
			481	$113.58	B	7	$250.00	$70.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
403.12	HOT BITUMINOUS PAVEMENT, HAND METHOD	TON	44	$343.18	C	3	$375.00	$275.00
			9738	$115.29	E	34	$150.00	$90.00
403.129	HOT BITUMINOUS PAVEMENT, HAND METHOD (NIGHT)	TON	100	$145.00	A	2	$175.00	$115.00
403.4	MATERIAL TRANSFER VEHICLE (MTV)	TON	1067880	$1.18	A	28	$1.75	$1.00
			192550	$1.25	E	9	$2.10	$1.00
403.911	HOT BITUMINOUS BRIDGE PAVEMENT, 1" BASE COURSE (F)	TON	3207.6	$126.01	A	18	$175.00	$100.00
			74	$366.08	C	3	$385.00	$325.00
			1142	$175.00	E	2	$200.00	$150.00
403.98	HOT BITUMINOUS CONCRETE LEVELING, MACHINE METHOD	TON	5665	$74.34	A	7	$85.00	$70.00
			2000	$84.00	E	2	$88.00	$80.00
403.989	HOT BITUMINOUS CONCRETE LEVELING, MACHINE METHOD (N	TON	5800	$67.35	A	2	$72.70	$62.00
403.99	TEMPORARY BITUMINOUS PAVEMENT	TON	82980	$68.49	A	21	$105.00	$60.00
			1672	$76.00	B	2	$77.00	$75.00
			2	$395.00	C	2	$440.00	$350.00
411.1	HOT BITUMINOUS CONCRETE LEVELING COURSE	TON	7530	$75.53	A	7	$82.00	$74.75
			19700	$76.33	E	7	$82.00	$64.00
411.19	HOT BITUMINOUS CONCRETE LEVELING COURSE (NIGHT)	TON	5750	$79.23	A	4	$80.00	$75.00
411.3	PLANT MIX SURFACE TREAT- MENT (AC), PAVER SHIM	TON	159500	$69.22	E	13	$80.00	$62.50
411.46	PLANT MIX SURFACE TREAT- MENT (ASPHALT CEMENT), 3/4 IN	TON	39500	$67.45	E	2	$68.00	$66.90
413.1	HOT-POURED CRACK SEALANT	LB	5000	$2.75	E	1	$2.75	$2.75
417.	COLD PLANING BITUMINOUS SURFACES	SY	1434200	$1.49	A	30	$10.00	$0.65
			2720	$12.72	B	7	$100.00	$4.00
			3448	$6.75	C	2	$7.00	$6.50
			1854990	$1.50	E	36	$25.00	$1.00
417.19	COLD PLANING BITUMINOUS SURFACES (NIGHT)	SY	906500	$1.44	A	8	$1.80	$1.25
			46800	$1.75	E	2	$2.00	$1.50
500.021	PREPARATION FOR FATIGUE CRACK INSPECTION - COVER PL	U	76	$150.00	A	2	$200.00	$100.00
502.	REMOVAL OF EXISTING BRIDGE STRUCTURE	U	2	$2,900.00	A	2	$3,600.00	$2,200.00
			4	$254,125.00	C	4	$480,000.00	$24,000.00
503.101	WATER DIVERSION STRUCTURE	U	7	$1,804.90	A	7	$3,800.00	$500.00
			2	$9,400.00	B	2	$14,000.00	$4,800.00
			2	$17,500.00	C	2	$20,000.00	$15,000.00
			4	$12,900.25	E	4	$21,000.00	$7,101.00
503.201	COFFERDAMS	U	9	$109,047.75	A	9	$234,079.71	$28,050.00
			2	$4,960.00	B	2	$6,500.00	$3,420.00
			6	$40,593.33	E	6	$80,000.00	$500.00
503.301	COFFERDAMS WITH SHEETING LEFT-IN-PLACE	U	2	$25,425.00	A	2	$26,000.00	$24,850.00
504.1	COMMON BRIDGE EXCAVATION (F)	CY	16886.5	$10.68	A	9	$30.00	$9.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
504.1	COMMON BRIDGE EXCAVATION (F)	CY	28	$33.00	C	2	$46.00	$20.00
			114.8	$50.00	E	2	$50.00	$50.00
504.2	ROCK BRIDGE EXCAVATION	CY	1697	$99.65	A	5	$1,000.00	$30.00
508.	STRUCTURAL FILL	CY	6015	$31.20	A	11	$44.00	$18.00
			70	$49.00	E	2	$58.00	$40.00
510.1	PILE DRIVING EQUIPMENT	U	2	$75,000.00	A	2	$125,000.00	$25,000.00
510.9	PILE SPLICES	EA	47	$32.42	A	5	$100.00	$1.05
511.00	CONCRETE BRIDGE DECK PAVEMENT REMOVAL (F)	SY	2152	$6.50	A	2	$8.00	$5.00
			1174	$16.00	C	2	$17.00	$15.00
511.01	PREPARATION FOR CONCRETE BRIDGE DECK OVERLAY (F)	SY	1890	$10.50	A	2	$12.00	$9.00
511.02	PREPARATION FOR PARTIAL DEPTH CONCRETE BRIDGE DECK	SY	2603	$346.74	A	13	$500.00	$165.00
			154	$341.00	C	2	$382.00	$300.00
			598	$287.50	E	2	$400.00	$175.00
511.03	PREPARATION FOR FULL DEPTH CONCRETE BRIDGE DECK RE	SY	1057	$548.83	A	12	$820.00	$175.00
			44	$570.00	C	2	$690.00	$450.00
			204	$350.00	E	2	$400.00	$300.00
512.0101	PREPARATION FOR CONCRETE REPAIRS, CLASS I	SY	22	$566.55	A	3	$600.00	$500.00
			2	$617.50	C	2	$835.00	$400.00
512.0201	PREPARATION FOR CONCRETE REPAIRS, CLASS II	SY	270	$614.36	A	5	$765.00	$475.00
			2	$850.00	C	2	$1,100.00	$600.00
520.01	CONCRETE CLASS AA	CY	561.8	$792.70	A	16	$1,250.00	$280.00
			18	$495.00	C	2	$500.00	$490.00
			69.6	$638.76	E	3	$800.00	$475.00
520.1	CONCRETE CLASS A	CY	1173	$652.62	A	16	$1,090.00	$155.00
520.12	CONCRETE CLASS A, ABOVE FOOTINGS (F)	CY	1120	$630.26	A	8	$745.00	$475.00
			60	$600.00	B	2	$700.00	$500.00
520.2	CONCRETE CLASS B	CY	1892	$273.87	A	7	$300.00	$253.00
520.21	CONCRETE CLASS B, FOOTINGS (F)	CY	990	$282.50	A	2	$290.00	$275.00
520.213	CONCRETE CLASS B, FOOTINGS (ON SOIL) (F)	CY	1858	$275.78	A	7	$400.00	$225.00
520.421	CONCRETE CLASS F, FLOWABLE FILL, EXCAVATABLE	CY	3920	$93.41	A	6	$200.00	$75.00
520.7002	CONCRETE BRIDGE DECK (QC/QA) (F)	CY	1844	$640.00	A	2	$730.00	$550.00
			1066	$887.50	C	2	$950.00	$825.00
520.7102	CONCRETE BRIDGE DECK (QC/QA) (F)	CY	926.4	$921.86	A	5	$1,300.00	$850.00
520.7202	CONCRETE BRIDGE DECK (QC/QA) (F)	CY	629	$952.35	A	3	$1,100.00	$811.00
520.7302	CONCRETE BRIDGE DECK (QC/QA) (F)	CY	44	$1,445.00	A	2	$1,600.00	$1,290.00
520.82	CONCRETE FOR BRIDGE DECK OVERLAY (F)	CY	158	$625.00	A	2	$750.00	$500.00
530.3	WATERPROOFING CONCRETE SURFACES (F)	SY	1900	$27.50	A	2	$30.00	$25.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
534.3	WATER REPELLENT (SILANE/ SILOXANE)	GAL	2519.4	$76.56	A	17	$100.00	$55.00
			252	$82.30	C	3	$85.00	$80.00
			936	$85.00	E	2	$105.00	$65.00
538.2	BARRIER MEMBRANE, PEEL AN STICK - VERTICAL SURFACES (	SY	1252	$52.43	A	11	$63.00	$40.80
			208	$42.50	E	2	$60.00	$25.00
538.5	BARRIER MEMBRANE, HEAT WELDED (F)	SY	9238	$19.04	A	8	$25.00	$17.00
			470	$52.50	C	2	$55.00	$50.00
			144	$45.80	E	2	$46.60	$45.00
541.1	PVC WATERSTOPS, NH TYPE 1 (F)	LF	821	$5.03	A	3	$5.15	$5.00
541.2	PVC WATERSTOPS, NH TYPE 2 (F)	LF	608	$7.25	A	9	$15.00	$5.00
541.3	PVC WATERSTOPS, NH TYPE 3 (F)	LF	288	$7.24	A	5	$10.00	$5.00
541.4	PVC WATERSTOPS, NH TYPE 4 (F)	LF	290	$10.14	A	3	$10.30	$10.00
541.5	PVC WATERSTOPS, NH TYPE 5 (F)	LF	2060	$10.32	A	13	$15.00	$8.00
			88	$10.50	C	2	$11.00	$10.00
			624	$9.00	E	2	$10.00	$8.00
544.	REINFORCING STEEL (F)	LB	82443	$1.43	A	8	$2.50	$1.00
			876	$4.05	E	2	$4.10	$4.00
544.1	REINFORCING STEEL (ROADWAY)	LB	388650	$1.01	A	12	$2.00	$0.85
			1120	$2.73	B	2	$4.20	$1.25
544.2	REINFORCING STEEL, EPOXY COATED (F)	LB	524416	$1.48	A	7	$2.25	$1.20
			158285	$1.23	C	3	$1.35	$1.20
544.21	REINFORCING STEEL, EPOXY COATED,MECHANICAL CONNEC	LB	17232	$3.01	A	4	$8.00	$2.45
			526	$6.25	C	2	$8.00	$4.50
547.	SHEAR CONNECTORS (F)	EA	46880	$5.13	A	9	$6.00	$4.00
548.21	ELASTOMERIC BEARING ASSEMBLIES (F)	EA	84	$1,873.57	A	6	$2,750.00	$930.00
550.1	STRUCTURAL STEEL (F)	LB	1988414	$1.71	A	2	$1.75	$1.66
			1700	$9.65	C	2	$10.00	$9.30
550.191	TEMPORARY GIRDER SUPPORT SYSTEM	U	2	$24,500.00	C	2	$25,000.00	$24,000.00
550.2	BRIDGE SHOES (F)	EA	6	$1,737.50	A	2	$2,500.00	$975.00
			12	$3,500.00	C	2	$4,000.00	$3,000.00
556.	PAINTING EXISTING STRUCTURAL STEEL	U	2	$308,000.00	A	2	$352,000.00	$264,000.00
			2	$64,500.00	E	2	$85,000.00	$44,000.00
559.4	ELASTOMERIC PLUG TYPE EXPANSION JOINT (F)	LF	756	$119.35	A	3	$120.00	$119.00
			302	$126.94	E	4	$130.00	$125.00
559.41	MODIFIED ELASTOMERIC PLUG TYPE FLEXIBLE JOINT (6" WIDE	LF	2696	$92.12	A	21	$115.00	$80.00
			80	$97.50	C	2	$100.00	$95.00
			2012	$79.00	E	2	$80.00	$78.00
559.412	MODIFIED ELASTOMERIC PLUG TYPE EXPANSION JOINT, 20" W	LF	1056	$100.42	A	4	$118.00	$98.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
559.412	MODIFIED ELASTOMERIC PLUG TYPE EXPANSION JOINT (20"	LF	2658	$104.50	E	2	$105.00	$104.00
560.101	PREFABRICATED COMPRESSION SEAL EXPANSION JOINT (F)	LF	448	$279.44	A	6	$850.00	$30.00
			88	$600.00	C	2	$600.00	$600.00
561.301	PREFABRICATED EXPANSION JOINT, FINGER JOINT (F)	LF	472	$1,361.77	A	4	$1,500.00	$1,200.00
563.22	BRIDGE RAIL T2 (F)	LF	2880	$101.51	A	11	$107.00	$95.00
563.221	BRIDGE RAIL T2 WITH PROTECTIVE SCREENING (F)	LF	978	$154.61	A	4	$160.00	$148.50
563.24	BRIDGE RAIL T4 (F)	LF	600	$170.94	A	4	$182.00	$140.00
563.241	BRIDGE RAIL T4 WITH PROTECTIVE SCREENING (F)	LF	962	$214.50	A	4	$224.00	$205.00
563.72	BRIDGE RAIL F (2-BAR) (F)	LF	4	$1,225.00	C	2	$1,250.00	$1,200.00
563.81	REHABILITATION OF BRIDGE RAIL (F)	LF	20	$750.00	C	2	$1,300.00	$200.00
565.222	BRIDGE APPROACH RAIL, T2 (STEEL POSTS) (F)	U	40	$4,327.90	A	10	$4,450.00	$4,100.00
565.232	BRIDGE APPROACH RAIL, T3 (STEEL POSTS)	U	12	$4,450.00	A	2	$4,450.00	$4,450.00
565.242	BRIDGE APPROACH RAIL T4 (STEEL POSTS)	U	10	$5,663.80	A	5	$5,800.00	$5,599.00
565.72	BRIDGE APPROACH RAIL F (2-BAR)	U	4	$5,685.00	A	2	$5,685.00	$5,685.00
566.1	ELASTOMERIC JOINT SEAL (F)	LF	2400	$8.50	A	2	$14.00	$3.00
570.4	MORTAR RUBBLE MASONRY (F)	CY	12	$475.00	A	2	$550.00	$400.00
572.1	RECONSTRUCTING STONE WALL ONE STONE WIDE	LF	260	$20.00	B	2	$25.00	$15.00
572.2	RECONSTRUCTING STONE WALL MULTIPLE STONES WIDE	LF	90	$450.00	A	2	$450.00	$450.00
582.1	SLOPE PAVING WITH CONCRETE (F)	SY	3466	$44.66	A	8	$50.00	$40.00
585.11	STONE FILL, CLASS A (BRIDGE)	CY	900	$48.50	C	2	$57.00	$40.00
585.2	STONE FILL, CLASS B	CY	31499	$27.80	A	18	$48.00	$16.50
			80	$34.00	B	2	$48.00	$20.00
			600	$31.00	E	2	$32.00	$30.00
585.21	STONE FILL, CLASS B (BRIDGE)	CY	4518	$27.07	A	3	$32.00	$22.00
585.3	STONE FILL, CLASS C	CY	41280	$26.87	A	22	$46.00	$18.61
			50	$42.50	B	2	$50.00	$35.00
			30	$50.00	E	2	$55.00	$45.00
585.4	STONE FILL, CLASS D	CY	30	$32.27	A	3	$35.00	$28.80
585.5	STONE FILL, CLASS E	CY	2510	$30.50	A	3	$33.00	$29.00
585.7	STONE FILL, CLASS G	CY	700	$31.78	A	3	$33.00	$30.00
587.1	KEYED STONE FILL	CY	1200	$32.92	A	3	$35.00	$30.00
592.1	MECHANICALLY STABILIZED EARTH RETAINING WALL	SF	19264	$37.00	A	2	$40.00	$34.00
603.00212	12" R.C. PIPE, 2000D	LF	194	$37.30	A	3	$42.00	$34.91
			429	$50.43	E	5	$80.00	$42.00
603.00215	15" R.C. PIPE, 2000D	LF	37605	$38.83	A	14	$55.00	$32.00
			80	$46.25	E	2	$50.00	$42.50

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603.00218	18" R.C. PIPE, 2000D	LF	11780	$42.38	A	7	$50.00	$36.00
603.00224	24" R.C. PIPE, 2000D	LF	12098	$48.91	A	13	$100.00	$42.00
603.00230	30" R.C. PIPE, 2000D	LF	3575	$61.96	A	11	$80.00	$55.00
603.00236	36" R.C. PIPE, 2000D	LF	3880	$75.13	A	7	$90.00	$73.41
603.00242	42" R.C. PIPE, 2000D	LF	994	$89.28	A	5	$100.00	$80.00
603.00248	48" R.C. PIPE, 2000D	LF	1430	$118.63	A	5	$125.00	$114.00
			216	$200.00	B	2	$210.00	$190.00
603.00254	54" R.C. PIPE, 2000D	LF	144	$156.50	A	2	$170.00	$143.00
603.00260	60" R.C. PIPE, 2000D	LF	720	$207.95	A	4	$260.00	$160.00
603.00272	72" R.C. PIPE, 2000D	LF	192	$320.00	E	2	$340.00	$300.00
603.00315	15" R.C. PIPE, 3000D	LF	4166	$38.86	A	8	$46.00	$35.00
603.00318	18" R.C. PIPE, 3000D	LF	3000	$41.06	A	3	$44.15	$38.00
603.00324	24" R.C. PIPE, 3000D	LF	5665	$51.80	A	5	$59.00	$45.00
603.00330	30" R.C. PIPE, 3000D	LF	1612	$62.23	A	5	$73.00	$58.00
603.00336	36" R.C. PIPE, 3000D	LF	740	$77.79	A	4	$87.00	$69.00
603.00342	42" R.C. PIPE, 3000D	LF	932	$99.71	A	4	$105.00	$94.00
603.00348	48" R.C. PIPE, 3000D	LF	340	$125.72	A	2	$126.43	$125.00
603.11012	12" CORR. STEEL PIPE, .064"	LF	38	$43.50	A	2	$50.00	$37.00
603.30112	12" R.C. END SECTIONS	EA	4	$577.06	A	2	$669.11	$485.00
603.30115	15" R.C. END SECTIONS	EA	18	$579.44	A	6	$755.00	$400.00
			4	$675.00	E	2	$1,000.00	$350.00
603.30118	18" R.C. END SECTIONS	EA	2	$605.00	A	2	$785.00	$425.00
603.30124	24" R.C. END SECTIONS	EA	20	$635.50	A	6	$855.00	$500.00
603.30130	30" R.C. END SECTIONS	EA	4	$893.33	A	2	$1,036.66	$750.00
603.30136	36" R.C. END SECTIONS	EA	8	$1,042.50	A	2	$1,200.00	$885.00
603.30148	48" R.C. END SECTIONS	EA	2	$1,640.00	A	2	$1,880.00	$1,400.00
603.34112	12" STEEL END SECTIONS	EA	110	$113.13	A	3	$125.00	$101.00
603.34115	15" STEEL END SECTIONS	EA	47	$144.98	A	5	$185.00	$121.00
603.34118	18" STEEL END SECTIONS	EA	44	$174.00	A	5	$250.00	$150.00
			4	$300.00	B	2	$400.00	$200.00
603.34124	24" STEEL END SECTIONS	EA	10	$262.50	A	2	$300.00	$225.00
			8	$372.50	B	2	$445.00	$300.00
603.391	RESETTING CONCRETE END SECTIONS	EA	4	$187.50	A	2	$250.00	$125.00
603.40012	12" PIPE FOR SLOPE DRAIN. DRAINAGE	LF	20	$33.49	A	2	$36.97	$30.00
603.44015	15" CORR. POLYETHYLENE PIPE FOR SLOPE DRAINAGE	LF	114	$32.50	A	2	$40.00	$25.00

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603.50012	12" PIPE FOR DRIVES & MINOR APPROACHES	LF	260	$27.50	A	2	$30.00	$25.00
603.6	RELAYING 0-24" DRAINAGE PIPE	LF	356	$60.11	A	4	$80.00	$40.00
603.60030	LAYING/RELAYING 30" DRAINAGE PIPE	LF	16	$80.00	A	2	$80.00	$80.00
603.60036	LAYING/RELAYING 36" DRAINAGE PIPE	LF	200	$60.00	A	2	$80.00	$40.00
603.82224	24" PE PIPE (TYPE S)	LF	19600	$41.08	A	6	$45.00	$39.00
			2600	$49.50	B	2	$53.00	$46.00
603.82230	30" PE PIPE (TYPE S)	LF	4050	$55.04	A	6	$60.00	$52.80
603.99012	12" TEMPORARY DRAIN. PIPE	LF	670	$42.35	A	3	$43.00	$27.00
603.99015	15" TEMPORARY DRAIN. PIPE	LF	196	$30.00	A	3	$30.00	$30.00
604.0007	POLYETHYLENE LINER	EA	2266	$187.91	A	26	$230.00	$120.00
			136	$122.50	B	2	$135.00	$110.00
			130	$124.42	E	7	$170.00	$110.00
604.11	CATCH BASINS TYPE A	U	368	$1,957.50	A	2	$2,140.00	$1,775.00
604.115	CATCH BASINS TYPE A, 5-FOOT DIAMETER	U	87	$2,553.10	A	3	$2,600.00	$2,500.00
604.116	CATCH BASINS TYPE A, 6-FOOT DIAMETER	U	18	$3,160.00	A	3	$3,230.00	$3,100.00
604.12	CATCH BASINS TYPE B	U	624.2	$1,953.56	A	18	$2,375.00	$1,700.00
			114	$2,250.00	B	2	$2,500.00	$2,000.00
			14	$2,200.00	E	3	$2,200.00	$2,200.00
604.125	CATCH BASINS TYPE B, 5-FOOT DIAMETER	U	40	$2,635.00	A	9	$3,000.00	$2,200.00
			26	$2,725.00	B	2	$2,750.00	$2,700.00
604.126	CATCH BASINS TYPE B, 6-FOOT DIAMETER	U	18	$2,998.78	A	6	$3,250.00	$2,500.00
			2	$3,500.00	B	2	$3,500.00	$3,500.00
604.128	CATCH BASINS TYPE B, 8-FOOT DIAMETER	U	4	$7,795.00	A	2	$8,000.00	$7,590.00
604.13	CATCH BASINS TYPE C	U	4	$1,813.36	A	2	$2,200.00	$1,426.72
604.15	CATCH BASINS TYPE E	U	326.6	$2,129.55	A	10	$2,600.00	$1,900.00
604.155	CATCH BASINS TYPE E, 5-FOOT DIAMETER	U	69	$2,829.13	A	5	$3,250.00	$2,480.00
604.156	CATCH BASINS TYPE E, 6-FOOT DIAMETER	U	14	$3,421.43	A	4	$3,800.00	$3,000.00
604.158	CATCH BASINS TYPE E, 8-FOOT DIAMETER	U	8	$7,003.75	A	4	$8,500.00	$5,200.00
604.16	CATCH BASINS TYPE F	U	193.6	$2,035.59	A	9	$2,250.00	$1,875.00
604.165	CATCH BASINS TYPE F, 5-FOOT DIAMETER	U	20	$2,897.50	A	5	$3,150.00	$2,375.00
604.191	SPECIAL CATCH BASINS	U	30	$2,800.00	A	2	$3,100.00	$2,500.00
604.22	DROP INLETS TYPE B	U	38.2	$1,438.79	A	7	$2,000.00	$1,212.72
			2	$1,900.00	B	2	$2,000.00	$1,800.00
604.241	DROP INLETS TYPE D-A	U	10	$1,450.00	A	2	$1,800.00	$1,100.00
604.242	DROP INLETS TYPE D-B	U	23	$1,177.17	A	3	$1,275.00	$1,100.00
604.245	DROP INLETS TYPE D-E	U	18	$1,550.00	A	2	$1,800.00	$1,300.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
604.25	DROP INLETS TYPE E	U	24	$1,601.25	A	4	$1,700.00	$1,500.00
604.258	DROP INLETS TYPE E, 8-FOOT DIAMETER	U	4	$4,000.00	A	2	$5,000.00	$3,000.00
604.26	DROP INLETS TYPE F	U	10	$1,730.00	A	3	$1,900.00	$1,650.00
604.32	DRAINAGE MANHOLES	U	107	$2,046.02	A	7	$2,300.00	$1,826.00
604.325	DRAINAGE MANHOLES, 5-FOOT DIAMETER	U	70.2	$2,574.93	A	8	$2,800.00	$2,100.00
			2	$2,725.00	B	2	$2,750.00	$2,700.00
604.326	DRAINAGE MANHOLES, 6-FOOT DIAMETER	U	28	$3,036.15	A	5	$3,300.00	$2,832.93
604.328	DRAINAGE MANHOLES, 8-FOOT DIAMETER	U	10	$6,805.00	A	2	$7,350.00	$6,260.00
604.39	SPECIAL MANHOLES	U	6	$3,385.00	A	2	$3,770.00	$3,000.00
604.4	RECONSTRUCTING/ADJUSTING CATCH BASIN & DROP INLET	LF	1640	$311.53	A	25	$395.00	$210.00
			14	$307.14	B	3	$350.00	$200.00
			8	$250.00	C	2	$400.00	$100.00
			681	$272.50	E	10	$400.00	$185.00
604.5	RECONSTRUCTING/ADJUSTING MANHOLES	LF	6	$325.00	B	2	$350.00	$300.00
			10	$337.00	E	5	$650.00	$185.00
604.51	RECONSTRUCTING/ADJUSTING SEWER MANHOLES	LF	56	$398.48	A	7	$450.00	$300.00
604.6	MANHOLE COVERS & FRAMES	EA	2	$475.00	A	2	$600.00	$350.00
604.71	GRATES & FRAMES, TYPE A	EA	88	$570.00	E	2	$600.00	$540.00
604.72	GRATES & FRAMES, TYPE B	EA	343	$369.38	A	7	$575.00	$230.00
			4	$275.00	B	2	$300.00	$250.00
			60	$383.33	E	4	$500.00	$300.00
604.75	GRATES & FRAMES, TYPE E	EA	4	$700.00	A	2	$790.00	$610.00
604.9101	OUTLET CONTROL STRUCTURE	U	6	$3,143.33	A	6	$4,200.00	$2,400.00
605.506	6" PERF. CORR. POLYETHYL PIPE UND.	LF	202065	$17.44	A	19	$25.00	$13.00
			400	$35.00	B	2	$40.00	$30.00
605.508	8" PERFORATED CORRUGATED POLYETHYLENE PIPE UND.	LF	18200	$16.92	A	4	$22.50	$14.00
605.512	12" PERF. CORR. POLYETHLY PIPE UND.	LF	6980	$25.65	A	6	$35.00	$22.00
605.515	15" PERF. CORR. POLYETHLY PIPE UND.	LF	3360	$32.66	A	9	$50.00	$25.00
			800	$45.00	B	2	$50.00	$40.00
605.518	18" PERF. CORR. POLYETHYL PIPE UND.	LF	1730	$33.95	A	4	$54.00	$28.00
605.79	UNDERDRAIN FLUSHING BASINS	EA	388	$407.72	A	18	$510.00	$300.00
605.812	24" AGGREGATE UNDERDRAIN, TYPE 1	LF	7400	$9.00	A	2	$10.00	$8.00
605.82251	24"AGGREGATE UNDERDRAIN TYPE 2 WITH 6" PERF. CORR. P	LF	30700	$22.72	A	5	$25.00	$20.06
605.82258	24" AGGREGATE UNDERDRAIN TYPE 2, WITH 8" PERF. CORR. P	LF	14500	$27.06	A	4	$31.00	$23.00
606.000	STEEL BEAM FOR BEAM GUARDRAIL	LF	14940	$8.02	A	11	$9.00	$7.00
			875	$7.40	E	3	$9.00	$7.00
606.014	6"X8" WOOD POST REPLACE- MENTS FOR BEAM GUARDRAIL P	EA	1433	$50.86	A	9	$60.00	$45.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
606.014	6"X8" WOOD POST REPLACE- MENTS FOR BEAM GUARDRAIL P	EA	45	$58.89	E	3	$70.00	$45.00
606.0142	6"X8" WOOD POST ASSEMBLIES FOR BEAM GUARDRAIL POST	EA	2253	$9.50	A	10	$15.00	$8.00
			175	$10.71	E	3	$15.00	$10.00
606.120	BEAM GUARDRAIL (STANDARD SECTION) (STEEL POST)	LF	222362.5	$16.34	A	26	$19.00	$14.50
			400	$20.50	B	2	$22.00	$19.00
			1524	$18.50	C	2	$19.00	$18.00
			13600	$17.53	E	11	$25.20	$16.00
606.140	BEAM GUARDRAIL (STANDARD SECTION- WOOD POSTS)	LF	16100	$17.21	A	11	$20.00	$16.14
			1150	$14.26	E	3	$17.00	$13.50
606.141	BEAM GUARDRAIL (CURVED W/CRT POSTS)	LF	941	$20.25	A	14	$23.90	$18.50
			150	$20.00	E	3	$20.00	$20.00
606.147	BEAM GUARDRAIL (TERMINAL UNIT TYPE G-2)	U	86	$483.85	A	11	$517.88	$450.00
			54	$480.58	E	5	$500.00	$475.00
606.212	DOUBLE-FACED BEAM GUARDRAIL (STANDARD SECTION STEE	LF	100	$24.00	A	2	$24.00	$24.00
606.312	SINGLE FACED TRANSITION RAIL, STEEL POSTS	U	14	$2,808.93	A	4	$2,945.00	$2,500.00
			16	$2,475.00	E	2	$2,500.00	$2,450.00
606.322	DOUBLE FACED TRANSITION RAIL, STEEL POST	U	4	$3,835.00	A	4	$4,200.00	$3,500.00
			4	$1,175.00	B	2	$1,250.00	$1,100.00
			24	$4,132.50	E	2	$4,175.00	$4,090.00
606.411	CONCRETE BARRIER, SINGLE- FACED, PRECAST	LF	80	$121.00	A	2	$210.00	$32.00
606.41211	TRANSITION MEDIAN CONCRET BARRIER, PRECAST	U	2	$2,050.00	A	2	$2,300.00	$1,800.00
606.41231	TRANSITION SINGLE SLOPE CONCRETE BARRIER, PRECAST	U	10	$5,750.00	A	2	$6,500.00	$5,000.00
			4	$2,250.00	B	2	$2,500.00	$2,000.00
			32	$1,675.00	E	2	$1,750.00	$1,600.00
606.417	PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL	LF	170070	$21.72	A	23	$41.66	$12.00
			370	$21.24	B	3	$28.00	$18.00
			5680	$34.99	C	4	$50.00	$15.00
			21900	$22.10	E	7	$33.00	$15.00
606.4171	PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL - LEF	LF	280	$38.50	A	2	$40.00	$37.00
606.4229	MODIFIED CONCRETE MEDIAN BARRIER, CAST-IN-PLACE	LF	40	$295.00	E	2	$400.00	$190.00
606.84	ANCHOR FOR CURVED GUARD- RAIL W/CRT POSTS	U	25	$1,236.68	A	13	$1,370.00	$1,150.00
			3	$1,200.00	E	3	$1,200.00	$1,200.00
606.91	RESETTING OR SETTING GUARDRAIL	LF	3900	$10.91	A	2	$11.50	$10.31
			60	$21.00	E	2	$21.00	$21.00
606.9147	RESETTING TERMINAL UNIT TYPE G-2	U	60	$102.73	A	6	$190.00	$60.00
606.93	TEMPORARY BEAM GUARDRAIL	LF	4850	$11.90	A	3	$12.35	$11.50
606.95	TEMPORARY TRAFFIC CONTROL BARRIER	LF	34500	$29.69	A	4	$37.00	$12.50
			3800	$16.00	B	2	$20.00	$12.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
607.1	WOVEN WIRE FENCE	LF	7300	$7.45	A	2	$8.00	$6.90
607.41	POST ASSEMBLIES FOR WOVEN WIRE FENCE	EA	40	$160.00	A	2	$160.00	$160.00
607.9	RESETTING RAILING & FENCING	LF	2800	$17.18	A	2	$17.35	$17.00
608.12	2" BITUMINOUS SIDEWALK (F)	SY	17971	$15.99	A	11	$22.00	$12.00
			120	$13.50	B	2	$14.00	$13.00
608.24	4" CONCRETE SIDEWALK (F)	SY	1219	$38.22	A	11	$55.00	$34.00
608.26	6" CONCRETE SIDEWALK (F)	SY	25304	$39.05	A	9	$45.00	$37.00
			550	$36.82	E	3	$40.00	$35.00
608.28	8" CONCRETE SIDEWALK (F)	SY	5105.14	$42.83	A	13	$60.00	$40.00
			550	$56.00	B	2	$58.00	$54.00
609.01	STRAIGHT GRANITE CURB	LF	37320	$17.22	A	10	$20.00	$15.75
			8560	$19.50	B	2	$20.00	$19.00
609.02	CURVED GRANITE CURB	LF	1007	$27.09	A	9	$32.00	$24.00
			900	$32.00	B	2	$33.00	$31.00
609.21	STRAIGHT GRANITE SLOPE CURB	LF	33060	$12.09	A	5	$13.90	$11.50
			100	$21.25	E	1	$21.25	$21.25
609.214	STRAIGHT GRANITE SLOPE CURB, 4" HIGH	LF	4800	$14.36	A	2	$14.71	$14.00
609.216	STRAIGHT GRANITE SLOPE CURB 6" HIGH	LF	46180	$13.77	A	8	$14.20	$13.00
609.22	STRAIGHT GRANITE SLOPE CURB WITH RADIAL JOINTS	LF	414	$12.33	A	7	$13.90	$11.20
609.226	STRAIGHT GRANITE SLOPE CURB WITH RADIAL JOINTS, 6" HIG	LF	442	$13.79	A	7	$14.20	$13.00
609.23	CURVED GRANITE SLOPE CURB	LF	346	$61.96	A	10	$64.50	$60.00
609.236	CURVED GRANITE SLOPE CURB 6" HIGH	LF	89	$60.21	A	3	$65.00	$60.00
609.5	RESET GRANITE CURB	LF	110034	$6.75	A	21	$13.50	$4.65
			230	$11.50	B	2	$15.00	$8.00
			230	$21.93	C	2	$25.00	$18.85
			3250	$9.00	E	5	$14.00	$7.00
609.55	RESET GRANITE CURB (BRIDGE)	LF	673	$74.49	A	5	$86.70	$60.00
			40	$90.00	C	2	$105.00	$75.00
			820	$80.00	E	2	$100.00	$60.00
609.811	BITUMINOUS CURB, TYPE B (4" REVEAL)	LF	91500	$4.23	A	26	$8.00	$2.00
			220	$14.50	E	2	$15.00	$14.00
611.05206	6" CEMENT LINED DUCTILE IRON WATER PIPE, CL 52	LF	190	$49.21	A	3	$50.00	$35.00
			20	$72.00	B	2	$75.00	$69.00
611.05208	8" CEMENT LINED DUCTILE IRON WATER PIPE, CL 52	LF	60	$73.50	B	2	$75.00	$72.00
611.05212	12" CEMENT LINED DUCTILE IRON WATER PIPE, CL 52	LF	6495	$70.29	A	5	$80.00	$60.00
			2800	$77.00	B	2	$80.00	$74.00
611.05216	16" CEMENT LINED DUCTILE IRON WATER PIPE, CL 52	LF	480	$99.03	A	2	$100.00	$98.05

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
611.05220	20" CEMENT LINED DUCTILE IRON WATER PIPE, CL 52	LF	3000	$117.50	A	2	$135.00	$100.00
611.50107	3/4" COPPER WATER PIPE	LF	150	$28.50	A	2	$50.00	$7.00
611.51007	3/4" CORPORATION STOP	EA	12	$160.00	A	2	$280.00	$40.00
611.52007	3/4" CURB STOP	EA	6	$102.50	A	2	$125.00	$80.00
611.70006	6" FITTING	EA	2	$475.00	A	2	$550.00	$400.00
611.70008	8" FITTING	EA	12	$350.00	A	2	$400.00	$300.00
			6	$385.00	B	2	$520.00	$250.00
611.70010	10" FITTING	EA	4	$420.00	A	2	$500.00	$340.00
611.70012	12" FITTING	EA	80	$666.63	A	6	$1,250.00	$540.00
			64	$602.50	B	2	$755.00	$450.00
611.71006	6" GATE VALVE	EA	6	$1,088.33	A	3	$1,170.00	$900.00
			2	$950.00	B	2	$1,000.00	$900.00
611.71008	8" GATE VALVE	EA	4	$1,167.50	A	2	$1,510.00	$825.00
			2	$1,195.00	B	2	$1,290.00	$1,100.00
611.71012	12" GATE VALVE	EA	9	$2,220.00	A	5	$2,500.00	$1,900.00
			12	$2,100.00	B	2	$2,200.00	$2,000.00
611.74	CHLORINE INJECTION TAP	EA	12	$425.33	A	4	$600.00	$250.00
			12	$850.00	B	2	$1,200.00	$500.00
611.81	HYDRANTS	EA	14	$3,814.29	A	4	$5,000.00	$2,400.00
611.811	ADJUSTING/RELOCATING HYDRANTS	EA	2	$1,825.00	B	2	$2,400.00	$1,250.00
611.90001	ADJUSTING WATER GATES AND SHUTOFFS SET BY OTHERS	EA	27	$123.33	A	3	$150.00	$80.00
611.951	WATER MAIN INSULATION	SY	710	$9.52	A	5	$15.00	$7.00
			40	$14.50	B	2	$20.00	$9.00
614.321	2" STEEL CONDUIT	LF	1516	$13.71	A	7	$16.20	$8.00
614.331	3" STEEL CONDUIT	LF	6560	$15.77	A	9	$17.25	$15.00
614.3429	4 INCH 2-DUCT STEEL CONDUIT (BRIDGE)	LF	254	$52.50	A	2	$60.00	$45.00
614.3439	4 INCH 3-DUCT STEEL CONDUIT (BRIDGE)	LF	690	$85.00	A	2	$85.00	$85.00
614.511	CONCRETE PULL BOX 14"	EA	116	$288.14	A	9	$304.95	$275.00
614.512	CONCRETE PULL BOX 18"	EA	44	$349.09	A	5	$375.00	$325.00
614.522	MOLDED PULL BOX 13"X24"	EA	148	$298.97	A	7	$319.00	$280.00
614.523	MOLDED PULL BOX 17"X30"	EA	280	$322.40	A	9	$380.00	$300.00
			12	$360.00	E	1	$360.00	$360.00
614.72114	2" PVC CONDUIT, SCHEDULE 40	LF	9690	$5.36	A	6	$5.50	$5.00
614.72118	2" PVC CONDUIT, SCHEDULE 80	LF	1716	$8.63	A	3	$9.00	$8.60
614.73114	3" PVC CONDUIT, SCHEDULE 40	LF	50992	$6.69	A	10	$7.50	$6.00
			70	$8.00	E	1	$8.00	$8.00

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614.73118	3" PVC CONDUIT, SCHEDULE 80	LF	16120	$12.44	A	11	$15.00	$9.75
614.74118	4" PVC CONDUIT, SCHEDULE 80	LF	800	$16.25	A	2	$17.30	$15.20
615.003	REMOVING TRAFFIC SIGNS	U	3120	$10.50	B	2	$11.00	$10.00
615.004	RELOCATING TRAFFIC SIGNS	U	18	$150.00	B	2	$200.00	$100.00
615.01	TRAFFIC SIGN TYPE A (F)	SF	1416	$55.19	A	4	$60.00	$41.00
615.012	TRAFFIC SIGN TYPE A, BREAKAWAY MOUNTS (F)	SF	2395	$56.61	A	9	$92.25	$49.00
			1144	$65.96	B	4	$95.00	$41.00
615.013	REMOVING TRAFFIC SIGN TYPE A	U	60	$346.33	A	6	$535.00	$200.00
615.014	RELOCATING TRAFFIC SIGN, TYPE A	U	30	$5,896.00	A	6	$7,125.00	$4,500.00
615.02	TRAFFIC SIGN TYPE B (F)	SF	1631.6	$44.85	A	12	$65.00	$24.00
			442.32	$45.02	B	5	$80.00	$18.00
615.022	TRAFFIC SIGN TYPE B, BREAKAWAY MOUNTS (F)	SF	2893.5	$98.14	A	17	$135.80	$76.00
			562.5	$78.48	B	3	$90.00	$68.00
615.024	RELOCATING TRAFFIC SIGN, TYPE B	U	94	$950.28	A	10	$2,035.00	$100.00
615.03	TRAFFIC SIGN TYPE C (F)	SF	5530.8	$42.49	A	22	$80.00	$24.80
			26125.74	$21.26	B	8	$53.00	$14.00
			120	$47.00	E	2	$50.00	$44.00
615.032	TRAFFIC SIGN TYPE C, BREAKAWAY MOUNTS (F)	SF	144	$112.25	A	4	$125.00	$95.00
			840	$111.00	B	2	$117.00	$105.00
615.034	RELOCATING TRAFFIC SIGN, TYPE C	U	79	$117.35	A	7	$250.00	$100.00
			12	$162.50	B	2	$200.00	$125.00
			6	$250.00	C	2	$300.00	$200.00
			2	$285.00	E	2	$350.00	$220.00
615.04	TRAFFIC SIGN TYPE AA (F)	SF	26496	$17.13	A	11	$18.83	$16.25
			20488	$16.87	B	4	$19.00	$15.00
			400	$27.00	C	2	$35.00	$19.00
			2726	$18.00	E	2	$18.00	$18.00
615.043	REMOVING TRAFFIC SIGN TYPE AA	U	53	$291.51	A	5	$350.00	$250.00
			66	$45.45	B	3	$150.00	$10.00
			18	$210.00	E	2	$210.00	$210.00
615.05	TRAFFIC SIGN TYPE BB (F)	SF	1951.8	$16.93	A	15	$36.50	$11.50
			2675	$15.19	B	3	$16.00	$13.00
615.06	TRAFFIC SIGN TYPE CC (F)	SF	1886.62	$10.67	A	20	$15.40	$9.25
			3746.7	$9.85	B	8	$15.00	$8.00
615.10001	FULL TRAFFIC SIGN STRUCTURE	U	8	$101,800.00	A	8	$168,000.00	$48,400.00
615.20001	CANTILEVER TRAFFIC SIGN STRUCTURE	U	5	$32,068.00	A	5	$36,400.00	$29,000.00
			2	$70,500.00	C	2	$76,000.00	$65,000.00
615.20301	REMOVING CANTILEVER TRAFFIC SIGN STRUCTURE	U	5	$2,471.00	A	5	$2,580.00	$2,375.00

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
615.20301	REMOVING CANTILEVER TRAFFIC SIGN STRUCTURE	U	2	$3,300.00	C	2	$5,000.00	$1,600.00
615.30001	BRIDGE MOUNTED TRAFFIC SIGN STRUCTURE	U	6	$10,228.33	A	6	$12,600.00	$8,000.00
			2	$9,750.00	B	2	$10,000.00	$9,500.00
616.161	TRAFFIC SIGNALS (TEMP.)	U	7	$12,157.14	A	7	$20,000.00	$5,000.00
616.191	ALTERATIONS TO TRAFFIC SIGNALS	U	8	$20,399.69	A	8	$42,900.00	$7,275.00
			1	$84,000.00	E	1	$84,000.00	$84,000.00
616.650	TRAFFIC SIGNAL DETECTOR LOOP 6 FT X 50 FT	EA	26	$500.00	A	3	$500.00	$500.00
			28	$553.57	E	3	$600.00	$550.00
618.7	FLAGGERS	HR	87500	$21.00	A	30	$26.00	$16.50
			17495	$22.81	B	12	$28.00	$18.00
			3816	$18.68	C	7	$24.00	$12.00
			28960	$21.52	E	33	$26.00	$18.00
619.25	PORTABLE CHANGEABLE MESSAGE SIGN	U	101	$6,637.62	A	33	$12,000.00	$1,000.00
			17	$1,340.00	B	5	$2,500.00	$520.00
			8	$6,875.00	C	4	$12,000.00	$1,000.00
			42	$1,828.12	E	19	$3,600.00	$600.00
619.253	PORTABLE CHANGEABLE MESSAGE SIGN (UNIT WEEK)	UWK	578	$247.61	A	14	$300.00	$150.00
			82	$193.66	B	5	$400.00	$50.00
			166	$213.64	E	25	$350.00	$140.00
619.27	TRAILER-MOUNTED SPEED LIMIT SIGN	U	71	$1,887.87	A	18	$6,069.47	$250.00
			5	$1,840.00	B	3	$2,000.00	$1,600.00
			12	$2,158.33	E	4	$4,500.00	$750.00
619.63	TRUCK-MOUNTED IMPACT ATTENUATOR, TEST LEVEL 3	U	23	$13,675.65	A	15	$25,000.00	$2,200.00
			4	$10,500.00	C	4	$20,000.00	$2,000.00
			10	$10,690.00	E	8	$17,500.00	$2,400.00
621.1	RETROREFLECTIVE MEDIAN BARRIER DELINEATOR	EA	180	$16.15	A	2	$17.50	$14.80
			24	$8.00	B	2	$9.00	$7.00
			270	$11.88	E	4	$15.30	$10.00
621.2	RETROREFLECTIVE BEAM GUARDRAIL DELINEATOR	EA	3911	$4.33	A	22	$5.00	$3.95
			1137	$3.16	E	9	$10.00	$3.00
621.31	SINGLE DELINEATOR WITH POST	EA	5594	$29.59	A	28	$35.20	$25.00
			172	$32.44	B	5	$35.00	$30.00
			715	$30.64	E	7	$36.00	$30.00
621.32	DOUBLE DELINEATOR WITH POST	EA	248	$33.58	A	7	$35.00	$31.55
621.33	SINGLE DELINEATOR DOUBLE FACED WITH POST	EA	30	$32.07	A	3	$32.65	$31.55
			1200	$30.00	E	3	$31.00	$29.00
622.1	STEEL WITNESS MARKERS	EA	1473	$27.86	A	19	$31.03	$24.00
			16	$32.50	B	2	$35.00	$30.00
622.2	CONCRETE BOUNDS	EA	330	$280.26	A	7	$330.00	$248.24

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
622.2	CONCRETE BOUNDS	EA	58	$312.50	B	2	$325.00	$300.00
622.4	STONE BOUNDS	EA	74	$275.28	A	4	$290.00	$248.24
625.2	CONCRETE LIGHT POLE BASES, TYPE B	EA	89	$678.82	A	9	$800.00	$600.00
625.52	LIGHT POLE	U	15	$2,283.33	A	3	$2,300.00	$2,250.00
628.2	SAWED BITUMINOUS PAVEMENT	LF	180980	$0.99	A	34	$2.50	$0.50
			5320	$1.31	B	5	$2.00	$1.00
			206	$3.25	C	2	$5.50	$1.00
			53160	$1.16	E	19	$7.00	$0.50
632.0104	RETROREFLECTIVE PAINT PAVE. MARKING, 4" LINE	LF	2788500	$0.10	A	26	$0.20	$0.09
			61400	$0.21	B	7	$1.50	$0.15
			4160	$0.53	C	2	$0.55	$0.50
			2890940	$0.10	E	13	$0.80	$0.08
632.0106	RETROREFLECTIVE PAINT PAVE. MARKING, 6" LINE	LF	4952010	$0.11	A	21	$0.17	$0.10
			883070	$0.12	E	7	$0.21	$0.11
632.0108	RETROREFLECTIVE PAINT PAVE. MARKING, 8" LINE	LF	980	$0.75	A	2	$0.75	$0.75
632.0112	RETROREFLECTIVE PAINT PAVE. MARKING, 12" LINE	LF	93350	$0.20	A	10	$0.25	$0.20
632.0118	RETROREFLECTIVE PAINT PAVE. MARKING, 18" LINE	LF	2185	$2.02	A	11	$3.00	$1.00
632.0124	RETROREFLECTIVE PAINT PAVE. MARKING, 24" LINE	LF	330	$4.00	A	2	$4.00	$4.00
632.02	RETROREFLECTIVE PAINT PAVEMENT MARKING, SYMBOL OR	SF	1360	$1.50	A	2	$1.50	$1.50
632.1104	PREFORMED RETROREFLECTIVE TAPE, TYPE I (REMOVABLE)	LF	19880	$1.01	A	7	$1.58	$0.70
			7402	$1.13	C	2	$1.50	$0.75
			109720	$1.08	E	5	$1.20	$0.85
632.3104	RETROREFLECT. THERMOPLAS. PAVE. MARKING, 4" LINE	LF	58360	$0.55	A	21	$0.75	$0.45
			2100	$0.60	B	2	$0.75	$0.45
			5000	$0.47	E	3	$0.50	$0.45
632.3106	RETROREFLECT. THERMOPLAS. PAVE. MARKING, 6" LINE	LF	28765	$0.83	A	15	$1.10	$0.75
			4600	$0.83	B	2	$1.00	$0.65
			116500	$0.51	E	6	$0.75	$0.41
632.3108	RETROREFLECT. THERMOPLAS. PAVE. MARKING, 8" LINE	LF	11280	$0.88	A	14	$1.32	$0.80
			8340	$1.20	B	2	$1.50	$0.90
			1710	$1.53	E	3	$2.20	$0.95
632.3112	RETROREFLECT. THERMOPLAS. PAVE. MARKING, 12" LINE	LF	102632	$1.54	A	19	$1.85	$1.35
			24600	$1.46	E	5	$1.50	$1.41
632.3118	RETROREFLECT. THERMOPLAS. PAVE. MARKING, 18" LINE	LF	8604	$3.26	A	27	$7.50	$2.00
			470	$3.13	B	2	$4.00	$2.25
			2060	$3.33	E	7	$4.40	$2.35
632.3124	RETROREFLECT. THERMOPLAS. PAVE. MARKING, 24" LINE	LF	758	$5.05	A	4	$6.00	$3.00
632.32	RETROREFLECT. THERMOPLAS. PAVEMENT MARKING, SYMBO	SF	21260	$5.47	A	20	$7.00	$4.25

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
632.32	RETROREFLECT. THERMOPLAS. PAVEMENT MARKING, SYMBO	SF	12	$9.98	B	2	$15.00	$4.95
			10140	$5.41	E	13	$7.00	$3.95
632.6106	PREFORMED RETROREFLECTIVE TAPE, LEVEL II, 6" LINE	LF	2600	$2.00	A	2	$2.00	$2.00
632.92	OBLITERATE PAVEMENT MARKING, SYMBOL OR WORD	SF	4430	$0.95	A	11	$2.00	$0.60
			11600	$1.50	E	6	$2.00	$0.50
641.	LOAM	CY	8165	$21.86	A	18	$45.00	$13.39
			2280	$21.00	B	2	$22.00	$20.00
			212	$27.34	E	3	$33.00	$26.00
643.22	FERTILIZER FOR REFERTILIZATION	TON	36.7	$825.00	A	3	$850.00	$800.00
644.62	WET BASIN/MEADOW SEED TYPE 62	LB	760	$191.59	A	3	$198.00	$185.00
644.82	SALT-TOLERANT GRASS SEED, TYPE 82	LB	50	$63.10	A	2	$64.20	$62.00
645.11	MULCH	A	12	$806.25	A	3	$825.00	$750.00
645.12	TEMPORARY MULCH	A	2.2	$1,237.50	A	2	$1,500.00	$975.00
645.3	EROSION STONE	TON	104185	$19.69	A	26	$40.00	$15.00
			2600	$25.00	B	4	$40.00	$16.00
			120	$27.50	C	2	$35.00	$20.00
			890	$26.82	E	9	$42.00	$20.00
645.51	HAY BALES FOR TEMPORARY EROSION CONTROL	EA	3520	$8.27	A	9	$10.00	$7.00
			100	$11.75	E	2	$15.00	$8.50
645.52	RYEGRASS FOR TEMPORARY EROSION CONTROL	LB	11484	$2.63	A	9	$5.00	$0.75
645.531	SILT FENCE	LF	306370	$1.98	A	28	$3.25	$1.25
			3950	$2.56	B	5	$3.00	$2.25
			1650	$3.00	C	2	$4.00	$2.00
			9820	$2.83	E	14	$4.00	$2.00
645.532	SILT FENCE WITH SUPPORT FENCE	LF	400	$3.75	B	2	$4.00	$3.50
			1200	$3.25	E	2	$4.00	$2.50
645.7	STORM WATER POLLUTION PREVENTION PLAN	U	24	$5,631.25	A	24	$35,000.00	$1,600.00
			5	$1,760.00	B	5	$2,200.00	$1,500.00
			2	$2,600.00	C	2	$2,700.00	$2,500.00
			12	$1,999.50	E	12	$2,650.00	$1,469.00
645.71	MONITORING SWPPP AND EROSION AND SEDIMENT CONTROL	HR	24252	$49.90	A	25	$68.00	$45.00
			495	$67.37	B	4	$80.00	$65.00
			150	$65.00	C	2	$70.00	$60.00
			366	$56.23	E	12	$90.00	$40.00
646.2	TURF ESTABLISHMENT WITHOUT MULCH	A	327.1	$1,595.64	A	7	$3,500.00	$850.00
646.3	TURF ESTABLISHMENT WITH MULCH AND TACKIFIERS	A	104.6	$1,703.97	A	13	$1,950.00	$1,650.00
			31.54	$1,769.34	E	7	$6,000.00	$1,650.00
646.31	TURF ESTABLISHMENT WITH MULCH AND TACKIFIERS	SY	57700	$0.47	A	9	$1.00	$0.40
			20200	$0.52	B	3	$1.00	$0.50

Item		Unit	Quantity * Bids	Average Unit Price	Group Code	No. Of Bidders	High Bid	Low Bid
646.31	TURF ESTABLISHMENT WITH MULCH AND TACKIFIERS	SY	1170	$1.60	C	2	$2.00	$1.20
			7350	$0.84	E	4	$2.00	$0.54
647.1	HUMUS	CY	237390	$15.37	A	31	$34.00	$8.00
			228	$22.15	B	3	$25.00	$20.00
			114	$30.50	C	2	$36.00	$25.00
			2450	$26.74	E	10	$45.00	$14.00
647.22	HUMUS, INTERMIXED, 2" DEE	CY	154	$19.20	A	3	$20.35	$17.75
659.401	LANDSCAPE ESTABLISHMENT CREW (4 MEN- 8 HR DAY)	DAY	16	$1,205.00	A	4	$1,320.00	$1,100.00
670.066	MAILBOX SUPPORT ASSEMBLIES	EA	25	$152.83	A	7	$200.00	$100.00
			6	$182.50	B	2	$200.00	$165.00
			2	$125.00	E	2	$150.00	$100.00
670.0661	MULTIPLE MAILBOXES SUPPORT ASSEMBLIES	EA	4	$225.00	A	2	$225.00	$225.00
670.101	TEMPORARY LIGHTING	U	5	$3,590.00	A	5	$5,600.00	$2,000.00
			2	$3,400.00	C	2	$3,500.00	$3,300.00
670.95	TEMPORARY SAFETY FENCE	LF	2150	$4.51	A	5	$5.00	$3.79
693.	ON-THE-JOB TRAINING OF UNSKILLED WORKERS	$	10	$2,040.00	A	10	$3,000.00	$1,200.00
			2	$600.00	E	2	$600.00	$600.00
698.11	FIELD OFFICE TYPE A	MON	160	$2,076.50	A	5	$2,340.00	$2,000.00
698.12	FIELD OFFICE TYPE B	MON	50	$1,654.00	A	3	$1,750.00	$1,600.00
			36	$1,000.00	C	2	$1,200.00	$800.00
			20	$2,350.00	E	2	$2,700.00	$2,000.00
698.13	FIELD OFFICE TYPE C	MON	166	$1,377.41	A	16	$2,000.00	$900.00
			54	$1,700.00	B	2	$2,000.00	$1,400.00
			20	$975.00	C	3	$1,000.00	$900.00
			20	$1,112.10	E	3	$1,250.00	$906.00
698.2	PHYSICAL TESTING LABORATORY	MON	350	$709.74	A	14	$1,100.00	$400.00
			16	$1,400.00	B	2	$1,500.00	$1,300.00
1002.1	REPAIRS OR REPLACEMENTS AS NEEDED - BRIDGE STRUCTU	$	12	$35,933.17	A	12	$70,000.00	$5,000.00
			6	$2,666.67	C	6	$5,000.00	$1,000.00
			4	$46,000.00	E	4	$90,000.00	$2,000.00

Three Types
Box Beams
Deck Bulb Tees
AASHTO Standard I Beams

Concrete

f'_c	= compressive strength of concrete at service	7000	psi
f'_{ci}	= compressive strength of concrete at time of initial prestress	5500	psi

Strands

Diameter	0.5	in
Wires	7	#
Strength	270	ksi
Center to Center Spacing	2	in
Initial Tension	202.5	ksi

None of the Precast Elements have span lengths of greater than 130'
Therefore two spans with a central pier is required.

Total Span Length 150 ft
Assume two equal spans Total Span Width 76 ft

Span 1 75 ft
Span 2 75 ft

Assume simple at central pier

Using PCI Preliminary Design Charts

AASHTO BOX BEAM 48" Wide — Chart BB-1 through BB-5

Depth (in)	Allowable Beam Spacing (ft)	Number of Beam Required, Assuming Max Overhang of .5 Beam Spacing	Number of Strands Per Beam
27	6	13.0000	40
33	8.33	10.0000	34
39	10.66	8.0000	38
42	12	7.0000	42
	(rough estimate)		

AASHTO BOX BEAM 36" Wide — Chart BB-6 through BB-5

Depth (in)	Allowable Beam Spacing (ft)	Number of Beam Required, Assuming Max Overhang of .5 Beam Spacing	Number of Strands Per Beam
27	5.5	14.00	41
33	7.5	11.00	36
39	9.66	9.00	38
42	11	7.00	50

7	7
9	
10	
14	

Deck Bulb Tee

65" DBT can be used to span over 150'

	Chart DBT-1, DBT-4		
Depth (in)	Allowable Beam Spacing (ft)	Number of Beam Required, Assuming Max Overhang of .5 Beam Spacing	Number of Strands Per Beam
35		Insufficient Capacity	
53		Insufficient Capacity	
65	5.75	14.0	57

Looking at the the Problem as two equal simple spans
Span Length= 75'

	Chart DBT-1, DBT-2		
Depth (in)	Allowable Beam Spacing (ft)	Number of Beam Required, Assuming Max Overhang of .5 Beam Spacing	Number of Strands Per Beam
35	6.66	12.0	35
53		Excess Capacity	
65		Excess Capacity	

AASHTO I Beams

Type VI can be used to span over 150'   Chart DBT-1, DBT-4

Type	Depth (in)	Allowable Beam Spacing (ft)	Number of Beam Required, Assuming Max Overhang of .5 Beam Spacing	Number of Strands Per Beam
I	28		Insufficient Capacity	
II	36		Insufficient Capacity	
III	45		Insufficient Capacity	
IV	54		Insufficient Capacity	
V	63		Insufficient Capacity	
VI	72	8	10	47

Looking at the the Problem as two equal simple spans

Span Length= 75'   Chart DBT-1, DBT-2

Type	Depth (in)	Allowable Beam Spacing (ft)	Number of Beam Required, Assuming Max Overhang of .5 Beam Spacing	Number of Strands Per Beam
I	28		Insufficient Capacity	
II	36		Insufficient Capacity	
III	45	9.75	8	28
IV	54		Excess Capacity	
V	63		Excess Capacity	
VI	72		Excess Capacity	

Pier Wall (Typical)

Length	70	
Height	32	
Thickness	2	
Volume		165.9259
Footing		
Width	12	
Length	70	
Depth	4	
Volume		124.4444
Total Volume		290.3704
Cost Installed		
Concrete Class B, Installed Average	$ 350.00	per CY
Total Estimated Cost	$ 101,629.63	Per Pier Wall
	101629.6296	

Bridge Deck			
Thickness	0.666666667	ft	
Width	76	ft	
Length	150	ft	
Weight	150	pcf	
Area	11400	ft^2	
Volume	281.4814815	CY	
Cost Installed			
Concrete Bridge Deck Average	$750.00	per CY	13933N Item List
Deck Cost	$ 211,111.11		
	211111.1111		

Structural Element	Cross Sectional Area (In^2)	Strands	Total Steel Per Member (LF)
AASHTO BOX BEAM 48" X 27"	692.5	40	3000
AASHTO BOX BEAM 48" X 33"	752.5	34	2550
AASHTO BOX BEAM 48" X 39"	812.5	38	2850
AASHTO BOX BEAM 48" X 42"	842.5	42	3150
AASHTO BOX BEAM 36" X 27"	560.5	41	3075
AASHTO BOX BEAM 36" X 33"	620.5	36	2700
AASHTO BOX BEAM 36" X 39"	680.5	38	2850
AASHTO BOX BEAM 36" X 42"	710.5	50	3750
DECK BULB TEE 65"	1003	57	8550
DECK BULB TEE 35"	850	35	2625
TYPE IV AASHTO I BEAM	1085	47	7050
TYPE III AASHTO I BEAM	560	28	2100

Cost Numbers		
Concrete	$ 97.57	per CY
Steel Strands	$ 725.75	per Ton
Concrete Class AA Super Structure(Precast, Installed)	$ 600.00	per CY
Concrete Class B (Footings, Substructure), Installed Average	$ 350.00	per CY
Elastomeric Bearings	$ 2,750.00	per ea.

Total Concrete Per Member (CY)	Number of Members In Bridge Cross Section	Spans	Total Number of Members in Bridge	Total Steel (LF)	Total Steel Cost
13.36	13	2	26	78000	$ 14,718.18
14.52	10	2	20	51000	$ 9,623.43
15.67	8	2	16	45600	$ 8,604.48
16.25	7	2	14	44100	$ 8,321.43
10.81	14	2	28	86100	$ 16,246.61
11.97	11	2	22	59400	$ 11,208.46
13.13	9	2	18	51300	$ 9,680.04
13.71	7	2	14	52500	$ 9,906.47
38.70	14	1	14	119700	$ 22,586.75
16.40	12	2	24	63000	$ 11,887.76
41.86	10	1	10	70500	$ 13,302.97
10.80	8	2	16	33600	$ 6,340.14

Total Concrete (CY)	Total Concrete Cost	Total Material Cost for Super Structure	Requires Topping (or Precast Panel Deck)	Volume of Deck Already Contained(Box Beam CY)
347.3186728	$ 33,887.88	$ 48,606.07	Yes	192.5925926
290.316358	$ 28,326.17	$ 37,949.59	Yes	148.1481481
250.7716049	$ 24,467.79	$ 33,072.26	Yes	118.5185185
227.5270062	$ 22,199.81	$ 30,521.24	Yes	103.7037037
302.7391975	$ 29,538.26	$ 45,784.87	Yes	155.5555556
263.3294753	$ 25,693.06	$ 36,901.52	Yes	122.2222222
236.2847222	$ 23,054.30	$ 32,734.34	Yes	100
191.878858	$ 18,721.62	$ 28,628.09	Yes	77.77777778
541.7438272	$ 52,857.95	$ 75,444.69	No	0
393.5185185	$ 38,395.60	$ 50,283.36	No	0
418.595679	$ 40,842.38	$ 54,145.35	Yes	0
172.8395062	$ 16,863.95	$ 23,204.09	Yes	0

Additional Deck Cost	Number of Required Elastomeric Bearings	Total Bearing Cost	Total Estimated Cost	Piers between Abutments
$ 66,666.67	26	$48,712.82	$ 338,488.87	1
$ 100,000.00	20	$37,471.40	$ 321,284.64	1
$ 122,222.22	16	$29,977.12	$ 311,266.78	1
$ 133,333.33	14	$26,229.98	$ 304,400.95	1
$ 94,444.44	28	$52,459.96	$ 344,794.53	1
$ 119,444.44	22	$41,218.54	$ 329,869.13	1
$ 136,111.11	18	$33,724.26	$ 321,286.24	1
$ 152,777.78	14	$26,229.98	$ 304,041.54	1
$ -	28	$52,459.96	$ 400,093.01	0
$ -	24	$44,965.68	$ 292,964.55	1
$ 211,111.11	20	$37,471.40	$ 513,042.89	0
$ 211,111.11	16	$29,977.12	$ 351,132.08	1

Total Pier Cost	Total Bridge Structure Estimated Cost (Stringers, Deck + Pier)		
$ 101,629.63	$ 440,118.50		
$ 101,629.63	$ 422,914.27		
$ 101,629.63	$ 412,896.41		
$ 101,629.63	$ 406,030.58		
$ 101,629.63	$ 446,424.16		
$ 101,629.63	$ 431,498.76		
$ 101,629.63	$ 422,915.87		
$ 101,629.63	$ 405,671.17		
$ -	$ 400,093.01	Is ths Feasable?	Depth Works
$ 101,629.63	$ 394,594.18		
$ -	$ 513,042.89		
$ 101,629.63	$ 452,761.70		

Steel Pre-Design Study Estimate

Single Span 75'

Girders:	8
Girder Weight (lbs):	13679
Avg. Unit Price ($/lb):	9.65
Girder Estimate ($):	1056018.8
Shear Connect. D=6":	45
Unit Cost ($/#):	5.13
Connector Estimate ($):	1846.8
Pier Volume :	1856.45
Unit Cost:	300
Pier Estimate ($):	556935
Span Estimate ($):	$ 2,926,228.15

Single Span 75'

Girders:	8
W36x231 Weight (lbs):	15750
Unit Cost ($/lb):	RS means->Material Bare Cost is 320
W36x210 Weight (lbs):	15750
Avg. Unit Price ($/lb):	9.65
W36x210 Estimate ($):	1215900
Shear Connect. D=6":	32
Unit Cost ($/#):	5.13
Connector Estimate ($):	1313.28
Pier Volume:	1322.93
Unit Cost:	300
Pier Estimate:	396879
Span Estimate ($):	$ 3,084,867.51

Single Span 140'

Girders:	8
Girder Weight (lbs):	39336
Avg. Unit Price ($/lb):	9.65
Girder Estimate ($):	3036739.2

Shear Connect. D=9":	20
Unit Cost ($/#):	5.13
Connector Estimate ($):	820.8
Span Estimate ($):	$ 3,291,121.95

Bridge Deck		
Thickness	0.666666667	ft
Width	76	ft
Length	150	ft
Weight	150	pcf
Area	11400	ft^2
Volume	281.4814815	CY
Cost Installed		
Concrete Bridge Deck Average	$887.50	per CY
Deck Cost	$ 249,814.81	
Elastomeric Bearings	$ 1,873.57	per ea.

Preliminary Price Estimate Summary Table

Superstructure Type	Total Reinforced Concrete For Structural Member (CY)	Total Structural Steel For Structural Members (Tons)	Total Reinforced Concrete for Deck not included in members	Total Pier Walls Required	Total Bearings Required	Total Price
Simple Two Span Precast	426.31	0	0	1	52	$ 500,415.50
Continuous Two Span Steel	0	61.64	308.33	1	24	$ 610,918.60
Single Span Steel	0	189.875	308.33	0	16	$ 928,417.50

Unit	Prices per unit According to NH DOT Contract 13933N
CY of Superstructure Concrete	$ 600.00
Pound of Structural Steel	$ 1.72
CY of Pier Concrete	$ 350.00
CY of Deck Concrete	$ 750.00
Elastomeric Bearing	$ 2,750.00

Appendix 3.1: Superstructure Design

This section contains the calculations used to design the primary bridge girders. The primary bridge girders are Deck Bulb Tees, and are designed for both moment and shear. The moment calculations are limited to positive moment, because it is assumed that both spans use simple end conditions.

AASHTO Moment Calculator for Precast Bridge

This calculation includes the calculation of the governing AASHTO load condition for the proposed bridge design for moment. The outputs of these calculations are the magnitudes of the positive movements created by various load conditions on the bridge girders.

AASHTO Shear Calculator for Precast Bridge

This calculation includes the calculation of the governing AASHTO load conditions for the proposed bridge design for shear. The outputs of these calculations are the magnitudes of the shear forces created by various load conditions on the bridge girders, at different positions along their length.

PG Super Imputs

This section is a table of the imput values into the PG-Super Program.

Bridge Geometry Report

This section is a report produced by PG super that checks the geometry of the proposed bridge against Federal specification, and the specification of the two states that created the program. It reports the coordinate positions of individual bridge components.

Spec Check Report

This section is a report produced by PG super that checks the flexural properties of the proposed bridge against, AASHTOP specifications, Federal specifications, and the design specification of the two states that created the program. The bridge is given a pass/fail for each section.

Flexural Prestressed Design

This section contains the long form calculations according to AASHTO to design the prestressed bridge girders for positive moment, transfer and service, and the placement and magnitude of their prestressing.

Shear Design Calculator for Final Precast Bridge

This section contains the long form calculations to design the prestressed bridge girders for shear, and the placement and magnitude of shear reinforcing.

Moment Calculator

Permanent Loads		Exterior Distı	Exterior Mor	Interior Distr	Interior Moment
Dead load of structural components and nonstructural attachments	**DC***	1.30542	**917.87**	0.88542	**622.56**
Downdrag force	**DD**	0	**0**	0	**0**
Dead load of wearing surfaces and utilities	**DW**	0.21	**147.66**	0.21	**147.65625**
Horizontal earth pressure	**EH**	0	**0**	0	**0**
Vertical pressure from dead load of earth fill	**EV**	0	**0**	0	**0**
Earth Surcharge Load	**ES**	0	**0**	0	**0**
Misc. locked-in force effects resulting from the construction process	**EL**	0	**0**	0	**0**
Secondary forces from post-tensioning	**PS**	0	**0**	0	**0**
Force effects due to creep	**CR**	0	**0**	0	**0**
Force effects due to shrinkage	**SH**	0	**0**	0	**0**

* Value determined using load analysis software

Span Length,	75

	Interior Beams Moment, k-ft			
	One Design Lane Loaded		) or More Design Lanes Loa	
	Maximum	Minimum	Maximum	Minimum
Strength I	**3883.99**	**3540.58**	**7134.69**	**6791.28**
Strength II	**3224.72**	**2881.31**	**5732.40**	**5389.00**
Strength III	**999.69**	**656.28**	**999.69**	**656.28**
Strength IV	**1155.33**	**656.28**	**1155.33**	**656.28**
Strength V	**3224.72**	**2881.31**	**5732.40**	**5389.00**
Extreme Eve	**1823.77**	**1480.37**	**2752.54**	**2409.14**
Extreme Eve	**1823.77**	**1480.37**	**2752.54**	**2409.14**
Service I		**2418.39**		**4275.93**
Service II		**2912.84**		**5327.65**
Service III		**2088.76**		**3574.79**
Service IV		**770.22**		**770.22**
Fatigue I		**2367.93**		**5154.25**
Fatigue II		**1183.97**		**2577.12**

	xterior Beams Moment, k-1	
	One Lane Loaded	
	Maximum	Minimum
Strength I	**3316.35**	**2869.59**
Strength II	**2871.20**	**2424.44**
Strength III	**1368.83**	**922.06**
Strength IV	**1598.29**	**922.06**
Strength V	**2871.20**	**2424.44**
Extreme Eve	**1925.26**	**1478.50**
Extreme Eve	**1925.26**	**1478.50**
Service I	**2178.40**	
Service II	**2512.26**	
Service III	**1955.83**	
Service IV	**1065.53**	
Fatigue I	**1564.98**	
Fatigue II	**782.49**	

Distribution of Live Loads for Moment in Interior Beams		
Beam Spacin	72	
Slab Thickne	6	
One Design L	**0.7781513**	
Two or More	**1.69379**	
Number of La	1	
Multiple Pres	1.2	
Distribution I	0.4285714	
Distribution Factor for Ext		**0.5142857**

Longitudinal Stiffness Parameter, K_g		
Modulus of Elasticity of Be		5255.14
Modulus of Elasticity of De		5255.14
Moment of Intertia of Bea		116071
Distance between centers of gravity of beam and deck, inches		11.96
Area of Beam, in^2		850

Kg	**237656.36**

Load Factors

Transient Loads		Load at Midspan, Ki	Momen
Vehicular live load	**LL***		1525.326
Vehicular centrifugal force	**CE**	0	0
Vehicular braking force	**BR***		69.55
Pedestrian live load	**PL**	0	0
Live load surcharge	**LS**	0	0
Water load and stream pressure	**WA**	0	0
Wind load on structure	**WS**		0
Wind on live load	**WL**		0
Friction load	**FR**	0	0
Force effect due to uniform temperature	**TU**	0	0
Force effect due to temperature gradient	**TG**	0	0
Force effect due to settlement	**SE**	0	0
Earthquake load	**EQ**	0	0
Blast loading	**BL**	0	0
Ice load	**IC**	0	0
Vehicular collision force	**CT**	0	0
Vessel collision force	**CV**	0	0

* Value determined using load analysis software

Dynamic Load Allowance, IM %	0.33

Component	IM
Deck Joints—All Limit States	75%
All Other Components:	
• Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

: for Permanent Loads, γ

Load Fact

Load Fact

	Maximum	Minimum
DC	1.25	0.9
DD	0	0
DW	1.5	0.65
EH	0	0
EL	1	1
EV	0	0
ES	0	0
PS	0	
CR	0	
SH	0	

Bridge Component	*PS*	*CR, SH*
Superstructures—Segmental Concrete Substructures supporting Segmental Superstructures (see 3.12.4, 3.12.5)	1.0	See γ_P for *DC*, Table 3.4.1-2
Concrete Superstructures—non-segmental	1.0	1.0
Substructures supporting non-segmental Superstructures • using I_g • using $I_{effectuve}$	 0.5 1.0	 0.5 1.0
Steel Substructures	1.0	1.0

Type of Load, Foundation Type, and Method Used to Calculate Downdrag		Load Factor	
		Maximum	Minimum
DC: Component and Attachments *DC:* Strength IV only		1.25 1.50	0.90 0.90
DD: Downdrag	Piles, α Tomlinson Method Piles, λ Method Drilled shafts, O'Neill and Reese (1999) Method	1.4 1.05 1.25	0.25 0.30 0.35
DW: Wearing Surfaces and Utilities		1.50	0.65
EH: Horizontal Earth Pressure • Active • At-Rest • *AEP* for anchored walls		 1.50 1.35 1.35	 0.90 0.90 N/A
EL: Locked-in Construction Stresses		1.00	1.00
EV: Vertical Earth Pressure • Overall Stability • Retaining Walls and Abutments • Rigid Buried Structure • Rigid Frames • Flexible Buried Structures o Metal Box Culverts and Structural Plate Culverts with Deep Corrugations o Thermoplastic culverts o All others		 1.00 1.35 1.30 1.35 1.5 1.3 1.95	 N/A 1.00 0.90 0.90 0.9 0.9 0.9
ES: Earth Surcharge		1.50	0.75

ors for Transient Loads (Strength States)	
TU	0
TG	
SE	

tors for Transient Loads (Service States)	
TU	0
TG	0.5
SE	

AASHTO Shear Calculator

Permanent Loads		Exterior Di	Exterior Sh	Interior Dis	Interior Shear, kips
Dead load of structural components	**DC**	1.30542	**48.95**	0.88542	**33.20**
Downdrag force	**DD**	0	**0**	0	**0**
Dead load of wearing surfaces and	**DW**	0.21	**7.88**	0.21	**7.88**
Horizontal earth pressure	**EH**	0	**0**	0	**0**
Vertical pressure from dead load of	**EV**	0	**0**	0	**0**
Earth Surcharge Load	**ES**	0	**0**	0	**0**
Misc. locked-in force effects resulting	**EL**	0	**0**	0	**0**
Secondary forces from post-tensi	**PS**	0	**0**	0	**0**
Force effects due to creep	**CR**	0	**0**	0	**0**
Force effects due to shrinkage	**SH**	0	**0**	0	**0**

Span Lengt	75

	Interior Beams Shear, kips			
	)ne Design Lane Loade		r More Design Lanes L	
	Maximum	Minimum	Maximum	Minimum
Strength I	**172.84**	**154.53**	**185.44**	**167.12**
Strength II	**145.52**	**127.21**	**155.24**	**136.92**
Strength II	**53.32**	**35.00**	**53.32**	**35.00**
Strength IV	**61.62**	**35.00**	**61.62**	**35.00**
Strength V	**145.52**	**127.21**	**155.24**	**136.92**
Extreme Ev	**87.47**	**69.15**	**91.07**	**72.75**
Extreme Ev	**87.47**	**69.15**	**91.07**	**72.75**
Service I		**109.38**		**116.58**
Service II		**129.87**		**139.23**
Service III		**95.72**		**101.48**
Service IV		**41.08**		**41.08**
Fatigue I		**102.45**		**113.25**
Fatigue II		**51.23**		**56.62**

	terior Beams Shear, ki	
	One Lane Loaded	
	Maximum	Minimum
Strength I	**177.19**	**153.36**
Strength II	**153.38**	**129.55**
Strength II	**73.00**	**49.18**

Strength IV	85.24	49.18
Strength V	153.38	129.55
Extreme Ev	102.77	78.94
Extreme Ev	102.77	78.94
Service I		116.36
Service II		134.22
Service III		104.46
Service IV		56.83
Fatigue I		89.30
Fatigue II		44.65

Longitudinal Stiffness Paramete	
Modulus of Elasticity (	5255.14
Modulus of Elasticity (	5255.14
Moment of Intertia of	116071
Distance between centers of	11.96
Area of Beam, in^2	850

Kg	237656.4

ve Loads for Moment		
Beam Spac	5.75	
Slab Thickn	6	
One Desigr	0.59	
Two or Mo	0.65	
Number of	1	
Multiple Pr	1.2	
Distributio	0.428571	
Distribution Factor fo		0.514286

Transient Loads	
Vehicular live load	**LL***
Vehicular centrifugal force	**CE**
Vehicular braking force	**BR***
Pedestrian live load	**PL**
Live load surcharge	**LS**
Water load and stream pressure	**WA**
Wind load on structure	**WS**
Wind on live load	**WL**
Friction load	**FR**
Force effect due to uniform temperature	**TU**
Force effect due to temperature gradient	**TG**
Force effect due to settlement	**SE**
Earthquake load	**EQ**
Blast loading	**BL**
Ice load	**IC**
Vehicular collision force	**CT**
Vessel collision force	**CV**

* Value determined using load analysis software

Dynamic Load Allowance, IM %	0.33

Component	IM
Deck Joints—All Limit States	75%
All Other Components:	
• Fatigue and Fracture Limit State	15%
• All Other Limit States	33%

Load at Midspan, Kips	Shear, kips
	87.04
0	0
	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0
0	0

Load Factors for Permanent L	
	Maximum
DC	1.25
DD	0
DW	1.5
EH	0
EL	1
EV	0
ES	0
PS	(
CR	(
SH	(

PG Super Program Imputs

PG Super Entry Field	Input for Analysis
Bridge Alignment	N 75 W
Girder Spacing	6ft
Girder Family	DBT
Girder Type	W35DG
Number of Girders	13
Same number of Girders in all Spans?	Yes
Spacing Type	Adjacent
Girder Connectivity	Sufficient to force girders to act as a unit
Girder Orientation	Normal to road at start of bridge
Additional Deck	No (Deck Bulb Tee) Girders
Span 1 Length	75ft
Span 2 Length	75ft
Abutment 1 Alignment	N 12 W
Pier 1 Alignment	N 12 W
Abutment 2 Alignment	N 12 W
Bridge Barrier Type	F-shape
Wearing Surface	3" HMA
Deck Reinforcement	No Deck
Corosion Conditions	Normal
Relative Humidity	75%
F'ci (Initial Estimate)	4800
F'c (Initial Estimate)	6000
Longitudinal Mild Steel	None
Transverse Mild Steel Reinforcement	Yes
Lift Loop Locations	1.75ft
Transportation Bearing Locations	5ft
Bridge Condition	Good
Units	US
Analysis Style	Simple Supported
Specification	AASHTO LRFD Specifications
Load Rating Criteria	AASHTO Bridge Manual
Loads	Design, and Legal Load States
Truck Load	HL-93 Truck
ADTT	4680
Dynamic Allowance	10% Truck 0% Lane
Permit Load Check	No
Rate For Shear	Yes
Effective Flange Widths	LRFD Calculations for effective flange widths
Additional Loads/Moments	None
Design for Flexure/ Shear	Yes

Bridge Geometry Report

February 23, 2013 12:36:55 pm

PGSuper™

Version 2.5.1 - Built on Aug 17 2011

Washington State Department of Transportation

Texas Department of Transportation

Project Properties

Bridge Name	MQP New
Bridge ID	
Company	MQP
Engineer	Joshua Nitso
Job Number	
Comments	
File	C:\Users\Nitso\Desktop\MQP Final.pgs

Library Usage

Master Library Publisher: WSDOT
Library and Template Package URL: ftp://ftp.wsdot.wa.gov/public/bridge/Software/PGSuper/Version_2.5.0/WSDOT.pgz
Master Library Date Stamp: December 12, 2011 11:46:13 am

Library	Entry	Source
Connections	End Type A (W35DG)	Master Library
Girders	W35DG	Master Library
Traffic Barriers	32" F Shape	Master Library
Project Criteria	WSDOT LRFD - US Units	Master Library
Vehicular Live Load	OL1	Master Library
Vehicular Live Load	OL2	Master Library
Load Rating Criteria	WSDOT	Master Library

Notes

Symbol	Definition
L_g	Length of Girder
L_s	Length of Span
FoS	Face of Support
Debond	Point where bond begins for a debonded strand
PSXFR	Point of prestress transfer
CS	Critical Section for Shear
H	H from end of girder or face of support
1.5H	1.5H from end of girder or face of support
HP	Harp Point
Pick Point	Support point where girder is lifted from form
Bunk Point	Point where girder is supported during transportation

Status Items

Level	Description
Info	All Live Load Distribution Factors are computed using the Lever Rule.
Warning	Left lift point is less than the minimum value of 3.000 ft

Alignment

Alignment Details

Direction: N 75° 00' 00.00" W
Ref. Point: 0+00.00 (E (X) 0.0000, N (Y) 0.0000)

Profile Details

Station: 0+00.00
Elevation: 0.0000 ft
Grade: 0%

Superelevation Details

Section	Station	Left Slope (ft/ft)	Right Slope (ft/ft)	Crown Point Offset (ft)
1	0+00.00	-0.02	-0.02	0.0000

Deck Elevations

Deck Elevations over Girder Webs

Notes
Web Offsets are measured from and normal to the centerline girder
Station, normal offset, and deck elevations are given for 10th points between bearings

Span 1

Girder	Web		CL Brg	$0.1L_s$	$0.2L_s$	$0.3L_s$	$0.4L_s$	$0.5L_s$	$0.6L_s$	$0.7L_s$	$0.8L_s$	$0.9L_s$	CL Brg
A	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+86.16	0+93.18	1+00.21	1+07.23	1+14.25	1+21.27	1+28.30	1+35.32	1+42.34	1+49.37	1+56.39
		Offset (ft)	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L
		Elev (ft)	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690
B	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+87.38	0+94.40	1+01.43	1+08.45	1+15.47	1+22.50	1+29.52	1+36.54	1+43.57	1+50.59	1+57.61
		Offset (ft)	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L
		Elev (ft)	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575
C	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+88.60	0+95.63	1+02.65	1+09.67	1+16.70	1+23.72	1+30.74	1+37.76	1+44.79	1+51.81	1+58.83
		Offset (ft)	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L
		Elev (ft)	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460
D	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+89.83	0+96.85	1+03.87	1+10.90	1+17.92	1+24.94	1+31.96	1+38.99	1+46.01	1+53.03	1+60.06
		Offset (ft)	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L
		Elev (ft)	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345
E	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+91.05	0+98.07	1+05.09	1+12.12	1+19.14	1+26.16	1+33.19	1+40.21	1+47.23	1+54.25	1+61.28
		Offset (ft)	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L
		Elev (ft)	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230
F	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+92.27	0+99.29	1+06.32	1+13.34	1+20.36	1+27.39	1+34.41	1+41.43	1+48.45	1+55.48	1+62.50
		Offset (ft)	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L
		Elev (ft)	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115
G	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+93.49	1+00.52	1+07.54	1+14.56	1+21.58	1+28.61	1+35.63	1+42.65	1+49.68	1+56.70	1+63.72
		Offset (ft)	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L
		Elev (ft)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
H	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+94.72	1+01.74	1+08.76	1+15.78	1+22.81	1+29.83	1+36.85	1+43.88	1+50.90	1+57.92	1+64.94
		Offset (ft)	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R
		Elev (ft)	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115
I	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+95.94	1+02.96	1+09.98	1+17.01	1+24.03	1+31.05	1+38.07	1+45.10	1+52.12	1+59.14	1+66.17
		Offset (ft)	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R
		Elev (ft)	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230
J	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+97.16	1+04.18	1+11.21	1+18.23	1+25.25	1+32.27	1+39.30	1+46.32	1+53.34	1+60.37	1+67.39
		Offset (ft)	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R
		Elev (ft)	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345
K	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+98.38	1+05.40	1+12.43	1+19.45	1+26.47	1+33.50	1+40.52	1+47.54	1+54.57	1+61.59	1+68.61
		Offset (ft)	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R

Girder	Web		CL Brg	$0.1L_s$	$0.2L_s$	$0.3L_s$	$0.4L_s$	$0.5L_s$	$0.6L_s$	$0.7L_s$	$0.8L_s$	$0.9L_s$	CL Brg
		Elev (ft)	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460
L	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	0+99.60	1+06.63	1+13.65	1+20.67	1+27.70	1+34.72	1+41.74	1+48.76	1+55.79	1+62.81	1+69.83
		Offset (ft)	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R
		Elev (ft)	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575
M	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+00.83	1+07.85	1+14.87	1+21.89	1+28.92	1+35.94	1+42.96	1+49.99	1+57.01	1+64.03	1+71.06
		Offset (ft)	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R
		Elev (ft)	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690

Span 2

Girder	Web		CL Brg	$0.1L_s$	$0.2L_s$	$0.3L_s$	$0.4L_s$	$0.5L_s$	$0.6L_s$	$0.7L_s$	$0.8L_s$	$0.9L_s$	CL Brg
A	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+61.16	1+68.18	1+75.21	1+82.23	1+89.25	1+96.27	2+03.30	2+10.32	2+17.34	2+24.37	2+31.39
		Offset (ft)	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L	34.500 L
		Elev (ft)	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690
B	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+62.38	1+69.40	1+76.43	1+83.45	1+90.47	1+97.50	2+04.52	2+11.54	2+18.57	2+25.59	2+32.61
		Offset (ft)	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L	28.750 L
		Elev (ft)	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575
C	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+63.60	1+70.63	1+77.65	1+84.67	1+91.70	1+98.72	2+05.74	2+12.76	2+19.79	2+26.81	2+33.83
		Offset (ft)	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L	23.000 L
		Elev (ft)	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460
D	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+64.83	1+71.85	1+78.87	1+85.90	1+92.92	1+99.94	2+06.96	2+13.99	2+21.01	2+28.03	2+35.06
		Offset (ft)	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L	17.250 L
		Elev (ft)	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345
E	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+66.05	1+73.07	1+80.09	1+87.12	1+94.14	2+01.16	2+08.19	2+15.21	2+22.23	2+29.25	2+36.28
		Offset (ft)	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L	11.500 L
		Elev (ft)	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230
F	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+67.27	1+74.29	1+81.32	1+88.34	1+95.36	2+02.39	2+09.41	2+16.43	2+23.45	2+30.48	2+37.50
		Offset (ft)	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L	5.750 L
		Elev (ft)	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115
G	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+68.49	1+75.52	1+82.54	1+89.56	1+96.58	2+03.61	2+10.63	2+17.65	2+24.68	2+31.70	2+38.72
		Offset (ft)	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000 L	0.000	0.000 L	0.000 L	0.000 L	0.000 L
		Elev (ft)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
H	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+69.72	1+76.74	1+83.76	1+90.78	1+97.81	2+04.83	2+11.85	2+18.88	2+25.90	2+32.92	2+39.94
		Offset (ft)	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R	5.750 R
		Elev (ft)	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115	-0.115
I	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+70.94	1+77.96	1+84.98	1+92.01	1+99.03	2+06.05	2+13.07	2+20.10	2+27.12	2+34.14	2+41.17
		Offset (ft)	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R	11.500 R
		Elev (ft)	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230	-0.230
J	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+72.16	1+79.18	1+86.21	1+93.23	2+00.25	2+07.27	2+14.30	2+21.32	2+28.34	2+35.37	2+42.39
		Offset (ft)	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R	17.250 R
		Elev (ft)	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345	-0.345
K	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+73.38	1+80.40	1+87.43	1+94.45	2+01.47	2+08.50	2+15.52	2+22.54	2+29.57	2+36.59	2+43.61
		Offset (ft)	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R	23.000 R
		Elev (ft)	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460	-0.460
L	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+74.60	1+81.63	1+88.65	1+95.67	2+02.70	2+09.72	2+16.74	2+23.76	2+30.79	2+37.81	2+44.83
		Offset (ft)	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R	28.750 R
		Elev (ft)	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575	-0.575
M	1	**Web Offset (ft)**	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
		Station	1+75.83	1+82.85	1+89.87	1+96.89	2+03.92	2+10.94	2+17.96	2+24.99	2+32.01	2+39.03	2+46.06
		Offset (ft)	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R	34.500 R
		Elev (ft)	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690	-0.690

Pier Geometry

Pier Layout

	Station	Bearing	Skew Angle	Alignment Intersection		
				East (X)	North (Y)	Elev (ft)
Abutment 1	0+90.00	S 3° 00' 00.00" W	12° 00' 00.00" L	-86.9333	23.2937	0.0000
Pier 2	1+65.00	S 3° 00' 00.00" W	12° 00' 00.00" L	-159.3778	42.7051	0.0000
Abutment 3	2+40.00	S 3° 00' 00.00" W	12° 00' 00.00" L	-231.8222	62.1166	0.0000

Girder Geometry

Girder Points

Span 1

Girder	Start of Girder									End of Girder								
	CL Pier			Girder End			CL Bearing			CL Bearing			Girder End			CL Pier		
	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)
A	-88.7793	-11.9287	-0.6900	-89.8491	-11.6420	-0.6900	-91.0834	-11.3113	-0.6900	-158.9195	6.8653	-0.6900	-160.1539	7.1961	-0.6900	-161.2237	7.4827	-0.6900
B	-88.4716	-6.0583	-0.5750	-89.5414	-5.7716	-0.5750	-90.7758	-5.4409	-0.5750	-158.6119	12.7357	-0.5750	-159.8462	13.0665	-0.5750	-160.9160	13.3531	-0.5750
C	-88.1639	-0.1879	-0.4600	-89.2337	0.0988	-0.4600	-90.4681	0.4295	-0.4600	-158.3042	18.6061	-0.4600	-159.5386	18.9369	-0.4600	-160.6084	19.2235	-0.4600
D	-87.8563	5.6825	-0.3450	-88.9261	5.9692	-0.3450	-90.1605	6.2999	-0.3450	-157.9965	24.4765	-0.3450	-159.2309	24.8073	-0.3450	-160.3007	25.0939	-0.3450
E	-87.5486	11.5529	-0.2300	-88.6184	11.8396	-0.2300	-89.8528	12.1703	-0.2300	-157.6889	30.3469	-0.2300	-158.9233	30.6777	-0.2300	-159.9931	30.9643	-0.2300
F	-87.2410	17.4233	-0.1150	-88.3108	17.7100	-0.1150	-89.5452	18.0407	-0.1150	-157.3812	36.2173	-0.1150	-158.6156	36.5481	-0.1150	-159.6854	36.8347	-0.1150
G	-86.9333	23.2937	0.0000	-88.0031	23.5804	0.0000	-89.2375	23.9111	0.0000	-157.0736	42.0877	0.0000	-158.3080	42.4185	0.0000	-159.3778	42.7051	0.0000
H	-86.6257	29.1641	-0.1150	-87.6955	29.4508	-0.1150	-88.9298	29.7815	-0.1150	-156.7659	47.9581	-0.1150	-158.0003	48.2889	-0.1150	-159.0701	48.5755	-0.1150
I	-86.3180	35.0345	-0.2300	-87.3878	35.3212	-0.2300	-88.6222	35.6519	-0.2300	-156.4583	53.8285	-0.2300	-157.6927	54.1593	-0.2300	-158.7625	54.4459	-0.2300
J	-86.0104	40.9049	-0.3450	-87.0802	41.1916	-0.3450	-88.3145	41.5223	-0.3450	-156.1506	59.6989	-0.3450	-157.3850	60.0297	-0.3450	-158.4548	60.3163	-0.3450
K	-85.7027	46.7753	-0.4600	-86.7725	47.0620	-0.4600	-88.0069	47.3927	-0.4600	-155.8430	65.5693	-0.4600	-157.0773	65.9001	-0.4600	-158.1471	66.1868	-0.4600
L	-85.3951	52.6457	-0.5750	-86.4648	52.9324	-0.5750	-87.6992	53.2631	-0.5750	-155.5353	71.4398	-0.5750	-156.7697	71.7705	-0.5750	-157.8395	72.0572	-0.5750
M	-85.0874	58.5161	-0.6900	-86.1572	58.8028	-0.6900	-87.3916	59.1335	-0.6900	-155.2277	77.3102	-0.6900	-156.4620	77.6409	-0.6900	-157.5318	77.9276	-0.6900

Span 2

Girder	Start of Girder									End of Girder								
	CL Pier			Girder End			CL Bearing			CL Bearing			Girder End			CL Pier		
	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)	East (X)	North (Y)	Deck Elev (ft)
A	-161.2237	7.4827	-0.6900	-162.2935	7.7694	-0.6900	-163.5279	8.1001	-0.6900	-231.3639	26.2768	-0.6900	-232.5983	26.6075	-0.6900	-233.6681	26.8942	-0.6900
B	-160.9160	13.3531	-0.5750	-161.9858	13.6398	-0.5750	-163.2202	13.9705	-0.5750	-231.0563	32.1472	-0.5750	-232.2907	32.4779	-0.5750	-233.3605	32.7646	-0.5750
C	-160.6084	19.2235	-0.4600	-161.6782	19.5102	-0.4600	-162.9126	19.8409	-0.4600	-230.7486	38.0176	-0.4600	-231.9830	38.3483	-0.4600	-233.0528	38.6350	-0.4600
D	-160.3007	25.0939	-0.3450	-161.3705	25.3806	-0.3450	-162.6049	25.7113	-0.3450	-230.4410	43.8880	-0.3450	-231.6754	44.2187	-0.3450	-232.7452	44.5054	-0.3450
E	-159.9931	30.9643	-0.2300	-161.0629	31.2510	-0.2300	-162.2972	31.5817	-0.2300	-230.1333	49.7584	-0.2300	-231.3677	50.0891	-0.2300	-232.4375	50.3758	-0.2300
F	-159.6854	36.8347	-0.1150	-160.7552	37.1214	-0.1150	-161.9896	37.4521	-0.1150	-229.8257	55.6288	-0.1150	-231.0601	55.9595	-0.1150	-232.1299	56.2462	-0.1150
G	-159.3778	42.7051	0.0000	-160.4476	42.9918	0.0000	-161.6819	43.3225	0.0000	-229.5180	61.4992	0.0000	-230.7524	61.8299	0.0000	-231.8222	62.1166	0.0000
H	-159.0701	48.5755	-0.1150	-160.1399	48.8622	-0.1150	-161.3743	49.1929	-0.1150	-229.2104	67.3696	-0.1150	-230.4447	67.7003	-0.1150	-231.5145	67.9870	-0.1150
I	-158.7625	54.4459	-0.2300	-159.8322	54.7326	-0.2300	-161.0666	55.0633	-0.2300	-228.9027	73.2400	-0.2300	-230.1371	73.5707	-0.2300	-231.2069	73.8574	-0.2300
J	-158.4548	60.3163	-0.3450	-159.5246	60.6030	-0.3450	-160.7590	60.9338	-0.3450	-228.5951	79.1104	-0.3450	-229.8294	79.4411	-0.3450	-230.8992	79.7278	-0.3450
K	-158.1471	66.1868	-0.4600	-159.2169	66.4734	-0.4600	-160.4513	66.8042	-0.4600	-228.2874	84.9808	-0.4600	-229.5218	85.3115	-0.4600	-230.5916	85.5982	-0.4600
L	-157.8395	72.0572	-0.5750	-158.9093	72.3438	-0.5750	-160.1437	72.6746	-0.5750	-227.9797	90.8512	-0.5750	-229.2141	91.1819	-0.5750	-230.2839	91.4686	-0.5750
M	-157.5318	77.9276	-0.6900	-158.6016	78.2142	-0.6900	-159.8360	78.5450	-0.6900	-227.6721	96.7216	-0.6900	-228.9065	97.0523	-0.6900	-229.9763	97.3390	-0.6900

Girder Offsets

Span 1

Girder	Start of Girder									End of Girder								
	CL Pier			Girder End			CL Bearing			CL Bearing			Girder End			CL Pier		
	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)
A	0+82.67	34.5000 L	-0.6900	0+83.77	34.5000 L	-0.6900	0+85.05	34.5000 L	-0.6900	1+55.28	34.5000 L	-0.6900	1+56.56	34.5000 L	-0.6900	1+57.67	34.5000 L	-0.6900
B	0+83.89	28.7500 L	-0.5750	0+85.00	28.7500 L	-0.5750	0+86.27	28.7500 L	-0.5750	1+56.50	28.7500 L	-0.5750	1+57.78	28.7500 L	-0.5750	1+58.89	28.7500 L	-0.5750
C	0+85.11	23.0000 L	-0.4600	0+86.22	23.0000 L	-0.4600	0+87.50	23.0000 L	-0.4600	1+57.73	23.0000 L	-0.4600	1+59.00	23.0000 L	-0.4600	1+60.11	23.0000 L	-0.4600
D	0+86.33	17.2500	-0.3450	0+87.44	17.2500	-0.3450	0+88.72	17.2500	-0.3450	1+58.95	17.2500	-0.3450	1+60.23	17.2500	-0.3450	1+61.33	17.2500	-0.3450

Girder	Start of Girder									End of Girder								
	CL Pier			Girder End			CL Bearing			CL Bearing			Girder End			CL Pier		
	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)
		L			L			L			L			L			L	
E	0+87.56	11.5000 L	-0.2300	0+88.66	11.5000 L	-0.2300	0+89.94	11.5000 L	-0.2300	1+60.17	11.5000 L	-0.2300	1+61.45	11.5000 L	-0.2300	1+62.56	11.5000 L	-0.2300
F	0+88.78	5.7500 L	-0.1150	0+89.89	5.7500 L	-0.1150	0+91.16	5.7500 L	-0.1150	1+61.39	5.7500 L	-0.1150	1+62.67	5.7500 L	-0.1150	1+63.78	5.7500 L	-0.1150
G	0+90.00	0.0000 L	0.0000	0+91.11	0.0000 L	0.0000	0+92.39	0.0000 L	0.0000	1+62.61	0.0000 L	0.0000	1+63.89	0.0000 L	0.0000	1+65.00	0.0000 L	0.0000
H	0+91.22	5.7500 R	-0.1150	0+92.33	5.7500 R	-0.1150	0+93.61	5.7500 R	-0.1150	1+63.84	5.7500 R	-0.1150	1+65.11	5.7500 R	-0.1150	1+66.22	5.7500 R	-0.1150
I	0+92.44	11.5000 R	-0.2300	0+93.55	11.5000 R	-0.2300	0+94.83	11.5000 R	-0.2300	1+65.06	11.5000 R	-0.2300	1+66.34	11.5000 R	-0.2300	1+67.44	11.5000 R	-0.2300
J	0+93.67	17.2500 R	-0.3450	0+94.77	17.2500 R	-0.3450	0+96.05	17.2500 R	-0.3450	1+66.28	17.2500 R	-0.3450	1+67.56	17.2500 R	-0.3450	1+68.67	17.2500 R	-0.3450
K	0+94.89	23.0000 R	-0.4600	0+96.00	23.0000 R	-0.4600	0+97.27	23.0000 R	-0.4600	1+67.50	23.0000 R	-0.4600	1+68.78	23.0000 R	-0.4600	1+69.89	23.0000 R	-0.4600
L	0+96.11	28.7500 R	-0.5750	0+97.22	28.7500 R	-0.5750	0+98.50	28.7500 R	-0.5750	1+68.73	28.7500 R	-0.5750	1+70.00	28.7500 R	-0.5750	1+71.11	28.7500 R	-0.5750
M	0+97.33	34.5000 R	-0.6900	0+98.44	34.5000 R	-0.6900	0+99.72	34.5000 R	-0.6900	1+69.95	34.5000 R	-0.6900	1+71.23	34.5000 R	-0.6900	1+72.33	34.5000 R	-0.6900

Span 2

Girder	Start of Girder									End of Girder								
	CL Pier			Girder End			CL Bearing			CL Bearing			Girder End			CL Pier		
	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)	Station	Offset (ft)	Deck Elev (ft)
A	1+57.67	34.5000 L	-0.6900	1+58.77	34.5000 L	-0.6900	1+60.05	34.5000 L	-0.6900	2+30.28	34.5000 L	-0.6900	2+31.56	34.5000 L	-0.6900	2+32.67	34.5000 L	-0.6900
B	1+58.89	28.7500 L	-0.5750	1+60.00	28.7500 L	-0.5750	1+61.27	28.7500 L	-0.5750	2+31.50	28.7500 L	-0.5750	2+32.78	28.7500 L	-0.5750	2+33.89	28.7500 L	-0.5750
C	1+60.11	23.0000 L	-0.4600	1+61.22	23.0000 L	-0.4600	1+62.50	23.0000 L	-0.4600	2+32.73	23.0000 L	-0.4600	2+34.00	23.0000 L	-0.4600	2+35.11	23.0000 L	-0.4600
D	1+61.33	17.2500 L	-0.3450	1+62.44	17.2500 L	-0.3450	1+63.72	17.2500 L	-0.3450	2+33.95	17.2500 L	-0.3450	2+35.23	17.2500 L	-0.3450	2+36.33	17.2500 L	-0.3450
E	1+62.56	11.5000 L	-0.2300	1+63.66	11.5000 L	-0.2300	1+64.94	11.5000 L	-0.2300	2+35.17	11.5000 L	-0.2300	2+36.45	11.5000 L	-0.2300	2+37.56	11.5000 L	-0.2300
F	1+63.78	5.7500 L	-0.1150	1+64.89	5.7500 L	-0.1150	1+66.16	5.7500 L	-0.1150	2+36.39	5.7500 L	-0.1150	2+37.67	5.7500 L	-0.1150	2+38.78	5.7500 L	-0.1150
G	1+65.00	0.0000 L	0.0000	1+66.11	0.0000 L	0.0000	1+67.39	0.0000 L	0.0000	2+37.61	0.0000 L	0.0000	2+38.89	0.0000 L	0.0000	2+40.00	0.0000 L	0.0000
H	1+66.22	5.7500 R	-0.1150	1+67.33	5.7500 R	-0.1150	1+68.61	5.7500 R	-0.1150	2+38.84	5.7500 R	-0.1150	2+40.11	5.7500 R	-0.1150	2+41.22	5.7500 R	-0.1150
I	1+67.44	11.5000 R	-0.2300	1+68.55	11.5000 R	-0.2300	1+69.83	11.5000 R	-0.2300	2+40.06	11.5000 R	-0.2300	2+41.34	11.5000 R	-0.2300	2+42.44	11.5000 R	-0.2300
J	1+68.67	17.2500 R	-0.3450	1+69.77	17.2500 R	-0.3450	1+71.05	17.2500 R	-0.3450	2+41.28	17.2500 R	-0.3450	2+42.56	17.2500 R	-0.3450	2+43.67	17.2500 R	-0.3450
K	1+69.89	23.0000 R	-0.4600	1+71.00	23.0000 R	-0.4600	1+72.27	23.0000 R	-0.4600	2+42.50	23.0000 R	-0.4600	2+43.78	23.0000 R	-0.4600	2+44.89	23.0000 R	-0.4600
L	1+71.11	28.7500 R	-0.5750	1+72.22	28.7500 R	-0.5750	1+73.50	28.7500 R	-0.5750	2+43.73	28.7500 R	-0.5750	2+45.00	28.7500 R	-0.5750	2+46.11	28.7500 R	-0.5750
M	1+72.33	34.5000 R	-0.6900	1+73.44	34.5000 R	-0.6900	1+74.72	34.5000 R	-0.6900	2+44.95	34.5000 R	-0.6900	2+46.23	34.5000 R	-0.6900	2+47.33	34.5000 R	-0.6900

Girder Spacing

Span 1

Girder	Start of Girder					End of Girder				
	Spacing at CL Pier		Spacing at CL Brg		Angle with CL Pier	Spacing at CL Brg		Spacing at CL Pier		Angle with CL Pier
	⊥ to Alignment (ft)	Along CL Pier (ft)	⊥ to Alignment (ft)	Along CL Brg (ft)		⊥ to Alignment (ft)	Along CL Brg (ft)	⊥ to Alignment (ft)	Along CL Pier (ft)	
A					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
B					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
C					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
D					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
E					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
F					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
G					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	

Girder	Start of Girder					End of Girder				
	Spacing at CL Pier		Spacing at CL Brg		Angle with CL Pier	Spacing at CL Brg		Spacing at CL Pier		Angle with CL Pier
	⊥ to Alignment (ft)	Along CL Pier (ft)	⊥ to Alignment (ft)	Along CL Brg (ft)		⊥ to Alignment (ft)	Along CL Brg (ft)	⊥ to Alignment (ft)	Along CL Pier (ft)	
H					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
I					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
J					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
K					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
L					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
M					102° 00' 00.00"					102° 00' 00.00"

Span 2

Girder	Start of Girder					End of Girder				
	Spacing at CL Pier		Spacing at CL Brg		Angle with CL Pier	Spacing at CL Brg		Spacing at CL Pier		Angle with CL Pier
	⊥ to Alignment (ft)	Along CL Pier (ft)	⊥ to Alignment (ft)	Along CL Brg (ft)		⊥ to Alignment (ft)	Along CL Brg (ft)	⊥ to Alignment (ft)	Along CL Pier (ft)	
A					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
B					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
C					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
D					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
E					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
F					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
G					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
H					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
I					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
J					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
K					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
L					102° 00' 00.00"					102° 00' 00.00"
	5.750	5.878	5.750	5.878		5.750	5.878	5.750	5.878	
M					102° 00' 00.00"					102° 00' 00.00"

⊥ to Alignment: spacing is measured along a line that is normal to the alignment and passes through the point where the CL Pier or CL Brg intersect the alignment.

Girder Ends

Span 1

Girder	Start of Girder					End of Girder				
	CL Pier to CL Brg		CL Pier to Girder End		CL Brg to Girder End Along Girder (ft)	CL Pier to CL Brg		CL Pier to Girder End		CL Brg to Girder End Along Girder (ft)
	⊥ to Pier (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)		⊥ to Pier (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)	
A	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
B	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
C	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
D	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
E	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
F	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
G	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
H	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
I	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
J	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
K	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278

Girder	Start of Girder					End of Girder				
	CL Pier to CL Brg		CL Pier to Girder End		CL Brg to Girder End	CL Pier to CL Brg		CL Pier to Girder End		CL Brg to Girder End
	⊥ to Pier (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)	Along Girder (ft)
L	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
M	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278

Span 2

Girder	Start of Girder					End of Girder				
	CL Pier to CL Brg		CL Pier to Girder End		CL Brg to Girder End	CL Pier to CL Brg		CL Pier to Girder End		CL Brg to Girder End
	⊥ to Pier (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)	⊥ to Pier (ft)	Along Girder (ft)	Along Girder (ft)
A	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
B	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
C	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
D	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
E	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
F	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
G	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
H	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
I	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
J	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
K	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
L	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278
M	2.333	2.385	1.083	1.108	1.278	2.333	2.385	1.083	1.108	1.278

Girder Lengths

C-C Pier = Centerline pier to centerline pier length measured along the girder
C-C Bearing = Centerline bearing to centerline bearing length measured along the girder
Girder Length, Horizontal = End to end length of the girder projected into a horizontal plane
Girder Length, Along Grade = End to end length of girder measured along grade of the girder (slope adjusted) = $L_g\sqrt{1+slope^2}$

Span 1

Girder	C-C Pier (ft)	C-C Bearing L_s (ft)	Girder Length		Girder Slope (ft/ft)	Direction
			Horizontal L_g (ft)	Along Grade (ft)		
A	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
B	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
C	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
D	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
E	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
F	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
G	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
H	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
I	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
J	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
K	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
L	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
M	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W

Span 2

Girder	C-C Pier (ft)	C-C Bearing L_s (ft)	Girder Length		Girder Slope (ft/ft)	Direction
			Horizontal L_g (ft)	Along Grade (ft)		
A	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
B	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
C	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
D	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
E	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
F	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
G	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
H	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
I	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
J	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
K	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W
L	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W

Girder	C-C Pier (ft)	C-C Bearing L_s (ft)	Girder Length		Girder Slope (ft/ft)	Direction
			Horizontal L_g (ft)	Along Grade (ft)		
M	75.000	70.229	72.785	72.785	0.0000	N 75° 00' 00.00" W

Spec Check Report

For

Span 1 Girder A

February 23, 2013 12:35:33 pm

PGSuper™

Version 2.5.1 - Built on Aug 17 2011

Washington State Department of Transportation

Texas Department of Transportation

Project Properties

Bridge Name	MQP New
Bridge ID	
Company	MQP
Engineer	Joshua Nitso
Job Number	
Comments	
File	C:\Users\Nitso\Desktop\MQP Final.pgs

Library Usage

Master Library Publisher: WSDOT
Library and Template Package URL: ftp://ftp.wsdot.wa.gov/public/bridge/Software/PGSuper/Version_2.5.0/WSDOT.pgz
Master Library Date Stamp: December 12, 2011 11:46:13 am

Library	Entry	Source
Connections	End Type A (W35DG)	Master Library
Girders	W35DG	Master Library
Traffic Barriers	32" F Shape	Master Library
Project Criteria	WSDOT LRFD - US Units	Master Library
Vehicular Live Load	OL1	Master Library
Vehicular Live Load	OL2	Master Library
Load Rating Criteria	WSDOT	Master Library

Notes

Symbol	Definition
L_g	Length of Girder
L_s	Length of Span
FoS	Face of Support
Debond	Point where bond begins for a debonded strand
PSXFR	Point of prestress transfer
CS	Critical Section for Shear
H	H from end of girder or face of support
1.5H	1.5H from end of girder or face of support
HP	Harp Point
Pick Point	Support point where girder is lifted from form
Bunk Point	Point where girder is supported during transportation

Status Items

Level	Description
Info	All Live Load Distribution Factors are computed using the Lever Rule.
Warning	Left lift point is less than the minimum value of 3.000 ft

Specification Check Summary

The Specification Check Was Not Successful

Ultimate vertical shear capacity check failed for Strength I Limit State for the Bridge Site Stage 3.

Splitting zone checks failed.

Confinement zone checks failed.

Specification Checks

Specification = WSDOT LRFD - US Units

Strand Stresses [5.9.3]

Loss Stage	Allowable Stress (KSI)	Straight		Harped	
		Strand Stress (KSI)	Status (C/D)	Strand Stress (KSI)	Status (C/D)
At Jacking	202.500	202.500	Pass (1.00)	202.500	Pass (1.00)
After All Losses	194.400	163.436	Pass (1.19)	163.436	Pass (1.19)

Required Concrete Strengths

Required f'_{ci} = 6.206 KSI ⇒ 6.300 KSI
Actual f'_{ci} = 6.800 KSI

Required f'_{c} = 7.875 KSI ⇒ 7.900 KSI
Actual f'_{c} = 8.000 KSI

Continuity [5.14.1.4.5]

	f_b (KSI)	Boundary Condition	Is Compressive?
Abutment 1	0.000	Hinged	No
Pier 2	0.000	Hinged	No
Abutment 3	0.000	Hinged	No

f_b is the calcuated stress at the bottom of the continuity diaphragm for the combination of superimposed permanent loads and 50% live load
Continuous connections are not fully effective.
Continuity is accounted for only in Strength Limit States.

Stress Check for Service I for Casting Yard Stage (At Release) [5.9.4.1.2]

For temporary stresses before losses in pretensioned components
Allowable tensile stress = $0.0948\sqrt{f'_{ci}}$ but not more than 0.200 KSI = 0.200 KSI
Allowable tensile stress = $0.2400\sqrt{f'_{ci}}$ = 0.626 KSI if at least 0.139 in^2 of mild reinforcement is provided
Allowable compressive stress = $-0.65f'_{ci}$ = -4.420 KSI

f'_{ci} required to satisfy this stress check = 5.988 KSI

Location from End of Girder (ft)	Prestress		Service I		Demand		Tension Status w/o rebar (C/D)	Tension Status w/ rebar (C/D)	Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)			
($0.0L_g$) 0.000	0.000	0.000	0.000	0.000	0.000	0.000	Pass (∞)	Pass (∞)	Pass (∞)
1.278	0.042	-1.681	-0.054	0.101	-0.012	-1.580	Pass (-)	Pass (-)	Pass (2.80)
(H) 2.917	0.141	-3.920	-0.120	0.226	0.021	-3.694	Pass (9.41)	Pass (10+)	Pass (1.20)
(PSXFR) 3.000	0.148	-4.036	-0.123	0.232	0.024	-3.804	Pass (8.21)	Pass (10+)	Pass (1.16)
4.695	0.195	-4.123	-0.188	0.354	0.007	-3.769	Pass (10+)	Pass (10+)	Pass (1.17)
($0.1L_g$) 7.278	0.268	-4.255	-0.281	0.528	-0.012	-3.727	Pass (-)	Pass (-)	Pass (1.19)
8.301	0.297	-4.307	-0.315	0.593	-0.018	-3.714	Pass (-)	Pass (-)	Pass (1.19)
($0.2L_g$) 14.557	0.471	-4.620	-0.499	0.939	-0.027	-3.681	Pass (-)	Pass (-)	Pass (1.20)
15.324	0.492	-4.658	-0.518	0.976	-0.026	-3.682	Pass (-)	Pass (-)	Pass (1.20)
($0.3L_g$) 21.835	0.671	-4.971	-0.655	1.233	0.017	-3.738	Pass (10+)	Pass (10+)	Pass (1.18)
22.347	0.685	-4.995	-0.663	1.249	0.022	-3.746	Pass (9.17)	Pass (10+)	Pass (1.18)
(HP, $0.4L_g$) 29.114	0.866	-5.301	-0.748	1.409	0.118	-3.892	Pass (1.70)	Pass (5.32)	Pass (1.14)
29.370	0.866	-5.302	-0.750	1.413	0.116	-3.889	Pass (1.73)	Pass (5.42)	Pass (1.14)
($0.5L_g$) 36.392	0.867	-5.308	-0.779	1.468	0.088	-3.841	Pass (2.28)	Pass (7.14)	Pass (1.15)
43.415	0.866	-5.302	-0.750	1.413	0.116	-3.889	Pass (1.73)	Pass (5.42)	Pass (1.14)
(HP, $0.6L_g$) 43.671	0.866	-5.301	-0.748	1.409	0.118	-3.892	Pass (1.70)	Pass (5.32)	Pass (1.14)
50.438	0.685	-4.995	-0.663	1.249	0.022	-3.746	Pass (9.17)	Pass (10+)	Pass (1.18)
($0.7L_g$) 50.949	0.671	-4.971	-0.655	1.233	0.017	-3.738	Pass (10+)	Pass (10+)	Pass (1.18)
57.461	0.492	-4.658	-0.518	0.976	-0.026	-3.682	Pass (-)	Pass (-)	Pass (1.20)
($0.8L_g$) 58.228	0.471	-4.620	-0.499	0.939	-0.027	-3.681	Pass (-)	Pass (-)	Pass (1.20)
64.484	0.297	-4.307	-0.315	0.593	-0.018	-3.714	Pass (-)	Pass (-)	Pass (1.19)
($0.9L_g$) 65.506	0.268	-4.255	-0.281	0.528	-0.012	-3.727	Pass (-)	Pass (-)	Pass (1.19)
68.090	0.195	-4.123	-0.188	0.354	0.007	-3.769	Pass (10+)	Pass (10+)	Pass (1.17)

Location from End of Girder (ft)	Prestress		Service I		Demand		Tension Status w/o rebar (C/D)	Tension Status w/ rebar (C/D)	Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)			
(PSXFR) 69.785	0.148	-4.036	-0.123	0.232	0.024	-3.804	Pass (8.21)	Pass (10+)	Pass (1.16)
(H) 69.868	0.141	-3.920	-0.120	0.226	0.021	-3.694	Pass (9.41)	Pass (10+)	Pass (1.20)
71.507	0.042	-1.681	-0.054	0.101	-0.012	-1.580	Pass (-)	Pass (-)	Pass (2.80)
($1.0L_g$) 72.785	0.000	0.000	0.000	0.000	0.000	0.000	Pass (∞)	Pass (∞)	Pass (∞)

Stress Check for Service I for Deck and Diaphragm Placement (Bridge Site 1) [5.9.4.2.2]

For stresses at service limit state after losses for components with bonded prestressing tendons other than piles
Allowable tensile stress = $0.1900\sqrt{f'_c}$ = 0.537 KSI
Allowable compressive stress = $-0.45f'_c$ = -3.600 KSI
f'_c required to satisfy this stress check = 7.875 KSI

Location from Left Support (ft)	Prestress		Service I		Demand		Tension Status (C/D)	Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)		
($0.0L_s$) 0.000	0.038	-1.531	0.000	0.000	0.038	-1.531	Pass (10+)	Pass (2.35)
1.639	0.129	-3.570	-0.067	0.126	0.062	-3.444	Pass (8.69)	Pass (1.05)
(PSXFR) 1.722	0.134	-3.676	-0.070	0.132	0.064	-3.544	Pass (8.35)	Pass (1.02)
(H) 3.417	0.178	-3.754	-0.136	0.255	0.042	-3.499	Pass (10+)	Pass (1.03)
6.001	0.244	-3.874	-0.229	0.431	0.015	-3.443	Pass (10+)	Pass (1.05)
($0.1L_s$) 7.023	0.270	-3.921	-0.264	0.497	0.006	-3.424	Pass (10+)	Pass (1.05)
13.279	0.428	-4.201	-0.450	0.847	-0.021	-3.354	Pass (-)	Pass (1.07)
($0.2L_s$) 14.046	0.448	-4.234	-0.469	0.884	-0.022	-3.350	Pass (-)	Pass (1.07)
20.558	0.609	-4.508	-0.608	1.145	0.001	-3.363	Pass (10+)	Pass (1.07)
($0.3L_s$) 21.069	0.621	-4.529	-0.617	1.162	0.004	-3.367	Pass (10+)	Pass (1.07)
(HP) 27.836	0.782	-4.787	-0.704	1.326	0.078	-3.461	Pass (6.92)	Pass (1.04)
($0.4L_s$) 28.092	0.782	-4.788	-0.706	1.330	0.076	-3.458	Pass (7.11)	Pass (1.04)
($0.5L_s$) 35.115	0.784	-4.799	-0.738	1.390	0.046	-3.409	Pass (10+)	Pass (1.06)
($0.6L_s$) 42.137	0.782	-4.788	-0.706	1.330	0.076	-3.458	Pass (7.11)	Pass (1.04)

Location from Left Support (ft)	Prestress		Service I		Demand		Tension Status (C/D)	Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)		
(HP) 42.393	0.782	-4.787	-0.704	1.326	0.078	-3.461	Pass (6.92)	Pass (1.04)
($0.7L_s$) 49.160	0.621	-4.529	-0.617	1.162	0.004	-3.367	Pass (10+)	Pass (1.07)
49.672	0.609	-4.508	-0.608	1.145	0.001	-3.363	Pass (10+)	Pass (1.07)
($0.8L_s$) 56.183	0.448	-4.234	-0.469	0.884	-0.022	-3.350	Pass (-)	Pass (1.07)
56.950	0.428	-4.201	-0.450	0.847	-0.021	-3.354	Pass (-)	Pass (1.07)
($0.9L_s$) 63.206	0.270	-3.921	-0.264	0.497	0.006	-3.424	Pass (10+)	Pass (1.05)
64.229	0.244	-3.874	-0.229	0.431	0.015	-3.443	Pass (10+)	Pass (1.05)
(H) 66.812	0.178	-3.754	-0.136	0.255	0.042	-3.499	Pass (10+)	Pass (1.03)
(PSXFR) 68.507	0.134	-3.676	-0.070	0.132	0.064	-3.544	Pass (8.35)	Pass (1.02)
68.590	0.129	-3.570	-0.067	0.126	0.062	-3.444	Pass (8.69)	Pass (1.05)
($1.0L_s$) 70.229	0.038	-1.531	0.000	0.000	0.038	-1.531	Pass (10+)	Pass (2.35)

Stress Check for Service I for Superimposed Dead Loads (Bridge Site 2) [5.9.4.2.1]

For stresses at service limit state after losses in other than segmentally constructed bridges due to permanent loads
Allowable compressive stress = $-0.45f'_c$ = -3.600 KSI
f'_c required to satisfy this stress check = 7.770 KSI

Location from Left Support (ft)	Prestress		Service I		Demand		Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	
($0.0L_s$) 0.000	0.038	-1.531	0.000	0.000	0.038	-1.531	Pass (2.35)
1.639	0.129	-3.572	-0.092	0.173	0.037	-3.399	Pass (1.06)
(PSXFR) 1.722	0.134	-3.679	-0.097	0.182	0.038	-3.497	Pass (1.03)
(H) 3.417	0.178	-3.761	-0.187	0.352	-0.009	-3.409	Pass (1.06)
6.001	0.245	-3.885	-0.316	0.595	-0.071	-3.291	Pass (1.09)
($0.1L_s$) 7.023	0.271	-3.934	-0.364	0.685	-0.093	-3.249	Pass (1.11)
13.279	0.431	-4.229	-0.620	1.168	-0.189	-3.061	Pass (1.18)
($0.2L_s$) 14.046	0.451	-4.264	-0.647	1.219	-0.196	-3.045	Pass (1.18)

Location from Left Support (ft)	Prestress		Service I		Demand		Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	
20.558	0.615	-4.554	-0.838	1.579	-0.224	-2.975	Pass (1.21)
($0.3L_s$) 21.069	0.627	-4.576	-0.850	1.602	-0.223	-2.974	Pass (1.21)
(HP) 27.836	0.792	-4.850	-0.970	1.828	-0.178	-3.023	Pass (1.19)
($0.4L_s$) 28.092	0.792	-4.851	-0.973	1.833	-0.181	-3.018	Pass (1.19)
($0.5L_s$) 35.115	0.794	-4.865	-1.016	1.914	-0.222	-2.951	Pass (1.22)
($0.6L_s$) 42.137	0.792	-4.851	-0.973	1.833	-0.181	-3.018	Pass (1.19)
(HP) 42.393	0.792	-4.850	-0.970	1.828	-0.178	-3.023	Pass (1.19)
($0.7L_s$) 49.160	0.627	-4.576	-0.850	1.602	-0.223	-2.974	Pass (1.21)
49.672	0.615	-4.554	-0.838	1.579	-0.224	-2.975	Pass (1.21)
($0.8L_s$) 56.183	0.451	-4.264	-0.647	1.219	-0.196	-3.045	Pass (1.18)
56.950	0.431	-4.229	-0.620	1.168	-0.189	-3.061	Pass (1.18)
($0.9L_s$) 63.206	0.271	-3.934	-0.364	0.685	-0.093	-3.249	Pass (1.11)
64.229	0.245	-3.885	-0.316	0.595	-0.071	-3.291	Pass (1.09)
(H) 66.812	0.178	-3.761	-0.187	0.352	-0.009	-3.409	Pass (1.06)
(PSXFR) 68.507	0.134	-3.679	-0.097	0.182	0.038	-3.497	Pass (1.03)
68.590	0.129	-3.572	-0.092	0.173	0.037	-3.399	Pass (1.06)
($1.0L_s$) 70.229	0.038	-1.531	0.000	0.000	0.038	-1.531	Pass (2.35)

Stress Check for Compressive Stresses for Service I for Final with Live Load (Bridge Site 3) [5.5.3.1]

For stresses at service limit state after losses in other than segmentally constructed bridges due to permanent and transient loads
Allowable compressive stress = $-0.6f'_c$ = -4.800 KSI
f'_c required to satisfy this stress check = 5.660 KSI

Location from Left Support (ft)	Prestress		Service I		Demand		Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	
($0.0L_s$) 0.000	0.037	-1.488	0.000	0.000	0.037	-1.488	Pass (3.22)

Location from Left Support (ft)	Prestress		Service I		Demand		Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	
1.639	0.125	-3.475	-0.221	0.173	-0.096	-3.301	Pass (1.45)
(PSXFR) 1.722	0.131	-3.578	-0.232	0.182	-0.101	-3.396	Pass (1.41)
(H) 3.417	0.173	-3.660	-0.448	0.352	-0.275	-3.308	Pass (1.45)
6.001	0.238	-3.786	-0.755	0.595	-0.516	-3.191	Pass (1.50)
(0.1L_s) 7.023	0.264	-3.835	-0.868	0.685	-0.604	-3.150	Pass (1.52)
13.279	0.421	-4.132	-1.466	1.168	-1.045	-2.964	Pass (1.62)
(0.2L_s) 14.046	0.441	-4.167	-1.528	1.219	-1.088	-2.948	Pass (1.63)
20.558	0.602	-4.460	-1.955	1.579	-1.353	-2.881	Pass (1.67)
(0.3L_s) 21.069	0.615	-4.482	-1.981	1.602	-1.367	-2.881	Pass (1.67)
(HP) 27.836	0.777	-4.760	-2.243	1.828	-1.466	-2.932	Pass (1.64)
(0.4L_s) 28.092	0.777	-4.761	-2.249	1.833	-1.472	-2.928	Pass (1.64)
(0.5L_s) 35.115	0.780	-4.776	-2.321	1.914	-1.541	-2.863	Pass (1.68)
(0.6L_s) 42.137	0.777	-4.761	-2.249	1.833	-1.472	-2.928	Pass (1.64)
(HP) 42.393	0.777	-4.760	-2.243	1.828	-1.466	-2.932	Pass (1.64)
(0.7L_s) 49.160	0.615	-4.482	-1.981	1.602	-1.367	-2.881	Pass (1.67)
49.672	0.602	-4.460	-1.955	1.579	-1.353	-2.881	Pass (1.67)
(0.8L_s) 56.183	0.441	-4.167	-1.528	1.219	-1.088	-2.948	Pass (1.63)
56.950	0.421	-4.132	-1.466	1.168	-1.045	-2.964	Pass (1.62)
(0.9L_s) 63.206	0.264	-3.835	-0.868	0.685	-0.604	-3.150	Pass (1.52)
64.229	0.238	-3.786	-0.755	0.595	-0.516	-3.191	Pass (1.50)
(H) 66.812	0.173	-3.660	-0.448	0.352	-0.275	-3.308	Pass (1.45)
(PSXFR) 68.507	0.131	-3.578	-0.232	0.182	-0.101	-3.396	Pass (1.41)
68.590	0.125	-3.475	-0.221	0.173	-0.096	-3.301	Pass (1.45)
(1.0L_s) 70.229	0.037	-1.488	0.000	0.000	0.037	-1.488	Pass (3.22)

Stress Check for Tensile Stresses for Service III for Final with Live Load (Bridge Site 3) [5.9.4.2.2]

For stresses at service limit state after losses which involve traffic loading in members with bonded prestressing tendons other than piles
Allowable tensile stress in the precompressed tensile zone = $0.0000\sqrt{f'_c}$ = 0.000 KSI

Location from Left Support (ft)	Prestress f_b (KSI)	Service III f_b (KSI)	Demand f_b (KSI)	Tension Status
($0.0L_s$) 0.000	-1.488	0.000	-1.488	Pass
1.639	-3.475	0.368	-3.107	Pass
(PSXFR) 1.722	-3.578	0.386	-3.192	Pass
(H) 3.417	-3.660	0.746	-2.915	Pass
6.001	-3.786	1.256	-2.530	Pass
($0.1L_s$) 7.023	-3.835	1.445	-2.390	Pass
13.279	-4.132	2.443	-1.689	Pass
($0.2L_s$) 14.046	-4.167	2.547	-1.621	Pass
20.558	-4.460	3.262	-1.198	Pass
($0.3L_s$) 21.069	-4.482	3.306	-1.177	Pass
(HP) 27.836	-4.760	3.745	-1.015	Pass
($0.4L_s$) 28.092	-4.761	3.756	-1.005	Pass
($0.5L_s$) 35.115	-4.776	3.881	-0.896	Pass
($0.6L_s$) 42.137	-4.761	3.756	-1.005	Pass
(HP) 42.393	-4.760	3.745	-1.015	Pass
($0.7L_s$) 49.160	-4.482	3.306	-1.177	Pass
49.672	-4.460	3.262	-1.198	Pass
($0.8L_s$) 56.183	-4.167	2.547	-1.621	Pass
56.950	-4.132	2.443	-1.689	Pass
($0.9L_s$) 63.206	-3.835	1.445	-2.390	Pass
64.229	-3.786	1.256	-2.530	Pass
(H) 66.812	-3.660	0.746	-2.915	Pass
(PSXFR) 68.507	-3.578	0.386	-3.192	Pass
68.590	-3.475	0.368	-3.107	Pass
($1.0L_s$) 70.229	-1.488	0.000	-1.488	Pass

Stress Check for Compressive Stresses for Fatigue I for Final with Live Load (Bridge Site 3) [5.5.3.1]

For stresses at service limit state after losses in other than segmentally constructed bridges due to the Fatigue I load combination and one-half the sum of effective prestress and permanent loads
Allowable compressive stress = $-0.4f'_c$ = -3.200 KSI
f'_c required to satisfy this stress check = 4.245 KSI

Location from Left Support (ft)	Prestress		Fatigue I		Demand		Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	
($0.0L_s$) 0.000	0.037	-1.488	0.000	0.000	0.019	-0.744	Pass (4.30)
1.639	0.125	-3.475	-0.158	0.087	-0.095	-1.651	Pass (1.94)
(PSXFR) 1.722	0.131	-3.578	-0.166	0.091	-0.100	-1.698	Pass (1.88)
(H) 3.417	0.173	-3.660	-0.319	0.176	-0.232	-1.654	Pass (1.93)
6.001	0.238	-3.786	-0.532	0.297	-0.413	-1.595	Pass (2.01)
($0.1L_s$) 7.023	0.264	-3.835	-0.610	0.343	-0.478	-1.575	Pass (2.03)
13.279	0.421	-4.132	-1.007	0.584	-0.796	-1.482	Pass (2.16)
($0.2L_s$) 14.046	0.441	-4.167	-1.046	0.610	-0.826	-1.474	Pass (2.17)
20.558	0.602	-4.460	-1.327	0.790	-1.026	-1.441	Pass (2.22)
($0.3L_s$) 21.069	0.615	-4.482	-1.345	0.801	-1.037	-1.440	Pass (2.22)
(HP) 27.836	0.777	-4.760	-1.492	0.914	-1.103	-1.466	Pass (2.18)
($0.4L_s$) 28.092	0.777	-4.761	-1.494	0.917	-1.106	-1.464	Pass (2.19)
($0.5L_s$) 35.115	0.780	-4.776	-1.471	0.957	-1.081	-1.431	Pass (2.24)
($0.6L_s$) 42.137	0.777	-4.761	-1.494	0.917	-1.106	-1.464	Pass (2.19)
(HP) 42.393	0.777	-4.760	-1.492	0.914	-1.103	-1.466	Pass (2.18)
($0.7L_s$) 49.160	0.615	-4.482	-1.345	0.801	-1.037	-1.440	Pass (2.22)
49.672	0.602	-4.460	-1.327	0.790	-1.026	-1.441	Pass (2.22)
($0.8L_s$) 56.183	0.441	-4.167	-1.046	0.610	-0.826	-1.474	Pass (2.17)
56.950	0.421	-4.132	-1.007	0.584	-0.796	-1.482	Pass (2.16)
($0.9L_s$) 63.206	0.264	-3.835	-0.610	0.343	-0.478	-1.575	Pass (2.03)
64.229	0.238	-3.786	-0.532	0.297	-0.413	-1.595	Pass (2.01)
(H) 66.812	0.173	-3.660	-0.319	0.176	-0.232	-1.654	Pass (1.93)
(PSXFR) 68.507	0.131	-3.578	-0.166	0.091	-0.100	-1.698	Pass (1.88)
68.590	0.125	-3.475	-0.158	0.087	-0.095	-1.651	Pass (1.94)

Location from Left Support (ft)	Prestress		Fatigue I		Demand		Compression Status (C/D)
	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	f_t (KSI)	f_b (KSI)	
($1.0L_s$) 70.229	0.037	-1.488	0.000	0.000	0.019	-0.744	Pass (4.30)

Positive Moment Capacity for Strength I Limit State for Final with Live Load Stage (Bridge Site 3) [5.7]

Location from Left Support (ft)	M_u (kip-ft)	ϕM_n (kip-ft)	ϕM_n Min (kip-ft)	Status	
				ϕM_n Min ≤ ϕM_n ($\phi M_n/\phi M_n$ Min)	$M_u \le \phi M_n$ ($\phi M_n/M_u$)
($0.0L_s$) 0.000	0.00	824.64	0.00	Pass (∞)	Pass (∞)
(FoS) 0.500	83.75	1144.52	111.38	Pass (10+)	Pass (10+)
1.639	269.65	1868.50	358.64	Pass (5.21)	Pass (6.93)
(PSXFR) 1.722	282.99	1921.91	376.38	Pass (5.11)	Pass (6.79)
2.187	356.84	1987.30	474.60	Pass (4.19)	Pass (5.57)
(CS) 2.611	423.02	2046.92	562.62	Pass (3.64)	Pass (4.84)
(H) 3.417	546.48	2160.86	726.82	Pass (2.97)	Pass (3.95)
4.375	688.91	2297.11	916.25	Pass (2.51)	Pass (3.33)
(1.5H) 4.875	761.33	2368.55	1012.57	Pass (2.34)	Pass (3.11)
6.001	919.62	2530.33	1223.10	Pass (2.07)	Pass (2.75)
($0.1L_s$) 7.023	1057.72	2678.50	1406.76	Pass (1.90)	Pass (2.53)
13.279	1784.88	3400.04	2373.89	Pass (1.43)	Pass (1.90)
($0.2L_s$) 14.046	1860.06	3421.97	2473.89	Pass (1.38)	Pass (1.84)
20.558	2375.85	3610.38	2746.67	Pass (1.31)	Pass (1.52)
($0.3L_s$) 21.069	2407.05	3625.34	2757.70	Pass (1.31)	Pass (1.51)
(HP) 27.836	2722.10	3825.62	2896.04	Pass (1.32)	Pass (1.41)
27.841	2722.25	3825.62	2896.05	Pass (1.32)	Pass (1.41)
($0.4L_s$) 28.092	2729.94	3825.63	2896.59	Pass (1.32)	Pass (1.40)
($0.5L_s$) 35.115	2813.11	3825.75	2904.27	Pass (1.32)	Pass (1.36)
	2729.94	3825.63	2896.59	Pass	Pass

Location from Left Support (ft)	M_u (kip-ft)	ϕM_n (kip-ft)	ϕM_n Min (kip-ft)	Status	
				ϕM_n Min ≤ ϕM_n (ϕM_n/ϕM_n Min)	M_u ≤ ϕM_n (ϕM_n/M_u)
($0.6L_s$) 42.137				(1.32)	(1.40)
42.388	2722.25	3825.62	2896.05	Pass (1.32)	Pass (1.41)
(HP) 42.393	2722.10	3825.62	2896.04	Pass (1.32)	Pass (1.41)
($0.7L_s$) 49.160	2407.05	3625.34	2757.70	Pass (1.31)	Pass (1.51)
49.672	2375.85	3610.38	2746.67	Pass (1.31)	Pass (1.52)
($0.8L_s$) 56.183	1860.06	3421.97	2473.89	Pass (1.38)	Pass (1.84)
56.950	1784.88	3400.04	2373.89	Pass (1.43)	Pass (1.90)
($0.9L_s$) 63.206	1057.72	2678.50	1406.76	Pass (1.90)	Pass (2.53)
64.229	919.62	2530.33	1223.10	Pass (2.07)	Pass (2.75)
(1.5H) 65.354	761.33	2368.55	1012.57	Pass (2.34)	Pass (3.11)
65.854	688.91	2297.11	916.25	Pass (2.51)	Pass (3.33)
(H) 66.812	546.48	2160.86	726.82	Pass (2.97)	Pass (3.95)
(CS) 67.618	423.02	2046.92	562.62	Pass (3.64)	Pass (4.84)
68.042	356.84	1987.30	474.60	Pass (4.19)	Pass (5.57)
(PSXFR) 68.507	282.99	1921.91	376.38	Pass (5.11)	Pass (6.79)
68.590	269.65	1868.50	358.64	Pass (5.21)	Pass (6.93)
(FoS) 69.729	83.75	1144.52	111.38	Pass (10+)	Pass (10+)
($1.0L_s$) 70.229	0.00	824.64	0.00	Pass (∞)	Pass (∞)

Ultimate Shears for Strength I Limit State for Bridge Site Stage 3 [5.8]

Location from Left Support (ft)	Stirrups Required	Stirrups Provided	$\|V_u\|$ (kip)	ϕV_n (kip)	Status (ϕV_n/V_u)
($0.0L_s$) 0.000	Yes	Yes	$	$	Pass
(FoS) 0.500	Yes	Yes	$	$	Pass
1.639	Yes	Yes	$	$	Pass

Location from Left Support (ft)	Stirrups Required	Stirrups Provided	$\|V_u\|$ (kip)	ϕV_n (kip)	Status ($\phi V_n/V_u$)
(PSXFR) 1.722	Yes	Yes	$	$	Pass
(CS) 2.611	Yes	Yes	159.04	266.48	Pass (1.68)
(H) 3.417	Yes	Yes	156.05	264.23	Pass (1.69)
(1.5H) 4.875	Yes	Yes	150.64	141.83	Fail (0.94)
6.001	Yes	Yes	146.49	140.03	Fail (0.96)
($0.1L_s$) 7.023	Yes	Yes	142.72	138.64	Fail (0.97)
13.279	Yes	Yes	119.89	142.26	Pass (1.19)
($0.2L_s$) 14.046	Yes	Yes	117.12	145.21	Pass (1.24)
20.558	Yes	Yes	93.81	144.03	Pass (1.54)
($0.3L_s$) 21.069	Yes	Yes	91.99	141.98	Pass (1.54)
(HP) 27.836	Yes	Yes	68.23	132.90	Pass (1.95)
($0.4L_s$) 28.092	Yes	Yes	67.34	110.45	Pass (1.64)
($0.5L_s$) 35.115	Yes	Yes	43.15	108.56	Pass (2.52)
($0.6L_s$) 42.137	Yes	Yes	67.34	110.45	Pass (1.64)
(HP) 42.393	Yes	Yes	68.23	132.90	Pass (1.95)
($0.7L_s$) 49.160	Yes	Yes	91.99	141.98	Pass (1.54)
49.672	Yes	Yes	93.81	144.03	Pass (1.54)
($0.8L_s$) 56.183	Yes	Yes	117.12	145.21	Pass (1.24)
56.950	Yes	Yes	119.89	142.26	Pass (1.19)
($0.9L_s$) 63.206	Yes	Yes	142.72	138.64	Fail (0.97)
64.229	Yes	Yes	146.49	140.03	Fail (0.96)
(1.5H) 65.354	Yes	Yes	150.64	141.83	Fail (0.94)
(H) 66.812	Yes	Yes	156.05	264.23	Pass (1.69)
(CS) 67.618	Yes	Yes	159.04	266.48	Pass (1.68)
(PSXFR) 68.507	Yes	Yes	$	$	Pass

Location from Left Support (ft)	Stirrups Required	Stirrups Provided	$\lvert V_u \rvert$ (kip)	ϕV_n (kip)	Status ($\phi V_n/V_u$)
68.590	Yes	Yes	$	$	Pass
(FoS) 69.729	Yes	Yes	$	$	Pass
($1.0L_s$) 70.229	Yes	Yes	$	$	Pass

$ [LRFD 5.8.3.2] The reaction introduces compression into the end of the girder. Sectional design is not used at this location.
[LRFD C5.8.3.2] Loads close to the support are transferred directly to the support by compressive arching action without causing additional stresses in the stirrups. A_v/S at this section has been compared to A_v/S at the critical section.

Optional Live Load Deflection Check (LRFD 2.5.2.6.2)

Allowable deflection span ratio = L/800
Allowable maximum deflection = ± 1.053 in
Minimum live load deflection along girder = -0.498 in
Maximum live load deflection along girder = 0.000 in
Status = Pass

Splitting Zone Stirrup Check [5.10.10.1]

Splitting Zone Length = 0.729 ft
Splitting Force = 41.91 kip
Splitting Resistance = 38.09 kip
Status = **Fail**

Confinement Stirrup Check [5.10.10.2]
Length of confinement zone is 4.375ft

Zone #	End Location From Girder End (ft)	S (in)	S_{max} (in)	Bar Size	Min Bar Size	Status
1	0.125	1.500	6.000	#3	#3	Pass
2	1.000	3.500	6.000	#3	#3	Pass
3	2.000	6.000	6.000	#3	#3	Pass
4	5.000	9.000	6.000	#3	#3	**Fail**
5	67.785	-	-	-	-	N/A
4	70.785	9.000	6.000	#3	#3	**Fail**
3	71.785	6.000	6.000	#3	#3	Pass
2	72.660	3.500	6.000	#3	#3	Pass
1	72.785	1.500	6.000	#3	#3	Pass

Longitudinal Reinforcement for Shear Check - Strength I [5.8.3.5]

$$A_s f_y + A_{ps} f_{ps} \geq \left[\frac{M_u}{d_v \varphi_f} + 0.5\frac{N_u}{\varphi_a} + \left(\left| \frac{V_u}{\varphi_v} - V_p \right| - 0.5 V_s \right) \cot\theta \right] \quad 5.8.3.5-1$$

$$A_s f_y + A_{ps} f_{ps} \geq \left(\frac{V_u}{\varphi_v} - V_p - 0.5 V_s \right) \cot\theta \quad 5.8.3.5-2$$

Location from Left Support (ft)	Capacity (kip)	Demand (kip)	Equation	Status (C/D)
(FoS) 0.500	376.37	128.71	5.8.3.5-2	Pass

Location from Left Support (ft)	Capacity (kip)	Demand (kip)	Equation	Status (C/D)
				(2.92)
1.639	613.95	128.71	5.8.3.5-2	Pass (4.77)
(PSXFR) 1.722	631.30	128.71	5.8.3.5-2	Pass (4.90)
(CS) 2.611	668.43	128.71	5.8.3.5-2	Pass (5.19)
(H) 3.417	702.07	382.32	5.8.3.5-1	Pass (1.84)
(1.5H) 4.875	762.93	569.63	5.8.3.5-1	Pass (1.34)
6.001	809.90	630.83	5.8.3.5-1	Pass (1.28)
(0.1L_s) 7.023	852.57	683.27	5.8.3.5-1	Pass (1.25)
13.279	1390.97	939.08	5.8.3.5-1	Pass (1.48)
(0.2L_s) 14.046	1507.69	961.98	5.8.3.5-1	Pass (1.57)
20.558	1514.06	1087.52	5.8.3.5-1	Pass (1.39)
(0.3L_s) 21.069	1514.55	1092.71	5.8.3.5-1	Pass (1.39)
(HP) 27.836	1520.81	1119.85	5.8.3.5-1	Pass (1.36)
(0.4L_s) 28.092	1520.81	1154.08	5.8.3.5-1	Pass (1.32)
(0.5L_s) 35.115	1520.87	1158.10	5.8.3.5-1	Pass (1.31)
(0.6L_s) 42.137	1520.81	1154.08	5.8.3.5-1	Pass (1.32)
(HP) 42.393	1520.81	1119.85	5.8.3.5-1	Pass (1.36)
(0.7L_s) 49.160	1514.55	1092.71	5.8.3.5-1	Pass (1.39)
49.672	1514.06	1087.52	5.8.3.5-1	Pass (1.39)
(0.8L_s) 56.183	1507.69	961.98	5.8.3.5-1	Pass (1.57)
56.950	1390.97	939.08	5.8.3.5-1	Pass (1.48)
(0.9L_s) 63.206	852.57	683.27	5.8.3.5-1	Pass (1.25)
64.229	809.90	630.83	5.8.3.5-1	Pass (1.28)
(1.5H) 65.354	762.93	569.63	5.8.3.5-1	Pass (1.34)
(H) 66.812	702.07	382.32	5.8.3.5-1	Pass (1.84)

Location from Left Support (ft)	Capacity (kip)	Demand (kip)	Equation	Status (C/D)
(CS) 67.618	668.43	128.71	5.8.3.5-2	Pass (5.19)
(PSXFR) 68.507	631.30	128.71	5.8.3.5-2	Pass (4.90)
68.590	613.95	128.71	5.8.3.5-2	Pass (4.77)
(FoS) 69.729	376.37	128.71	5.8.3.5-2	Pass (2.92)

Check for Lifting In Casting Yard [5.5.4.3]
Lifting Stresses and Factor of Safety Against Cracking

Maximum allowable concrete compressive stress = $-0.65f'_{ci}$ = -4.420 KSI

Maximum allowable concrete tensile stress = $0.0948\sqrt{f'_{ci}}$ but not more than: 0.200 = 0.200 KSI

Maximum allowable concrete tensile stress = $0.1900\sqrt{f'_{ci}}$ = 0.495 KSI if at least 0.353 in^2 of mild reinforcement is provided

Allowable factor of safety against cracking = 1

f'_{ci} required to satisfy this stress check = 6.206 KSI

Location from Left Pick Point (ft)	Min Stress (KSI)	Max Stress (KSI)	Tension Status w/o Rebar	Tension Status w/ Rebar	Compression Status	FS_{cr}	FS Status
-1.750	0.000	0.000	Pass	Pass	Pass	17.420	Pass
-0.472	-1.683	0.043	Pass	Pass	Pass	17.420	Pass
(Pick Point, $0.0L_s$) 0.000	-2.320	0.067	Pass	Pass	Pass	17.420	Pass
(PSXFR) 1.250	-3.945	0.099	Pass	Pass	Pass	17.420	Pass
5.528	-3.868	0.062	Pass	Pass	Pass	17.273	Pass
6.551	-3.855	0.057	Pass	Pass	Pass	15.973	Pass
($0.1L_s$) 6.928	-3.851	0.055	Pass	Pass	Pass	15.553	Pass
12.807	-3.822	0.048	Pass	Pass	Pass	11.311	Pass
13.574	-3.823	0.049	Pass	Pass	Pass	10.942	Pass
($0.2L_s$) 13.857	-3.824	0.050	Pass	Pass	Pass	10.812	Pass
20.085	-3.879	0.091	Pass	Pass	Pass	8.503	Pass
20.597	-3.887	0.097	Pass	Pass	Pass	8.344	Pass
($0.3L_s$) 20.785	-3.890	0.099	Pass	Pass	Pass	8.286	Pass
(HP) 27.364	-4.034	0.193	Pass	Pass	Pass	6.390	Pass
27.620	-4.030	0.191	Pass	Pass	Pass	6.398	Pass
($0.4L_s$) 27.714	-4.029	0.190	Pass	Pass	Pass	6.401	Pass
($0.5L_s$) 34.642	-3.982	0.163	Pass	Pass	Pass	6.505	Pass
($0.6L_s$) 41.571	-4.029	0.190	Pass	Pass	Pass	6.401	Pass
41.665	-4.030	0.191	Pass	Pass	Pass	6.398	Pass
(HP) 41.921	-4.034	0.193	Pass	Pass	Pass	6.390	Pass
($0.7L_s$) 48.499	-3.890	0.099	Pass	Pass	Pass	8.286	Pass
48.688	-3.887	0.097	Pass	Pass	Pass	8.344	Pass

Location from Left Pick Point (ft)	Min Stress (KSI)	Max Stress (KSI)	Tension Status w/o Rebar	Tension Status w/ Rebar	Compression Status	FS_{cr}	FS Status
49.199	-3.879	0.091	Pass	Pass	Pass	8.503	Pass
($0.8L_s$) 55.428	-3.824	0.050	Pass	Pass	Pass	10.812	Pass
55.711	-3.823	0.049	Pass	Pass	Pass	10.942	Pass
56.478	-3.822	0.048	Pass	Pass	Pass	11.311	Pass
($0.9L_s$) 62.356	-3.851	0.055	Pass	Pass	Pass	15.553	Pass
62.734	-3.855	0.057	Pass	Pass	Pass	15.973	Pass
63.756	-3.868	0.062	Pass	Pass	Pass	17.273	Pass
(PSXFR) 68.035	-3.945	0.099	Pass	Pass	Pass	17.420	Pass
(Pick Point, $1.0L_s$) 69.285	-2.320	0.067	Pass	Pass	Pass	17.420	Pass
69.757	-1.683	0.043	Pass	Pass	Pass	17.420	Pass
71.035	0.000	0.000	Pass	Pass	Pass	17.420	Pass

Factor of Safety Against Failure

Factor of Safety Against Failure (FS_f)	7.077
Allowable Factor of Safety Against Failure	1.500
Status	Pass

Check for Hauling to Bridge Site [5.5.4.3]
Hauling Stresses and Factor of Safety Against Cracking

Maximum allowable concrete compressive stress = $-0.65f'_c$ = -5.200 KSI

Maximum allowable concrete tensile stress, plumb girder with impact = $0.0948\sqrt{f'_c}$ = 0.268 KSI

Maximum allowable concrete tensile stress, plumb girder with impact = $0.1900\sqrt{f'_c}$ = 0.537 KSI if at least 1.441 in^2 of mild reinforcement is provided

Maximum allowable concrete tensile stress, inclined girder without impact = $f_r = 0.2400\sqrt{f'_c}$ = 0.679 KSI

Allowable factor of safety against cracking = 1

f'_c required to satisfy stress and stability criteria = 6.672 KSI

Location from Left Bunk Point (ft)	Min Stress# (KSI)	Max Stress# (KSI)	Min Stress* (KSI)	Max Stress* (KSI)	Tension Status w/o Rebar	Tension Status w/ Rebar	Compression Status	FS_{cr}	FS Status
-5.000	0.000	0.000	0.000	0.000	Pass	Pass	Pass	5.478	Pass
-3.722	-1.635	0.042	-1.635	0.042	Pass	Pass	Pass	5.478	Pass
(PSXFR) -2.000	-3.930	0.149	-3.932	0.150	Pass	Pass	Pass	5.478	Pass
(Bunk Point, $0.0L_s$) 0.000	-4.048	0.215	-4.053	0.216	Pass	Pass	Pass	5.478	Pass
2.278	-4.011	0.203	-4.033	0.207	Pass	Pass	Pass	5.478	Pass
3.301	-3.999	0.201	-4.032	0.208	Pass	Pass	Pass	5.478	Pass
($0.1L_s$) 6.278	-3.973	0.201	-4.037	0.214	Pass	Pass	Pass	5.458	Pass
9.557	-3.965	0.212	-4.058	0.230	Pass	Pass	Pass	5.319	Pass
10.324	-3.966	0.216	-4.065	0.235	Pass	Pass	Pass	5.288	Pass
($0.2L_s$) 12.557	-3.975	0.230	-4.091	0.253	Pass	Pass	Pass	5.200	Pass
16.835	-4.017	0.271	-4.161	0.299	Fail	Pass	Pass	5.028	Pass

Location from Left Bunk Point (ft)	Min Stress# (KSI)	Max Stress# (KSI)	Min Stress* (KSI)	Max Stress* (KSI)	Tension Status w/o Rebar	Tension Status w/ Rebar	Compression Status	FS_{cr}	FS Status
17.347	-4.024	0.277	-4.171	0.306	Fail	Pass	Pass	5.006	Pass
($0.3L_s$) 18.835	-4.048	0.296	-4.203	0.326	Fail	Pass	Pass	4.942	Pass
(HP) 24.114	-4.162	0.378	-4.337	0.412	Fail	Pass	Pass	4.663	Pass
24.370	-4.158	0.376	-4.334	0.411	Fail	Pass	Pass	4.664	Pass
($0.4L_s$) 25.114	-4.149	0.372	-4.327	0.406	Fail	Pass	Pass	4.668	Pass
($0.5L_s$) 31.392	-4.113	0.352	-4.298	0.389	Fail	Pass	Pass	4.684	Pass
($0.6L_s$) 37.671	-4.149	0.372	-4.327	0.406	Fail	Pass	Pass	4.668	Pass
38.415	-4.158	0.376	-4.334	0.411	Fail	Pass	Pass	4.664	Pass
(HP) 38.671	-4.162	0.378	-4.337	0.412	Fail	Pass	Pass	4.663	Pass
($0.7L_s$) 43.949	-4.048	0.296	-4.203	0.326	Fail	Pass	Pass	4.942	Pass
45.438	-4.024	0.277	-4.171	0.306	Fail	Pass	Pass	5.006	Pass
45.949	-4.017	0.271	-4.161	0.299	Fail	Pass	Pass	5.028	Pass
($0.8L_s$) 50.228	-3.975	0.230	-4.091	0.253	Pass	Pass	Pass	5.200	Pass
52.461	-3.966	0.216	-4.065	0.235	Pass	Pass	Pass	5.288	Pass
53.228	-3.965	0.212	-4.058	0.230	Pass	Pass	Pass	5.319	Pass
($0.9L_s$) 56.506	-3.973	0.201	-4.037	0.214	Pass	Pass	Pass	5.458	Pass
59.484	-3.999	0.201	-4.032	0.208	Pass	Pass	Pass	5.478	Pass
60.506	-4.011	0.203	-4.033	0.207	Pass	Pass	Pass	5.478	Pass
(Bunk Point, $1.0L_s$) 62.785	-4.048	0.215	-4.053	0.216	Pass	Pass	Pass	5.478	Pass
(PSXFR) 64.785	-3.930	0.149	-3.932	0.150	Pass	Pass	Pass	5.478	Pass
66.507	-1.635	0.042	-1.635	0.042	Pass	Pass	Pass	5.478	Pass
67.785	0.000	0.000	0.000	0.000	Pass	Pass	Pass	5.478	Pass

based on inclined girder without impact
* based on plumb girder with impact

Factor of Safety Against Rollover

Factor of Safety Against Rollover (FS_r)	2.948
Allowable Factor of Safety Against Rollover	1.500
Status	Pass

Spacing Between Truck Supports for Hauling

Distance Between Supports	62.785 ft
Max. Allowable Distance Between Supports	230.000 ft
Status	Pass

Girder Support Configuration

Leading Overhang (closest to cab of truck)	5.000 ft
Max. Allowable Leading Overhang	15.000 ft
Status	Pass

Maximum Girder Weight

Girder Weight	67.05 kip
Maximum Allowable Weight	200.00 kip
Status	Pass

Girder Dimensions Detailing Check [5.14.1.2.2]

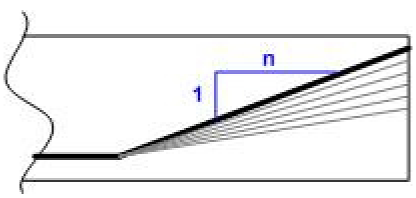
Dimension	Minimum (in)	Actual (in)	Status
Top Flange Thickness	2.000	6.000	Pass
Web Thickness	5.000	6.000	Pass
Bottom Flange Thickness	5.000	6.000	Pass

Stirrup Detailing Check [5.8.2.5, 5.8.2.7, 5.10.3.1.2]

Location from Left Support (ft)	Bar Size	S (in)	S_{max} (in)	S_{min} (in)	A_v/S (in^2/ft)	A_v/S_{min} (in^2/ft)	Status
($0.0L_s$) 0.000	#5	6.000	10.080	1.625	1.240	0.107	Pass
(FoS) 0.500	#5	6.000	10.080	1.625	1.240	0.107	Pass
1.639	#5	9.000	10.080	1.625	0.827	0.107	Pass
(PSXFR) 1.722	#5	9.000	10.080	1.625	0.827	0.107	Pass
(CS) 2.611	#5	9.000	10.132	1.625	0.827	0.107	Pass
(H) 3.417	#5	9.000	10.188	1.625	0.827	0.107	Pass
(1.5H) 4.875	#4	18.000	18.000	1.500	0.267	0.107	Pass
6.001	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.1L_s$) 7.023	#4	18.000	18.000	1.500	0.267	0.107	Pass
13.279	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.2L_s$) 14.046	#4	18.000	18.000	1.500	0.267	0.107	Pass
20.558	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.3L_s$) 21.069	#4	18.000	18.000	1.500	0.267	0.107	Pass
(HP) 27.836	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.4L_s$) 28.092	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.5L_s$) 35.115	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.6L_s$) 42.137	#4	18.000	18.000	1.500	0.267	0.107	Pass
(HP) 42.393	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.7L_s$) 49.160	#4	18.000	18.000	1.500	0.267	0.107	Pass
49.672	#4	18.000	18.000	1.500	0.267	0.107	Pass

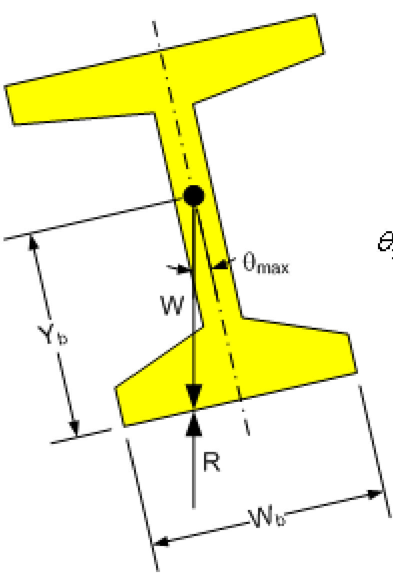
Location from Left Support (ft)	Bar Size	S (in)	S_{max} (in)	S_{min} (in)	A_v/S (in^2/ft)	A_v/S_{min} (in^2/ft)	Status
($0.8L_s$) 56.183	#4	18.000	18.000	1.500	0.267	0.107	Pass
56.950	#4	18.000	18.000	1.500	0.267	0.107	Pass
($0.9L_s$) 63.206	#4	18.000	18.000	1.500	0.267	0.107	Pass
64.229	#4	18.000	18.000	1.500	0.267	0.107	Pass
(1.5H) 65.354	#4	18.000	18.000	1.500	0.267	0.107	Pass
(H) 66.812	#5	9.000	10.188	1.625	0.827	0.107	Pass
(CS) 67.618	#5	9.000	10.132	1.625	0.827	0.107	Pass
(PSXFR) 68.507	#5	9.000	10.080	1.625	0.827	0.107	Pass
68.590	#5	9.000	10.080	1.625	0.827	0.107	Pass
(FoS) 69.729	#5	6.000	10.080	1.625	1.240	0.107	Pass
($1.0L_s$) 70.229	#5	6.000	10.080	1.625	1.240	0.107	Pass

Strand Slope



	1 : n
Allowable Slope	8.000
Strand Slope	12.940
Status	Pass

Global Stability of Girder



$$\theta_{max} = \frac{W_b}{6Y_b}$$

W_b (in)	Y_b (in)	Incline from Vertical (θ_{max}) (ft/ft)	Max Incline (ft/ft)	Status

W_b (in)	Y_b (in)	Incline from Vertical (θ_{max}) (ft/ft)	Max Incline (ft/ft)	Status
25.000	22.863	0.020	0.182	Pass

AASHTO Flexural Prestressed Design

Loads

AASHTO Max Lane Moments

Dead Loads			Live Load		
M_{SW}	622.56	kip*ft	M_{LL}	1525.33	kip*ft
M_{DC}	0	kip*ft	Dynamic Load Allowance		
M_{Ba}	0	kip*ft	IM	503.359	kip*ft
M_{WC}	147.66	kip*ft			

Live Load Per Beam 1413.170194 kip*ft

Total Beam Moment 2183.39 kip*ft

Estimate Required Prestress

fb=(Msw+Mdi+Mba+Mwc+MLL+I)/Sb

fb= 5.160860163 ksi

Stress Limits for Concrete

Limit=.19*sqrt(f'c)

Limit= 0.537401154

Tensile Stress Limit 0.5374 ksi

Required Number of Strands

fpb=fb-tensile stress limit

fpb= -4.62345901

ec= 19.773 in

fpb=Ppe/A+Ppe*ec/Sb

5.407=Ppe/823+Ppe*20/5076.805

Ppe= 882.8115632 kips

Final Prestress Force=(area of strand)*fpj*(1-final losses)

Assume 20% losses

Final Prestress Force= 35.154

Number of Strands Reqiured 25.11269

I will try 26 .6" diameter, 270ksi strands

New As 5.642 in^2

Strand Pattern

Ybs	3.09	in
ec	19.773	in

Strength Limit State

Mu=1.25*(DC)+1.5(DW)+1.75(LL+IM)
Mu= 3472.737839 ft*kips

fps=fpu(1-k*(c/dp))

k=2*(1.04-fpy/fpu)
k= 0.28
dp=H-Ys
dp= 31.91

c=(Apsfpu+AsFy-As'fy')/(.85f'c*B1b+kAps(fpu/dp))
c= 3.91879316

a=b1*c
a= 3.135034528 <= 6 in
Okay to continue

fps=fpu(1-k*(c/dp))
fps 262.5725913

Mn=Aps*fps*(dp-a/2)+.85*f'c*(b-bw)*B1*hf*(a/2-hf/2)
Mn= 3504.292992 ft*kips

Mr=ΦMn
Φ= 1
Mr= 3504.292992
Mu= 3472.737839
So Beam is Sufficient

Total Prestress Loss Δfp(ES+SR+CR+R2)

Elastic Shortening
ΔfpES=Ep/Eci*fcgp

Fcgp
Fpii= fpi*(1-.05)
Fpii= 192.375

Pii 1085.37975 kips

Fcgp=Pi/A+Pi*ec^2/I-(Mg+MD)*ec/I

Fcgp= 3.70212633

Elastic Shortening

ΔfpES=Ep/Eci*fcgp

Pi= 1023.429095

Fcgp=Pi/A+Pi*ec^2/I-(Mg+MD)*ec/I

Fcgp= 3.418178639

ΔfpES=Ep/Eci*fcgp

ΔfpES= 19.48652195 ksi

ΔfpES= 21.105265 assumed

Elastic Shortening

ΔfpES=Ep/Eci*fcgp

Pi= 1032.562043

Fcgp=Pi/A+Pi*ec^2/I-(Mg+MD)*ec/I

Fcgp= 3.460039045

ΔfpES=Ep/Eci*fcgp

ΔfpES= 19.72516183 ksi

ΔfpES= 19.48652195 assumed

Elastic Shortening

ΔfpES=Ep/Eci*fcgp

Pi= 1031.215637

Fcgp=Pi/A+Pi*ec^2/I-(Mg+MD)*ec/I

Fcgp= 3.453867861

ΔfpES=Ep/Eci*fcgp

ΔfpES= 20.45349956 ksi

ΔfpES= 19.72516183 assumed

Elastic Shortening

ΔfpES=Ep/Eci*fcgp

Pi= 1027.106355

Fcgp=Pi/A+Pi*ec^2/I-(Mg+MD)*ec/I

Fcgp= 3.435033177

ΔfpES=Ep/Eci*fcgp

ΔfpES= 19.58260714 ksi Use

ΔfpES= 20.45349956 assumed

ΔfpES=Ep/Eci*fcgp

ΔfpES= 21.105265 ksi

ΔfpES=		10.125	assumed	
ΔfpES=		**19.58260714**	**ksi**	

Shrinkage

ΔfpSr=(17-.15H)				
ΔfpSr=		6.5	ksi	

Prestress Loss Due to Creep Pretensioned Case

Kcr		2		Ec	4999.255
Fcs=	same as above=				
Fcsd=MDL*e/I					
MDL (no SW)=		147.66	k*ft		
fcs=Pi/A-Pi*e*e/I+Msw*e/I					
fcs		-2.196418417	ksi		
Fcsd=MDL*e/I					
Fcsd		0.30185123	ksi		
ΔfpCR=KCR*Eps/Ec*(fcs-fcsd)					
ΔfpCR		**19.91540051**	**ksi**		

Prestress Loss Due To Relaxation

ΔfpiR2=30%(20-.4*fpES-.2(ΔfpSR+ΔfpCR)					
ΔfpiR2=		2.87	ksi		
Total loss at transfer					
Δfpi=ΔfpES+ΔfpSr+ΔfpCR+ΔfpiR2					
Δfpi=		42.37			
Fpe=		160.13	ksi		
Check	Ok		<=.8fpy		=
.8fpy=		194.4			
Ppe		903.4642092			
Final Losses					
	%=ΔfpT/fpi=			20.92%	

Required Number of Strands

fpb=fb-tensile stress limit			
fpb=		4.62345901	

ec= 19.773 in

fpb=Ppe/A+Ppe*ec/Sb
5.407=Ppe/823+Ppe*20/5076.805
Ppe= 896.8574452 kips

Final Prestress Force=(area of strand)*fpj*(1-final losses)
Assume 23% losses
Final Prestress Force= 33.8357
Number of Strands Reqiured 26.50623

I will try 28 .6" diameter, 270ksi strands
New As 6.076 in^2
Strand Pattern
Ybs 3.09 in

ec 19.773 in

Strength Limit State

Mu=1.25*(DC)+1.5(DW)+1.75(LL+IM)
Mu= 3472.737839 ft*kips

fps=fpu(1-k*(c/dp))

k=2*(1.04-fpy/fpu)
k= 0.28
dp=H-Ys
dp= 31.91

c=(Apsfpu+AsFy-As'fy')/(.85f'c*B1b+kAps(fpu/dp))
c= 4.209105331

a=b1*c
a= 3.367284265 <= 6 in
Okay to continue

fps=fpu(1-k*(c/dp))
fps 262.0223538

Mn=Aps*fps*(dp-a/2)+.85*f'c*(b-bw)*B1*hf*(a/2-hf/2)
Mn= 3784.579516 ft*kips

Mr=ΦMn
Φ= 1
Mr= 3784.579516
Mu= 3472.737839

So Beam is Sufficient

Stresses at Transfer

Jacking force per strand=fpj*strand area
= 43.9425 k
Force per strand after initial losses=(fpj-ΔfpEs)*Astrand
= 39.69307425 kips

Pi 1032.019931 kips

Stress Limits for Concrete

Compressiohn: 0.6f'ci
= 4.08 ksi

Tension
w.o auxiliary reinforcement=.0948*sqrt(f'ci) <=.2
= 0.247208155
= 0.2
w auxiliary reinforcement=.22*sqrt(f'ci) 0.51595
Stresss at Transfer Length Section

Transfer length=60*strand diameter
= 36 in
= 3 ft

Equivalent w, Msw	Msw*8/L^2	w=	0.94026
Equivalent w, Md	Md*8/L^2	w=	0

at 3ft

Msw=	w*x(L-x)	196.8336372	ft*kips
Md=	w*x(L-x)	0	ft*kips

ft=Pi/A+Pi*ee/St-(Mg+Md)/St
ft= -0.911749208 compression

fb=Pi/A+Pi*ec/Sb-(Mg+Md)/Sb
fb -1.89863552 compression

Compression Stress limit is okay

Stresses at midspan

ft=Pi/A-Pi*ec/St+(Mg+Md)/St

ft=	-0.098621971	compression

fb=Pi/A+Pi*ec/Sb-(Mg+Md)/Sb

fb	-3.801916088	ksi
		compression
fb allowable	4.08	

Service Stress Limits for Concrete

Compression	3.6	ksi
Tension	0.537401154	ksi

ft=Pe/A-Pe*ec/St+(Mg+Md+Mws+Mb)/St

ft=	-0.20188582	compression
OKAY		

fb=Pe/A+Pe*ec/Sb-(Mg+Md+Mws+Mb)/Sb

fb	-2.76223491	compression
OKAY		

Service III

ft=Pe/A-Pe*ec/St+(Mg+Md+Mws+Mb)/St

ft=	-1.975108759	Compression
OKAY		
Compression		

fb=Pe/A+Pe*ec/Sb-(Mg+Md+Mws+Mb)/Sb

fb	0.578063026	Tension
OKAY		

Limits of Reinforcement

Maximum Reinforcement

c/de<=.42

de=Apsfpsdp+AsFyds/Apsfps+asfy

de	31.91	
c=	4.209105331	
c/de=	0.131905526	<= 0.42
		OK

Final Design

W35-DG		
H=	35	in
Dp=	31.91	in
f'c	8000	psi
f'ci	6800	psi
Harped Strands	8	
Strait Strands	18	

Strands are allowed acording to Wash Dot specs.

Givens/Imputs

Prestress Applied	U.W. Concrete	Ecenter	Eend	Beam Type	Width	Length	Span	Steel Relax Period	#Strands
7	150	19.8	5.46	W35DG	74.75	72.78	70.229	60	23
years	pcf	in	in		in	ft	ft	day	#
									26

RH	f'ci	f'c	Strand Dia.	Strand Area	Aps	Fpu
70.00	6800	8000	0.6	0.217	4.991	270
%	psi	psi	in	in	in^2	ksi

Strait	Harped	Ac	Ic	r^2	Fpj	Eps	ys	Distanc to Ec	Slab Th.
16	7	823	1E+05		202.5	28500	3.09	27.836	6
#	#	in^2	in^4	in^2	ksi	ksi	in	ft	in
18	8								

fpy	Fy	Es	Ec	Eci	H	Yb	Yt	Sb	St
243	60	####	5422	4999.25	35	22.86	12.137	5076.805319	9563.4
plf	in								

SW	Effective Flange
857.2916667	69

Shear Design Calculator

Length, ft.	75
Eccentricity at end, in.	12.248
C top, in.	12.863
Beam Height, in.	35
Self Weight, plf	885.42
Superimposed Dead Load, plf	210
Live Load, plf	2300
Moment of Inertia, in^4	116071
f'c, psi	8000
fpe, psi	157500
fps, psi	243000
Area of Prestress, in^2	6.076
Pe, lb,	**956970**
Lambda, λ	1
y tension, in.	22.137
Area of Beam, in^2	823
Section Modulus of Tension Side, St, in^3	5076.805
Web Thickness, in.	6
Compression at Centroid, fpc, psi	**1162.78**

	Percent of Length		
	1.940%	10%	20%
Eccentricity, in.	**12.839**	**14.095**	**15.941**
Depth of Prestressing, dp, in.	**28.0**	**28.0**	**28.8**
Shear Due to Self Weight, Vd, lb.	**31,915**	**26,563**	**19,922**
Moment Due to External Loads, M max, lb-in	**4,126,369**	**19,906,516**	**36,325,459**
Shear Due to External Loads, Vi, lb.	**61,819**	**50,440**	**41,958**
fpe at x, psi	**3,583**	**3,820**	**4,168**
Stress Due to Self Weight at Tension Face, fd, psi	**112**	**530**	**942**
Mcre, lb-in	**21,013,032**	**20,063,865**	**19,727,960**
Flexural Shear, Vci, lb.	**350,857**	**81,940**	**47,842**
Shear Check, lb.	**25,545**	**25,545**	**26,278**
Tangent Angle, radians, α	0.1825	0.1825	0.1825
Vertical Component of Prestress, Vp, lb.	**173,679**	**173,679**	**173,679**
Web Shear, Vcw, lb.	**284,876**	**284,876**	**288,069**
ΦVc, lb.	213,657	61,455	35,881
Vu, lb.	103,348	83,334	66,417
ΦVn, lb.	258,657	106,455	80,881

Area of Shear Reinforcement, in^2	0.2		Spacing Requir
Number of Legs	2		Spacing, in.
fy, psi	60000		
Height of Section, in.	35		
Spacing, in.	24		
As minimum, in^2	**0.12**	**or**	**0.160996894**

Capacity Needed from Shear Reinforcement, lb	-110,309	21,879	30,536
ΦVs, lb.	26,250	26,250	26,250

Parabolic Strand Equation, y=ax^2+bx+c		
a	b	c
0.001777778	-0.1333333	0

Percent of Length					
30%	40%	50%	60%	70%	80%
17.786	19.632	19.632	19.632	17.786	15.941
30.6	32.5	32.5	32.5	30.6	28.8
13,281	6,641	0	6,641	13,281	19,922
49,664,696	58,277,501	62,462,509	59,573,405	51,811,909	37,632,653
33,476	23,330	14,848	4,702	11,068	15,770
4,515	4,863	4,863	4,863	4,515	4,168
1,236	1,413	1,472	1,413	1,236	942
20,008,327	20,906,940	20,598,311	20,906,940	20,008,327	19,727,960
32,495	21,332	11,218	14,612	23,282	33,322
27,962	29,646	29,646	29,646	27,962	26,278
0.1825	0	0	0	0.1825	0.1825
173,679	0	0	0	173,679	173,679
295,396	129,048	129,048	129,048	295,396	288,069
24,371	15,999	8,413	10,959	17,462	24,991
49,500	31,004	14,086	16,917	35,413	52,330
69,371	42,249	34,663	37,209	62,462	69,991

Percent of Length		
90%	98.055%	
14.095	12.839	
28.0	28.0	
26,563	31,912	
20,567,629	4,384,708	
71,624	130,182	
3,820	3,583	
530	112	
20,063,865	21,011,549	
100,971	659,877	*Values must be greater than the shear check
25,545	25,545	
0.1825	0.1825	
173,679	173,679	
284,876	284,876	
75,728	213,657	
117,776	185,438	

120,728	258,657				
80					
59.6284794		Maximum Spacing		24	
26.25					
24					
25,129	15,005	5,673	5,958	17,951	27,339
26,250	26,250	26,250	26,250	26,250	26,250
42,048	-28,219				
26,250	26,250				

Percent of Length	0%	10%	20%	30%	40%	50%	60%	70%	80%	90%	100%
Shear from Live Load	32.96	26.8	22.68	18.56	13.4	9.28	4.12	9.28	13.4	49.49	87.04
Shear from DC	33203.25	26562.60	19921.95	13281.30	6640.65	0.00	6640.65	13281.30	19921.95	26562.60	33203.25
Shear from DW	7875.00	6300.00	4725.00	3150.00	1575.00	0.00	1575.00	3150.00	4725.00	6300.00	7875.00
Ultimate Shear (Strength I), lbs.	103347.92	83334.09	66416.85	49499.62	31003.73	14086.50	16917.23	35413.12	52330.36	117776.18	185438.21
Moment from Superimposed LL, lb-ft	83429	414743	786076	1135242	1376474	1525326	1443969	1247076	854159	449176	96552
Ultimate Moment, lb-ft	133,486.40	996,322.63	1,849,248.40	##########	3,089,648.60	3,364,782.23	3,197,640.60	2,771,700.53	1,958,181.20	1,051,415.43	154,483.20
Vu*dp/Mu	9.9402182	1.178929317	0.572530482	0.33956024	0.197001447	0.082188422	0.103863799	0.227245986	0.426006645	1.578876685	15.4116507

Appendix 3.2: Pier Design

Below are all the spreadsheets used for the design of the intermediate pier. Section 3.9.4 of the methodology illustrates the process used for the design and section 5.2 displays the results. The spreadsheets used for the design are broken down into the categories shown below.

Wind Loads on Pier

The wind loads on the pier (both transverse and longitudinal) required guess-and-check work, warranting its own spreadsheet. This spreadsheet was done in accordance with the *AASHTO LRFD 2012 Specifications*.

Pier Cap Design

This spreadsheet was created to design the pier cap given the maximum loads and dimensions. The spreadsheet contains relevant calculations for the shear, skin, temperature, and upper and lower flexural reinforcement. All relevant AASHTO sections are noted and AASHTO limits are checked to ensure an adequate design.

Pier Column Design

This spreadsheet was used to design the pier columns in tandem with the column-interaction diagram found in section 5.2.3. This spreadsheet also ensures the column in accordance with the *AASHTO LRFD 2012 Specifications*. The transverse and longitudinal forces were combined using the moment magnification method outlined by AASHTO.

Bearing Capacity Calculation

The geometry of the pier footing was determined using the bearing capacity and settlement spreadsheets. The spreadsheets were obtained through the purchase of the (Cudotu) book. The spreadsheet used the shown inputs in order to determine the minimum size of the footings to meet bearing capacity requirements.

Settlement Calculation

The settlement calculation was done using the displayed spreadsheet obtained from the same way as the bearing capacity spreadsheet. This spreadsheet was highly dependent on the soil information to calculate the settlement of the structure.

Wind Loads on Pier

Columns						**Pier Cap**			
Longitudinal						Longitudinal			
P_B:	0.04	ksf				P_B:	0.04	ksf	
D_{column}:	3.5	ft				Fw:	2.99	k/ft	
Fw:	0.14	k/ft							
Transverse						Transverse			
P_B:	0.04	ksf				P_B:	0.04	ksf	
D_{column}:	3.5	ft				Fw:	0.16	k/ft	
Fw:	0.028	k/ft							
Superstructure						**Live Load**	*Apply 6 feet above deck		
Longitudinal		*60 degrees				Longitudinal		*60 degrees	
P_D:	0.019	ksf				P_B:	0.04	klf	
H:	2.67	ft				Fw:	6	k	
L:	150	ft							
Fw:	7.6095	k							
Transverse		*0 degrees				Transverse		*0 degrees	
P_B:	0.05	ksf				P_B:	0.1	klf	
A:	218.75	ft^2				Fw:	7.5	k	
Fw:	10.9375	k							
http://www.fhwa.dot.gov/bridge/lrfd/pscus72.htm									

Pier Cap Design

Pier Cap Flexural Resistance (S5.7.3.2)				
f'c:	3	ksi		
β_1:	0.85			
fy:	60	ksi		
b:	4	ft		
h:	3.833	ft		
Av:	0.31	in^2	*per leg	(#5 bars d=0.625 in)
Cover:	2	in		

Bottom Steel							
n_{bars}:	6						
As:	4.74	in^2	(#8 bars, d=1 in)				
d_e:	42.871	in					
a:	2.323529	in					
c:	2.733564	in					
Mn:	988.5089	k-ft					
ϕ:	0.9						
Mr:	889.658	k-ft					
Mu:	849.1	k-ft					
Mr>Mu		ADEQUATE					
$c/d_e \leq$	0.42	ADEQUATE		(S5.7.3.3.1-1)			
fr:	0.415692	ksi		(S5.4.2.6)			
S:	16925.06	in^3					
$1.2M_{cr}$:	703.5614	k-ft		889.658	>	703.5614	ADEQUATE
$1.33M_u$:	1129.303	k-ft					
Z:	170	k/in					
d_c:	2.5	in					
A:	40	in^2					
$f_{s,allow}$:	36.62539	ksi					
$f_{s,allow}$>	36	ADEQUATE					

Top Steel							
n_{bars}:	13						
As:	10.27	in^2	(#8 bars, d=1 in)				
d'_e:	42.871	in					
a:	5.034314	in					
c:	5.922722	in					
Mn:	2072.17	k-ft					
ϕ:	0.9						
Mr:	1864.953	k-ft					
Mu:	1814.9	k-ft					
Mr>Mu		ADEQUATE					
$c/d_e \leq$	0.42	ADEQUATE		(S5.7.3.3.1-1)			
fr:	0.415692	ksi		(S5.4.2.6)			
S:	16925.06	in^3					
$1.2M_{cr}$:	762.1915	k-ft		1864.953	>	762.1915	ADEQUATE
$1.33M_u$:	2413.817	k-ft					
Z:	170	k/in					
d_c:	2.5	in					
A:	18.46154	in^2					
$f_{s,allow}$:	47.39286	ksi					
$f_{s,allow}$>	36	ADEQUATE					

Temperature & Shrinkage Steel (S5.10.8)			
$A_{s,min1}$:	4.047648	in^2	(S5.10.8.2-1)
$A_{s\ prov}$:	2.4	in^2	(4 #7 bars)
$A_{s\ prov}$>	$A_{s,min1}/2$		ADEQUATE

Skin Reinforcement (S5.7.3.4)				
A_{sk}:	0.154452	≤	3.5	
A_{sk}:	0.617808	≤	2.4	(4 #7 bars)

Shear							
bv:	48	in					
dv:	41.70924	in					
0.9de:	38.5839	in					
0.72h:	33.11712	in					
β:	2	(S5.8.3.4)					
Vc:	219.1549	k					
θ:	0.785398	rad	(S5.8.3.4)				
α:	1.570796	rad					
n_{Vlegs}:	4						
Av:	1.24	in^2	(4 legs of #5 bars)				
s:	8	in					
Vs:	387.896	k	(S5.8.3.3-4)				
Vn:	607.0509	*lower value:	607.0509	k			
	1501.532						
ϕ:	0.9						
Vr:	546.3458	k					
Vu:	524.9	k					
Vr>Vu	ADEQUATE						
Av_{min}:	0.35029	in^2					
Av_{min}<	Av	ADEQUATE					
v:	0.291313	ksi					
0.125f'c:	0.375	ksi					
*If v<0.125f'c		TRUE					
		s_{max}:	33.36739	in	≤	24	(s_{max} must be less than or equal to)
*If v>0.125f'c		FALSE					
		s_{max}:	16.68369	in	≤	12	

Pier Column Design

Column Properties							
f'c:	4	ksi					
Ec:	3321	ksi					
n:	9						
fy:	60	ksi					
d:	3.5	ft					
Ag:	9.62	ft^2					
Cover:	2	in					
Vert. r.f.:	#8 bars						
	d:	1	in				
	A:	0.79	in^2				
	n_{bars}:	16					
	A_{LT}:	12.64	in^2				
Trans r.f.:	Ties	(#3 bars)					
	s:	12	in				
	d:	0.375	in				
	A_T:	0.11	in/bar				
Longitudinal Limits (S5.6.4.2)							
As/Ag:	0.0091245	≤	0.08		**ADEQUATE**		
Asfy/Agf'c:	0.1368676	≥	0.135		**ADEQUATE**		
K:	1.2	*in the plane of the bent					
	2.1	*in the direction perpendicular to the bent					
l_u:	23	ft					
r:	0.875	ft					
Slenderness Ratios							
K*l_u/r:	31.542857	>	22	**Slender Column**		*in the plane of the bent	
	55.2	>	22	**Slender Column**		*in the direction perpendicular to the bent	
Moment Magnification							
Cm:	1	(S4.5.3.2.2b)					
Pu:	824.3	k					
ϕ:	0.75						
M_2:	1056.7	k-ft					
Ig:	152745.02	in^4					
βd (L):	0.0785464						
Es:	60000	ksi					
Is:	0.7853982						
EI:	94108482	k-in^2	*Take greater of the values			188129580	k-in^2
	188129580	k-in^2					
lu:	216	in					
Pe:	27636.742	k					
δ_b:	1.0414153	≥	1	**ADEQUATE**			
δ_s:	1.0414153						
Mcl:	1100.4636	k-ft					
βd (T):	0						
EI:	101500366	k-in^2	*Take greater of the values			202906485	k-in^2
	202906485	k-in^2					
Pe:	29807.509	k					
δ_b:	1.0382837						
δ_s:	1.0382837						
Mct:	86.177551	k-ft					
Mu:	1103.8327	k-ft					

Column Interaction Diagram										
ϕ:	0.75	(S5.5.4.2)	*compression controlled							
	0.9		*tension controlled							
gamma:	0.9047619									
Mu:	1103.8327	k-ft								
Pu:	824.3	k								
P_{least}:	197	k								
ρ:	0.0091245									
Kn (max):	0.1983474		Rn:	0.16		Mr:	2792.724	k-ft	**ADEQUATE**	
Kn (least):	0.0474032		Rn:	0.14		Mr:	2443.634	k-ft	**ADEQUATE**	
Rn:	0.0632405		Kn:	1.02		Pr:	4238.957	k	**ADEQUATE**	

Bearing Capacity Calculation

BEARING CAPACITY OF SHALLOW FOUNDATIONS
Terzaghi and Vesic Methods

Date	April 6, 2013	
Identification	Intermediate Pier	

Input

Units of Measurement		
	E	SI or E
Foundation Information		
Shape	sq	SQ, CI, CO, or RE
B =	4.5	ft
L =	4.5	ft
D =	8	ft
Soil Information		
c =	0	lb/ft^2
phi =	42	deg
gamma =	125	lb/ft^3
Dw =	9999	ft
Factor of Safety		
F =	3.5	

Results

	Terzaghi	Vesic
Bearing Capacity		
q ult =	149,805 lb/ft^2	222,346 lb/ft^2
q a =	42,801 lb/ft^2	63,527 lb/ft^2
Allowable Column Load		
P =	867 k	1,286 k

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Unit conve	1000
Gamma w	62.4
phi (radian	0.733038
Terzaghi Computatior	
a theta =	5.998514
Nc =	119.67
Nq =	108.75
N gamma	182.46
gamma' =	125
coefficient	1.3
coefficient	0.4
sigma zD'	1000
Vesic Computation	
Nc =	93.71
sc =	1.91
dc =	1.42
Nq =	85.37
sq =	1.90
dq =	1.21
N gamma	155.54
s gamma	0.60
d gamma	1.00
B/L =	1
k =	1.058407
W sub f	0

Settlement Calculation

SETTLEMENT ANALYSIS OF SHALLOW FOUNDATIONS
Classical Method

Date	April 6, 2013
Identification	Intermediate Pier

Input

Units	E	E or SI
Shape	SQ	SQ, CI, CO, or RE
B =	10	ft
L =	10	ft
D =	8	ft
P =	197	k
Dw =	999	ft
r =	0.85	

Results

q =	3170	lb/ft^2
delta =	1.99	in

Depth to Soil Layer Top (ft)	Bottom (ft)	Cc/(1+e)	Cr/(1+e)	sigma m' (lb/ft^2)	gamma (lb/ft^3)	zf (ft)	sigma c' (lb/ft^2)	sigma zo' (lb/ft^2)	delta sigma (lb/ft^2)	sigma zf' (lb/ft^2)	strain (%)	delta (in)
0.0	8.0				125							
8.0	8.5	0.07	0.06	1350	125	0.25	2381	1031	2170	3201	2.62	0.157
8.5	9.0	0.07	0.06	1350	125	0.75	2444	1094	2165	3258	2.52	0.151
9.0	9.5	0.07	0.06	1350	125	1.25	2506	1156	2146	3303	2.43	0.146
9.5	10.0	0.07	0.06	1350	125	1.75	2569	1219	2110	3329	2.32	0.139
10.0	10.5	0.07	0.06	1350	125	2.25	2631	1281	2053	3335	2.21	0.132
10.5	11.0	0.07	0.06	1350	125	2.75	2694	1344	1978	3322	2.08	0.125
11.0	11.5	0.07	0.06	1350	125	3.25	2756	1406	1888	3294	1.95	0.117
11.5	12.0	0.07	0.06	1350	125	3.75	2819	1469	1788	3257	1.82	0.109
12.0	12.5	0.07	0.06	1350	125	4.25	2881	1531	1682	3213	1.68	0.101
12.5	13.0	0.07	0.06	1350	125	4.75	2944	1594	1574	3168	1.55	0.093
13.0	13.5	0.07	0.06	1350	125	5.25	3006	1656	1468	3124	1.42	0.085
13.5	14.0	0.07	0.06	1350	125	5.75	3069	1719	1365	3084	1.30	0.078
14.0	14.5	0.07	0.06	1350	125	6.25	3131	1781	1268	3049	1.19	0.071
14.5	15.0	0.07	0.06	1350	125	6.75	3194	1844	1176	3020	1.09	0.066
15.0	15.5	0.09	0.04	7600	145	7.25	9511	1911	1091	3002	0.67	0.040
15.5	16.0	0.09	0.04	7600	145	7.75	9584	1984	1012	2996	0.61	0.037
16.0	16.5	0.09	0.04	7600	145	8.25	9656	2056	939	2996	0.56	0.033
16.5	17.0	0.09	0.04	7600	145	8.75	9729	2129	873	3001	0.51	0.030
17.0	17.5	0.09	0.04	7600	145	9.25	9801	2201	811	3013	0.46	0.028
17.5	18.0	0.09	0.04	7600	145	9.75	9874	2274	756	3029	0.42	0.025
18.0	18.5	0.09	0.04	7600	145	10.25	9946	2346	704	3051	0.39	0.023
18.5	19.0	0.09	0.04	7600	145	10.75	10019	2419	658	3076	0.36	0.021
19.0	19.5	0.09	0.04	7600	145	11.25	10091	2491	615	3106	0.33	0.020
19.5	20.0	0.09	0.04	7600	145	11.75	10164	2564	576	3139	0.30	0.018
20.0	20.5	0.09	0.04	7600	145	12.25	10236	2636	540	3176	0.28	0.017
20.5	21.0	0.09	0.04	7600	145	12.75	10309	2709	507	3215	0.25	0.015
21.0	21.5	0.09	0.04	7600	145	13.25	10381	2781	476	3258	0.23	0.014
21.5	22.0	0.09	0.04	7600	145	13.75	10454	2854	449	3302	0.22	0.013
22.0	22.5	0.09	0.04	7600	145	14.25	10526	2926	423	3349	0.20	0.012
22.5	23.0	0.09	0.04	7600	145	14.75	10599	2999	399	3398	0.18	0.011
23.0	23.5	0.09	0.04	7600	145	15.25	10671	3071	378	3449	0.17	0.010
23.5	24.0	0.09	0.04	7600	145	15.75	10744	3144	357	3501	0.16	0.010
24.0	24.5	0.09	0.04	7600	145	16.25	10816	3216	339	3555	0.15	0.009
24.5	25.0	0.09	0.04	7600	145	16.75	10889	3289	321	3610	0.14	0.008
25.0	25.5	0.09	0.04	7600	145	17.25	10961	3361	305	3667	0.13	0.008
25.5	26.0	0.09	0.04	7600	145	17.75	11034	3434	290	3724	0.12	0.007
26.0	26.5	0.09	0.04	7600	145	18.25	11106	3506	276	3783	0.11	0.007
26.5	27.0	0.09	0.04	7600	145	18.75	11179	3579	263	3842	0.10	0.006

Appendix 3.3: Abutment and Wingwall Design

All relevant information pertaining to the abutments and wingwalls are displayed in this appendix. The key components of the abutment design are broken down into components and discussed below.

<u>North SB Boring Logs</u>

The boring logs obtained from the NHDOT for the Northern portion of the bridge are shown. The site layout and locations of the boring logs are shown in the methodology section of the abutment design (3.9.5).

<u>South SB Boring Logs</u>

The boring logs for the Southern portion of the bridge are shown. These boring logs were done by the same company at the same time as the Northern boring logs but are separated for organizational purposes.

<u>Northern Soil Profile</u>

The soil profiles including the groundwater tables and elevations for each of the boring logs along Northern portion of the bridge are displayed. These profiles were then combined to make a conservative approximation for the representative soil profile which is also displayed.

<u>Southern Soil Profile</u>

This section contains the same information for soil profiles, except it is for the Southern portion.

<u>Abutment Design</u>

The MathCad sheet used to design the abutments in accordance with the *AASHTO LRFD Specifications* is displayed. This file was created by the New York State Department of Transportation and other individuals noted on the cover page.

<u>Wingwall Design</u>

This MathCad sheet provided by the New York State Department of Transportation was used in the same manner as the abutment design and is displayed below.

North SB Boring Logs

WALL "J" TOP OF LEVELING PAD
EL = 152.5 (AT END OF WALL)

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME: SALEM-MANCHESTER 13933E
BRIDGE NO.: 067/077
DESCRIPTION: I-93 SB over Route 97 (Pelham Road)

BORING NO.	B13-03
SHEET NO.	1 OF 2
STA.	3159+45
OFF.	LT 66
BASELINE	I-93 SB Mainline CL
ELEVATION (ft)	153.3
START/END	6/14/04 / 6/15/04
DRILLER	J. Kibbee (NHDOT)
INSPECTOR	Scott Myers
CLASSIFIER	SFM
EAST/NORTH (ft)	1097382/101369

GROUNDWATER

DATE	TIME	DEPTH (ft)	ELEV. (ft)	BOTTOM OF CASING	BOTTOM OF HOLE
6/15/04	8:00 am	17.2	136.1	26	30.2

EQUIPMENT	SAMPLER	CASING	CORE
TYPE:	S	NW	NX
SIZE I.D. (in):	1.375	3	1.875
HAMMER WT. (lb):	140	DRILL RIG	
HAMMER FALL (in):	30	CME 45-C Track rig	
HAMMER TYPE:	Automatic		

DEPTH (ft)	STRATUM CHANGE (ft) DEPTH	STRATUM CHANGE (ft) ELEVATION	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS
0	1.0	152.3	1 / 3 / 4 / 6	S1	1.3 [65]	0.0 - 2.0	Very dark grayish brown to dark brown, LOAMY TOPSOIL Olive, fine sandy SILT, trace gravel, trace coarse - medium sand, cobbles possible -FILL-
	4.0	149.3					
5			87 / 23 / 8 / 6	S2	0.1 [5]	5.0 - 7.0	Dense, very poor recovery of an apparent upper sandy till with stoniness (cobbles and possibly small boulder) included. this stone factor blocked the shoe of the spoon from receiving more soil to observe -GLACIAL TILL-
			14 / 11 / 11 / 12	S3	1.3 [65]	8.0 - 10.0	Medium dense, pale olive, silty FINE SAND, trace gravel and coarse - medium sand, cobbles likely over olive gray, SILT, some - little fine sand, trace gravel, trace coarse - medium sand, cobbles likely
10			7 / 7 / 7 / 9	S4	1.2 [60]	10.0 - 12.0	Medium dense, olive, similar to low end of S3 with trace clay and yellow brown stains
15			7 / 8 / 14 / 16	S5	1.2 [60]	15.0 - 17.0	Medium dense, dark greenish gray, SILT, little fine sand, trace gravel, trace coarse - medium sand, trace clay, cobbles possible, w/ pocket of light yellow brown, FINE SAND, some silt, trace coarse - medium sand, occasional trace gravel
20			10 / 11 / 11 / 14	S6	1.5 [75]	20.0 - 22.0	Medium dense, dark greenish gray, SILT, little fine sand, strong trace clay, trace gravel, trace coarse - medium sand, cobbles possible w/ one 1" zone of SILT, little clay and SILT (sorted) inclusion
25			50/0.4	S7	0.4 [100]	25.0 - 25.4	Very dense, similar to S6
							Advanced through this portion of the profile with diamond coring tools indicating dark greenish gray, SILT, little clay, trace gravel, trace coarse - medium sand, cobbles and boulders evident

Sampler	Identification
S	Standard Split Spoon
SL	Large Spoon (O.D.= 3 in)
T	Thin Wall Tube
U	Undisturbed Piston
O	Open End Rod
A	Auger Flight
C	Core Barrel
NR	Not Recorded

COHESIVE SOILS

Blows/foot	Consistency
0 - 1	Very Soft
2 - 4	Soft
5 - 8	Medium Stiff
9 - 15	Stiff
16 - 30	Very Stiff
31 - 60	Hard
> 60	Very Hard

NON-COHESIVE SOILS

Blows/foot	Density
0 - 4	Very Loose
5 - 10	Loose
11 - 24	Medium Dense
25 - 50	Dense
> 50	Very Dense

WOR - Weight of Rod
WOH - Weight of Hammer

Soil Descriptions	Proportion
Capitalized Soil Name	Major Component
Lower Case Adjective	35% - 50%
Some	20% - 35%
Little	10% - 20%
Trace	1% - 10%

ENGLISH

TB-06 S:\GINTW\PROJECTS\SALEM TO MANCHESTER (10418-13933)\BRIDGES\BRIDGE 13-14 (I 93 NB SB OVER PELHAM RD)\BR 13 (I-93 SB OVER ROUTE 97 PELHAM RD)\B13-01 TO B13-06 FINAL.GPJ 12/22/2008 9:51:23 AM TB-06

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME SALEM-MANCHESTER 13933E BRIDGE NO. 067/077
DESCRIPTION I-93 SB over Route 97 (Pelham Road)

BORING NO. B13-03
SHEET NO. 2 OF 2
STA. 3159+45 OFF. LT 66
BASELINE I-93 SB Mainline CL
ELEVATION (ft) 153.3

DEPTH (ft)	STRATUM CHANGE (ft) DEPTH	STRATUM CHANGE (ft) ELEVATION	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS	STRATUM SYMBOL
30							-GLACIAL TILL-	
			7 9 13 15	S8	1.7 [85]	30.2 - 32.2	Medium dense, similar to soil above this sample depth, w/ little - trace fine sand, cobbles likely	
	34.0	119.3					-APPROXIMATE BEDROCK SURFACE-	
35				C1	4.8 [96]	34.5 - 39.3	Very hard, slightly to very slightly weathered, sound, gray and greenish gray(w/ yellow brown stained rock first 1.4' of recovery), medium grained, GRANITE, w/ few pegmatitic intrusions, w/ few garnets RQD = 4.3/4.8 = 90%	
40							Bottom of Exploration @ 39.3 ft (El. 114.0)	
45								
50								
55								
60								
65								

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME: SALEM-MANCHESTER 13933E   BRIDGE NO. 067/077
DESCRIPTION: I-93 SB over Route 97 (Pelham Road)

BORING NO. B13-04
SHEET NO. 1 OF 2
STA. 3159+55 OFF. RT 36
BASELINE: I-93 SB Mainline CL
ELEVATION (ft): 166.9
START/END: 6/3/04 / 6/7/04
DRILLER: J. Kibbee (NHDOT)
INSPECTOR: Scott Myers
CLASSIFIER: SFM
EAST/NORTH (ft): 1097445/101429

GROUNDWATER					
DATE	TIME	DEPTH (ft)	ELEV. (ft)	BOTTOM OF CASING	BOTTOM OF HOLE
6/7/04	6:00 am	15.3	151.6	24.8	20.8

EQUIPMENT	SAMPLER	CASING	CORE
TYPE:	S	NW	NX
SIZE I.D. (in):	1.375	3	1.875
HAMMER WT. (lb):	140	DRILL RIG	
HAMMER FALL (in):	30	CME 45-C Track rig	
HAMMER TYPE:	Automatic		

DEPTH (ft)	STRATUM CHANGE (ft) DEPTH	STRATUM CHANGE (ft) ELEVATION	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS
0	0.7	166.2	1 1 4 11	S1	1.1 [55]	0.0 - 2.0	Very dark brown, / very dark grayish brown., ORGANIC to LOAMY TOPSOIL Light yellow brown w/ light brownish gray, fine sandy SILT, trace gravel, trace coarse - medium sand, cobbles likely -FILL-
5			12 10 8 6	S2	1.0 [50]	5.0 - 7.0	Medium dense, pale olive and light yellow brown, silty FINE SAND, strong trace gravel, trace coarse - medium sand, cobbles possible
10			6 5 10 22	S3	0.6 [30]	10.0 - 12.0	Medium dense, similar to S2 w/ cobbles evident
	13.5	153.4	22 45 57 8	S4	0.7 [35]	12.0 - 14.0	Very dense, light brownish gray, silty FINE SAND, trace gravel, trace coarse - medium sand, w/ cobbles Black and very dark grayish brown, SILTY MUCK, little - trace organic matter, little fine sand, trace gravel and cobbles pressed into old ground surface
15	16.6	150.3	10 6 21 23	S5	1.0 [50]	15.5 - 17.5	Black to very dark brown, SILT, little fine sand, little - trace organic matter, trace gravel w/ small lenses of gray FINE SAND, occasional small pocket of yellow brown, MEDIUM - FINE SAND Gray, sorted silty FINE SAND, w/ cobbles evident and possibly small boulders Stoniness blocked out most of the recovery form this lower soil -GLACIAL TILL- Advanced exploration through upper till soil w/ cobbles evident and boulders possible
20			10 16 17 30	S6	0.2 [10]	20.0 - 22.0	Dense, olive gray, silty FINE SAND, little - trace gravel, little - trace coarse - medium sand, w/ cobbles likely and boulders possible
	23.8	143.1					-APPROXIMATE BEDROCK SURFACE-
25				C1	4.8 [100]	25.0	Hard to very hard, very slightly weathered to fresh, slightly fractured to sound (w/ occasionally moderately fractured zone), gray w/ yellow, medium grained, GRANITE, w/ several pegmatitic intrusions, low angle fractures *RQD combined w/ C2

WALL "K" TOP OF LEVELING PAD
EL = 147.50

Sampler	Identification
S	Standard Split Spoon
SL	Large Spoon (O.D.= 3 in)
T	Thin Wall Tube
U	Undisturbed Piston
O	Open End Rod
A	Auger Flight
C	Core Barrel
NR	Not Recorded

COHESIVE SOILS

Blows/foot	Consistency
0 - 1	Very Soft
2 - 4	Soft
5 - 8	Medium Stiff
9 - 15	Stiff
16 - 30	Very Stiff
31 - 60	Hard
> 60	Very Hard

NON-COHESIVE SOILS

Blows/foot	Density
0 - 4	Very Loose
5 - 10	Loose
11 - 24	Medium Dense
25 - 50	Dense
> 50	Very Dense

WOR - Weight of Rod
WOH - Weight of Hammer

Soil Descriptions	Proportion
Capitalized Soil Name	Major Component
Lower Case Adjective	35% - 50%
Some	20% - 35%
Little	10% - 20%
Trace	1% - 10%

ENGLISH

TB-08 S:\GINTW\PROJECTS\SALEM TO MANCHESTER (10418-13933)\BRIDGES\BRIDGE 13-14 (I-93 NB SB OVER PELHAM RD)\BR 13 (I-93 SB OVER ROUTE 97 PELHAM RD)\B13-01 TO B13-06 FINAL.GPJ 12/22/2008 9:51:38 AM TB-08

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME: SALEM-MANCHESTER 13933E — BRIDGE NO. 067/077
DESCRIPTION: I-93 SB over Route 97 (Pelham Road)

BORING NO. B13-06
SHEET NO. 1 OF 2
STA. 3160+24 OFF. RT 42
BASELINE: I-93 SB Mainline CL
ELEVATION (ft): 161.4
START/END: 8/24/08 / 8/25/08
DRILLER: J. Woodward (NHDOT)
INSPECTOR: Edward Birckhead
CLASSIFIER: EFB
EAST/NORTH (ft): 1097415/101492

GROUNDWATER

DATE	TIME	DEPTH (ft)	ELEV. (ft)	BOTTOM OF CASING	BOTTOM OF HOLE
8/25/06	7:30 am	9.7	151.7	20	22

EQUIPMENT

	SAMPLER	CASING	CORE
TYPE:	S	NW	NX
SIZE I.D. (in):	1.375	3	1.875
HAMMER WT. (lb):	140		
HAMMER FALL (in):	30		
HAMMER TYPE:	Automatic		

DRILL RIG: CME 45-C Track rig

DEPTH (ft)	STRATUM CHANGE DEPTH (ft)	STRATUM CHANGE ELEVATION	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS
0			1, 1, 2, 2	S1	1.1 [55]	0.0 – 2.0	Loose, grey to brown, silty FINE SAND, with thin layer organic silt, trace sand at 1.5 feet
							-FILL-
5			6, 8, 14, 14	S2	1.5 [75]	6.0 – 7.0	Medium dense, brown, silty FINE SAND, little gravel
	7.0	154.4					-GLACIAL TILL-
10			8, 12, 14, 16	S3	1.3 [65]	10.0 – 12.0	Dense, grey, silty FINE SAND, little clay, little gravel
15			10, 16, 14, 17	S4	2.0 [100]	15.0 – 17.0	Dense, grey, silty FINE SAND, little clay, little gravel
20			3, 8, 11, 18	S5	2.0 [100]	20.0 – 22.0	Medium dense, grey, silty FINE SAND, little clay, little gravel
25			11, 12, 11, 80/0.4	S6	2.0 [105]	25.0 – 26.9	Medium dense, grey, silty FINE SAND, little clay, little gravel
	26.9	134.5				28.0	-APPROXIMATE BEDROCK SURFACE-

Sampler	Identification
S	Standard Split Spoon
SL	Large Spoon (O.D.= 3 in)
T	Thin Wall Tube
U	Undisturbed Piston
O	Open End Rod
A	Auger Flight
C	Core Barrel
NR	Not Recorded

COHESIVE SOILS

Blows/foot	Consistency
0 - 1	Very Soft
2 - 4	Soft
5 - 8	Medium Stiff
9 - 15	Stiff
16 - 30	Very Stiff
31 - 60	Hard
> 60	Very Hard

NON-COHESIVE SOILS

Blows/foot	Density
0 - 4	Very Loose
5 - 10	Loose
11 - 24	Medium Dense
25 - 50	Dense
> 50	Very Dense

WOR - Weight of Rod
WOH - Weight of Hammer

Soil Descriptions	Proportion
Capitalized Soil Name	Major Component
Lower Case Adjective	35% - 50%
Some	20% - 35%
Little	10% - 20%
Trace	1% - 10%

ENGLISH

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME SALEM-MANCHESTER 13933E — BRIDGE NO. 067/077
DESCRIPTION I-93 SB over Route 97 (Pelham Road)

BORING NO. B13-06
SHEET NO. 2 OF 2
STA. 3160+24 OFF. RT 42
BASELINE I-93 SB Mainline CL
ELEVATION (ft) 161.4

DEPTH (ft)	STRATUM CHANGE (ft) DEPTH	STRATUM CHANGE (ft) ELEVATION	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS	STRATUM SYMBOL
30				C1	4.8 [100]	32.8	Moderately hard to hard, slightly weathered, slightly fractured, coarse grained, light grey GRANITE, fractures shallow, some oxidation on planes RQD: 4.6 / 4.8 = 96%	
35				C2	4.8 [100]	32.8 37.6	Moderately hard to hard, slightly weathered, slightly fractured, coarse grained, light grey GRANITE, fractures shallow to steep, some oxidation on planes RQD: 4.0 / 4.8 = 83%	
							Bottom of Exploration @ 37.6 ft (El. 123.8)	
40								
45								
50								
55								
60								
65								

South SB Boring Logs

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME: SALEM-MANCHESTER 13933E   BRIDGE NO. 067/077
DESCRIPTION: I-93 SB over Route 97 (Pelham Road)

BORING NO.	B13-01
SHEET NO.	1 OF 1
STA.	3158+24 OFF. RT 39
BASELINE	I-93 SB Mainline CL
ELEVATION (ft)	162.0
START/END	6/8/04 / 6/9/04
DRILLER	J. Kibbee (NHDOT)
INSPECTOR	Scott Myers
CLASSIFIER	SFM
EAST/NORTH (ft)	1097512/101315

GROUNDWATER

DATE	TIME	DEPTH (ft)	ELEV. (ft)	BOTTOM OF CASING	BOTTOM OF HOLE
6/9/04	8:00 am	Dry		10	12

EQUIPMENT	SAMPLER	CASING	CORE
TYPE:	S	NW	NX
SIZE I.D. (in):	1.375	3	1.875
HAMMER WT. (lb):	140	DRILL RIG	
HAMMER FALL (in):	30	CME 45-C Track rig	
HAMMER TYPE:	Automatic		

DEPTH (ft)	STRATUM CHANGE DEPTH (ft)	STRATUM CHANGE ELEVATION (ft)	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS
0	0.7	161.3	1 2 4 7	S1	0.8 [40]	0.0 – 2.0	Very dark grayish brown to brown, LOAMY TOPSOIL Light olive brown and light yellow brown, fine sandy SILT, trace gravel, trace coarse - medium sand, cobbles likely -FILL-
5			10 8 6 10	S2	0.2 [10]	5.0 – 7.0	Medium dense, silty FINE SAND, strong trace gravel, trace coarse - medium sand, cobbles likely
			11 7 8 7	S3	0.4 [20]	7.0 – 9.0	Medium dense, similar to S2
10			13 10 11 7	S4	0.5 [25]	10.0 – 12.0	Medium dense, light yellow brown, FINE SAND, some silt to silty FINE SAND, little - trace gravel, trace coarse - medium sand, cobbles likely
	11.7	150.3					Dark grayish brown, SILT, little fine sand, trace organic matter, trace coarse - medium sand and gravel -REMNANT TOPSOIL-
	12.5	149.5					-APPROXIMATE BEDROCK SURFACE-
15				C1	4.6 [96]	12.9 – 17.7	Very hard, moderately weathered, extremely - moderately fractured (from 12.9 to 15.1) then sound to slightly fractured, yellow brown, coarse- medium grained, GRANITE, w/ pegmatitic intrusions, w/ some minor pitting, w/ soil coated joints, w/ one moderately dipping fracture, w/ few garnets evident RQD = 2.5/4.8 = 52%
							Bottom of Exploration @ 17.7 ft (El. 144.3)
20							
25							

WALL "H" TOP OF LEVELING PAD
EL = 149.50

TB-06 S:\GINT\PROJECTS\SALEM TO MANCHESTER (10418-13933)\BRIDGES\BRIDGE 13-14 (I-93 NB SB OVER PELHAM RD)\BR 13 (I-93 SB OVER ROUTE 97 PELHAM RD)\B13-01 TO B13-06 FINAL.GPJ 12/22/2008 9:52:29 AM TB-06

Sampler	Identification
S	Standard Split Spoon
SL	Large Spoon (O.D.= 3 in)
T	Thin Wall Tube
U	Undisturbed Piston
O	Open End Rod
A	Auger Flight
C	Core Barrel
NR	Not Recorded

COHESIVE SOILS

Blows/foot	Consistency
0 - 1	Very Soft
2 - 4	Soft
5 - 8	Medium Stiff
9 - 15	Stiff
16 - 30	Very Stiff
31 - 60	Hard
> 60	Very Hard

NON-COHESIVE SOILS

Blows/foot	Density
0 - 4	Very Loose
5 - 10	Loose
11 - 24	Medium Dense
25 - 50	Dense
> 50	Very Dense

WOR - Weight of Rod
WOH - Weight of Hammer

Soil Descriptions	Proportion
Capitalized Soil Name	Major Component
Lower Case Adjective	35% - 50%
Some	20% - 35%
Little	10% - 20%
Trace	1% - 10%

ENGLISH

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME SALEM-MANCHESTER 13933E BRIDGE NO. 067/077
DESCRIPTION I-93 SB over Route 97 (Pelham Road)

BORING NO.	B13-02
SHEET NO.	1 OF 2
STA.	3158+01 OFF. LT 60
BASELINE	I-93 SB Mainline CL
ELEVATION (ft)	150.4
START/END	6/15/04 / 6/16/04
DRILLER	J. Woodward (NHDOT)
INSPECTOR	Scott Myers
CLASSIFIER	SFM
EAST/NORTH (ft)	1097435/101248

GROUNDWATER

DATE	TIME	DEPTH (ft)	ELEV. (ft)	BOTTOM OF CASING	BOTTOM OF HOLE
6/16/04	8:00 am	5.0	145.4	10	21

EQUIPMENT	SAMPLER	CASING	CORE
TYPE:	S	NW	NX
SIZE I.D. (in):	1.375	3	1.875
HAMMER WT. (lb):	140	DRILL RIG	
HAMMER FALL (in):	30		
HAMMER TYPE:	Automatic	CME 45-C Track rig	

DEPTH (ft)	STRATUM CHANGE DEPTH (ft)	STRATUM CHANGE ELEVATION (ft)	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS
0	0.5	149.9	2 4 6 6	S1	1.5 [75]	0.0 - 2.0	Very dark grayish brown to dark brown, LOAMY TOPSOIL Loose, olive, fine sandy SILT, trace gravel, trace coarse - medium sand -FILL-
	3.0	147.4					
5			14 16 12 30	S2	1.3 [65]	4.0 - 6.0	Dense, pale olive, fine sandy SILT, trace gravel, trace coarse - medium sand, cobbles evident, w/ occasional yellow brown stains -GLACIAL TILL-
10			15 24 75/0.4	S3	1.3 [93]	9.0 - 10.4	Very dense, olive gray to dark greenish gray, SILT, little fine sand, trace clay, trace gravel, trace coarse - medium sand, cobbles likely, boulders possible
15			13 20 20 20	S4	1.6 [80]	14.0 - 16.0	Dense, dark greenish gray, SILT, little clay, trace gravel, trace coarse - medium sand, cobbles and boulders possible
20			12 10 8 10	S5	1.6 [80]	19.0 - 21.0	Medium dense, similar to S4 w/ stronger little clay
25			9 17 75	S6	1.3 [87]	24.0 - 25.5	Very dense, Similar to S4
	27.6	122.8				28.0	-APPROXIMATE BEDROCK SURFACE-

Sampler	Identification
S	Standard Split Spoon
SL	Large Spoon (O.D.= 3 in)
T	Thin Wall Tube
U	Undisturbed Piston
O	Open End Rod
A	Auger Flight
C	Core Barrel
NR	Not Recorded

COHESIVE SOILS

Blows/foot	Consistency
0 - 1	Very Soft
2 - 4	Soft
5 - 8	Medium Stiff
9 - 15	Stiff
16 - 30	Very Stiff
31 - 60	Hard
> 60	Very Hard

NON-COHESIVE SOILS

Blows/foot	Density
0 - 4	Very Loose
5 - 10	Loose
11 - 24	Medium Dense
25 - 50	Dense
> 50	Very Dense

WOR - Weight of Rod
WOH - Weight of Hammer

Soil Descriptions	Proportion
Capitalized Soil Name	Major Component
Lower Case Adjective	35% - 50%
Some	20% - 35%
Little	10% - 20%
Trace	1% - 10%

ENGLISH

TB-06 S:\GINT\PROJECTS\SALEM TO MANCHESTER (10418-13933)\BRIDGES\BRIDGE 13-14 (I-93 NB SB OVER PELHAM RD)\BR 13 (I-93 SB OVER ROUTE 97 PELHAM RD)\B13-01 TO B13-06 FINAL.GPJ 12/22/2006 9:51:08 AM TB-06

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME SALEM-MANCHESTER 13933E BRIDGE NO. 067/077
DESCRIPTION I-93 SB over Route 97 (Pelham Road)

BORING NO. B13-02
SHEET NO. 2 OF 2
STA. 3158+01 OFF. LT 60
BASELINE I-93 SB Mainline CL
ELEVATION (ft) 150.4

DEPTH (ft)	STRATUM CHANGE (ft) DEPTH	STRATUM CHANGE (ft) ELEVATION	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS	STRATUM SYMBOL
30				C1	4.7 [98]	32.8	Very hard, fresh, sound, gray, coarse to very coarse(w/ occasional medium grained zone), GRANITE, w/ near horizontal joints, w/ few garnets RQD = 4.7/4.8 = 98%	
							Bottom of Exploration @ 32.8 ft (El. 117.6)	
35								
40								
45								
50								
55								
60								
65								

TB-08 S:\GINTW\PROJECTS\SALEM TO MANCHESTER (10418-13933)\BRIDGES\BRIDGE 13-14 (I-93 NB SB OVER PELHAM RD)\BR 13 (I-93 SB OVER ROUTE 97 PELHAM RD)\B13-01 TO B13-06 FINAL.GPJ 12/22/2008 9:51:09 AM TB-08

TEST BORING REPORT

STATE OF NEW HAMPSHIRE DEPARTMENT OF TRANSPORTATION
MATERIALS & RESEARCH BUREAU - GEOTECHNICAL SECTION

New Hampshire DOT Department of Transportation

PROJECT NAME SALEM-MANCHESTER 13933E BRIDGE NO. 067/077
DESCRIPTION I-93 SB over Route 97 (Pelham Road)

BORING NO. B13-05
SHEET NO. 1 OF 1
STA. 3157+71 OFF. RT 41
BASELINE I-93 SB Mainline CL
ELEVATION (ft) 162.5
START/END 9/10/08 / 9/10/08
DRILLER J. Woodward (NHDOT)
INSPECTOR Edward Birckhead
CLASSIFIER EFB
EAST/NORTH (ft) 1097539/101266

WALL "C" TOP OF LEVELING PAD
EL = 159.50 (AT END OF WALL)

GROUNDWATER						EQUIPMENT	SAMPLER	CASING	CORE
DATE	TIME	DEPTH (ft)	ELEV. (ft)	BOTTOM OF CASING	BOTTOM OF HOLE	TYPE:	S	NW	NX
9/11/08	7:30 am	7.3	155.2	5	19.8	SIZE I.D. (in):	1.375	3	1.875
						HAMMER WT. (lb):	140	DRILL RIG	
						HAMMER FALL (in):	30	CME 45-C Track rig	
						HAMMER TYPE:	Automatic		

DEPTH (ft)	STRATUM CHANGE (ft) DEPTH	ELEVATION	BLOWS PER 0.5 ft	SAMPLE NUMBER	SAMPLER RECOVERY (ft) [%]	DEPTH RANGE (ft)	FIELD CLASSIFICATION AND REMARKS
0			WOH, WOH, 2, 3	S1	1.3 [65]	0.0 – 2.0	Very loose, light brown, silty FINE SAND, trace gravel -FILL-
5	5.0	157.5	7, 10, 8, 5	S2	1.5 [75]	5.0 – 7.0	Medium dense, brown, silty FINE SAND, trace gravel -GLACIAL TILL- casing refusal with spin shoe at 8.5', boring continued with tri-cone to 10'
	8.5	154.0					-APPROXIMATE BEDROCK SURFACE-
10				C1	4.8 [100]	10.0 – 14.8	Hard, slightly weathered to fresh, sound, medium to coarse grained, GRANITE RQD: 4.8 / 4.8 = 100%
15				C2	4.1 [85]	14.8 – 19.6	Hard, slightly weathered to fresh, sound, medium to coarse grained, GRANITE, weathered and fractured zone 17.6' to 18.4' RQD: 3.1 / 4.8 = 65%
20							Bottom of Exploration @ 19.8 ft (El. 142.9)
25							

Sampler	Identification
S	Standard Split Spoon
SL	Large Spoon (O.D.= 3 in)
T	Thin Wall Tube
U	Undisturbed Piston
O	Open End Rod
A	Auger Flight
C	Core Barrel
NR	Not Recorded

COHESIVE SOILS

Blows/foot	Consistency
0 - 1	Very Soft
2 - 4	Soft
5 - 8	Medium Stiff
9 - 15	Stiff
16 - 30	Very Stiff
31 - 60	Hard
> 60	Very Hard

NON-COHESIVE SOILS

Blows/foot	Density
0 - 4	Very Loose
5 - 10	Loose
11 - 24	Medium Dense
25 - 50	Dense
> 50	Very Dense

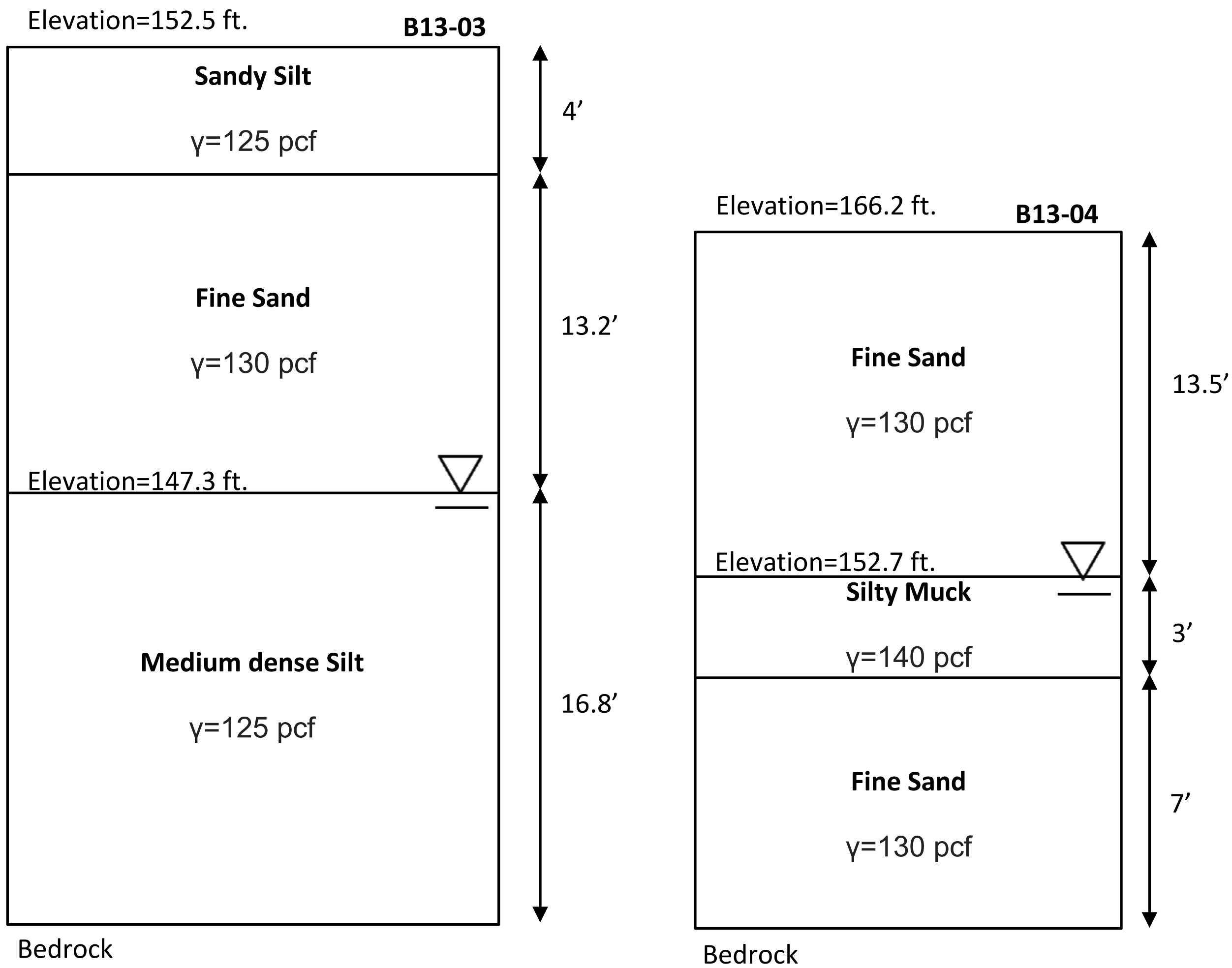
WOR - Weight of Rod
WOH - Weight of Hammer

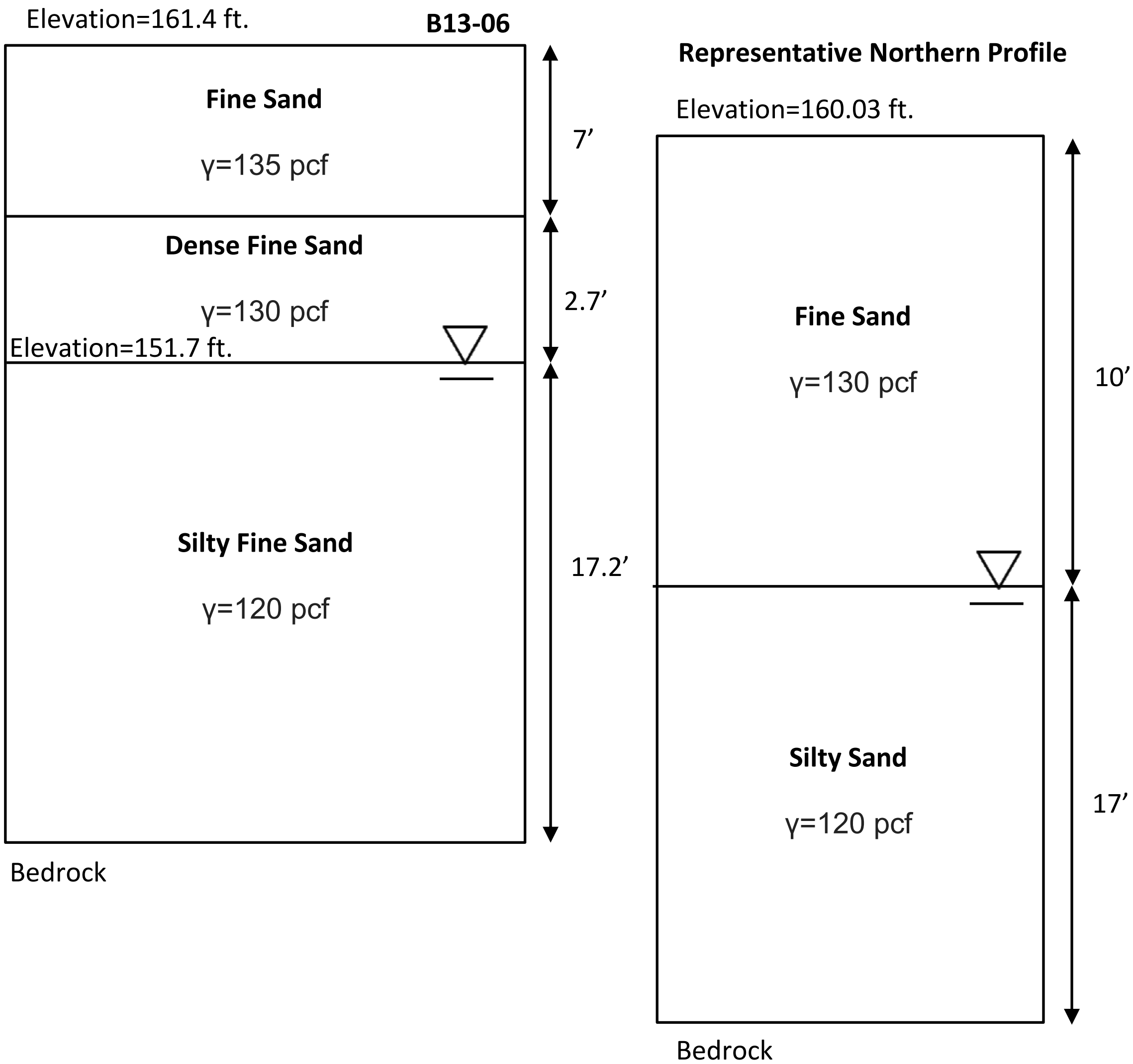
Soil Descriptions	Proportion
Capitalized Soil Name	Major Component
Lower Case Adjective	35% - 50%
Some	20% - 35%
Little	10% - 20%
Trace	1% - 10%

ENGLISH

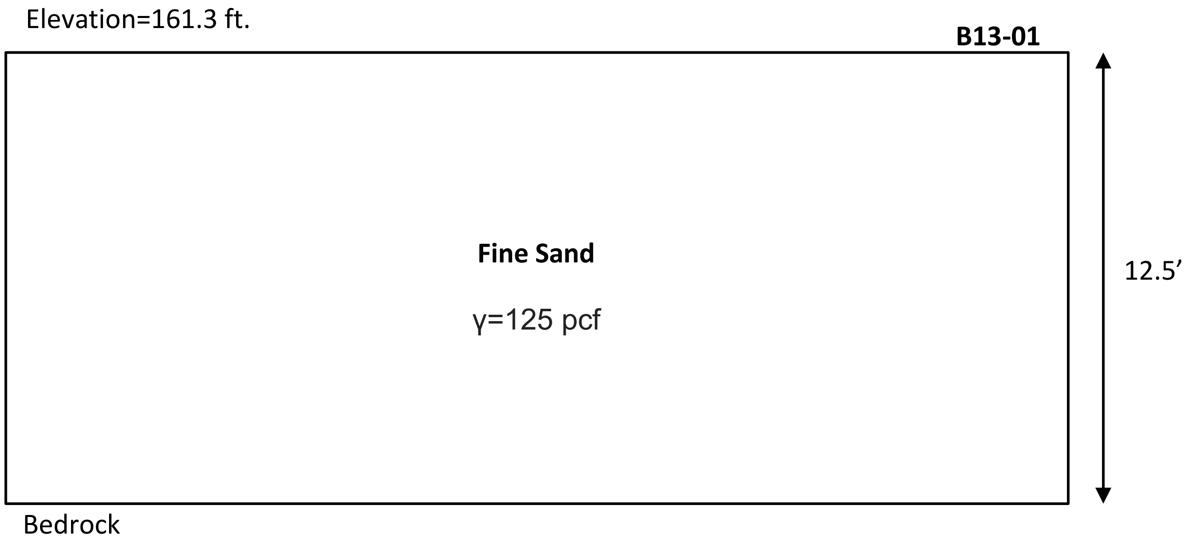
TB-08 S:\O\NTWPROJECTS\SALEM TO MANCHESTER (10418-13933)\BRIDGES\BRIDGE 13-14 (I-93 NB SB OVER PELHAM RD)\BR 13 (I-93 SB OVER ROUTE 97 PELHAM RD)\B13-01 TO B13-05 FINAL.GPJ 12/22/2008 9:51:51 AM TB-08

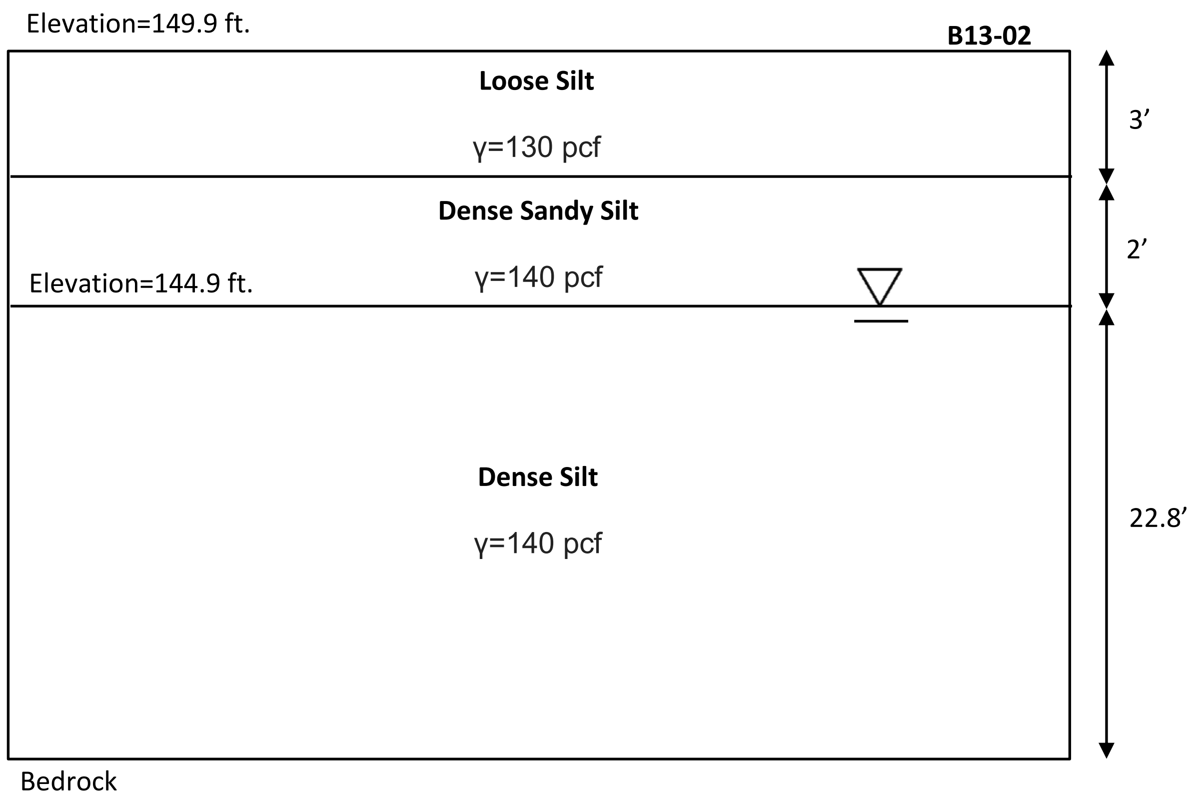
Representative Soil Profiles from Northern Boring Logs

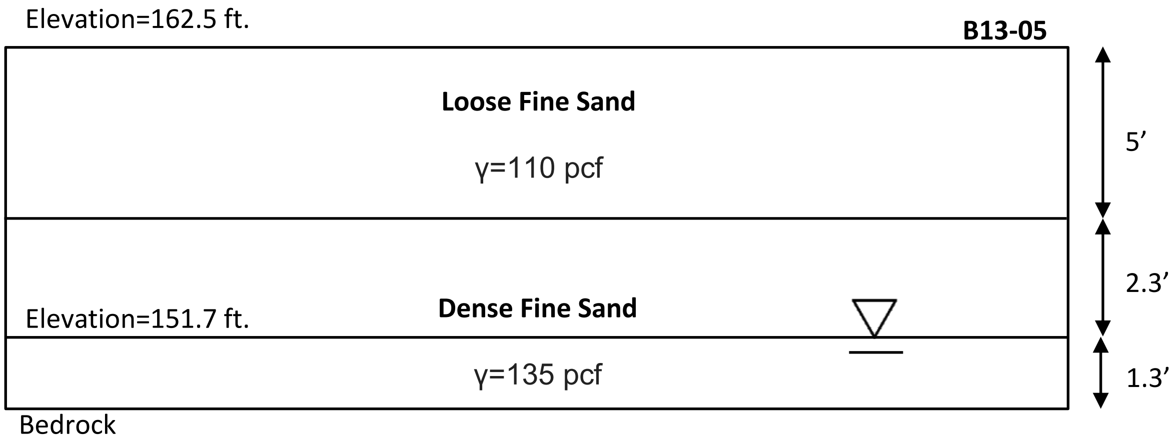


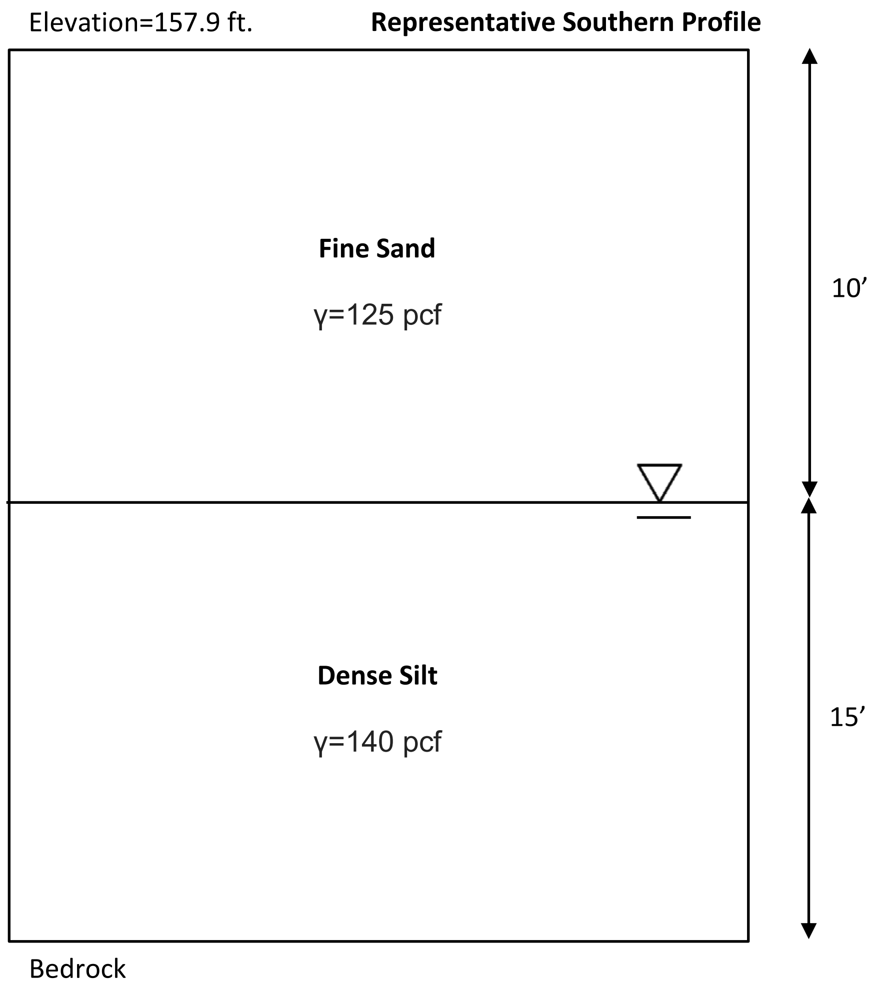

Elevation=161.4 ft.
B13-06
Fine Sand
γ=135 pcf
7’
Dense Fine Sand
γ=130 pcf
2.7’
Elevation=151.7 ft.
Silty Fine Sand
γ=120 pcf
17.2’
Bedrock
Representative Northern Profile
Elevation=160.03 ft.
Fine Sand
γ=130 pcf
10’
Silty Sand
γ=120 pcf
17’
Bedrock

Representative Soil Profiles from Southern Boring Logs






Elevation=162.5 ft.
B13-05
Loose Fine Sand
γ=110 pcf
5’
2.3’
Elevation=151.7 ft.
Dense Fine Sand
γ=135 pcf
1.3’
Bedrock


Elevation=157.9 ft.
Representative Southern Profile
Fine Sand
γ=125 pcf
10’
Dense Silt
γ=140 pcf
15’
Bedrock

NYSDOT
CANTILEVER ABUTMENT ANALYSIS
SPREAD FOOTING ON SOIL
AASHTO LRFD - FOURTH EDITION 2007
(2009 INTERIM)

Disclaimer

Disclaimer:

If you have any questions or comments contact Md. Ratan of the office of Structures : mratan@dot.state.ny.us

Disclaimer

About this worksheet

Created by : **Md Ratan, PE, Office of Structures.**
Acknowledgement : **Arthur Yannotti, Brian Edinger, Denise Carman, Khelifa Abdurahman, Ranjit Singh, Rohit Dagli, Scott VanSlyke, Shamim Hydery, Sonjoy Sikder, Stephanie Winkelhake,Troy Soka, William LeBlanc and Wahid Albert; Office of Structures.**
Paul Bailey; Geotechnical Engineering Bureau.
Version 1 : **June 2008, AASHTO LRFD 2007.**
Version 2 : **June 2009, AASHTO LRFD 2007 with 2008 Interim.**
Version 3 : **July 2010, AASHTO LRFD 2007 with 2009 Interim.**

About this worksheet

JOB DESCRIPTION:

INTERSTATE 93 SB OVER NH 97

PIN: **BIN:**

ABUTMENT: **BEGINNING**

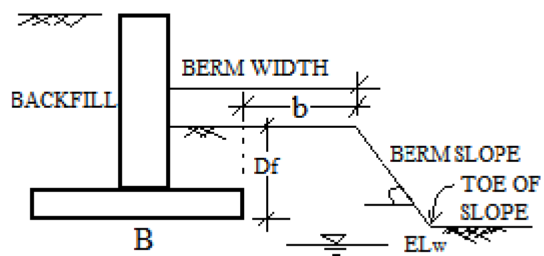
DESIGNED BY: **MQP**

CHECKED BY: **MQP**

Material and Geotechnical data inputs:

60	ksi	Yield strength of reinforcing bars
4	ksi	Compressive strength of concrete
2.5	in	Clear Cover of reinforcement in Backwall and Stem
3	in	Clear Cover of reinforcement in Footing
1		Exposure Factor. 1.0 for Class 1 Exposure & 0.75 for Class 2 Exposure. LRFD Art. 5.7.3.4 (2007)
6	ksf	Service bearing resistance of soil
0.8		Coefficient of sliding (friction)

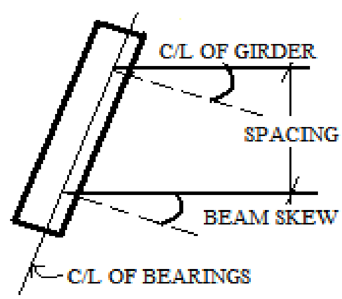
0.8		**Resistance factor for Sliding**
35	deg	**Internal angle of friction for backfill**
125	pcf	**Unit weight of backfill**
30	deg	**Internal angle of friction for soil under the footing**
125	pcf	**Unit weight of soil under the footing**
0.45		**Resistance Factor for bearing.** **LRFD T 10.5.5.2.2-1 (2007)**
0	ft	**Berm Width**
0	deg	**Berm Slope**



Note: Input Berm Width = 0 and Berm Slope = 0 for following conditions:
1.The ground surface in front of abutment is level.
2.There is a very long berm with a slope. (Roughly: b/B > 6, see Figure.)
3.There is a reasonable berm with slope but the bottom of footing is below the toe of slope.

147.9	ft	**Elevation of water table**

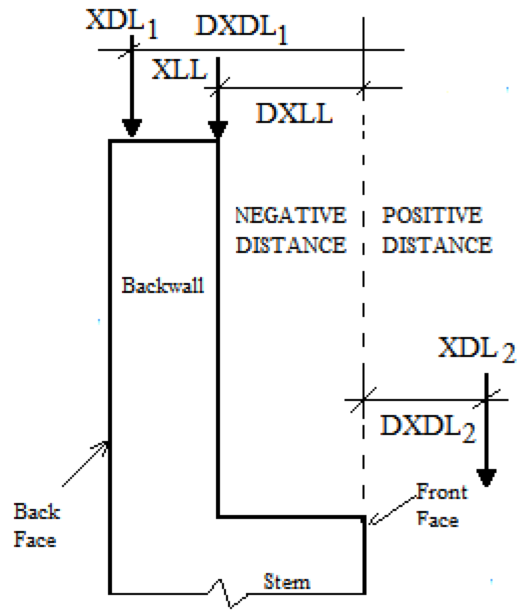
Superstructure data inputs:



5.75	ft	**Spacing of Girder**
0	deg	**Beam Skew**
47.8	kip	**Unfactored Girder Reaction due to DL+SDL**
7.8	kip	**Unfactored Girder Reaction due to DW (FWS+UTILITY)**
120	kip	**Maximum Unfactored Girder Reaction due to LL** **(without Impact and with a distribution factor of 1.0).**
9	in	**Distance from center line of bearings to the front face of abutment stem.**
0	kip	**Axial girder load due to temperature fall (TU). Expand the following region for calculation details.**
6		**Number of Design Lanes (the integer part of the ratio of the clear roadway width divided by 12 feet (3600 mm).**

Note: All extra loads are per unit length of abutment.

6.4	kip/ft	XDL_1 **Extra Dead Load 1**
-35	in	$DXDL_1$ **Distance of Extra Load 1 from the front face of abutment stem**
0	kip/ft	XDL_2 **Extra Dead Load 2**
0	in	$DXDL_2$ **Distance of Extra Load 2 from the front face of abutment stem**
2.69	kip/ft	XLL **Extra Live Load**
-35	in	DXLL **Distance of Extra Live Load from the front face of abutment stem**
50	%	**Percentage of Live Load Surcharge**



Distance is negative at left / positive at right of stem face

Note: For approach slabs which are supported at one edge by the backwall of an abutment, a corresponding reduction in the surcharge loads may be permitted. Ref: A 3.11.6.5 and C 3.11.6.5

Substructure Geometric Inputs:

174.55	ft	**Elevation of top of Back Wall**
171.63	ft	**Elevation of Bridge Seat**
147.63	ft	**Elevation at bottom of Footing**
1	ft	**Fill height over toe**
36	in	**Thickness of Footing**
10	ft	**Heel Width**
4	ft	**Toe Width**
18	in	**Thickness of Back Wall**
62	in	**Thickness of Stem**
75	ft	**Length of Abutment**

Input Program

Live Load Surcharge

Figure:

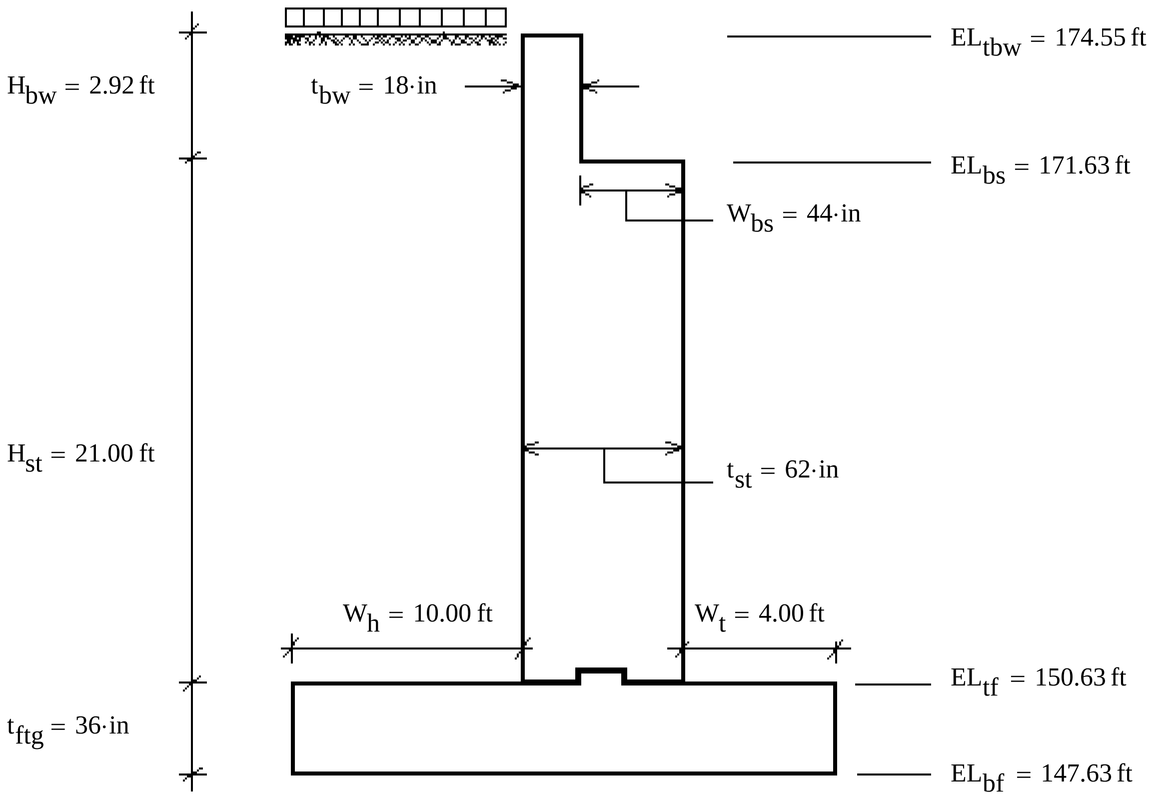


Figure:

Note: All loads and resistances are for <u>one foot</u> of abutment length.

Loads Calculation

Load Factors & Design Data

Analyze and Combine Force Effect

Rebar sizes

Temp. and Shrinkage Reinforcement in Backwall

programming for structural design of Backwall

Temp. and Shrinkage Reinforcement in Stem

Program for Reinforcement in Stem at Section (e)

Program for Reinforcement in Stem at Section (d)

Program for Reinforcement in Stem at Section (c)

Program for Reinforcement in Stem at Section (b)

Stem Shear Check

Reinforcement Graph

Temp & Shrinkage Reinforcement Graph

Overturning Check

Sliding check

Structural design of Footing

Bearing Resistance Calculation

Bearing resistance check

Programming for Reinforcement in bottom footing

Programming for Reinforcement in top footing

Shear design of footing

RESULTS:

LOAD FACTORS:

LF :=

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"
"BACKWALL DL"	0.900	1.250	1.250	1.000
"STEM DL"	0.900	1.250	1.250	1.000
"FOOTING DL"	0.900	1.250	1.250	1.000
"EXTRA DL1"	0.900	1.250	1.250	1.000
"EXTRA DL2"	0.900	1.250	1.250	1.000
"EXTRA LL"	0.000	1.750	0.000	1.000
"SUP.STR. DC"	0.900	1.250	0.000	1.000
"SUP. STR. DW"	0.650	1.500	0.000	1.000
"SUP. STR. LL"	0.000	1.750	0.000	1.000
"FILL AT HEEL (EV)"	1.000	1.350	1.250	1.000
"FILL AT TOE (EV)"	1.000	1.350	1.250	1.000
"VERT. LS AT HEEL"	0.000	1.750	1.75	1.000
"EARTH LOAD (EH)"	1.500	1.500	1.250	1.000
"HOR. LS"	1.750	1.750	1.75	1.000
"TEMP. LOAD (TU)"	0.500	0.500	0.000	1.000

Strength I-a: Min. vertical load and Max. lateral load (without sup. LL and vertical componant of LS)

Strength I-b: Max. vertical load and Max. lateral load

Strength I-c: Construction condition without superstructure

Service I: Unfactored load for bearing and serviceability check

EQUIVALENT HEIGHT OF LIVE LOAD SURCHARGE:

$EqH = 2\ ft$ **(Based on the height of wall. 100% is applied to Strength I-c only)**

FACTORED VERTICAL LOADS ON BACKWALL:

$VL_{bw} =$

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"BACKWALL DL"	0.59	0.82	0.82	0.66	"kip"
"EXTRA DL1"	5.76	8.00	8.00	6.40	"kip"
"EXTRA DL2"	0.00	0.00	0.00	0.00	"kip"
"EXTRA LL"	0.00	0.00	0.00	0.00	"kip"
"TOTAL"	6.35	8.82	8.82	7.06	"kip"

FACTORED HORIZONTAL LOADS ON BACKWALL:

HL_{bw} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"EARTH LOAD (EH)"	0.22	0.22	0.18	0.14	"kip"
"HOR. LS"	0.17	0.17	0.35	0.10	"kip"
"TEMP. LOAD (TU)"	0.00	0.00	0.00	0.00	"kip"
"TOTAL"	0.39	0.39	0.53	0.24	"kip"

FACTORED VERTICAL LOADS ON STEM:

VL_{st} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"BACKWALL DL"	0.59	0.82	0.82	0.66	"kip"
"STEM DL"	14.65	20.34	20.34	16.27	"kip"
"EXTRA DL1"	5.76	8.00	8.00	6.40	"kip"
"EXTRA DL2"	0.00	0.00	0.00	0.00	"kip"
"EXTRA LL"	0.00	4.71	0.00	2.69	"kip"
"SUP.STR. DC"	7.48	10.39	0.00	8.31	"kip"
"SUP. STR. DW"	0.88	2.03	0.00	1.36	"kip"
"SUP. STR. LL"	0.00	10.92	0.00	6.24	"kip"
"TOTAL"	29.36	57.22	29.16	41.93	"kip"

FACTORED HORIZONTAL LOADS ON STEM:

$HL_{st} =$

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"EARTH LOAD (EH)"	14.54	14.54	12.11	9.69	"kip"
"HOR. LS"	1.42	1.42	2.84	0.81	"kip"
"TEMP. LOAD (TU)"	0.00	0.00	0.00	0.00	"kip"
"TOTAL"	15.95	15.95	14.95	10.50	"kip"

FACTORED VERTICAL LOADS ON FOOTING:

$VL_{ftg} =$

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"BACKWALL DL"	0.59	0.82	0.82	0.66	"kip"
"STEM DL"	14.65	20.34	20.34	16.27	"kip"
"FOOTING DL"	7.76	10.78	10.78	8.62	"kip"
"EXTRA DL1"	5.76	8.00	8.00	6.40	"kip"
"EXTRA DL2"	0.00	0.00	0.00	0.00	"kip"
"EXTRA LL"	0.00	4.71	0.00	2.69	"kip"
"SUP.STR. DC"	7.48	10.39	0.00	8.31	"kip"
"SUP. STR. DW"	0.88	2.03	0.00	1.36	"kip"
"SUP. STR. LL"	0.00	10.92	0.00	6.24	"kip"
"FILL AT HEEL (EV)"	29.90	40.37	37.38	29.90	"kip"
"FILL AT TOE (EV)"	0.50	0.68	0.63	0.50	"kip"
"VERT. LS AT HEEL"	0.00	2.19	4.38	1.25	"kip"
"TOTAL"	67.52	111.23	82.32	82.21	"kip"

FACTORED HORIZONTAL LOADS ON FOOTING:

$HL_{ftg} =$

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"EARTH LOAD (EH)"	18.41	18.41	15.34	12.27	"kip"
"HOR. LS"	1.60	1.60	3.19	0.91	"kip"
"TEMP. LOAD (TU)"	0.00	0.00	0.00	0.00	"kip"
"TOTAL"	20.01	20.01	18.53	13.19	"kip"

MOMENT ARMS:

NOTE:ARM SIGNS ARE ACCORDING TO COORDINATE SYSTEMS

M_{arms} =

"*********"	"About Backwall CG (ft)."	"About Stem CG. (ft)"	"About Footing Toe (ft)"
"BACKWALL DL"	0.00	–1.83	–8.42
"STEM DL"	0.00	0.00	–6.58
"FOOTING DL"	0.00	0.00	–9.58
"EXTRA DL1"	1.50	–0.33	–6.92
"EXTRA DL2"	0.00	0.00	0.00
"EXTRA LL"	0.00	–0.33	–6.92
"SUP.STR. DC"	0.00	1.83	–4.75
"SUP. STR. DW"	0.00	1.83	–4.75
"SUP. STR. LL"	0.00	1.83	–4.75
"FILL AT HEEL (EV)"	0.00	0.00	–14.17
"FILL AT TOE (EV)"	0.00	0.00	–2.00
"VERT. LS AT HEEL"	0.00	0.00	–14.17
"EARTH LOAD (EH)"	0.97	7.97	8.97
"HOR. LS"	1.46	11.96	13.46
"TEMP. LOAD (TU)"	0.00	21.00	24.00

MOMENT ABOUT BACKWALL CENTER OF GRAVITY:

M_{bw} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"BACKWALL DL"	0.00	0.00	0.00	0.00	"kip-ft"
"EXTRA DL1"	8.64	12.00	12.00	9.60	"kip-ft"
"EXTRA DL2"	0.00	0.00	0.00	0.00	"kip-ft"
"EXTRA LL"	0.00	0.00	0.00	0.00	"kip-ft"
"EARTH LOAD (EH)"	0.21	0.21	0.18	0.14	"kip-ft"
"HOR. LS"	0.25	0.25	0.51	0.14	"kip-ft"
"TEMP. LOAD (TU)"	0.00	0.00	0.00	0.00	"kip-ft"
"TOTAL"	9.10	12.46	12.68	9.88	"kip-ft"

MOMENT ABOUT STEM CENTER OF GRAVITY:

M_{stem} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"BACKWALL DL"	–1.08	–1.51	–1.51	–1.20	"kip-ft"
"STEM DL"	0.00	0.00	0.00	0.00	"kip-ft"
"EXTRA DL1"	–1.92	–2.67	–2.67	–2.13	"kip-ft"
"EXTRA DL2"	0.00	0.00	0.00	0.00	"kip-ft"
"EXTRA LL"	0.00	–1.57	0.00	–0.90	"kip-ft"
"SUP.STR. DC"	13.72	19.05	0.00	15.24	"kip-ft"
"SUP. STR. DW"	1.62	3.73	0.00	2.49	"kip-ft"
"SUP. STR. LL"	0.00	20.02	0.00	11.44	"kip-ft"
"EARTH LOAD (EH)"	115.90	115.90	96.58	77.27	"kip-ft"
"HOR. LS"	16.96	16.96	33.92	9.69	"kip-ft"
"TEMP. LOAD (TU)"	0.00	0.00	0.00	0.00	"kip-ft"
"TOTAL"	145.19	169.92	126.33	111.89	"kip-ft"

MOMENT ABOUT FOOTING TOE:

M_{toe} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"BACKWALL DL"	–4.98	–6.91	–6.91	–5.53	"kip-ft"
"STEM DL"	–96.43	–133.93	–133.93	–107.14	"kip-ft"
"FOOTING DL"	–74.39	–103.32	–103.32	–82.66	"kip-ft"
"EXTRA DL1"	–39.84	–55.33	–55.33	–44.27	"kip-ft"
"EXTRA DL2"	0.00	0.00	0.00	0.00	"kip-ft"
"EXTRA LL"	0.00	–32.56	0.00	–18.61	"kip-ft"
"SUP.STR. DC"	–35.54	–49.36	0.00	–39.49	"kip-ft"
"SUP. STR. DW"	–4.19	–9.67	0.00	–6.44	"kip-ft"
"SUP. STR. LL"	0.00	–51.87	0.00	–29.64	"kip-ft"
"FILL AT HEEL (EV)"	–423.58	–571.84	–529.48	–423.58	"kip-ft"
"FILL AT TOE (EV)"	–1.00	–1.35	–1.25	–1.00	"kip-ft"
"VERT. LS AT HEEL"	0.00	–30.99	–61.98	–17.71	"kip-ft"
"EARTH LOAD (EH)"	165.21	165.21	137.67	110.14	"kip-ft"
"HOR. LS"	21.48	21.48	42.96	12.27	"kip-ft"
"TEMP. LOAD (TU)"	0.00	0.00	0.00	0.00	"kip-ft"
"TOTAL"	–493.26	–860.44	–711.57	–653.65	"kip-ft"

MOMENT ABOUT FOOTING CENTER OF GRAVITY:

$M_{ftg.cg}$ =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"BACKWALL DL"	0.69	0.96	0.96	0.77	"kip-ft"
"STEM DL"	43.94	61.03	61.03	48.82	"kip-ft"
"FOOTING DL"	0.00	0.00	0.00	0.00	"kip-ft"
"EXTRA DL1"	15.36	21.33	21.33	17.07	"kip-ft"
"EXTRA DL2"	0.00	0.00	0.00	0.00	"kip-ft"
"EXTRA LL"	0.00	12.55	0.00	7.17	"kip-ft"
"SUP.STR. DC"	36.16	50.22	0.00	40.18	"kip-ft"
"SUP. STR. DW"	4.26	9.83	0.00	6.56	"kip-ft"
"SUP. STR. LL"	0.00	52.78	0.00	30.16	"kip-ft"
"FILL AT HEEL (EV)"	–137.04	–185.01	–171.30	–137.04	"kip-ft"
"FILL AT TOE (EV)"	3.79	5.12	4.74	3.79	"kip-ft"
"VERT. LS AT HEEL"	0.00	–10.03	–20.05	–5.73	"kip-ft"
"EARTH LOAD (EH)"	165.21	165.21	137.67	110.14	"kip-ft"
"HOR. LS"	21.48	21.48	42.96	12.27	"kip-ft"
"TEMP. LOAD (TU)"	0.00	0.00	0.00	0.00	"kip-ft"
"TOTAL"	153.85	205.49	77.34	134.16	"kip-ft"

SLIDING,OVERTURNING AND BEARING RESISTANCE CHECK:

$Stability_{ftg}$ =

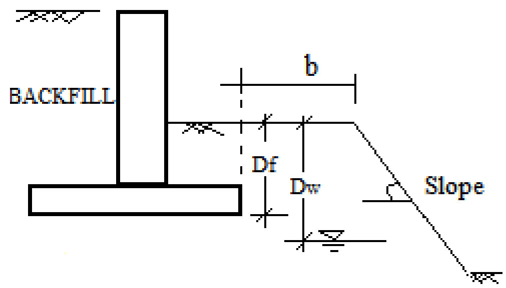
"STABILITY/BEARING CHECK:"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"Vertical Force"	67.52	111.23	82.32	82.21	"kip"
"Lateral Force"	20.01	20.01	18.53	13.19	"kip"
"Sliding Resistance"	43.22	71.19	52.69	52.61	"kip"
"Sliding Check"	"OK"	"OK"	"OK"	"OK"	"**"
"Moment about toe"	–493.26	–860.44	–711.57	–653.65	"kip-ft"
"Moment about cg. of footing"	153.85	205.49	77.34	134.16	"kip-ft"
"Eccentricity"	2.28	1.85	0.94	1.63	"ft"
"Allowable Eccentricity"	4.79	4.79	4.79	3.19	"ft"
"Eccentricity Check"	"OK"	"OK"	"OK"	"OK"	"**"
"Bearing Pressure"	4.62	7.19	4.76	5.17	"ksf"
"Bearing Resistance"	8.81	8.81	8.81	6.00	"ksf"
"Bearing Check"	"OK"	"OK"	"OK"	"OK"	"**"

GEOTECHNICAL OUTPUT:

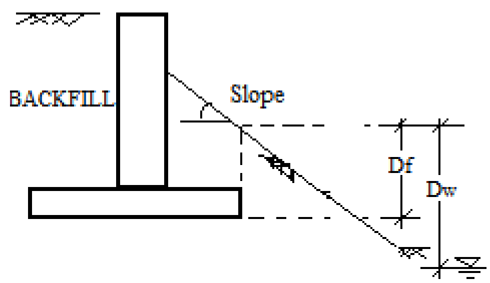
GEOTECH =

"Is footing on or near slope?"	"No"	"*****"
"Int.Angle.of.Friction"	30.00	"deg"
"Df"	4.00	"ft"
"Dw"	3.73	"ft"
"B"	19.17	"ft"
"N.gama"	17.00	"*****"
"S.gama"	0.90	"*****"
"N.gama.m"	15.26	"*****"
"C.w.gamma"	0.50	"*****"
"N.q"	18.00	"*****"
"S.q"	1.15	"*****"
"d.q"	1.04	"*****"
"N.qm"	21.58	"*****"
"C.w.q"	0.97	"*****"
"q.n"	19.57	"ksf"
"q.R"	8.81	"ksf"

FOOTING IS NEAR SLOPE



FOOTING IS ON SLOPE



LRFD A 10.6.3.1 (2007) US

Note: For factored bearing resistance calculation the AASHTO LRFD approach along with relative NYSDOT blue pages is followed. The load inclination factors are omitted according to LRFD C 10.6.3.1.2a, page 10-63, 2nd paragraph. Figure 10.6.3.1.2c-2 is used for modified bearing capacity factors.

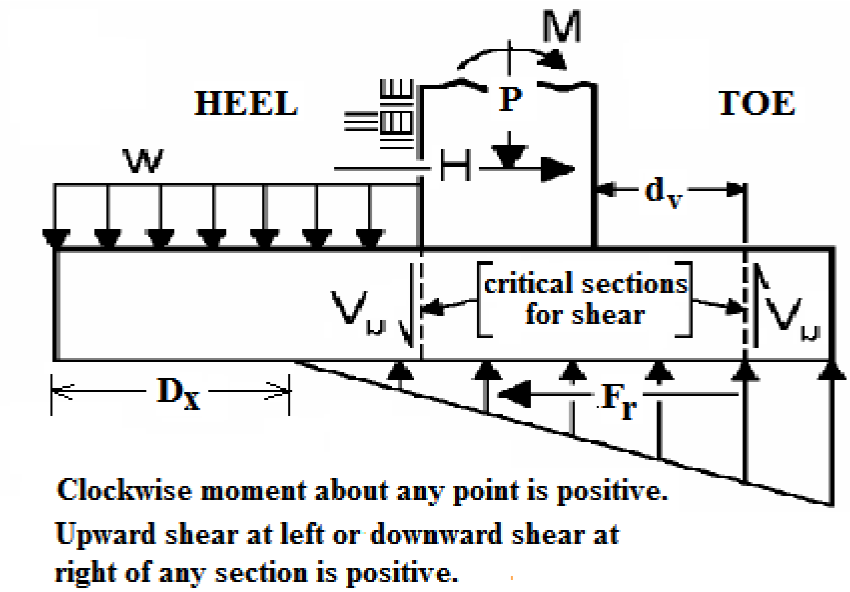
$q_R = \ _b \cdot q_n$

Footing not on or near slope

$q_n = \ _s \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \ _s \cdot B \cdot N \ _m \cdot C_w$

Footing on or near slope:

$q_n = 0.5 \cdot \ _s \cdot B \cdot N \ _m \cdot C_w$



MOMENT & SHEAR FOR FOOTING DESIGN:

$Footing_{table}$ =

"**************"	"Strength I-a"	"Strength I-b"	"Strength I-c"	"Service I"	"UNIT"
"Toe Pressure"	6.04	9.16	5.56	6.48	"ksf"
"Heel Pressure"	1.01	2.45	3.03	2.10	"ksf"
"Dx"	0.00	0.00	0.00	0.00	"ft"
"Toe Moment"	–42.25	–65.04	–38.56	–45.80	"kip-ft"
"Heel Moment"	–88.04	–66.11	–57.06	–41.50	"kip-ft"
"Toe Shear"	–7.39	–11.33	–6.65	–7.96	"kip"
"Shear cap.at toe"	45.10	35.64	47.23	NaN	"kip"
"Heel Sheer"	–13.24	–7.38	–9.22	–4.49	"kip"
"Shear cap.at heel"	38.82	46.86	47.97	NaN	"kip"
"Shear Check"	"OK"	"OK"	"OK"	NaN	"****"

FLAGS:

BEARING = "OK"

OVERTURNING = "OK"

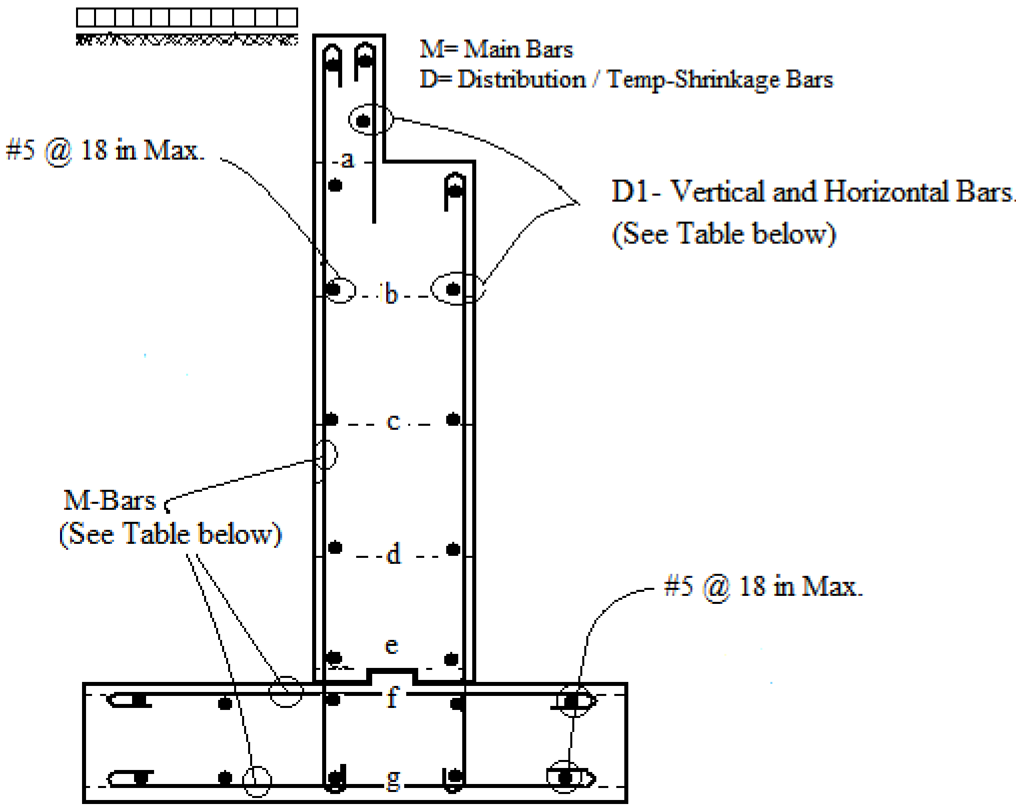
SLIDING = "OK"

FOOTING_SHEAR = "OK"

STEM_SHEAR = "OK"

GO TO DATA INPUT

REINFORCEMENT SUMMARY:



D1-Bars =

"**********"	"D1-Bar."	"S(in)"	"Design Control"
"Backwall"	5	18	"Shri-Temp. A 5.10.8"
"Stem"	5	6	"Shri-Temp. A 5.10.8"

M-Bars =

"**********"	"Ht.FromTop(ft)"	"M-Bar."	"S(in)"	"Design Control"	"Gov. Limit"
"Backwall at (a)"	2.92	5	11	"Crack Con. A 5.7.3.4"	"Strength I-c"
"Stem at (b)"	8.17	5	17	"Min. Req. A5.7.3.3.2"	"Strength I-b"
"Stem at (c)"	13.42	5	11	"Min. Req. A5.7.3.3.2"	"Strength I-b"
"Stem at (d)"	18.67	5	7	"Min. Req. A5.7.3.3.2"	"Strength I-b"
"Stem at (e)"	23.92	6	6	"Min. Req. A5.7.3.3.2"	"Strength I-b"
"Footing at (f)"	24.17	6	6	"Min. Req. A5.7.3.3.2"	"Strength I-a"
"Footing at (g)"	26.644	5	6	"Min. Req. A5.7.3.3.2"	"Strength I-b"

Expand the folllowing region for alternative bar sizes.

Reinforcement Table in Detail (Expand/Collapse)

Reinforcement in Back Face of Backwall at Section a: $H_{bw.sec.a} = 21\,ft$ **from the <u>top of footing</u>**

$Bars_{bw} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	11	"Crack Con. A 5.7.3.4"	"Strength I-c"
6	13	"Crack Con. A 5.7.3.4"	"Strength I-c"
7	18	"NYSDOT BM A 15.2"	"Strength I-c"
8	18	"NYSDOT BM A 15.2"	"Strength I-c"
9	18	"NYSDOT BM A 15.2"	"Strength I-c"
10	18	"NYSDOT BM A 15.2"	"Strength I-c"

Reinforcement in Back Face of Stem at Section b: $H_{st.sec.b} = 15.75\,ft$ **from the <u>top of footing</u>**

$Bars_{st.sec.b} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	17	"Min. Req. A5.7.3.3.2"	"Strength I-b"
6	18	"NYSDOT BM A 15.2"	"Strength I-b"
7	18	"NYSDOT BM A 15.2"	"Strength I-b"
8	18	"NYSDOT BM A 15.2"	"Strength I-b"
9	18	"NYSDOT BM A 15.2"	"Strength I-b"
10	18	"NYSDOT BM A 15.2"	"Strength I-b"

Reinforcement in Back Face of Stem at Section c: $H_{st.sec.c} = 10.5\,ft$ **from the <u>top of footing</u>**

$Bars_{st.sec.c} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	11	"Min. Req. A5.7.3.3.2"	"Strength I-b"
6	16	"Min. Req. A5.7.3.3.2"	"Strength I-b"
7	18	"NYSDOT BM A 15.2"	"Strength I-b"
8	18	"NYSDOT BM A 15.2"	"Strength I-b"
9	18	"NYSDOT BM A 15.2"	"Strength I-b"
10	18	"NYSDOT BM A 15.2"	"Strength I-b"

Reinforcement in Back Face of Stem at Section d: $H_{st.sec.d} = 5.25\,ft$ **from the <u>top of footing</u>**

$Bars_{st.sec.d} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	7	"Min. Req. A5.7.3.3.2"	"Strength I-b"
6	10	"Min. Req. A5.7.3.3.2"	"Strength I-b"
7	13	"Min. Req. A5.7.3.3.2"	"Strength I-b"
8	18	"NYSDOT BM A 15.2"	"Strength I-b"
9	18	"NYSDOT BM A 15.2"	"Strength I-b"
10	18	"NYSDOT BM A 15.2"	"Strength I-b"

Reinforcement in Back Face of Stem at Section e: $H_{st.sec.e} = 0$ from the top of footing

$Bars_{st.sec.e} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	4	"Min. Req. A5.7.3.3.2"	"Strength I-b"
6	6	"Min. Req. A5.7.3.3.2"	"Strength I-b"
7	8	"Min. Req. A5.7.3.3.2"	"Strength I-b"
8	11	"Min. Req. A5.7.3.3.2"	"Strength I-b"
9	13	"Min. Req. A5.7.3.3.2"	"Strength I-b"
10	17	"Crack Con. A 5.7.3.4"	"Strength I-b"

Reinforcement in Footing Top at section f:

$Bars_{ftg.top} =$

"Bar No."	"Trans. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	4	"Min. Req. A5.7.3.3.2"	"Strength I-a"
6	6	"Min. Req. A5.7.3.3.2"	"Strength I-a"
7	8	"Min. Req. A5.7.3.3.2"	"Strength I-a"
8	11	"Min. Req. A5.7.3.3.2"	"Strength I-a"
9	14	"Min. Req. A5.7.3.3.2"	"Strength I-a"
10	18	"NYSDOT BM A 15.2"	"Strength I-a"

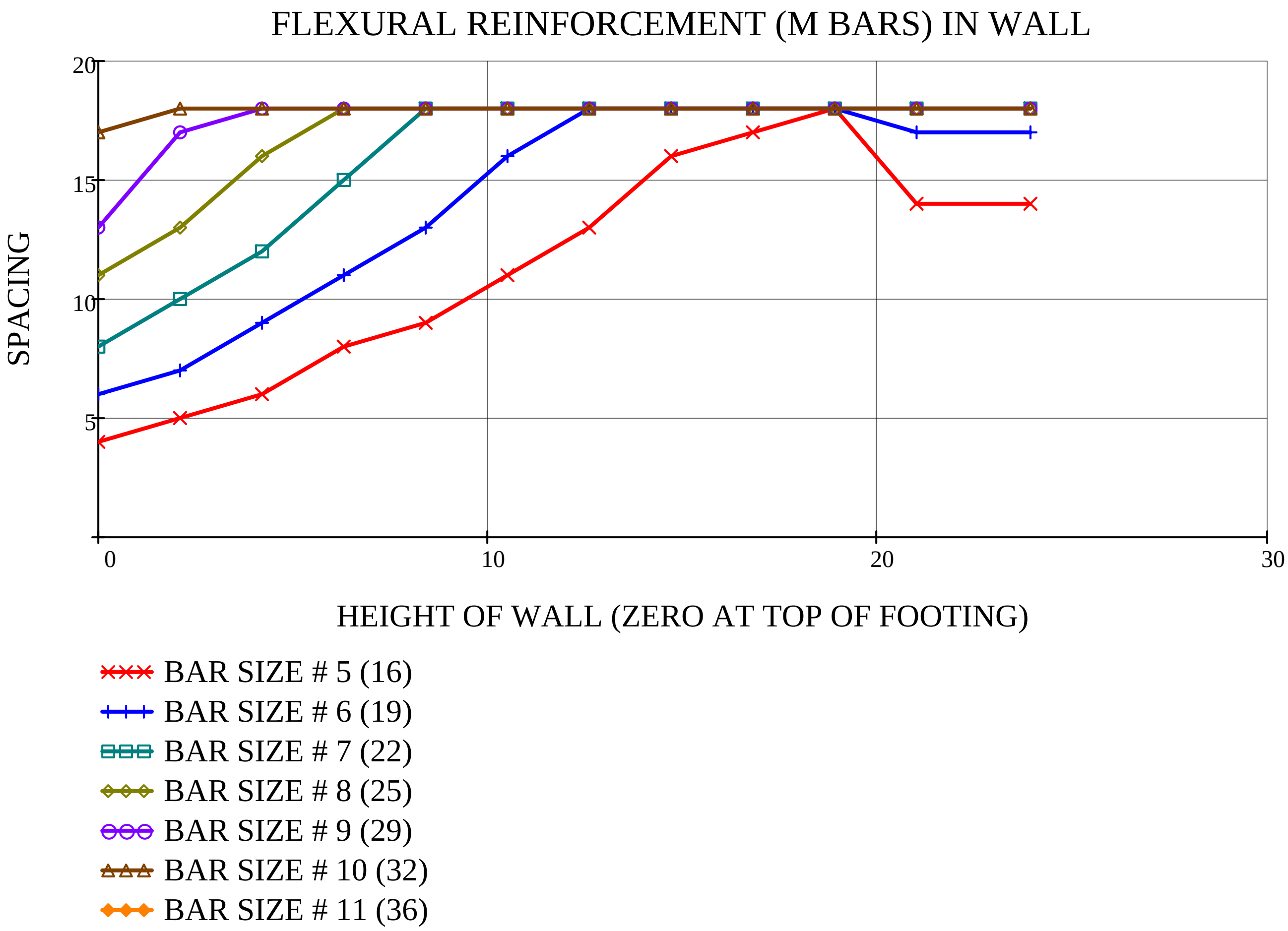
Reinforcement in Footing Bottom at section g :

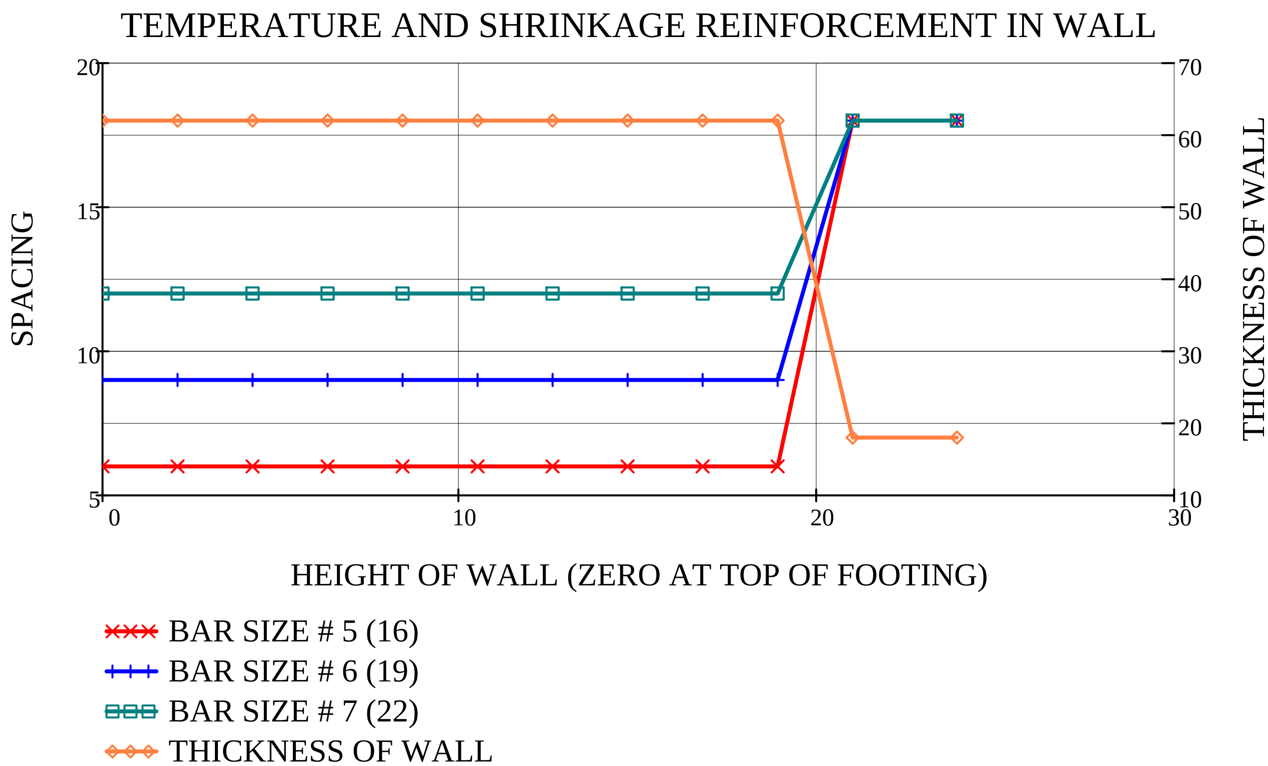
$Bars_{ftg.bot} =$

"Bar No."	"Trans. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	6	"Min. Req. A5.7.3.3.2"	"Strength I-b"
6	8	"Min. Req. A5.7.3.3.2"	"Strength I-b"
7	12	"Min. Req. A5.7.3.3.2"	"Strength I-b"
8	15	"Min. Req. A5.7.3.3.2"	"Strength I-b"
9	18	"NYSDOT BM A 15.2"	"Strength I-b"
10	18	"NYSDOT BM A 15.2"	"Strength I-b"

Reinforcement Table in Detail (Expand/Collapse)

Reinforcement Graph





Reinforcement Graph

GO TO DATA INPUT

NYSDOT
CANTILEVER RETAINING WALL ANALYSIS
SPREAD FOOTING ON SOIL
AASHTO LRFD - FOURTH EDITION 2007
(2009 INTERIM)

Disclaimer

About this worksheet

JOB DESCRIPTION:

HIGHWAY BRIDGE DESIGN MQP

PIN: N/A **BIN:** N/A

WALL: NORTH ABUTMENT

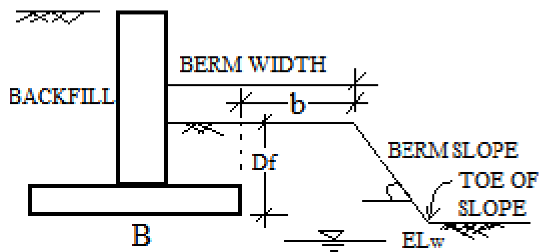
DESIGNED BY: MQP

CHECKED BY: MQP

Material and Geotechnical data inputs:

60	ksi	Yield strength of reinforcing bars
3	ksi	Compressive strength of concrete
2.5	in	Clear Cover of reinforcement in wall
3	in	Clear Cover of reinforcement in Footing
1		Exposure Factor. 1.0 for Class 1 Exposure & 0.75 for Class 2 Exposure. LRFD Art. 5.7.3.4 (2007)

6	ksf	**Service bearing resistance of soil**
0.8		**Coefficient of sliding (friction)**
0.8		**Resistance factor for Sliding**
35	deg	**Internal angle of friction for backfill**
125	pcf	**Unit weight of backfill**
30	deg	**Internal angle of friction for soil under the footing**
125	pcf	**Unit weight of soil under the footing**
0.45		**Resistance Factor for bearing.** LRFD T 10.5.5.2.2-1 (2007)
0	ft	**Berm Width**
0	deg	**Berm Slope**



Note: Input Berm Width = 0 and Berm Slope = 0 for following conditions:
1.The ground surface in front of abutment is level.
2.There is a very long berm with a slope. (Roughly: b/B > 6, see Figure.)
3.There is a reasonable berm with slope but the bottom of footing is below the toe of slope.

147.9	ft	**Elevation of water table**
147.63	ft	**Elevation at bottom of Footing**
4	ft	**Distance of Traffic edge (See following Figure)**

Extra Loads:

0	kip/ft	XDL_1 **Extra Dead Load 1**
0	in	$DXDL_1$ **Distance of Extra Load 1 from the front face of wall**
0	kip/ft	XDL_2 **Extra Dead Load 2**
0	in	$DXDL_2$ **Distance of Extra Load 2 from the front face of wall**

Wall Geometric Inputs:

24	in	Thickness of Footing
8	ft	Heel Width
3	ft	Toe Width
24	ft	Fill height over heel (See following Figure)
0	in	Height of Reveal (See following Figure)
0	deg	Slope of backfill over heel (See following Figure)
0	ft	Distance of slope of backfill over heel (See following Figure)
18	in	Thickness of wall at top (See following Figure)
24	in	Thickness of step (See following Figure)
16	ft	Height of Step (See following Figure)
1	ft	Fill height over toe (See following Figure)
58	ft	Length of wall

Input Program

Live Load Surcharge

Figure:

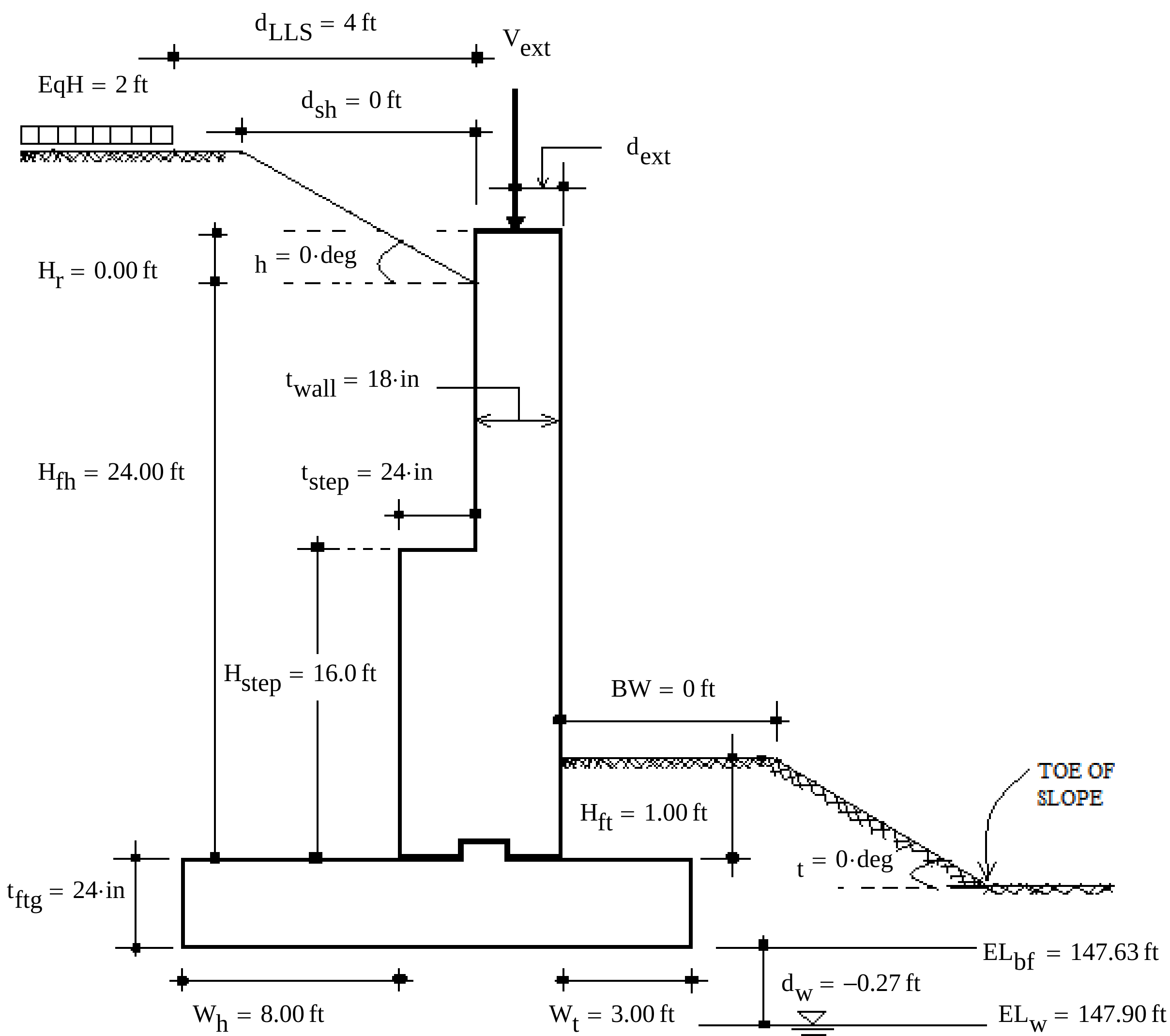


Figure:

Note: All loads and resistances are for one foot of wall length.

Loads Calculation

Load Factors & Design Data

Analyze and Combine Force Effect

Rebar sizes

Temp & Shrinkage Reinforcement for wall

Reinforcement Graph

Temp & Shrinkage Reinforcement Graph

Program for Reinforcement in Wall at Section (d)

Program for Reinforcement in Wall at Section (c)

Program for Reinforcement in Wall at Section (b)

Program for Reinforcement in Wall at Section (a)

Wall Shear Check

Overturning Check

Sliding check

Structural design of Footing

Bearing Resistance Calculation

Bearing resistance check

Programming for Reinforcement in bottom footing

Programming for Reinforcement in top footing

Shear design of footing

RESULTS:

LOAD FACTORS:

LOAD_FACTORS =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"
"WALL DL"	0.9	1.25	1
"FOOTING DL"	0.9	1.25	1
"EXTRA DL 1"	0.9	1.25	1
"EXTRA DL 2"	0.9	1.25	1
"FILL.AT.HEEL (EV)"	1	1.35	1
"FILL.AT.TOE (EV)"	1	1.35	1
"VERT. LS AT HEEL"	0	1.75	1
"VERT.COM. OF EH"	1.5	1.5	1
"VERT.COM. OF LS"	1.75	1.75	1
"HOR. COM OF EH"	1.5	1.5	1
"HOR. COM OF LS"	1.75	1.75	1

Strength I-a: Min. vertical load and Max. lateral load (without the vertical component of LS)

Strength I-b: Max. vertical load and Max. lateral load

Service I: Unfactored load for bearing and serviceability check

EQUIVALENT HEIGHT OF LIVE LOAD SURCHARGE:

EqH = 2 ft

FACTORED VERTICAL LOADS ON WALL:

VL_{wall} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"WALL DL"	9.18	12.75	10.2	"kip"
"EXTRA DL 1"	0	0	0	"kip"
"EXTRA DL 2"	0	0	0	"kip"
"TOTAL"	9.18	12.75	10.2	"kip"

ADJUSTED BACKFILL SLOPE ANGLE: $h = 0$

COEFFICIENT OF ACTIVE EARTH PRESSURE: $K_a = 0.271$

FACTORED HORIZONTAL LOADS ON WALL:

$HL_{wall} =$

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"HOR. COM OF EH"	14.633	14.633	9.756	"kip"
"HOR. COM OF LS"	2.845	2.845	1.626	"kip"
"TOTAL"	17.479	17.479	11.382	"kip"

FACTORED VERTICAL LOADS ON FOOTING:

$VL_{ftg} =$

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"WALL DL"	9.18	12.75	10.2	"kip"
"FOOTING DL"	3.915	5.437	4.35	"kip"
"EXTRA DL 1"	0	0	0	"kip"
"EXTRA DL 2"	0	0	0	"kip"
"FILL.AT.HEEL (EV)"	26	35.1	26	"kip"
"FILL.AT.TOE (EV)"	0.375	0.506	0.375	"kip"
"VERT. LS AT HEEL"	0	2.625	1.5	"kip"
"VERT.COM. OF EH"	0	0	0	"kip"
"VERT.COM. OF LS"	0	0	0	"kip"
"TOTAL"	39.47	56.419	42.425	"kip"

FACTORED HORIZONTAL LOADS ON FOOTING:

HL_{ftg} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"HOR. COM OF EH"	17.174	17.174	11.449	"kip"
"HOR. COM OF LS"	3.083	3.083	1.761	"kip"
"TOTAL"	20.257	20.257	13.211	"kip"

MOMENT ARMS:

NOTE:ARM SIGNS ARE ACCORDING TO COORDINATE SYSTEMS

M_{arms} =

"*********"	"About Wall CG. (ft)"	"About Footing Toe (ft)"
"WALL DL"	0	–4.574
"FOOTING DL"	0	–7.25
"EXTRA DL 1"	0	0
"EXTRA DL 2"	0	0
"FILL.AT.HEEL (EV)"	0	–10.115
"FILL.AT.TOE (EV)"	0	–1.5
"VERT. LS AT HEEL"	0	–11.5
"VERT.COM. OF EH"	0	–14.5
"VERT.COM. OF LS"	0	–14.5
"HOR. COM OF EH"	8	8.667
"HOR. COM OF LS"	12	13

MOMENT ABOUT WALL CENTER OF GRAVITY:

M_{wall} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"WALL DL"	0	0	0	"kip-ft"
"EXTRA DL 1"	0	0	0	"kip-ft"
"EXTRA DL 2"	0	0	0	"kip-ft"
"HOR. COM OF EH"	117.07	117.07	78.05	"kip-ft"
"HOR. COM OF LS"	34.14	34.14	19.51	"kip-ft"
"TOTAL"	151.21	151.21	97.56	"kip-ft"

MOMENT ABOUT FOOTING TOE:

M_{toe} =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"WALL DL"	–41.99	–58.31	–46.65	"kip-ft"
"FOOTING DL"	–28.38	–39.42	–31.54	"kip-ft"
"EXTRA DL 1"	0	0	0	"kip-ft"
"EXTRA DL 2"	0	0	0	"kip-ft"
"FILL.AT.HEEL (EV)"	–263	–355.05	–263	"kip-ft"
"FILL.AT.TOE (EV)"	–0.56	–0.76	–0.56	"kip-ft"
"VERT. LS AT HEEL"	0	–30.19	–17.25	"kip-ft"
"VERT.COM. OF EH"	0	0	0	"kip-ft"
"VERT.COM. OF LS"	0	0	0	"kip-ft"
"HOR. COM OF EH"	148.84	148.84	99.23	"kip-ft"
"HOR. COM OF LS"	40.07	40.07	22.9	"kip-ft"
"TOTAL"	–145.02	–294.82	–236.87	"kip-ft"

MOMENT ABOUT FOOTING CENTER OF GRAVITY:

$M_{ftg.cg}$ =

"LOADS / CASES"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"WALL DL"	24.57	34.12	27.3	"kip-ft"
"FOOTING DL"	0	0	0	"kip-ft"
"EXTRA DL 1"	0	0	0	"kip-ft"
"EXTRA DL 2"	0	0	0	"kip-ft"
"FILL.AT.HEEL (EV)"	–74.5	–100.58	–74.5	"kip-ft"
"FILL.AT.TOE (EV)"	2.16	2.91	2.16	"kip-ft"
"VERT. LS AT HEEL"	0	–11.16	–6.38	"kip-ft"
"VERT.COM. OF EH"	0	0	0	"kip-ft"
"VERT.COM. OF LS"	0	0	0	"kip-ft"
"HOR. COM OF EH"	148.84	148.84	99.23	"kip-ft"
"HOR. COM OF LS"	40.07	40.07	22.9	"kip-ft"
"TOTAL"	141.14	114.22	70.71	"kip-ft"

SLIDING,OVERTURNING AND BEARING RESISTANCE CHECK:

$Stability_{ftg}$ =

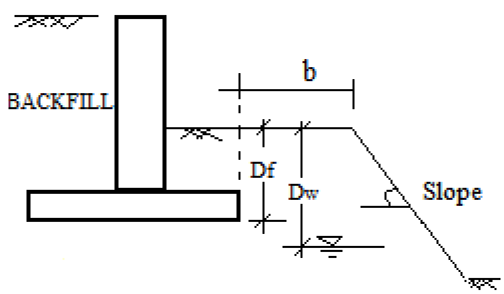
"STABILITY/BEARING CHECK:"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"Vertical Force"	39.47	56.419	42.425	"kip"
"Lateral Force"	20.257	20.257	13.211	"kip"
"Sliding Resistance"	25.261	36.108	27.152	"kip"
"Sliding Check"	"OK"	"OK"	"OK"	"**"
"Moment about toe"	–145.017	–294.817	–236.874	"kip-ft"
"Moment about cg. of footing"	141.14	114.219	70.707	"kip-ft"
"Eccentricity"	3.576	2.024	1.667	"ft"
"Allowable Eccentricity"	3.625	3.625	2.417	"ft"
"Eccentricity Check"	"OK"	"OK"	"OK"	"**"
"Bearing Pressure"	5.371	5.398	3.799	"ksf"
"Bearing Resistance"	6.571	6.571	6	"ksf"
"Bearing Check"	"OK"	"OK"	"OK"	"**"

GEOTECHNICAL OUTPUT:

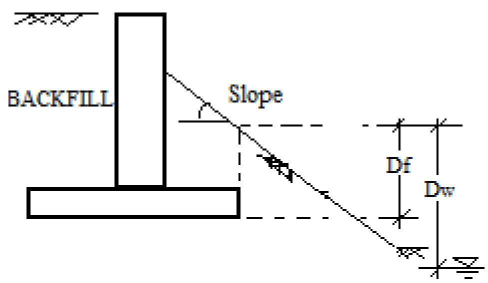
GEOTECH =

"Is footing on or near slope?"	"No"	"****"
"Int.Angle.of.Friction"	30	"deg"
"Df"	3	"ft"
"Dw"	2.73	"ft"
"B"	14.5	"ft"
"N.gama"	17	"****"
"S.gama"	0.9	"****"
"N.gama.m"	15.3	"****"
"C.w.gamma"	0.5	"****"
"N.q"	18	"****"
"S.q"	1.144	"****"
"d.q"	1.04	"****"
"N.qm"	21.413	"****"
"C.w.q"	0.955	"****"
"q.n"	14.601	"ksf"
"q.R"	6.571	"ksf"

FOOTING IS NEAR SLOPE



FOOTING IS ON SLOPE



LRFD A 10.6.3.1 (2007) US

Note: For factored bearing resistance calculation the AASHTO LRFD approach along with relative NYSDOT blue pages is followed. The load inclination factors are omitted according to LRFD C 10.6.3.1.2a, page 10-63, 2nd paragraph. Figure 10.6.3.1.2c-2 is used for modified bearing capacity factors.

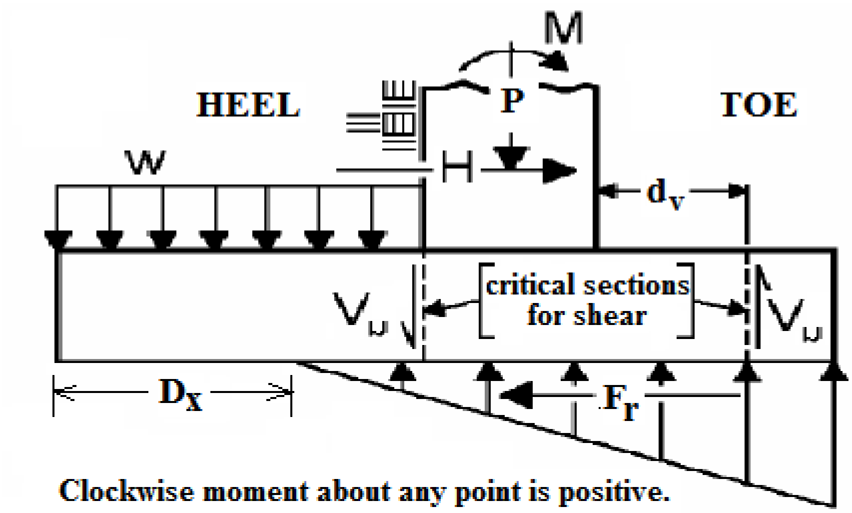
$q_u = \ _b \cdot q_n$

Footing not on or near slope

$q_n = \ _s \cdot D_f \cdot N_{qm} \cdot C_{wq} + 0.5 \cdot \ _s \cdot B \cdot N \ _m \cdot C_w$

Footing on or near slope:

$q_n = 0.5 \cdot \ _s \cdot B \cdot N \ _m \cdot C_w$



Clockwise moment about any point is positive.
Upward shear at left or downward shear at right of any section is positive.

MOMENT & SHEAR FOR FOOTING DESIGN:

Footing$_{table}$ =

"**************"	"Strength I-a"	"Strength I-b"	"Service I"	"UNIT"
"Toe Pressure"	7.162	7.15	4.944	"ksf"
"Heel Pressure"	0	0.631	0.908	"ksf"
"Dx"	3.478	0	0	"ft"
"Toe Moment"	–27.527	–27.707	–19.082	"kip-ft"
"Heel Moment"	–94.624	–96.154	–60.292	"kip-ft"
"Toe Shear"	–8.8	–8.771	–6.027	"kip"
"Shear cap.at toe"	20.654	20.613	NaN	"kip"
"Heel Sheer"	–19.516	–18.587	–11.729	"kip"
"Shear cap.at heel"	20.554	20.55	NaN	"kip"
"Shear Check"	"OK"	"OK"	NaN	"****"

FLAGS:

FOUNDATION:

BEARING = "OK"

OVERTURNING = "OK"

SLIDING = "OK"

FOOTING_SHEAR = "OK"

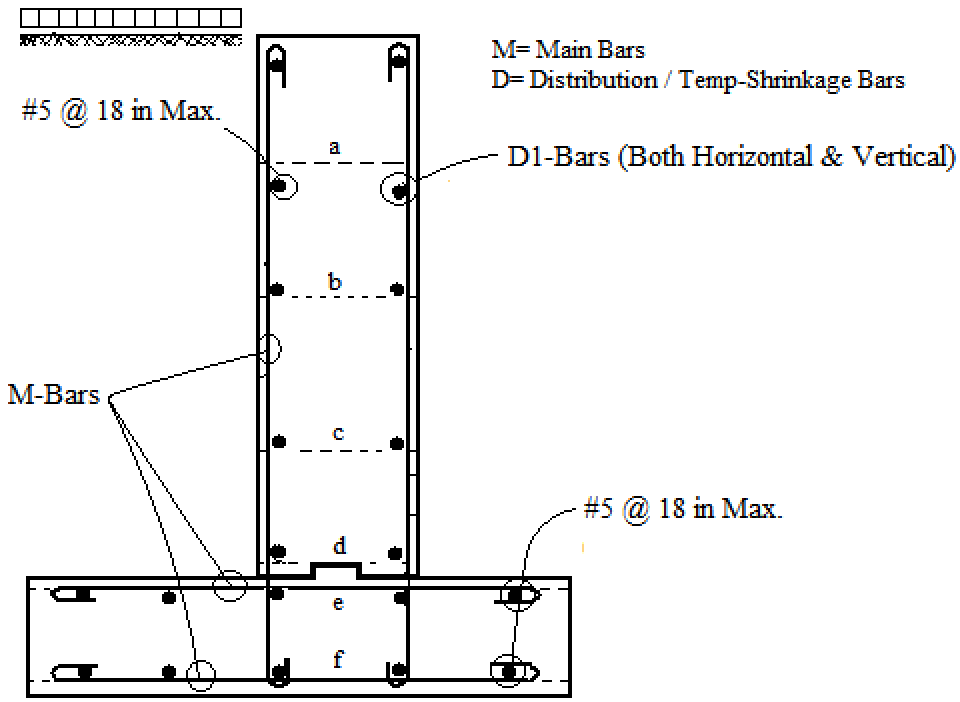
WALL:

SHEAR_AT_STEP = "OK"

SHEAR_AT_BOTTOM = "OK"

GO TO DATA INPUT

REINFORCEMENT SUMMARY



D1-Bars =

"**********"	"D1-Bar."	"S(in)"	"Design Control"
"Wall at (a)"	5	18	"Shri-Temp. A 5.10.8"
"Wall at (d)"	5	9	"Shri-Temp. A 5.10.8"

M-Bars =

"**********"	"Ht.FromTop(ft)"	"M-Bar."	"S(in)"	"Design Control"	"Gov. Limit"
"Wall at (a)"	6	5	18	"NYSDOT BM A 15.2"	"Strength I-a"
"Wall at (b)"	12	5	18	"NYSDOT BM A 15.2"	"Strength I-a"
"Wall at (c)"	18	5	7	"Min. Req. A5.7.3.3.2"	"Strength I-a"
"Wall at (d)"	24	7	6	"Min. Req. A5.7.3.3.2"	"Strength I-a"
"Footing at (e)"	24.25	7	6	"Moment"	"Strength I-b"
"Footing at (f)"	25.724	5	9	"Min. Req. A5.7.3.3.2"	"Strength I-b"

Expand the folllowing region for alternative bar sizes.

Reinforcement Table in Detail (Expand/Collapse)

Reinforcement in Back Face of Wall at Section a: $H_{w.sec.a} = 18\,ft$ **from the top of footing**

$Bars_{w.sec.a} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	18	"NYSDOT BM A 15.2"	"Strength I-a"
6	18	"NYSDOT BM A 15.2"	"Strength I-a"
7	18	"NYSDOT BM A 15.2"	"Strength I-a"
8	18	"NYSDOT BM A 15.2"	"Strength I-a"
9	18	"NYSDOT BM A 15.2"	"Strength I-a"
10	18	"NYSDOT BM A 15.2"	"Strength I-a"
11	18	"NYSDOT BM A 15.2"	"Strength I-a"

Reinforcement in Back Face of Wall at Section b: $H_{w.sec.b} = 12\,ft$ **from the top of footing**

$Bars_{w.sec.b} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	18	"NYSDOT BM A 15.2"	"Strength I-a"
6	18	"NYSDOT BM A 15.2"	"Strength I-a"
7	18	"NYSDOT BM A 15.2"	"Strength I-a"
8	18	"NYSDOT BM A 15.2"	"Strength I-a"
9	18	"NYSDOT BM A 15.2"	"Strength I-a"
10	18	"NYSDOT BM A 15.2"	"Strength I-a"
11	18	"NYSDOT BM A 15.2"	"Strength I-a"

Reinforcement in Back Face of Wall at Section c: $H_{w.sec.c} = 6\,ft$ **from the top of footing**

$Bars_{w.sec.c} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	7	"Min. Req. A5.7.3.3.2"	"Strength I-a"
6	10	"Min. Req. A5.7.3.3.2"	"Strength I-a"
7	13	"Min. Req. A5.7.3.3.2"	"Strength I-a"
8	17	"Crack Con. A 5.7.3.4"	"Strength I-a"
9	18	"NYSDOT BM A 15.2"	"Strength I-a"
10	18	"NYSDOT BM A 15.2"	"Strength I-a"
11	18	"NYSDOT BM A 15.2"	"Strength I-a"

Reinforcement in Back Face of Wall at Section d:

$H_{w.sec.d} = 0$ **from the <u>top of footing</u>**

$Bars_{w.sec.d} =$

"Bar No."	"Vert. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	3	"Min. Req. A5.7.3.3.2"	"Strength I-a"
6	4	"Min. Req. A5.7.3.3.2"	"Strength I-a"
7	6	"Min. Req. A5.7.3.3.2"	"Strength I-a"
8	8	"Min. Req. A5.7.3.3.2"	"Strength I-a"
9	10	"Min. Req. A5.7.3.3.2"	"Strength I-a"
10	12	"Min. Req. A5.7.3.3.2"	"Strength I-a"
11	15	"Min. Req. A5.7.3.3.2"	"Strength I-a"

Reinforcement in Footing Top:

$Bars_{ftg.top} =$

"Bar No."	"Trans. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	3	"Moment"	"Strength I-b"
6	4	"Moment"	"Strength I-b"
7	6	"Moment"	"Strength I-b"
8	8	"Moment"	"Strength I-b"
9	10	"Moment"	"Strength I-b"
10	13	"Moment"	"Strength I-b"
11	16	"Moment"	"Strength I-b"

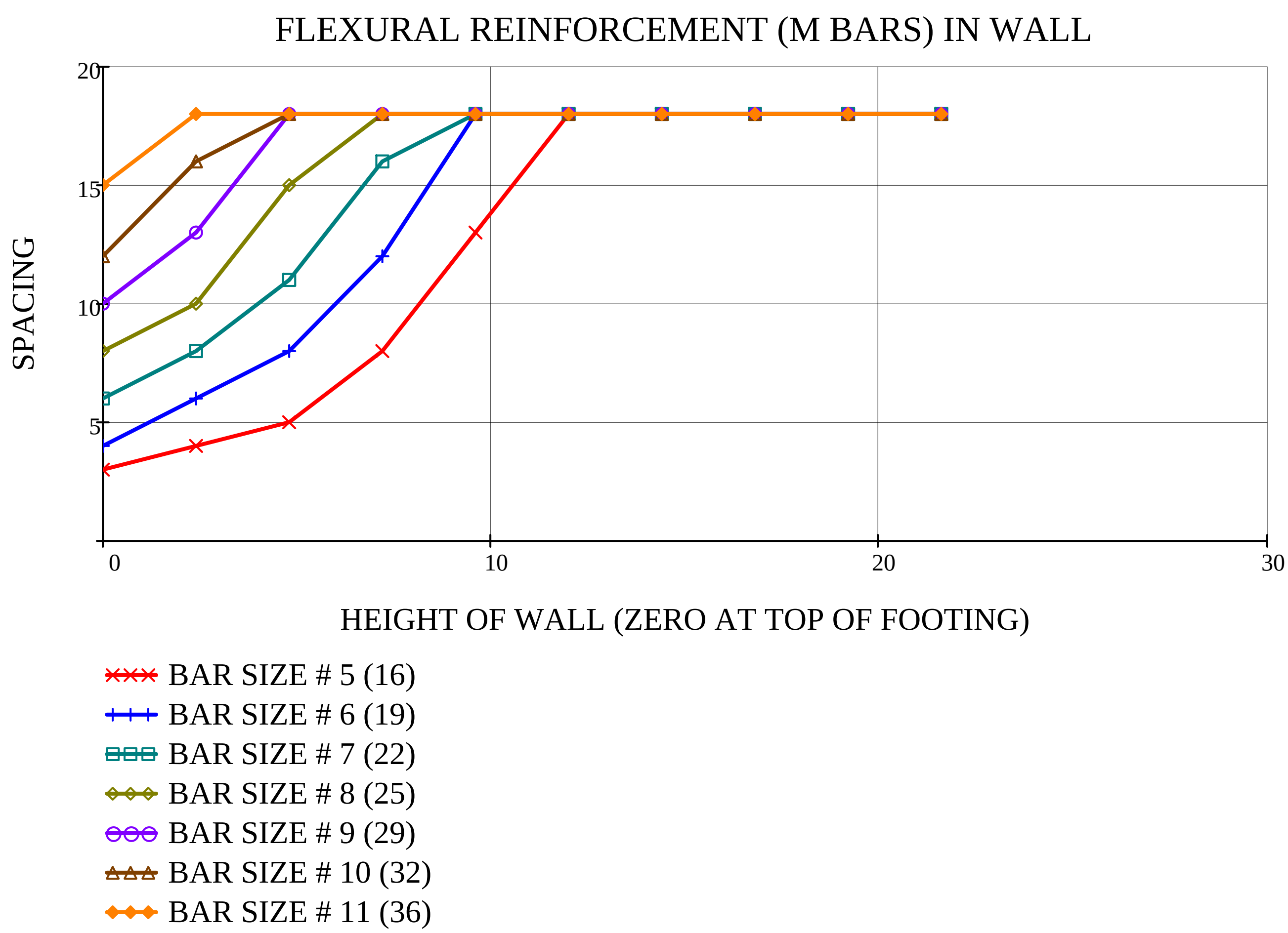
Reinforcement in Footing Bottom :

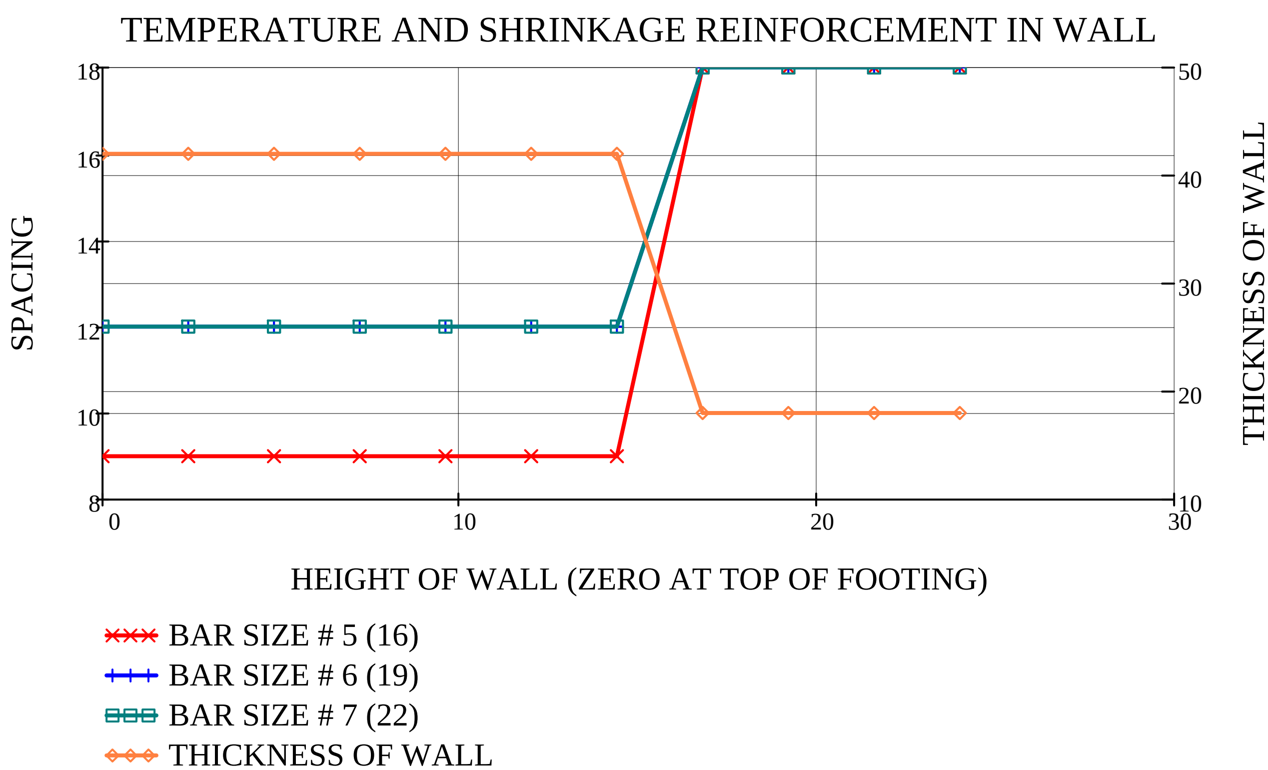
$Bars_{ftg.bot} =$

"Bar No."	"Trans. Bar Sp. (in)"	"Design Control"	"Governing Limit State"
5	9	"Min. Req. A5.7.3.3.2"	"Strength I-b"
6	13	"Min. Req. A5.7.3.3.2"	"Strength I-b"
7	17	"Min. Req. A5.7.3.3.2"	"Strength I-b"
8	18	"NYSDOT BM A 15.2"	"Strength I-b"
9	18	"NYSDOT BM A 15.2"	"Strength I-b"
10	18	"NYSDOT BM A 15.2"	"Strength I-b"
11	18	"NYSDOT BM A 15.2"	"Strength I-b"

Reinforcement Table in Detail (Expand/Collapse)

Reinforcement Graph





Reinforcement Graph

GO TO DATA INPUT

Appendix 4.1: Cost Estimate

This section contains the calculations used to develop the final cost estimate for the bridge calculations. It is a line by line estimation of the tasks included in both the actual state design and the proposed MQP design.

Final Proposal and State Plan Estimates

This section contains the quantities, prices, adjustments, and totals by task for both the actual state bridge cost estimate, and the proposed bridge cost estimate.

State Bid Data

This section is a published report made available by NHDOT for all bid projects. It contains the pricing and bid information for the actual replacement project in real life. The unit prices provided by the winning bidder were used to cost estimate the cost of just the SB bridge for the actual state project.

Final Esitmates

Task	Unit	Quantity	Price Per Unit	2013 NH
Demolish Existing SB Bridge	each	1	$ 137,900.00	$ 137,900.00
Construct North Abutment MSE Wall of New SB Bridge	each	1	$ 307,335.00	$ 307,335.00
Construct South Abutment MSE Wall for New SB	each	1	$ 307,335.00	$ 307,335.00
Form/Rebar/Pour/Strip North Abutment Footing for	CY	45	$ 275.00	$ 12,405.56
Form/Rebar/Pour/Strip South Abutment Footing for	CY	45	$ 275.00	$ 12,405.56
Form/Rebar/Pour/Strip North Stub Abutment for New	CY	42	$ 670.00	$ 27,905.50
Form/Rebar/Pour/Strip South Stub Abutment for New	CY	41.7	$ 670.00	$ 27,905.50
Form/Rebar/Pour/Strip North Abutment Backwall for	CY	33	$ 1,090.00	$ 36,318.80
Form/Rebar/Pour/Strip South Abutment Backwall for	CY	33	$ 1,090.00	$ 36,318.80
Set Structural Steel & Set Blocking Grades for New SB	each	8	$ 90,500.00	$ 724,000.00
Set Precast Deck Panels for New SB Bridge	CY	122	$ 795.00	$ 96,614.58
Form/Rebar/Pour/Strip Deck Overpour, Overhang,	CY	156	$ 862.00	$ 134,687.50
Form/Rebar/Pour/Strip North Abutment Approach	CY	69	$ 259.00	$ 17,986.11
Form/Rebar/Pour/Strip South Abutment Approach	CY	69	$ 259.00	$ 17,986.11
Install Bridge Rail for New SB Bridge	LF	300	$ 142.00	$ 42,600.00
Install Barrier Membrane On SB Bridge Deck	Sq Ft.	1250	$ 24.75	$ 30,937.50
Install Bridge Joint for SB Bridge	each	1.0	$ 58,548.00	$ 58,548.00
Pave Base & Temporary Course for SB Bridge Deck Sta.	Tons	102	$ 160.00	$ 16,312.50

Prices sourced from actual bid information

Total	$ 2,045,502.02

Task	Unit	Quantity	Price Per Unit	2013 NH
Mobilization	each	1	$ 20,000.00	$ 19,929.02
Clearing & grubbing	acre	2	$ 10,000.00	$ 19,929.02
Sheet Piles	sq ft.	4722	$ 26.50	$ 124,675.73
Excavation	CY	3303	$ 2.02	$ 6,661.10
Abutment wall footings	CY	117	$ 259.00	$ 30,109.44
pier footing	CY	33	$ 395.00	$ 13,119.94
Wing wall footing	CY	94.5	$ 259.00	$ 24,393.42
Abutment wall	CY	313.6	$ 325.00	$ 101,543.92
Wing walls	CY	189	$ 325.00	$ 61,052.05
Pier columns	CY	49.2	$ 835.00	$ 40,915.12
Backfill	CY	1055	$ 22.93	$ 24,111.65
Pier cap	CY	27	$ 1,100.00	$ 29,229.24
DBTs (14)	each	14	$ 18,029.00	$ 251,510.27
Bearings	each	28	$ 1,200.00	$ 33,480.76
Approach slabs	CY	69	$ 259.00	$ 17,922.28
Expansion joints	each	1	$ 50,250.00	$ 50,071.68
Bridge membrane	sq yd.	625	$ 22.50	$ 14,012.60
Bridge pavement	tons	105.5	$ 97.50	$ 10,246.71
Precast Barrier Perm.	lf	150	$ 465.00	$ 69,502.47
Bridge Approach Rail	lf	40	$ 145.00	$ 5,779.42
Temporary Barriers	lf	190	$ 39.50	$ 7,478.37
Removal of Existing Bridge	each	1	$ 150,000.00	$ 149,467.69
Sheet Piles	sq ft.	4722	$ 26.50	$ 124,675.73
Excavation	CY	3303	$ 2.02	$ 6,661.10
Abutment wall footings	CY	117	$ 259.00	$ 30,109.44
pier footing	CY	22	$ 395.00	$ 8,746.63
Wing wall footing	CY	94.5	$ 259.00	$ 24,393.42
Abutment wall	CY	314	$ 325.00	$ 101,543.92
Wing walls	CY	189	$ 325.00	$ 61,052.05
Pier columns	CY	33	$ 835.00	$ 27,276.75
Backfill	CY	1055	$ 39.50	$ 41,535.55
Pier cap	CY	18	$ 1,100.00	$ 19,486.16
DBTs (12)	each	12	$ 18,029.00	$ 215,580.23
Bearings	each	24	$ 1,200.00	$ 28,697.80
Approach slabs	CY	69	$ 259.00	$ 17,922.28
Expansion joints	each	1	$ 50,250.00	$ 50,071.68
Bridge membrane	sq yrd.	625	$ 22.50	$ 14,012.60
Bridge pavement	tons	105.5	$ 97.50	$ 10,246.71
Precast Barrier Perm.	lf	150	$ 465.00	$ 69,502.47
Bridge Approach Rail	lf	40	$ 145.00	$ 5,779.42
Temporary Barriers Removal	lf	190	$ 23.50	$ 4,449.15
Concrete Sealant	sq ft.	9648	$ 0.06	$ 563.09
Loam & Seeding	acre	1072	28	$ 29,909.48

Information sourced from RS Means and actual bid information.

Total	**$ 1,997,357.55**

THE STATE OF NEW HAMPSHIRE

DEPARTMENT OF TRANSPORTATION

PROJECT: SALEM-MANCHESTER
BI-A000(125), 13933E

COUNTIES AND CODES: ROCKINGHAM 015

DATE BIDS OPEN: JUNE 7, 2012

SCOPE OF WORK: ROADWAY AND BRIDGE REHABILITATION

LOCATION: I-93 EXIT 2 INTERCHANGE AND
PELHAM ROAD IN THE TOWN OF SALEM

COMPLETION DATE: JULY 10, 2015

A GEORGE R. CAIRNS AND SONS, INC.
8 LEDGE RD., WINDHAM, NH 03087
$40,908,383.67

B R.S. AUDLEY, INC.
609 ROUTE 3A, BOW, NH 03304
$40,967,760.95

C THE MIDDLESEX CORPORATION
ONE SPECTACLE POND ROAD, LITTLETON, MA 01460
$43,877,573.20

ITEM NO.	DESCRIPTION	UNIT	QUANTITY	A UNIT PRICE	A TOTAL	B UNIT PRICE	B TOTAL	C UNIT PRICE	C TOTAL
201.1	CLEARING AND GRUBBING (F)	A	31.	3,760.00	116,560.00	3,500.00	108,500.00	7,500.00	232,500.00
201.21	REMOVING SMALL TREES	EA	21.	225.00	4,725.00	300.00	6,300.00	200.00	4,200.00
201.6	CLEARING FOR FENCE LINES (F)	A	1.	4,225.00	4,225.00	1,800.00	1,800.00	7,000.00	7,000.00
201.881	INVASIVE SPECIES CONTROL TYPE I	SY	18,500.	1.10	20,350.00	1.25	23,125.00	.10	1,850.00
201.882	INVASIVE SPECIES CONTROL TYPE II	SY	14,200.	1.10	15,620.00	1.25	17,750.00	1.50	21,300.00
202.201	DEMOLISHING BUILDINGS	U	1.	32,900.00	32,900.00	15,000.00	15,000.00	30,000.00	30,000.00
202.301	BUILDING ASBESTOS ABATEMENT	U	1.	6,050.00	6,050.00	2,900.00	2,900.00	10,000.00	10,000.00
202.31	FILL ABANDONED PIPE	CY	240.	175.00	42,000.00	170.00	40,800.00	170.00	40,800.00
202.32	FILL ABANDONED STUCTURE	CY	6.	175.00	1,050.00	135.00	810.00	115.00	690.00
202.41	REMOVAL OF EXISTING PIPE 0-24" DIAMETER	LF	4,650.	14.85	69,052.50	8.00	37,200.00	9.00	41,850.00
202.42	REMOVAL OF EXISTING PIPE								

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            OVER 24" DIAMETER       LF          215.           19.75     4,246.25
   11.00     2,365.00        17.00     3,655.00
  202.5     REMOVAL OF CATCH BASINS,
             DROP INLETS, AND
             MANHOLES               EA           65.          380.00    24,700.00
  280.00    18,200.00      300.00    19,500.00
  202.7     REMOVAL OF GUARDRAIL    LF       19,175.            1.85    35,473.75
    1.50    28,762.50        2.00    38,350.00
  203.1     COMMON EXCAVATION       CY      342,000.            6.40 2,188,800.00
    4.00 1,368,000.00        6.90 2,359,800.00
  203.2     ROCK EXCAVATION         CY       22,500.           11.85   266,625.00
   25.00    562,500.00      23.00    517,500.00
  203.4     MUCK EXCAVATION         CY        7,500.            5.00    37,500.00
   14.00    105,000.00       7.50     56,250.00
  203.49    WETLAND SOIL EXCAVATION CY        2,300.            5.00    11,500.00
    9.00    20,700.00        7.50     17,250.00
  203.52    IMPERVIOUS MATERIAL (F) CY        5,950.           12.40    73,780.00
    9.00    53,550.00       20.00    119,000.00
  203.5525  PORTABLE CHANGEABLE
             MESSAGE SIGN PLATFORM  U             6.        1,360.00     8,160.00
  850.00     5,100.00      800.00     4,800.00
  203.5554  GUARDRAIL 50' EAGRT
             PLATFORM               U             7.        1,000.00     7,000.00
1,200.00     8,400.00    1,800.00    12,600.00
     13933E

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 ITEM                                                                           A
          B                          C
  NO.         DESCRIPTION          UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
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  203.5555  GUARDRAIL 25' EAGRT
             PLATFORM               U             4.          690.00     2,760.00
1,100.00     4,400.00      800.00     3,200.00
  203.6     EMBANKMENT-IN-PLACE (F) CY      345,200.            4.35 1,501,620.00
    4.00 1,380,800.00        3.75 1,294,500.00
  203.62    EMBANKMENT IN PLACE
             REPLACEMENT MATERIAL   CY          750.            7.00     5,250.00
    4.00     3,000.00       14.00    10,500.00
  206.1     COMMON STRUCTURE
             EXCAVATION             CY        5,400.           44.00   237,600.00
   12.00    64,800.00       22.00    118,800.00
  206.19    COMMON STRUCTURE
             EXCAVATION EXPLORATORY CY          505.           32.75    16,538.75
   60.00    30,300.00       90.00    45,450.00
  206.2     ROCK STRUCTURE EXCAVATION CY      1,050.           14.00    14,700.00
   80.00    84,000.00       70.00    73,500.00
  209.1     GRANULAR BACKFILL       CY          290.           28.30     8,207.00
   30.00     8,700.00       38.00    11,020.00
  209.201   GRANULAR BACKFILL
             (BRIDGE) (F)           CY          795.           27.00    21,465.00
   40.00    31,800.00       28.00    22,260.00
  209.5     GRANULAR BACKFILL FOR
             MSE WALLS              CY       12,760.           25.75   328,570.00
   50.00    638,000.00      24.00    306,240.00
  214.      FINE GRADING            U             1.      635,000.00   635,000.00
400,000.00   400,000.00   280,000.00   280,000.00
  214.41    FINE GRADING- WETLAND
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            MITIGATION                 U              1.      42,000.00   42,000.00
45,000.00   45,000.00   100,000.00  100,000.00
 304.1     SAND (F)                  CY      84,600.          20.75 1,755,450.00
   20.00 1,692,000.00        20.00 1,692,000.00
 304.11    SAND FOR SHIMMING         CY       2,000.          23.00   46,000.00
   21.00   42,000.00         22.00   44,000.00
 304.32    CRUSHED GRAVEL FOR
            SHOULDER LEVELING        TON        300.          27.40    8,220.00
   21.00    6,300.00         18.00    5,400.00
 304.4     CRUSHED STONE (FINE
            GRADATION) (F)           CY      66,050.          18.50 1,221,925.00
   23.00 1,519,150.00        19.25 1,271,462.50
 304.41    CRUSHED STONE (FINE
            GRADATION) FOR SHIM      CY       4,500.          21.25   95,625.00
   24.00   108,000.00        24.00   108,000.00
 304.45    CRUSHED STONE (FINE
            GRADATION) FOR DRIVES    CY          45.          31.00    1,395.00
   24.00    1,080.00         30.00    1,350.00
 304.5     CRUSHED STONE (COARSE
            GRADATION) (F)           CY      29,717.          17.60   523,019.20
   23.00   683,491.00        21.50   638,915.50
 403.11    HOT BITUMINOUS PAVEMENT,
            MACHINE METHOD           TON      1,192.          85.80   102,273.60
   75.00   89,400.00         62.00   73,904.00
 403.11001 HOT BITUMINOUS PAVEMENT,
            MACHINE METHOD
            (QC/QA TIER 1)           TON     76,400.          63.80 4,874,320.00
   60.00 4,584,000.00        62.00 4,736,800.00
 403.1109  HOT BITUMNOUS PAVEMENT
            MACHINE METHOD HIGH
            STRENGTH                 TON      2,550.          77.55   197,752.50
   70.00   178,500.00        71.00   181,050.00
 403.12    HOT BITUMINOUS PAVEMENT,
            HAND METHOD              TON        610.         117.70   71,797.00
  105.00   64,050.00        117.00   71,370.00
 403.4     MATERIAL TRANSFER VEHICLE
            (MTV)                    TON     79,800.           1.35   107,730.00
    1.00   79,800.00          1.25   99,750.00
 403.6     PAVEMENT JOINT ADHESIVE   LF     188,500.            .28   52,780.00
     .22   41,470.00           .25   47,125.00
 403.911   HOT BITUMINOUS BRIDGE
            PAVEMENT, 1" BASE
            COURSE (F)               TON        136.         160.00   21,760.00
  145.00   19,720.00        210.00   28,560.00
 403.98    HOT BITUMINOUS CONCRETE
            LEVELING, MACHINE METHOD TON        905.          90.00   81,450.00
   80.00   72,400.00         62.00   56,110.00
 403.99    TEMPORARY BITUMINOUS
            PAVEMENT                 TON     12,800.          73.50   940,800.00
   60.00   768,000.00        65.00   832,000.00
     13933E

     PAGE  3
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 ITEM                                                                         A
           B                       C
  NO.         DESCRIPTION         UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
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 415.8     GLASS FIBER PAVEMENT
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             REINFORCING MESH         SY       1,750.            15.40    26,950.00
    9.00    15,750.00        14.00    24,500.00
  417.      COLD PLANING BITUMINOUS
             SURFACES                 SY      19,000.             2.90    55,100.00
    1.75    33,250.00         2.75    52,250.00
  501.1     TEMPORARY BRIDGE          U              1.     585,000.00   585,000.00
600,000.00   600,000.00   800,000.00   800,000.00
  502.101   REMOVAL OF EXISTING
             BRIDGE STRUCTURE         U              1.     137,900.00   137,900.00
165,000.00   165,000.00   120,000.00   120,000.00
  502.102   REMOVAL OF EXISTING
             BRIDGE STRUCTURE         U              1.     130,360.00   130,360.00
160,000.00   160,000.00   120,000.00   120,000.00
  503.101   WATER DIVERSION STRUCTURE U              1.       3,800.00     3,800.00
   500.00       500.00    80,000.00    80,000.00
  503.102   WATER DIVERSION STRUCTURE U              1.      10,335.00    10,335.00
3,500.00     3,500.00     7,000.00     7,000.00
  503.103   WATER DIVERSION STRUCTURE U              1.      12,935.00    12,935.00
2,100.00     2,100.00     8,000.00     8,000.00
  503.104   WATER DIVERSION STRUCTURE U              1.      11,335.00    11,335.00
2,800.00     2,800.00    16,000.00    16,000.00
  503.105   WATER DIVERSION STRUCTURE U              1.       4,115.00     4,115.00
2,200.00     2,200.00     5,000.00     5,000.00
  503.106   WATER DIVERSION STRUCTURE U              1.       4,115.00     4,115.00
1,100.00     1,100.00     7,000.00     7,000.00
  503.107   WATER DIVERSION STRUCTURE U              1.       4,115.00     4,115.00
2,100.00     2,100.00     7,000.00     7,000.00
  503.108   WATER DIVERSION STRUCTURE U              1.      10,235.00    10,235.00
1,200.00     1,200.00     6,000.00     6,000.00
  503.110   WATER DIVERSION STRUCTURE U              1.       8,230.00     8,230.00
2,000.00     2,000.00     6,000.00     6,000.00
  503.201   COFFERDAMS                U              1.     113,300.00   113,300.00
178,000.00   178,000.00   240,000.00   240,000.00
  503.202   COFFERDAMS                U              1.     246,000.00   246,000.00
280,000.00   280,000.00   330,000.00   330,000.00
  503.203   COFFERDAMS                U              1.      53,130.00    53,130.00
120,000.00   120,000.00   160,000.00   160,000.00
  503.204   COFFERDAMS                U              1.       3,100.00     3,100.00
8,500.00     8,500.00    17,000.00    17,000.00
  503.205   COFFERDAMS                U              1.       3,100.00     3,100.00
2,000.00     2,000.00    17,000.00    17,000.00
  503.206   COFFERDAMS                U              1.       3,100.00     3,100.00
2,500.00     2,500.00    17,000.00    17,000.00
  503.207   COFFERDAMS                U              1.       3,100.00     3,100.00
2,500.00     2,500.00    17,000.00    17,000.00
  503.208   COFFERDAMS                U              1.       3,100.00     3,100.00
1,000.00     1,000.00    17,000.00    17,000.00
  503.209   COFFERDAMS                U              1.       3,100.00     3,100.00
   500.00       500.00    22,000.00    22,000.00
  503.210   COFFERDAMS                U              1.       3,650.00     3,650.00
1,400.00     1,400.00    10,000.00    10,000.00
  503.211   COFFERDAMS                U              1.       3,650.00     3,650.00
9,500.00     9,500.00    10,000.00    10,000.00
  503.212   COFFERDAMS                U              1.       3,650.00     3,650.00
3,800.00     3,800.00    10,000.00    10,000.00
  503.213   COFFERDAMS                U              1.      39,630.00    39,630.00
26,500.00    26,500.00    16,000.00    16,000.00
  503.214   COFFERDAMS                U              1.       3,100.00     3,100.00
1,400.00     1,400.00    17,000.00    17,000.00
  503.215   COFFERDAMS                U              1.      17,710.00    17,710.00
25,000.00    25,000.00    28,000.00    28,000.00
  503.216   COFFERDAMS                U              1.       3,650.00     3,650.00
3,900.00     3,900.00     9,000.00     9,000.00
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 504.101   COMMON BRIDGE EXCAVATION  CY      10,740.          5.20    55,848.00
   14.20   152,508.00         7.00    75,180.00
 504.2     ROCK BRIDGE EXCAVATION    CY         365.        165.00    60,225.00
  150.00   54,750.00         50.00    18,250.00
 506.4     STEEL SHEETING FOR
            ENVIRONMENTAL
            PROTECTION               SY       4,750.         88.00   418,000.00
   95.00   451,250.00       105.00   498,750.00
 508.      STRUCTURAL FILL           CY         740.         40.00    29,600.00
   44.00   32,560.00         30.00    22,200.00
 509.1     MOBILIZATION AND DEMOBIL
            OF DRILLED SHAFT
            DRILLING EQUIPMENT       U            1.      5,500.00     5,500.00
28,000.00    28,000.00    12,000.00    12,000.00
 509.2     DRILLED SHAFT             LF       1,476.        240.00   354,240.00
  200.00   295,200.00       240.00   354,240.00
 509.3     OBSTRUCTION REMOVAL       LF          77.        220.00    16,940.00
   80.00     6,160.00       160.00    12,320.00
 520.01    CONCRETE CLASS AA         CY          16.        835.00    13,360.00
1,500.00   24,000.00     1,900.00     30,400.00
 520.01127 CONCRETE CLASS AA (QC/QA)
            (PRECAST OPTION) (F)     CY         158.        687.00   108,546.00
1,100.00   173,800.00       600.00    94,800.00
 520.01227 CONCRETE CLASS AA (QC/QA)
            (PRECAST OPTION) (F)     CY         161.        670.00   107,870.00
1,100.00   177,100.00       600.00    96,600.00
     13933E

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 ITEM                                                                      A
          B                          C
  NO.         DESCRIPTION           UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
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 520.03017 CONCRETE CLASS AA
            (PRECAST OPTION) (F)     CY          92.        410.00    37,720.00
  450.00   41,400.00        300.00    27,600.00
 520.03027 CONCRETE CLASS AA
            (PRECAST OPTION) (F)     CY          90.        410.00    36,900.00
  450.00   40,500.00        300.00    27,000.00
 520.03127 CONCRETE CLASS AA,
            APPROACH SLABS (QC/QA)
            (PRECAST OPTION) (F)     CY         136.        312.00    42,432.00
  340.00   46,240.00        280.00    38,080.00
 520.03227 CONCRETE CLASS AA,
            APPROACH SLABS (QC/QA)
            (PRECAST OPTION) (F)     CY         136.        285.00    38,760.00
  340.00   46,240.00        280.00    38,080.00
 520.1     CONCRETE CLASS A          CY         110.      1,090.00   119,900.00
1,200.00   132,000.00       800.00    88,000.00
 520.12    CONCRETE CLASS A,
            ABOVE FOOTINGS (F)       CY         197.        670.00   131,990.00
  745.00   146,765.00       670.00   131,990.00
 520.2     CONCRETE CLASS B          CY         340.        275.00    93,500.00
  271.00   92,140.00        280.00    95,200.00
 520.213   CONCRETE CLASS B,
            FOOTINGS (ON SOIL) (F)   CY         180.        428.00    77,040.00
  400.00   72,000.00        290.00    52,200.00
 520.71026 CONCRETE BRIDGE DECK
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            (QC/QA)
            (PANEL OPTION) (F)       CY         355.          862.00   306,010.00
  800.00   284,000.00       820.00   291,100.00
 520.72026 CONCRETE BRIDGE DECK
            (QC/QA)
            (PANEL OPTION) (F)       CY         361.          795.00   286,995.00
  750.00   270,750.00       750.00   270,750.00
 534.3     WATER REPELLENT (SILANE/
            SILOXANE)                GAL        225.           63.00    14,175.00
   68.00   15,300.00         90.00    20,250.00
 538.2     BARRIER MEMBRANE, PEEL AN
            STICK - VERTICAL SURFACE
            (F)                      SY         114.           63.00     7,182.00
   55.00    6,270.00         60.00    6,840.00
 538.5     BARRIER MEMBRANE, HEAT
            WELDED (F)               SY         110.           24.75     2,722.50
   24.00    2,640.00         22.00    2,420.00
 538.6     BARRIER MEMBRANE,
            HEAT WELDED -
            MACHINE METHOD (F)       SY       2,420.           24.75    59,895.00
   18.50   44,770.00         24.00    58,080.00
 541.2     PVC WATERSTOPS, NH
            TYPE 2 (F)               LF          48.           17.00       816.00
    6.00      288.00         18.00       864.00
 541.3     PVC WATERSTOPS, NH
            TYPE 3 (F)               LF          38.           18.20       691.60
    8.00      304.00         22.00       836.00
 541.5     PVC WATERSTOPS, NH
            TYPE 5 (F)               LF         308.           19.20     5,913.60
   10.00    3,080.00         18.00    5,544.00
 544.1     REINFORCING STEEL
            (ROADWAY)                LB      42,150.            1.16    48,894.00
     .95   40,042.50          1.50    63,225.00
 544.3     REINFORCING STEEL
            (CONTRACTOR DETAILED)    LB      30,000.            1.18    35,400.00
    1.22   36,600.00          1.10    33,000.00
 544.31    REINFORCING STEEL, EPOXY
            COATED (CONTRACTOR
            DETAILED)                LB     313,020.            1.45   453,879.00
    1.25   391,275.00          .90   281,718.00
 544.7     SYNTHETIC FIBER
            REINFORCEMENT (F)        LB       1,904.           11.00    20,944.00
    6.00   11,424.00          9.00    17,136.00
 547.      SHEAR CONNECTORS (F)      EA       7,232.            5.25    37,968.00
    5.60   40,499.20          5.50    39,776.00
 548.21    ELASTOMERIC BEARING
            ASSEMBLIES (F)           EA          16.        1,200.00    19,200.00
  930.00   14,880.00      1,500.00    24,000.00
    13933E

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 ITEM                                                                        A
           B                       C
  NO.         DESCRIPTION         UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
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 548.22    ELASTOMERIC BEARING
            ASSEMBLIES (F)           EA          16.        1,200.00    19,200.00
  930.00   14,880.00      1,500.00    24,000.00
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550.101     STRUCTURAL STEEL (F)       LB    376,570.          1.90   715,483.00
   1.52     572,386.40         2.00    753,140.00
550.102     STRUCTURAL STEEL (F)       LB    371,110.          1.87   693,975.70
   1.54     571,509.40         2.00    742,220.00
559.4       ELASTOMERIC PLUG TYPE
             EXPANSION JOINT (F)       LF        492.        119.00    58,548.00
 108.00     53,136.00        115.00     56,580.00
562.1       SILICONE JOINT SEALANT (F LF        575.         32.00    18,400.00
  13.00      7,475.00         12.00      6,900.00
563.22      BRIDGE RAIL T2 (F)         LF        465.        107.00    49,755.00
 100.00     46,500.00        130.00     60,450.00
563.23      BRIDGE RAIL T3 (F)         LF        155.        138.00    21,390.00
 126.00     19,530.00        170.00     26,350.00
563.24      BRIDGE RAIL T4 (F)         LF        251.        182.00    45,682.00
 170.00     42,670.00        230.00     57,730.00
563.923     SNOW SCREENING FOR
             OVERPASS STRUCTURES
             T2 OR T4 RAIL             LF        608.         41.75    25,384.00
  38.00     23,104.00         50.00     30,400.00
565.222     BRIDGE APPROACH RAIL T2
             (STEEL POSTS)             U           6.      4,890.00    29,340.00
4,400.00    26,400.00      4,500.00     27,000.00
565.2325    BRIDGE APPROACH RAIL T3
             (STEEL POSTS) SPECIAL
             WITH RESET                U           2.      5,815.00    11,630.00
7,500.00    15,000.00      5,000.00     10,000.00
565.242     BRIDGE APPROACH RAIL T4
             (STEEL POSTS)             U           2.      6,170.00    12,340.00
5,600.00    11,200.00      5,600.00     11,200.00
570.4       MORTAR RUBBLE MASONRY (F) CY           6.        550.00     3,300.00
 400.00      2,400.00        500.00      3,000.00
582.1       SLOPE PAVING WITH
             CONCRETE (F)              SY        354.         44.00    15,576.00
  40.00     14,160.00         60.00     21,240.00
585.2       STONE FILL, CLASS B        CY      6,500.         16.50   107,250.00
  31.00     201,500.00        36.00    234,000.00
585.25      COBBLE-GRAVEL-SAND         CY        770.         46.00    35,420.00
  36.00     27,720.00         32.00     24,640.00
585.3       STONE FILL, CLASS C        CY      5,250.         28.70   150,675.00
  35.00     183,750.00        36.00    189,000.00
585.4       STONE FILL, CLASS D        CY         10.         28.80       288.00
  35.00        350.00         60.00        600.00
585.5       STONE FILL, CLASS E        CY        755.         30.00    22,650.00
  33.00     24,915.00         36.00     27,180.00
585.7       STONE FILL, CLASS G        CY        285.         30.00     8,550.00
  33.00      9,405.00         46.00     13,110.00
592.11      MECHANICALLY STABILIZED
             EARTH RETAINING WALL      SF      7,600.         42.00   319,200.00
  46.00     349,600.00        49.00    372,400.00
592.111     MECHANICALLY STABILIZED
             EARTH RETURN WALLS
             LEFT-IN-PLACE             U           1.     80,000.00    80,000.00
150,000.00   150,000.00    85,000.00    85,000.00
592.12      MECHANICALLY STABILIZED
             EARTH RETAINING WALL      SF      7,509.         44.65   335,276.85
  46.00     345,414.00        49.00    367,941.00
593.411     GEOTEXTILE; PERM CONTROL
             CL.1, NON-WOVEN           SY     17,300.          1.75    30,275.00
   4.00     69,200.00          2.50     43,250.00
594.2       WOOD PANEL SOUND
             ABATEMENT WALL            SF     31,214.         21.65   675,783.10
  20.00     624,280.00        27.00    842,778.00
603.0001    VIDEO INSPECTION           LF     23,300.           .95    22,135.00
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ITEM NO.	DESCRIPTION	UNIT	QUANTITY	A UNIT PRICE	A TOTAL	B UNIT PRICE	B TOTAL	C UNIT PRICE	C TOTAL
						.70	16,310.00	1.00	23,300.00
603.00215	15" R.C. PIPE, 2000D	LF	1,250.	43.00	53,750.00	45.00	56,250.00	32.00	40,000.00
603.00218	18" R.C. PIPE, 2000D	LF	470.	46.00	21,620.00	50.00	23,500.00	39.00	18,330.00
603.00224	24" R.C. PIPE, 2000D	LF	840.	52.70	44,268.00	55.00	46,200.00	52.00	43,680.00
603.00230	30" R.C. PIPE, 2000D	LF	390.	63.80	24,882.00	65.00	25,350.00	60.00	23,400.00
603.00236	36" R.C. PIPE, 2000D	LF	175.	76.00	13,300.00	80.00	14,000.00	80.00	14,000.00
603.00242	42" R.C. PIPE, 2000D	LF	97.	100.00	9,700.00	100.00	9,700.00	110.00	10,670.00
603.00248	48" R.C. PIPE, 2000D	LF	105.	114.00	11,970.00	125.00	13,125.00	100.00	10,500.00
603.00254	54" R.C. PIPE, 2000D	LF	72.	143.00	10,296.00	170.00	12,240.00	150.00	10,800.00
603.00260	60" R.C. PIPE, 2000D	LF	55.	172.00	9,460.00	260.00	14,300.00	240.00	13,200.00

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ITEM NO.	DESCRIPTION	UNIT	QUANTITY	A UNIT PRICE	A TOTAL	B UNIT PRICE	B TOTAL	C UNIT PRICE	C TOTAL
603.00315	15" R.C. PIPE, 3000D	LF	28.	43.35	1,213.80	46.00	1,288.00	40.00	1,120.00
603.00324	24" R.C. PIPE, 3000D	LF	65.	56.20	3,653.00	64.00	4,160.00	75.00	4,875.00
603.00330	30" R.C. PIPE, 3000D	LF	52.	68.00	3,536.00	80.00	4,160.00	75.00	3,900.00
603.00424	24" R.C. PIPE, 3750D	LF	61.	56.50	3,446.50	75.00	4,575.00	52.00	3,172.00
603.00430	30" R.C. PIPE, 3750D	LF	30.	76.50	2,295.00	110.00	3,300.00	70.00	2,100.00
603.30115	15" R.C. END SECTIONS	EA	2.	720.00	1,440.00	500.00	1,000.00	650.00	1,300.00
603.30124	24" R.C. END SECTIONS	EA	4.	360.00	1,440.00	600.00	2,400.00	900.00	3,600.00
603.36115	15" ALUMINIZED STEEL END SECTION	EA	12.	495.00	5,940.00	300.00	3,600.00	400.00	4,800.00
603.36118	18" ALUMINIZED STEEL END SECTION	EA	9.	530.00	4,770.00	325.00	2,925.00	400.00	3,600.00
603.36130	30" ALUMINIZED STEEL END SECTION	EA	1.	800.00	800.00	500.00	500.00	650.00	650.00
603.36136	36" ALUMINIZED STEEL END SECTION	EA	1.	1,080.00	1,080.00	725.00	725.00	900.00	900.00
603.82115	15" PE PIPE (TYPE C)	LF	97.	35.50	3,443.50	35.00	3,395.00	38.00	3,686.00
603.82124	24" PE PIPE (TYPE C)	LF	97.	44.75	4,340.75	45.00	4,365.00	55.00	5,335.00
603.82212	12" PE PIPE (TYPE S)	LF	64.	35.00	2,240.00	34.00	2,176.00	25.00	1,600.00

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 603.82215 15" PE PIPE (TYPE S)      LF      14,200.          35.20  499,840.00
   35.00   497,000.00         26.00   369,200.00
 603.82218 18" PE PIPE (TYPE S)      LF       3,250.          39.00  126,750.00
   43.00   139,750.00         30.00    97,500.00
 603.82224 24" PE PIPE (TYPE S)      LF       2,400.          44.00  105,600.00
   45.00   108,000.00         44.00   105,600.00
 603.82230 30" PE PIPE (TYPE S)      LF         945.          54.00   51,030.00
   60.00    56,700.00         62.00    58,590.00
 603.82236 36" PE PIPE (TYPE S)      LF         265.          62.50   16,562.50
   65.00    17,225.00         80.00    21,200.00
 603.82248 48" PE PIPE (TYPE S)      LF          51.         100.00    5,100.00
   90.00     4,590.00        130.00     6,630.00
 604.0007  POLYETHYLENE LINER        EA         135.         225.00   30,375.00
  215.00    29,025.00        200.00    27,000.00
 604.12    CATCH BASINS TYPE B       U          100.       1,760.00  176,000.00
2,100.00   210,000.00      1,700.00   170,000.00
 604.125   CATCH BASINS TYPE B,
            5-FOOT DIAMETER          U            8.       2,250.00   18,000.00
3,000.00    24,000.00      2,200.00    17,600.00
 604.1252  CATCH BASINS TYPE B,
            5-FOOT DIA. DOUBLE GRATE U           18.       2,250.00   40,500.00
3,500.00    63,000.00      2,600.00    46,800.00
 604.126   CATCH BASINS TYPE B,
            6-FOOT DIAMETER          U            4.       2,882.00   11,528.00
3,600.00    14,400.00      2,800.00    11,200.00
 604.128   CATCH BASINS TYPE B,
            8-FOOT DIAMETER          U            2.       7,590.00   15,180.00
8,000.00    16,000.00      4,400.00     8,800.00
 604.15    CATCH BASINS TYPE E       U           52.       1,985.00  103,220.00
2,350.00   122,200.00      1,900.00    98,800.00
 604.155   CATCH BASINS TYPE E,
            5-FOOT DIAMETER          U            7.       2,480.00   17,360.00
3,250.00    22,750.00      2,900.00    20,300.00
 604.156   CATCH BASINS TYPE E,
            6-FOOT DIAMETER          U            5.       3,100.00   15,500.00
3,800.00    19,000.00      2,900.00    14,500.00
 604.158   CATCH BASINS TYPE E,
            8-FOOT DIAMETER          U            2.       7,815.00   15,630.00
8,500.00    17,000.00      4,900.00     9,800.00
 604.16    CATCH BASINS TYPE F       U           24.       1,880.00   45,120.00
2,250.00    54,000.00      1,800.00    43,200.00
 604.165   CATCH BASINS TYPE F,
            5-FOOT DIAMETER          U            2.       2,375.00    4,750.00
3,150.00     6,300.00      2,500.00     5,000.00
 604.166   CATCH BASINS TYPE F,
            6-FOOT DIAMETER          U            2.       3,000.00    6,000.00
3,700.00     7,400.00      1,900.00     3,800.00
 604.22    DROP INLETS TYPE B        U            8.       1,370.00   10,960.00
1,450.00    11,600.00      1,300.00    10,400.00
 604.25    DROP INLETS TYPE E        U            6.       1,555.00    9,330.00
1,650.00     9,900.00      1,300.00     7,800.00
 604.32    DRAINAGE MANHOLES         U           24.       1,826.00   43,824.00
2,300.00    55,200.00      1,900.00    45,600.00
 604.325   DRAINAGE MANHOLES,
            5-FOOT DIAMETER          U            8.       2,320.00   18,560.00
3,100.00    24,800.00      2,600.00    20,800.00
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 ITEM                                                                   A
           B                        C
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NO.	DESCRIPTION	UNIT	QUANTITY	UNIT PRICE	TOTAL	UNIT PRICE	TOTAL	UNIT PRICE	TOTAL
604.326	DRAINAGE MANHOLES, 6-FOOT DIAMETER	U	2.	2,945.00	5,890.00	3,600.00	7,200.00	2,000.00	4,000.00
604.4	RECONSTRUCTING/ADJUSTING CATCH BASIN & DROP INLET	LF	2.	270.00	540.00	350.00	700.00	1,500.00	3,000.00
604.51	RECONSTRUCTING/ADJUSTING SEWER MANHOLES	LF	4.	270.00	1,080.00	400.00	1,600.00	400.00	1,600.00
604.54	RECONSTRUCTING/ADJUSTING TELEPHONE MANHOLES	LF	8.	270.00	2,160.00	400.00	3,200.00	500.00	4,000.00
604.62	DRAINAGE MANHOLE COVERS AND FRAMES	EA	1.	630.00	630.00	550.00	550.00	600.00	600.00
604.75	GRATES & FRAMES, TYPE E	EA	2.	790.00	1,580.00	610.00	1,220.00	700.00	1,400.00
604.9101	OUTLET CONTROL STRUCTURE	U	1.	3,670.00	3,670.00	4,200.00	4,200.00	7,000.00	7,000.00
604.9102	OUTLET CONTROL STRUCTURE	U	1.	3,390.00	3,390.00	3,000.00	3,000.00	5,200.00	5,200.00
604.9103	OUTLET CONTROL STRUCTURE	U	1.	3,670.00	3,670.00	3,100.00	3,100.00	5,200.00	5,200.00
604.9104	OUTLET CONTROL STRUCTURE	U	1.	4,330.00	4,330.00	4,500.00	4,500.00	7,000.00	7,000.00
604.9105	OUTLET CONTROL STRUCTURE	U	1.	3,885.00	3,885.00	3,800.00	3,800.00	7,000.00	7,000.00
604.9113	OUTLET CONTROL STRUCTURE	U	2.	11,910.00	23,820.00	11,000.00	22,000.00	18,000.00	36,000.00
604.9127	OUTLET CONTROL STRUCTURE	U	1.	13,215.00	13,215.00	9,500.00	9,500.00	17,000.00	17,000.00
604.9128	OUTLET CONTROL STRUCTURE	U	1.	17,890.00	17,890.00	12,500.00	12,500.00	21,000.00	21,000.00
604.921	LEACHING CHAMBER TYPE 1	U	4.	820.00	3,280.00	2,100.00	8,400.00	1,800.00	7,200.00
604.922	LEACHING CHAMBER TYPE 2	U	4.	820.00	3,280.00	2,100.00	8,400.00	1,800.00	7,200.00
605.506	6" PERF. CORR. POLYETHYL PIPE UND.	LF	23,900.	21.45	512,655.00	19.00	454,100.00	18.00	430,200.00
605.508	8" PERFORATED CORRUGATED POLYETHYLENE PIPE UND.	LF	2,800.	22.50	63,000.00	20.00	56,000.00	23.00	64,400.00
605.79	UNDERDRAIN FLUSHING BASINS	EA	47.	510.00	23,970.00	350.00	16,450.00	700.00	32,900.00
605.798	UNDERDRAIN FLUSHING BASINS, 8"	EA	9.	565.00	5,085.00	425.00	3,825.00	750.00	6,750.00
605.82251	24"AGGREGATE UNDERDRAIN								

	TYPE 2 WITH 6" PERF. CORR. POLYETHYL PIPE	LF	6,700.	24.20	162,140.00	30.00	201,000.00	21.00	140,700.00
605.82258	24" AGGREGATE UNDERDRAIN TYPE 2, WITH 8" PERF. CORR. POYLETHY PIPE	LF	4,300.	25.25	108,575.00	31.00	133,300.00	24.00	103,200.00
606.0194	CABLE RAIL POST ASSEMBLY WITH SOIL PLATE - 4 STRAND (SUPPLY TO DEPT)	EA	220.	59.40	13,068.00	45.00	9,900.00	47.00	10,340.00
606.01941	CABLE RAIL COMMON POST ASSEMBLY W/SOIL PLATE 4 STRAND (SUPPLY TO DEPT)	EA	2.	59.40	118.80	45.00	90.00	50.00	100.00
606.01942	CABLE RAIL ANCHOR ASSEMBLY (4 STRAND) (SUPPLY TO DEPT)	EA	8.	2,000.00	16,000.00	1,600.00	12,800.00	1,700.00	13,600.00

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ITEM NO.	DESCRIPTION	UNIT	QUANTITY	A UNIT PRICE	A TOTAL	B UNIT PRICE	B TOTAL	C UNIT PRICE	C TOTAL
606.120	BEAM GUARDRAIL (STANDARD SECTION-STEEL POST)	LF	9,100.	19.00	172,900.00	17.00	154,700.00	18.00	163,800.00
606.141	BEAM GUARDRAIL (CURVED W/CRT POSTS)	LF	41.	26.70	1,094.70	20.00	820.00	22.00	902.00
606.1454	BEAM GUARDRAIL (TERM. UNIT TYPE EAGRT 50 FT.)	U	7.	2,065.00	14,455.00	2,100.00	14,700.00	2,100.00	14,700.00
606.1455	BEAM GUARDRAIL (TERM. UNIT TYPE EAGRT 25 FT.)	U	4.	1,845.00	7,380.00	1,900.00	7,600.00	1,900.00	7,600.00
606.147	BEAM GUARDRAIL (TERMINAL UNIT TYPE G-2)	U	8.	563.00	4,504.00	500.00	4,000.00	600.00	4,800.00
606.417	PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL	LF	14,100.	28.00	394,800.00	30.00	423,000.00	31.00	437,100.00
606.4175	PORTABLE CONCRETE BARRIER FOR TRAFFIC CONTROL - ANCHORED	LF	600.	51.00	30,600.00	45.00	27,000.00	60.00	36,000.00
606.72104	CABLE MEDIAN RAIL LOW TENSION, (4 STRAND)	LF	3,700.	15.80	58,460.00	12.50	46,250.00	13.00	48,100.00
606.72114	ANCHORAGE UNIT FOR LOW TENSION CABLE MEDIAN RAIL (4 STRAND)	U	10.	2,664.00	26,640.00	2,400.00	24,000.00	2,500.00	25,000.00
606.93	TEMPORARY BEAM GUARDRAIL	LF	2,300.	12.35	28,405.00	11.50	26,450.00	13.00	29,900.00
606.9523	TEMP. IMPACT ATTENUATION								

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             DEVICE (NON-REDIRECTIVE)
             TEST LEVEL 3             U           11.       5,325.00    58,575.00
4,200.00    46,200.00     7,500.00    82,500.00
 606.9612  TEMPORARY GUARDRAIL TO
             BARRIER TRANSITION,
             STEEL POST               U            2.       1,860.00     3,720.00
2,300.00     4,600.00     3,000.00     6,000.00
 607.1     WOVEN WIRE FENCE           LF       3,650.           6.90    25,185.00
    8.00    29,200.00         7.00    25,550.00
 607.350   CHAIN LINK FENCE WITH
             VINYL COATED STEEL
             FABRIC, 5' HIGH          LF       6,250.          21.30   133,125.00
   17.00   106,250.00        16.50   103,125.00
 607.360   CHAIN LINK FENCE WITH
             VINYL COATED STEEL
             FABRIC, 6' HIGH          LF       4,500.          22.70   102,150.00
   17.00    76,500.00        18.00    81,000.00
 607.41    POST ASSEMBLIES FOR
             WOVEN WIRE FENCE         EA          20.         160.00     3,200.00
  160.00     3,200.00       175.00     3,500.00
 607.4350  POST ASSEMBLIES FOR CHAIN
             LINK FENCE WITH VINYL
             CTD STL FABRIC, 5' HIGH  EA          23.         174.00     4,002.00
  155.00     3,565.00       175.00     4,025.00
 607.4360  POST ASSEMBLIES FOR CHAIN
             LINK FENCE WITH VINYL
             CTD STL FABRIC, 6' HIGH  EA          36.         188.00     6,768.00
  165.00     5,940.00       185.00     6,660.00
 607.73610 10' OPENING DOUBLE GATE,
             CHAIN LINK VINYL COATED
             STEEL FABRIC, 6' HIGH    U            1.       1,300.00     1,300.00
1,150.00     1,150.00     2,200.00     2,200.00
 607.73618 18' OPENING DOUBLE GATE,
             CHAIN LINK VINYL COATED
             STEEL FABRIC, 6' HIGH    U            2.       1,676.00     3,352.00
1,500.00     3,000.00     2,300.00     4,600.00
 608.12    2" BITUMINOUS
             SIDEWALK (F)             SY       2,050.          14.00    28,700.00
   13.00    26,650.00        15.00    30,750.00
    13933E

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 ITEM                                                                      A
             B                        C
  NO.          DESCRIPTION           UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
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 608.24    4" CONCRETE SIDEWALK (F)  SY          95.          37.00     3,515.00
   34.00     3,230.00        70.00     6,650.00
 608.26    6" CONCRETE SIDEWALK (F)  SY       8,700.          40.70   354,090.00
   37.00   321,900.00        40.00   348,000.00
 608.28    8" CONCRETE SIDEWALK (F)  SY         600.          44.00    26,400.00
   40.00    24,000.00        43.00    25,800.00
 608.54    DETECTABLE WARNING
             DEVICES, CAST IRON       SY          16.         412.50     6,600.00
  375.00     6,000.00       400.00     6,400.00
 609.01    STRAIGHT GRANITE CURB      LF       1,850.          19.60    36,260.00
   15.90    29,415.00        16.00    29,600.00
 609.02    CURVED GRANITE CURB        LF         155.          30.25     4,688.75
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  25.00     3,875.00        32.00     4,960.00
 609.21    STRAIGHT GRANITE SLOPE
            CURB                      LF       2,000.          13.90   27,800.00
  11.50    23,000.00        15.00    30,000.00
 609.216   STRAIGHT GRANITE SLOPE
            CURB 6" HIGH              LF       4,000.          17.00   68,000.00
  14.00    56,000.00        17.00    68,000.00
 609.22    STRAIGHT GRANITE SLOPE
            CURB WITH RADIAL JOINTS   LF          84.          13.90    1,167.60
  12.00     1,008.00        15.00     1,260.00
 609.23    CURVED GRANITE SLOPE
            CURB                      LF           8.          78.00      624.00
  60.00       480.00        85.00       680.00
 609.236   CURVED GRANITE SLOPE CURB
            6" HIGH                   LF          85.          78.00    6,630.00
  60.00     5,100.00        80.00     6,800.00
 609.5     RESET GRANITE CURB         LF      12,180.          10.00  121,800.00
   4.80    58,464.00         9.00   109,620.00
 609.811   BITUMINOUS CURB, TYPE B
            (4" REVEAL)               LF       8,450.           4.70   39,715.00
   4.00    33,800.00         4.50    38,025.00
 609.924   SPECIAL GRANITE CURB, 12"
            REVEAL                    LF          73.          38.75    2,828.75
  35.00     2,555.00        40.00     2,920.00
 611.05212 12" CEMENT LINED DUCTILE
            IRON WATER PIPE, CL 52    LF         470.          80.00   37,600.00
  60.00    28,200.00        85.00    39,950.00
 611.70012 12" FITTING                EA           2.       1,230.00    2,460.00
1,250.00    2,500.00     1,400.00     2,800.00
 611.71012 12" GATE VALVE             EA           2.       2,430.00    4,860.00
2,500.00    5,000.00     3,400.00     6,800.00
 611.74    CHLORINE INJECTION TAP     EA           2.         352.00      704.00
 500.00     1,000.00       900.00     1,800.00
 611.81    HYDRANTS                   EA           3.       5,000.00   15,000.00
3,600.00   10,800.00     4,000.00    12,000.00
 611.951   WATER MAIN INSULATION      SY          10.          20.00      200.00
  15.00       150.00        20.00       200.00
 612.141   FORCE MAIN SYSTEM
            CONNECTION                U            1.       7,295.00    7,295.00
6,000.00    6,000.00    30,000.00    30,000.00
 612.2524  24" DUCTILE IRON SEWER
            FORCE MAIN CASING         LF          20.          97.50    1,950.00
 180.00     3,600.00       650.00    13,000.00
 612.2708  8" HDPE SEWER FORCE MAIN,
            SDR 15                    LF       4,400.          42.50  187,000.00
  65.00   286,000.00        85.00   374,000.00
 612.3106  SEWER MANHOLE, 6 FOOT
            DIAMETER                  LF          15.         255.00    3,825.00
 420.00     6,300.00       450.00     6,750.00
 612.6     AIR RELEASE VALVE          EA           1.       4,631.00    4,631.00
4,336.00    4,336.00     7,800.00     7,800.00
 612.62512 12" PVC FORCE MAIN SDR 35 LF           10.          45.00      450.00
  70.00       700.00        80.00       800.00
 612.951   SEWER MAIN INSULATION      SY         127.          14.00    1,778.00
  15.00     1,905.00        13.00     1,651.00
 614.321   2" STEEL CONDUIT           LF          53.           8.80      466.40
   8.00       424.00         9.00       477.00
 614.331   3" STEEL CONDUIT           LF         270.          16.50    4,455.00
  15.00     4,050.00        16.00     4,320.00
 614.511   CONCRETE PULL BOX 14"      EA          24.         319.00    7,656.00
 285.00     6,840.00       300.00     7,200.00
 614.512   CONCRETE PULL BOX 18"      EA           8.         357.50    2,860.00
 325.00     2,600.00       330.00     2,640.00
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 614.51821 CONCRETE FIBER OPTIC
           SPLICE VAULT 48"X48"X48" U            9.       2,530.00    22,770.00
2,500.00   22,500.00    2,500.00    22,500.00
 614.51823 CONCRETE FIBER OPTIC
           SPLICE VAULT 48"X96"X48" U            4.       4,070.00    16,280.00
3,950.00   15,800.00    3,900.00    15,600.00
 614.5183  30-INCH CONCRETE FIBER
           OPTIC MANHOLE            EA           7.       2,200.00    15,400.00
1,750.00   12,250.00    2,200.00    15,400.00
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 ITEM                                                                         A
          B                         C
  NO.         DESCRIPTION          UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
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 614.522   MOLDED PULL BOX 13"X24"   EA          32.         319.00    10,208.00
  285.00    9,120.00      300.00     9,600.00
 614.523   MOLDED PULL BOX 17"X30"   EA          26.         341.00     8,866.00
  310.00    8,060.00      320.00     8,320.00
 614.72114 2" PVC CONDUIT,
           SCHEDULE 40              LF         430.           5.50     2,365.00
    5.00    2,150.00        6.00     2,580.00
 614.73114 3" PVC CONDUIT,
           SCHEDULE 40              LF       4,550.           7.15    32,532.50
    6.50   29,575.00        7.50    34,125.00
 614.73118 3" PVC CONDUIT,
           SCHEDULE 80              LF       1,750.          14.30    25,025.00
   13.50   23,625.00       15.00    26,250.00
 614.73214 3" 2-DUCT PVC CONDUIT,
           SCHEDULE 40              LF         110.          11.00     1,210.00
   10.00    1,100.00       12.00     1,320.00
 614.74221 4" 2-DUCT HDPE CONDUIT,
           SDR 13.5                 LF      11,300.          18.70   211,310.00
   17.00  192,100.00       19.00   214,700.00
 615.01    TRAFFIC SIGN TYPE A (F)   SF         189.          43.00     8,127.00
   41.00    7,749.00       42.00     7,938.00
 615.012   TRAFFIC SIGN TYPE A,
           BREAKAWAY MOUNTS (F)     SF         189.          74.00    13,986.00
   55.00   10,395.00       70.00    13,230.00
 615.014   RELOCATING TRAFFIC SIGN,
           TYPE A                   U            6.       6,380.00    38,280.00
7,125.00   42,750.00    6,000.00    36,000.00
 615.02    TRAFFIC SIGN TYPE B (F)   SF          49.          54.00     2,646.00
   54.00    2,646.00       50.00     2,450.00
 615.022   TRAFFIC SIGN TYPE B,
           BREAKAWAY MOUNTS (F)     SF         198.5        124.00    24,614.00
   95.00   18,857.50      115.00    22,827.50
 615.024   RELOCATING TRAFFIC SIGN,
           TYPE B                   U           10.       2,035.00    20,350.00
  715.00    7,150.00    1,900.00    19,000.00
 615.03    TRAFFIC SIGN TYPE C (F)   SF         552.5        120.00    66,300.00
   47.50   26,243.75      120.00    66,300.00
 615.034   RELOCATING TRAFFIC SIGN,
           TYPE C                   U            5.         825.00     4,125.00
  235.00    1,175.00      900.00     4,500.00
 615.04    TRAFFIC SIGN TYPE AA (F)  SF       1,160.          19.60    22,736.00
   17.00   19,720.00       20.00    23,200.00
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615.05	TRAFFIC SIGN TYPE BB (F)	SF	83.2	62.60	5,208.32	11.50	956.80	60.00	4,992.00
615.06	TRAFFIC SIGN TYPE CC (F)	SF	144.2	15.40	2,220.68	9.50	1,369.90	16.00	2,307.20
615.10001	FULL TRAFFIC SIGN STRUCTURE	U	1.	48,400.00	48,400.00	51,000.00	51,000.00	46,000.00	46,000.00
615.10002	FULL TRAFFIC SIGN STRUCTURE	U	1.	52,800.00	52,800.00	57,500.00	57,500.00	50,000.00	50,000.00
615.20001	CANTILEVER TRAFFIC SIGN STRUCTURE	U	1.	38,500.00	38,500.00	36,400.00	36,400.00	37,000.00	37,000.00
615.20002	CANTILEVER TRAFFIC SIGN STRUCTURE	U	1.	39,600.00	39,600.00	37,500.00	37,500.00	38,000.00	38,000.00
615.20003	CANTILEVER TRAFFIC SIGN STRUCTURE	U	1.	41,800.00	41,800.00	37,500.00	37,500.00	40,000.00	40,000.00
615.20004	CANTILEVER TRAFFIC SIGN STRUCTURE	U	1.	41,800.00	41,800.00	37,500.00	37,500.00	40,000.00	40,000.00
616.101	TRAFFIC SIGNALS	U	1.	102,850.00	102,850.00	95,000.00	95,000.00	105,000.00	105,000.00
616.102	TRAFFIC SIGNALS	U	1.	107,800.00	107,800.00	99,000.00	99,000.00	105,000.00	105,000.00
616.103	TRAFFIC SIGNALS	U	1.	112,860.00	112,860.00	105,000.00	105,000.00	110,000.00	110,000.00
616.141	COMPUTER EQUIPMENT AND TRAFFIC SIGNAL SOFTWARE	U	1.	11,550.00	11,550.00	10,000.00	10,000.00	12,000.00	12,000.00
616.151	TRAFFIC SIGNAL (FIBER OPTIC)	U	1.	70,400.00	70,400.00	52,000.00	52,000.00	66,000.00	66,000.00
616.163	TRAFFIC SIGNALS (TEMP.)	U	1.	22,000.00	22,000.00	26,000.00	26,000.00	24,000.00	24,000.00
616.164	TRAFFIC SIGNALS (TEMP.)	U	1.	27,500.00	27,500.00	25,000.00	25,000.00	29,000.00	29,000.00
616.191	ALTERATIONS TO TRAFFIC SIGNALS	U	1.	42,900.00	42,900.00	40,000.00	40,000.00	42,000.00	42,000.00

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ITEM NO.	B DESCRIPTION	C UNIT	QUANTITY	UNIT PRICE	A TOTAL	UNIT PRICE	TOTAL	UNIT PRICE	TOTAL
616.192	ALTERATIONS TO TRAFFIC SIGNALS	U	1.	5,500.00	5,500.00	5,000.00	5,000.00	7,000.00	7,000.00
616.96	WEIGH IN MOTION TRAFFIC STUDY EQUIPMENT	U	2.	68,750.00	137,500.00	65,000.00	130,000.00	63,000.00	126,000.00
618.6	UNIFORMED OFFICERS	$	1.	260,000.00	260,000.00	260,000.00	260,000.00	260,000.00	260,000.00
618.61	UNIFORMED OFFICERS WITH VEHICLE	$	1.	990,000.00	990,000.00				

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990,000.00   990,000.00   990,000.00   990,000.00
 618.7      FLAGGERS                  HR        3,600.           23.65    85,140.00
  17.00     61,200.00        24.00    86,400.00
 619.1      MAINTENANCE OF TRAFFIC    U            1.       345,000.00   345,000.00
400,000.00   400,000.00   500,000.00   500,000.00
 619.25     PORTABLE CHANGEABLE
             MESSAGE SIGN             U            6.        16,500.00    99,000.00
18,000.00   108,000.00   12,000.00    72,000.00
 619.253    PORTABLE CHANGEABLE
             MESSAGE SIGN (UNIT WEEK) UWK        160.           285.00    45,600.00
  300.00    48,000.00       250.00    40,000.00
 619.27     TRAILER-MOUNTED SPEED
             LIMIT SIGN               U            4.         8,800.00    35,200.00
4,500.00    18,000.00     5,100.00    20,400.00
 619.63     TRUCK-MOUNTED IMPACT
             ATTENUATOR, TEST
             LEVEL 3                  U            2.        25,000.00    50,000.00
10,000.00    20,000.00   55,000.00   110,000.00
 621.2      RETROREFLECTIVE BEAM
             GUARDRAIL DELINEATOR     EA          95.             4.85       460.75
   4.00       380.00          4.00       380.00
 621.31     SINGLE DELINEATOR WITH
             POST                     EA         340.            35.20    11,968.00
  23.00     7,820.00         32.00    10,880.00
 621.61     FIBER OPTIC DELINEATOR    EA          29.            55.00     1,595.00
  45.00     1,305.00         50.00     1,450.00
 622.1      STEEL WITNESS MARKERS     EA         130.            29.70     3,861.00
  30.00     3,900.00         30.00     3,900.00
 622.2      CONCRETE BOUNDS           EA          64.           256.30    16,403.20
 280.00     17,920.00       300.00    19,200.00
 625.2      CONCRETE LIGHT POLE
             BASES, TYPE B            EA          12.           660.00     7,920.00
 650.00     7,800.00        700.00     8,400.00
 628.2      SAWED BITUMINOUS PAVEMENT LF      14,600.             1.35    19,710.00
    .80     11,680.00         1.50    21,900.00
 632.0104   RETROREFLECTIVE PAINT
             PAVE. MARKING, 4" LINE   LF      80,500.              .11     8,855.00
    .09     7,245.00           .12     9,660.00
 632.0106   RETROREFLECTIVE PAINT
             PAVE. MARKING, 6" LINE   LF     343,700.              .13    44,681.00
    .12     41,244.00          .14    48,118.00
 632.0112   RETROREFLECTIVE PAINT
             PAVE. MARKING, 12" LINE  LF       9,100.              .22     2,002.00
    .20     1,820.00           .25     2,275.00
 632.3104   RETROREFLECT. THERMOPLAS.
             PAVE. MARKING, 4" LINE   LF       3,100.              .66     2,046.00
    .50     1,550.00           .65     2,015.00
 632.3106   RETROREFLECT. THERMOPLAS.
             PAVE. MARKING, 6" LINE   LF       1,100.             1.10     1,210.00
    .80       880.00          1.10     1,210.00
 632.3108   RETROREFLECT. THERMOPLAS.
             PAVE. MARKING, 8" LINE   LF         250.             1.32       330.00
   1.00       250.00          1.25       312.50
 632.3112   RETROREFLECT. THERMOPLAS.
             PAVE. MARKING, 12" LINE  LF       4,000.             2.20     8,800.00
   1.50     6,000.00          2.10     8,400.00
 632.3118   RETROREFLECT. THERMOPLAS.
             PAVE. MARKING, 18" LINE  LF         920.             4.40     4,048.00
   2.25     2,070.00          4.10     3,772.00
 632.32     RETROREFLECT. THERMOPLAS.
             PAVEMENT MARKING, SYMBOL
             OR WORD                  SF       1,800.             6.05    10,890.00
   4.40     7,920.00          5.75    10,350.00
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 632.911   OBLITERATE PAVE. MARKING
            LINE, 12" WIDE & UNDER   LF     158,700.             .28    44,436.00
    .25    39,675.00         .30    47,610.00
 632.912   OBLITERATE PAVE. MARKING
            LINE, OVER 12" WIDE      LF         125.            2.20       275.00
   1.50       187.50        3.00       375.00
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 ITEM                                                                      A
          B                         C
  NO.        DESCRIPTION           UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
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 641.      LOAM                     CY          545.           17.40     9,483.00
  21.00    11,445.00        20.00   10,900.00
 644.62    WET BASIN/MEADOW SEED    LB          350.          198.00    69,300.00
 185.00    64,750.00       210.00   73,500.00
 644.741   UPLAND SEED MIX          LB            5.8          97.90       567.82
  85.00       493.00       170.00      986.00
 644.75    DRY FLOODPLAIN SEED MIX  LB           21.3          37.20       792.36
  35.00       745.50       170.00    3,621.00
 644.77    WET FLOODPLAIN SEED MIX  LB           13.4          49.50       663.30
  40.00       536.00       210.00    2,814.00
 644.78    STREAMSIDE SEED MIX      LB           16.8         198.00     3,326.40
 180.00     3,024.00       190.00    3,192.00
 645.0001  TURBIDITY BARRIER        LF        1,000.           19.00    19,000.00
   9.00     9,000.00        30.00   30,000.00
 645.11    MULCH                    A             3.          825.00     2,475.00
 750.00     2,250.00       810.00    2,430.00
 645.113   HYDRAULIC MULCH          LB       50,000.             .55    27,500.00
    .50    25,000.00         1.65   82,500.00
 645.119   MULCH WITH TACKIFIERS    A            31.        1,400.00    43,400.00
 900.00    27,900.00       910.00   28,210.00
 645.2535  COIR FIBER MATTING FOR
            EROSION CONTROL (STREAM
            CHANNEL)                SY        2,200.            3.45     7,590.00
   4.00     8,800.00        10.00   22,000.00
 645.3     EROSION STONE            TON      15,000.           19.65   294,750.00
  23.00   345,000.00        22.00  330,000.00
 645.44    TEMPORARY SLOPE
            STABILIZATION TYPE D
            (WILDLIFE FRIENDLY)     SY      306,100.            1.42   434,662.00
   1.28   391,808.00         1.90  581,590.00
 645.46    PERMANENT CHANNEL
            STABILIZATION TYPE B    SY       10,200.            3.48    35,496.00
   3.00    30,600.00         3.60   36,720.00
 645.48    EROSION CONTROL MIX      CY        2,000.           20.00    40,000.00
  35.00    70,000.00        33.00   66,000.00
 645.481   WOODCHIPS FOR TEMPORARY
            EROSION CONTROL         CY        1,000.           22.00    22,000.00
  26.00    26,000.00        13.00   13,000.00
 645.482   STUMP GRINDINGS FOR
            TEMPORARY EROSION
            CONTROL                 CY        7,000.           17.50   122,500.00
  26.00   182,000.00        20.00  140,000.00
 645.51    HAY BALES FOR TEMPORARY
            EROSION CONTROL         EA        1,010.            8.80     8,888.00
   8.00     8,080.00        12.00   12,120.00
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645.512 COMPOST SOCK FOR EROSION
AND SEDIMENT CONTROL LF 16,500. 3.85 63,525.00
4.00 66,000.00 6.00 99,000.00
645.52 RYEGRASS FOR TEMPORARY
EROSION CONTROL LB 2,110. 2.15 4,536.50
4.00 8,440.00 5.00 10,550.00
645.531 SILT FENCE LF 17,600. 1.65 29,040.00
2.50 44,000.00 3.00 52,800.00
645.611 BONDED FIBER MATRIX LB 56,000. 1.06 59,360.00
1.00 56,000.00 1.90 106,400.00
645.612 FIBER REINFORCED MATRIX LB 10,500. 1.18 12,390.00
1.15 12,075.00 2.30 24,150.00
645.613 STABILIZED MULCH MATRIX LB 7,500. .62 4,650.00
1.00 7,500.00 1.70 12,750.00
645.7 STORM WATER POLLUTION
PREVENTION PLAN U 1. 44,000.00 44,000.00
40,000.00 40,000.00 60,000.00 60,000.00
645.71 MONITORING SWPPP AND
EROSION AND SEDIMENT
CONTROLS HR 2,500. 55.00 137,500.00
50.00 125,000.00 54.00 135,000.00
645.81 STORM WATER REMOVAL
TRUCKING HR 400. 110.00 44,000.00
100.00 40,000.00 90.00 36,000.00
645.8103 EROSION CONTROL PUMP
300 GAL/MIN UWK 100. 1,300.00 130,000.00
1,400.00 140,000.00 800.00 80,000.00
645.8105 EROSION CONTROL PUMP
500 GAL/MIN UWK 100. 1,400.00 140,000.00
1,500.00 150,000.00 820.00 82,000.00
645.8110 EROSION CONTROL PUMP
1000 GAL/MIN UWK 100. 1,600.00 160,000.00
1,900.00 190,000.00 1,000.00 100,000.00
645.8203 EROSION CONTROL PUMP
(300 GAL./MIN. DISCHARGE
CAPACITY) U 1. 133,000.00 133,000.00
80,000.00 80,000.00 80,000.00 80,000.00
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ITEM A
B C
NO. DESCRIPTION UNIT QUANTITY UNIT PRICE TOTAL
UNIT PRICE TOTAL UNIT PRICE TOTAL

645.8205 EROSION CONTROL PUMP
(500 GAL./MIN. DISCHARGE
CAPACITY) U 1. 138,000.00 138,000.00
90,000.00 90,000.00 82,000.00 82,000.00
645.8210 EROSION CONTROL PUMP
(1000 GAL/MIN. DISCHARGE
CAPACITY) U 3. 145,000.00 435,000.00
120,000.00 360,000.00 100,000.00 300,000.00
645.85 FLOCCULANT TREATMENT
SYSTEM (FTS) MON 30. 10,000.00 300,000.00
10,500.00 315,000.00 24,500.00 735,000.00
645.851 WATER QUALITY MONITORING
FOR CONSTRUCTION
STORMWATER HR 500. 71.00 35,500.00

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  65.00    32,500.00        67.00    33,500.00
 646.2     TURF ESTABLISHMENT
            WITHOUT MULCH            A            55.          935.00    51,425.00
 850.00    46,750.00     1,600.00    88,000.00
 647.1     HUMUS                     CY       30,700.          10.25   314,675.00
  18.00    552,600.00       12.00   368,400.00
 647.22    HUMUS, INTERMIXED, 2" DEE CY           34.           20.35       691.90
  30.00     1,020.00        30.00     1,020.00
 647.29    WETLAND HUMUS             CY       12,750.           15.70   200,175.00
  17.00    216,750.00       14.00   178,500.00
 650.2     LANDSCAPING               U             1.       64,000.00    64,000.00
60,000.00    60,000.00   60,000.00    60,000.00
 650.3     LANDSCAPING, WETLAND
            MITIGATION               U             1.      164,000.00   164,000.00
135,000.00   135,000.00  155,000.00   155,000.00
 659.401   LANDSCAPE ESTABLISHMENT
            CREW (4 MEN- 8 HR DAY)   DAY           4.        1,320.00     5,280.00
1,200.00     4,800.00    1,500.00    6,000.00
 662.281   DECOMMISION DRILLED WELL  LF           74.           11.00       814.00
  12.50       925.00        80.00     5,920.00
 670.02    SEDIMENT SUMP
            MEASURING BLOCK          EA            7.          550.00     3,850.00
 240.00     1,680.00       550.00     3,850.00
 670.04501 CONSTRUCT AND REMOVE
            DIVERSION                U             1.      208,000.00   208,000.00
340,000.00   340,000.00  600,000.00   600,000.00
 670.04502 CONSTRUCT AND REMOVE
            DIVERSION                U             1.        6,740.00     6,740.00
8,000.00     8,000.00   33,000.00    33,000.00
 670.04503 CONSTRUCT AND REMOVE
            DIVERSION                U             1.       39,540.00    39,540.00
50,000.00    50,000.00   85,000.00    85,000.00
 670.04504 CONSTRUCT AND REMOVE
            DIVERSION                U             1.       33,200.00    33,200.00
50,000.00    50,000.00  120,000.00   120,000.00
 670.04506 CONSTRUCT AND REMOVE
            DIVERSION                U             1.       24,775.00    24,775.00
25,000.00    25,000.00   30,000.00    30,000.00
 670.04507 CONSTRUCT AND REMOVE
            DIVERSION                U             1.        3,990.00     3,990.00
12,000.00    12,000.00  110,000.00   110,000.00
 670.04508 CONSTRUCT AND REMOVE
            DIVERSION                U             1.        9,095.00     9,095.00
25,000.00    25,000.00  190,000.00   190,000.00
 670.04509 CONSTRUCT AND REMOVE
            DIVERSION                U             1.        3,160.00     3,160.00
8,000.00     8,000.00   38,000.00    38,000.00
 670.04510 CONSTRUCT AND REMOVE
            DIVERSION                U             1.          994.00       994.00
3,500.00     3,500.00   20,000.00    20,000.00
 670.04511 CONSTRUCT AND REMOVE
            DIVERSION                U             1.        4,635.00     4,635.00
9,500.00     9,500.00   20,000.00    20,000.00
 670.04512 CONSTRUCT AND REMOVE
            DIVERSION                U             1.       15,800.00    15,800.00
23,000.00    23,000.00   55,000.00    55,000.00
 670.04513 CONSTRUCT AND REMOVE
            DIVERSION                U             1.       54,450.00    54,450.00
85,000.00    85,000.00  135,000.00   135,000.00
 670.04515 CONSTRUCT AND REMOVE
            DIVERSION                U             1.        8,560.00     8,560.00
6,200.00     6,200.00   24,000.00    24,000.00
     13933E
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ITEM NO.	DESCRIPTION	UNIT	QUANTITY	A UNIT PRICE	A TOTAL	B UNIT PRICE	B TOTAL	C UNIT PRICE	C TOTAL
670.04516	CONSTRUCT AND REMOVE DIVERSION	U	1.	3,150.00	3,150.00	8,500.00	8,500.00	9,000.00	9,000.00
670.04517	CONSTRUCT AND REMOVE DIVERSION	U	1.	2,040.00	2,040.00	7,500.00	7,500.00	9,000.00	9,000.00
670.04519	CONSTRUCT AND REMOVE DIVERSION	U	1.	1,362.00	1,362.00	2,900.00	2,900.00	5,000.00	5,000.00
670.04520	CONSTRUCT AND REMOVE DIVERSION	U	1.	7,900.00	7,900.00	17,500.00	17,500.00	48,000.00	48,000.00
670.04521	CONSTRUCT AND REMOVE DIVERSION	U	1.	8,000.00	8,000.00	26,000.00	26,000.00	37,000.00	37,000.00
670.04522	CONSTRUCT AND REMOVE DIVERSION	U	1.	1,780.00	1,780.00	5,500.00	5,500.00	7,000.00	7,000.00
670.04523	CONSTRUCT AND REMOVE DIVERSION	U	1.	1,215.00	1,215.00	2,800.00	2,800.00	5,000.00	5,000.00
670.101	TEMPORARY LIGHTING	U	1.	3,850.00	3,850.00	3,500.00	3,500.00	6,000.00	6,000.00
670.104	TEMPORARY PORTABLE LIGHTING	U	5.	11,375.00	56,875.00	12,000.00	60,000.00	48,000.00	240,000.00
670.1501	RELOCATING PRIMARY UNDERGROUND ELECTRICAL SERVICE	U	1.	10,230.00	10,230.00	8,300.00	8,300.00	10,000.00	10,000.00
670.1502	RELOCATING PRIMARY UNDERGROUND ELECTRICAL SERVICE	U	1.	10,230.00	10,230.00	8,300.00	8,300.00	10,000.00	10,000.00
670.1503	RELOCATING PRIMARY UNDERGROUND ELECTRICAL SERVICE	U	1.	13,640.00	13,640.00	11,400.00	11,400.00	13,000.00	13,000.00
670.1504	RELOCATING PRIMARY UNDERGROUND ELECTRICAL SERVICE	U	1.	16,280.00	16,280.00	13,800.00	13,800.00	19,000.00	19,000.00
670.1505	RELOCATING PRIMARY UNDERGROUND ELECTRICAL SERVICE	U	1.	9,900.00	9,900.00	8,000.00	8,000.00	8,500.00	8,500.00
670.1506	RELOCATING PRIMARY UNDERGROUND ELECTRICAL SERVICE	U	1.	9,900.00	9,900.00	8,000.00	8,000.00	8,500.00	8,500.00
670.1507	RELOCATING PRIMARY UNDERGROUND ELECTRICAL								

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            SERVICE                 U             1.       9,900.00     9,900.00
8,000.00     8,000.00    8,500.00    8,500.00
 670.811   CONSTRUCT TURN LANE(S)
            AND ASSOCIATED WIDENING  U             1.      67,273.49    67,273.49
20,000.00    20,000.00   60,000.00    60,000.00
 671.603   3" PVC CONDUIT SCH. 80
            DIRECTIONAL BORE        LF          395.          38.50    15,207.50
   40.00   15,800.00        36.00   14,220.00
 677.31    WIRELESS COMMUNICATIONS
            EQUIPMENT               U             1.      12,650.00    12,650.00
10,500.00    10,500.00   12,000.00    12,000.00
 677.41    CCTV SYSTEM              U             1.      63,580.00    63,580.00
58,000.00    58,000.00   60,000.00    60,000.00
 677.43    VSL SYSTEM               U             1.      63,140.00    63,140.00
58,000.00    58,000.00   60,000.00    60,000.00
 677.51012 12-STRAND SINGLE MODE
            FIBER OPTIC CABLE       LF        1,350.           6.60     8,910.00
    3.50    4,725.00         7.00    9,450.00
 677.51072 72-STRAND SINGLE MODE
            FIBER OPTIC CABLE       LF        4,100.           6.60    27,060.00
    5.00   20,500.00         6.50   26,650.00
     13933E

     PAGE 15
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------------------------------------------------
 ITEM                                                                      A
            B                       C
  NO.          DESCRIPTION         UNIT     QUANTITY       UNIT PRICE     TOTAL
UNIT PRICE     TOTAL     UNIT PRICE     TOTAL
------------------------------------------------------------------------------------
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 677.53    FIBER OPTIC SPLICE       EA           24.          80.00     1,920.00
   80.00     1,920.00       80.00    1,920.00
 677.541   GROUND MOUNTED ITS
            EQUIPMENT CABINET
            (CCTV 20)               U             1.      17,050.00    17,050.00
12,000.00    12,000.00   16,000.00    16,000.00
 677.542   POLE MOUNTED ITS
            EQUIPMENT CABINET
            (VSL 403 & VSL 404)     U             1.      17,050.00    17,050.00
14,000.00    14,000.00   16,000.00    16,000.00
 677.561   12-POSITION FIBER
            OPTIC PATCH PANEL       EA            2.       1,100.00     2,200.00
  900.00     1,800.00    1,200.00    2,400.00
 677.5821  100 MBPS FIBER ETHERNET
            SWITCH                  EA            2.       4,510.00     9,020.00
4,500.00     9,000.00    4,500.00    9,000.00
 677.6301  METER AND DISCONNECT
            PEDESTAL                U             1.       3,850.00     3,850.00
3,000.00     3,000.00    4,000.00    4,000.00
 677.6302  METER AND DISCONNECT
            PEDESTAL                U             1.       3,850.00     3,850.00
3,500.00     3,500.00    4,000.00    4,000.00
 677.64    UNINTERUPTIBLE POWER
            SUPPLY (UPS)            EA            2.      14,850.00    29,700.00
10,000.00    20,000.00   14,000.00    28,000.00
 677.9300  3-CONDUCTOR #00 AWG
            CABLE                   LF          245.          13.20     3,234.00
   12.00    2,940.00        14.00    3,430.00
 677.9304  3-CONDUCTOR #4 AWG
            CABLE                   LF          110.          13.20     1,452.00
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  12.00     1,320.00        14.00     1,540.00
 677.9314  3-CONDUCTOR #14 AWG
            CABLE                   LF          130.          8.80     1,144.00
   7.50       975.00        10.00     1,300.00
 692.      MOBILIZATION             U             1.  2,443,000.00 2,443,000.00
2,950,000.00 2,950,000.00 2,561,000.00 2,561,000.00
 693.      ON-THE-JOB TRAINING OF
            UNSKILLED WORKERS       $             1.      3,000.00     3,000.00
3,000.00    3,000.00     3,000.00     3,000.00
 697.11    INVASIVE SPECIES CONTROL
            AND MANAGEMENT PLAN     U             1.      2,750.00     2,750.00
2,500.00    2,500.00     3,000.00     3,000.00
 698.11    FIELD OFFICE TYPE A      MON          36.      2,340.00    84,240.00
2,400.00   86,400.00     4,000.00   144,000.00
 698.2     PHYSICAL TESTING
            LABORATORY              MON          36.        520.00    18,720.00
 800.00    28,800.00     1,200.00    43,200.00
 699.      MISCELLANEOUS TEMPORARY
            EROSION AND SEDIMENT
            CONTROL                 $             1.    670,000.00   670,000.00
670,000.00  670,000.00  670,000.00   670,000.00
1008.52    ALTERATIONS AND ADDITIONS
            AS NEEDED - DEWATERING  $             1.     20,000.00    20,000.00
20,000.00   20,000.00   20,000.00    20,000.00
1008.8     ALTERATION AND ADDITIONS
            AS NEEDED - WINTER
            MAINTENANCE             $             1.     20,000.00    20,000.00
20,000.00   20,000.00   20,000.00    20,000.00
1010.15    FUEL ADJUSTMENT          $             1.    250,000.00   250,000.00
250,000.00  250,000.00  250,000.00   250,000.00
1010.2     ASPHALT CEMENT ADJUSTMENT $            1.     75,000.00    75,000.00
75,000.00   75,000.00   75,000.00    75,000.00
1010.3     QUALITY CONTROL
            QUALITY ASSURANCE (QC/QA
            ASPHALT                 $             1.    300,000.00   300,000.00
300,000.00  300,000.00  300,000.00   300,000.00
1010.41    QUALITY CONTROL
            QUALITY ASSURANCE (QC/QA
            FOR CONCRETE            $             1.     25,000.00    25,000.00
25,000.00   25,000.00   25,000.00    25,000.00
1010.42    QUALITY CONTROL
            QUALITY ASSURANCE (QC/QA
            FOR CONCRETE            $             1.     25,000.00    25,000.00
25,000.00   25,000.00   25,000.00    25,000.00


                                                                $40,908,383.67
       $40,967,760.95            $43,877,573.20
```

STATE OF NEW HAMPSHIRE
DEPARTMENT OF TRANSPORTATION
A-PS&E COMPARISION

PROJECT: SALEM-MANCHESTER
BI-A000(125), 13933E

COUNTIES AND CODES: ROCKINGHAM 015

DATE BIDS OPEN: JUNE 7, 2012

SCOPE OF WORK: ROADWAY AND BRIDGE REHABILITATION

LOCATION: I-93 EXIT 2 INTERCHANGE AND
PELHAM ROAD IN THE TOWN OF SALEM

Appendix 4.2: Time Estimate

This section contains the calculations used to develop the final time estimate for the bridge designs. This section contains both calculations and scheduling software outputs

4.2 Construction Time Estimates and Methodology

This section includes the calculations, tasks and crews from RS Means used to estimate the duration of proposed construction tasks and the entire proposed project duration.

4.2 Actual State CPM

This section is a piece of the actual project CPM, including expected durations, a scheduling time period, float, and a Gantt chart.

4.2 Project CPM Schedule

This is the data calculated in the construction time estimates and methodology part of the appendix as put into scheduling software. This includes a showing of task dependencies, and a Gantt Chart for the proposed design.

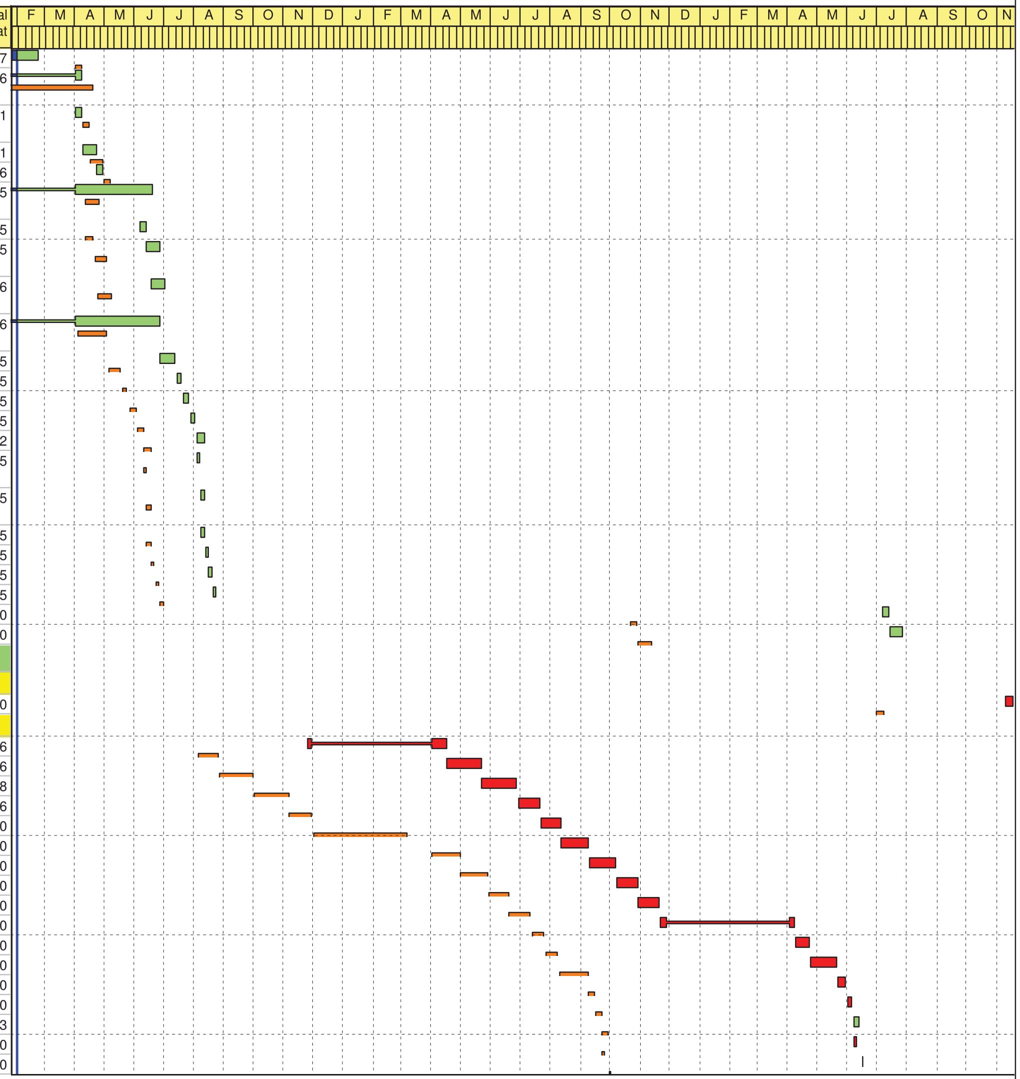
4.2 Task and Time Estimates, State and Project Bridges

This appendix contains a summary of the time estimates by task for both the actual state bridge and the proposed MQP bridge design.

Proposal Time Estimate					
Phase	Task	Time (days)	Description	Method of Estimating Time	RS Means Main Crews Needed
1	1	2	Mobilization, clearing and grubbing of first half of abutment locations.	RS Means Heavy Construction 2011. Clearing and Grubbing is governing factor	B-30
1	2	6	Installation of temporary steel sheet piles, Excavation of western half of abutment and pier locations. Driving Steel piles for first half of bridge	RS Means Heavy Construction 2011. Excavation and then sheet piles are are the governing factor	B-12A, B-40, B-19
1	3	20	First half of construction of Precast footings, CIP abutment retaining walls, and central pier, and precast abutment footings and bridge seats.	Project timeline from Laconia Total precast bridge. RS Means Heavy Construction 2011. Dot project expirience. Governing Factor is footings, then CIP pier, then, CIP pier cap.	1 Clab, B-10A, C-17B
1	4	1	Placement of western 7 of DBT girder lines and accompanying apparatus	Project timeline from Laconia Total precast bridge. Project expirience during work for NHDOT. RS Means Heavy Construction 2011. Crew Daily output is 14 beams, exactly as required for 1/2 of the bridge	C-14
1	5	10	Construction of western half of bridge joints, bridge membrane, bridge pavement and western half of highway structural box, adjustment of ramp paths. Placement of western face precast bridge barrier.	Project timeline from Laconia Total precast bridge. RS Means Heavy Construction 2011. Governing Factor is subgrade construction	1-Rofc, C-12, B-25, 2*B-36C
1	6	7	Final paving of first half of roadway box, line painting, placement of temporary barriers and traffic diversion onto new span	RS Means Heavy Construction 2011. Governing Factor is highway paving and lining	2*B-25B
2	1	1	Closure of existing bridge, removal of asphalt pavement, precutting of concrete deck, beam removal and transport off site.	Project timeline from Laconia Total precast bridge.Project expirience during work for NHDOT	B-9
2	2	2	Removal of aboveground pier cap and columns, existing bridge abutments, and slope paving	Project timeline from Laconia Total precast bridge. Project expirience during work for NHDOT	B-9

2	3	2	Removal to below new footings existing pier and abutment footings.	Project timeline from Laconia Total precast bridge.	B-38
3	1	6	Installation of sheet piles and structural piles, excavation of remaining half of bridge abutments and pier footing	See Step 2 Phase 1	B-12A, B-40, B-19
3	2	20	Second half of construction of CIP footings, abutment retaining walls, and central pier, and precast abutment footings and bridge seats.	See Step 3 Phase 1	1 Clab, B-10A, C-17B
3	3	1	Placement of remaining 6 DBTs and accompanying apparatus	See Step 4 Phase 1	C-14
3	4	10	Construction of remaining half of bridge joints, bridge membrane, bridge pavement and remaining half of highway structural box, adjustment of ramp paths. Placement of eastern face precast bridge barrier	See Step 5 Phase 1	1-Rofc, C-12, B-25, 2*B-36C
3	5	7	Final paving of remaining half of roadway box, line painting, removal of temporary barriers and traffic diversion onto new span. Concrete sealant and final slope seeding and loam	See Step 6 Phase 2, NH BID Results, 13933 E	2*B-25B, B-78

Activity ID	Activity Name	Original Duration	Start	Finish	Total Float
SB0310	Install South Abutment Sheeting for SB Diversion A Temporary Bridge	5	28-Jan-13 A	22-Feb-13	57
SB0240	Install Drainage for SB Diversion A South of Temporary Bridge Sta. 745+00 to 760+50	20	03-Sep-12 A	08-Apr-13	66
SB0320	Backfill & Install Walers/Tiebacks at South Abutment for SB Diversion A Temporary Bridge	5	02-Apr-13	08-Apr-13	31
SB0350	Install South Abutment Bridge Seats for SB Diversion A Temporary Bridge	10	09-Apr-13	23-Apr-13	31
SB0370	Build South Approach & Roller Pads for SB Diversion A Temporary Bridge	5	24-Apr-13	30-Apr-13	36
SB0300	Exc/Fill to Grade for SB Diversion A North of Temporary Bridge Sta. 763+00 to 788+81	10	15-Oct-12 A	20-Jun-13	-15
SB0330	Install North Abutment Sheeting for SB Diversion A Temporary Bridge	5	07-Jun-13	13-Jun-13	-15
SB0340	Backfill & Install Walers/Tiebacks at North Abutment for SB Diversion A Temporary Bridge	10	14-Jun-13	27-Jun-13	-15
SB0280	Install Select Roadbase materials for SB Diversion A North of Temporary Bridge Sta. 763+00 to 788+81	10	19-Jun-13	02-Jul-13	16
SB0270	Install Drainage for SB Diversion A North of Temporary Bridge Sta. 763+00 to 788+81	20	22-Aug-12 A	27-Jun-13	16
SB0360	Install North Abutment Bridge Seats for SB Diversion A Temporary Bridge	10	28-Jun-13	12-Jul-13	-15
SB0380	Build North Approach & Roller Pads for SB Diversion A Temporary Bridge	5	15-Jul-13	19-Jul-13	-15
SB0390	Assemble SB Diversion A Temporary Bridge to Launch Configuration	5	22-Jul-13	26-Jul-13	-15
SB0400	Launch & Construct SB Diversion A Temporary Bridge to Use Configuration	5	29-Jul-13	02-Aug-13	-15
SB0410	Install Bridge Rail & Snow Screen for SB Diversion A Temporary Bridge	6	05-Aug-13	12-Aug-13	-12
SB0420	Install Select Roadbase materials for SB Diversion A Temporary Bridge South Abutment	3	05-Aug-13	07-Aug-13	-15
SB0430	Install Select Roadbase materials for SB Diversion A Temporary Bridge North Abutment	3	08-Aug-13	12-Aug-13	-15
SB0440	Fine Grade SB Diversion A South of Temporary Bridge	3	08-Aug-13	12-Aug-13	-15
SB0450	Fine Grade SB Diversion A North of Temporary Bridge	3	13-Aug-13	15-Aug-13	-15
SB0460	Temporary Pave SB Diversion A North of Temporary Bridge	3	16-Aug-13	20-Aug-13	-15
SB0470	Place PCB for SB Diversion A	3	21-Aug-13	23-Aug-13	-15
SB2120	Dismantle/Remove Temporary Bridge for Diversion A	5	07-Jul-15	13-Jul-15	-70
SB2130	Remove Temporary Bridge Abutments for Diversion A	10	14-Jul-15	27-Jul-15	-70
Southbound Bridge					
Ultimate Roadway					
SB2710	Install Bridge Joint for SB Bridge	5	10-Nov-15	17-Nov-15	-90
Ultimate SB Mainline Sta. 3157+00 to 3161+00					
SB1340	Demolish Existing SB Bridge	15	25-Nov-13	16-Apr-14	-76
SB1350	Construct North Abutment MSE Wall of New SB Bridge	25	17-Apr-14	22-May-14	-76
SB1360	Construct South Abutment MSE Wall for New SB Bridge	25	23-May-14	27-Jun-14	-108
SB1370	Form/Rebar/Pour/Strip North Abutment Footing for New SB Bridge	15	30-Jun-14	21-Jul-14	-76
SB1380	Form/Rebar/Pour/Strip South Abutment Footing for New SB Bridge	15	22-Jul-14	11-Aug-14	-90
SB1390	Form/Rebar/Pour/Strip North Stub Abutment for New SB Bridge	20	12-Aug-14	09-Sep-14	-90
SB1400	Form/Rebar/Pour/Strip South Stub Abutment for New SB Bridge	20	10-Sep-14	07-Oct-14	-90
SB1410	Form/Rebar/Pour/Strip North Abutment Backwall for New SB Bridge	15	08-Oct-14	29-Oct-14	-90
SB1420	Form/Rebar/Pour/Strip South Abutment Backwall for New SB Bridge	15	30-Oct-14	20-Nov-14	-90
SB1430	Set Structural Steel & Set Blocking Grades for New SB Bridge	10	21-Nov-14	08-Apr-15	-90
SB1440	Set Precast Deck Panels for New SB Bridge	10	09-Apr-15	23-Apr-15	-90
SB1450	Form/Rebar/Pour/Strip Deck Overpour, Overhang, and Ends for New SB Bridg	20	24-Apr-15	21-May-15	-90
SB1460	Form/Rebar/Pour/Strip North Abutment Approach Slab for New SB Bridge	5	22-May-15	29-May-15	-90
SB1470	Form/Rebar/Pour/Strip South Abutment Approach Slab for New SB Bridge	5	01-Jun-15	05-Jun-15	-90
SB1480	Install Bridge Rail for New SB Bridge	5	08-Jun-15	12-Jun-15	-83
SB0030	Install Barrier Membrane On SB Bridge Deck	3	08-Jun-15	10-Jun-15	-90
SB0040	Pave Base & Temporary Course for SB Bridge Deck Sta. 3157+00 to 3161+00	1	16-Jun-15	16-Jun-15	-90



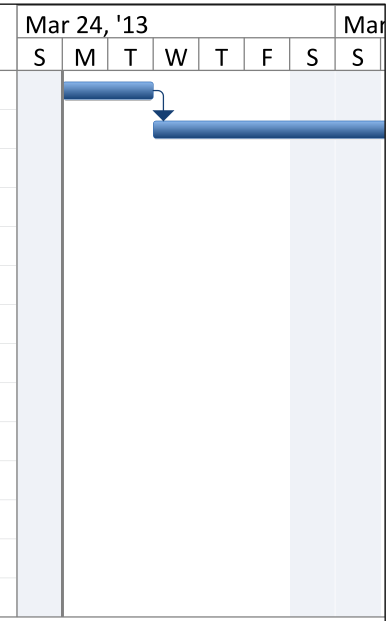
Remaining Level of Effort
Project Baseline
Actual Work
Remaining Work
Critical Remaining Work
Start Constraint
Milestone

NHDOT I93 Salem to Manchester Exit 2

Project schedule

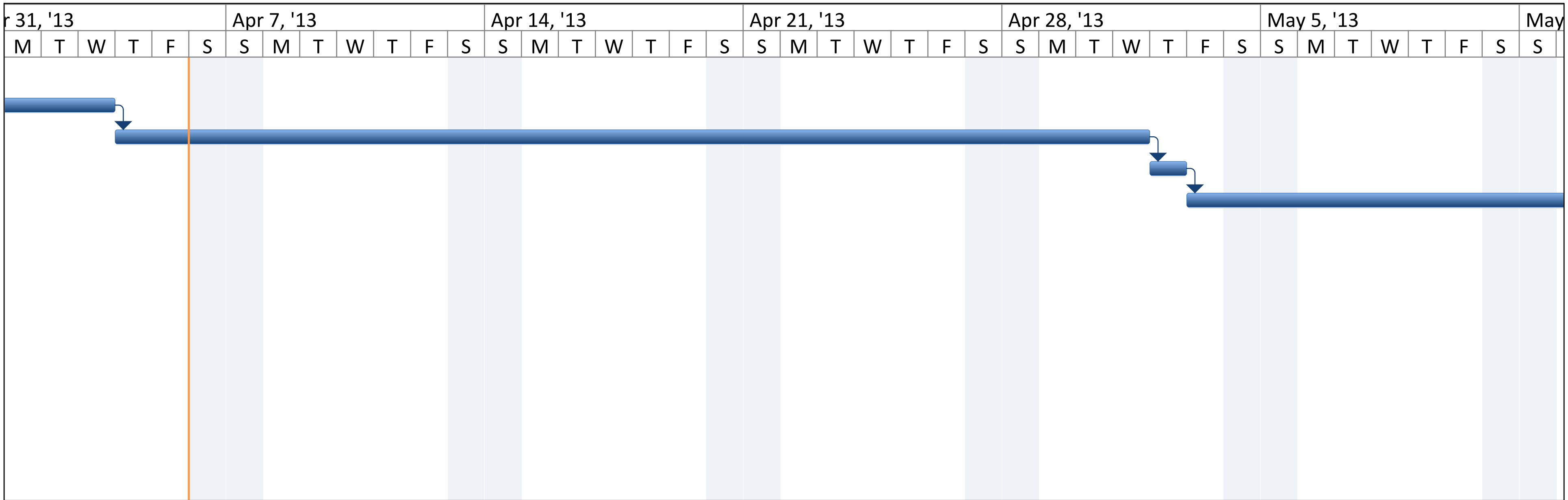
DD 01-Feb-13

ID		Task Mode	Task Name	Duration	Start	Finish	Predecessors	Resource Names
1			Phase 1-1	2 days	Mon 3/25/13	Tue 3/26/13		
2			Phase 1-2	6 days	Wed 3/27/13	Wed 4/3/13	1	
3			Phase 1-3	20 days	Thu 4/4/13	Wed 5/1/13	2	
4			Phase 1-4	1 day	Thu 5/2/13	Thu 5/2/13	3	
5			Phase 1-5	10 days	Fri 5/3/13	Thu 5/16/13	4	
6			Phase 1-6	7 days	Fri 5/17/13	Mon 5/27/13	5	
7			Phase 2-1	1 day	Tue 5/28/13	Tue 5/28/13	6	
8			Phase 2-2	2 days	Wed 5/29/13	Thu 5/30/13	7	
9			Phase 2-3	2 days	Fri 5/31/13	Mon 6/3/13	8	
10			Phase 3-1	6 days	Tue 6/4/13	Tue 6/11/13	9	
11			Phase 3-2	20 days	Wed 6/12/13	Tue 7/9/13	10	
12			Phase 3-3	1 day	Wed 7/10/13	Wed 7/10/13	11	
13			Phase 3-4	10 days	Thu 7/11/13	Wed 7/24/13	12	
14			Phase 3-5	7 days	Thu 7/25/13	Fri 8/2/13	13	



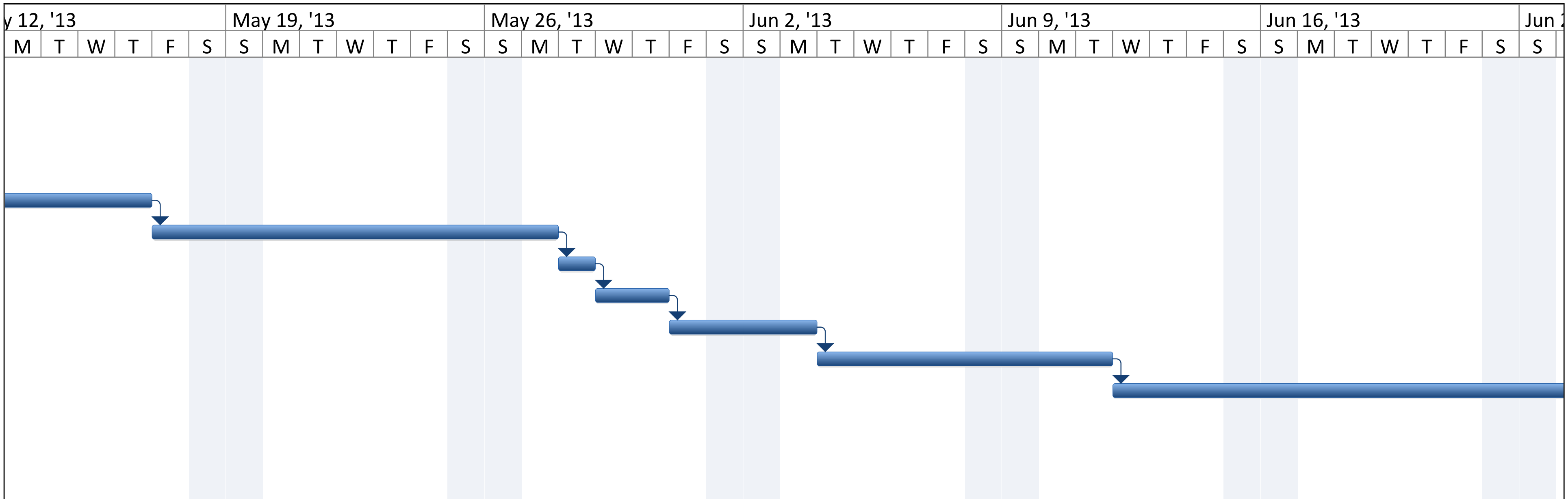
Project: 4-3 Project CPM Schedule
Date: Sat 4/6/13

Task	External Milestone	Manual Summary Rollup
Split	Inactive Task	Manual Summary
Milestone	Inactive Milestone	Start-only
Summary	Inactive Summary	Finish-only
Project Summary	Manual Task	Deadline
External Tasks	Duration-only	Progress



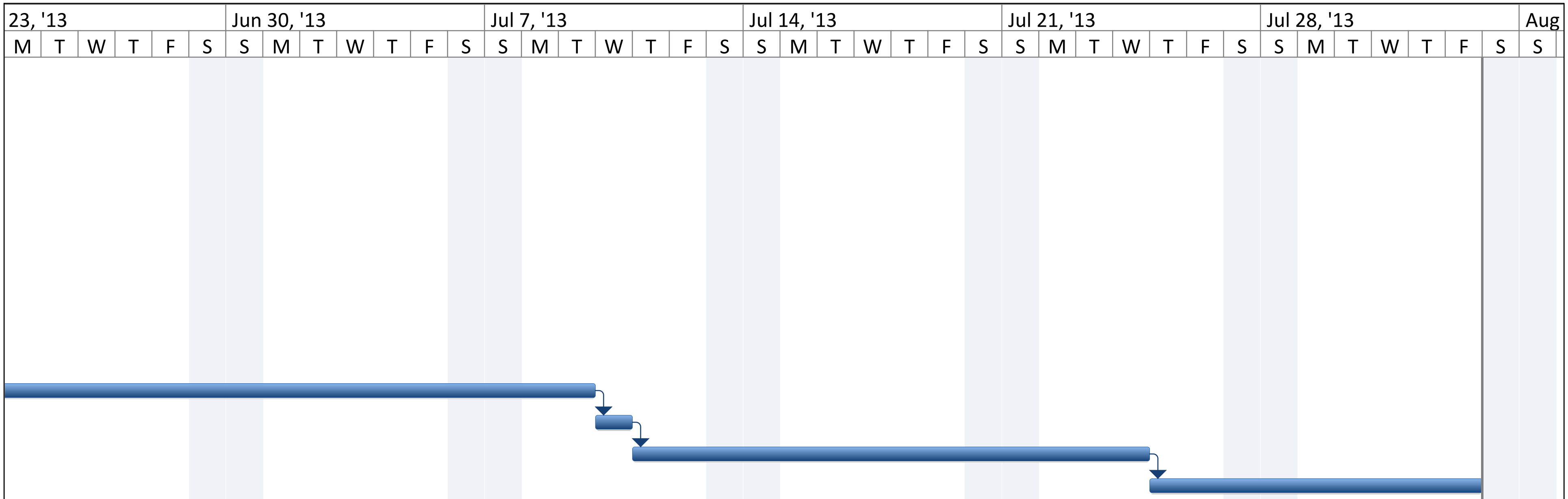
Task		External Milestone		Manual Summary Rollup	
Split		Inactive Task		Manual Summary	
Milestone		Inactive Milestone		Start-only	
Summary		Inactive Summary		Finish-only	
Project Summary		Manual Task		Deadline	
External Tasks		Duration-only		Progress	

Project: 4-3 Project CPM Schedule
Date: Sat 4/6/13



Project: 4-3 Project CPM Schedule
Date: Sat 4/6/13

Task	External Milestone	Manual Summary Rollup
Split	Inactive Task	Manual Summary
Milestone	Inactive Milestone	Start-only
Summary	Inactive Summary	Finish-only
Project Summary	Manual Task	Deadline
External Tasks	Duration-only	Progress



Project: 4-3 Project CPM Schedule Date: Sat 4/6/13	Task	External Milestone	Manual Summary Rollup
	Split	Inactive Task	Manual Summary
	Milestone	Inactive Milestone	Start-only
	Summary	Inactive Summary	Finish-only
	Project Summary	Manual Task	Deadline
	External Tasks	Duration-only	Progress

Final Time Estimates

Task Estimated Durations for State Bridge	
Task	Working Days
Install Bridge Joint for SB Bridge	5
Demolish Existing SB Bridge	15
Construct North Abutment MSE Wall of New SB Bridge	25
Construct South Abutment MSE Wall for New SB Bridge	25
Form/Rebar/Pour/Strip North Abutment Footing for New SB Bridge	15
Form/Rebar/Pour/Strip South Abutment Footing for New SB Bridge	15
Form/Rebar/Pour/Strip North Stub Abutment for New SB Bridge	20
Form/Rebar/Pour/Strip South Stub Abutment for New SB Bridge	20
Form/Rebar/Pour/Strip North Abutment Backwall for New SB Bridge	15
Form/Rebar/Pour/Strip South Abutment Backwall for New SB Bridge	15
Set Structural Steel & Set Blocking Grades for New SB Bridge	10
Set Precast Deck Panels for New SB Bridge	10
Form/Rebar/Pour/Strip Deck Overpour, Overhang, and Ends for New SB Bridge	20
Form/Rebar/Pour/Strip North Abutment Approach Slab for New SB Bridge	5
Form/Rebar/Pour/Strip South Abutment Approach Slab for New SB Bridge	5
Install Bridge Rail for New SB Bridge	5
Install Barrier Membrane On SB Bridge Deck	3
Pave Base & Temporary Course for SB Bridge Deck Sta.	1
TOTAL	229

Scheduling Information State Bridge	
State Early Start	11/25/2013
State Late Finish	6/16/2015
Total Days Elasped	568
Percent of Weekdays Utillized	56%

Task Estimated Durations for Project Bridge	
Task	Working Days
Mobilization, clearing and grubbing of first half of abutment locations.	2
Installation of temporary steel sheet piles, Excavation of western half of abutment and pier locations.	6
First half of construction of Precast footings, CIP abutment retaining walls, and central pier, and precast abutment footings and bridge seats.	20
Placement of western 7 of DBT girder lines and accompanying apparatus	1
Placement of approach slabs. Construction of western half of bridge joints, bridge membrane, bridge pavement and western half of highway structural box, adjustment of ramp paths. Placement of western face precast bridge barrier.	10
Final paving of first half of roadway box, line painting, placement of temporary barriers and traffic diversion onto new span	7
Closure of existing bridge, removal of asphalt pavement, precutting of concrete deck, beam removal and transport off site.	1
Removal of aboveground pier cap and columns, existing bridge abutments, and slope paving	2

Removal to below new footings existing pier and abutment footings.	2
Installation of sheet piles and structural piles, excavation of remaining half of bridge abutments and pier footing	6
Second half of construction of CIP footings, abutment retaining walls, and central pier, and precast abutment footings and bridge seats.	20
Placement of remaining 6 DBTs and accompanying apparatus	1
Placement of approach slabs. Construction of remaining half of bridge joints, bridge membrane, bridge pavement and remaining half of highway structural box, adjustment of ramp paths. Placement of eastern face precast bridge barrier	10
Final paving of remaining half of roadway box, line painting, removal of temporary barriers and traffic diversion onto new span. Concrete Sealant and final loaming and seeding of slopes	7
TOTAL	95

Scheduling Information Project Bridge	
Project Early Start	3/25/2013
Project Late Finish	8/2/2013
Total Days Elasped	130
Percent of Weekdays Utillized	100%

Appendix 4.3: Work Zone Road User Cost (WZ RUC)

Below are all spreadsheets and information relevant to the calculation of the WZ RUC. This Appendix is broken down into the three different sections outlined below.

Traffic Data

All relevant traffic data was obtained from the NHDOT website under the "Traffic Volume Reports". The traffic data was taken on both I-93 and Pelham Road. The traffic data offered an in depth look at the hourly breakdown of traffic for all the days of the week and made it possible to accurately estimate the WZ RUC.

I-93 WZ RUC

This spreadsheet below displays calculated the WZ RUC for the decrease in the speed limit for I-93 for both the project and the State design. The methodology for the WZ RUC can be seen in section 3.11.2 and the results can be seen in section 5.4.1.

Pelham Road WZ RUC

This spreadsheet displays the calculation of the WZ RUC for the detour along Pelham Road as explained in section 3.11.2 of the methodology and 5.4.1 of the results.

Traffic Data

STATE OF NEW HAMPSHIRE, DEPARTMENT OF TRANSPORTATION - BUREAU OF TRAFFIC 4/3/2012
IN COOPERATION WITH U.S. DEPARTMENT OF TRANSPORTATION FEDERAL HIGHWAY ADMINISTRATION
AUTOMATIC TRAFFIC RECORDER DATA FOR THE MONTH OF OCTOBER 2010

62 399062 SALEM- I-93 BETWEEN EXITS 1-2 (SB-NB) (61399062-61399061)

MON	DATE	DAY	12 AM	1 AM	2 AM	3 AM	4 AM	5 AM	6 AM	7 AM	8 AM	9 AM	10 AM	11 AM	12 PM	1 PM	2 PM	3 PM	4 PM	5 PM	6 PM	7 PM	8 PM	9 PM	10 PM	11 PM	Total
10	31	1	1255	858	492	318	278	535	879	1375	2074	3059	4202	5180	5317	5309	5334	5533	4892	3711	2893	3209	3290	2398	1405	942	64738
10	28	5	574	360	348	454	1025	3522	5324	5834	5146	4881	4184	4167	4283	4580	5591	5798	6505	6711	5437	3921	2907	2430	1828	1389	87199
10	29	6	723	421	377	446	1044	3389	5041	5741	5217	4356	4475	4749	4941	5321	6391	7048	6854	6549	5848	4473	3407	2841	2493	1837	93982
10	30	7	1200	697	441	325	552	1069	1716	2598	3553	4411	5015	5574	5638	5491	5682	5825	6122	5679	4841	3884	3138	2838	2523	2022	80834

TYPE	STATION	YEAR	MONTH	NO. DAYS	AVERAGE SUNDAY	AVERAGE WEEKDAY	AVERAGE SATURDAY	AVERAGE DAILY	COMPUTED VOLUME	PERCENT GAIN	PERCENT LOSS
62	399062	2010	October	4	64738	90590	80834	84847	2630260		

PEAK HOUR VOLUMES:	AVERAGE AM:	AVERAGE MIDDAY:	AVERAGE PM:	
SUNDAY	3059	5317	5533	AM - 6 AM TO 10 AM
WEEKDAY	5788	4950	6880	MIDDAY - 10 AM TO 2 PM
SATURDAY	4411	5638	6122	PM - 2 PM TO 8 PM

STATE OF NEW HAMPSHIRE, DEPARTMENT OF TRANSPORTATION - BUREAU OF TRAFFIC 4/3/2012
IN COOPERATION WITH U.S. DEPARTMENT OF TRANSPORTATION FEDERAL HIGHWAY ADMINISTRATION
AUTOMATIC TRAFFIC RECORDER DATA FOR THE MONTH OF OCTOBER 2010

62 399062 SALEM- I-93 BETWEEN EXITS 1-2 (SB-NB) (61399062-61399061)

MON	DATE	DAY	12 AM	1 AM	2 AM	3 AM	4 AM	5 AM	6 AM	7 AM	8 AM	9 AM	10 AM	11 AM	12 PM	1 PM	2 PM	3 PM	4 PM	5 PM	6 PM	7 PM	8 PM	9 PM	10 PM	11 PM	Total
10	31	1	1255	858	492	318	278	535	879	1375	2074	3059	4202	5180	5317	5309	5334	5533	4892	3711	2893	3209	3290	2398	1405	942	64738
10	28	5	574	360	348	454	1025	3522	5324	5834	5146	4881	4184	4167	4283	4580	5591	5798	6505	6711	5437	3921	2907	2430	1828	1389	87199
10	29	6	723	421	377	446	1044	3389	5041	5741	5217	4356	4475	4749	4941	5321	6391	7048	6854	6549	5848	4473	3407	2841	2493	1837	93982
10	30	7	1200	697	441	325	552	1069	1716	2598	3553	4411	5015	5574	5638	5491	5682	5825	6122	5679	4841	3884	3138	2838	2523	2022	80834

TYPE	STATION	YEAR	MONTH	NO. DAYS	AVERAGE SUNDAY	AVERAGE WEEKDAY	AVERAGE SATURDAY	AVERAGE DAILY	COMPUTED VOLUME	PERCENT GAIN	PERCENT LOSS
62	399062	2010	October	4	64738	90590	80834	84847	2630260		

PEAK HOUR VOLUMES:	AVERAGE AM:	AVERAGE MIDDAY:	AVERAGE PM:	
SUNDAY	3059	5317	5533	AM - 6 AM TO 10 AM
WEEKDAY	5788	4950	6880	MIDDAY - 10 AM TO 2 PM
SATURDAY	4411	5638	6122	PM - 2 PM TO 8 PM

STATE OF NEW HAMPSHIRE, DEPARTMENT OF TRANSPORTATION - BUREAU OF TRAFFIC 4/3/2012
IN COOPERATION WITH U.S. DEPARTMENT OF TRANSPORTATION FEDERAL HIGHWAY ADMINISTRATION
AUTOMATIC TRAFFIC RECORDER DATA FOR THE MONTH OF NOVEMBER 2008

82 399073 SALEM- PELHAM RD WEST OF POLICY ST

MON	DATE	DAY	12 AM	1 AM	2 AM	3 AM	4 AM	5 AM	6 AM	7 AM	8 AM	9 AM	10 AM	11 AM	12 PM	1 PM	2 PM	3 PM	4 PM	5 PM	6 PM	7 PM	8 PM	9 PM	10 PM	11 PM	Total
11	2	1	185	130	47	31	34	66	168	258	457	630	751	855	891	892	806	815	727	694	584	414	271	235	142	136	10219
11	1	7	181	89	80	56	68	112	225	399	624	823	956	1228	1250	1164	1041	979	1028	1188	946	725	604	752	779	382	15679

TYPE	STATION	YEAR	MONTH	NO. DAYS	AVERAGE SUNDAY	AVERAGE WEEKDAY	AVERAGE SATURDAY	AVERAGE DAILY	COMPUTED VOLUME	PERCENT GAIN	PERCENT LOSS
82	399073	2008	November	2	10219	0	15679	*	*		

PEAK HOUR VOLUMES:	AVERAGE AM:	AVERAGE MIDDAY:	AVERAGE PM:	
SUNDAY	630	892	815	AM - 6 AM TO 10 AM
WEEKDAY	*	*	*	MIDDAY - 10 AM TO 2 PM
SATURDAY	823	1250	1188	PM - 2 PM TO 8 PM

I-93 WZ RUC

I-93 Traffic Data from Exits 1-2 Using Statistics from October 30-November 6, 2010

Unit Cost/Hour		*Data taken from AASHTO Red Book and updated Consumer Price Index (CPI) for cars and Producer Price Index (PPI) for trucks.
Car:	$ 25.59	
Truck:	$ 41.83	

Average Daily Traffic Hourly Breakdown

Time	**Saturday**	**Sunday**	**Monday**	**Tuesday**	**Wednesday**	**Thursday**	**Friday**
12:00 AM	1200	1255	554	550	567	673	659
1:00 AM	697	858	305	332	370	386	404
2:00 AM	441	492	276	305	327	350	388
3:00 AM	325	318	407	437	395	408	479
4:00 AM	552	278	1156	1058	1052	1085	1015
5:00 AM	1069	535	3708	3488	3475	3500	3206
6:00 AM	1716	879	5272	5454	5505	5421	4862
7:00 AM	2598	1375	5751	6068	6106	6110	5308
8:00 AM	3553	2074	5814	5124	5720	5471	5028
9:00 AM	4411	3059	4371	4385	4242	4201	4267
10:00 AM	5015	4202	3874	3825	3770	3820	4090
11:00 AM	5574	5180	3957	3901	3765	3702	4607
12:00 PM	5638	5317	3967	3991	3812	4219	4947
1:00 PM	5491	5309	4115	4147	4151	4336	5396
2:00 PM	5682	5334	5061	5297	5074	5117	6368
3:00 PM	5825	5533	6079	6247	6241	5843	7051
4:00 PM	6122	4892	6647	6339	6743	5790	6791
5:00 PM	5679	3711	6632	6541	6640	5792	6493
6:00 PM	4841	2893	4740	4765	5335	4901	5816
7:00 PM	3884	3209	3135	3168	3379	3498	4063
8:00 PM	3138	3290	2259	2261	2496	2589	2980
9:00 PM	2838	2398	1935	1961	2029	2113	2600
10:00 PM	2523	1405	1340	1302	1599	1690	2002
11:00 PM	2022	942	945	968	1083	1093	1554
Total:	**80834**	**64738**	**82300**	**81914**	**83876**	**82108**	**90374**

Daily Avg. % Breakdown

Time	**Saturday**	**Sunday**	**Monday**	**Tuesday**	**Wednesday**	**Thursday**	**Friday**
12:00 AM	1.48%	1.94%	0.67%	0.67%	0.68%	0.82%	0.73%
1:00 AM	0.86%	1.33%	0.37%	0.41%	0.44%	0.47%	0.45%
2:00 AM	0.55%	0.76%	0.34%	0.37%	0.39%	0.43%	0.43%
3:00 AM	0.40%	0.49%	0.49%	0.53%	0.47%	0.50%	0.53%
4:00 AM	0.68%	0.43%	1.40%	1.29%	1.25%	1.32%	1.12%
5:00 AM	1.32%	0.83%	4.51%	4.26%	4.14%	4.26%	3.55%
6:00 AM	2.12%	1.36%	6.41%	6.66%	6.56%	6.60%	5.38%
7:00 AM	3.21%	2.12%	6.99%	7.41%	7.28%	7.44%	5.87%
8:00 AM	4.40%	3.20%	7.06%	6.26%	6.82%	6.66%	5.56%
9:00 AM	5.46%	4.73%	5.31%	5.35%	5.06%	5.12%	4.72%
10:00 AM	6.20%	6.49%	4.71%	4.67%	4.49%	4.65%	4.53%
11:00 AM	6.90%	8.00%	4.81%	4.76%	4.49%	4.51%	5.10%
12:00 PM	6.97%	8.21%	4.82%	4.87%	4.54%	5.14%	5.47%
1:00 PM	6.79%	8.20%	5.00%	5.06%	4.95%	5.28%	5.97%
2:00 PM	7.03%	8.24%	6.15%	6.47%	6.05%	6.23%	7.05%
3:00 PM	7.21%	8.55%	7.39%	7.63%	7.44%	7.12%	7.80%
4:00 PM	7.57%	7.56%	8.08%	7.74%	8.04%	7.05%	7.51%
5:00 PM	7.03%	5.73%	8.06%	7.99%	7.92%	7.05%	7.18%
6:00 PM	5.99%	4.47%	5.76%	5.82%	6.36%	5.97%	6.44%
7:00 PM	4.80%	4.96%	3.81%	3.87%	4.03%	4.26%	4.50%
8:00 PM	3.88%	5.08%	2.74%	2.76%	2.98%	3.15%	3.30%
9:00 PM	3.51%	3.70%	2.35%	2.39%	2.42%	2.57%	2.88%
10:00 PM	3.12%	2.17%	1.63%	1.59%	1.91%	2.06%	2.22%
11:00 PM	2.50%	1.46%	1.15%	1.18%	1.29%	1.33%	1.72%
Total:	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**	**100.00%**

Truck Traffic Breakdown

Time	**% of Trucks**	**Saturday**	**Sunday**	**Monday**	**Tuesday**	**Wednesday**	**Thursday**	**Friday**
12:00 AM	35.01%	420.12	439.38	193.96	192.56	198.51	235.62	230.72
1:00 AM	39.95%	278.45	342.77	121.85	132.63	147.82	154.21	161.40
2:00 AM	43.48%	191.75	213.92	120.00	132.61	142.18	152.18	168.70
3:00 AM	43.26%	140.60	137.57	176.07	189.05	170.88	176.50	207.22
4:00 AM	38.18%	210.75	106.14	441.36	403.94	401.65	414.25	387.53
5:00 AM	29.61%	316.53	158.41	1097.94	1032.80	1028.95	1036.35	949.30
6:00 AM	23.50%	403.26	206.57	1238.92	1281.69	1293.68	1273.94	1142.57
7:00 AM	21.45%	557.27	294.94	1233.59	1301.59	1309.74	1310.60	1138.57
8:00 AM	22.61%	803.33	468.93	1314.55	1158.54	1293.29	1236.99	1136.83
9:00 AM	23.30%	1027.76	712.75	1018.44	1021.71	988.39	978.83	994.21
10:00 AM	23.21%	1163.98	975.28	899.16	887.78	875.02	886.62	949.29
11:00 AM	22.90%	1276.45	1186.22	906.15	893.33	862.19	847.76	1055.00
12:00 PM	22.64%	1276.44	1203.77	898.13	903.56	863.04	955.18	1120.00
1:00 PM	22.17%	1217.35	1177.01	912.30	919.39	920.28	961.29	1196.29
2:00 PM	21.30%	1210.27	1136.14	1077.99	1128.26	1080.76	1089.92	1356.38
3:00 PM	19.96%	1162.67	1104.39	1213.37	1246.90	1245.70	1166.26	1407.38
4:00 PM	18.83%	1152.77	921.16	1251.63	1193.63	1269.71	1090.26	1278.75
5:00 PM	18.51%	1051.18	686.91	1227.58	1210.74	1229.06	1072.10	1201.85
6:00 PM	19.88%	962.39	575.13	942.31	947.28	1060.60	974.32	1156.22
7:00 PM	21.76%	845.16	698.28	682.18	689.36	735.27	761.16	884.11
8:00 PM	23.38%	733.66	769.20	528.15	528.62	583.56	605.31	696.72
9:00 PM	24.84%	704.96	595.66	480.65	487.11	504.00	524.87	645.84
10:00 PM	27.27%	688.02	383.14	365.42	355.06	436.05	460.86	545.95
11:00 PM	30.31%	612.87	285.52	286.43	293.40	328.26	331.29	471.02
Total:		**18408.0051**	**14779.1822**	**18628.12**	**18531.54**	**18968.56**	**18696.67**	**20481.84**

WZ RUC for Speed Limit Changes		
Initial Speed Limit:	65	mph
Const. Speed Limit:	55	mph
Work Zone Distance:	1.7	m
Hourly Traffic Flow:	3369.90	vph
Car %:	73.45%	
Truck %:	26.55%	
Reduced Speed Cost	$ 123.24	/hr
Project Duration:	130	days
Work Days:	24	hours
Reduced Speed Cost	$384,523.04	

Pelham Road WZ RUC

Pelham Road Traffic Data Using Statistics from November 1-2, 2008

Unit Cost/Hour *Data taken from AASHTO Red Book and updated Consumer Price Index (CPI) for cars and Producer Price Index (PPI) for trucks.

Car:	$ 25.59
Truck:	$ 41.83

Average Daily Traffic Hourly Breakdown

Time	Saturday	Sunday
12:00 AM	181	185
1:00 AM	89	130
2:00 AM	80	47
3:00 AM	56	31
4:00 AM	68	34
5:00 AM	112	66
6:00 AM	225	168
7:00 AM	399	258
8:00 AM	624	457
9:00 AM	823	630
10:00 AM	956	751
11:00 AM	1228	855
12:00 PM	1250	891
1:00 PM	1164	892
2:00 PM	1041	806
3:00 PM	979	815
4:00 PM	1028	727
5:00 PM	1188	694
6:00 PM	946	584
7:00 PM	725	414
8:00 PM	604	271
9:00 PM	752	235
10:00 PM	779	142
11:00 PM	382	136
Total:	**15679**	**10219**

Daily Avg. % Breakdown

Time	Saturday	Sunday
12:00 AM	1.15%	1.81%
1:00 AM	0.57%	1.27%
2:00 AM	0.51%	0.46%
3:00 AM	0.36%	0.30%
4:00 AM	0.43%	0.33%
5:00 AM	0.71%	0.65%
6:00 AM	1.44%	1.64%
7:00 AM	2.54%	2.52%
8:00 AM	3.98%	4.47%
9:00 AM	5.25%	6.16%
10:00 AM	6.10%	7.35%
11:00 AM	7.83%	8.37%
12:00 PM	7.97%	8.72%
1:00 PM	7.42%	8.73%
2:00 PM	6.64%	7.89%
3:00 PM	6.24%	7.98%
4:00 PM	6.56%	7.11%
5:00 PM	7.58%	6.79%
6:00 PM	6.03%	5.71%
7:00 PM	4.62%	4.05%
8:00 PM	3.85%	2.65%
9:00 PM	4.80%	2.30%
10:00 PM	4.97%	1.39%
11:00 PM	2.44%	1.33%
Total:	**100.00%**	**100.00%**

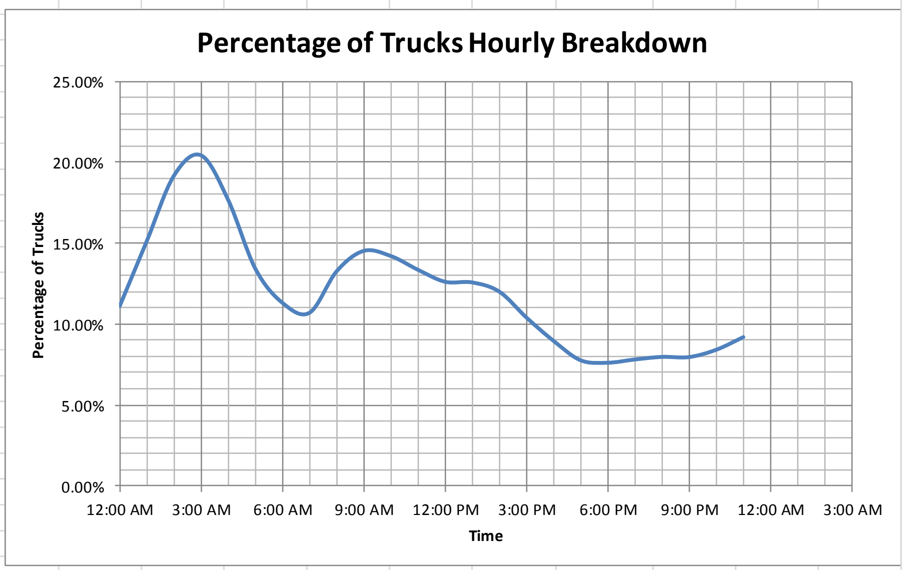
Time	% of Trucks	Saturday	Sunday
12:00 AM	11.12%	20.1272	20.572
1:00 AM	15.14%	13.4746	19.682
2:00 AM	19.17%	15.336	9.0099
3:00 AM	20.42%	11.4352	6.3302
4:00 AM	17.66%	12.0088	6.0044
5:00 AM	13.42%	15.0304	8.8572
6:00 AM	11.31%	25.4475	19.0008
7:00 AM	10.70%	42.693	27.606
8:00 AM	13.26%	82.7424	60.5982
9:00 AM	14.52%	119.4996	91.476
10:00 AM	14.20%	135.752	106.642
11:00 AM	13.35%	163.938	114.1425
12:00 PM	12.61%	157.625	112.3551
1:00 PM	12.57%	146.3148	112.1244
2:00 PM	12.00%	124.92	96.72
3:00 PM	10.40%	101.816	84.76
4:00 PM	8.96%	92.1088	65.1392
5:00 PM	7.76%	92.1888	53.8544
6:00 PM	7.61%	71.9906	44.4424
7:00 PM	7.81%	56.6225	32.3334
8:00 PM	7.97%	48.1388	21.5987
9:00 PM	7.96%	59.8592	18.706
10:00 PM	8.41%	65.5139	11.9422
11:00 PM	9.19%	35.1058	12.4984
Total:		**1709.689**	**1156.395**

Detour Delay Cost

Original Distance:	0.7	miles	1 min
Detour Distance:	1.43	miles	3 mins
NH-97 Speed Limit:	40	mph	
Detour Speed Limit:	30	mph	
Average AADT:	711	vehicles/8 hours	
Duration of Closure:	3	days	
Avg. Truck %:	11.98%		

<u>Car</u>

Original Travel Time:	63	s
Detour Travel Time:	171.6	s
Delay:	0.0301667	hr
Cost/car:	$ 0.77	
Cost/truck:	$ 1.26	
Total Cost:	$ 1,771.97	
Avg. Cost/Day:	$ 590.66	



Appendix 5.1: Comparisons and Grading

The project was compared with the State design and graded in order to determine the weaknesses and strengths of each project. The relevant calculations can be seen in the spreadsheet described below.

Grading Calculations

This spreadsheet was used to determine the grade of the projects based on the weight and scores of each individual category. Section 6.3 illustrates the process used for the calculation of these grades and this spreadsheet shows the raw data.

Grading Calculations

	Cost (million)	Cost Score	Constructability	Aesthetics	Maintenance	WZ RUC	Incentives	Total Grade
Project	$ 1.997	5.03	8.5	6.5	8	0.392		5.20
State Plan	$ 2.045	4.55	8	8	9	1.688		5.15
	A	B		**Aesthetics**	**Constructability**	**Cost**	**Maintenance**	**Incentives**
10	$ 1.50	$ -		7.5%	20.0%	40.0%	12.5%	20.0%
9	$ 1.60	$ 0.20						
8	$ 1.70	$ 0.40						
7	$ 1.80	$ 0.60						
6	$ 1.90	$ 0.80						
5	$ 2.00	$ 1.00						
4	$ 2.10	$ 1.20						
3	$ 2.20	$ 1.40						
2	$ 2.30	$ 1.60						
1	$ 2.40	$ 1.80						
0	$ 2.50	$ 2.00						