Design of Culvert Replacement Structure at Depot Road

Prepared by

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Submitted to Susannah Howe, Jim Hyslip, and Bill Turner

Executive Summary

Depot Road in Williamsburg, MA, frequently floods due to an undersized culvert for Wright Brook, a stream that crosses the road. The flooding occurs during heavy storms, requiring drivers to take an alternate route. In addition, the flooding is exacerbating the damage to the already deteriorating road. The road above the culvert is hazardous to drive on because of its poor condition as well as its unsafe curvature. These issues became more of a concern in early 2012 when the road collapsed under the weight of a teenager. Team HyGround, consisting of Lindsay Duran, Sarah Pong, and Eunice Zhao, designed a replacement structure for the culvert as well as a realigned road. Our liaisons include Jim Hyslip, President of HyGround Engineering, and Bill Turner, Superintendent of the Williamsburg Department of Public Works.

From our research, we determined that a precast concrete open bottom box culvert would be the most suitable structure for the site. After performing a hydrologic analysis, we found that the flow rate for a 50-year 24-hour design storm in the area would be about 1900 cfs. Using this information, we modeled various culvert sizes in HEC-RAS to narrow our options. At first, the team intended to use an opening similar in size to a culvert downstream of the site. However, in order to comply with the *Massachusetts Stream Crossing Handbook* standards, the new replacement structure must be 1.2 times wider than the bankfull stream width, which translates to a 35 ft span. According to Federal standards, this structure would be classified as a bridge.

The bridge was then oriented slightly rotated from the current culvert's orientation. This position better aligns with the stream and reduces the need for stream training into the bridge. Even so, stepped back face gabion walls were designed to allow for some stream training. From there, the foundation for the bridge was designed. We decided to use a strip footing that will be laid at bedrock based on the soil boring taken from the site and a soil analysis.

The new road design was based on the standards set in Massachusetts, as well as the input of our liaisons. A design speed of 35 mph and a superelevation of 4% were used to determine the new radius of the road curvature. Bill Turner originally wanted to recondition at least 270 ft of Depot Road. Combining this and the length of road that needs to be realigned at the curve, a total new road length of 355 ft must be paved.

The deliverables for this project include construction plans, a full cost estimate, an executive report, and documents to support permitting. These materials are in preparation for construction to begin in summer of 2013.

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1.0 Introduction

The culvert on Depot Road in Williamsburg, MA needs to be redesigned due to flooding and structural deterioration. We worked as Team HyGround on behalf of the town of Williamsburg and HyGround Engineering, LLC to develop a design for a replacement structure for the existing culvert as well as realigning the section of road running above it (Figure 1). In addition to this, we performed hydrologic, hydraulic, and soil analyses in order to develop construction drawings and specifications, to support future permitting for construction in August 2013.

Figure 1. Inside view of the existing culvert (left) and Depot Road (right)

2.0 Background

In 2009, Bill Turner, the Department of Public Works (DPW) Superintendent in Williamsburg, approached HyGround Engineering on behalf of the town residents regarding the replacement of the culvert on Depot Road and realignment of the section of road above. The motivation for replacing the existing culvert was frequent flooding at that section of Depot Road during storm events. The existing culvert is undersized and unable to allow high volumes of water to pass at an adequate rate during storm flow. In addition to the flooding problem, the road above the culvert must be realigned due to deteriorating conditions and an unsafe geometry. This challenge of designing a new stream crossing and roadway was designated as a capstone design project for the Picker Engineering Program at Smith College. This section describes the project site location, motivation, and stakeholders.

2.1 Site Overview

Williamsburg, located in the foothills of the Berkshires in Western Massachusetts, borders Northampton. Williamsburg is considered a rural New England town with a population of approximately 2,600 residents (1). The proposed site is located on Depot Road, just off Route 9 (Figure 2). Depot Road is a secondary road connecting Route 9 with residential and agricultural areas in Williamsburg. Overall, the site is in a primarily rural area with a few neighborhoods and businesses.

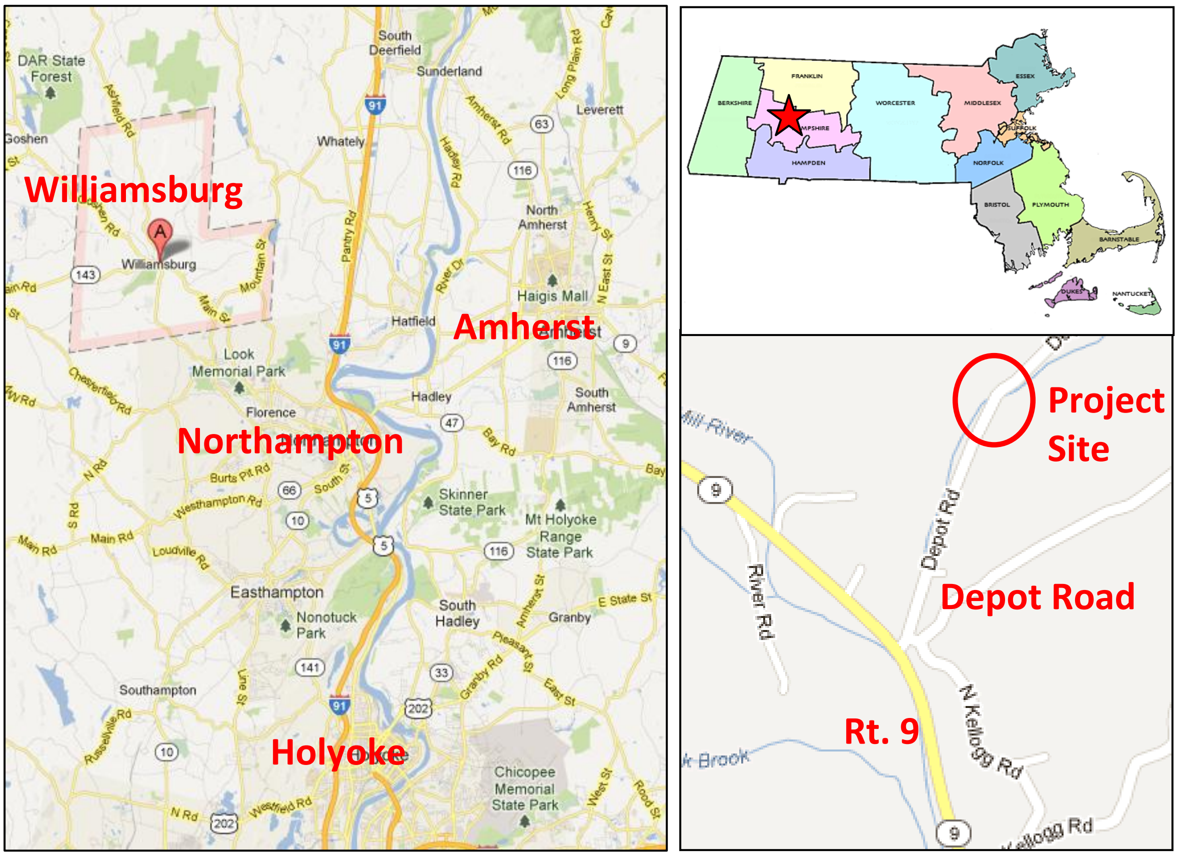


Figure 2. Location of Williamsburg in Western MA and Depot Road (2)

As seen in Figure 3, the body of water being conveyed through the existing culvert is Wright Brook. Figure 4 provides an overall view of a section of Depot Road that runs over the existing culvert and shows evidence of road deterioration present in the cracking of the pavement. Figure 5 shows the guardrails along the wing wall of the culvert inlet. The reinforcing steel bars within the guardrails are exposed due to crumbling of the original concrete. Because Wright Brook naturally flows parallel to the road (as seen in Figure 3), wing walls were placed to train the stream to bend and enter the culvert.

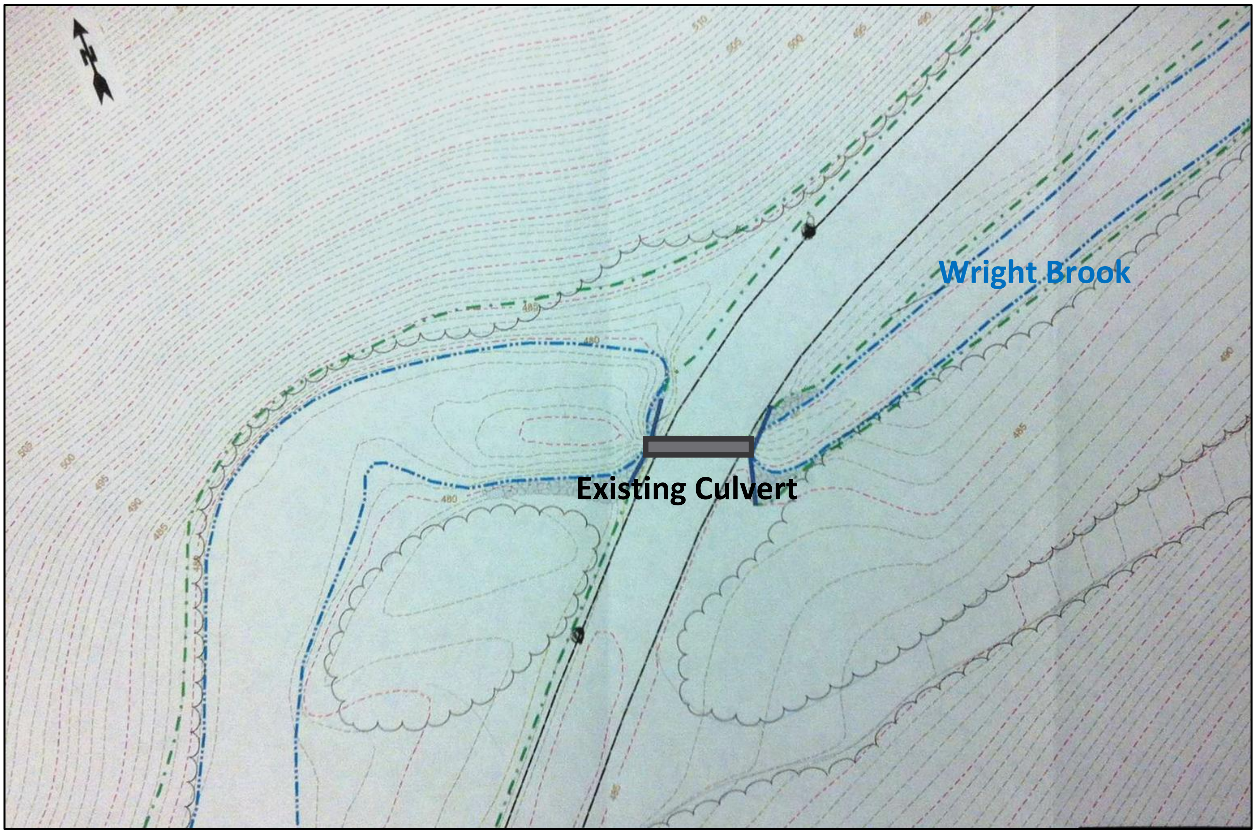


Figure 3. Site plan for the existing culvert

Figure 4. Re-patched shoulder above culvert

Figure 5. Worn down guardrail along the culvert inlet wing wall

Figure 6 provides a view of Wright Brook running through the outlet of the existing culvert. After visiting the site, we determined that the outlet side of the stream was much deeper than the inlet. We believe this is due to scouring.

Figure 6. Outlet of existing culvert

On a site visit, the team noted a box culvert downstream of the project site that was constructed in 1954. It is situated at the intersection of Rt. 9 and Depot Road. The opening of this culvert is 60 ft^2 and facilitates the merging of Wright Brook and Mill River. Wright Brook flows through the culvert and under Rt. 9 before being deposited into the Mill River (Figure 7). As of 2012, there have been no reports of flooding on Rt. 9 above the culvert, indicating that the culvert is adequately sized for the flow it experiences.

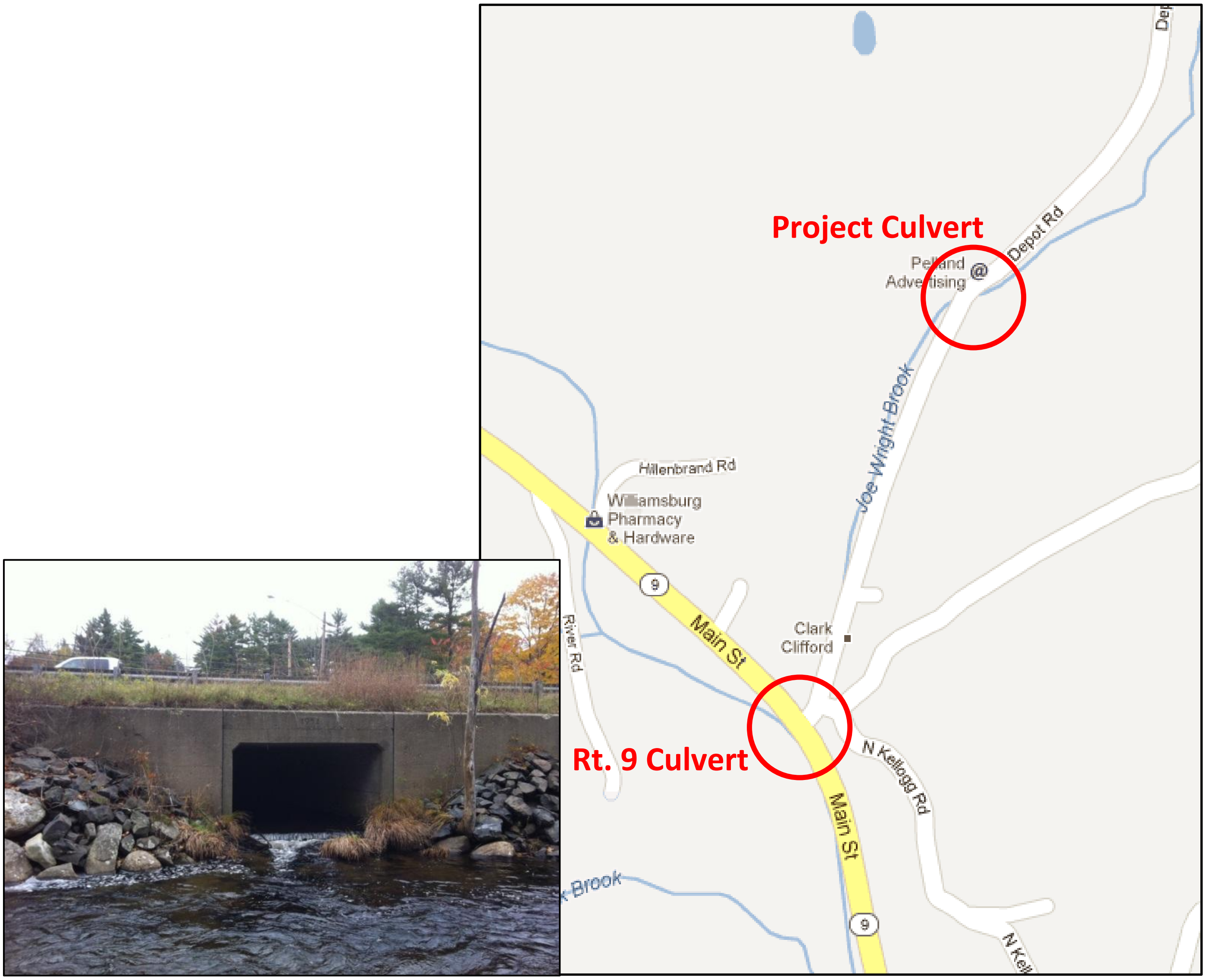


Figure 7. Rt. 9 culvert located downstream of the project site

2.2 Project Motivation

The motivation for replacing the culvert stems from the need to alleviate problems with flooding and structural deterioration (Figure 8). The section of Depot Road at our project site primarily floods during heavy storm events and has flooded at least once a year in recent years. This is especially true of the hurricanes and tropical storms that Western Massachusetts experienced in 2011-2012. The existing culvert is a 5 ft diameter corrugated metal pipe (CMP) and is considered to be undersized and in poor condition (Figure 9).

Figure 8. Flooding of the road and structural deterioration

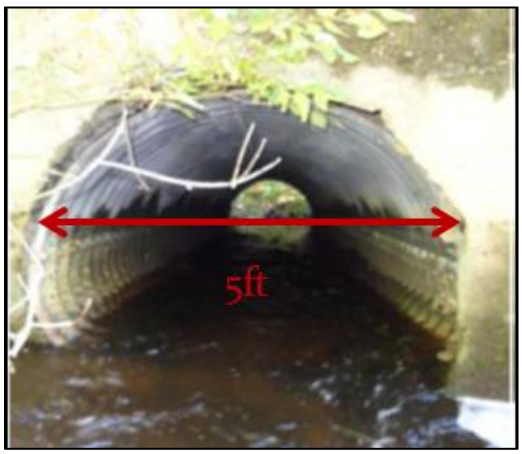


Figure 9. Head on view of existing CMP culvert with 5 ft diameter and rusting interior

The segment of road overlaying the culvert has been deteriorating for years. Our liaisons stressed the urgency for this project, which was fueled when a portion of the road crumbled under the weight of a teenage girl earlier this year. The section of road was given a temporary patch (as seen in Figure 4), but must be fully replaced. A minimum road length of 270 ft must be reconstructed because of its poor condition. While the majority of Depot Road was repaved in recent years, the section of road at our design site was left in its original state to avoid interrupting the existing culvert and the stream below. Aside from needing replacement, the road also requires realignment. The current layout of the road does not provide a safe radius of curvature for the existing speed limit. For this reason, the goal in reshaping the road is to provide a safer alignment for vehicular traffic as well as accommodate the stream direction under the road for minimal stream training.

2.3 Stakeholders

Jim Hyslip and Bill Turner served as the liaisons for Team HyGround and assisted with the development of our design. Jim Hyslip is the President of HyGround Engineering, LLC, an engineering consulting firm located in Williamsburg, which provides geotechnical and railroad engineering services while specializing in railroad substitution, investigation, diagnosis and problem solutions (3). Bill Turner is the DPW Superintendent of Williamsburg and is responsible for infrastructure projects for the area within its jurisdiction. The projects include public buildings, public spaces, transportation and other physical assets and facilities. The DPW maintains and improves the quality of drinking water, wastewater treatment, stormwater management, streets and highways, flood control and engineering (4).

The Williamsburg Conservation Commission works on behalf of the town to maintain natural resources and preserve the quality of life of those who live in the town. The Conservation Commission administers the Massachusetts Wetlands Protections Act and the Massachusetts Rivers Protection Act. Any projects near ponds, rivers, streams, wetlands and other environmental resources require review and approval from the Conservation Commission (5).

3.0 Design Requirements

Through communication with Jim Hyslip and Bill Turner, the team determined the design needs of the project: meeting the project budget, accommodating town interests, and improving current road conditions. The team also researched restrictions and regulations imposed by government and state agencies. For the road realigning aspect of the project, the Massachusetts Highway has regulations the design must meet (6). The replacement structure for the culvert must also be safe and environmentally nonintrusive according to standards stipulated by the Department of Environmental Protection (7), the Massachusetts Department of Transportation (Mass DOT) (8), and the Conservation Commission of Williamsburg (5). The environmental design requirements for the new structure are based upon the *Massachusetts Stream Crossing Handbook* (7) which provides standards with which all new stream crossings must comply. The design needs, as well as the quantitative requirements of each, are listed in Table 1 below.

Table 1. Project Design Needs and Requirements

Design Need	**Design Requirement**	**Stakeholders**
The road design will improve existing roadway geometry.	The new road geometry will safely accommodate a 35mph design speed	• Drivers/Town, • Bill Turner, • Jim Hyslip
The new road will include all existing road requiring reconstruction.	The new road will have a minimum length of 270 ft.	• Bill Turner, • Drivers/Town
The new road will meet state standards.	The percent banking, or superelevation, on the curve of the roadway will be no more than 6% (9).	• Mass. Highway, • Drivers
The new road must be able to handle the load above.	The new road must be able to support the live load of traffic, approximately 135 psf (calculations based on maximum live loading expected).	• DEP, • Bill Turner, • Drivers/Town, Jim Hyslip
The replacement structure can withstand the load above.	The replacement structure must support both the dead load of the road and soil as well as the live load of the traffic. If the structure requires a separate foundation, the foundation must be able to support the total load of the structure as well as the load of the soil, road, and traffic above the structure.	• DEP, • Bill Turner, • Drivers/Town, • Jim Hyslip
The new structure must be able to handle a 50-year, 24-hour design storm.	Due to its location on a secondary road, structure must be able to accommodate the 50-year, 24-hour storm flow rate (8).	• Mass. DOT, • Jim Hyslip, • Bill Turner
The structure does not disrupt wildlife passing through.	1. The crossing has a bottom that matches the stream's natural substrate. 2. The crossing spans the channel width with a minimum 1.2 times bankfull width of stream. 3. The openness ratio (cross sectional area/crossing length) is at least 0.82. 4. The water depth and velocity must be comparable to the natural channel. The maximum velocity through the structure is 10 ft/s (Chapter 8, (7)).	• Conservation Commission, • DEP, • Mass. Highway, • Aquatic life
The design must withstand environmental conditions.	The road and structure will not need to be replaced within 100 years.	• Conservation Commission, • Bill Turner, • Town
The proposed structure and road design is inexpensive.	The final design cost estimate does not exceed the town's allotted budget of $250,000 for this project.	• Town, • Bill Turner
The design is aesthetically suitable.	The final design must receive approval from Jim Hyslip and Bill Turner.	• Town

4.0 Design Development

There are two main components to this project: redesigning a replacement structure for an undersized culvert and realigning the road above it. The first part of the project was to design the new road, which required pavement design as well realigning the road. We performed background research and calculations for horizontal curves on roadways. Before pursuing a specific structure, the team researched two common structure options, bridges and culverts, as well as possible shapes and materials. After selecting a structure and new road alignment, we explored different structure orientations. Before recommending a final design, the team performed hydrologic and hydraulic analyses to ensure the performance of the new structure. We evaluated the soil properties at Depot Road to determine the foundation needed for the structure. Combining the road and structure design, an optimal orientation for the structure was chosen along with the foundation and retaining wall design. All of these components make up the final design. We then drew up construction plans in order for construction to begin in summer 2013.

4.1 Road Design

In order to redesign the road running over the existing culvert, various aspects of road design had to be considered including both the pavement design and the parameters required to safely realign the road. As a first step, Team HyGround identified the project area and project roadway type. The Massachusetts Department of Transportation (Mass. DOT) provides a design guide on various types of roadway and areas (6). Using this information and the fact that Depot Road is considered a secondary road in a rural location, we determined that the site was a rural area of a natural type and that the roadway type was classified as a Minor Rural Arterial. This classification was later important in identifying a suitable design for the project site.

4.1.1 Pavement Design

Pavement design is typically based on many factors relating to the roadway classification and environmental features such as soil type present and traffic flow. The design guide provided by the Mass. DOT provides useful information on different pavement types: flexible pavement, rigid pavement, and composite pavement. In Massachusetts, it is common practice to choose a flexible pavement (6); the most commonly used form is Hot Mix Asphalt (HMA), also known as bituminous concrete. The standard layers of HMA used when creating a road are a sub base, a base course, an intermediate course, a subsurface course, and a friction course.

In pavement design, the thickness of each layer is determined using the bearing capacity. However, as pavement design was not a focus of this project, we chose to use pavement materials and thickness that are common for road construction in Williamsburg: the top layer of asphalt consists of 1.5 inches of top coarse and 2.5 inches of binder, and the layer beneath these materials consists of 6 inches of ¾ inch coarse gravel (Figure 10).

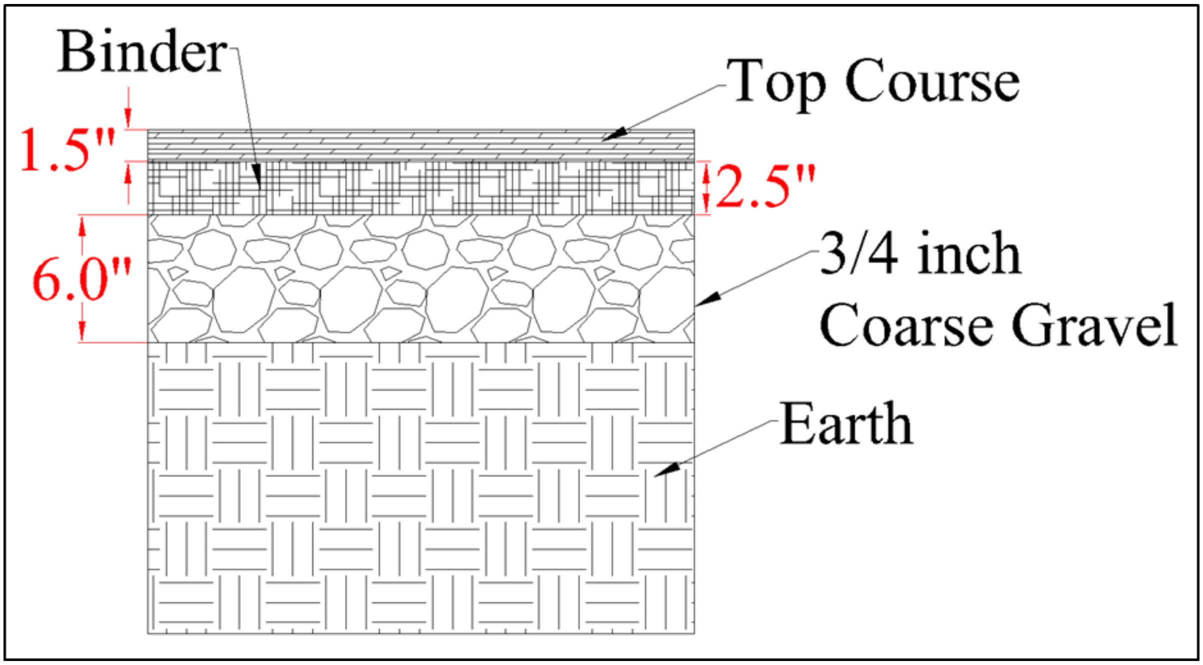


Figure 10. Pavement layering design of realigned road

4.1.2 Road Realignment

To realign a roadway, factors such as road curvature, design speed, and superelevation must be considered. The Mass. DOT design guide and liaison Jim Hyslip both recommended starting with a design speed of 35 mph. For our project area and roadway type, typical design speeds range from 35 mph - 60 mph (10). The design speed is the maximum safe speed that can be taken on the worst part of a curve. The radius for a curve is dependent on the design speed and the superelevation of the road.

Superelevation is the percent banking of a roadway around a curve (9). The purpose of superelevation is to counteract the outward pull that a car experiences when travelling on a horizontal curve (Figure 11). The maximum superelevation allowed is controlled by factors such as climate condition, terrain, area type, etc. In Massachusetts, the maximum superelevation is 6% due to winter snow and ice conditions (9). If a road were banked at any higher percentage, a car traveling in winter conditions would be at risk of sliding toward the center of the curve. However, it is recommended not to use any superelevation for design speeds less than or equal to 35 mph (9). It is also recommended that a normal crown section be used for the roadway (9).

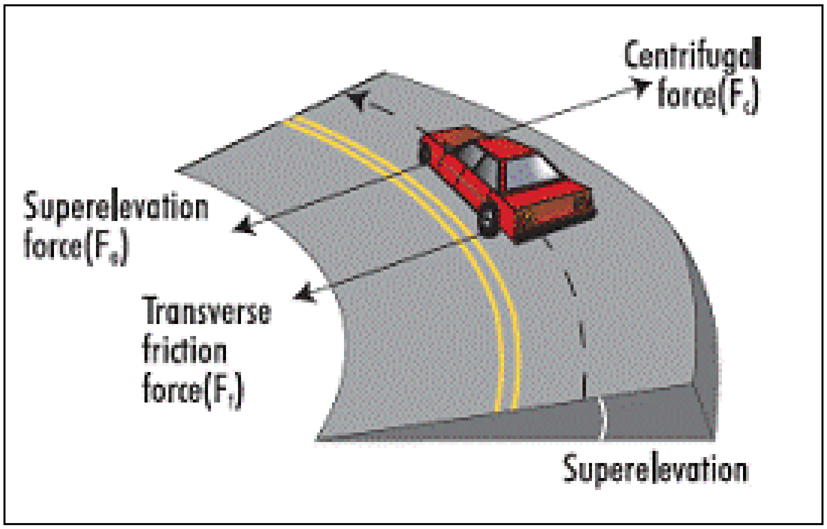


Figure 11. Superelevation on a roadway curve (9)

In order to calculate the radius of road curvature, we determined a minimum radius (R_{min}) of 340 ft. using Equation 1; detailed calculations are provided in Appendix A.

$$R_{min} = \frac{V^2}{15(0.01e_{max}+f_{max})} \qquad \text{[Eq. 1]}$$

where V = design speed (mph)
e_{max} = maximum superelevation
f_{max} = maximum side friction factor
(the friction between the tire and the pavement)

The existing tangents on Depot Road provided an upper limit on the radius the new road could support. We used the minimum radius, the design speed, and existing road constraints to narrow our design superelevation to a few viable options. Ultimately a design superelevation of 4% with a radius of 371 ft was chosen (see Appendix A). A required transition length for the superelevation would be the governing factor on the total length of new road required. At a minimum, we had to reconstruct the section of Depot Rd. that was not repaved in recent years. The section of road that required repaving spanned 270 ft (as designated by liaison Bill Turner).

When superelevation is used on a horizontal curve, superelevation transitioning is required; road changes from a normal crown to full superelevation bank and then back down to normal crown. This is accomplished by rotating the pavement about an axis. For two lane roads, rotation about the centerline profile of the travelled lane is recommended (11). The sections of road that are involved in the transitioning are referred to as the normal crown, tangent runout, superelevation runoff, and the fully superelevated curve as seen in Figure 12.

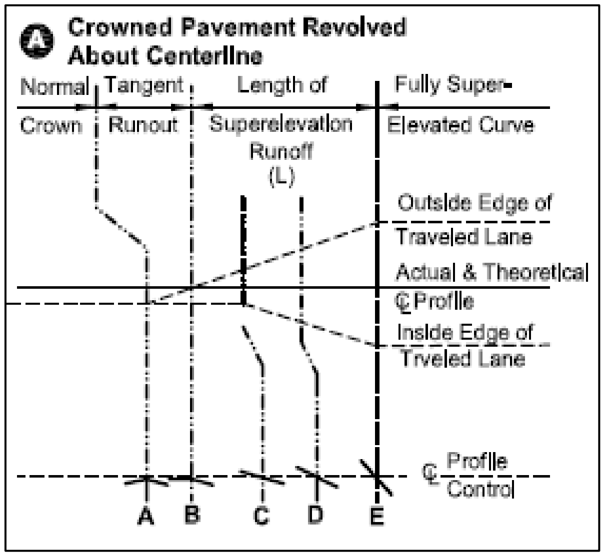


Figure 12. Banking transition for an undivided highway with rotation about the centerline (12)

The tangent runout denotes the length of highway required to accomplish the change in cross slope from a normal section with the adverse crown removed. Superelevation runoff is the length of highway required to accomplish the change in cross slope from a section with the adverse crown removed to a fully superelevated section. The purpose of these road sections is to introduce a superelevation in a safe way. It is common practice to place two-thirds of the superelevation runoff on the tangent section and one-third on the horizontal curve (12).

We used equations provided by both the Mass. Highway Design Guide and the Iowa DOT to calculate the lengths of each section (see Appendix A). Using these values and the AutoCAD model depicted in Figure 13, we calculated a total length of road required of 355 ft. All calculated values are displayed in Table 2 below.

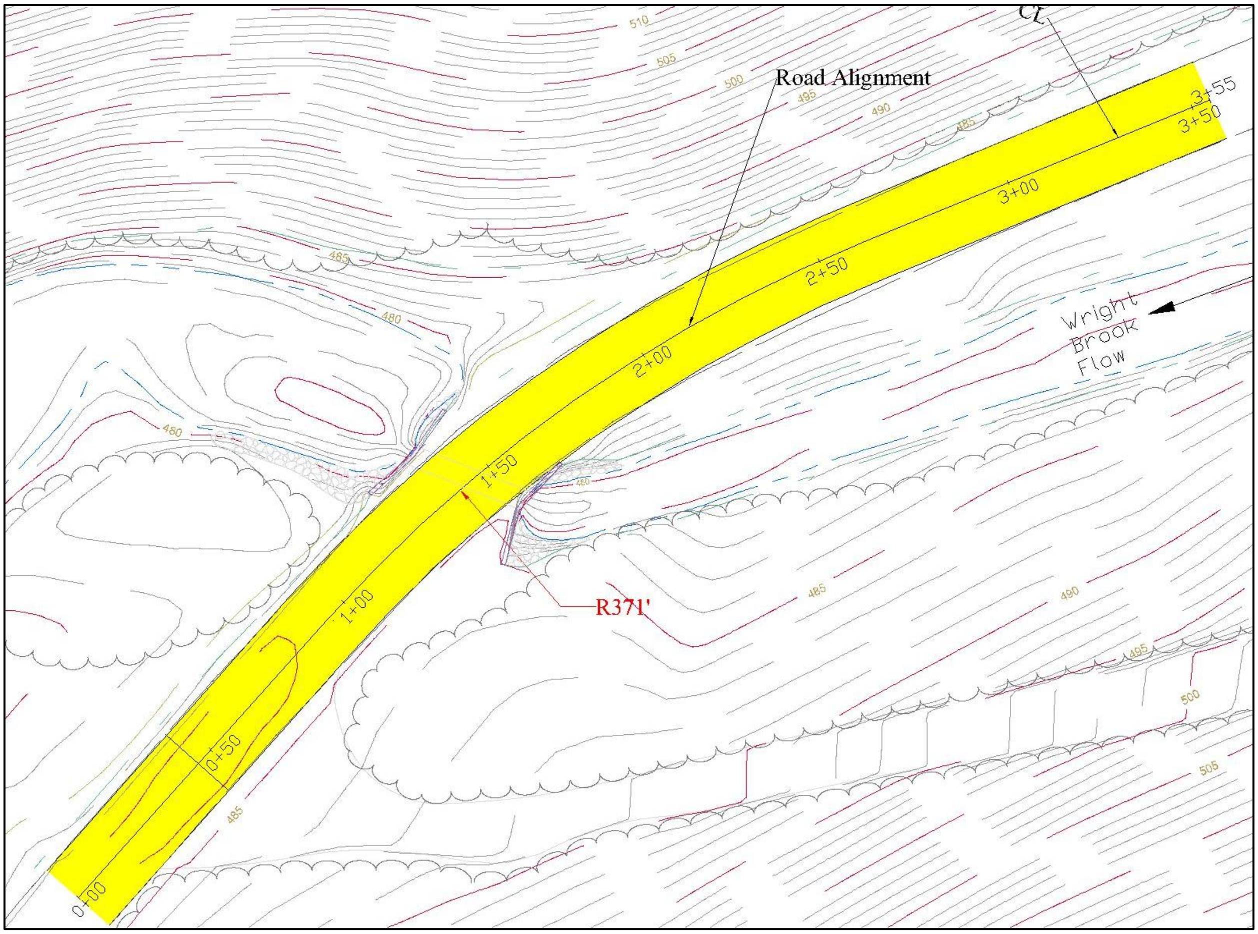


Figure 13. Road design for a superelevation of 4% and design speed of 35 mph

Table 2. Calculated Values for Road Design

Road Design	
Superelevation	4%
Design Speed	35 mph
Radius	371 ft
Superelevation Runoff	84 ft
Tangent Runout	42 ft
Total Length of Road	355 ft

4.2 Stream Crossings

A stream crossing is a structure placed across running water to allow vehicles and pedestrians to pass over. There are two main types of stream crossings: culverts and bridges. We researched both options to make an informed decision on which structure is best suited for the Depot Road site.

4.2.1 Culverts

A culvert is an embedded structure that allows water to pass through and under a road. Culverts can be made from various materials in many different shapes and sizes. The five basic and most common shapes are circular, elliptical, box, arch, and pipe arch (13). The advantages and disadvantages for these shapes are listed in Table 3 below, and comparisons of different materials are listed in Table 4.

Table 3. Advantages and Disadvantages of Different Culvert Shapes (14), (15), (16)

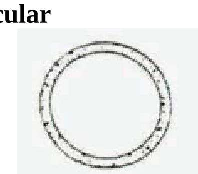
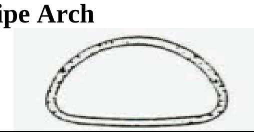
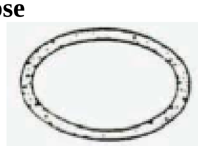
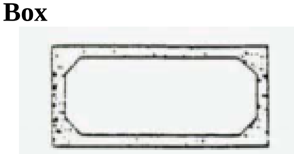
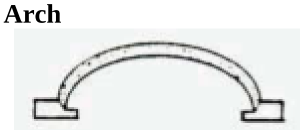
Shape	Advantages	Disadvantages
Circular 	• hydraulically and structurally efficient under most conditions • medium to high stream banks • most common culvert shape • least expensive	• prone to clogging due to diminished free surface as the pipe fills past the midpoint
Pipe Arch 	• optimal when wider sections are needed for low flow areas	• prone to clogging • not as structurally sound as circular pipes
Ellipse 	• when the distance between channel to the pavement is limited or when a wider section is desirable for low flow • typically used for storm water and sewer drainage	• typically more expensive than circular culverts for equal hydraulic capacity • not as structurally efficient • prone to clogging
Box 	• easily adaptable to a wide range of site conditions, especially at sites that require low profile structures • alternative where circular pipes cannot provide adequate flow capacity	• due to the flat sides and top, not as structurally efficient as other shapes
Arch 	• natural streambed • useful for maintaining the streambed integrity (e.g. fish passage) • offers less obstruction to waterway than pipe arches • resembles a bridge and can often be used in lieu of bridges	• important to carefully consider the structural stability of the streambed and scouring potential

Table 4. Advantages and Disadvantages of Different Culvert Materials (17), (18), (19), (20), (21) (22)

Material	Advantages	Disadvantages
Concrete	• prepared either in precast or cast-in-place • readily available • highly durable and strong	• labor intensive to install as it requires heavy equipment due to its weight • weak in sulfate environments • needs to be carefully installed to avoid cracking
Plastic	• made of polyethylene, making them corrosion resistant • very flexible and lightweight • can be laid down by hand without heavy machinery • can also come as corrugated pipes	• least environmentally friendly
Vitrified Clay/Brick	• strong and resistive to acids, alkaline, scouring, and erosion	• not readily available • difficult to install • available only in large diameters • brittle with joints susceptible to chemical attacks
Wood	• ideal for forest road construction • can protect natural stream features while being efficient and durable	• needs to be treated • not as strong as other materials, and depends on defects in the wood • can decay, be damaged easily
Smooth Steel	• can last over 50 years in any climate • can be coated to resist corrosion • lightweight	• prone to rusting and cracking • in some cases can distort and cave in if improperly set
Corrugated Steel	• constructed from a single piece of galvanized steel • corrugated surface slows down water flow making it more fish friendly • flexible and lightweight	• has the same flaws as the smooth steel culverts • can also catch and hold onto debris and silt
Cast Iron	• high in strength, ductile, and corrosion resistant when coated	• heavy and susceptible to corrosion from acidic waste waters
Corrugated Aluminum	• one of the most common materials used today • lightweight and more flexible than steel • high resistance to corrosion, making it useful in high erosion environments like salt water applications	• relatively new material for culverts • abrasion can occur in fast flowing streams with significant loads of sand or rock • requires more care in installation
Stainless Steel	• highly durable and strong	• not a common material • used only in specialized situations

4.2.2 Bridges

A bridge is an elevated structure that allows water or a path to pass under it. According to the Mass. DOT, (6) a bridge is legally defined as any structure that spans over 20 ft. The most common types of bridges are beam, arch, suspension, and truss. Suspension and truss bridges were found to be unfit for the project site due to their large scale and the fact that they require a straight length of road, which cannot be accommodated by our site due to the curvature of the road.

A beam bridge is a horizontal structure that is supported at two ends. Under a load, the top of the beam bridge compresses while the bottom stretches in tension (23). The beam bridge must be strong enough as to not bend under its own weight and the added weight of traffic; as a result, they rarely span more than 250 ft (23). It is common to use concrete or steel beams for these bridges (24). Figure 14 depicts a beam bridge located on Route 9, a few miles down from the project site.

Figure 14. Beam bridge on Route 9 in Williamsburg, MA

Arch bridges, shown in Figure 15, are composed of a curved structure with abutments at each end. The abutments transfer the thrust from an arch to the earth underneath while keeping the ends of the bridge from spreading outward (25). Therefore, the abutments must be heavy and installed only in areas where the ground is solid and stable. The ability of the curve of the arch to dissipate forces outward reduces the tensile force on the bridge to be virtually negligible (26). However, the greater the degree of curvature, the greater the effect tension has on the underside of the bridge.

Figure 15. Arch bridge located in Delaware, NY (22)

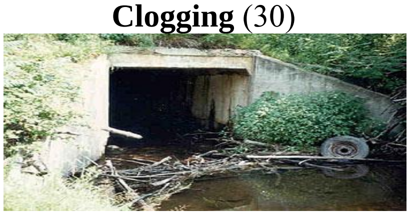
4.2.3 Stream Crossing Selection

Before deciding which structure to pursue, we researched relevant regulations in Massachusetts. One such regulation states that all new stream crossings must adhere to the General Standards presented in the *Massachusetts Stream Crossing Handbook* (7).

According to the handbook, there are three main problems that may occur with stream crossings (7). The first is an undersized crossing that restricts flow. The second is a shallow crossing that has low water depths. Third is a perched crossing that is above the natural stream level. Each of these problems, shown below in Figure 16, can lead to bigger consequences, as outlined in Table 5 (7).

Figure 16. The three main problems that can occur: undersized, shallow, perched crossings

Table 5. Consequences of Stream Crossing Problems (adapted from (7))

Consequence of Problem	Description	Cause
Low Flow (27) 	Low flow is a problem for aquatic life that requires enough water depth to pass through a crossing. Low velocities may also lead to stagnant conditions within the crossing.	• Shallow • Perched
Scouring and Erosion (28) 	In undersized crossings, high water velocities can scour natural substrate, degrading habitat. High velocities may also erode stream banks. Scour pools develop downstream of perches culverts, which may undercut the culvert.	• Undersized • Perched
Unnatural Bed Material (29) 	Certain materials, like concrete and metal, are not suitable for species that travel along the streambed. Crossing substrate should match natural substrate to maintain natural conditions.	• Shallow • Perched
Clogging (30) 	Leaves, debris, and other materials can clog a crossing, making passage impossible. Clogging can also exacerbate the impact of floods.	• Undersized
Ponding (31) 	Water can back up at the upstream of a crossing, leading to property damage, road and bank erosion, and changes in upstream habitat. It can also create new, undesirable wetlands.	• Undersized • Perched

According to the *Massachusetts Stream Crossing Handbook* (7), the best option would be to use either a bridge or a bottomless arch culvert. Box or circular culverts are an acceptable alternative, but only if they are embedded to have a natural streambed. For the Depot Road site, a typical bridge may not be the most practical option due the short, curved road that needs to be redesigned. There is no ideal way to install the bridge without having to reconstruct a large portion of the road leading to it. The site has a limited height available for the stream crossing. Therefore, to have the desired opening area, a longer span is required, while keeping a shorter rise. This rise-to-span ratio is not sufficient for supporting an arch culvert. A circular culvert is also restrictive as a taller height is required for the same opening area. Therefore, the best option would be to use an open bottom box culvert, which is a rigid arch culvert. A comparison of these three types of culverts is shown in Figure 17.

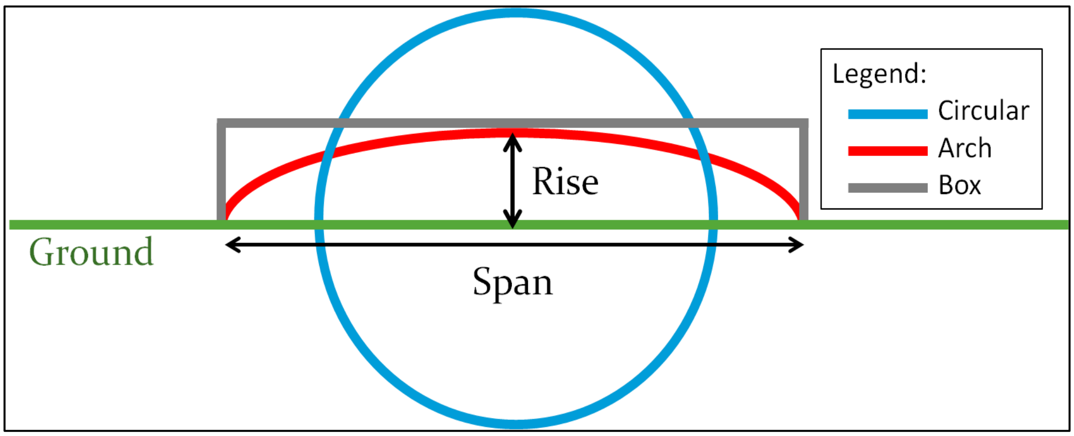


Figure 17. Sizing of three culverts based on a set rise to span ratio

4.2.4 Materials Selection

There are many types of materials used for manufacturing culverts. However, based on the advantages and disadvantages listed in Table 4, we identified concrete and corrugated metal as the two best options. Before selecting which to use, we compared the materials using several factors.

According to the US Army Corp of Engineers, reinforced concrete has an expected lifespan of 70-100 years, while corrugated metal does not exceed 50 years (32). As for price, the initial cost of concrete is high, but the cost of corrugated metal can add up over time due to replacement. The amount of load culverts can carry depend on the material packed above them, due to soil-structure interaction. This is especially true for corrugated metal, as it is flexible and relies on the surrounding fill, or soil cover, to prevent it from collapsing under the load (33).

Today, the most common type of culvert is Corrugated Metal Pipes (CMP) because of its price and ease of handling (34). However, CMP culverts can fail due to a number of reasons including buckling, corrosion, and erosion (Figure 18). Even galvanized corrugated metal can corrode, due to invert corrosion damage. This can occur from reactions with chemically active groundwater contaminated by road salt (35). Concrete culverts are vulnerable to abrasion of structurally critical areas (34). Still, there are more reported cases of failed CMP systems than concrete culverts (36).

Figure 18. CMP culverts failing due to erosion and buckling (36), (37)

In 2011, a CMP culvert failed in Buffalo, TX after only 25 years of use. According to the Texas Department of Transportation (TX DOT), this is the second CMP culvert to fail at that exact location. TX DOT has now replaced the culvert with a reinforced concrete pipe. Another CMP culvert installed in the same town is already buckling and corroding after only 12 years of service (38).

The US Army Corp of Engineers stated in a 1998 report that a 100 year minimum service life should be used when accounting for life cycle design, and the only material that meets this 100 year standard in concrete (32). From this information, our team decided that concrete is the best material option for our box culvert.

4.3 Structure Sizing

In order to find the required dimensions of the replacement structure, we first completed a hydrologic analysis to find the flow rate for a 50-year 24-year design storm and used it to determine the size of the structure through hydraulic modeling. Once we determined the size of the structure, we chose an orientation that would reduce the need for stream training.

4.3.1 Hydrologic Analysis

Hydrology is the study of the properties, distribution, and effects of water on the soils and rocks of the earth's surface (39). The hydrologic cycle (Figure 19) describes the transformation of water from one phase to another and its movement from one location to another (40). The cycle starts with condensation, then precipitation in either the form of rain, hail, sleet or snow which then falls on a watershed. The precipitation that does not evaporate back into the atmosphere will fall on the ground or a water surface. The water at the surface will either evaporate or infiltrate into the soil. If there is more water than the soil can absorb, the water continues on to become groundwater and feeds into streams and lakes which eventually reach the ocean. This process is repeated in the hydrologic cycle.

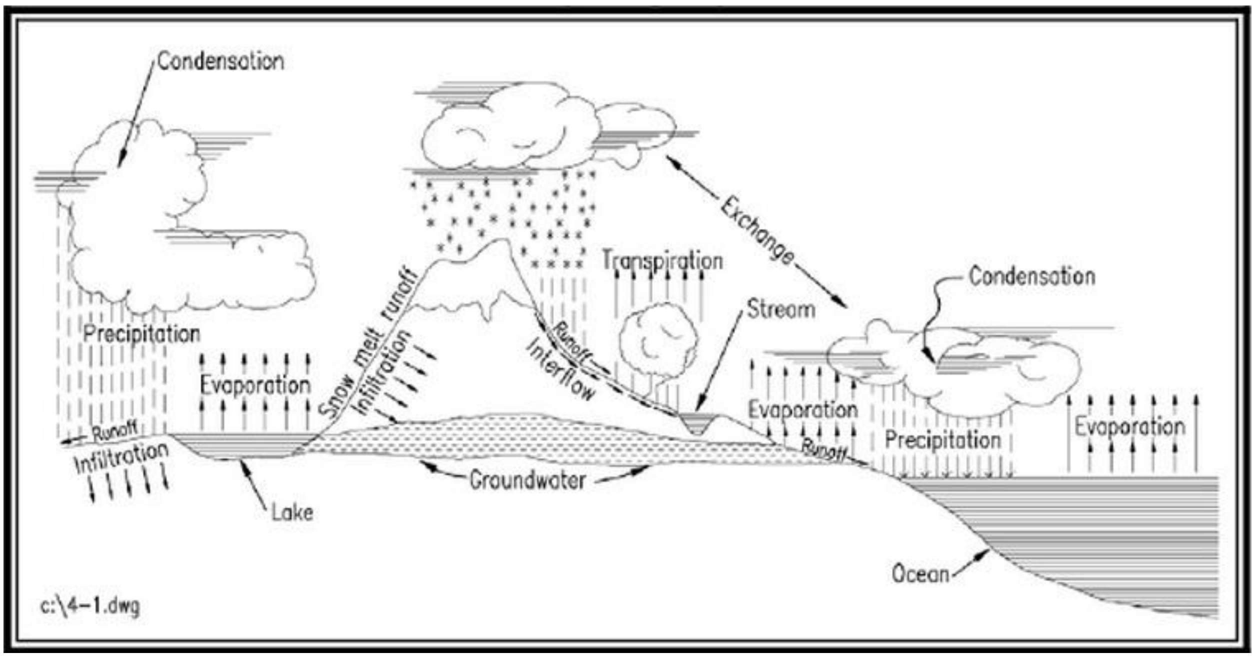


Figure 19. The hydrologic cycle (40).

To perform the hydrologic analysis, we used TR-55, a single-event rainfall-runoff small watershed hydrologic model (41). This National Resources Conservation Services (NRCS) model is the most widely used approach to hydrology in the US when modeling small watersheds. Based on rainfall observations, the Intensity-Duration-Frequency (IDF) curve can be found which indicates the relationship between intensity and duration of a rainfall event with a given return period. TR-55 also provides a way to calculate the predicted runoff using the Rational Method. Using this method, the peak runoff is found by multiplying the runoff coefficient, rainfall intensity, and area. The runoff coefficient is based on the soil, ground cover, and other factors. The rainfall intensity is obtained from the IDF curve for a given return period and duration (42).

To run TR-55, the minimum data requirements include the watershed sub-areas (expressed in acres or square miles), reaches, rainfall storm data, hydrologic soil groups, soil areas, and time of concentration. A watershed is composed of sub-areas (land areas) and reaches (major flow paths) (43).

4.3.1.1 Watershed Delineation

To find the watershed sub-areas, we delineated the watershed. We used a topographic map from ArcGIS online (44) as seen in Figure 20. We located every body of water that led to or would eventually lead to the stream that runs through our current culvert. Then we identified the boundaries of each area that contains all the water that would drain into an individual body of water to find the sub-areas. We found a total of seven sub-areas, shown below.

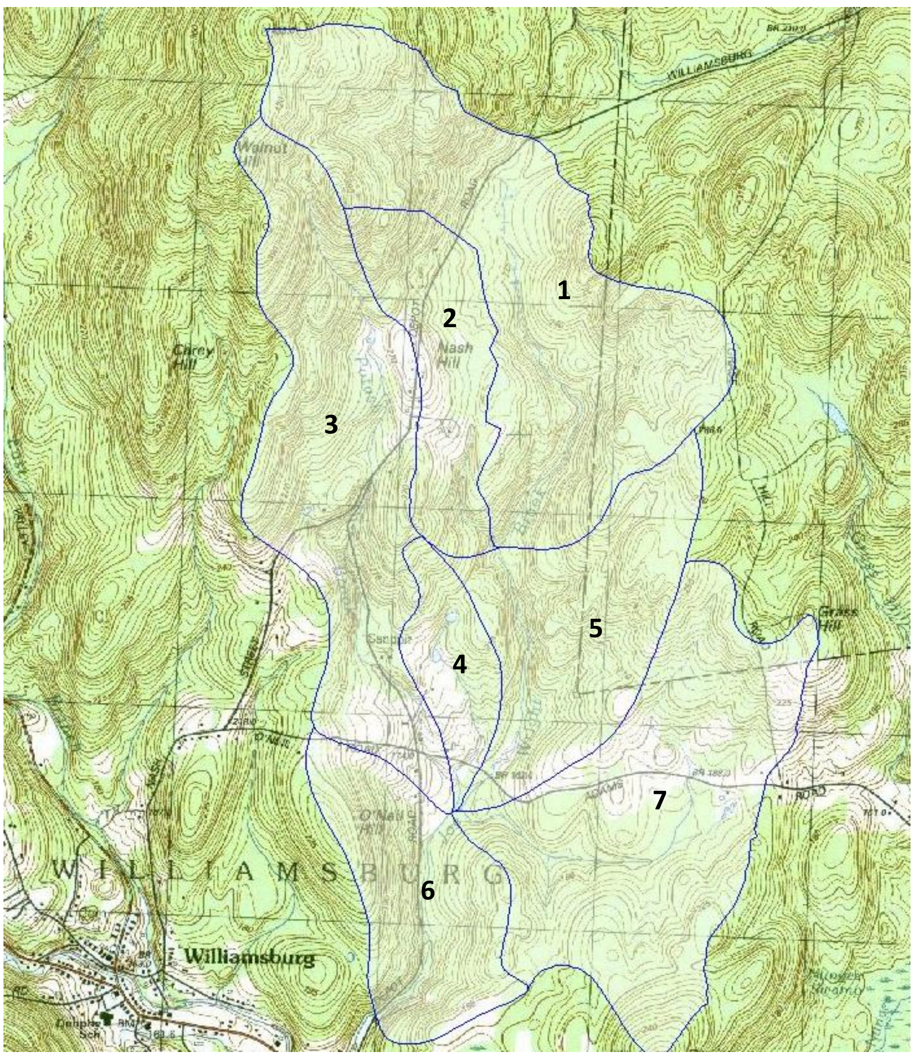


Figure 20. Watershed delineation with 7 sub-areas

4.3.1.2 Runoff Curve Number

To find the runoff factor, we classified all of the soil in the watershed into a hydrologic soil group in order to indicate the minimum rate of infiltration (45). The four hydrologic soil groups are described in Table 6 below (46).

Table 6. Hydrologic Soil Groups

Hydrologic Soil Group	Definition (when thoroughly wetted)	Description
A	Low runoff potential and high infiltration rates - water transmitted freely through soil	sandy, loamy sand, or sandy loam
B	Moderately low runoff potential and moderate infiltration rates – water transmission is unimpeded	silt loam or loam
C	Moderately high runoff potential and low infiltration rates – water transmission somewhat restricted	sandy clay loam
D	High runoff potential and very low infiltration rates – water transmission is restricted or very restricted	clay loam, silty clay loam, sandy clay, silty clay, or clay

We obtained a soil map (47) and found the area that each type of soil covered and then classified each type (see Appendix B). This information is an input in TR-55 used to calculate the runoff curve number. TR-55 calculates the runoff curve number of each sub-area given the total area of each hydrologic soil group in each sub-area. It combines the NRCS runoff equation (Eq. 2) with unit hydrograph theory for the computation of runoff rates.

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \qquad \text{[Eq. 2]}$$

where Q = runoff (in)
P = rainfall (in)
S = potential max retention after runoff begins (in)

4.3.1.3 Time of Concentration

The time concentration is the time it takes for runoff to travel to a point of interest from the hydraulically most distant point (48). To find the time of concentration, we used our watershed map and found the length from the furthest point of the watershed to the culvert and categorized that length into sheet flow, shallow concentrated flow, and channel flow for each sub-area. Sheet flow occurs when the water is traveling above ground (maximum of 100 ft in TR-55). Shallow concentrated flow is calculated as the length of flow above ground (after 100 ft) until it meets a stream. Channel flow represents the length of the channel. An example of the time of concentration worksheet for sub-area 1 can be seen in Appendix C where we detail the lengths of each flow and Manning's coefficients. From the worksheet, we obtained a time of concentration for each sub-area as shown in Table 7.

Table 7. Time of Concentration for Each Sub-area in Watershed

Sub-area	Time of Concentration (hr)
1	1.26
2	0.75
3	1.16
4	0.71
5	1.22
6	1.39
7	1.17

4.3.1.4 Results

After inputting all the necessary information, we ran TR-55 to obtain flow rates for 6 different 24-hour design storms (summarized in Table 8).

Table 8. Flow Rates of Different Design Storms for Hydrologic Modeling

Design Storm (24-hour)	Flow Rate (cfs)
2-year	373
5-year	741
10-year	1023
25-year	1379
50-year	1755
100-year	2034

4.3.2 Hydraulic Analysis

A hydraulic analysis is based on the principle of open-channel fluid mechanics. Hydraulic modeling takes the flow rates found from a hydrologic analysis and a stream's geometry to determine the velocity of the stream for different conditions. Hydraulic modeling iteratively solves Manning's Equation, shown in Equation 3, for open-channel flow in one dimension at cross sections along the stream. This is done by relating volumetric flow to cross sectional area and the hydraulic gradient (downward slope of the channel) while also accounting for energy loss from water flowing along banks and bed (42).

$$Q = \frac{k}{n} A (R_h)^{\frac{2}{3}} (S_f)^{\frac{1}{2}} \qquad \text{[Eq. 3]}$$

where Q = volumetric flow rate discharge (ft^3/s)
k = constant value (1.49 for US units)
n = Manning's roughness coefficient
A = area of cross section (ft^2)
R_h = hydrology radius (ft)
S_f = longitudinal slope of stream (ft/ft)

4.3.2.1 Setting Up HEC-RAS

In order to perform the analysis, the team used the computer program HEC-RAS, created by the Army Corp of Engineers, to model the hydraulics of flow in the channel (49). HEC-RAS requires the geometry of cross sections along the stream channel. However, due to weather restrictions from winter snowfall, the team was unable to physically survey the site and thus relied on the elevation map of the site provided by HyGround (see Appendix D). The team understands that the contours on the map will not be as accurate as actual measurements, but they serve as a good approximation. Using the map, the basic geometry of the stream as well as cross sections were drawn onto the map using Civil-3D and then exported to HEC-RAS (refer to Appendix E). HEC-RAS was then able to extract the cross section data directly from the elevation map.

After transferring the geometry, the channel conditions were entered. The values used for the roughness coefficients were based on the stream's natural features and the corresponding Manning's number as suggested by the HEC-RAS manual (50). The values are summarized in Table 9 below.

Table 9: Manning's Numbers Used in HEC-RAS Model

Location of Material	**Type of Material**	**Manning's Number** (50)
Stream Channel	Natural Earth	0.02
Culvert	Smooth Concrete	0.013
Gabion Walls	Rough Gravel	0.05

The design storm flow rates (see Table 9) from the hydrologic analysis were then inputted into HEC-RAS.

4.3.2.2 Modeling with HEC-RAS

As a baseline, the current 5 ft diameter CMP culvert was the first geometry modeled in HEC-RAS. Based on the results, the current culvert is extremely undersized as it overtops more than 1.5 ft and the velocities are over 12 ft/s at just the 2-year 24-hour design storm. After seeing these results, the team modeled different culvert sizes, while keeping to a maximum culvert height of 4 ft. This cap was set because of the amount of soil cover and road required above the culvert as well as the limited space between the road and streambed.

As the Rt. 9 culvert downstream has an area of 60 ft^2, the team looked into modeling a 4'x16' open bottom precast concrete box culvert (area of 64 ft^2). HEC-RAS does not have a specific option for an open bottom culvert, therefore the stream channel's Manning's number was used for the inside bottom of the culvert while the Manning's number for the walls were set as smooth concrete.

According to the HEC-RAS model, the 4'x16' culvert would still be undersized, having over a foot of overtop and velocities close to 12 ft/s at the 10-year 24-hour storm. Trying to stay within the opening area of the Rt. 9 culvert, the team modeled a 4'x20' culvert, a 30% increase in area. This modeled seemed more promising as it seemed to better accommodate the 10-year 24-hour storm with less than a foot of overtop and velocities closer to 11 ft/s. However, at the 25-year 24-hourstorm, the 4'x20'culvert performed similarly to the 4'x16' culvert at the 10-year 24-hour storm.

Using HEC-RAS, the team was able to see the cross sections along the channel. Through this the team noticed that the natural bankfull width of the channel is 29 ft. In order to meet the standards listed in the *MA Stream Crossing Handbook*, the new culvert needs to be at least 1.2 times the bankfull width (7). This meant that the new culvert needs to be closer to a 35 ft span. This also meant that the replacement structure will now be considered a bridge (6).

The HEC-RAS model was updated to reflect the new size, but further changes were made to the geometry of the channel. The stream has a natural linear slope leading into the culvert, but there is a large drop in the streambed at the outlet due to ponding (shown by the blue line in Figure 21). The team noted that the outlet of the bridge would have a larger opening than the inlet due to this sudden drop (though the opening is 4 ft, the bridge will be at least 7 ft tall in order to properly embed the footings, as discussed further in Section 4.3.3). In order to account for this, the team and liaisons agreed that the streambed slope should be altered. During construction, the upstream streambed will be dug out to keep a gradual slope while the streambed at the outlet will be filled in (as shown by the green line in Figure 21).

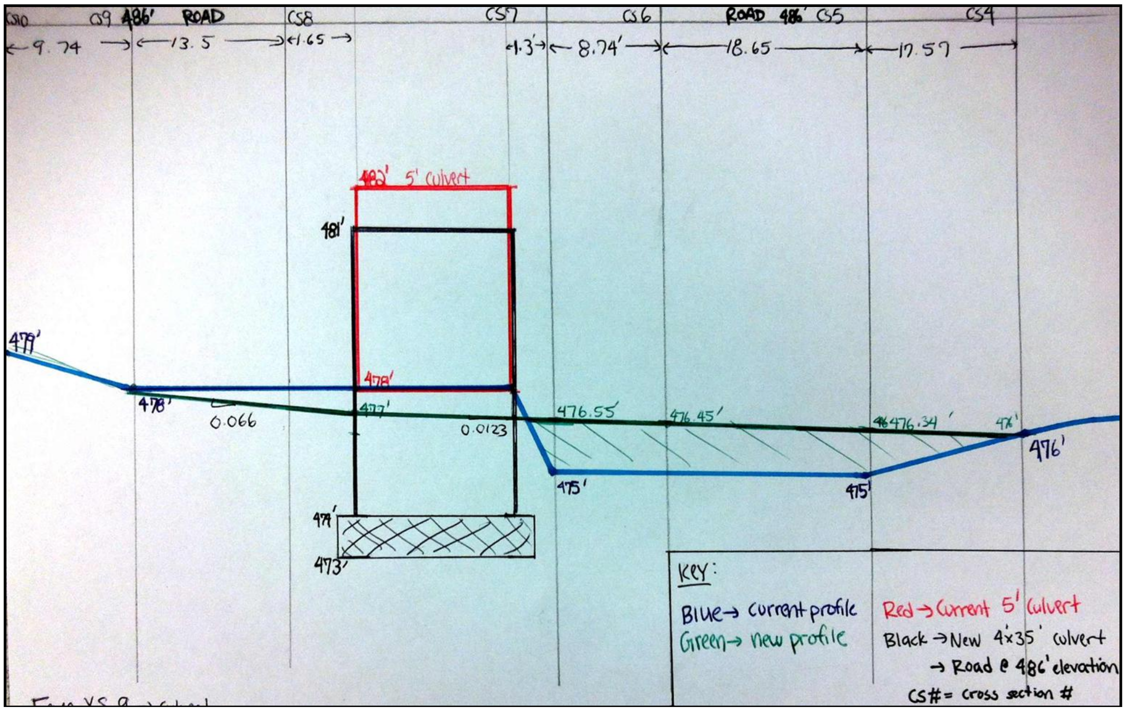


Figure 21. Profile view of streambed's slope

Further revisions were also made to the cross sections to make them smoother. The upstream cross sections were tapered such that the channel slowly widened up to the size of the bridge opening as seen in Figure 22 below (examples of cross sections are found in Appendix F). A similar adjustment was made to the downstream cross sections so that they opened wider after the bridge.

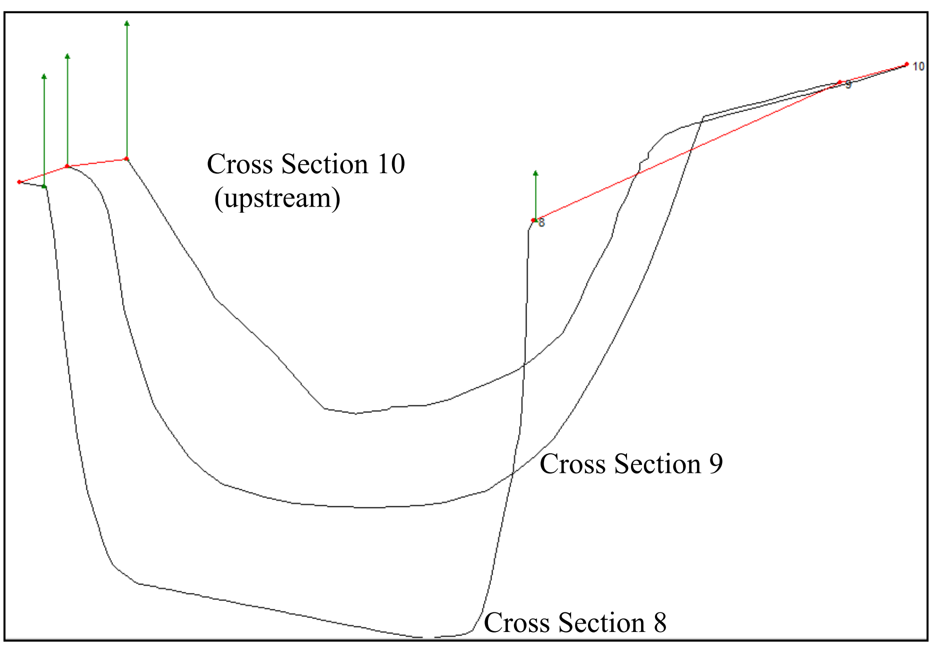


Figure 22. 3D view of upstream cross sections 8-10 tapering out

4.3.2.3 Results from HEC-RAS

The final model combined the new opening of 4’x35’, the new slope of the streambed, and the tapering of the channel before and after the bridge. A different Manning’s number (Table 9) was used for the banks of the channel to model potential gabion walls that would help train the stream toward the bridge and protect the bank from erosion. The velocities of the cross sections before and after the bridge, as well as the velocities through the bridge at different design storms were recorded. Table 9 summarizes these results as well as the predicted amount of overtop onto the road.

Table 10: Velocities and Height of Overtop at Different Design Storms for Final Model

Design Storm	**2-year**	**5-year**	**10-year**	**25-year**	**50-year**	**100-year**	
Flow Rate (cfs)	373	741	1023	1379	1755	2034	
X-Section	**Velocity (ft/s)**						
6	1.9	3.1	3.8	4.6	5.3	5.8	DOWNSTREAM
7	2.2	3.6	4.4	5.4	6.2	6.8	DOWNSTREAM
Culvert OUT	2.7	5.3	7.3	9.9	11.6	12.3	
Culvert IN	2.7	5.3	7.3	9.9	11.6	12.3	
8	2.9	4.2	4.6	4.9	5.1	5.4	UPSTREAM
9	5.2	6.6	6.6	6.2	6.0	6.1	UPSTREAM
	Overtop (ft)						
Culvert IN	0	0	0	0	0.8	1.6	
Culvert OUT	0	0	0	0	0.7	1.2	

4.3.3 Sizing

The final span of the 3-sided bridge was selected to be 35 feet, based primarily on the *MA Stream Crossing Handbook* (7) which states a stream crossing must span at least 1.2 times the bankfull width. The bankfull width of Wright Brook is approximately 29 ft, so the opening must be 35 ft. The length (in the direction of water flow) of the bridge needs to be 35 ft due to the orientation under the road. The height of the exposed opening was determined to be 4 ft to fit site constraints and the need for 2 ft of soil cover (6). Since we are using a 3-sided bridge, the actual height will need to be greater than 4 ft to allow for embedding. The minimum amount of embedding the bridge is 2 ft (51). After taking soil borings at the site, we found that bedrock was approximately 10 ft below the road. We decided to increase the height of the bridge by 1 ft so that the footings could be on bedrock. The bridge will be embedded 3 ft. with a 4 ft opening for a total bridge height of 7 ft.

4.3.2 Orientation

Given the road alignment design discussed in Section 4.1.2 we had to consider the orientation options for our bridge. Initially our goal was to avoid the need for a stream training structure. To do this, we tried aligning the bridge parallel with the stream so that the water could flow straight through. However, if the bridge were placed with the outlet at the existing culvert outlet location, the bridge would have to extend substantially beyond the road and would be visible. This could conflict with our design requirement of keeping the design aesthetically pleasing and fitting with the existing site, so we eliminated this option.

Next we considered placing the bridge downstream of the existing culvert. This design had the bridge parallel to the stream at the existing inlet and shifted the outlet further downstream. This alignment would require excavating the ground and displacing the location of Wright Brook downstream of our structure. Not only would this design require permission from the downstream landowner, but also there might be negative impacts related to disrupting the existing environment. These reasons motivated us to also reject this option. The selected orientation option is shown in Figure 23 below.

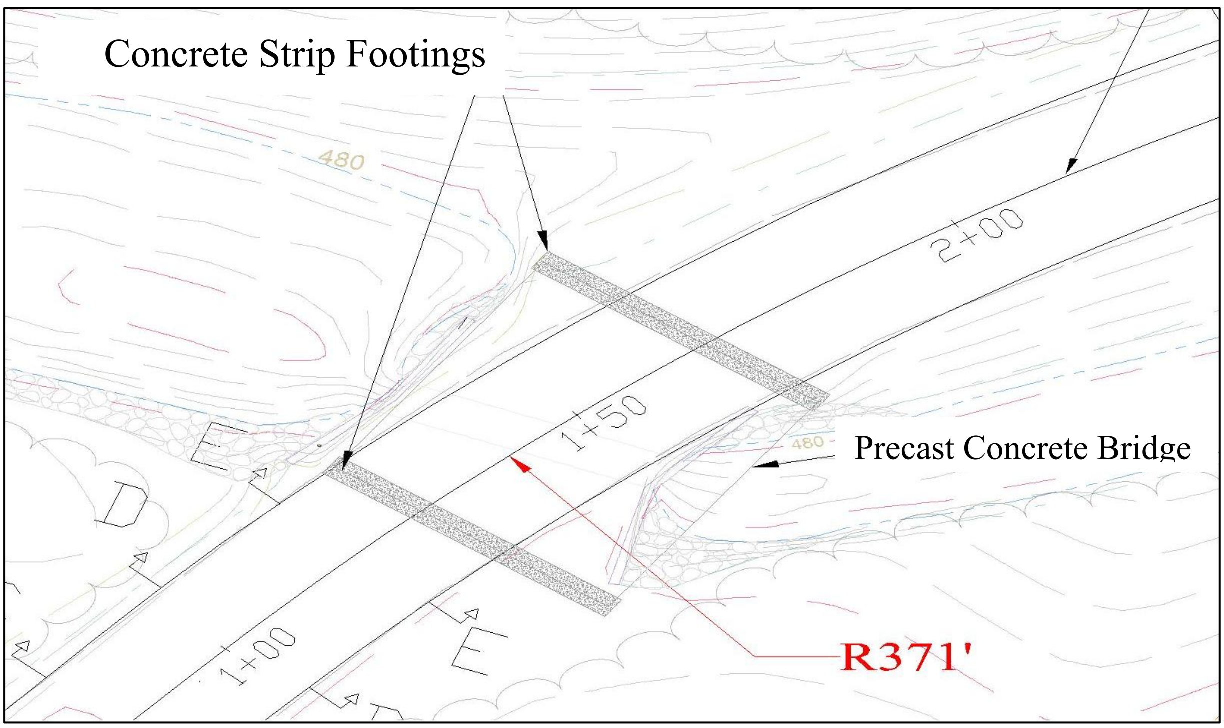


Figure 23. Orientation for culvert replacement structure

The selected orientation requires a stream training structure to help direct channel flow into the bridge and protect the area around the inlet from scouring. Figure 24 shows an example of a wingwall, which is one such steam training structure.

Figure 24. Example of a wingwall (52)

There are several types of wingwalls available. According to Glenn Robie, a representative from the culvert manufacturing company Atlantic Industries, cast-in-place concrete wingwalls are typically more expensive and take longer to install, but they are a good option for smaller structures (53). On the other hand, he advised that precast concrete may not be practical for a small wingwall. Corrugated steel wingwalls are durable, moderately priced, and fast to install. Wirewall is ideal for a fast and economical installation. Wirewall uses wire components and a selected backfill material.

Our liaison, Bill Turner, suggested using a gabion wall, a type of wirewall with which he has worked previously. Gabion walls are usually made of stacked stone-filled gabions tied together with wire (Figure 25). An advantage of gabion walls is that they can shift and conform to ground movement (54). They can also disperse energy from moving water while draining freely. Soil can also fill the voids within the wall, reinforcing it. The lifespan of a gabion wall is mainly dependent on the wire used; the wall only fails when the wiring fails. Based on Bill Turner's suggestion and further research, we decided to use gabion walls for our design.

Figure 25. Examples of gabion walls and gabion headwall (55), (56)

4.4 Foundation & Retaining Wall Design

In order ensure proper support of the replacement structure, we had to design a set of foundations. A foundation is an engineered interface that transmits loads to the underlying soils (57). Foundations can either be deep or shallow. Deep foundations are designed to support highly loaded structures, while shallow foundations are suitable to support lightly loaded structures. Since the structure we were designing for our site would be considered lightly loaded, we chose to design a shallow foundation, specifically a strip footing.

Strip footings are typically used to support wall loads (Figure 26). For our design, the sides of the bridge were modeled as walls, hence a strip footing was a suitable foundation. Design of a strip footing is based on soil present at a site and the associated properties, the expected loads, and various environmental factors present. In-situ tests and laboratory tests were used to identify the physical and mechanical properties of the soil at our site.

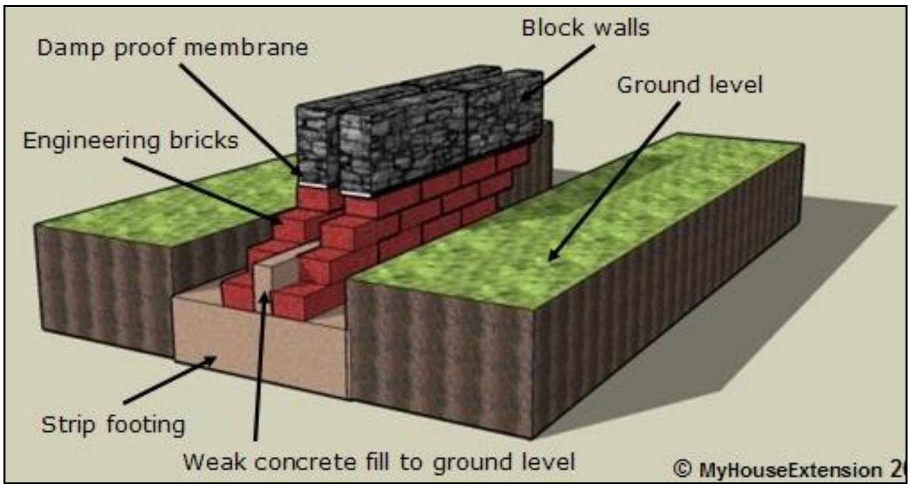


Figure 26. Example of a strip footing under a wall (58)

4.4.1 Subsurface Investigation

We performed the standard penetration test in-situ, following American Standard Testing Method (ASTM) D1586, on December 15, 2012 to characterize the soils at our project site. To obtain the soil types and the associated properties present at the site, we performed a subsurface investigation that consisted of drilling four borings on site (at locations B1-B4 noted in Figure 27). The drilling device we chose to use in our investigation was a Hollow Stem Auger (HSA). This is a commonly used drilling device that acts as a casing for each borehole during sampling. We used the HSA to advance each boring and a obtained samples using a Split Spoon Sampler. A more detailed explanation of the subsurface investigation and the data obtained can be seen in Appendix G. We determined that the bedrock at our site was located at an approximate depth of 12 feet below the surface. During drilling we observed the groundwater table to be at a depth of 5 feet below the ground surface.

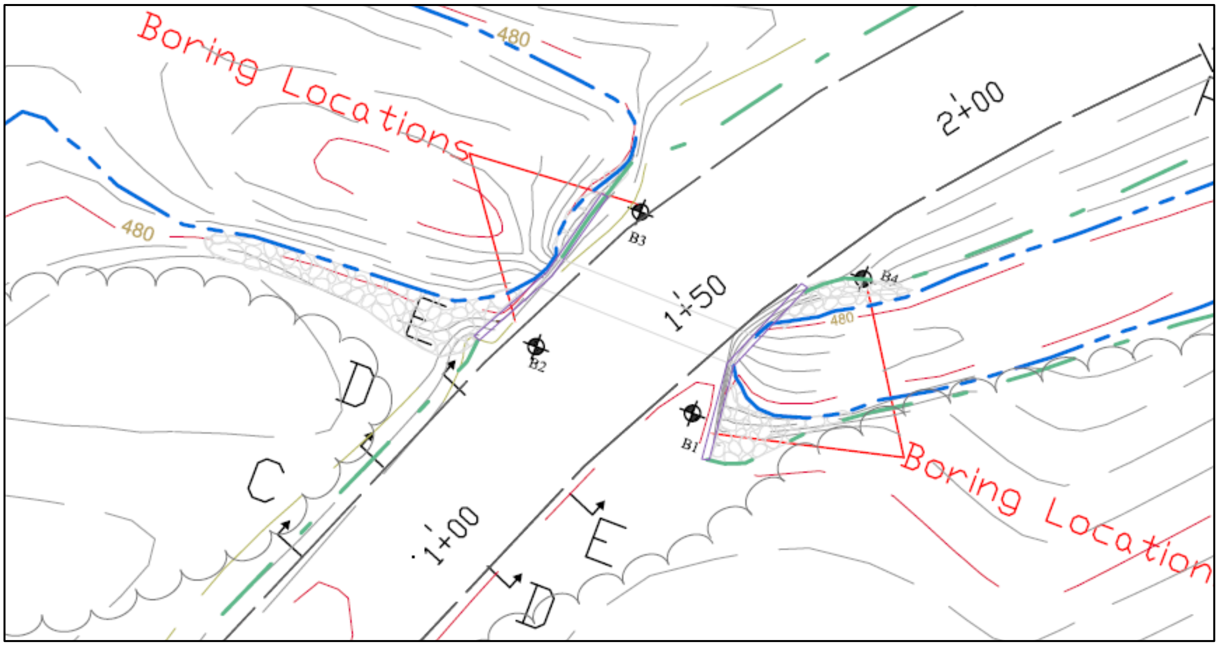


Figure 27. Boring location plan for borings 1, 2, 3, and 4

4.4.2 Soil Classification

Soil classification is an important task when creating designs for support or retention of a structure. Soil can be classified using both in-situ and laboratory methods. The common laboratory test performed for soil classification is for a grain size analysis, which provides a general classification of either coarse-grained soil or fine-grained soil. Silts and clays are considered fine-grained soil, while sands and gravels are coarse-grained. More distinct classification is based on the quantity of particular soil particle sizes. To obtain this information, we followed ASTM-D422 (59). For the purposes of our project, we followed the grain size scale defined by the Unified Soil Classification System (USCS) (60). The classification provided from this system was based on material passing the 75 mm sieve (No. 200 sieve). In addition to utilizing grain size and grain size distribution in the classification of coarse-grained soils, the USCS can determine the engineering behavior and classification of fine-grained soils by utilizing the related plasticity (see Appendix G for details).

After performing the soil classification following ASTM-D422, we determined that the soil from our project site was primarily coarse-grained. However, after reviewing the results from the testing we concluded that the results were not reliable enough to be used for more detailed classification. Reasoning behind this conclusion involved alterations made on the testing method as discussed in more detail in Appendix G. The in-situ classification was based on the Standard Penetration Testing results obtained throughout our subsurface investigation, as well as a physical classification of the soil performed on the collected samples. To perform a physical classification of the soil, we followed the Visual Manual Procedure detailed in ASTM D2488

(61). Overall we determined that the primary soils present at the site were course grained sands with some fines (see Appendix G for details final on classification).

4.4.3 Foundation Calculations

For design of each strip footing, we calculated the sizing of the footing and the reinforcement required. In a strip footing the width is the governing dimension for design since that is the direction in which failure would most likely occur. The width dimension parameter for each footing is dependent upon the ultimate bearing capacity and the expected settlement. The overall load of the new structure and everything above it was calculated to be approximately 31kips. This load was divided between the two desired footings. An ultimate bearing capacity of 19 kips for a 3 foot wide, 1 foot long section of a strip footing was calculated using Equation 4, and as detailed in Appendix G.

$$q_u = \gamma D_f (N_q - 1) s_q d_q r_q w_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma r_\gamma w_\gamma \qquad \text{[Eq. 4]}$$

where q_u = ultimate net bearing capacity
N_q and N_γ = bearing capacity factors
s_q and s_γ = shape factors
d_q and d_γ = depth factors
r_q and r_γ = inclination factors
w_q and w_γ = groundwater factors

The allowable bearing capacity was primarily calculated using a conservative unit weight value for the soil encountered at our site, the computed ultimate bearing capacity and an assumed factor of safety of 3 as is typical for shallow foundations (57). The allowable bearing capacity for the 3 foot wide footing was determined to be 7 ksf. Typically if the allowable bearing capacity is greater than the applied load, then the chosen dimensions for the footing design are acceptable. Based on the load experienced by the 3 foot wide footing, an applied load of 5 ksf was calculated. This value was much less than the calculated allowable bearing capacity, subsequently confirming that the chosen dimensions were adequate as well as overdesigned.

In order to further verify our width, we checked the elastic settlement. Our liaison Jim Hyslip requested that the settlement be no greater than 1 inch. Using the dimensions and allowable bearing capacity, we calculated an elastic settlement of 0.4 inches by utilizing Equation 5.

$$\rho_e = \frac{2Q_a}{EL_1}(1 - v^2{}_u)\mu_s\mu_{emb}\mu_{wall} \qquad \text{[Eq. 5]}$$

where ρ_e = Elastic Settlement
Q_a = allowable vertical stress
E = elastic modulus
L_1 = length of circumscribed rectangle
ν_u = Poisons ratio for undrained consolidation
μ_s = shape factor
μ_{enb} = embedment factor
μ_{wall} = wall adhersion factor

Because this was less than 1 inch we had greater confidence in the chosen design. The overall footing design selected was a 3'x 1'x 36' footing (Figure 28).

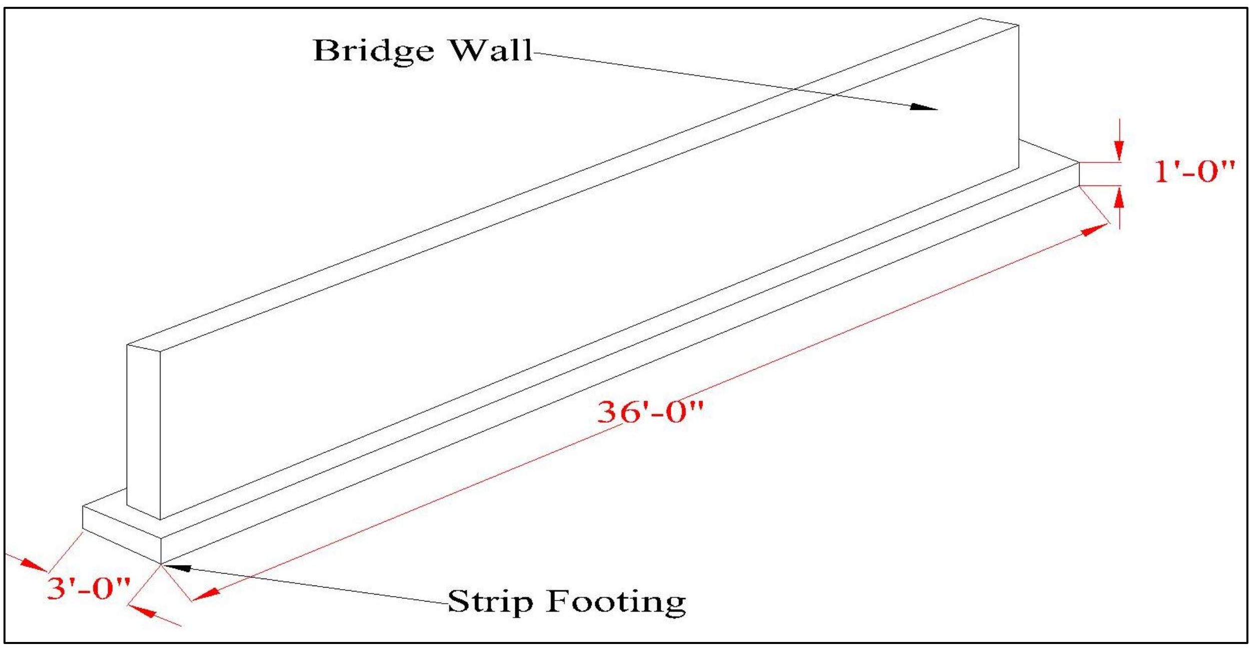


Figure 28. Schematic of strip footing design

Reinforcement is required in footing design to counteract expected flexure, prevent shearing, and to aid in the transfer of structural loads (62). When calculating the required reinforcement of the strip footing, we examined both the shear and the moment that the footing would experience under a given the given wall load. Overall, we found that a No. 5 bar with 9 inch spacing would adequately counteract the moment that our footing would experience (see Appendix G for all moment calculations). We determined that 3 No. 5 bars with 12 inch spacing would adequately serve as perpendicular shrinkage reinforcement (63). The overall reinforcement design for the footing design can be seen in Figure 29 below.

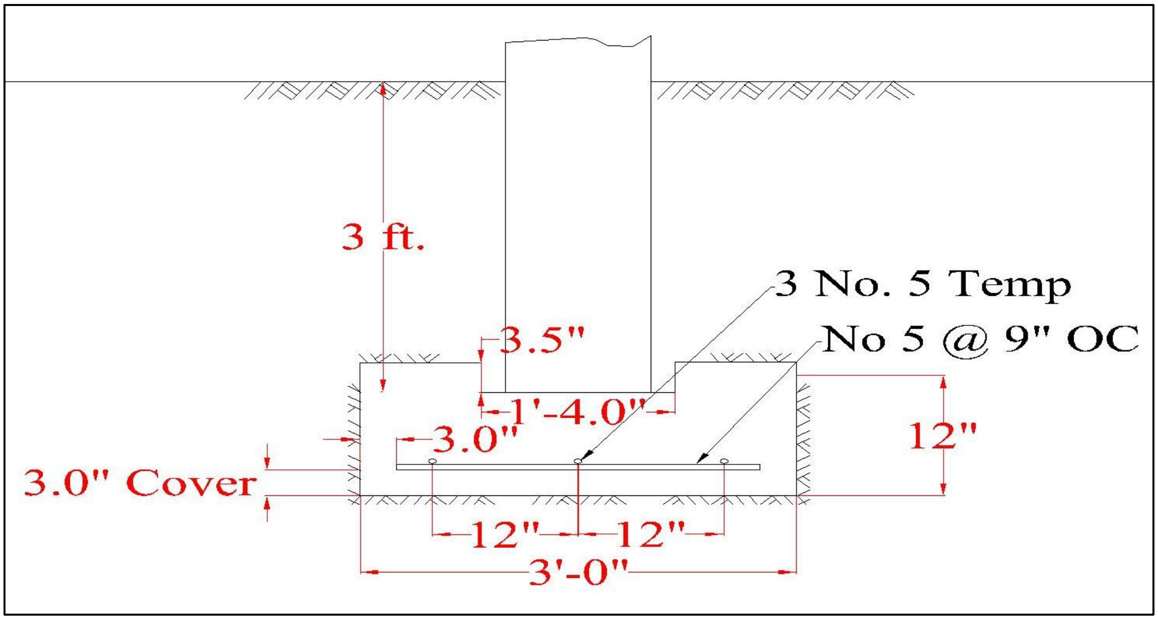


Figure 29. Details of strip footing reinforcement

In order to attach each bridge wall to its respective footing, a cold joint was designed (See Figure 29). This joint consisted of a 3.5 inch build up from the top of each footing. Once the joint for each footing has been poured and set, they will be leveled and the bridge walls will be placed within. When the walls have been properly situated and positioned, a grout will be used to fill in any gaps and secure the wall in place.

4.4.4 Retaining Wall Calculations

After finalizing the foundation design, we designed a retaining structure to safely maintain the soil around the inlet and outlet of the bridge as well as train the stream into the bridge. The type of retaining structure we chose was a gabion gravity wall. In order to conserve space by the stream, a general orientation chosen for our gabion wall design was for a stepped back face with a 6 ft batter (see Figure 30).

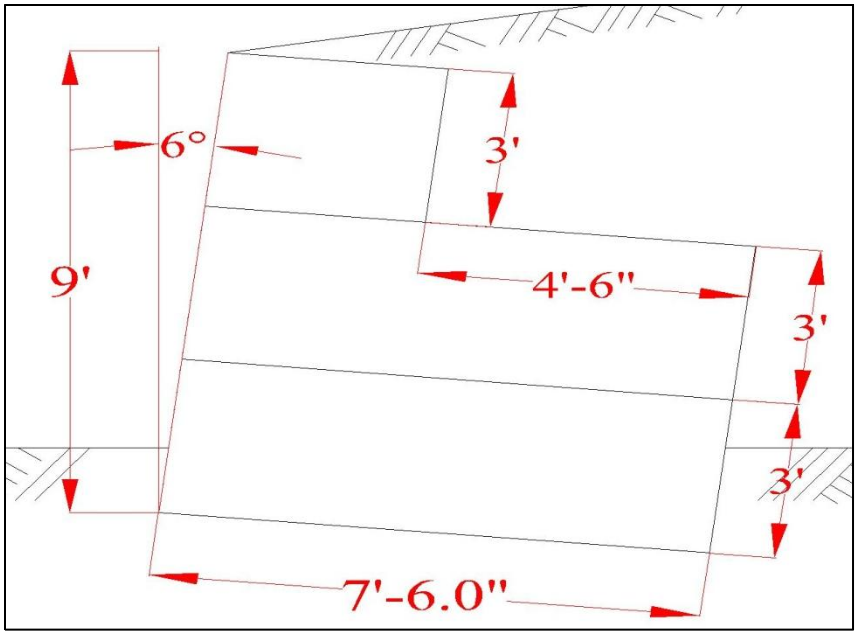


Figure 30. Stepped back face gabion wall orientation with dimensions (64)

In order to determine whether or not each gabion wall design was adequate and could safely meet our needs, we compared the resisting moment of the wall to the overturning moment that would be created by the force imposed from the soil (64). The calculations that governed our design can be seen in Appendix G6. The final design chosen for walls has a total height of 9 ft, consisting of 3 layered sections, and a bottom width of 7.5 ft with a span of 6 ft per unit. The layout at the site can be seen in Figure 31 below.

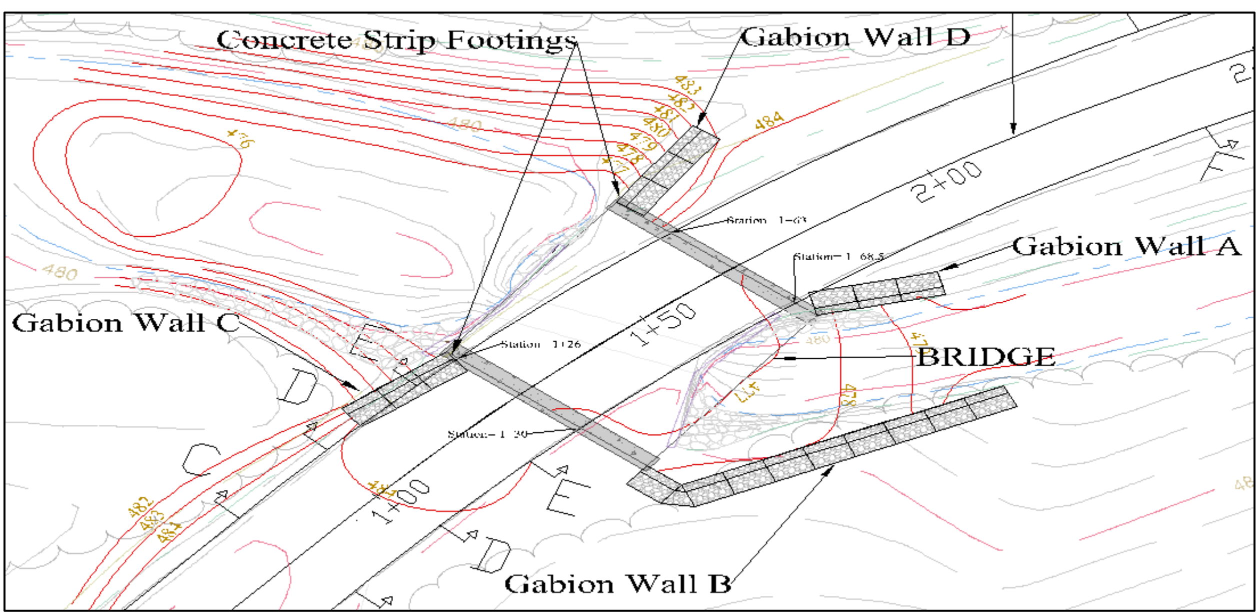


Figure 31. Gabion wall locations at project site

5.0 Final Design/Deliverables

The final structure design is a 3-sided precast concrete bridge with a 35' span x 7' height x 35' length. It will be supported by 2 strip footings with dimensions 3' span x 1' height x 36' length. The new road realignment design will safely support a 35 mph design speed and 4% superelevation.

Our final deliverable is a complete design package consisting of construction plans, a full cost estimate, an executive report, and documents to support future permitting. The construction plans include drawings depicting the bridge specifications, placement, and foundation details. Plans will also be drawn to guide the construction of the new road alignment. We have included a schedule for construction staging. The full cost estimate includes pricing for the bridge, road, foundation, retaining walls, equipment and labor.

6.0 Design Verification

As detailed in Table 11 below, the final design meets most of the design requirements specified. However, the biggest issue is that the replacement structure is not designed to handle the intended 50-year 24-hour design storm. Instead, the 4'x35' opening can handle flow up to a 25-year storm with no overtopping and with velocities under 10 ft/s. For a structure to accommodate the flow of a 50-year storm, it would require a span of at least 46 ft. This span is extremely long and does not seem fitting or reasonable for such a small project site. Also, given the 60 ft^2 Rt. 9 culvert downstream, it is irrational to place a culvert upstream that has a 300% increase in area of opening.

Table 11. Verification of Final Design Against Design Requirements

Design Requirement	Design Verification
The new road geometry will safely accommodate a design speed of 35 mph.	The specified 4% superelevation and radius of curvature of 371 ft will ensure safe driving at 35 mph.
The new road will have a minimum length of 270 ft.	Bill Turner planned on reconstructing 270 ft of the road due to poor conditions. The road realignment calls for 355 ft of new road.
The percent banking, or superelevation, on the curve of the roadway will be no more than 6% (9).	The specified superelevation is 4%.
The new road must be able to support the live load of traffic, approximately 250 psf (based on maximum live loading expected).	The road cross-section is the same as that used elsewhere in Williamsburg that has a proven performance with the expected loading.

Table 11. Verification of Final Design Against Design Requirements (continued)

Design Requirement	**Design Verification**
The replacement structure must support both the dead load of the road and soil as well as the live load of the traffic. If the structure requires a separate foundation, the foundation must be able to support the total load of the structure.	The verification of the structural stability of the bridge will be done by the manufacturer. For the bridge, the strip footing foundation was designed to support 15 tons, or the total load above.
Due to its location on a secondary road, structure must be able to accommodate the 50-year, 24-hour storm flow rate (8).	Based on the hydrologic modeling, the final proposed structure will be able to handle a 25-year 24-hour design storm without overtopping and with no greater than a 10ft/s velocity. In order to handle a 50-year storm, the bridge would need at least a 46 ft span, which seems unreasonable for the site especially given the downstream culvert that is only 15 ft wide (and also 4 ft high).
The road and structure will not need to be replaced within 100 years.	According to manufacturer of the culvert, the expected life is 100 years. Though the road will most likely need to be reconditioned every 20 years (personal correspondence (65)), it will not need to be redesigned.
The final design cost estimate does not exceed the town's allotted budget of $250,000 for this project	The total project cost estimate is less than $200,000 which is below the budget.
The final design must receive approval from Jim Hyslip and Bill Turner	The final design was discussed and approved by both liaisons.

7.0 Future Implementation

As construction of the project is intended to begin in the summer of 2013, there are measures that must be taken to ensure the process is implemented properly. The work involved with this project requires disrupting natural surroundings and precautions must be taken to reduce the impact construction will have on the environment. Therefore, erosion and sedimentation control will be implemented before any construction begins. After the controls are in place, construction may begin following a detailed staging plan and drawings.

7.1 Erosion & Sedimentation Control

During construction, the natural vegetation surrounding the current culvert will be removed in order to make room for the new bridge as well as allow the stream channel to be modified. Since much of the land will be disturbed, the soil can be displaced by erosion into

Wright Brook and sedimentation can occur. Measures will need to be taken in order to prevent sedimentation (66).

Silt fencing will need to be placed along the banks of the inlet and outlet where construction will occur. A silt fence is a temporary sediment barrier consisting of filter fabric attached to supporting posts (67). The silt fences must be 2’-3’ high above ground and the sediment must be closely monitored especially after rain. The sediment caught by the fence must be removed before it reaches 1/3 the height of the fence (68). It is estimated that about 800 ft of silt fence will be needed along the banks of the inlet and outlet. A silt curtain will also be needed downstream to trap any sedimentation that is in the water. Silt curtains are designed to control the dispersion of particulate matter in the body of water. It is estimated that about 80 ft of silt curtain will be needed to span across the downstream of Wright Brook beyond the area that needs to be re-graded.

7.2 Construction Staging

The project liaisons intend to begin construction in the summer of 2013. In order to have an idea of how long this process will take, the team has created a construction staging plan (Table 12) with the input of Bill Turner, who will be overseeing the actual construction. This staging plan covers the steps that will be taken during construction while the more detailed and final procedure will be decided by the future contractor. Figures demonstrating each construction phase can be found in Appendix H.

Table 12: Construction Staging Plan

Performance Time Estimate	
Task	**Duration**
STAGE 1: Site Preparation	
Lay out construction site boundaries, silt fences, silt curtain and set up detour.	
Strip away current road surface needing replacement.	
Dig out soil to top of current culvert (482’).	

Table 12: Construction Staging Plan (continued)

STAGE 2: Right Footing & Gabion Wall
Place sandbags Right of Bank (ROB) to contain stream (still flowing into culvert).
Remove current scour protection rocks at both inlet & outlet ROB.
Use a maximum 34° slope (Soil Type C) starting at the top of the culvert down to bedrock (473'), to excavate footing location.
If bedrock cannot be reached, excavate to top of rock and pour lean concrete.
Break current headwall to remove Right half of headwalls at inlet & outlet.
Pour concrete for Right Footing.
Fill above Right Footing until 476'.
From road level, place gabion baskets (embed from 476') for up and downstream ROB wing walls.
STAGE 3: Regrading Streambed
Dig down to 477' (right side of culvert) to regrade streambed to have gradual slope.
Install temporary stream diversion pipe that extends downstream 80' from culvert entrance into the regraded right side. Place sandbags to direct flow as necessary.
Place sandbags to direct stream away and into the temporary pipe (no flow in culvert).
Pump out the still pond water downstream of culvert.
Remove current scour protection rocks at outlet Left of Bank (LOB).
Fill in the downstream ponded area to continue the gradual slope from culvert exit (slope = 0.0123). At 45' downstream, elevation should be 476'.

Table 12: Construction Staging Plan (continued)

STAGE 4: Left Footing & Gabion Wall
Pump out water at LOB that may be left from the stream training.
Remove current scour protection rocks at inlet LOB.
Use a maximum 34° slope (Soil Type C) starting at the top of the culvert down to bedrock (473'), to excavate footing location.
If bedrock cannot be reached, excavate to top of rock and pour lean concrete.
Remove Left half of headwalls at inlet & outlet.
Pour concrete for Left Footing.
Fill above Left Footing until 476'.
Place a temporary path from the road down to upstream LOB for excavators.
Using the path, place gabion baskets (embed from 476') for LOB wing wall, working way back up the path to the road.
From road level, place gabion baskets (embed from 476') for downstream LOB wing walls.
STAGE 5: Regrading & Installation
Remove existing culvert.
Dig down to 477' (left side of bridge) to regrade streambed to have gradual slope.
Remove sandbags and temporary stream diversion pipe to allow free flow of stream.
Add flowable fill to inner sides of bridge while installing each of the 5 segments.
Fill in soil cover above and around bridge.
STAGE 6: Road Construction
Fill in layers of road material following the new alignment.
Pave the asphalt.
Install guardrails and traffic signs (speed limit, curved road).
Total Time ~ 1 month

After construction is complete, the bridge will need to be inspected by the Federal Highway Administration under the National Bridge Inspection Standards (NBIS). Since the new structure will be placed on a new alignment, NBIS inspection is to be completed within 90 days of being open to public travel. The state's inventory must be updated with the new Structure Inventory and Appraisal (SI&A) sheet within the 90 days. The structure should be re-inspected every 24 months (69). Any necessary maintenance must be carried out by the town to meet inspection standards.

7.3 Construction Plans

The construction plans for this project highlight the key elements of the final design as well as provide an overview of its integration to the project site. The plans were drawn to aid in the initial permitting process as a supporting document for the Notice of Intent. These drawings can be found in Appendix I.

8.0 Summary

On behalf of the Town of Williamsburg and in partnership with HyGround Engineering, LLC, Team HyGround designed a replacement structure for an undersized and deteriorating culvert on Depot Road. As the road tends to flood during storms, the main concern for the new structure was to follow stream crossing standards which include the ability to handle a 50-year 24-hour storm flow rate of 1900 cfs as well as keep the stream velocity below 10 ft/s. The overall project was to stay within the town's budget of $250,000.

The first aspect of the design was to realign the segment of Depot Road above the culvert. In order to do this, the common practice of pavement design as well as the speed limit in Williamsburg set the requirements for the new road. The next focus was the design of a replacement structure for the undersized culvert. Based on research, a pre-cast concrete open bottom box culvert was chosen as the best option. To determine the opening of the culvert, a hydrologic analysis was performed to find the flow rates of the stream during different design storms. The results from this analysis were used to model various sized culverts in a hydraulic modeling program. The 35 ft span for the culvert was chosen based on the standard listed in the *Massachusetts Stream Crossing Handbook* as well as the results from the hydraulic modeling. Because the span is over 20 ft, the replacement structure was then classified as a bridge. Once sized, the bridge was positioned at the site in order to minimize the need for stream training. The final orientation led to the length of the bridge (in the direction of flow) to be 35 ft. The bridge dimensions were then used to design the strip footing foundations and the gabion walls for stream training and soil support. All these individual design components were then compiled to develop construction drawings, staging plans, and full cost estimate.

The final proposed design for the replacement structure is a 3-sided precast concrete bridge with a 7 ft rise, 35 ft span, and 35 ft length. However, as the bridge will be embedded for stability purposes, the actual opening of the bridge will be 4'x35'. The bridge will require strip footings as well as stepped back face gabion walls to train the stream. The new road design will realign the current road to have a safer curve for a 35 mph design speed and 4% superelevation. According to these design specifications, the expected cost of the project is under $200,000. To enable construction in the summer of 2013, the team has submitted the final design along with construction plans, final cost estimates, and relevant permit documents.

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Appendix A: Road Design Calculations

The following sections explain the process by which we calculated the minimum radius, design superelevation road curve radius, and the transition length of road required for the selected superelevation. We ultimately selected a design superelevation of 4% due to the reduction in overall road length requiring construction.

Calculating minimum radius required:

$$R_{min} = \frac{V^2}{15(0.01e_{max} + f_{max})}$$

Where

V = design speed

e_{max} = maximum superelevation

f_{max} = maximum side friction factor

We chose a design speed of 35mph to match the speed limit of the road. To get maximum values for superelevation in Massachusetts is 6% (9). The maximum side friction factor was obtained from Exhibit 3-15 from AASHTO Geometric Design of Highways and Streets (70). This gave the following values.

e_{max} = 6.0 for MA

f_{max} = 0.18

Thus,

$$R_{min} = 340\ ft$$

We could then use any radius larger than this that would fit our site and design speed and consequently get a design superelevation. Using Exhibit 3-25 (70), we found the radius that would fit the existing tangents of the road was 371' for a superelevation of 4%.

R=371' for e = 4%

Next we obtained the required superelevation transition length using the superelevation runoff value correlating to each superelevation option. The overall goal of determining each transition was to model each road option in AutoCAD and decide which seemed the most practical.

The superelevation runoff for each option was obtained from Exhibit 3-32 (70). The following equation was used to obtain the transitions.

Calculating Superelevation Transition Required:

The superelevation runoff for each option was obtained from Exhibit 3-32 (70). The following equation was used to obtain the transitions.

$$TR = \frac{(S_{normal})(L_r)}{e}$$

Where

TR = Tangent Runout

S_{normal} = superelevation on the normal crown

L_r = superelevation runoff

e = design superelevation

Total transition Length = Tangent runout + superelevation runoff

The values for each option are shown in Table A1. The total length of road was obtained in AutoCAD, as shown in Figure A1.

Note: 2/3 superelevation runoff is on the tangent section and 1/3 superelevation runoff is on the arc.

Table A1. Calculated values for new road

Road Design	
Superelevation	4%
Design Speed	35mph
Radius	371ft
Superelevation Runoff	84ft
Tangent Runout	42ft
Total Length of Road*	355ft

*The total length of road was obtained from AutoCAD

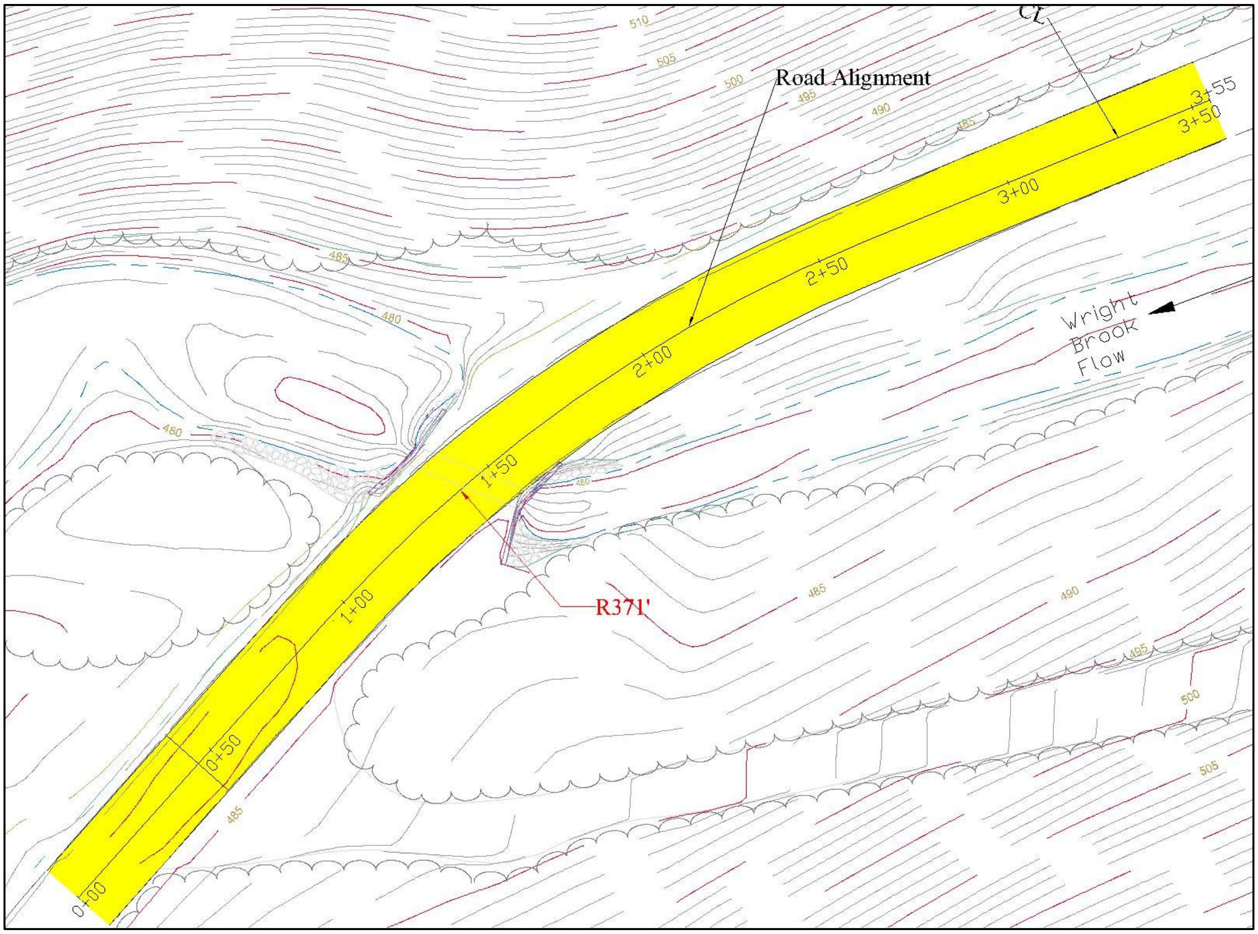


Figure A1. Road design for a superelevation of 4% and design speed of 35mph

Note: 2/3 superelevation runoff is on the tangent section and 1/3 superelevation runoff is on the arc.

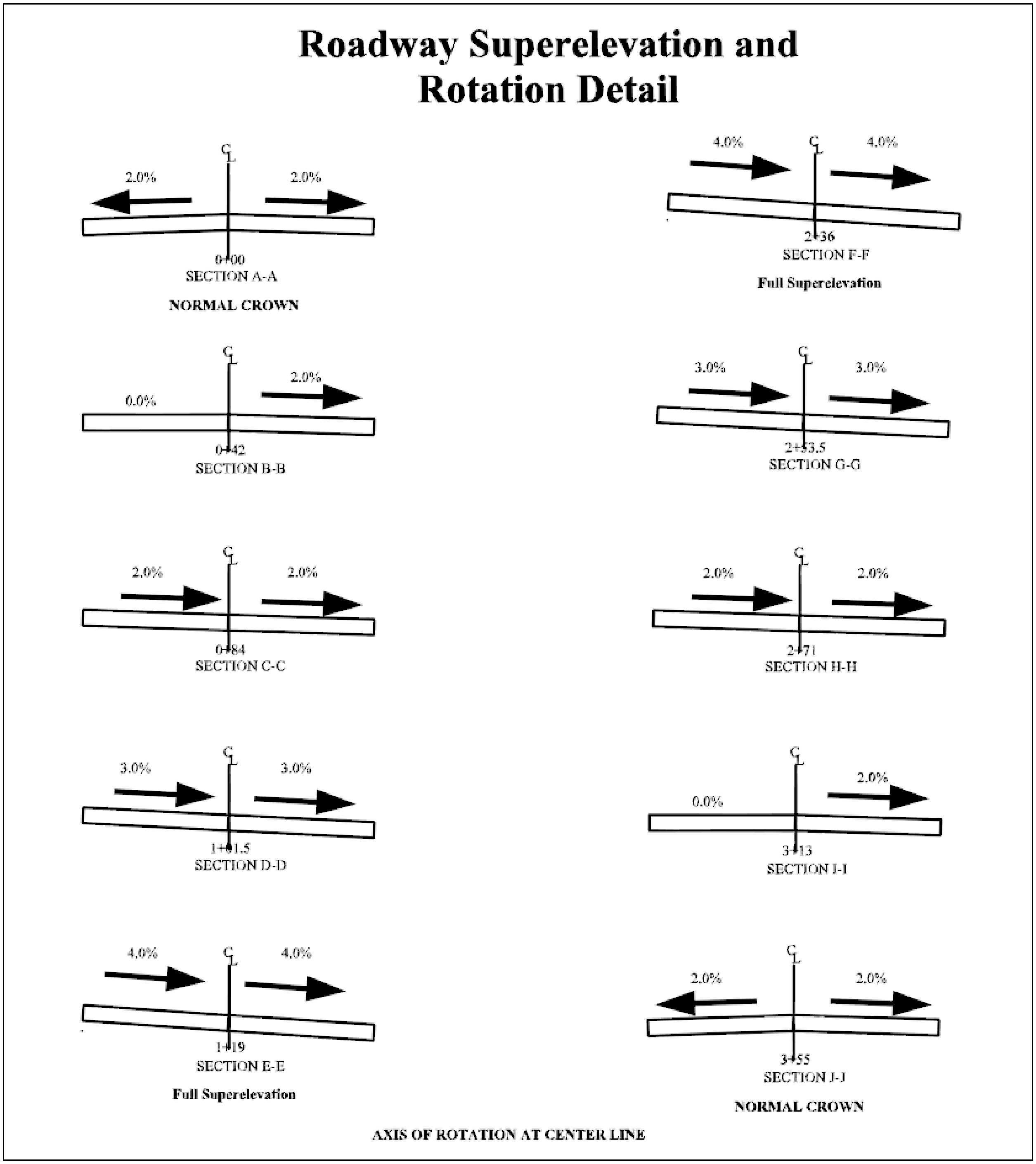


Figure A2. Superelevation transition plan

Appendix B: Watershed Soil Group Classification and Maps

The soils in the watershed were grouped by sub-areas and the far right column indicates which hydrologic soil group each soil type belongs to. The hydrologic soil groups and the area each soil group consists of (in a sub-area) are used in TR-55 to find the runoff curve number from rainfall.

Table B.1. Classification of Watershed Soils into their Respective Hydrologic Soil Group (71)

Sub-area 1				
Map Unit Symbol	**Map Unit Name**	**Area (sq.ft)**	**Area (ac)**	**Group**
12A	Maybid silt loam, 0 to 3 percent slopes	1841971.9	42.3	D
52A	Freetown muck, 0 to 1 percent slopes	167926.1	3.9	D
71B	Ridgebury gravelly fine sandy loam, 0 to 5 percent slopes, extremely stony	60485.0	1.4	D
109B	Chatfied-Hollis complex, 3 to 8 percent slopes, rocky	58028.1	1.3	B
109C	Chatfied-Hollis complex, 8 to 15 percent slopes, rocky	1556482.2	35.7	B
253E	Hinckley loamy sand, 25 to 35 percent slopes	1810532.3	41.6	A
254C	Merrimac fine sandy loam, 8 to 15 percent slopes	2223921.4	51.1	A
306B	Paxton fine sandy loam, 3 to 8 percent slopes, very stony	158869.5	3.6	C
306C	Paxton fine sandy loam, 8 to 15 percent slopes, very stony	1637474.4	37.6	C
711C	Charlton-Rock outcrop-Hollis complex, sloping	1532299.0	35.2	D
711E	Charlton-Rock outcrop-Hollis complex, steep	8657278.8	198.7	D
Total		19705268.6	452.4	

Sub-area 2				
Map Unit Symbol	**Map Unit Name**	**Area (sq.ft)**	**Area (ac)**	**Group**
254C	Merrimac fine sandy loam, 8 to 15 percent slopes	1339683.6	30.8	A
305C	Paxton fine sandy loam, 8 to 15 percent slopes	335815.0	7.7	C
306C	Paxton fine sandy loam, 8 to 15 percent slopes, very stony	1176822.0	27.0	C
306D	Paxton fine sandy loam, 15 to 25 percent slopes, very stony	198631.6	4.6	C
711E	Charlton-Rock outcrop-Hollis complex, steep	363582.6	8.3	D
Total		3414534.7	78.4	

Sub-area 3				
Map Unit Symbol	Map Unit Name	Area (sq.ft)	Area (ac)	Group
30A	Raynham silt loam, 0 to 3 percent slopes	744909.5	17.1	C
88B	Ridgebury fine sandy loam, 3 to 8 percent slopes, very stony	1319022.6	30.3	C
103C	Charlton-Hollis-Rock outcrop complex, 3 to 15 percent slopes	1388784.4	31.9	D
225A	Belgrade silt loam, 0 to 3 percent slopes	165142.4	3.8	B
225B	Belgrade silt loam, 3 to 8 percent slopes	763567.6	17.5	B
254C	Merrimac fine sandy loam, 8 to 15 percent slopes	914167.0	21.0	A
254D	Merrimac fine sandy loam, 15 to 25 percent slopes	197710.6	4.5	A
305B	Paxton fine sandy loam, 3 to 8 percent slopes	619859.5	14.2	C
305C	Paxton fine sandy loam, 8 to 15 percent slopes	734506.9	16.9	C
306C	Paxton fine sandy loam, 8 to 15 percent slopes	756910.3	17.4	C
306D	Paxton fine sandy loam, 15 to 25 percent slopes, very stony	428826.8	9.8	C
311C	Woodbridge fine sandy loam, 8 to 15 percent slopes, very stony	1903444.1	43.7	C
537C	Paxton fine sandy loam, 8 to 15 percent slopes, stony	455551.1	10.5	C
537D	Paxton fine sandy loam, 15 to 25 percent slopes, stony	77720.5	1.8	C
711C	Charlton-Rock outcrop-Hollis complex, sloping	1488409.1	34.2	D
711E	Charlton-Rock outcrop-Hollis complex, steep	3320369.9	76.2	D
Total		15278902.2	350.8	

Sub-area 4				
Map Unit Symbol	Map Unit Name	Area (sq.ft)	Area (ac)	Group
103C	Charlton-Hollis-Rock outcrop complex, 3 to 15 percent slopes	225332.6	5.2	D
103D	Charlton-Hollis-Rock outcrop complex, 15 to 25 percent slopes	653597.0	15.0	D
254B	Merrimac fine sandy loam, 3 to 8 percent slopes	747000.7	17.1	A
254C	Merrimac fine sandy loam, 8 to 15 percent slopes	376899.0	8.7	A
254D	Merrimac fine sandy loam, 15 to 25 percent slopes	56353.4	1.3	A
260B	Sudbury fine sandy loam, 3 to 8 percent slopes	408257.8	9.4	B
711E	Charlton-Rock outcrop-Hollis complex, steep	251908.9	5.8	D
Total		2719349.5	62.4	

Sub-area 5				
Map Unit Symbol	**Map Unit Name**	**Area (sq.ft)**	**Area (ac)**	**Group**
103C	Charlton-Hollis-Rock outcrop complex, 3 to 15 percent slopes	153710.3	3.5	D
103D	Charlton-Hollis-Rock outcrop complex, 15 to 25 percent slopes	167176.3	3.8	D
109C	Chatfied-Hollis complex, 8 to 15 percent slopes, rocky	829850.1	19.1	B
109F	Chatfied-Hollis complex, 25 to 60 percent slopes, rocky	3050222.2	70.0	B
253E	Hinckley loamy sand, 25 to 35 percent slopes	21735.0	0.5	A
254B	Merrimac fine sandy loam, 3 to 8 percent slopes	1425393.8	32.7	A
260B	Sudbury fine sandy loam, 3 to 8 percent slopes	295181.7	6.8	B
305B	Paxton fine sandy loam, 3 to 8 percent slopes	336644.6	7.7	C
305C	Paxton fine sandy loam, 8 to 15 percent slopes	333828.3	7.7	C
305D	Paxton fine sandy loam, 15 to 25 percent slopes	70095.5	1.6	C
306B	Paxton fine sandy loam, 3 to 8 percent slopes, very stony	649576.8	14.9	C
306C	Paxton fine sandy loam, 8 to 15 percent slopes	1872733.1	43.0	C
311C	Woodbridge fine sandy loam, 8 to 15 percent slopes, very stony	16871.4	0.4	C
711C	Charlton-Rock outcrop-Hollis complex, sloping	365570.1	8.4	D
Total		9588589.2	220.1	

Sub-area 6				
Map Unit Symbol	**Map Unit Name**	**Area (sq.ft)**	**Area (ac)**	**Group**
225A	Belgrade silt loam, 0 to 3 percent slopes	117238.0	2.7	B
225B	Belgrade silt loam, 3 to 8 percent slopes	305425.0	7.0	B
254C	Merrimac fine sandy loam, 8 to 15 percent slopes	79304.0	1.8	A
305B	Paxton fine sandy loam, 3 to 8 percent slopes	238515.9	5.5	C
305C	Paxton fine sandy loam, 8 to 15 percent slopes	483314.9	11.1	C
306B	Paxton fine sandy loam, 3 to 8 percent slopes, very stony	598893.0	13.7	C
306C	Paxton fine sandy loam, 8 to 15 percent slopes	47911.4	1.1	C
310C	Woodbridge fine sandy loam, 8 to 15 percent slopes	65228.9	1.5	C
311C	Woodbridge fine sandy loam, 8 to 15 percent slopes, very stony	623857.0	14.3	C
311D	Woodbridge fine sandy loam, 15 to 25 percent slopes, very stony	251977.8	5.8	C
537B	Paxton fine sandy loam, 3 to 8 percent slopes, stony	282289.3	6.5	C
537C	Paxton fine sandy loam, 8 to 15 percent slopes, stony	733583.9	16.8	C
711C	Charlton-Rock outcrop-Hollis complex, sloping	141262.2	3.2	D
Total		3968801.3	91.1	

Sub-area 7				
Map Unit Symbol	**Map Unit Name**	**Area (sq.ft)**	**Area (ac)**	**Group**
73A	Whitman fine sandy loam, 0 to 3 percent slopes, extremely stony	1353999.8	31.1	D
88A	Ridgebury fine sandy loam, 0 to 3 percent slopes, very stony	122358.0	2.8	C
88B	Ridgebury fine sandy loam, 3 to 8 percent slopes, very stony	2179766.7	50.0	C
109C	Chatfied-Hollis complex, 8 to 15 percent slopes, rocky	89343.6	2.1	B
254B	Merrimac fine sandy loam, 3 to 8 percent slopes	83165.0	1.9	A
305B	Paxton fine sandy loam, 3 to 8 percent slopes	686152.0	15.8	C
305C	Paxton fine sandy loam, 8 to 15 percent slopes	44713.6	1.0	C
305D	Paxton fine sandy loam, 15 to 25 percent slopes	131754.3	3.0	C
306B	Paxton fine sandy loam, 3 to 8 percent slopes, very stony	856972.5	19.7	C
306C	Paxton fine sandy loam, 8 to 15 percent slopes	6912218.7	158.7	C
306D	Paxton fine sandy loam, 15 to 25 percent slopes, very stony	1165274.4	26.8	C
310B	Woodbridge fine sandy loam, 3 to 8 percent slopes	918462.1	21.1	C
311C	Woodbridge fine sandy loam, 8 to 15 percent slopes, very stony	540926.7	12.4	C
537C	Paxton fine sandy loam, 8 to 15 percent slopes, stony	1661137.9	38.1	C
711C	Charlton-Rock outcrop-Hollis complex, sloping	1315511.7	30.2	D
711E	Charlton-Rock outcrop-Hollis complex, steep	545000.8	12.5	D
Total		18606757.8	427.2	

Figure B1. Soil map showing all the different soil types in the watershed (47)

Appendix C: Time of Concentration Worksheet

The time of concentration worksheet details the equations and variables needed to find the travel time for sheet flow, shallow concentrated flow, and channel flow. The time of concentration is the sum of the travel times for each flow segment. The time of concentration found for sub-area 1 is 1.3 hrs as seen in Table C-1.

Table C-1. Time of Concentration Worksheet

Sheet Flow	
Surface description	
Manning's roughness coefficient,n	0.24
Flow length, L	100
Two-year 24-hour rainfall, P_2	3
Land slope, s	0.02
T_t = (0.007*(n*L)^0.08)/($P_2^{0.5}$*$s^{0.4}$)	0.25
Shallow Concentrated Flow	
Surface description	
Flow length, L	4369
Watercourse slope, s	0.02
Average Velocity, V (from Fig. C-1)	2.32
Tt = L/(3600*V)	0.523108
Channel Flow	
Cross sectional flow area,a	30
Wetted perimeter,pw	32
Hydraulic radius, r=a/pw	0.9375
Channel slope, s	0.02
Manning's roughness coefficient, n	0.05
V = (1.49*r^(2/3)*s^(1/2))/n	4.036876
Flow length, L	7005
Tt = L/3600V	0.482015
Tc (hr)	1.25

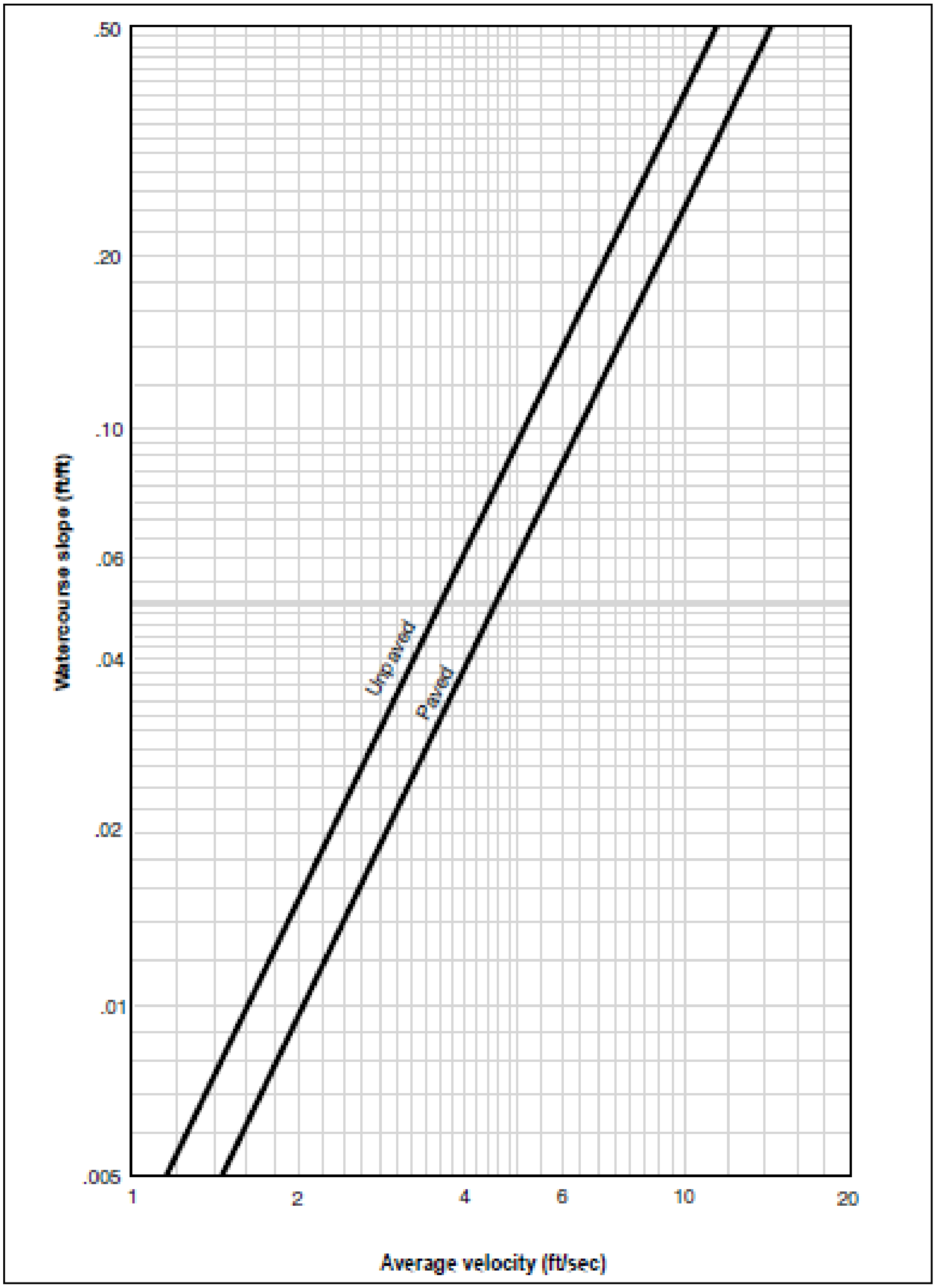


Figure C-1. Average velocities for estimating travel time for shallow concentrated flow (72)

Appendix D: Contour Map of Project Site

The maps below show the location of the project. The bottom right map of Massachusetts indicates Williamsburg with a red star. The map on the left orients Williamsburg (Depot Road designated with an "A") with the surrounding towns of Northampton, Amherst, and Holyoke. The top right map locates the project site with Depot Road and Route 9.

In order to perform a hydrologic analysis, the cross sections of the stream are required. However, due to inclement weather, we were unable to physically measure the cross sections. Therefore, we had to rely on the contour map (next page) surveyed by HyGround, for this information.

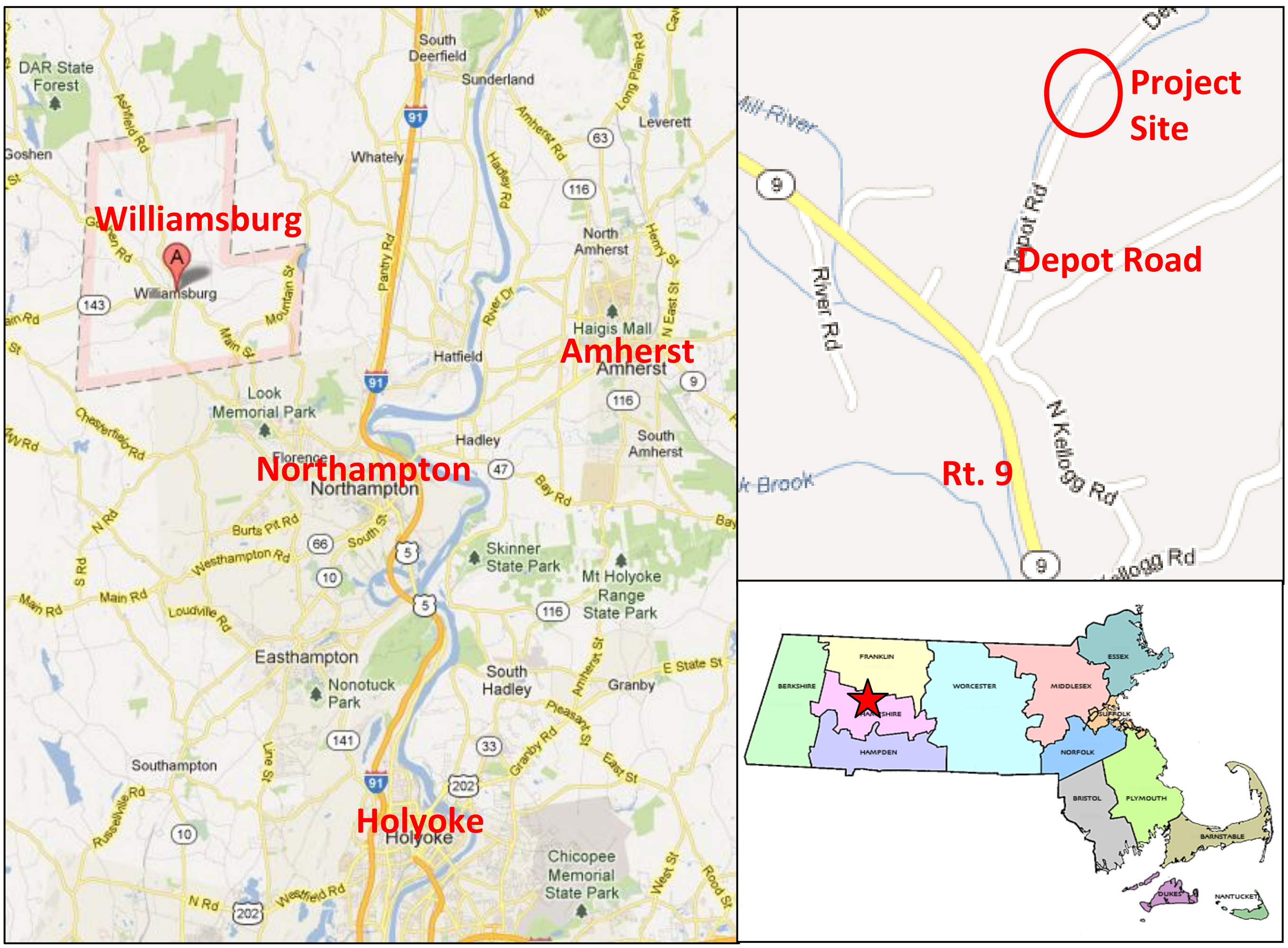


Figure D1. Maps showing location of Williamsburg and the project site

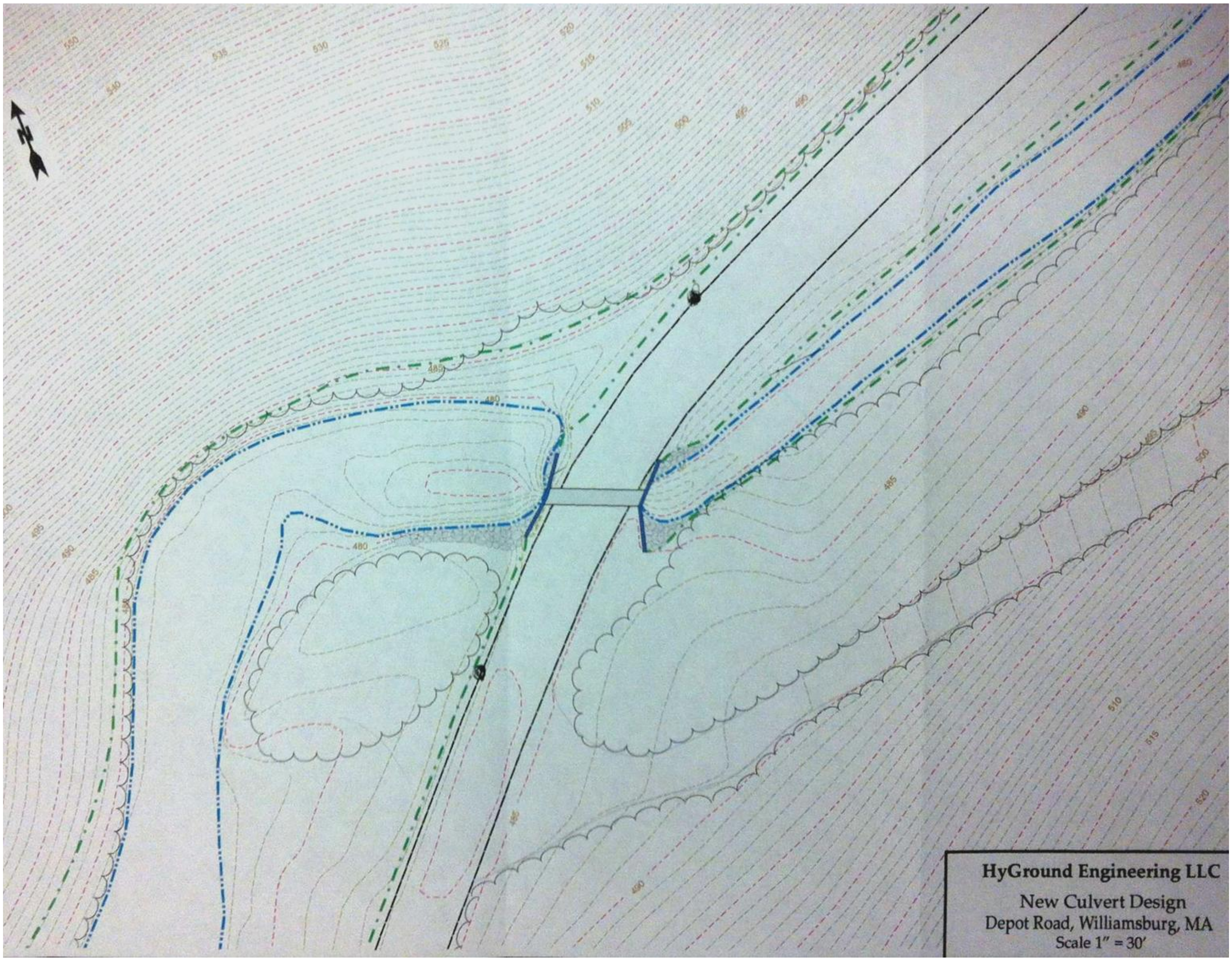
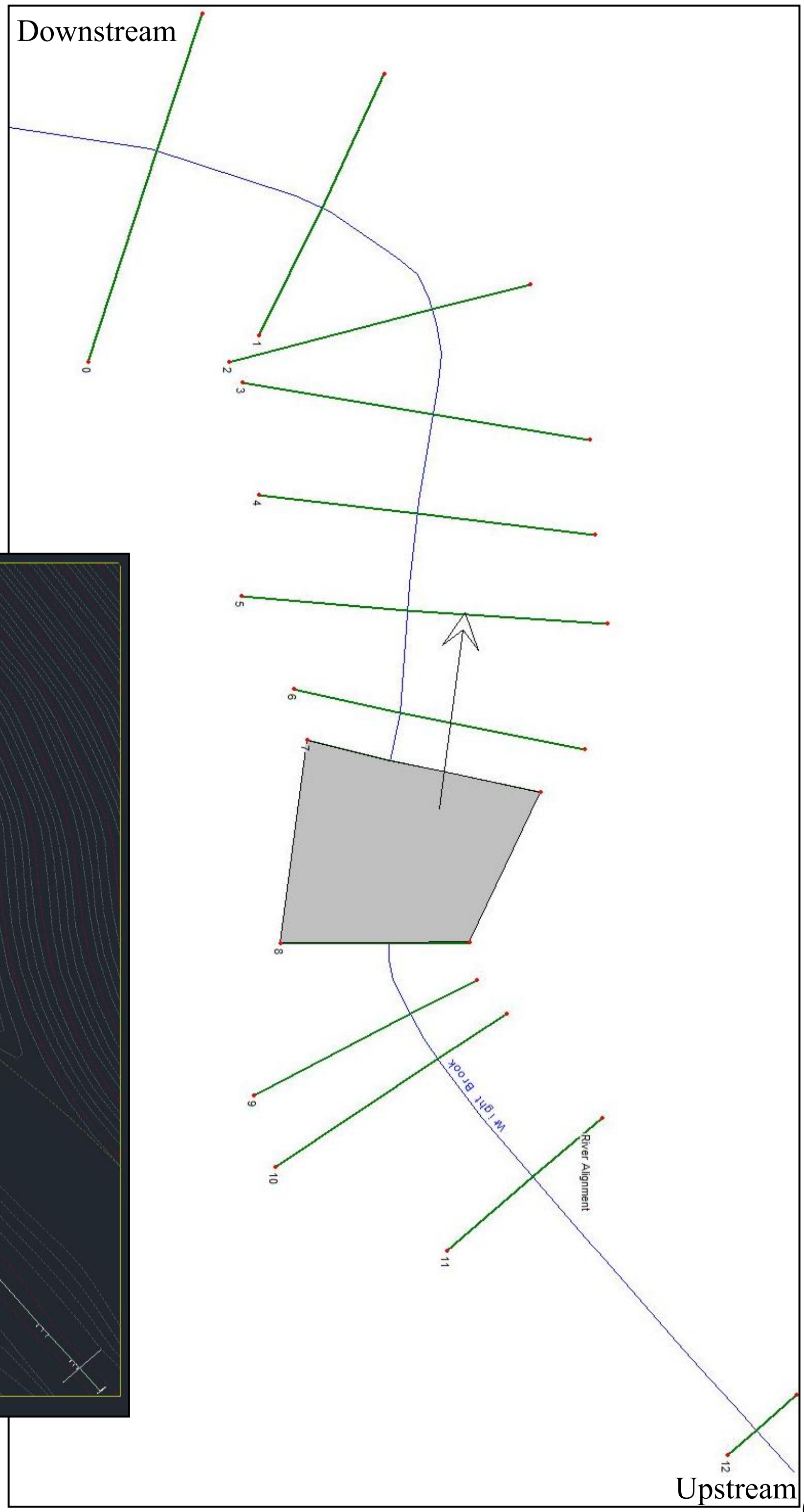


Figure D2. Contour map of project site surveyed by HyGround

Appendix E: Site Geometry and Cross Sections (HEC-RAS)

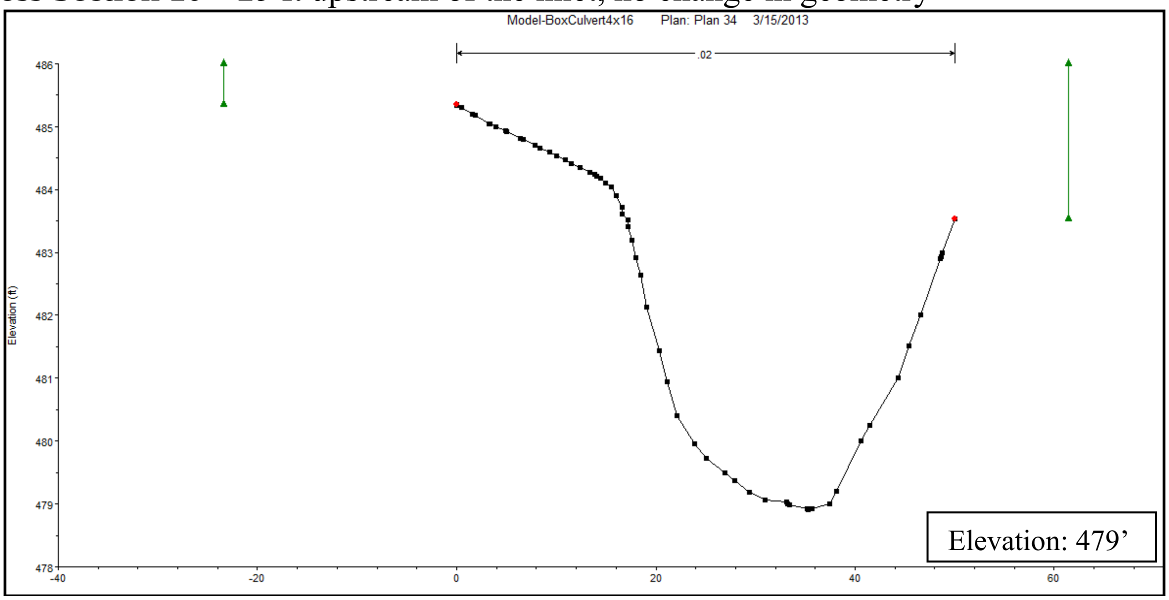
The black image is a Civil 3D version of the site contour map. After drawing the cross sections and stream geometry on Civil 3D, we exported the map into HEC-RAS (shown in white) onto HEC-RAS, which can then extract necessary stream geometry information.



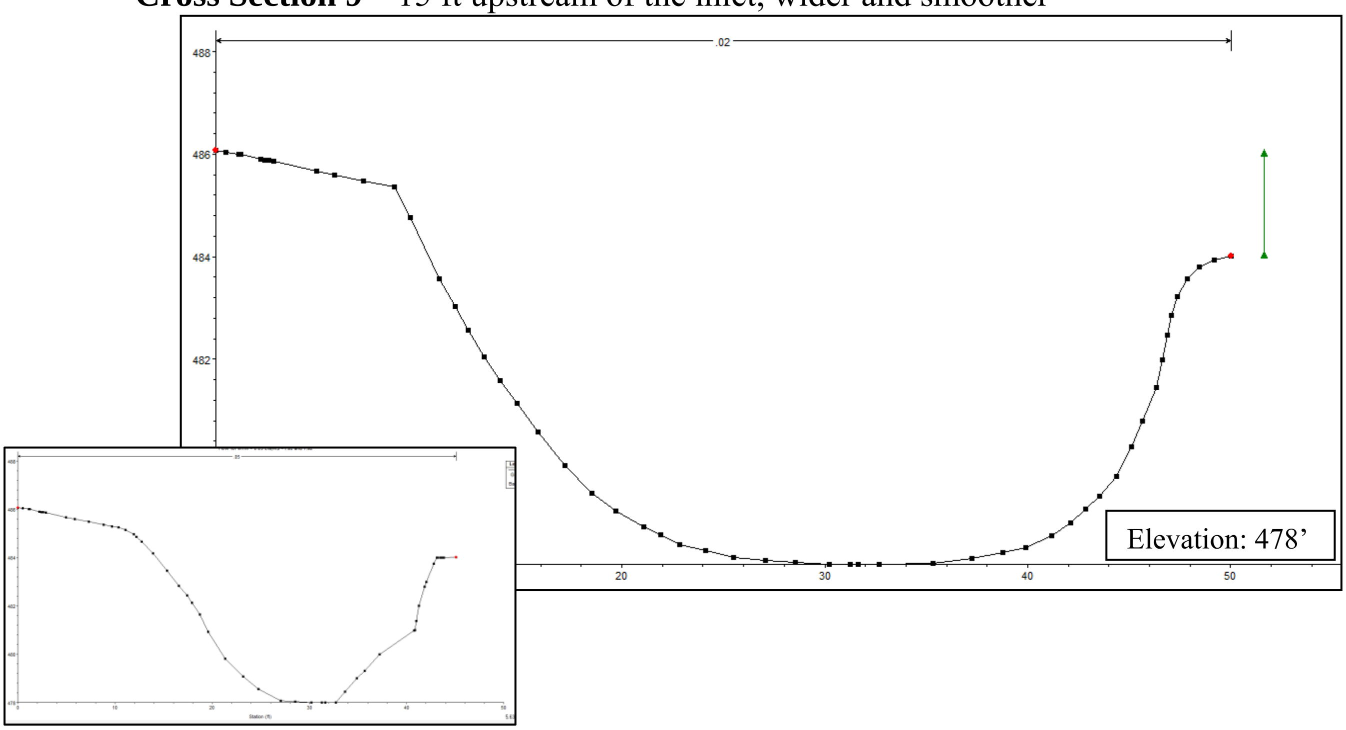
Appendix F: Cross Sections 10-8 (Upstream) and 7-5 (Downstream)

In order to remove the sudden drop in the streambed at the outlet of the current culvert, the stream will be regraded to continue the gradual slope from upstream. When regrading the slope during construction, the channel cross sections will be tapered to the size of the bridge opening. These changes were modeled in HEC-RAS by altering the shape of the cross sections as shown by the figures below. The larger images represent the new cross sections and the insets, the original.

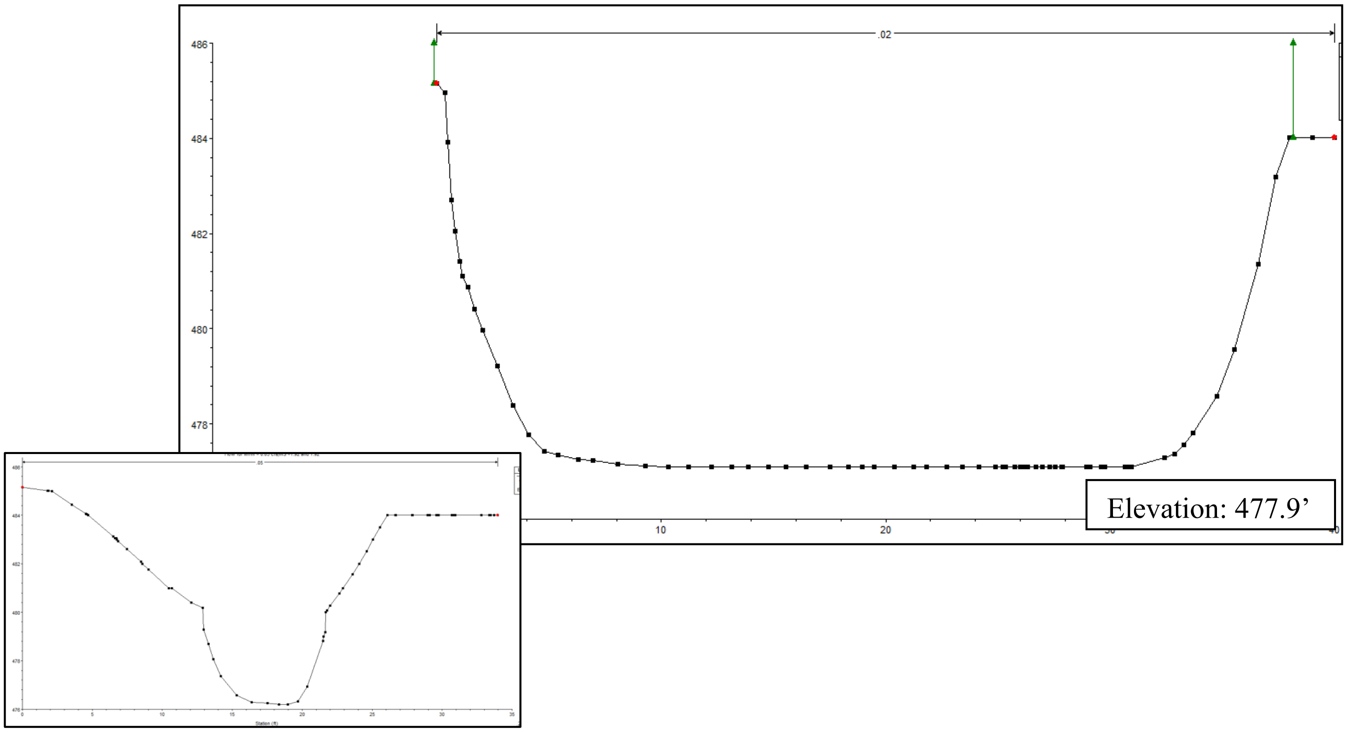
Cross Section 10 – 25 ft upstream of the inlet, no change in geometry



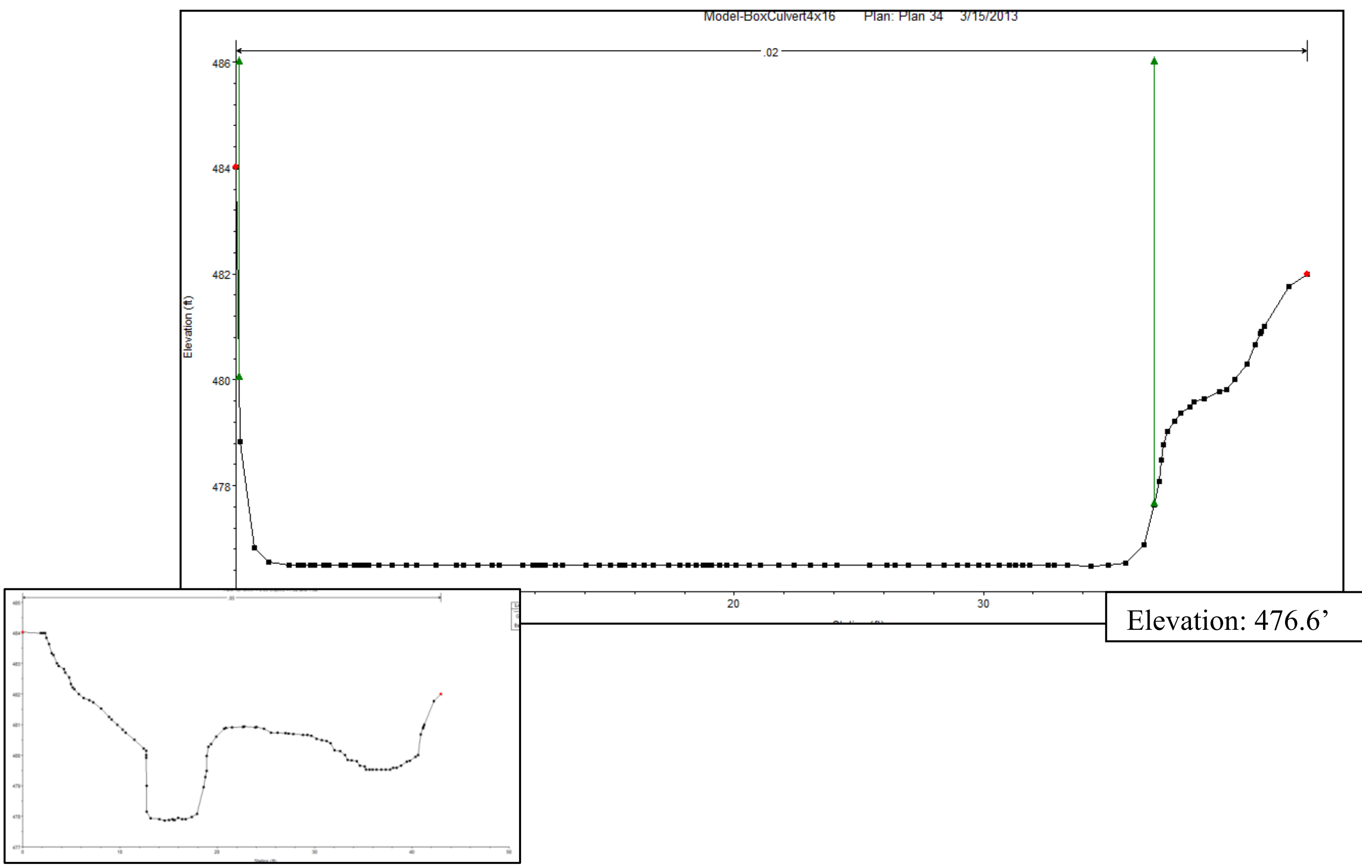
Cross Section 9 – 15 ft upstream of the inlet, wider and smoother



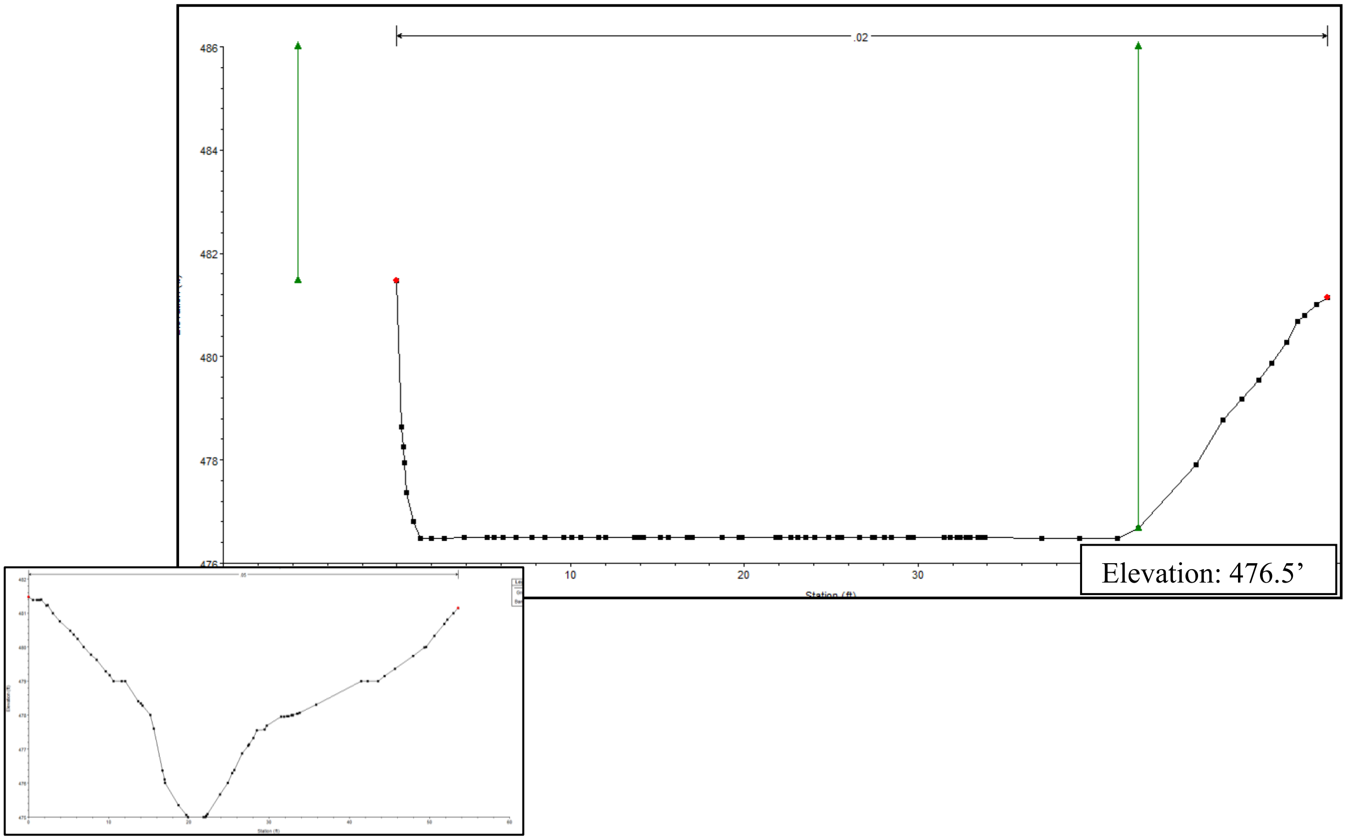
Cross Section 8 - 2 ft upstream of the inlet, wider and smoother



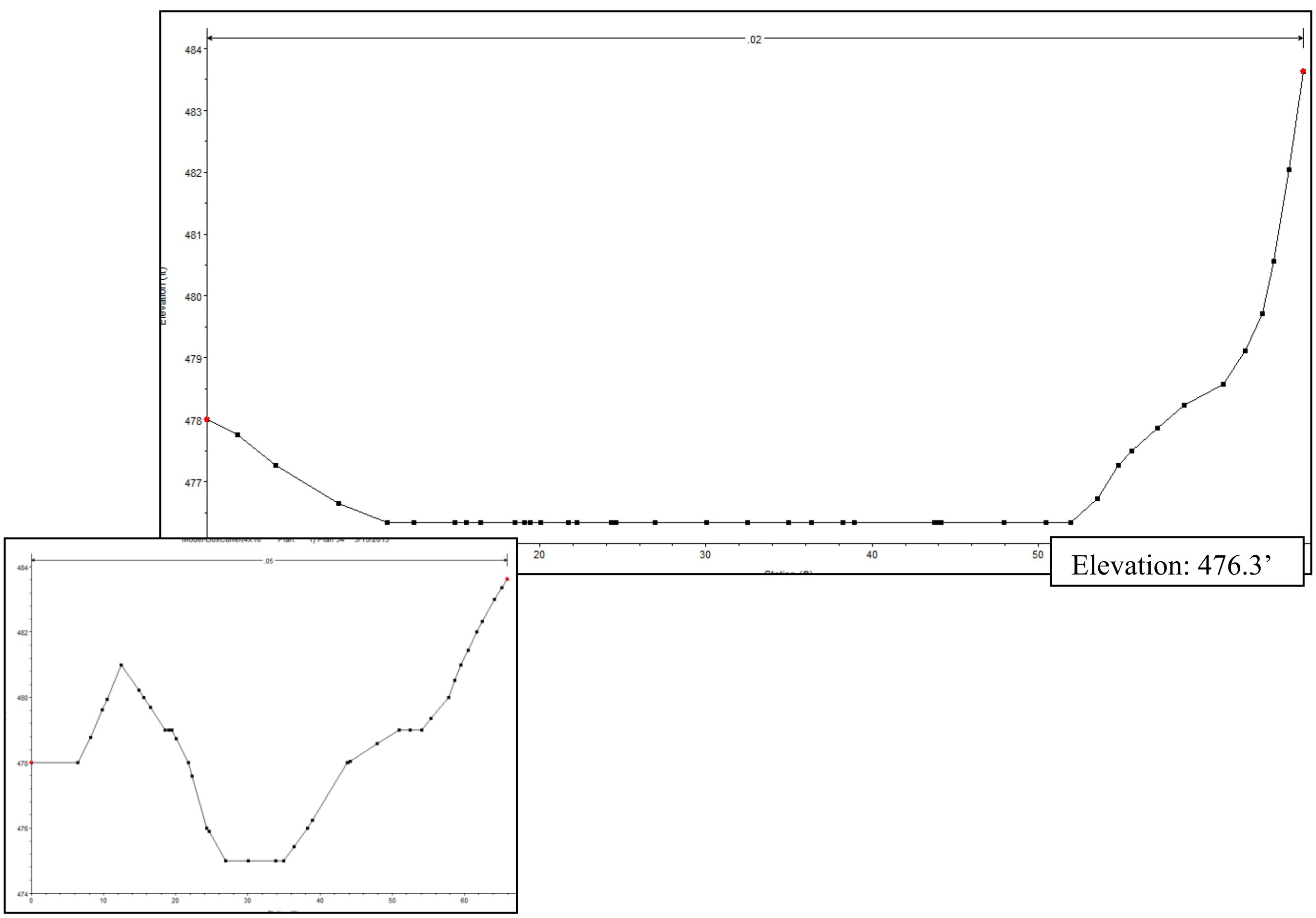
Cross Section 7 - 1 ft downstream of the outlet, wider and smoother



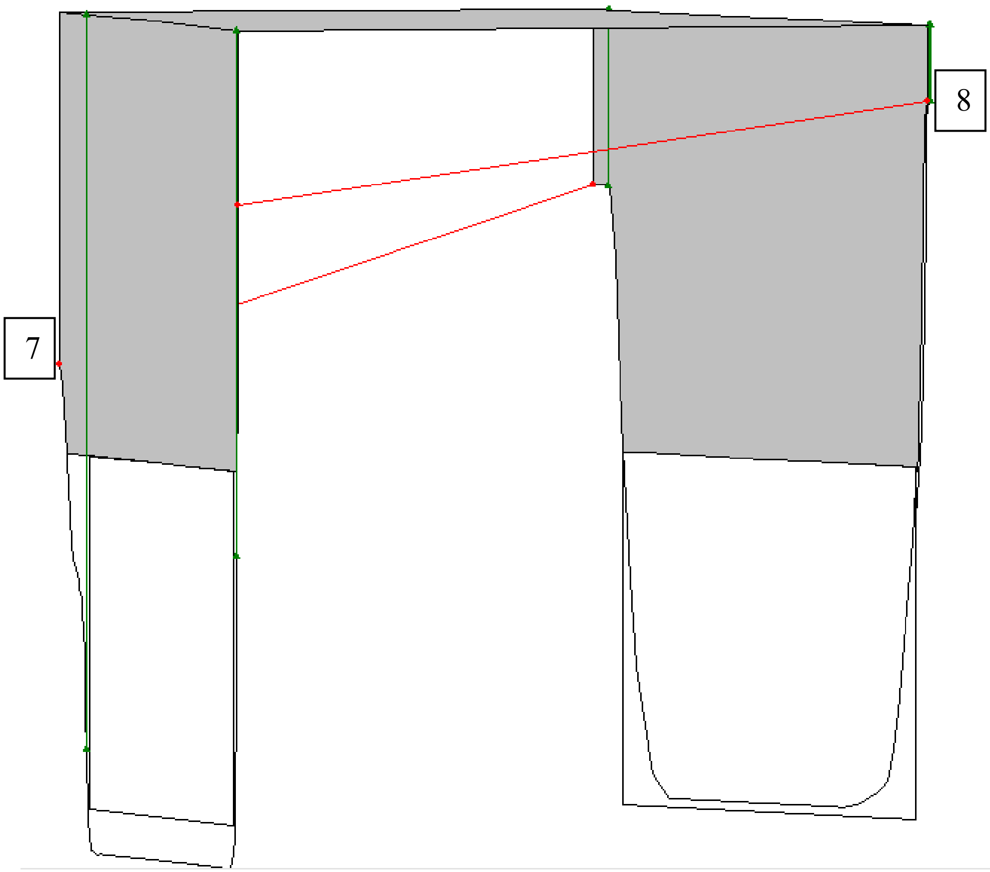
Cross Section 6 - 10 ft downstream of the outlet, wider and smoother



Cross Section 5 - 29 ft downstream of the outlet, wider and smoother



View of Cross Section 7, Culvert, and Cross Section 8



Appendix G: Foundation Calculations

The following sections explain the process by which we designed strip footing dimensions, footing reinforcement, and retaining walls required to support both our structure and the surrounding area. We ultimately selected a strip footing design of 1 foot in thickness, 3 feet in width, and 36 feet in length. No. 5 bars with 9 inches off center were chosen for flexure reinforcement and 3 No. 5 bars were chosen for perpendicular temperature reinforcement. The type of retaining walls chosen for our site were gabion walls in 3’x3’x6’blocks.

Section G1: Standard Penetration Test Results

Table G1 details the results from the Standard Penetration Test performed at the project site. Standard Penetration Testing was performed and blow count values were recorded at 3 consecutive 6 inch intervals and placed into boring logs. The second and third value for each grouping of intervals was added to determine the blowcount, N. It is common practice to discount the first value due to potential disturbance (Foundation source).

Table. G1.1 Standard Penetration Test Blowcount Data Collected from Project Site

Boring Number	Depth (ft.)	Blowcount, N (bpf)
1	1.5	12
	3	20
	4.5	16
	6.5	91
	8.5	45
	11.5	87
2	2	22
	3.5	16
	6.5	9
	8.5	23
3	1.5	27
	3.5	15
	6.5	7
	8.5	5
	11.5	38
	14.5	62
4	2	27
	3.5	27
	6.5	11
	8.5	45
	11.5	87
	13.5	107

The blowcount value was used to determine the variation in friction angle throughout the soil. To do this, each value had to be corrected for a variety of factors. Equation G1 below was used to provide this correction.

$$N_{1,60} = C_R C_S C_B C_N C_E N \qquad \text{[Eq. G1]}$$

Where:

$N_{1,60} = corrected\ blowcount$

$C_R = rode\ length\ below\ anvil = 0.15L + .61\ for\ 13ft < L < 20ft$

$C_S = standard\ sampler = 1.0$

$C_B = Borehole\ Diameter = 1.0\ because\ OD = 2"$

$C_N = \left(\frac{2000}{\sigma'_z}\right)^{\frac{1}{2}}\ if\ C_N \le 2$

$C_E = Hammer\ energy\ correction = 0.7 - 1.2\ because\ safety\ hammer\ was\ used$

Each of these correction factors could either be based off of the characteristics of the sampling equipment used or based off of best judgment from ranges provided. Table G1.2 lists the chosen values for the different correlation values. Each value is obtained from a listing of corrected factors for rod length, sampler type, and borehole size in the Foundation and Earth Retaining Structures textbook (57).

Table. G1.2 Standard Penetration Test Blowcount Correction Factors

Groundwater Depth	5	ft
Sampler Type, CS	1	for standard
Borehole Size, CB	1	for 2" OD
Equipment, CE	0.75	for safety hammer ranges 0.7-1.2
Rod Length from Surface	2.5	ft

Because there was an indication of some fine –grained soils at the site, we assumed a conservative unit weight value of 180 pcf for all of the soil present at the site. This value caused the correction for overburden pressure to lower each blowcount value. This indicated a lower strength of soil overall. The overburden pressure correction along with the corrected N values and correlated friction angles can be seen in Table G1.3 below. Figure G1.1 depicts the corrected blowcounts increasing with depth, showing that the soil strength increases with depth.

Table. G1.3 Standard Penetration Test Corrected Blowcount with Correlated Friction Angle

Boring Number	Depth	Unit Weight	Rod Length Correction	Vertical total stress	Porewater Pressure	Vertical Effective Stress	C_N	C_N	N	$N_{1,60}$	Friction Angle
	ft	pcf	C_R	psf	psf	psf	calc	use			Degrees
B1	1.5	130	1.21	195	0	195	3.203	1	12	11	33
	3	130	1.435	390	0	390	2.265	1	20	22	38
	4.5	130	1.66	585	0	585	1.849	1.85	16	37	44
	6.5	130	1.96	845	93.6	751.4	1.631	1.63	91	218	78
	8.5	130	2.26	1105	218.4	886.6	1.502	1.5	45	115	62
	11.5	130	2.71	1495	405.6	1089.4	1.355	1.35	87	240	81
B3	2	130	1.285	260	0	260	2.774	1	22	21	38
	3.5	130	1.51	455	0	455	2.097	1	16	18	37
	6.5	130	1.96	845	93.6	751.4	1.631	1.63	9	22	38
	8.5	130	2.26	1105	218.4	886.6	1.502	1.5	23	59	50
B3	1.5	130	1.21	195	0	195	3.203	1	27	25	39
	3.5	130	1.51	455	0	455	2.097	1	15	17	36
	6.5	130	1.96	845	93.6	751.4	1.631	1.63	7	17	36
	8.5	130	2.26	1105	218.4	886.6	1.502	1.5	5	13	34
	11.5	130	2.71	1495	405.6	1089.4	1.355	1.35	38	105	60
	14.5	130	3.16	1885	592.8	1292.2	1.244	1.24	62	183	73
B4	2	130	1.285	260	0	260	2.774	1	27	26	40
	3.5	130	1.51	455	0	455	2.097	1	27	31	42
	6.5	130	1.96	845	93.6	751.4	1.631	1.63	11	26	40
	8.5	130	2.26	1105	218.4	886.6	1.502	1.5	45	115	62
	11.5	130	2.71	1495	405.6	1089.4	1.355	1.35	87	240	81
	13.5	130	3.01	1755	530.4	1224.6	1.278	1.28	107	309	89

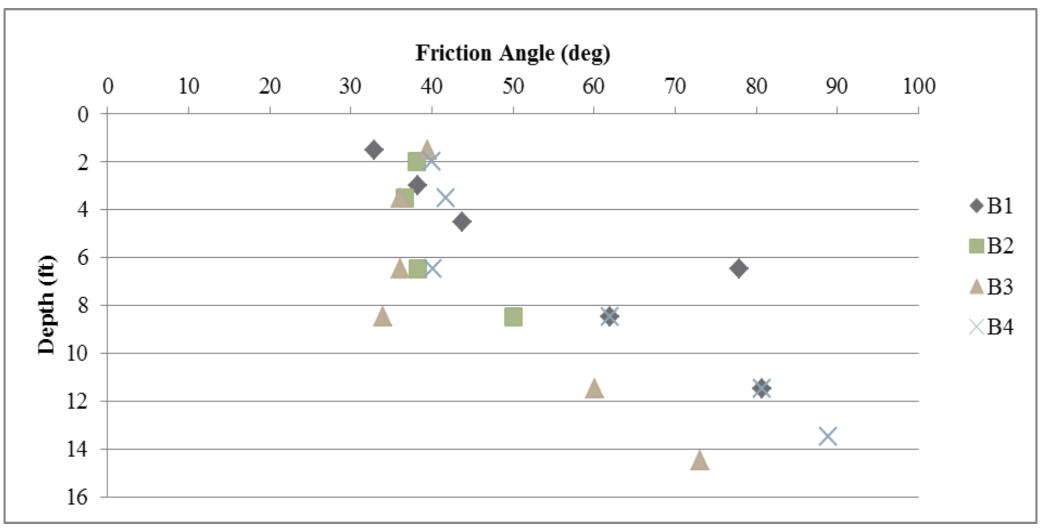


Figure G1.1 Plot of corrected blowcount quantity as a function of depth

Section G2: Laboratory Soil Classification Results

The purpose of the laboratory testing was to provide a classification of the soils on site. This classification could later be utilized to obtain a general idea of the associated soil properties related to strength. In determining the grain size distribution of the samples obtained from the site, we followed ASTM-D422. This required us to utilize a mechanical sieve shaker and a stack of sieves to separate oven dried soils into a grouping of set diameters. The mass retained on each diameter was obtained to determine the percent of soil with particles finer than the No. 200 sieve. The No. 200 sieve represents particle diameters of 75mm and serves are the threshold for coarse and fine-grained soils. The soil was classified according to the United Soil Classification System (Figure G2.1) based on the percent of soil passing the sieve.

TABLE 2.7 Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[a]				Soil Classification	
				Group Symbol	Group Name[b]
(1)	(2)	(3)	(4)	(5)	(6)
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	GRAVELS More than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVELS Less than 5% fines[c]	C_u^3 4 and $1 \le C_u \le 3$	GW	Well-graded gravel[f]
			$C_u < 4$ and/or $1 > C_u > 3$	GP	Poorly graded gravel[f]
		GRAVELS WITH FINES More than 12% fines[c]	Fines classify as ML or MH	GM	Silty gravel[f,g,h]
			Fines classify as CL or CH	GC	Clayey gravel[f,g,h]
	SANDS 50% or more of coarse fraction passes No. 4 sieve	CLEAN SANDS Less than 5% fines[d]	C_u^3 6 and $1 \le C_u \le 3$	SW	Well-graded sand[i]
			$C_u < 6$ and/or $1 > C_u > 3$	SP	Poorly graded sand[i]
		SANDS WITH FINES More than 12% fines[d]	Fines classify as ML or MH	SM	Silty sand[g,h,i]
			Fines classify as CL or CH	SC	Clayey sand[g,h,i]
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILTS AND CLAYS Liquid limit less than 50	Inorganic	PI > 7 and plots on or above "A"-line[j]	CL	Lean clay[k,l,m]
			PI < 4 and plots below "A"-line[j]	ML	Silt[k,l,m]
		Organic	$\frac{LL_{oven\text{-}dried}}{LL_{natural}} < 0.75$	OL	Organic clay[k,l,m,n] Organic silt[k,l,m,o]
	SILTS AND CLAYS Liquid limit 50 or more	Inorganic	PI plots on or above "A"-line	CH	Fat clay[k,l,m]
			PI plots below "A"-line	MH	Elastic silt[k,l,m]
		Organic	$\frac{LL_{oven\text{-}dried}}{LL_{natural}} < 0.75$	OH	Organic clay[k,l,m,p] Organic silt[k,l,m,q]
Highly organic soils		Primarily organic matter, dark in color, having organic odor		Pt	Peat

Figure G2.1 Unified Soil Classification System Chart (60)

The USCS method of classifying soil is fairly straight forward. If 50% of the soil is retained on the No. 200 sieve, then the soil is considered coarse-grained. To determine whether the soil is gravel or sand, one must then determine the coarse fraction. If the soil is gravel, then 50% of the coarse fraction will be retained on the No. 4 sieve. Otherwise the soil is sand. Further classification will depend on the percentage of fines and the coefficient of uniformity.

If 50% of the soil passes the No. 200 sieve then the soil is classified as fine grained and further analysis into classification will require an understanding of the soils plasticity and utilizing Casagrande's plasticity chart. Plasticity is dependent on the water content of a soil. Water content provides information on the amount of water present in the void spaces relative to the solids in a soil. For the USCS, one needs to know the plasticity index and the liquid limit. The plasticity index is equal to the difference between the liquid limit and the plastic limit. The liquid limit corresponds to the threshold water content where the soil begins to act like a liquid. The plastic limit is the threshold water content where the soil behaves as a plastic. The plastic and liquid behavior is related to engineering behavior. These values all fall under the category of Atterburg limits.

After speaking with Jim Hyslip, we predicted mostly coarse grained soils present at the site and therefore planned to perform only a grain size distribution analysis for the soil and did not pursue any further testing in the case of fine soil being present. When we began laboratory testing, we had to implement certain procedural alterations due to insufficient soil sample quantity obtained. ASTM-D422 requires 500g of oven dried soil to be tested. For the purposes of proceeding with the testing in a relatively timely manner, we decided to adjust the quantity of dried soil to 80g for each sample to maintain consistency and account for our testing limitations. However, we expected the possibility of skewed results from an inaccurate representation of soil distribution in each sample.

The results from this testing were not reliable enough to determine the exact soil classification (see results listed in Tables G2.1-G2.4 below). Early on we had anticipated only primary layers of soil to be present for such shallow depth below the ground surface. When using these results and obtaining the coefficients of uniformity and curvature, we found that the soil that could be classified did not indicate two layers of soil (Table G2.5). This realization is what showed the invalidity of the results. The unclassified soils required further testing for Atterburg limits, which we decided against doing since this indication stemmed from invalid data. However, we were able to determine with relative certainty that the majority of the samples were considered coarse-grained due to more than 50% of each sample being retained on the No. 200 sieve.

Table. G2.1 Soil Laboratory Results for Boring 1 (including percent finer)

Sample ID	Tin Wt. (g)	Wet Wt. (g)	Dry Wt. w/ tin (G)	Water Content (%)	#4	#10	#20	#40	#100	#200	Pan	
B-1 S-1	14.2	90.1	96.7	7.6	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					3.2	7.4	14.7	19.7	25.8	6.9	4.8	Dry sieve soil wt.
Mass Retained (%)					3.9	9.0	17.8	23.9	31.3	8.4	5.8	82.5
(%)					3.9	12.8	30.7	54.5	85.8	94.2	100.0	%fines
Percent Finer(%)					96.1	87.2	69.3	45.5	14.2	5.8	0.0	5.8
B-1 S-2	14.1	90.1	94.2	10	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					7.3	5.9	10.6	11.5	17.2	8.7	18.2	Dry sieve soil wt.
Mass (%)					9.1	7.4	13.2	14.4	21.5	10.9	22.7	79.4
Accumulated Mass					9.1	16.5	29.7	44.1	65.5	76.4	99.1	%fines
Percent Finer(%)					90.9	83.5	70.3	55.9	34.5	23.6	0.9	22.9
B-1 S-3	13.9	90.2	93.4	10.7	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					5.9	6.0	9.8	10.0	16.9	10.5	19.6	Dry sieve soil wt.
Mass (%)					7.4	7.5	12.3	12.6	21.3	13.2	24.7	78.7
Accumulated Mass					7.4	15.0	27.3	39.9	61.1	74.3	99.0	%fines
Percent Finer(%)					92.6	85.0	72.7	60.1	38.9	25.7	1.0	24.9
B-1 S-4	14	90.1	93.3	10.8	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					8.4	9.8	14.4	11.4	15.7	8.3	10.9	Dry sieve soil wt.
Mass (%)					10.6	12.4	18.2	14.4	19.8	10.5	13.7	78.9
Accumulated Mass					10.6	23.0	41.1	55.5	75.3	85.8	99.5	%fines
Percent Finer(%)					89.4	77.0	58.9	44.5	24.7	14.2	0.5	13.8
B-1 S-5	14.1	90	84.2	19.9	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					0.9	4.8	9.0	19.7	18.5	2.1	4.0	Dry sieve soil wt.
Mass (%)					1.3	6.8	12.8	28.1	26.4	3.0	5.7	59
Accumulated Mass					1.3	8.1	21.0	49.1	75.5	78.5	84.2	%fines
Percent Finer(%)					98.7	91.9	79.0	50.9	24.5	21.5	15.8	6.8
B-1 S-6	14	90.1	NA	NA	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					12.2	9.4	11.3	10.3	20.4	8.1	5.4	Dry sieve soil wt.
Mass (%)					N/A	N/A	N/A	N/A	N/A	N/A	N/A	77.1
Accumulated Mass					N/A	N/A	N/A	N/A	N/A	N/A	N/A	%fines
Percent Finer(%)					N/A	N/A	N/A	N/A	N/A	N/A	N/A	7.0

Table. G2.2 Soil Laboratory Results for Boring 2 (including percent finer)

Sample ID	Tin Wt. (g)	Wet Wt. (g)	Dry Wt. w/ tin (G)	Water Content (%)	#4	#10	#20	#40	#100	#200	Pan	
B-2 S-1	14.1	90	95.8	8.3	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					7.8	8.7	13.7	11.3	16.0	9.0	14.9	Dry sieve soil wt.
Mass (%)					9.5	10.6	16.8	13.8	19.6	11.0	18.2	81.4
Accumulated Mass					9.5	20.2	37.0	50.8	70.4	81.4	99.6	%fines
Percent Finer(%)					90.5	79.8	63.0	49.2	29.6	18.6	0.4	18.3
B-2 S-2	14.1	90.2	95.9	8.4	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					7.0	8.1	13.3	11.3	16.0	9.0	14.9	Dry sieve soil wt.
Mass (%)					8.6	9.9	16.3	13.8	19.6	11.0	18.2	79.6
Accumulated Mass					8.6	18.5	34.7	48.5	68.1	79.1	97.3	%fines
Percent Finer(%)					91.4	81.5	65.3	51.5	31.9	20.9	2.7	18.7
B-2 S-3	14	90.1	77.9	26.2	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					0.0	0.1	2.3	9.8	25.5	13.5	12.4	Dry sieve soil wt.
Mass (%)					0.0	0.2	3.6	15.3	39.9	21.1	19.4	63.6
Accumulated Mass					0.0	0.2	3.8	19.1	59.0	80.1	99.5	%fines
Percent Finer(%)					100.0	99.8	96.2	80.9	41.0	19.9	0.5	19.5
B-2 S-4	14	90.1	90.2	13.9	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					5.7	8.0	14.8	17.1	22.7	4.7	2.9	Dry sieve soil wt.
Mass (%)					7.5	10.5	19.4	22.4	29.8	6.2	3.8	75.9
Accumulated Mass					7.5	18.0	37.4	59.8	89.6	95.8	99.6	%fines
Percent Finer(%)					92.5	82.0	62.6	40.2	10.4	4.2	0.4	3.8

Table. G2.3 Soil Laboratory Results for Boring 3 (including percent finer)

Sample ID	Tin Wt. (g)	Wet Wt. (g)	Dry Wt. w/ tin (G)	Water Content (%)	#4	#10	#20	#40	#100	#200	Pan	
B-3 S-1	14	90.1	96.6	7.5	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					11.5	18.3	15.3	13.9	15.1	4.6	3.4	Dry sieve soil wt.
Mass (%)					13.9	22.2	18.5	16.8	18.3	5.6	4.1	82.1
Accumulated Mass					13.9	36.1	54.6	71.4	89.7	95.3	99.4	%fines
Percent Finer(%)					86.1	63.9	45.4	28.6	10.3	4.7	0.6	4.1
B-3 S-2	14.2	90.1	95.4	8.9	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					16.5	14.9	13.6	10.8	12.4	5.6	7.4	Dry sieve soil wt.
Mass (%)					20.3	18.3	16.7	13.3	15.3	6.9	9.1	81.2
Accumulated Mass					20.3	38.7	55.4	68.7	84.0	90.9	100.0	%fines
Percent Finer(%)					79.7	61.3	44.6	31.3	16.0	9.1	0.0	9.1
B-3 S-3	14	90	83.5	20.5	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					0.6	2.4	10.1	10.6	18.8	13.2	13.7	Dry sieve soil wt.
Mass (%)					0.9	3.5	14.5	15.3	27.1	19.0	19.7	69.4
Accumulated Mass					0.9	4.3	18.8	34.1	61.2	80.1	99.9	%fines
Percent Finer(%)					99.1	95.7	81.2	65.9	38.8	19.9	0.1	19.7
B-3 S-4	14	90.1	83.5	20.6	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					11.4	11.6	14.8	11.9	14.0	3.0	1.8	Dry sieve soil wt.
Mass (%)					16.4	16.7	21.3	17.1	20.1	4.3	2.6	68.5
Accumulated Mass					16.4	33.1	54.4	71.5	91.7	96.0	98.6	%fines
Percent Finer(%)					83.6	66.9	45.6	28.5	8.3	4.0	1.4	2.6
B-3 S-5	14.2	151	142.6	22.6	4.75	2	0.85	0.425	0.15	0.075		Opening(mm)
Mass Retained (g)					48.0	19.5	14.9	11.6	15.3	7.4	11.0	Dry sieve soil wt.
Mass (%)					37.4	15.2	11.6	9.0	11.9	5.8	8.6	127.7
Accumulated Mass					37.4	52.6	64.2	73.2	85.1	90.9	99.5	%fines
Percent Finer(%)					62.6	47.4	35.8	26.8	14.9	9.1	0.5	8.6
B-3 S-6	14.1	90.1	91.8	12.4	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					14.7	8.1	6.0	5.6	17.6	12.3	12.9	Dry sieve soil wt.
Mass (%)					18.9	10.4	7.7	7.2	22.7	15.8	16.6	77.2
Accumulated Mass					18.9	29.3	37.1	44.3	66.9	82.8	99.4	%fines
Percent Finer(%)					81.1	70.7	62.9	55.7	33.1	17.2	0.6	16.7

Table. G2.4 Soil Laboratory Results for Boring 4 (including percent finer)

Sample ID	Tin Wt. (g)	Wet Wt. (g)	Dry Wt. w/ tin (G)	Water Content (%)	#4	#10	#20	#40	#100	#200	Pan	
B-4 S-1	14.2	90.1	96.2	8.1	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					8.4	11.5	12.7	12.2	16.4	8.5	12.3	Dry sieve soil wt.
Mass (%)					10.2	14.0	15.5	14.9	20.0	10.4	15.0	82
Accumulated Mass					10.2	24.3	39.8	54.6	74.6	85.0	100.0	%fines
Percent Finer(%)					89.8	75.7	60.2	45.4	25.4	15.0	0.0	15.0
B-4 S-2	14.1	90	94.9	9.2	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					7.1	10.1	12.6	11.3	18.0	8.7	12.4	Dry sieve soil wt.
Mass (%)					8.8	12.5	15.6	14.0	22.3	10.8	15.3	80.2
Accumulated Mass					8.8	21.3	36.9	50.9	73.1	83.9	99.3	%fines
Percent Finer(%)					91.2	78.7	63.1	49.1	26.9	16.1	0.7	15.5
B-4 S-3	13.8	90.1	84.3	19.6	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					0.2	1.1	2.3	5.6	36.9	16.1	8.3	Dry sieve soil wt.
Mass (%)					0.3	1.6	3.3	7.9	52.3	22.8	11.8	70.5
Accumulated Mass					0.3	1.8	5.1	13.0	65.4	88.2	100.0	%fines
Percent Finer(%)					99.7	98.2	94.9	87.0	34.6	11.8	0.0	11.8
B-4 S-4	14	145	129	30	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					37.0	21.3	19.0	13.9	14.6	4.7	4.7	Dry sieve soil wt.
Mass (%)					32.2	18.5	16.5	12.1	12.7	4.1	4.1	115.2
Accumulated Mass					32.2	50.7	67.2	79.3	92.0	96.1	100.2	%fines
Percent Finer(%)					67.8	49.3	32.8	20.7	8.0	3.9	-0.2	4.1
B-4 S-5	14.1	90.3	92.9	11.5	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					6.5	12.6	21.2	14.7	15.1	4.7	4.6	Dry sieve soil wt.
Mass (%)					8.2	16.0	26.9	18.7	19.2	6.0	5.8	79.4
Accumulated Mass					8.2	24.2	51.1	69.8	89.0	94.9	100.8	%fines
Percent Finer(%)					91.8	75.8	48.9	30.2	11.0	5.1	-0.8	5.8
B-4 S-6	14.1	90.9	96.5	8.5	4.75	2	0.85	0.425	0.15	0.075		Sieve
Mass Retained (g)					40.8	11.9	7.8	5.6	7.8	3.9	4.3	Dry sieve soil wt.
Mass (%)					49.5	14.4	9.5	6.8	9.5	4.7	5.2	82.1
Accumulated Mass					49.5	64.0	73.4	80.2	89.7	94.4	99.6	%fines
Percent Finer(%)					50.5	36.0	26.6	19.8	10.3	5.6	0.4	5.2

Table. G2.5 Calculated Values for Coefficient of Curvature and Coefficient of Uniformity and Soil Classification Based on the Percent of Grain Size Passing the No.200 sieve

Sample ID	D10	D30	D60	Cc	Cu	% fines	Classification
B-1 S-1	0.12	0.305	0.6	1.29	5.00	5.82	Poorly Graded Sand (SP)
B-1 S-2	0.027	0.12	0.5	1.07	18.52	22.92	
B-1 S-3	0.031	0.1	0.36	0.90	11.61	24.90	
B-1 S-4	0.054	0.213	0.85	0.99	15.74	13.81	
B-1 S-5	0.053	0.16	0.05	9.66	0.94	6.78	Poorly Graded Sand (SP)
B-1 S-6	N/A	N/A	N/A	N/A	N/A	N/A	
B-2 S-1	0.05	0.17	0.7	0.83	14.00	18.30	
B-2 S-2	0.035	0.165	0.63	1.23	18.00	18.72	
B-2 S-3	0.053	0.13	0.26	1.23	4.91	19.50	
B-2 S-4	0.15	0.38	0.82	1.17	5.47	3.82	Poorly Graded Sand (SP)
B-3 S-1	0.165	0.49	0.17	8.56	1.03	4.14	Well Graded Sand(SW)
B-3 S-2	0.1	0.36	1.65	0.79	16.50	9.11	Poorly Graded Sand (SP)
B-3 S-3	0.047	0.13	0.385	0.93	8.19	19.74	
B-3 S-4	0.2	0.48	1.65	0.70	8.25	2.63	Poorly Graded Sand (SP)
B-3 S-5	0.084	0.52	1.42	2.27	16.90	8.61	
B-3 S-6	0.051	0.14	0.64	0.60	12.55	16.71	
B-4 S-1	0.05	0.18	0.93	0.70	18.60	15.00	
B-4 S-2	0.047	0.17	0.7	0.88	14.89	15.46	
B-4 S-3	0.067	0.135	0.21	1.30	3.13	11.77	
B-4 S-4	0.2	0.8	3.7	0.86	18.50	4.08	
B-4 S-5	0.15	0.44	1.25	1.03	8.33	5.79	
B-4 S-6	0.15	1.3	7.05	1.60	47.00	5.24	

Section G3: Calculation of Ultimate Bearing Capacity

The following section calculates the dimensions required for each strip footing to safely support the load above. The first step in determining a safe design was to calculate the ultimate bearing capacity, the allowable load, and the applied load acting on the structure. A strip footing has a small width (B) to length (L) ratio (B/L>>10).

Effective Stress Analysis (ESA)

<u>Ultimate Net Bearing Capacity</u> for vertical loads (assuming no incline)

$$q_u = \gamma D_f\left(N_q - 1\right)s_q d_q r_q w_q + 0.5\gamma B N_\gamma s_\gamma d_\gamma r_\gamma w_\gamma$$

Where:

$q_u = ultimate\ bearing\ capacity$

$N_q\ and\ N_\gamma\ are\ bearing\ capacity\ factors$

$s_q\ and\ s_\gamma\ are\ shape\ factors$

d_q *and* d_γ *are depth factors*

r_q *and* r_γ *are inclination factors*

w_q *and* w_γ *are groundwater factors*

D_f = *embedment depth*

B = *width of footing*

γ = *unit weight of soil*

- For our site, the GWL is located at 5 ft below the ground surface
- Assuming no inclination, the inclination factor is set to 1.
- For a shallow foundation the factory of safety is assumed to be 3.
- A friction angle is needed to determine the bearing capacity factors. The friction angle would normally use corrected blow count $N_{1,60}$. However, to be on the conservative side of bearing capacity calculations, we chose a friction angle of 34° and a unit weight of soil of 130 pcf.

The ultimate bearing capacity can be calculated with the chosen footing dimensions. For a strip footing, the only parameter we were interested in was the width of the footing. The ultimate bearing capacity was calculated to be 19.4 ksf, based on the following values.

Ultimate Bearing Capacity		
friction angle	34	degrees
Unit Weight	130	lb/ft^3
Depth of Embedment	3	ft
rq	1	
rgam	1	
Ngam	34	
Nq	32	
B	3	ft
L	1	ft
D/B	1	
dgam	1	
dq	1	
Sq	1	
Sgam	1	
unitweight bouyant	67.6	lb/ft^3
wq	1.32	
wgam	0.52	
qult	19406	lb/ft^2
FS	3	
Load for one foundation	19335	lb/ft
qapplied	5093	lb/ft^2
qallowed	6859	lb/ft^2

To determine if this ultimate bearing is based an acceptable footing design; the applied load must be less than the allowable bearing capacity, which in turn can be calculated as shown in Equations G3.1 and G3.2.

To calculate the applied load the total weight acting on the footing needed to be determined.

Load Breakdown for 4'x35' culvert opening					
Looking at 1ft. Strip segments					
Dead Load	Structure		Volume, ft^3	Unit Weight, lb/ft^3	Weight, lb
	Culvert	Side Wall	7.00	150.00	1050.00
		Side Wall	7.00	150.00	1050.00
		Side Wall	52.50	150.00	7875.00
	Soil	Soil	98.05	130.00	12746.50
	Road	Top Base	4.63	4.15	19.19
		Sub Base	7.71	4.15	31.99
		3/4" Gravel	18.50	101.85	1884.23
	ConcreteFoundation	Footing	3.00	150.00	450.00
		Footing	3.00	150.00	450.00
Live Load	Truck	Type 3 Vehicle			5000.00
Total Weight Felt By	Both Footings	30556.91 lb			
	Single Footing	15278.45 lb			

Calculation of applied load

$$q_{applied} = \frac{\frac{total\ weight\ on\ single\ footing}{1ft\ segment}}{width\ of\ footing} = \frac{\frac{15278lb}{1ft}}{3ft} = 5093\ psf = 5.09\ ksf \qquad \text{[Eq. G3.1]}$$

Calculation of allowable load

$$q_{allowable} = \frac{q_{ult}}{F.S.} + \gamma D_f \qquad \text{[Eq. G3.2]}$$

Where:

$q_{ult} = ultimate\ bearing\ capacity$

$F.S = Factor\ of\ Safety$

$\gamma = soil\ unit\ weight$

$D_f = embedment\ depth$

The allowable bearing capacity was calculated to be the following

$$q_{allowable} = \frac{19406psf}{3} + (130pcf)3ft = 6859\ psf = 6.86\ ksf$$

Because 5.09 ksf < 6.85 ksf, the applied load is less than the allowable. Therefore, the foundation design is acceptable.

Section G4: Calculation of Settlement

To further determine whether or not the chosen width of the strip footing was acceptable, we calculated the settlement. The maximum settlement allowed for our structure was set at 1 inch by our liaison Jim Hyslip. Because of the nature of our site and structure, we were only interested in the immediate settlement. To determine this, we calculated the elastic settlement using Equation G4.

$$\rho_e = \frac{2Q_a}{EL_1}(1 - v^2{}_u)\mu_s\mu_{emb}\mu_{wall} \qquad \text{[Eq. G4]}$$

Where:

$\rho_e = Elastic\ Settlement$

$Q_a = allowable\ vertical\ stress$

$E = elastic\ modulus$

$L_1 = length\ of\ circumscribed\ rectangle$

$v_u = Poisons\ ratio\ for\ undrained\ consolidation$

$\mu_s = shape\ factor$

$\mu_{enb} = embedment\ factor$

$\mu_{wall} = wall\ adhersion\ factor$

Immediate Settlement		
Qa	20576	lb
E	7000	psi
L1	12	inches
vu	0.3	
µs	1	
µemb	1	
µwall	0.9	
Ab/L1^2	0	
Ab	432	
Aw	144	
B1	36	inches
ρe	0.4	inches

The elastic settlement calculated for using the values listed in the table above was determined to be 0.4 inch. Since this is less than the 1 inch maximum settlement stipulated by Jim Hyslip, this further verified the foundation design.

Section G5: Calculation of Reinforcement

Each footing requires steel bar reinforcements to prevent flexure and shrinkage. The calculation for reinforcement was based on the chosen width and thickness of the footings. Before beginning the reinforcing calculations, we first verified the shear within a single footing since this is what usually governs the thickness of a footing.

<u>Calculating shear:</u>

The shear had to be checked a distance *d* away from where the face of the culvert wall would be acting on the footing. The calculation for this distance utilized Equation G5.1.

$$d = b_w - 3\ inch\ cover - \frac{1}{2} bar\ diameter \qquad \text{[Eq. G5.1]}$$

Where b_w is the chosen thickness for the footing. We chose a thickness of 12 inches.

$$d = 12\ in. - 3\ in. - \frac{1}{2}(0.625\ in.) = 8.69\ in.$$

Next we had to determine the factored shear force at the section, V_u, using Equation G4. 2

$$V_u = q_u\left(\frac{b_w - d}{12\ in.} \times 1ft\ segment\right) = 19.4ksf\left(\frac{3.31in}{12in} \times 1ft.\right) = 5.35kips/ft \qquad \text{[Eq. G5.2]}$$

Then we had to compare V_u to the critical shear force (Equation G5.3) provided by our footing design.

$$\varphi V_c = \varphi 2\sqrt{f'_c} b_w d \qquad \text{[Eq. G5.3]}$$

Where

f'_c = specified compressive strength for the concrete = 5000psi

$\varphi = strength\ reduction\ factor\ for\ concrete = 0.85$

$$\varphi V_c = 0.85 \times 2 \times \sqrt{5000psi} \times 12\ in. \times 8.69\ in. = 12.53\ kips/ft$$

Since $V_u < \varphi V_u$ the shear is acceptable for our design.

Calculating flexure reinforcement:

To calculate the reinforcing bar required to account for any flexure, we first had to determine the moment at the critical section at the face of the wall and then calculate the countering moment produced by the chosen reinforcing bar.

The critical moment acting at the face of the wall was determined using Equation G5.4.

$$M_u = q_u \frac{\left(\frac{Distance\ of\ footing\ out\ from\ wall}{12}\right)^2 \times 1\ ft\ segment}{2} \qquad \text{[Eq. G5.4]}$$

Where

$q_u = ultimate\ bearing\ capacity = 19.4ksf$

$M_u = critical\ moment\ due\ to\ factored\ loads$

$$M_u = 19.4ksf \times \frac{\left(\frac{12in}{12in}\right)^2 \times 1ft}{2} = 9.7ft - kip/ft$$

Next, to calculate the countering moment we used Equation G5.5 and the following assumptions.

$$M_u = \varphi M_n = \varphi A_s f_y jd \qquad \text{[Eq. G5.5]}$$

Where

$\varphi = strength\ reduction\ factor\ for\ steel = 0.9$

$M_n = nominal\ moment\ strength, in.-lb$

$A_s = area\ of\ nonstressed\ tension\ reinforcement, in.^2 = to\ be\ calculated$

f_y = specified compressive strength for the steel = 60,000psi

$jd = dist. between\ \ resultants\ of\ internal\ compressive\ and\ tensile\ forces\ on\ x\ sec$

For the purposes of obtaining an initial calculation, we assumed j=0.95 and jd =8.256. We then calculated the area of nonstressed tension reinforcement using Equation G5.6.

$$A_s = \frac{M_u \times 12{,}000}{\varphi \times f_y(jd)} = \frac{(9.7ft-kip/ft)\times 12{,}000}{0.9\times 60{,}000psi\times 8.256in.} = 0.261\ in^2/ft \qquad \text{[Eq. G5.6]}$$

The minimum area of nonstressed tension reinforcement was in Equation G5.7 from specification in ACI Secs. 10.5.3 and 7.12.2.

$$A_{s_{min}} = 0.0018bh = 0.0018 \times 12in \times 12in = 0.26in^2/ft \qquad \text{[Eq. G5.7]}$$

Since $A_s > A_{smin}$, it is acceptable.

From ACI Sec. 7.6.5, the maximum spacing of bar is 3h or 18 inches. We chose to use 18 inches as our maximum spacing. After trial and error, we chose to use No. 5 bars at 9 inch spacing off center. For No. 5 bars with 9 inch spacing,

$$A_s = 0.41\ in.^2/ft$$

Before recalculating the moment, we had to check the value of j using Equation G5.8 for the depth of equivalent rectangular stress block.

$$a = \frac{A_s \times f_y}{\varphi \times f'_x \times 12in} = 0.482\ in. \qquad \text{[Eq. G5.8]}$$

Where

$a = depth\ of\ equivalent\ rectangular\ stress\ block$

Since a/d=0.482 in/8.69 in= 0.05 is much less than 0.75(a_b/d) = 0.377 from Table A-5 of Reinforced Concrete text. We determined the actual distance between internal compression and tensile forces on the cross section using Equation G5.9

$$jd = d - \frac{a}{2} = 8.69in. - \frac{0.482\ in.}{2} = 8.45\ in. \qquad \text{[Eq. G5.9]}$$

Since this value is close to our jd value when we assumed j, our assumption was acceptable.

Using the jd value based on No. 5 bar at 9in. spacing, the countering moment was calculated as follows

$$M_u = \frac{0.9 \times 0.41 \times 60{,}000 \times 8.45in.}{12{,}000} = 15.59ft - kip/ft$$

Since this moment is greater than the critical moment, calculated earlier, our flexural reinforcement of **No. 5 bars at 9 inch on-center** is acceptable.

Calculating Temperature Reinforcement:

The temperature reinforcement is meant to reinforce the footing against shrinkage. By ACI Sec 7.12.2 we require

$$A_{s_{min}} = 0.0018bh = 0.0018 \times 36in. \times 12in = 0.78in^2$$

This is the minimum area of our bar reinforcement that must be met. The maximum spacing for the bar is 5 times the footing thickness (From ACI 7.12.2.2), but cannot exceed 18 inch. Since 5×12 = 60 inch, we used 18 inch as our limit.

To meet the area we chose 3- No. 5 bars with 12 inch spacing. This provided the following A_s

$$A_s = (0.29) \times 3 = 0.87\ in^2/ft$$

The value 0.29 was the A_s listed in Table A-11 of the Concrete Reinforcement Text for a No. 5 bar with 12 inch spacing. Since the total area of the bar in a 1 ft section was calculated to be greater than the minimum, our shrinkage reinforcement of **3- No. 5 with 12 inch spacing** can be used in our design.

Section G6: Calculation of Gabion Walls

In order to design the gabion walls, we first determined the height of wall required at each location to retain the soil. Overall we chose a standard height of 9 ft. The base required for this height, with regards to a stepped back face, was 7.5 ft (64). The overall design chosen was a 3 layer stepping system that stepped up 3 feet for each layer and stepped in 1.5 feet (see Figure G6.1 below).

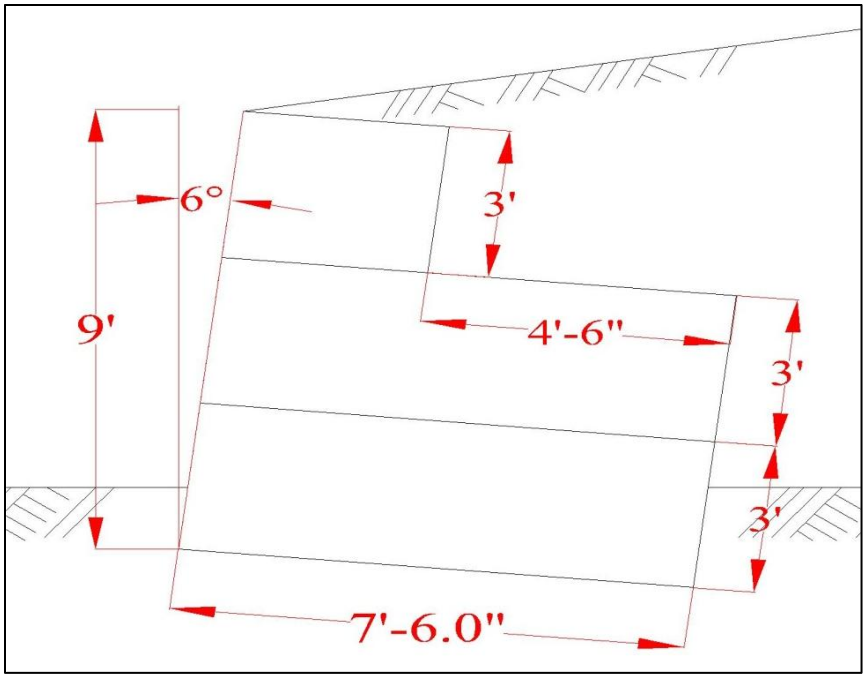


Figure G6.1 Dimensions and orientation of Gabion A

Overturning Moment Check

The main goal of checking the overturning moment is to find a ratio of the resisting moment to the overturning moment that will produce a factor of safety of at least 2 (64). First, we the forces acting on our gabion wall design were calculated. These forces consisted of the active earth pressure and the horizontal component of the active earth pressure. Equation G6.1 was used to calculate the active earth pressure, assuming no surcharge was present and that the unit weight of the soil at our site was 130 pcf. To calculate k_a, we used Equation G6.2.

$$P_a = \frac{k_a w_s H^2}{2} \qquad \text{[Eq. G6.1]}$$

Where:

$P_a = active\ earth\ pressure, \frac{lb}{ft}$

$k_a = pressure\ coefficient$

$w_s = unit\ weight\ of\ soil = 130\ pcf$

$H = wall\ height$ = 9 ft

$$k_a = \frac{cos^2(\emptyset-\beta)}{cos^2\beta\cos(\delta+\beta)\left[\sqrt{\frac{\sin(\emptyset+\delta)\sin(\emptyset-\alpha)}{\cos(\delta+\beta)\cos(\alpha-\beta)}}\right]}2 \quad \text{[Eq. G6.2]}$$

Where

$\emptyset = coefficient\ of\ internal\ friction = 34°$

$\beta = batter = 6°$

$\propto = sliope\ angle\ of\ backfill = assumed\ to\ be\ zero$

$\delta = angle\ of\ wall\ friction = assumed\ to\ be\ zero$

The calculated pressure coefficient for our design was

$$k_a = 0.54$$

Therefore

$$P_a = 2{,}843 lb/ft$$

To calculate the horizontal component of the active earth pressure we used Equation G6.3.

$$P_h = P_a cos\beta \quad \text{[Eq. G6.3]}$$

Where

$P_h = horizontal\ component\ of\ the\ active\ earth\ pressure, \frac{lb}{ft}$

$P_a = active\ earth\ pressure = 2{,}843\frac{lb}{ft}$

$\beta = batter = 6°$

The horizontal component of the active earth pressure was calculated to be

$$P_h = \frac{2{,}827lb}{ft}$$

Now that all the forces were calculated, the distances and moments from for the overturning and resisting forces were calculated.

Since no wall friction was assumed, the active earth force would act normal to the slope of the back face at a distance of H/3, shown by Equation G6.4.

$$d_a = \frac{H}{3} = 3ft \qquad \text{[Eq. G6.4]}$$

Where

$d_a = distance\ of\ total\ active\ force\ above\ the\ toe$

$H = wall\ height$= 9 ft

The overturning moment was determined using Equation G6.5.

$$M_o = d_a P_h \qquad \text{[Eq. G6.5]}$$

Where

$M_o = overturning\ moment$

$d_a = distance\ of\ total\ active\ force\ above\ the\ toe$ = 3 ft

$P_h = horizontal\ component\ of\ the\ active\ earth\ pressure = 2{,}827\ \frac{lb}{ft}$

The overturning moment was calculated to be

$$M_o = 8{,}483\ ft - lb/ft$$

Equation G6.6 was used to calculate the weight of the gabion for a 1 foot long section.

$$W_g = V_g \gamma_g \qquad \text{[Eq. G6.6]}$$

Where

$W_g = weight\ of\ gabion$, lb/ft

$A_g = total\ area\ \ of\ gabion\ for\ a\ 1\ foot\ secion$ = 36 ft^3

$\gamma_g = unit\ weight\ of\ gabion\ stone\ material = assumed\ to\ be\ basalt = 180\ pcf$

The weight of the gabion section was calculated to be

$$W_g = 6{,}480\ lb/ft$$

The weight of the gabion acts vertically through the centroid of its cross section the location of which was found using Equation G6.7.

$$d_g = \sum A_x / \sum A \qquad \text{[Eq.G6.7]}$$

Where

$d_g = Horizontal\ distance$

$\sum A_x = moments\ of\ areas\ about\ the\ toe$

$\sum A = total\ area$ = 36 ft

This distance was calculated as follows

$$d_g = \lfloor 13.5(2.25\cos(6) + 1.5\sin(6)) + 13.5(3\cos(6) + 4.5\sin(6)) + 9(3.75\cos(6) + 7.5\sin(6)\rfloor / 36$$

$$d_g = 3.04 ft$$

Equation G6.8 was used to calculate the resisting moment.

$$M_r = d_g W_g \qquad \text{[Eq. G6.8]}$$

Where

$M_r = resisting\ moment$

$d_g = horizontal\ distance = 3.04\ ft$

$W_g = weight\ of\ gabion$= 6,480 lb/ft

The resisting moment was calculated as

$$M_r = 19{,}699\ ft - lb/ft$$

Finally we checked that the safety factor against overturning was less than 2, the common safety factor for overturning (64). Equation G6.9 was used to determine this.

$$SF_o = \frac{M_r}{M_o} \qquad \text{[G6.9]}$$

Where

$SF_o = safety\ factor\ against\ overturning$

$M_r = resisting\ moment =\ 19{,}699\ ft - lb/ft$

$M_o = overturning\ moment = 8{,}483\ ft - lb/ft$

The safety factor for our design was calculated to be

$$SF_o = 2.3$$

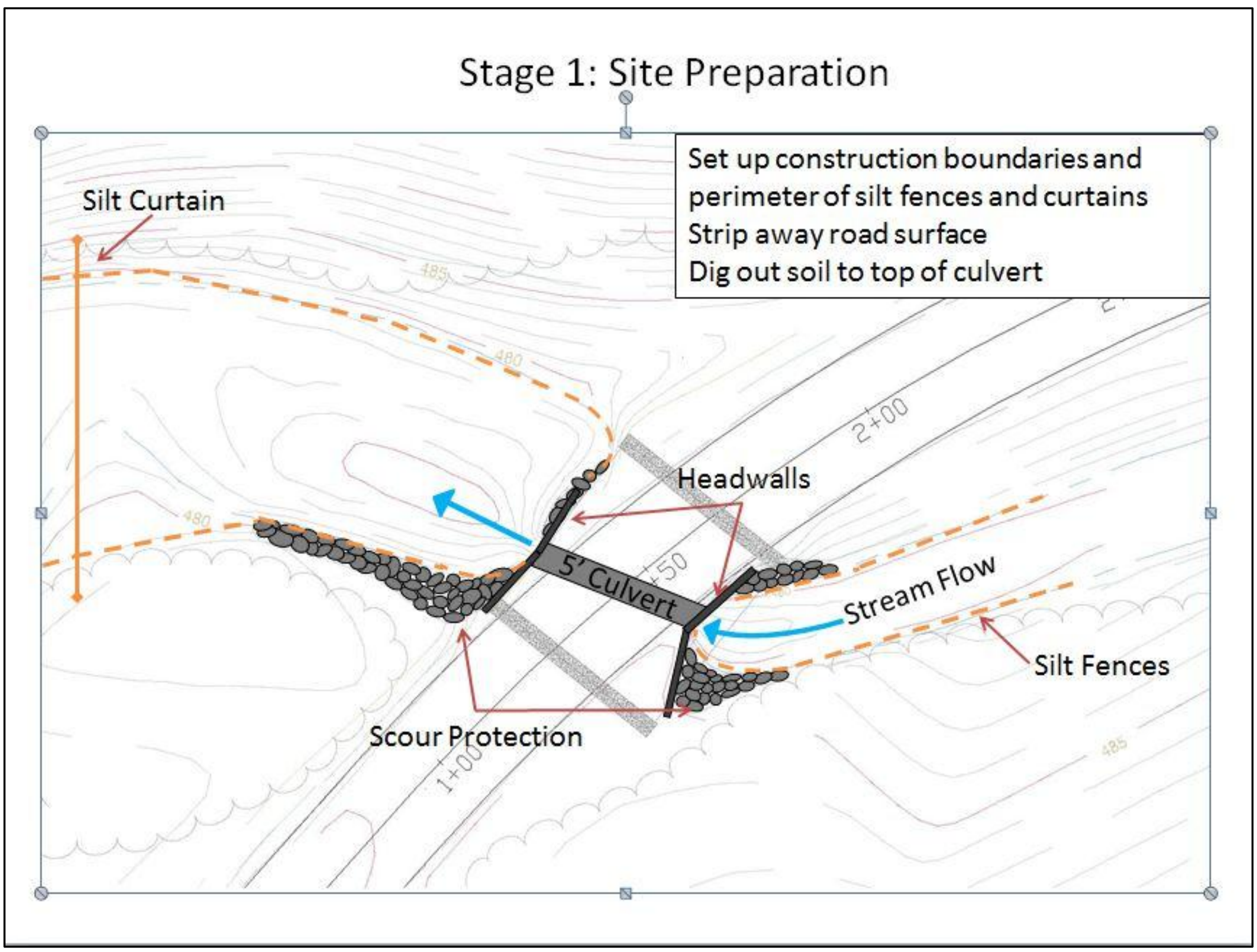
Since 2.3 > 2, our design is stable against overturning

Appendix H: Construction Staging Plan & Diagrams

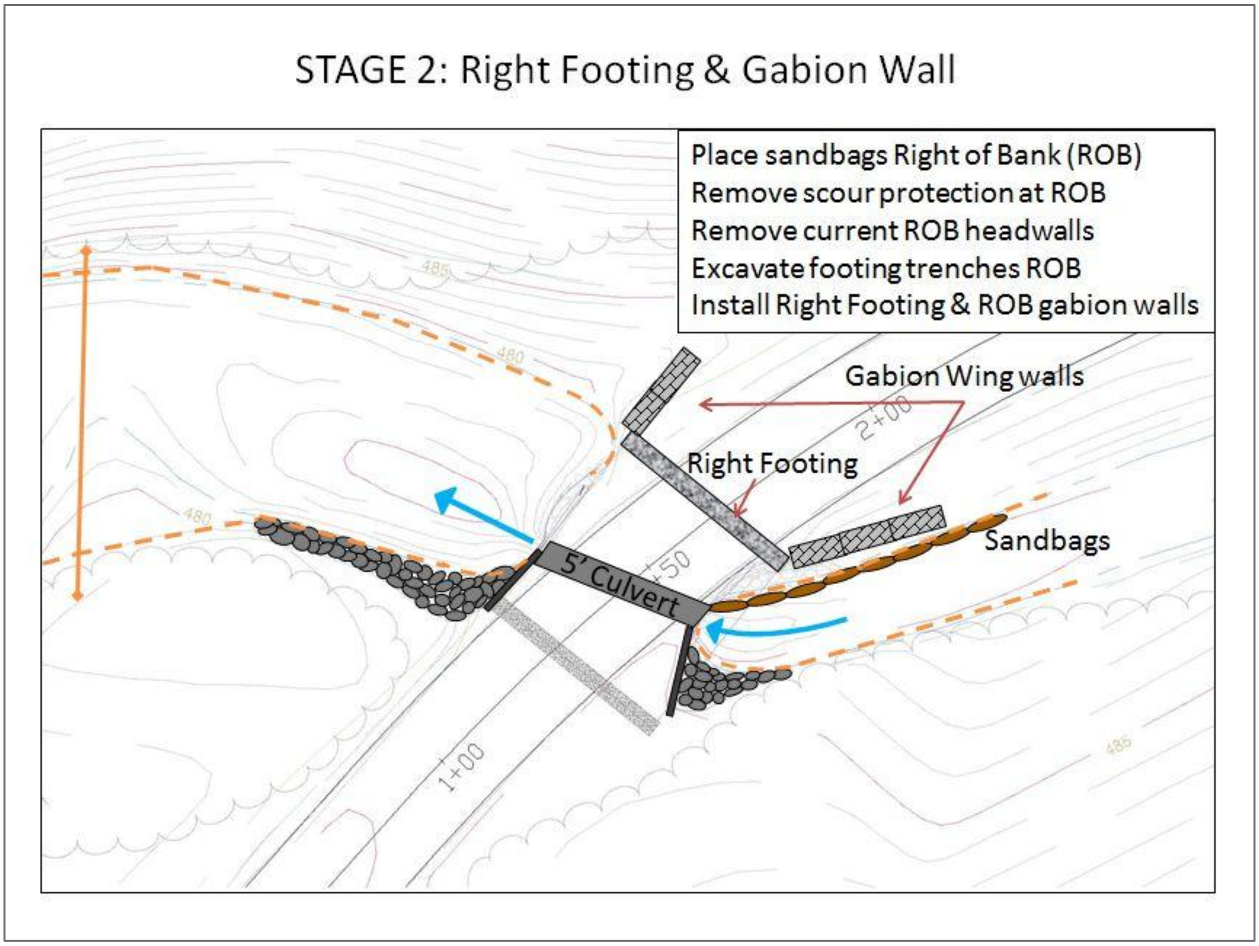
The diagrams here are to help guide the phasing of construction by depicting the different stages outlined in the Construction Staging Plan.

Construction Staging Plan & Corresponding Phase Diagram

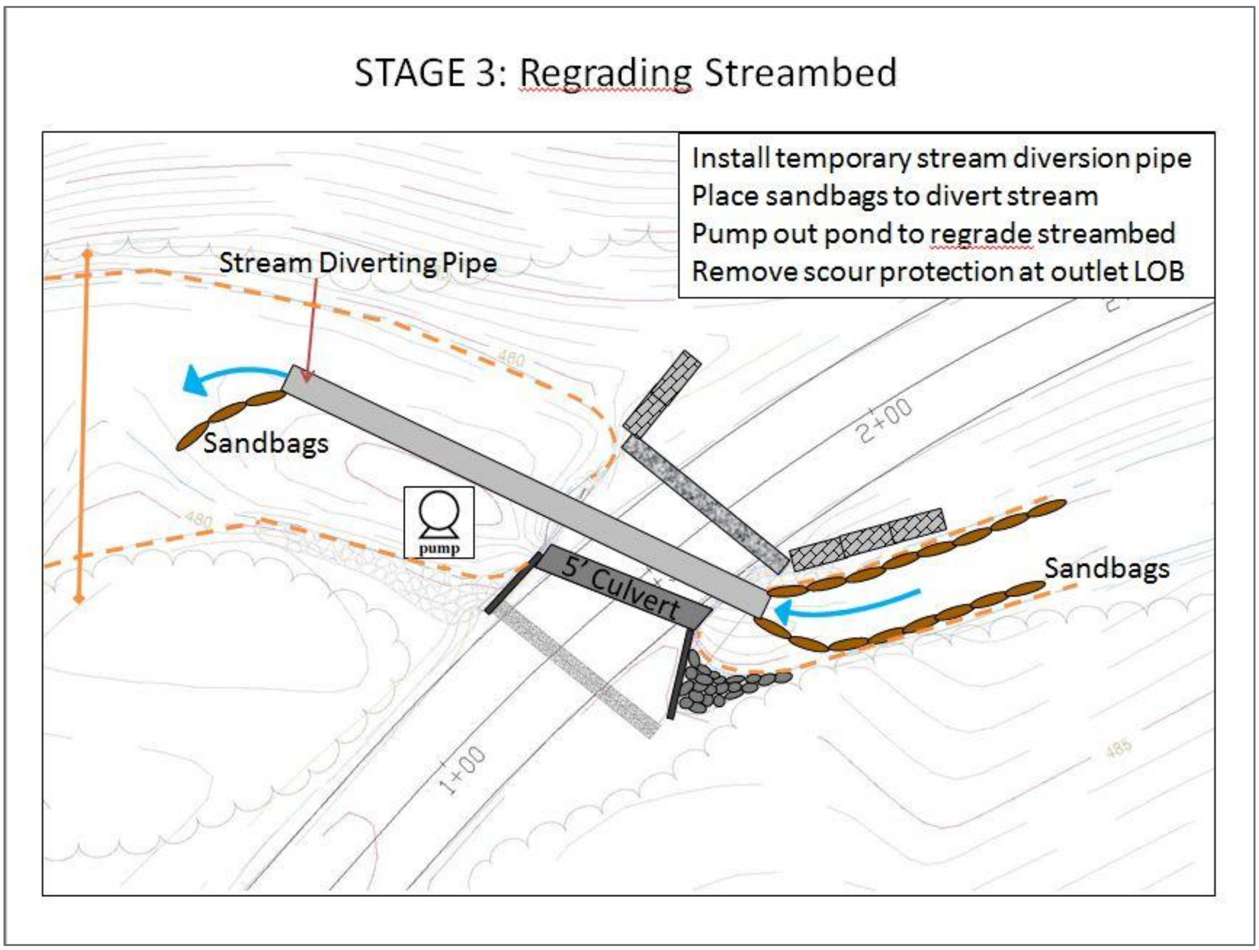
STAGE 1: Site Preparation
Lay out construction site boundaries, silt fences, and set up detour.
Strip away current road surface needing replacement.
Dig out soil to top of current culvert (482’).



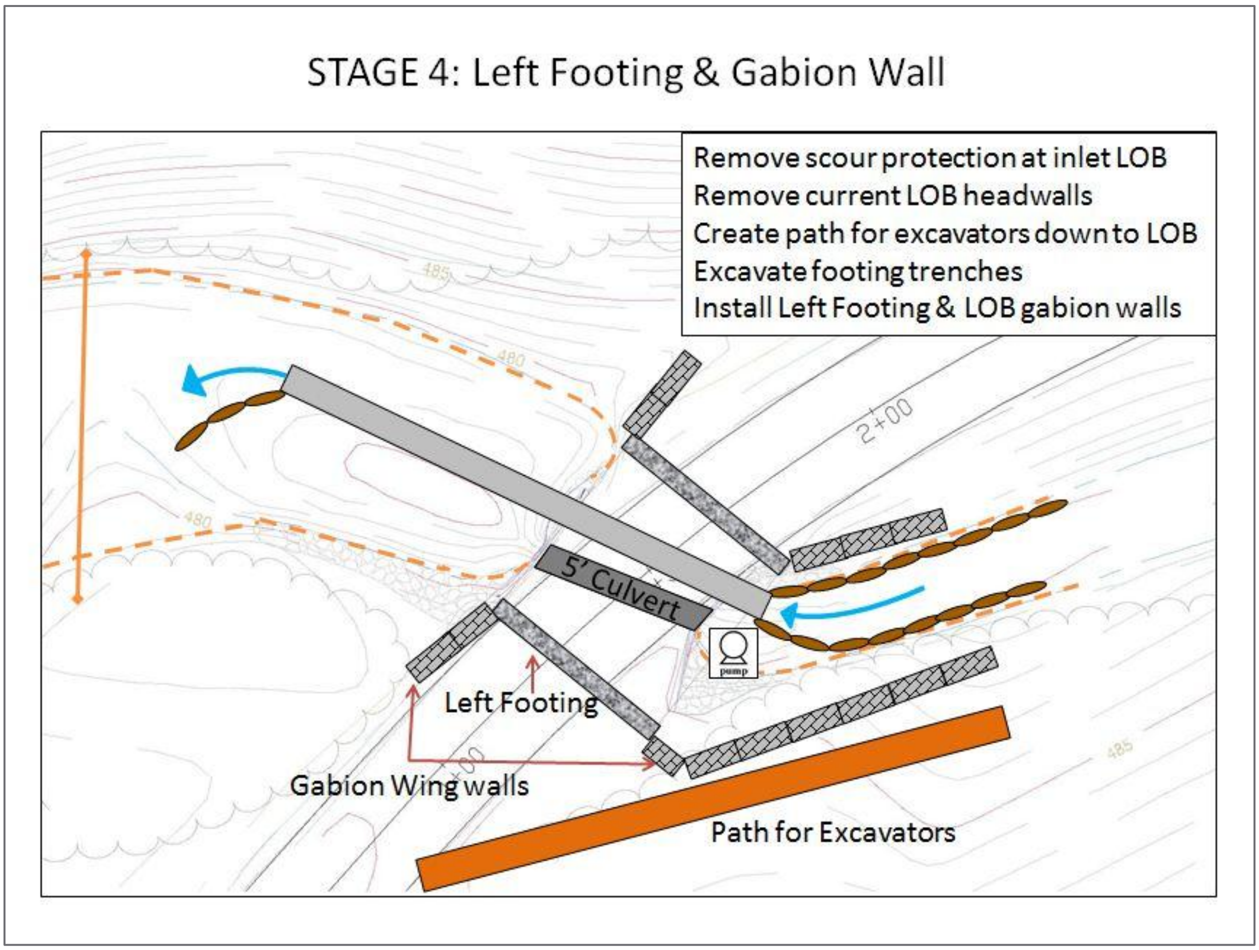
STAGE 2: Right Footing & Gabion Wall
Place sandbags ROB to contain stream (still flowing into culvert).
Remove current stream training rocks at both inlet & outlet ROB.
Using a maximum 53° (45°,34°) slope starting from the top of the culvert, excavate down to bedrock (473') to footing location.
If bedrock cannot be reached, excavate to top of rock and pour lean concrete.
Break current headwall to remove Right half of headwalls at inlet & outlet.
Pour concrete for Right Footing.
Fill above Right Footing until 476'.
From road level, place gabion baskets (embed from 476') for up and downstream ROB wing walls.



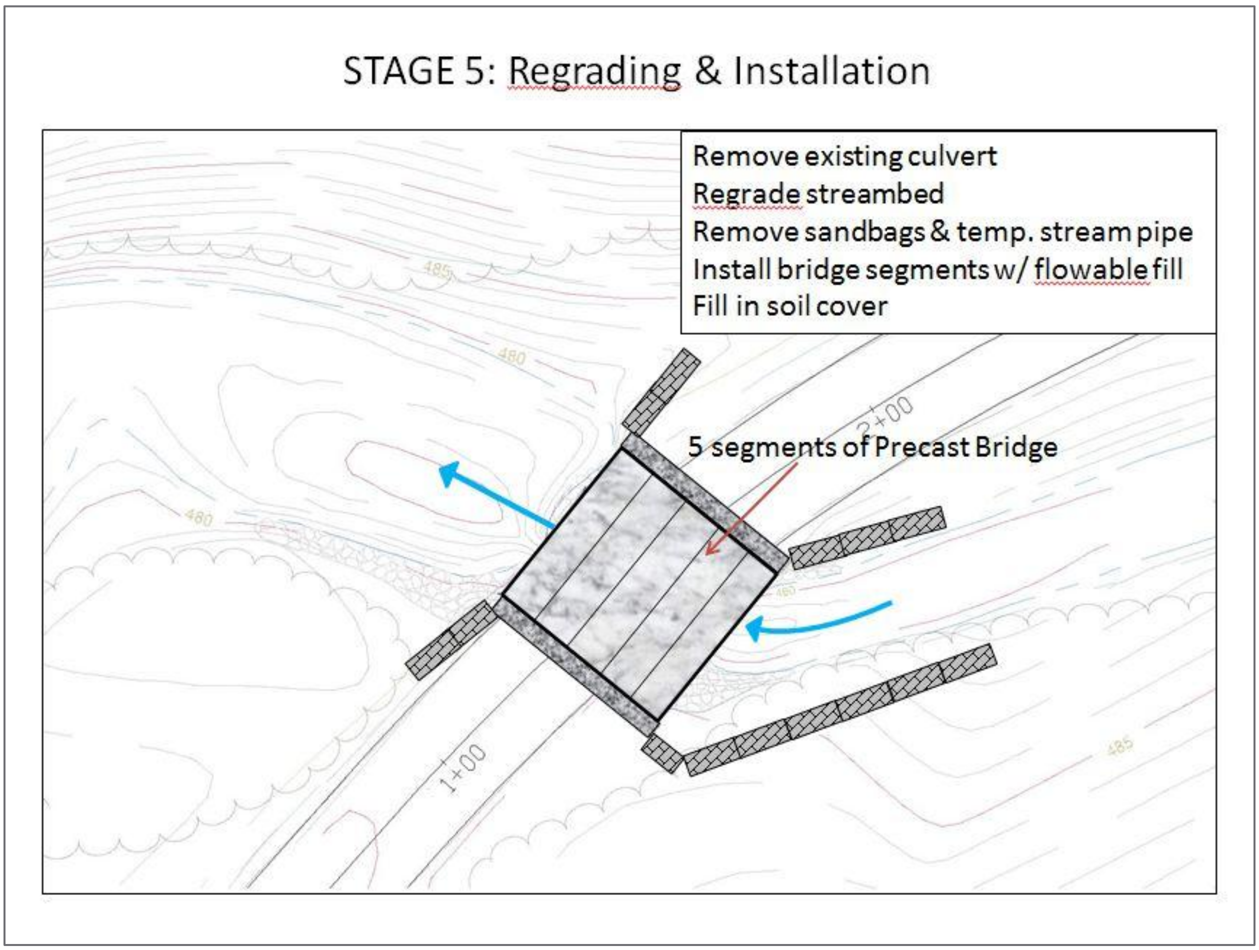
STAGE 3: Regrading Streambed
Dig down to 477’ (right side of culvert) to regrade streambed to have gradual slope.
Install temporary stream diversion pipe that extends downstream 80’ from culvert entrance into the regraded right side.
Remove sandbags and place them on LOB to direct stream away into the temporary pipe (no flow in culvert).
Pump out the still pond water downstream of culvert.
Remove current stream training rocks at outlet LOB.
Fill in the downstream ponded area to continue the gradual slope from culvert exit (slope = 0.0123). At 45’ downstream, elevation should be 476’.



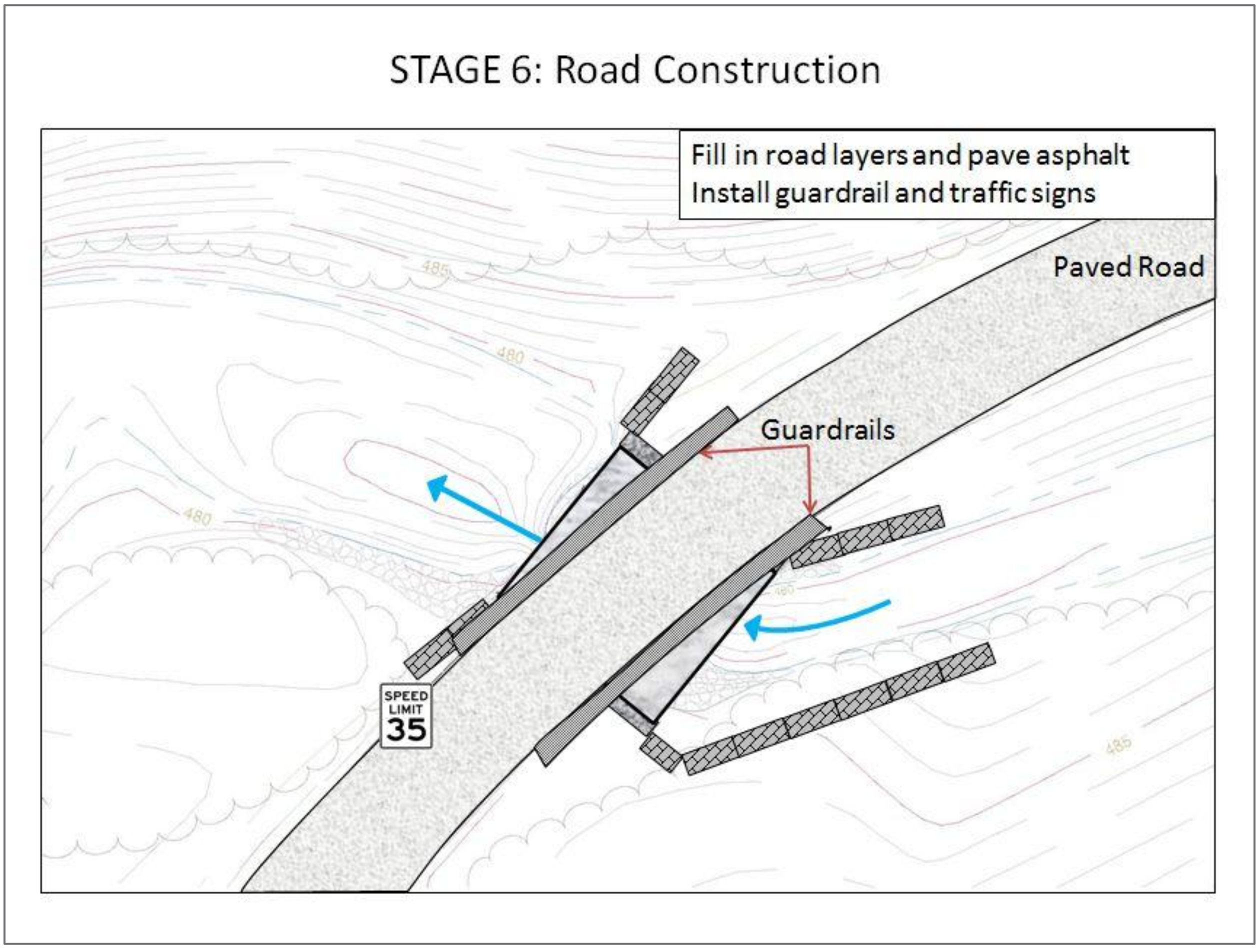
STAGE 4: Left Footing & Gabion Wall
Pump out water at LOB that may be left from the stream training.
Remove current stream training rocks at inlet LOB.
Using a maximum 53° (45°,34°) slope starting from the top of the culvert, excavate down to bedrock (473’) to footing location.
If bedrock cannot be reached, excavate to top of rock and pour lean concrete.
Remove Left half of headwalls at inlet & outlet.
Pour concrete for Left Footing.
Fill above Left Footing until 476’.
Place a temporary path from the road down to upstream LOB for excavators.
Using the path, place gabion baskets (embed from 476’) for LOB wing wall, working way back up the path to the road.
From road level, place gabion baskets (embed from 476’) for downstream LOB wing walls.



STAGE 5: Regrading & Installation
Remove current culvert.
Dig down to 477’ (left side of bridge) to regrade streambed to have gradual slope.
Remove sandbags and temporary stream diversion pipe to allow free flow of stream.
Add flowable fill to inner sides of bridge while installing each of the 5 segments.
Fill in soil cover above and around bridge.

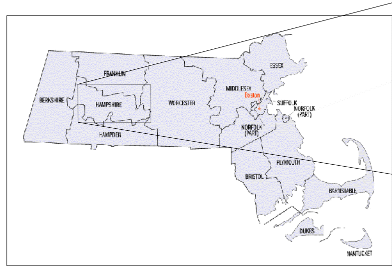


STAGE 6: Road Construction	
Fill in layers of road material following the new alignment.	
Pave the asphalt.	
Install guardrails and traffic signs (speed limit, curved road).	
Total Time	**1 month**

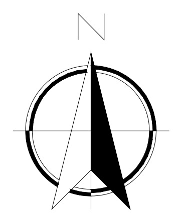


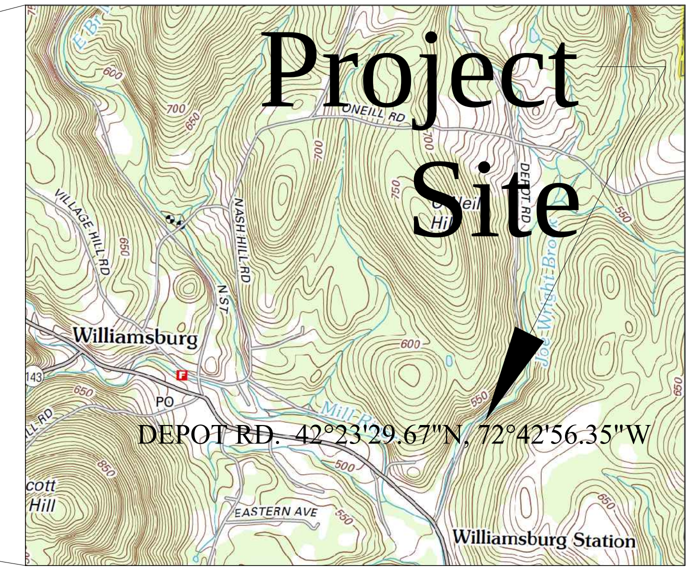
Appendix I: Construction Drawings

Depot Road Culvert Replacement Construction Documents



State of Massachusetts
Hampshire County
Not To Scale





Project Vicinity Map
Not To Scale

move	
Sheet Number	Sheet Title
1	Cover Sheet
2	Road Alignment Plan
3	Replacement Structure and Road Elevation Plan
4	Road Layout and Elevation and Superelevation Detail
5	Bridge Dimension, Orientation, and Placement Detail
6	Strip Footing Location, Dimension, and Reinforcement Detail
7	Retaining Wall Design and Location Detail
8	Depot Rd. Culvert Replacement Bore Hole Logs B1 and B2
9	Depot Rd. Culvert Replacement Bore Hole Logs B3 and B4

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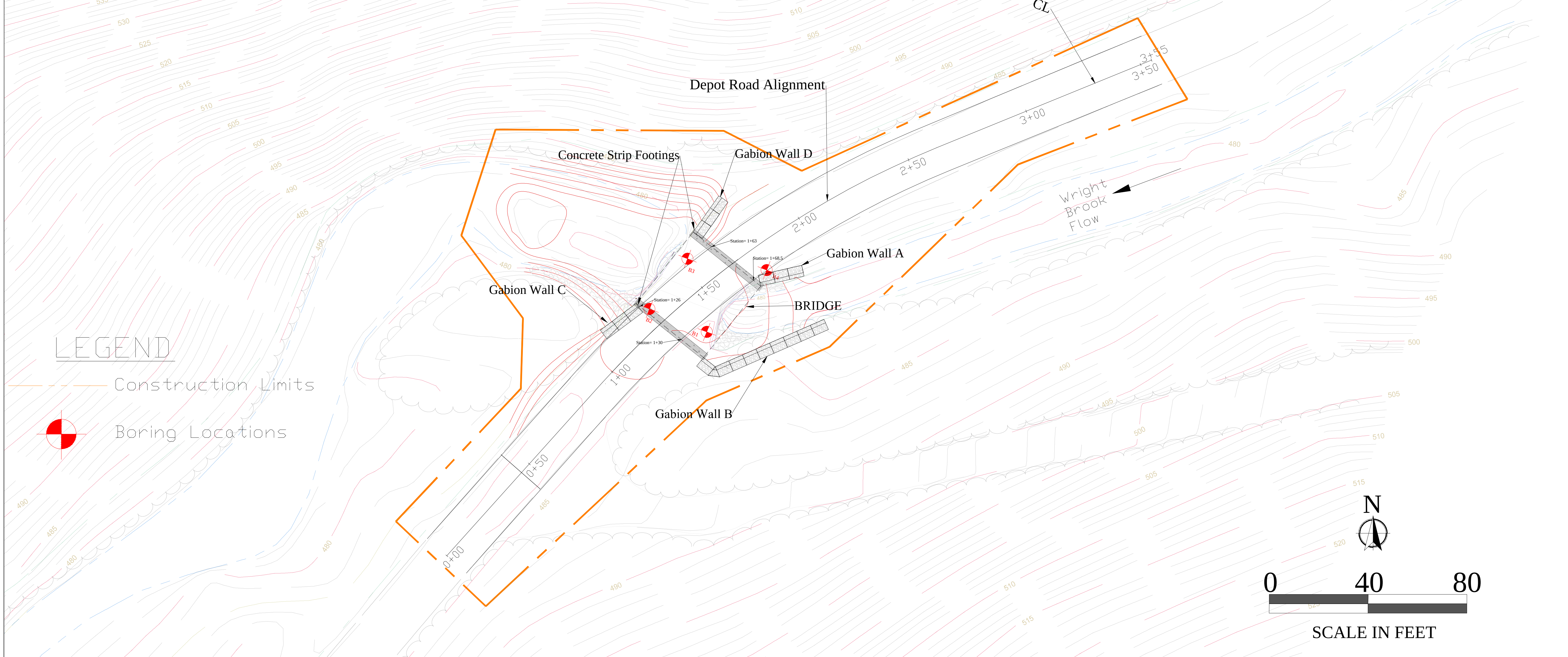
Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

Cover Sheet,Vicinity Map, and Sheet Index

SHEET
1 OF 9

CURVE TABLE						
	PC	PT	RADIUS	LENGTH	TANGENT	DELTA
CURVE	0 + 94	2 + 61	371'-0"	167'-0"	85'-8"	26 DEG



PLAN: DEPOT ROAD CULVERT REPLACEMENT AND ROAD REALIGNMENT

NO.	BY	Date	REVISION DESCRIPTION

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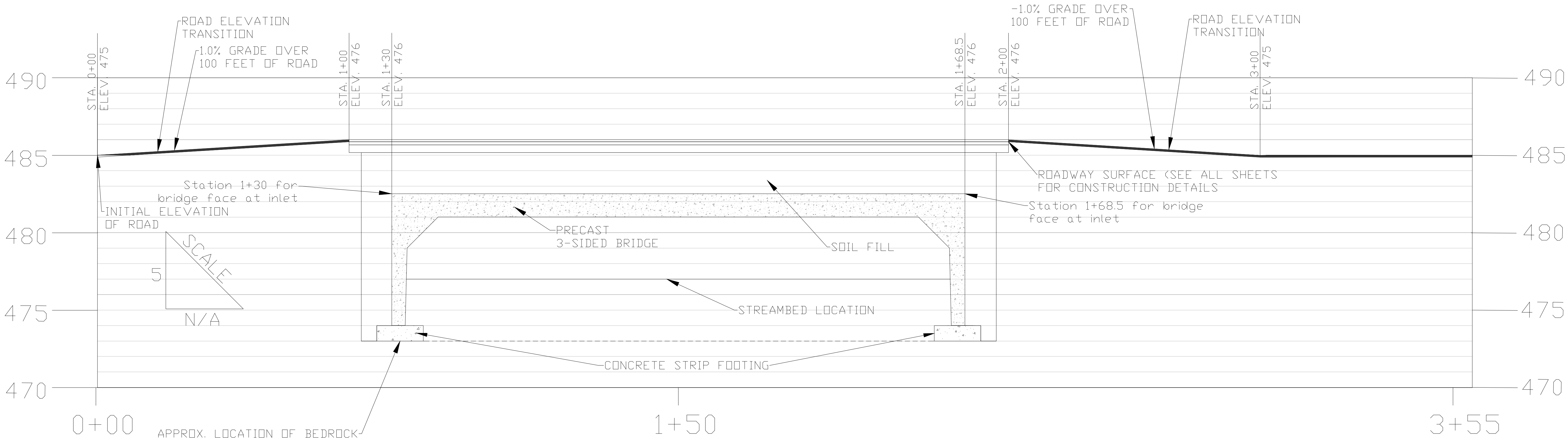
APPROVED DATE PROJECT

Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

Road Alignment Plan

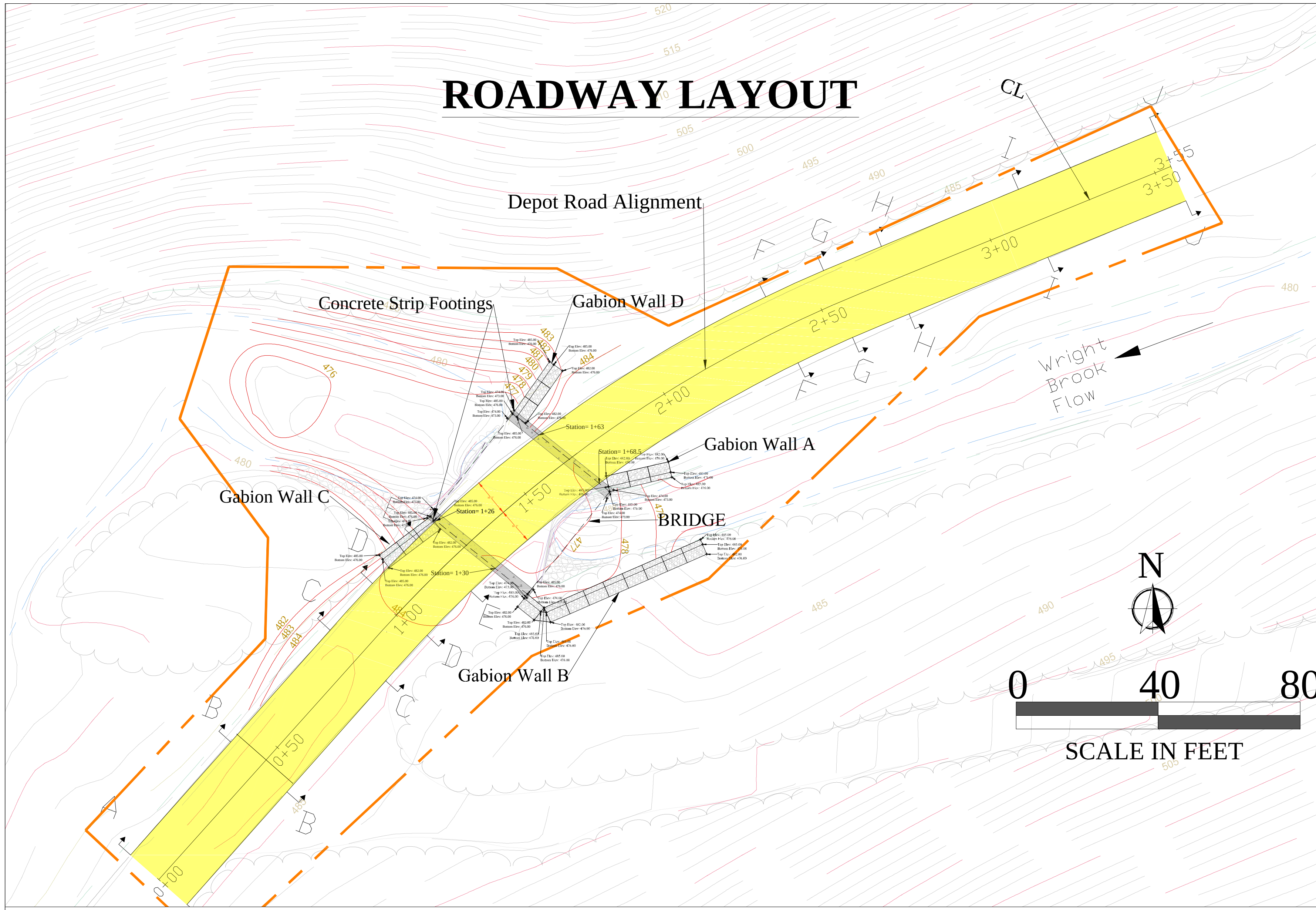
SHEET 2 OF 9



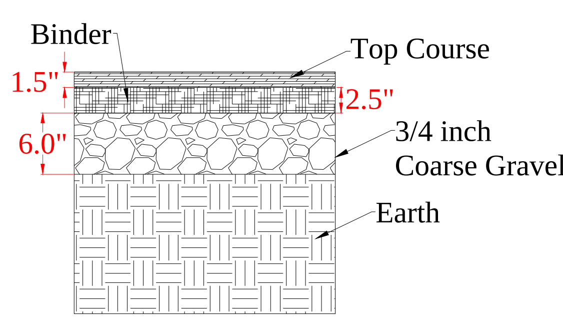
PROFILE VIEW THROUGH ROAD ALIGNMENT

NO.	BY	Date	REVISION DESCRIPTION	DRAWN / DESIGNED / CHECKED / APPROVED / DATE / PROJECT	Design of Culvert Replacement Structure at Depot Road, Williamsburg MA	Replacement Structure and Road Elevation Plan	SHEET 3 OF 9

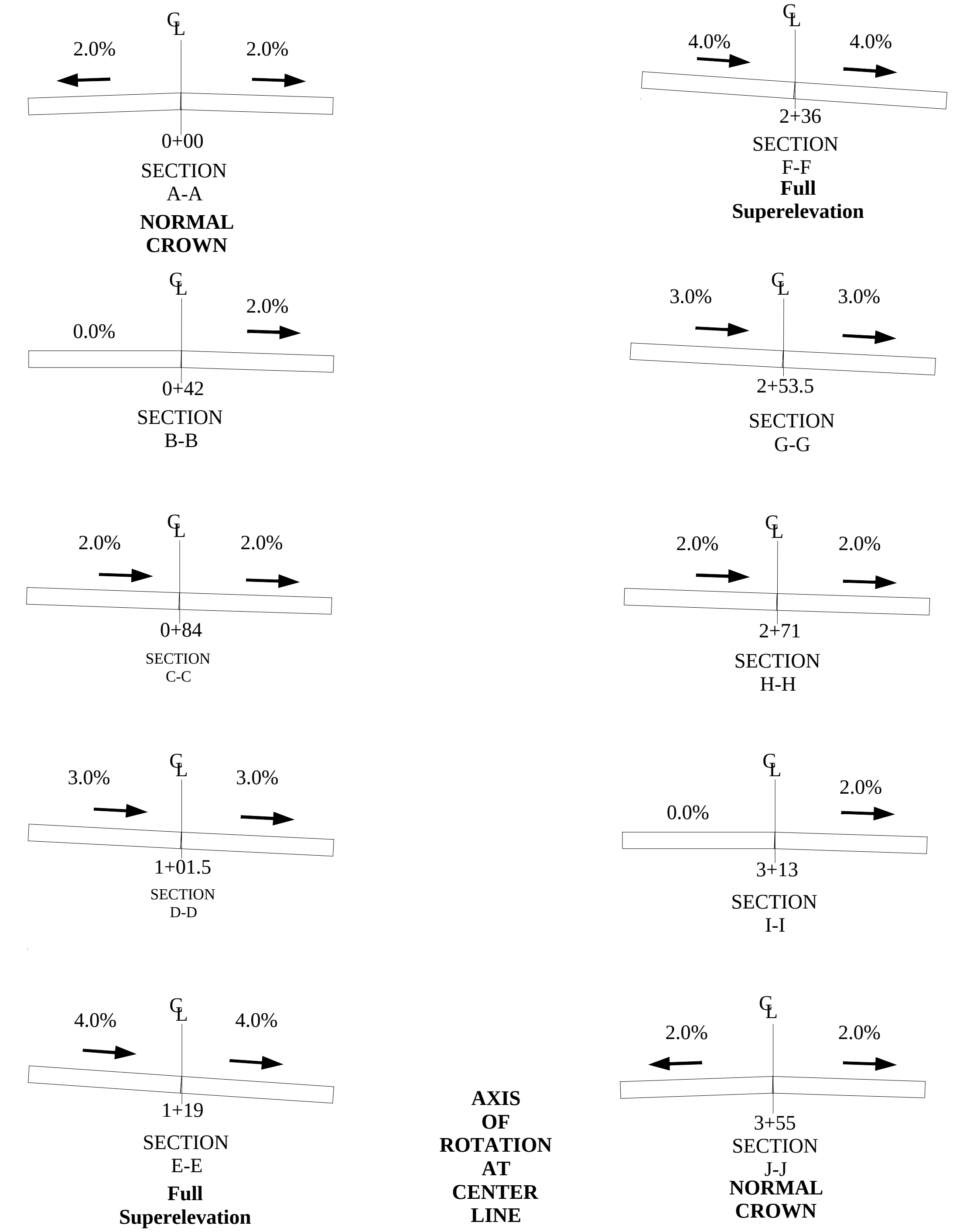
ROADWAY LAYOUT



ROADWAY PAVEMENT DESIGN



ROADWAY SUPERELEVATION AND ROTATION DETAIL



NO.	BY	Date	REVISION DESCRIPTION

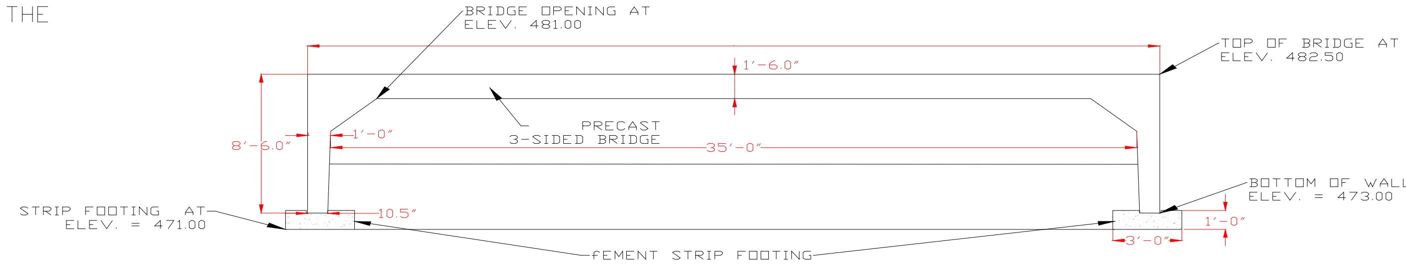
DRAWN | DESIGNED | CHECKED
APPROVED | DATE | PROJECT

Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

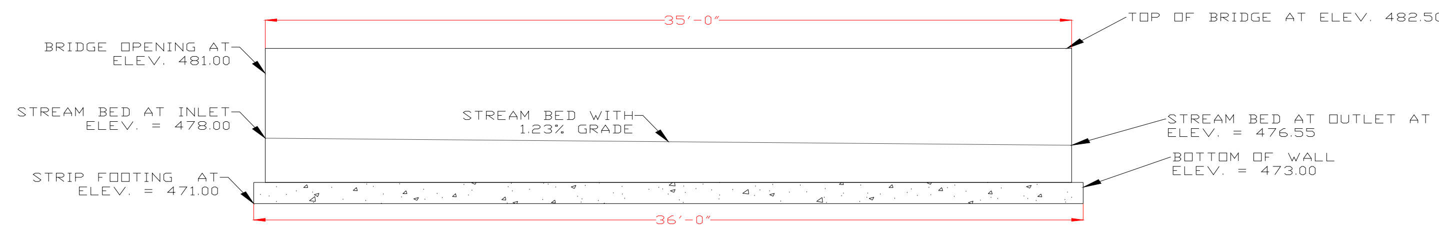
Road Layout with Elevation and Superelevation Detail

SHEET 4 OF 9

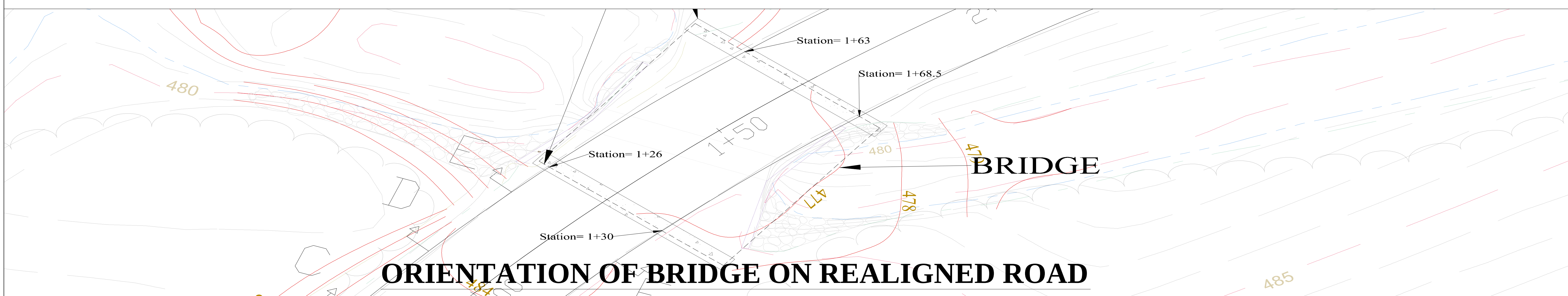
CONSTRUCTION NOTE: ORIENT THE BRIDGE CORNERS TO THE STATIONS SPECIFIED. EACH WALL SHOULD BE PLACED INTO THE COLD JOINTS OF THE STRIP FOOTINGS .PLACE THE FIVE 35'BY7' SEGMENTS TOGETHER TO CREATE THE STRUCTURE DEPICTED BELOW.



PROFILE VIEW OF BRIDGE



SIDE VIEW OF BRIDGE WITH STREAMBED GRADE



ORIENTATION OF BRIDGE ON REALIGNED ROAD

NO.	BY	Date	REVISION DESCRIPTION

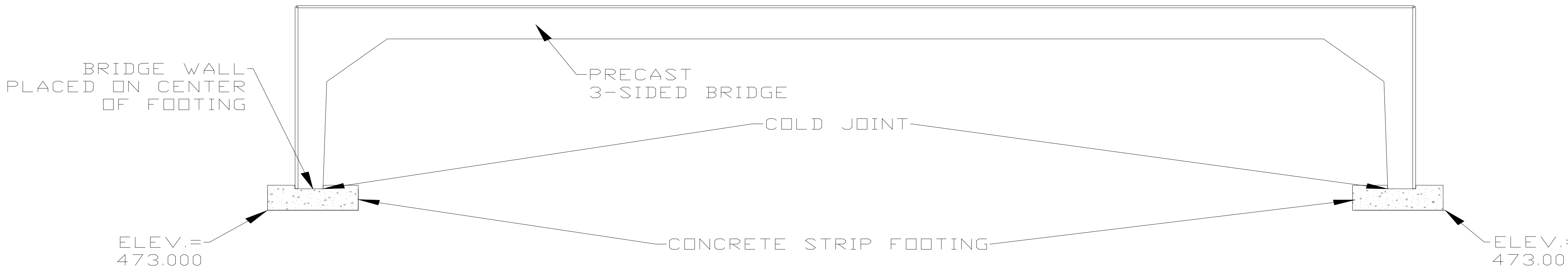
DRAWN DESIGNED CHECKED
APPROVED DATE PROJECT

Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

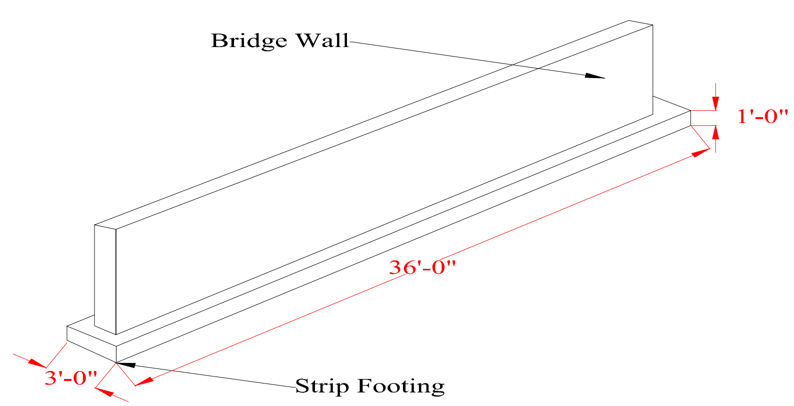
Bridge Dimension, Orientation, and Placement Detail

SHEET 5 OF 9

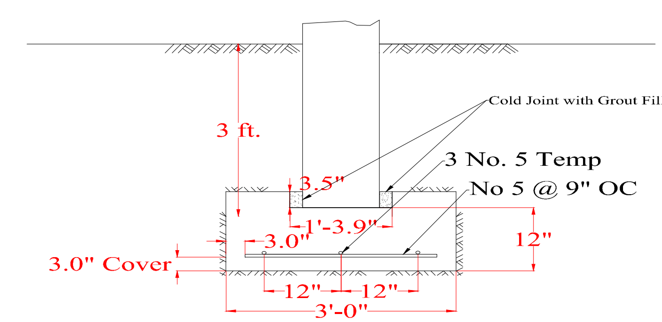
CONSTRUCTION NOTE: EXCAVATE TO BEDROCK. USE FLOWABLE FILL TO REACH ELEVATION 473. POUR FOOTINGS WITH COLD JOINT AND REINFORCEMENT. LEVEL COLD JOINT WITH HIGH STRENGTH GROUT. WHEN BRIDGE IS TO BE SET. PLACE EACH WALL IN COLD JOINT AND FILL IN WITH CEMENT GROUT.



STRIP FOOTING LOCATIONS AND ELEVATIONS



STRIP FOOTING DIMENSIONS



STRIP FOOTING REINFORCEMENT

NO.	BY	Date	REVISION DESCRIPTION

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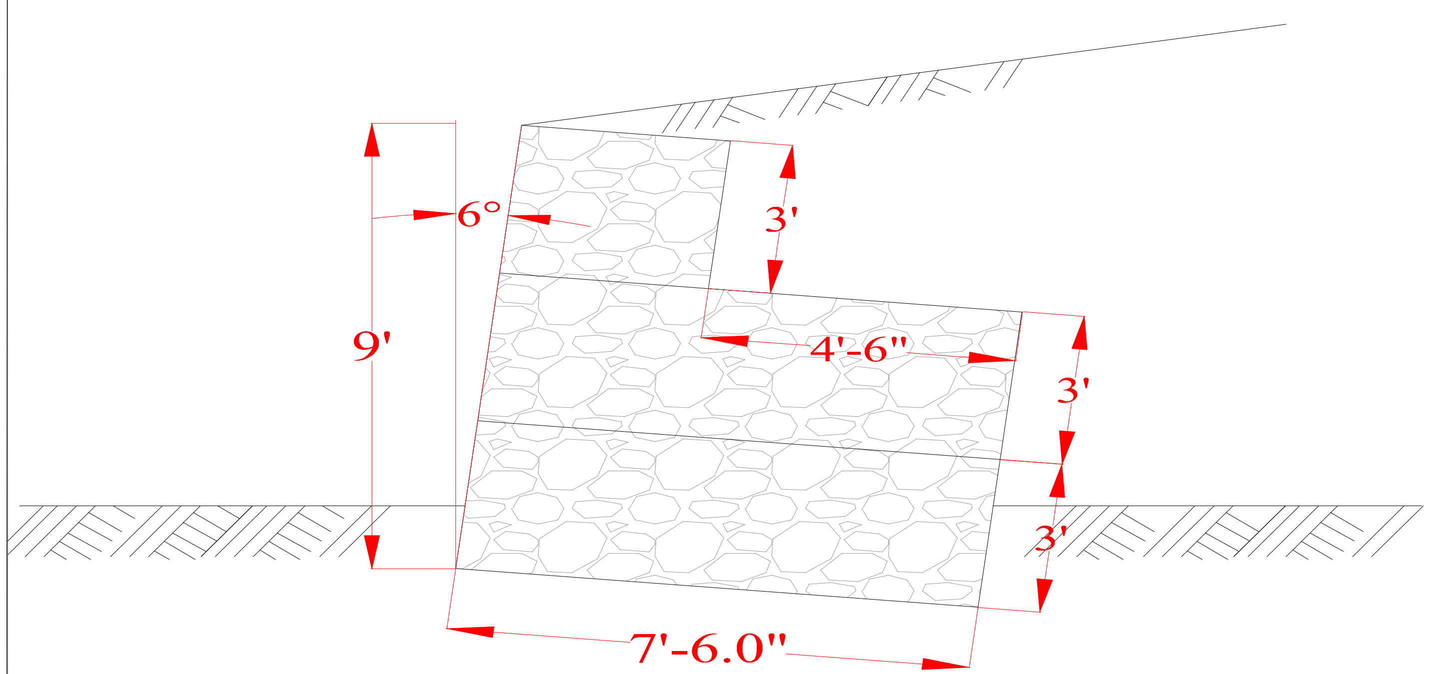
APPROVED | DATE | PROJECT

Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

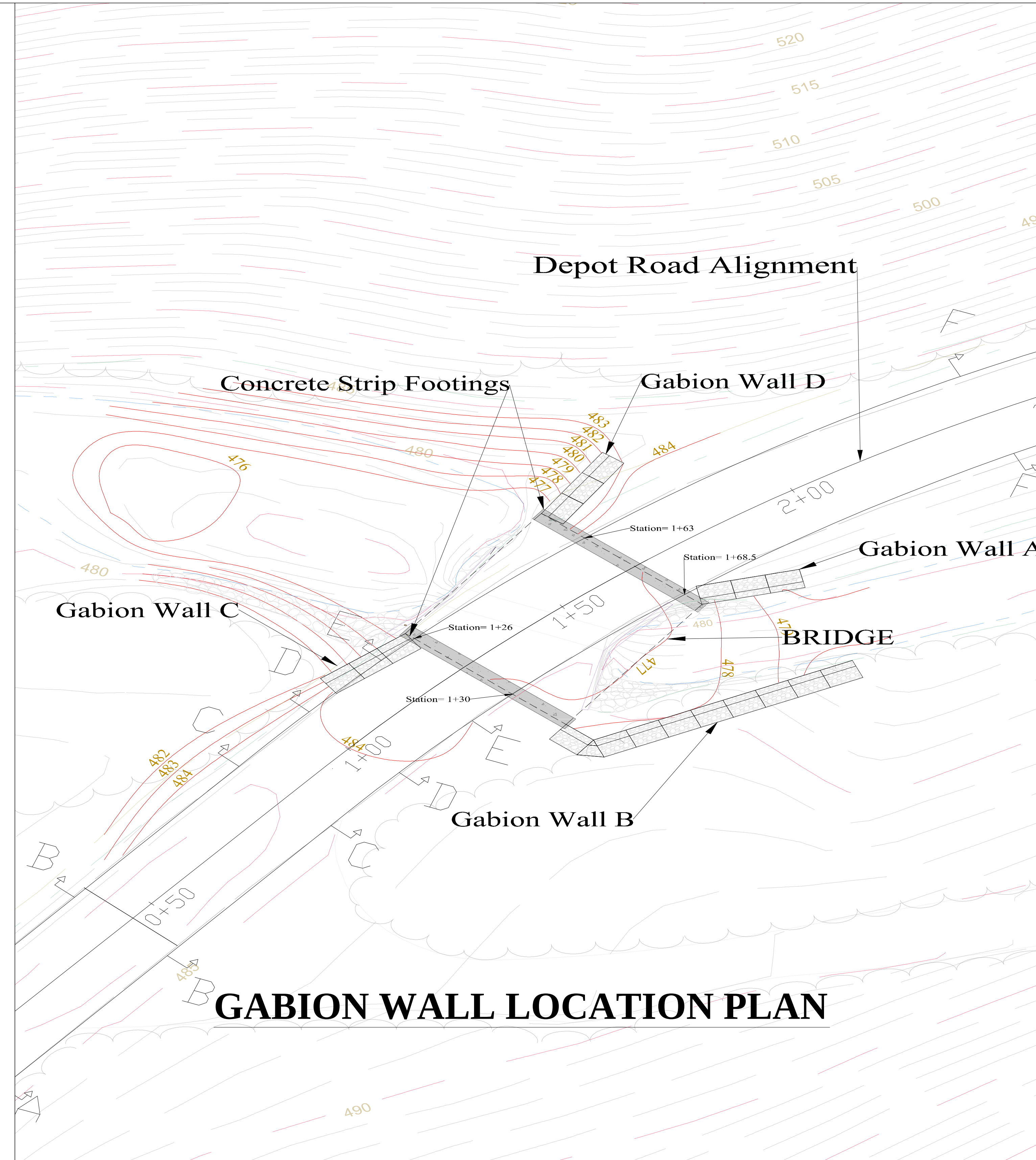
Strip Footing Location, Dimension, and Reinforcement Detail

SHEET 6 OF 9

CONSTRUCTION NOTE: THE GABION WALLS SHALL BE CONSTRUCTED IN 6' SEGMENTS, AND ORIENTED WITH A STEPPED BACK FACE AS SHOWN BELOW.



GABION GRAVITY WALL DESIGN: STEPPED BACK FACE



GABION WALL LOCATION PLAN

NO.	BY	Date	REVISION DESCRIPTION

DRAWN | DESIGNED | CHECKED
APPROVED | DATE | PROJECT

Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

Retaining Wall Design and Location Detail

SHEET 7 OF 9

Date Started: 12/14/2012
Date Finished: 12/14/2012
Client: HyGround Engineering
Driller: Frank Herrington
Drilling Co.: Dave Robeau
Drill Rig: Mobil B53

Team HyGround
Smith College
Northampton, MA 01063

Project: Depot Road Culvert
Location: Williamsburg, MA
Inspector: Lindsay Duran

Borehole: TB1
Sheet: 1 of 4
Surface Elevation: Est. 530'
Sampler: 2" OD SS/HSA
Hammer Wt.-Fall: 140#-30" (Automatic)

In-Situ Test & Instrume	RQD %	Run-Rec (in.)	Spoon Blows	Bottom Depth of Sample	Depth (ft.)	Description	Remarks
			5			Dark brown, well graded sand with gravel, moist	
			7				
		Rec-15	5	1.5			
			7				
			12				
		Rec-16	8	3	3		
			6				
			7				
		Rec-12	9	4.5			
					5		
			38				
			62				
		Rec-6	29	6.5			GW 5.0'
					7		@ 0 hrs.
			4			Dark grey-brown, well graded sand with gravel,	
			12				
		Rec-12	33	8.5			
					10		
			18			Brown, well graded sand with gravel, wet, some	
			69				
		Rec-15	97	11.5			
				12	12		Auger Sample @ 12'

Date Started: 12/14/2012
Date Finished: 12/14/2012
Client: HyGround Engineering
Driller: Frank Herrington
Drilling Co.: Dave Robeau
Drill Rig: Mobil B53

Team HyGround
Smith College
Northampton, MA 01063

Project: Depot Road Culvert
Location: Williamsburg, MA
Inspector: Lindsay Duran

Borehole: TB2
Sheet: 2 of 4
Surface Elevation: Est. 530'
Sampler: 2" OD SS/HSA
Hammer Wt.-Fall: 140#-30" (Safety)

In-Situ Test & Instrume	RQD %	Run-Rec (in.)	Spoon Blows	Bottom Depth of Sample	Depth (ft.)	Description	Remarks
						Brown,well graded sand with gravel, moist	
			14				
			15				
		Rec-18	7	2	2		
			17				
			8				
		Rec-3	8	3.5			
					5		
			2			Dark grey-brown, clayey sand, wet	
			2				
		Rec-20	2	6.5			GW 5.0'
						Brown, well graded sand with gravel, wet	@ 0 hrs.
			4				
			9				Auger
		Rec-6	14	8.5	8.5		Sample
							@
							10'
				10	10		

NO.	BY	Date	REVISION DESCRIPTION

DRAWN DESIGNED CHECKED
APPROVED DATE PROJECT

Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

Depot Rd. Culvert Replacement Bore Hole Logs B1 and B2

SHEET 8 OF 9

Date Started: 12/14/2012	**Team HyGround**	Borehole: TB3
Date Finished: 12/14/2012	**Smith College**	Sheet: 3 of 4
Client: HyGround Engineering	**Northampton, MA 01063**	Surface Elevation: Est. 530'
Driller: Frank Herrington	Project: Depot Road Culvert	Sampler: 2" OD SS/HSA
Drilling Co.: Dave Robeau	Location: Williamsburg, MA	Hammer Wt.-Fall: 140#-30" (Safety)
Drill Rig: Mobil B53	Inspector: Lindsay Duran	

In-Situ Test & Instrume	RQD %	Run-Rec (in.)	Spoon Blows	Bottom Depth of Sample	Depth (ft.)	Description	Remarks
			14			Dark brown, well graded sand with gravel, moist	
			13				
		Rec-6	14	1.5			
			13				
			8				
		Rec-2	7	3.5			
					5		
			2			Dark brown, clayey sand, moist	
			2				
		Rec-6	5	6.5	6.5		GW 5.0'
						Very dark brown, well graded sand with fines, moist	@ 0 hrs.
			6				
			3				
		Rec-12	2	8.5			
					10		
			6			Dark brown clayey sand, wet, some gravel	
			16				
		Rec-8	22	11.5			
					13		
			12			Dark brown, well graded sand with weathered rock, wet	
			103				
		Rec-6	50/0	14.5	14.5		

Date Started: 12/14/2012	**Team HyGround**	Borehole: TB4
Date Finished: 12/14/2012	**Smith College**	Sheet: 4 of 4
Client: HyGround Engineering	**Northampton, MA 01063**	Surface Elevation: Est. 530'
Driller: Frank Herrington	Project: Depot Road Culvert	Sampler: 2" OD SS/HSA
Drilling Co.: Dave Robeau	Location: Williamsburg, MA	Hammer Wt.-Fall: 140#-30" (Safety)
Drill Rig: Mobil B53	Inspector: Lindsay Duran	

In-Situ Test & Instrume	RQD %	Run-Rec (in.)	Spoon Blows	Bottom Depth of Sample	Depth (ft.)	Description	Remarks
						Dark brown, well graded sand with gravel, moist	
			8				
			12				
		Rec-6	15	2			
			12			Brown, well graded sand with gravel, moist	
			12				
		Rec-6	15	3.5	3.5		
						Grey dark brown, well graded sand with gravel, moist	
			8				
			5				
		Rec-9	6	6.5	6.5		
						Grey-brown, Gravelly sand, saturated	GW 5.0'
			12				@ 0 hrs.
			14				
		Rec-9	31	8.5			
			41			Brown, well graded sand with gravel, wet	
			33				
		Rec-6	54	11.5	11.5		
						Brown, well graded sand with gravel and weathered rock, wet.	
			107				5%
							@
		Rec-2		13.5	13.5		last
							sample

NO. BY Date REVISION DESCRIPTION

DRAWN DESIGNED CHECKED

APPROVED DATE PROJECT

Design of Culvert Replacement Structure at Depot Road, Williamsburg MA

Depot Rd. Culvert Replacement Bore Hole Logs B1 and B2

SHEET 9 OF 9

Appendix J: Project Cost Estimate

The project cost estimate is divided into sections for materials pertaining to the structure, materials pertaining to the road, and the cost for labor and cost for equipment needed during construction as shown below in Table J-1. The cost of the 3-sided bridge is the largest expense and the road materials were given by the main suppliers confirmed by the contractor, Bill Turner. The prices for construction equipment are overestimations as there is a 10% contingency included in the total prices to ensure that this project stays within the $250,000 budget. The detailed price breakdown can be found in Table J-1 below.

Table J-1. Costs for Implementation of Replacement Structure and Realigned Road

Item No.	Work / Material	Quantity	Unit	Unit Price ($)	Amount ($)
Structure Materials					
1	3-sided bridge	1	Segment	14,700	$73,500
2	Foundations (concrete)	8	yd^3	112.5	$900
3	Gabion Wall	20	baskets	1,500	$30,000
4	Headway	2	ea.	5000	$10,000
5	Rebar	52	ea.	11.73	$610
Road Materials					
6	Gravel	250	Tons	8.5	$2,125
7	Hot Mix (Top course)	41	Tons	80	$3,280
8	Hot Mix (Sub-base)	68	Tons	80	$5,440
9	Speed Sign	1	Sign	100	$100
10	Guardrails	80	ft	20	$1600
11	Guardrail anchors	4	piece	50	$200
Labor					
12	Worker (4)	320/worker	Man-hours	22/hr	$28,160
Construction Equipment					
13	Excavator	2	month	6000/month	$12,000
14	Silt Fencing	800	ft.	5	$4,000
15	Silt Curtain	80	ft.	30	$240
16	Diversion Pipe	80	ft.	1	$80
17	Water pump	1	ea.	800	$800
18	Dewatering bag (15' x 30')	1	ea.	200	$200
19	Flowable Fill	3	yd^3	40	$120
			Subtotal		$173,355
			10% Contingency (of Subtotal)		$17,336
			Total< $200,000		
			Budget		$250,000

Appendix K: Project Expenses

Team HyGround spent a total of $475 throughout the year for purchasing gifts, printing, traveling, and meals. The cost of each can be found below in Table K-1.

Table K-1. Project Expenses

Category	Justification	Expected Cost
Supplies	The team does not need any extra supplies other than ones that are already provided by the engineering department	$0
Gifts	The team purchased gifts for the liaisons and anyone else who has worked closely with the team.	$50
Printing/Copying	The team used $100 for printing each semester. The materials that needed to be printed include weekly progress reports, agendas, proposals, and other materials for the project.	$200
Travel	The team used Zipcar to visit the site twice throughout the year. Each visit was around 5 hours at a rate of $7/hr. The Zipcar membership is $25/year. The team also once took a taxi to get to and from the site for $60.	$155
Meals	The team had a meal with the liaisons before the end of the school year (5 person meal).	$120
Equipment	No equipment or software needed to be purchased.	$0
Total		$475

Appendix L: Gantt Chart

	WBS	Quarter 3, 2012 – Quarter 2, 2013 (September – May)
1	1	Background Research
2	2	Culvert research
3	3	Figure out type of structure needed
4	4	Research Structures in Close Proximity of Project Site
5	5	Road research
6	6	Road designs
7	7	Structure & road designs
8	8	Conceptual Design Options
9	9	Soil Analysis
10	10	Hydrologic Analysis (TR-55)
11	11	Hydraulic Analysis
12	12	Design Development
13	13	Detail Design
14	14	Final Delivera
15	14.1	Construction Plans
16	14.2	Design Packa
17	15	Class
18	15.1	Design Reviews
19	15.1.1	Design Review 1
20	15.1.2	Design Review 2
21	15.1.3	Design Review 3
22	15.2	Project Proposal
23	15.3	Mid Year Report & Presentation
24	15.4	Final Report
25	15.5	Project Poster
26	15.6	Final V
27	15.7	End of Year

WBS	Task
1	**Background Research**
1.1	Research history & purpose of culvert
1.2	**Site Research/Specifications**
1.2.1	Review material provided on existing culvert location
1.2.2	Identify Questions about site
1.2.3	Site Visit
2	**Culvert research**
2.1	Types of culverts
2.2	Research culvert shapes
2.3	Culvert materials
2.4	End treatments
2.5	**Compile preliminary costs**
2.5.1	Obtain Pricing for a Bridge
2.5.2	Obtain Pricing for Various Size Culverts
2.5.3	Gather Pricing for Road
2.6	Research pros and cons of shapes and materials
2.7	Factors affecting culvert performance
3	**Figure out type of structure needed**
3.1	Identify Equation
3.2	Find variables required for equation
3.3	Find needed area of opening
3.3	Obtain variables through site visit
3.4	Identify site constraints
4	**Research Structures in Close Proximity of Project Site**
4.1	Site Visit
4.2	Draft Letter to Private Bridge Owner
4.3	Ask Dave Foulis (DEP) about Rt 9 Culvert
5	**Road research**
5.1	Identify road design regulations
5.2	Research into pavement layers
5.3	Information on pavement design
6	**Road designs**
6.1	**Come up with road placement options**
6.1.1	Model Road Alignments on AutoCAD
6.1.2	Create an accurate placement of the existing road
6.1.3	Create Updated Road Alignments
6.2	**Look into road design given feedback from Jim**
6.2.1	Obtain information of radius of cuvature and factors of road design
6.2.1.1	Meet With Prof. Knodler to Discuss Road Design
6.2.1.2	Read Hickerson Resouce on Road Design and Summarze Key Facts
6.3	Site visit
7	**Structure & road designs**
7.1	Improve and finalize preliminary road alignment designs

WBS	Task
7.2	Create Preliminary Culvert Alignments
7.3	Narrow structure options down and create designs
7.4	Cost analysis for road and structure designs
8	**Conceptual Design Options**
8.1	Creat Comparison of refined designs or culvert
8.2	Create refined designs of the road
8.3	Create a proposal for liaisons
8.4	Present conceptual designs to liaisons
9	**Soil Analysis**
9.1	Soil Borings
9.2	Soil Testing
9.3	Classify Soil
9.4	Foundation Research & Calculations
9.5	Foundation Design
10	**Hydrologic Analysis (TR-55)**
10.1	Delineate Watershed
10.2	Find Soil types and Areas
10.3	Find Curve Number
10.4	Find Time of Concentration
10.5	Run and Compare Results
11	**Hydraulic Analysis**
11.1	**Compile Components**
11.1.1	Research Hydraulic Analysis
11.1.2	Research Into Relationship to Culverts
11.1.3	Research into Manning Equation for Open Channel Flow
11.2	**Collaborate with NRCS team**
11.2.1	Reach out to NRCS and Student Team
11.2.2	Meet with NRCS to refresh on HEC-RAS
11.3	**Perform Cacluations**
11.3.1	Perform Rough Calculations to Obtain Size of Opening
11.4	**HEC-RAS**
11.4.1	Set Up HEC-RAS
11.4.2	Run Different Models on HEC-RAS
12	**Design Development**
12.1	Finalize Design Components
12.2	Design Proposal (Meeting with Jim)
13	**Detail Design**
13.1	Foundation Specifications
13.2	Culvert Dimensions & Orientation
13.3	Wingwall Specs
13.4	Construction Staging Specs
14	**Final Deliverables**
14.1	**Construction Plans**

WBS	Task
14.1.1	Create First Plans (Road)
14.1.2	Continue Road Construction Plans
14.1.3	Construction Plans for Final Design
14.2	Design Package with Cost Estimate & Executive Report
15	**Class Documentation**
15.1	**Design Reviews**
15.1.1	Design Review 1
15.1.2	Design Review 2
15.1.3	Design Review 3
15.2	**Project Proposal**
15.2.1	Review criteria
15.2.1.1	Initial Proposal
15.2.1.2	Create outline and assign parts
15.2.1.3	Create Project Proposal
15.2.1.4	Project Proposal Presentation
15.2.2	Revise Project Proposal
15.3	**Mid Year Report & Presentation**
15.3.1	Mid Year Report Breakdown
15.3.2	Create and practice presentation
15.3.3	Compile information and write report
15.3.4	Hand in Mid-Year Report
15.4	**Final Report**
15.4.1	Write Individual Parts
15.4.2	Compile And Submit Draft
15.4.3	Edit and Submit Final Version
15.5	**Project Poster**
15.5.1	Create Poster
15.5.2	In Class Poster Review
15.5.3	Collaborations (Sat - 4/20)
15.6	**Final Video**
15.6.1	In Class Video Presentation
15.6.2	Film and Edit Video
15.7	**End of Year Presentation**
15.7.1	Create and Practice Presentation
15.7.2	Design Celebration