# Sackett Harbor Bridge Design

A Project Report for

A Major Qualifying Project
To be submitted to the faculty of
Worcester Polytechnic Institute
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Degree of Bachelor of Science

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## Table of Contents


Table of Contents ..... 2
List of Figures ..... 5
List of Tables ..... 6
ABSTRACT ..... 7
INTRODUCTION ..... 8
BACKGROUND ..... 10
AASHTO LRFD Bridge Specification ..... 10
Loading ..... 10
Deck System ..... 11
Railings ..... 12
Massachusetts Bridge Manual ..... 12
Design Constraints ..... 13
Cost ..... 13
Constructability ..... 14
Using Local Materials ..... 14
SITE ASSESMENT ..... 15
Site Survey ..... 15
Soil and Material Testing ..... 16
Existing Conditions ..... 16
Existing Bridge Assessment ..... 17
Assessment Findings ..... 17
Decay ..... 18
Summary of Bridge Inspection ..... 20
Scour Analysis ..... 21
Hydraulic Analysis ..... 21
Hydrologic Data ..... 22
DESIGN ..... 24
Bridge Design ..... 24
Structural Design ..... 24
Structural Design Options ..... 25
Preliminary Bridge Design ..... 26
Bridge Span ..... 26

Simple Span Design ........ 26
Truss Design ........ 28
Covered Bridge ........ 28
Final Bridge Design Options ........ 28
Decking ........ 29
Connections ........ 30
Simple Span ........ 30
Spanning Members ........ 31
Railing ........ 32
Truss ........ 32
Cost Estimate ........ 34
Bridge to Foundation Transition ........ 36
Constructability ........ 36
Foundation Design ........ 36
Functions ........ 36
Determination of Design Parameters ........ 37
Loading ........ 37
Geotechnical Analysis ........ 37
Slope Stability ........ 38
Retaining Wall Foundation ........ 39
Design ........ 39
Footing External Stability ........ 40
Footing Internal Stability ........ 41
Stem Internal Stability ........ 41
Overall External Stability ........ 41
Constructability ........ 42
Cost Estimate ........ 43
Concrete Pour Foundation ........ 43
External Stability ........ 43
Constructability ........ 44
Cost Estimate ........ 45
Weaknesses ........ 45
Slope Runoff Diversion Design ........ 46
Considerations ........ 46
Possible solutions ........ 46
Waterbar ........ 47
Construction Materials ........ 49
Impact on the immediate area ........ 50
PROJECT IMPLEMENTATION ........ 51
Permitting ........ 51

Cost Analysis ........ 52
Construction Scheduling ........ 52
RECOMMENDATIONS ........ 54
Structural Design ........ 54
Foundation Recommendations ........ 55
APPENDIX A: Profile of The Trustees of Reservations ........ 56
APPENDIX B: Map of Brooks Woodland Preserve ........ 57
APPENDIX C: Map of Watershed Area ........ 58
APPENDIX D: Hydrologic Calculations ........ 59
SMADA 6.0 for Windows: Watershed Information ........ 59
SMADA 6.0 for Windows: Time of Concentration Calculation Methods ........ 59
APPENDIX E: Notice of Intent Procedure ........ 61
Section A: General Information ........ 61
Section B: Resource Area Effects ........ 61
Section D: Other Applicable Standards and Requirements ........ 61
NOI Wetland Fee Transmittal Form ........ 62
Stormwater Management Form ........ 62
Section A: Property Information ........ 62
Section B: Stormwater Management Standards ........ 62
APPENDIX F: Preliminary Designs ........ 64
APPENDIX G: Simple Span Design ........ 66
APPENDIX H: Railing Design ........ 67
APPENDIX I: Howe Truss Member Forces ........ 69
APPENDIX J: Truss Connections ........ 70
APPENDIX K: Decking Connections ........ 74
APPENDIX L: Soil Analysis Tables ........ 75
APPENDIX M: Retaining Wall Design ........ 76
REFERENCES ........ 84

## List of Figures

Figure 1: Beaver Activity Upstream of Bridge ........ 15
Figure 2: Excessive deflection due to overloading of remaining beams (note presence of temporary structure) ........ 18
Figure 3: Rotting of entire beam length ........ 19
Figure 4: Failure on compression due to decay ........ 19
Figure 5: Decay most evident at contact with soil and water ........ 20
Figure 6: Contour Image of Project Watershed Area ........ 22
Figure 7: Rainfall Hyetograph 25yr, 24hr Storm ........ 23
Figure 8: Hydrograph showing flow from 25-yr 24-hr storm event ........ 23
Figure 9: Round Timbers ........ 27
Figure 10: Glulam ........ 27
Figure 11: Example of Howe Truss ........ 28
Figure 12: Example of Covered Bridge ........ 28
Figure 13: Example of Unused Design Cross Section ........ 29
Figure 14: Cross Section of Simple Span Design ........ 31
Figure 15: Howe Truss Evaluation in Risa-2D ........ 33
Figure 16: Truss Top Chord Connection ........ 33
Figure 17: Truss Bottom Chord Connection ........ 34
Figure 18: Sieve Analysis Semi-Log, well-graded gravelly sand with silt ........ 38
Figure 19: Foundation Wall Cross Section ........ 42
Figure 20: Cross Section of Massive Gravity Foundation ........ 45
Figure 21: Log Waterbar Plan View ........ 47
Figure 22: Log Waterbar Cross Section ........ 48
Figure 23: Plank, Rubber, Stone Waterbar Plan View ........ 48
Figure 24: Plank, Rubber, Stone Waterbar Cross Section ........ 49

## List of Tables

Table 1: Cost Estimate for Truss Design ........................................................................ 35
Table 2: Cost Estimate for Simple Span .......................................................................... 35
Table 3: Cost Estimate of Retaining Wall Foundation ................................................... 43
Table 4: Cost Estimate of Concrete Pour Foundation ................................................... 45
Table 5: Shear in Wood from Connections...................................................................... 73

## ABSTRACT

The purpose of this project was to design and analyze alternative bridge designs for a property on the East Branch of the Swift River for The Trustees of Reservations, located in Petersham, MA. Recommendations include design of wood bridge structure, foundation design and slope stability, environmental impact, constructability guidelines and permitting. Results of the design and analysis processes are outlined and recommendations for construction are given based on conclusions drawn from the design options. The final recommendations can be followed to complete the project on location.

# INTRODUCTION

The Northeast is a place of great cultural and environmental diversity.  With a dense regional population, it is important to preserve portions of the land for the enjoyment of the people now and for future generations.

The Trustees of Reservations (TTOR) is a non-profit conservation organization that was formed in 1891 as a regional land trust.  Their organization is devoted to preserving pieces of land that exhibit exceptional scenic, historic, and ecological characteristics; especially those that comprise Massachusetts' unique landscape and culture (see Appendix A).

The Brooks Woodland Preserve (see Appendix B) in Petersham, Massachusetts consists of 558 acres, currently accessed by walking paths and a trail road.  The upkeep of this trail system is vital to the fulfillment of the mission of the Trustees.  The ability of the public to fully enjoy the area hinges on regular trail cleanup of blow downs and debris, and maintained access to the preserve by way of the trails and trail roads.

A portion of the trail road follows the East Branch of the Swift River through the property.  This road crosses the river at a point about 4000 feet into the conservation.  The bridge currently in place here has failed recently, and the Trustees are searching for a replacement to regain full access to the property.  A temporary bridge has been constructed over the failed structure; however this is seen as a small step towards future improvement and reconstruction of the site.

The goals of this project are to find a best fit solution to the failed bridge and provide the Trustees with a set of recommendations with multiple options for the design and construction of a replacement bridge.
Specific objectives include:

- Conduct an analysis of the site and structure and determine contributing causes of bridge failure.

- Design and analyze alternative replacement structures.
- Suggest a designed solution within constraints (site, construction, labor, cost, materials, environmental impact).
- Determine the legal permits required to construct the structure, and develop a procedure for procuring the permits.
- Develop a cost estimate of the proposed alternatives.
- Develop a construction schedule.

# BACKGROUND

Before the design process starts information on the project requirements, constraints, and building or structural limitations should be researched. Some of this information can be gathered through site visits and in design manuals. This information includes but is not limited to the span, anticipated use, environmental effect, and existing conditions of the site. In the case of the Sackett's Harbor Bridge, the span is 22 feet and crosses the Swift River at a width of about 12 feet. The anticipated use is that it will be a pedestrian bridge, occasionally used as an access road for maintenance vehicles. In addition to designing a structurally sound bridge it is necessary for it to impact the environment as little as possible. The environmental impact from this project should be minimal because there is an existing bridge which failed, and current foundations that, while requiring upkeep, should be able to fit into the new design. Knowledge of the site can be used in addition to manuals, codes and specifications to determine the requirements of the bridge and for environmental solutions.

## *AASHTO LRFD Bridge Specification*

One important resource published by the American Association of State Highway and Transportation Officials is the AASHTO LRFD Bridge Specification (American Association of State Highway and Transportation Officials, 1998). This describes all of the technical requirements of bridge design. This focuses on large scale bridges on main roads but gives information for projects of a smaller magnitude. This covers everything to do with bridge design from design loads to various building materials to foundation design.

### Loading

In order to design a bridge to be structurally sound it is necessary to know what kind of loading it will have to withstand. In the case of the Sackett's Harbor Bridge the design is to be just pedestrian loads (85 pounds per square foot) or static vehicle loads in addition to the weight of the structure. This is because in the AASHTO LRFD Bridge Specification it is stated that "where sidewalks, pedestrian, and/or bicycle bridges are intended to be used by maintenance and/or other incidental vehicles, these loads shall not be considered in the design. The dynamic load allowance need not be considered for these vehicles." (AASHTO, 3.6.1.6) This is the case because the vehicles are traveling at a slow speed and there is not enough room on the bridge for both a vehicle and pedestrians.

These loads are not the only forces that a bridge must withstand. It must be able to withstand lateral loads which are caused by wind and earthquakes. The loading caused by wind is a function of the height of the structure, the friction of the structure, and the location and therefore the velocity of the wind. Similarly the force from an earthquake is dependent on the height and magnitude of the earthquake as well as the weight of the bridge. (AASHTO, 3.8 & 3.10)

The other forces that the bridge abutments will have to withstand are from the stream that is flowing under the span. These forces are the static pressure of water, stream pressure and forces caused by ice. The static pressure is dependent on the height of water that the abutment is exposed to. The greater the height of water exposed to the greater the force. The last force is caused by ice flowing down stream and hitting the abutments. The thickness and strength of the ice affects the forces that the abutments must withstand. (AASHTO, 3.7 & 3.9)

## Deck System

The decking of bridges can be made from a variety of building materials with each variety having its strengths and weaknesses. One of the most common deck systems is concrete because it is very strong in compression and it can be made to be any size and shape. The negative aspect is that is lacks strength in tension which would require reinforcement. This material would not work well in this specific aspect because it would be extremely difficult to get a substantial amount to the site. In addition this material would not maintain the rustic appeal that is desired. Another material that is commonly used for decking is steel. This is because steel is strong in both compression and tension. The drawback from using steel in the decking is that it loses strength when the forces applied to it change from tension to compression called fatigue failure. This bridge will not be exposed to the number of cycles when fatigue failure will influence the strength of the decking. Also the bridge is located in an environment where it will constantly be subjected to moisture which will corrode which will reduce the strength of the material. The third major construction material is wood. This material is popular because it is strong in both compression and tension while being a lighter construction material. It does not diminish in strength when subjected to alternating compression and tension. The downside of using wood is that it does not last as long as other materials if the environment is not favorable. These conditions include cycles of wetting and drying or exposure to fungus or other decay mechanisms.

Many specifications for a wooden deck system are defined in the AASHTO LRFD Bridge Specification. One of these is that the minimum nominal thickness of the planks is four inches. The existing decking on the failed bridge meets this criterion so it can be used again if it is still structurally sound. Another specification is that each plank shall be nailed to each support with two nails with a minimum length of twice the plank thickness. (AASHTO, 9.9.2 & 9.9.7.2)

### Railings

In addition to providing a way to cross an obstacle a bridge must ensure that the users are safe which is why railings are necessary. There are different specifications for pedestrian and bicycle bridges but because both will be used it is necessary to design for the worst case scenario. This means that even though the minimum height for a pedestrian bridge is 42 inches it is necessary to use the minimum height of 54 inches for a bicycle. In addition to the height requirements these railings must be able to support people in case they fall and need the railing for support. It must be able to support a force of 50 pounds per linear foot both horizontally and vertically as well as a concentrated load of 200 pounds. (AASHTO, 13.8.1, 13.8.2, & 13.9.2)

## *Massachusetts Bridge Manual*

The specifications that are provided by the Massachusetts Bridge Manual are based upon the AASHTO Bridge Specification (Massachusetts Highway Department, 2005). This manual also provides details of what needs to be surveyed around the structure. The details that need to be obtained from the survey are the elevations of the land around the site, elevations at both abutments, minimum clearance under bridge, and the elevations of the river if necessary. (Mass Highway, 1.1 & 3.1.1.1) The other information that is included in the AASHTO Bridge Specification is not applicable and includes highway design and other large scale specifications.

## *Design Constraints*

In addition to the design challenges, the bridge must be constructed on site due to the limited access to the site. The site is located in the middle of the woods with an access road that is just wide enough for a pickup truck to pass through. In addition to having no access to large machinery the site has no utilities including electricity. This means that designing the bridge in such a way that it is easy to construct is a large concern for the Trustees of Reservation. Another issue that proves why constructability is a huge issue on this project is that it will be built by a group of volunteers. These people will not be experts in construction and if the design is simple it will be easier to be built.

In order for the bridge to be able to sit on a solid surface one of the abutments must be rebuilt or the span must be increased to around forty feet. The large stones from the failed abutment would require some large machine to move and rebuild. A larger span brings up an additional problem in that the decking is going to be reused and additional decking will be required. The additional span will cause larger members to be necessary. One solution that could be optimized is to use salvaged steel members.

Reusing materials like steel beams, as well as decking material will help the Trustees to achieve a goal of a small environmental footprint. This goal is influenced by the nature of the Trustees of Reservations' organization. The project is located in an ecologically sensitive area, near wetlands and high animal activity. This can be seen through the newly constructed beaver dam that is just upstream of the project.

## *Cost*

One of the major factors to any project is the cost of materials and the cost to construct. The two designs that will be analyzed in detail will vary in cost due to the differences in designs. The simple span will accumulate its cost because of the beams spanning the abutments and the truss will require more members.

## *Constructability*

It is necessary for the structure to be able to be constructed easily and in an area which normal constructions techniques will be difficult.  The first problem that must be addressed is how to get the materials to the site.  The bridge is located in the woods and is accessible from dirt roads. This is not a problem for the smaller members but the steel beams and large timbers will be difficult to move down the road.  The simply supported bridge option will be more difficult in this aspect because it may require a greater number of longer members than the truss design.

## *Using Local Materials*

One of the ways to reduce the environmental effect of this project is to use local materials.  One way to do this it to obtain the materials from a local lumber yard that gets the materials locally. T.S. Mann Lumber Co. is the local lumber yard and gets local materials both new and salvaged. Salvaged materials could be used, but their exact structural properties are not known so tests would have to be performed in order to figure out their exact properties.  Another way to obtain local lumber is to utilize the vast number of trees on the property and cut a few down and utilize them as structural members.  Ideally this would work but the structural properties of the wood are dependent on many properties that cannot be determined until the tree is cut down.  Some of these properties are density, presence of decay, slope of grain, knots, and pitch pockets (Faherty & Williamson, 1997).  In addition to this it is not easy to learn how to accurately visually grade lumber.  The Appalachian Hardwood Center offers a three day class as an introduction to lumber inspection.

## SITE ASSESMENT

### *Site Survey*

The location of the project is a very influential part of the bridge design and its constructability. An analysis of the site and surrounding area was therefore a key portion of the pre-design phase.

A survey was conducted of the project site. The purpose was to gather basic data about the site conditions to allow for the proper design and analysis of the bridge, surrounding slopes, and abutments. A surveying level was the primary instrument used in order to gain relative elevations between the adjacent hills, corners of the bridge, and the water level. These elevations then allowed for a number of calculations to take place, including slope of the adjacent Northwesterly hill and a more accurate height of the bridge. A tape measure was used to record distances between elevation points. Given the relative elevations and the distances between them, all changes in elevation, whether on the side of the hill or the corners of the bridge were measured. From this the tilt of the bridge, as well as the slope of the hill were calculated.

**Figure 1: Beaver Activity Upstream of Bridge**

The tape measure was also used during this site visit to measure the water level from the bottom of the slump of the bridge. The bridge is currently exhibiting a slump with the lowest point being about 1/3 of the distance from the Northwesterly side. The change in height was measured with the tape measure between the most slumped point and another point that was closer to level with the entirety of the bridge. This will be compared to the values for the water level taken at an earlier time because a noticeable drop in water level has taken place due to beaver activity (Figure 1).

During the site survey a sample for soil analysis was collected. The purpose of the soil sample was to determine the composition of the soil through a sieve analysis. This analysis determined the constituents of the soil and through this knowledge proper analysis of the stability of the slope as well as of the abutments' retaining wall feature was conducted.

In general, only rough numbers were gathered from the site during this visit. The purpose was to quickly obtain data in order to do preliminary analysis. Further surveying before construction begins is suggested.

## *Soil and Material Testing*

If the construction materials used on this project are recycled and design values are not available, adequate testing must be conducted to determine the necessary design values.

Although the abutment stones are an important structural portion of the bridge, no actual tests will be run on them. The structural capacity of the stones used in the abutments will be determined by noting the type of stone and using known values for regional rocks. Generally accepted values for the type of rocks being used in the abutment will be more than adequate to determine the load bearing capacity of the abutment stones.

In addition to all the structural values of the bridge materials, the constructability of the materials has been analyzed due to many of the constraints of the design. For example, any materials to be transported into the site will require proper lengths to be loaded into the bed of the available truck. This puts limits on the design specifications of these materials. Also, ease of construction is important, as well as weight of individual portions of the bridge. Because man power will be used in the construction process, weight is an important factor in the design and therefore all weights of materials should be discussed in relation to manpower and in conjunction with their importance to the design itself.

## *Existing Conditions*

The existing conditions were determined using information gathered on the site visit, as well as external sources of information, such as MassGIS and Google Earth. Assessments of the site are also discussed in this section.

## Existing Bridge Assessment

An assessment of the bridge based on the Bridge Inspector's Manual had to be conducted to determine the mode of failure of the bridge. The bridge underwent an inspection by the team and digital photos taken were compared to descriptions and diagrams in the Bridge Inspector's Manual. The following is the results of that inspection and the conclusion reached.

### *Assessment Findings*

The current structure is a solid-sawn lumber multi-beam bridge. The most important locations for inspection of this type of bridge is the ends of the beams, for shear inspection; lower half of beams, for tension inspection; ends of beam, for horizontal shear crack inspection; and any exposed surface, especially where in contact with water or soils, to look for decay of members. An inspection for shear damage of the ends of the beams showed that none of the beams had failed or begun to fail under this type of shear. No vertical cracks were noticed, most cracks ran horizontally, as will be described in horizontal shear and decay. The next important location of inspection is the center of the span below the center-line of the beam. This location is where the highest tension is found and therefore any failure in tension can be determined here. Inspection found that there was some cracking in this region. Also, excessive deflection or sagging was inspected and turned out to be quite dramatic, as seen in Figure 2. The remaining structural beams are unable to support the bridge's load alone and have begun to sag under the weight.

**Figure 2: Excessive deflection due to overloading of remaining beams (note presence of temporary structure)**

### ***Decay***

An inspection of decay was conducted, although during the entire inspection process it was obvious that decay played a large part in the bridge failure.  Most of the failures in the above categories were caused by the weakening of members due to decay.  Rotting is most evident where the soil was in contact with the bridge, however, it spanned the entire bridge, with some structural beams rotting to the point of falling off from the decking.

**Figure 3: Rotting of entire beam length**

As a result of the excessive deterioration, many of the beams failed in *compression* at the point where they rest between the deck and the stone abutment, this was not expected as a mode of failure for the wooden beams.

**Figure 4: Failure on compression due to decay**

Causes for the bridge decay were determined through visual inspection of decayed members. This was documented with digital photography.

For timber structures one of the most common forms of decay is insect damage. Evidence of burrowing within the damaged beams is clear, although the exact type of insect damage is unknown. No infestation of insects was noted at time of inspection and damage was likely due to individual insects, rather than infestation from carpenter ants or termites.

The site of most decay was in the "channel" of the abutment, a low spot in the abutment that channels water from the hill above. In this location most of the support beams had deteriorated completely. Further deterioration was noted where soils came in contact with the beams as shown in Figure 5 It is almost certain that water exposure increased the members' susceptibility to insect damages and other forms of decay (Baker, 1969). The decay spread through entire beam when water was able to soak up do to prolonged exposure.

**Figure 5: Decay most evident at contact with soil and water**

### *Summary of Bridge Inspection*

Overall, through inspection, the mode of failure was very clear. Failure was caused by excessive decay resulting from contact of the members with the soil and water. Although the remaining two beams have withstood the decay well (due to not being in as much direct contact with soil) they have begun to fail due to excessive loading.

It is recommended that any future design using timber be kept out of contact with soils. Also, diversion of water flow would protect the bridge and abutment from further decay and erosion. An additional protection measure would be to use pressure treated wood, once again, to prevent decay.

### Scour Analysis

A preliminary investigation into the scour affects caused by the stream on the bridge was conducted. However, due to overall inapplicability of the current methods for scour analysis to a small scale example this investigation was abandoned. The most effective method of determining if scour is acting significantly on the bridge abutment would be through visual inspection of the bridge and the soils in the stream below. However, considering the length of time that the abutment has stood without noticeable removal of soil from below, it is unlikely scour is the cause of failure, or will be a cause of failure, in the future.

## *Hydraulic Analysis*

The watershed surrounding the project location contributes many factors to the bridge project. The amount of rainfall that contacts this area contributes to the amount of water that makes its way over and through the land to the location of the bridge and into the waterway. A map of the watershed can be seen in Appendix C.

The water that infiltrates the soil (infiltrate) contributes to the weight of the soil pushing on the abutments. It affects the daily conditions of the bridge structure and, as already seen in the failed structure, has a great impact on its deterioration. To determine the conditions that the redesigned bridge will need to withstand a hydraulic analysis of the watershed was conducted. This included collecting data of the area from the USGS and Google Earth, as well as gathering field data from a hand survey at the site. Information gathered included the slope of hills in the area, a definition of the watershed, determination of the time of concentration during typical storm events, analysis to determine the amount of infiltration, and the path of the runoff, including the flow. Figure 6 shows a contour image of the project area.

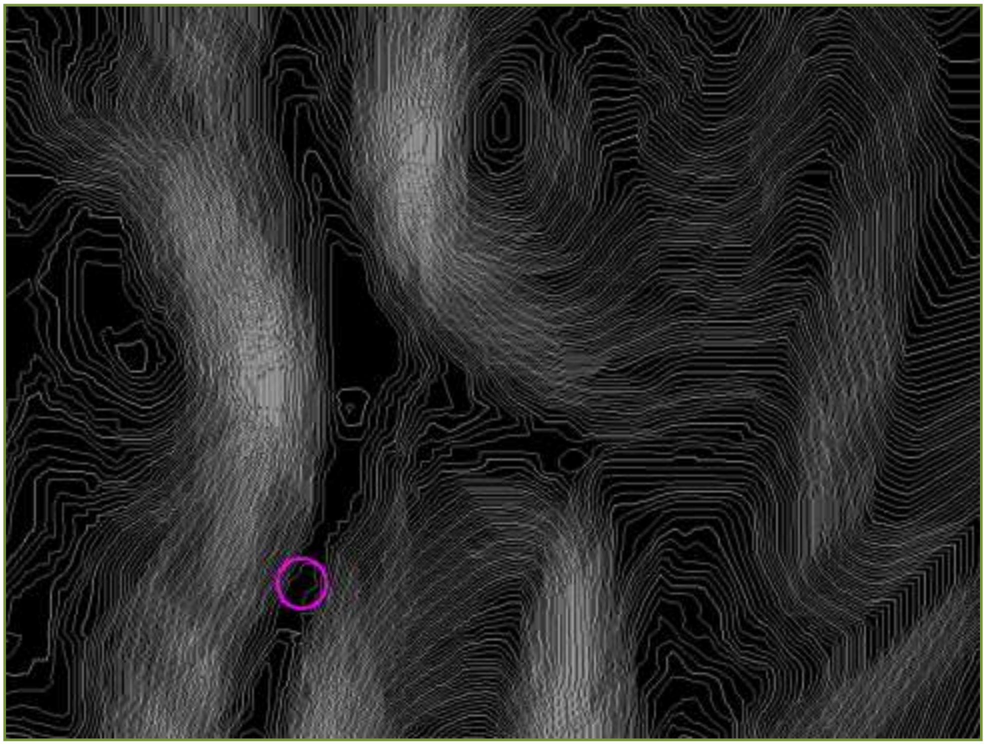


**Figure 6: Contour Image of Project Watershed Area**

The SMADA software package was employed to calculate the hyetographs for rainfall intensity and the hydrograph showing runoff volume during the storm event.  These can be seen in Figures 7 & 8.

Much of this work contributes to the formation of an understanding of the watershed, used in the permitting process.  The data generated also contributed to determining the soil conditions that the abutments are currently withstanding and that the foundation will continue to sustain.

## Hydrologic Data

The rainfall in the project area is a large factor in the stability of the environment.  Runoff from storm events will directly contribute to water levels in the stream, as well as infiltration and runoff in the ground surrounding the site.

A map indicating the area of the watershed can be seen in Appendix C.  All data used in generating calculations can be seen in Appendix D  (Eaglin).

It was found that, during a 24-hour 25-year storm event, the total rainfall is 5.23 inches. The watershed overall was determined to have a time of concentration of 20.3 minutes. The rainfall hyetograph can be seen in Figure 7.

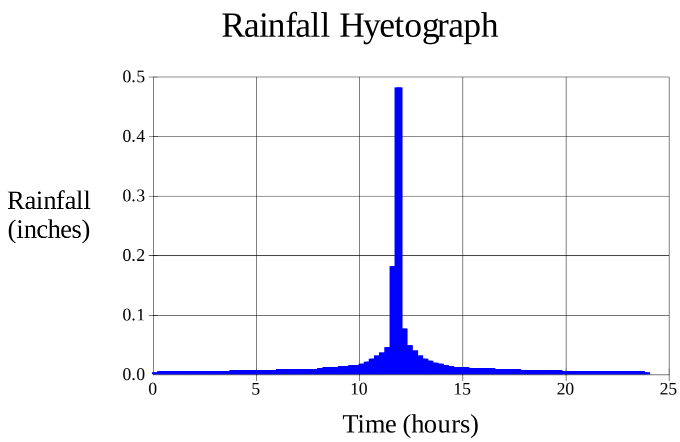



**Figure 7: Rainfall Hyetograph 25yr, 24hr Storm**

The flow, in runoff, generated by the occurrence of such an event can be seen in Figure 8 Using the Santa Barbara method, a maximum flow of 799.2 cfs is calculated. The maximum infiltration rate of 0.273 inches per hour was calculated. This data can be used in calculating effects of storm events on scour and sedimentation, as well as predicting runoff velocities and flows in the project area.

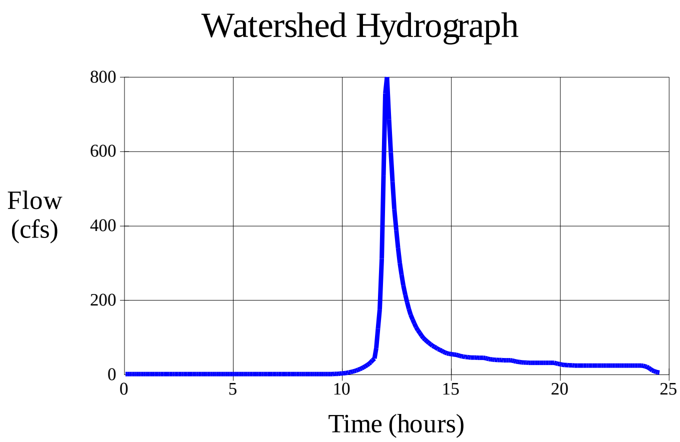



**Figure 8: Hydrograph showing flow from 25-yr 24-hr storm event**

The SMADA software package was employed to calculate the hyetograph for rainfall intensity and the hydrograph showing rainfall volume during the storm event.  This volume is based on all rainfall in the watershed (Appendix C) arriving at one point under the bridge.  The time of concentration is considered in this calculation.  This number is very large, and must be tempered with the knowledge that the majority of water will infiltrate into the ground and continue at much slower velocities, slowing and dispersing its arrival at this point.  Still, this data indicates that the amount of runoff arriving at the project location is significant enough to warrant design considerations.  A solution addressing runoff volume and velocities is included in this project.

# DESIGN

The design portion intends to incorporate the knowledge gained from the site assessment into structural solutions.  These structures consist of several options as bridge designs, foundation designs and solutions to address hydrologic problems.  Each solution has been analyzed for effectiveness and is discussed in the following sections.

## *Bridge Design*

### Structural Design

In order to determine the sizes of the bridge members a structural analysis was conducted.  The first step in this process is to determine the loading that the bridge must withstand.  According to AASHTO Bridge Specification a pedestrian bridge must be able to support a load of 85 psf (American Association of State Highway and Transportation Officials, 1998).  The snow loads that the bridge must withstand are less than the pedestrian loads.  In addition to pedestrian traffic this bridge must be able to support a pickup truck.  Due to the fact that for the probability of both pedestrian and vehicle traffic to be on the bridge at the same time is very low, only the single critical load will be used.   Both the pedestrian and truck loads were evaluated separately for every design because they influence the structure in different ways.  Once the applied loads are known, RISA-2D was utilized to determine the member forces for the various truss designs and moment forces for the simple span designs.  Once these forces are known member sizes can be determined.

For the truss-less design the maximum stress that it will have to withstand is $M \times C/I$. The moment is (M), the distance from the center of the beam to the extreme fibers is (C) and the moment of inertia is (I).

For the truss bridges there are two different calculations which must be performed, one for tension and one for compression. For the members in tension the stress is Force/Area. The maximum force that a member that is in compression which is treated as a short column and follows Eulers buckling equation is $\pi^2 \times E \times I/L^2$. The length (L) is equal to the actual length because the column is pin connected on both ends, (I) is the moment of inertia, and (E) is the modulus of elasticity. The actual stress must be less than the allowable stresses for each sized member multiplied by the adjustment factors. These known values as well as the adjustment factors are found in NDS specifications (American Wood Council, 2005).

## Structural Design Options

Several design options for the bridge were developed including sawn lumber, round timbers, glulam, steel beams, and truss options, to allow for a choice of the best design. One of the first options that must be decided upon is whether there should be a truss design or a simple span. The truss design will use more linear feet of material but each member will not be as thick as those in the simple span. Some other options are to use a truss design under the roadway or a covered bridge. A covered bridge will have a more rustic appeal; however it will be more complex and costly.

An important factor in the selection of the proposed options, is the possible reuse of the current abutments to carry the bridge. Due to the wear on the current abutments it may be preferred that the abutments are not used as the primarily support for the loading of the structure. However, if the best design included the use of the current abutments, efforts would need to be made to rebuild the Northwesterly abutment due to its current state of disrepair. Reconstruction will require more resources to accomplish. If the current abutments are not used then the bridge would be extended then the load will be carried by the compacted soil. This would require an increase in the size and therefore the cost of the bridge.

## *Preliminary Bridge Design*

Various design options are proposed that consider the various constraints including cost, environmental impact, sustainability and aesthetics. The designs that were explored are a simple span and truss design. Many of these design options were explored for various spans due to the fact that one of the abutments failed. The longer spans around 40 feet would bring the edges of the structure back to solid ground and spans around 25 feet require rebuilding the failed abutment.

### Bridge Span

In order to complete the analysis it is necessary to determine whether to rebuild the abutments or expand the bridge span so that the beams are on a shallow foundation supported by the soil. The shorter beams would be supported by the current abutments or a reconstructed abutment. Although there is a great cost to rebuild the failed abutment it is necessary to rebuild it for a few reasons. In addition to providing a solid surface to support the bridge it provides a way to keep the soil from eroding away. The abutment acts as a retaining wall for the slope that leads down to the bridge. Without that retaining wall there is nothing that is preventing the entire slope from being carried into the river. In addition to acting as a retaining wall the abutment reduces the span of the bridge which allows smaller length and sized members to be used. This will greatly reduce the cost of materials that are required to provide enough support for the structure. If the abutment is used then the span will be about 22 feet and if the abutment is not used then the span will be around 35 feet.

### Simple Span Design

The easiest way to cross an obstacle is to use a simple span. In this design option where the only support is obtained from the moment resistance of the spanning member, multiple materials were evaluated including sawn lumber, round timber, glued laminated timber (glulam), and steel members.

**Figure 9: Round Timbers**

In the designs involving sawn lumber and round timbers the sizes of members were unreasonably large. For example a thirty foot span would require four eighteen inch diameter round timbers (Figure 9). If sawn lumber was used for the same thirty foot span it would require four members larger than 8"x24". From contacting the local lumber yard (T.S. Mann Lumber Co.) the largest members that are readily available are 8"x12" although larger members could be special ordered.

**Figure 10: Glulam**

Another way that wood members can be used for this purpose is to use glued laminated timber. This is an engineered wood which is made up of multiple layers of wood glued together which allows smaller sized members to be used to support the same loading conditions (Figure 10). For the same thirty foot span as the previous examples four 5"x11" members with a stress rating of 24F-1.8E provides enough support for the applied loading. Although this design alternative provides enough structural support it does not follow the "green" building techniques of the Trustees of Reservations. The glulam members would have to be shipped from a company that is a large distance away. The two closest supplier to Petersham, MA are located in Maine or Ohio which will require a large delivery cost and a large carbon footprint.

The final design alternative is to use steel beams that are either new or salvaged. These steel members are extremely strong but they are heavy and getting them to the site could be difficult. The design values for steel construction were obtained through the Steel Construction Manual (AISC, 2005).

## Truss Design

One way to minimize the size of members is to use a truss, however this increases the length of wood required.  In addition to using smaller members the truss design can also be used as a railing which optimizes the required members.

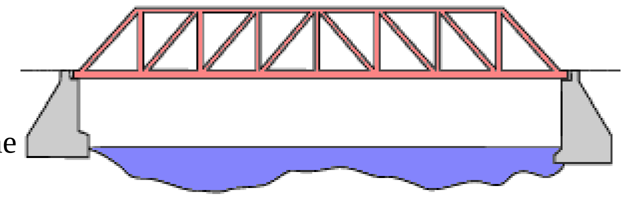


Figure 11: Example of Howe Truss

This Howe truss shown in Figure 11 was designed so that the height of the truss is the same height as a typical railing.  This was analyzed at various spans to determine whether it was better to expand the bridge so that it spans back to solid ground or if the abutments should be reconstructed.  The results of these calculations are in Appendix J.  Similarly to the simple span the longer the span is the larger the size of the members.

## Covered Bridge

Figure 12: Example of Covered Bridge

One way to increase the lifespan of a wooden bridge is to put a roof on it which reduces the amount of water and other harmful materials that gets on the structure.  An example of a covered bridge is shown in Figure 12.

This increases the total weight of the bridge and therefore larger members are needed to support the roofing system.  This roof is placed upon a truss design which is tall enough to allow vehicular traffic to pass.

## Final Bridge Design Options

The various calculations for each design option are shown in Appendices G through K.  The design values for the wood and bolts were obtained through the 2005 National Design

Specification for Wood Construction (American Wood Council, 2005). Various assumptions were made during the calculations and they are stated when applicable.

### *Decking*

The only structural component of a bridge that directly comes into contact with the applied forces is the decking. The decking must be strong enough to transfer the applied loads into the spanning members. Due to the fact that there are many decking members that must be used it is advantageous to minimize their sizes in order to reduce costs of the entire bridge. This bridge will use the minimum thickness allowable by AASHTO Bridge Manual which is four inches and the other members are designed based upon these calculations (American Association of State Highway and Transportation Officials, 1998). These members will be eight inches wide and fourteen feet long. After multiple calculations it became apparent that there should be some support directly below the wheel path of the truck in order to reduce the shear forces which the decking must withstand. One design option that shows this is shown in Figure 13. In this option there is a member at the end of the decking and another member three and a half feet from the end. Under this loading condition the decking will fail.

In addition to the need for support members to be under the wheel path the decking cannot support a long span. If there is a large span then a large moment is created which the decking is not strong enough to support. The two ways to prevent the member from failing in this way is to either increase the size of the member or add a support in order to reduce the span. Another aspect of the decking that must be addressed is the amount of deflection that occurs. If a member is not included in the center then the deflection that occurs will be large and influence the functionality of the bridge.

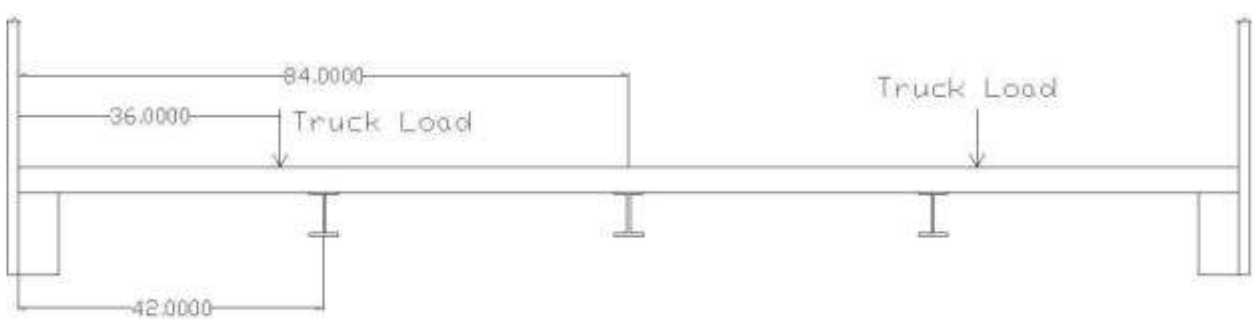



**Figure 13: Example of Unused Design Cross Section**

## Connections

The decking must be connected to the spanning members. In accordance with the AASHTO Bridge Specifications when nails are used then they must be twice the thickness of the decking and a minimum of two are used in each location where the decking crosses a spanning member. After loading the bridge in every loading condition possible the maximum uplift force was determined to be 33 pounds. A nail with a diameter of .099 inches (12.5 gauge) and a length of eight inches provides ample withdrawing support. It is very unlikely that a nail this thin would be available that long so any nail that is at least eight inches is strong enough. It is necessary for there to be four of these nails for each decking member connecting to the two wooden 6x12. In order to connect the decking to the steel I-beams 3/16" diameter steel carriage bolts are used. The bolt will go through the decking and past the flange of the beam. A washer and nut are attached in order to secure the bolt as shown in Figure 14. Carriage bolts were chosen instead of hex-head bolts because the heads of the carriage bolts are rounded. This will allow the decking to be smoother and countersinking holes for the hex-heads are not necessary.

### *Simple Span*

When considering all of the restrictions due to the location of the bridge it is apparent that a simple design is the best option. A simple span is the simplest option and therefore it will be easier to construct. After analyzing multiple options for a simple span bridge design to cross the east branch of the Swift River it was determined that the design shown in Figure 14 is the best alternative.

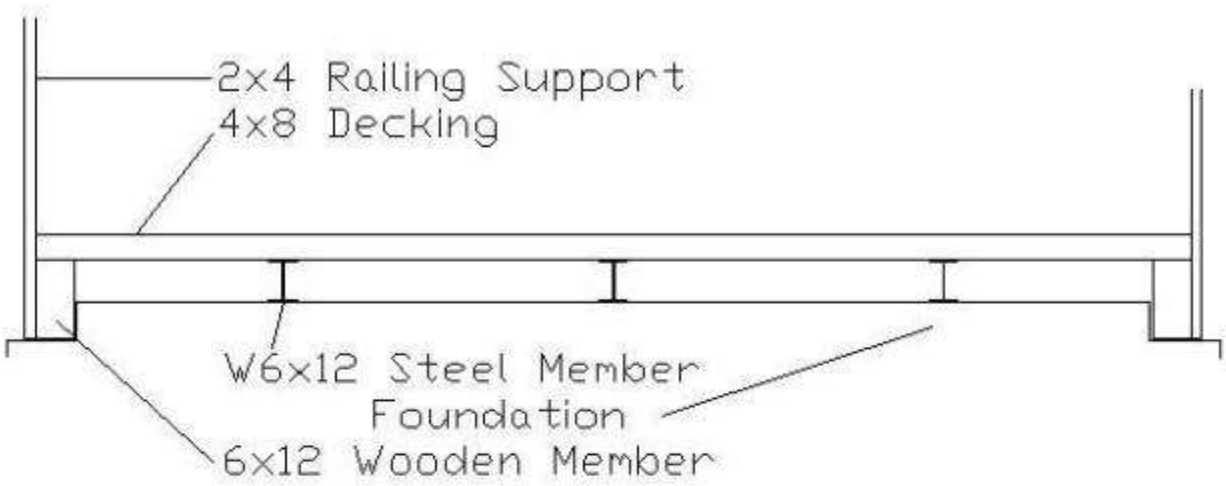



**Figure 14: Cross Section of Simple Span Design**

## Spanning Members

The design options that were not used ranged from using two steel members crossing directly under where the truck tires would be to using ten wooden 6x10s. The unused design options are shown in Appendix E. The chosen design uses three steel and two wooden members to span the river. Two of the steel members are spaced so that they are directly under the wheel path of a typical vehicle which occurs at three feet from the edge of the decking. This was done so that the weight of the truck would be supported by the steel members directly. If the wheel path occurred at some other place then the forces would have to be transferred by the wooden decking. This design minimizes the required size of the decking members. The third steel member is located in the center of the cross section. This was done so that the steel members support loads from the greatest area possible. The remaining area will be supported by the wooden members. The steel members could support all of the loads that the bridge will encounter but under certain loading conditions there will be large uplift forces due to the three foot cantilever. In order to prevent these forces the wooden members were added to the edges of the bridge. The wooden members support part of the pedestrian loads between the steel member and the edge of the bridge.

## Railing

Once the structural integrity of the structure was established then it was necessary to ensure the safety of the people utilizing the structure.  This was done by designing a railing which gives enough support to ensure that when it is needed then it will provide enough strength to prevent someone from falling off.  This strength was achieved by attaching each vertical member to the wooden spanning member with two bolts that are spaced far enough apart to provide ample moment support from the horizontal loads created by the anticipated loads.

The anticipated loads and other specifications were obtained through the AASHTO Bridge Specification (1997).  The railing must support horizontal and vertical loads of 50 pounds per linear foot and a concentrated load of 200 pounds.  The moment created from these loads are carried by two ¼” bolts that are spaced a minimum of four inches.  The strength of the wooden spanning members is checked again while compensating for the bolt holes.  When this was done there was a large factor of safety for the spanning member so the distance between bolt holes was expanded to five inches.  The spanning member still contained ample strength for the applied loads and the railing can support an even greater load than is required. These bolts also provide enough shear support so that the vertical forces associated with a railing are supported.

### ***Truss***

In an effort to minimize the required size of the wooden spanning members a truss option was evaluated.  This Howe Truss design is shown in Figure 15.  The truss is 54 inches above the top of the decking which is the minimum height for a railing when people on bikes will cross.  Similarly to the simple span this design requires three steel members in the same locations as the simple span.  In addition to these this design also requires a 6x12 wooden member for the bottom chord of the truss.  The truss has two functions, one as a structural component of the bridge and the other as a railing.  The railing component of the truss is done similarly to the simple span.

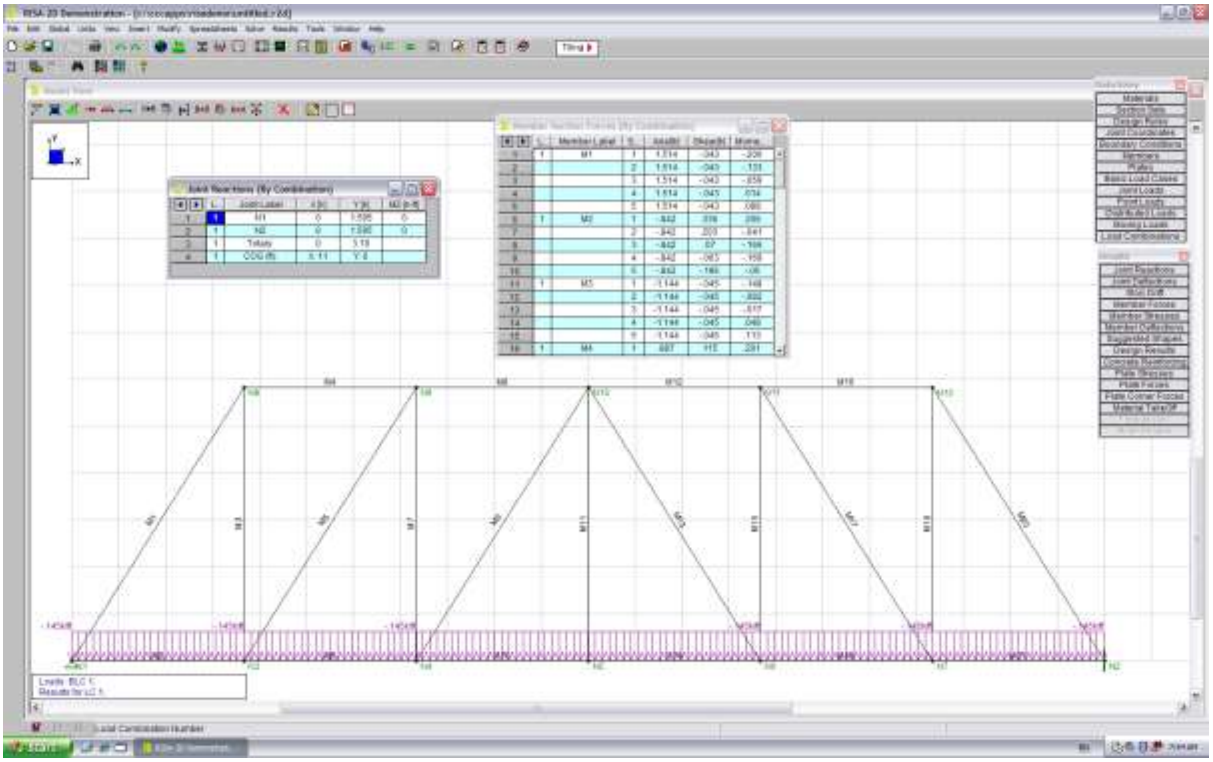


**Figure 15: Howe Truss Evaluation in Risa-2D**

There are two bolts which are spaced five inches apart in order to provide ample moment resistance. These bolts need to be ¾” diameter to be strong enough in shear. Similar calculations were conducted for every connection for both the top and bottom shear. The top chord needs to be a 2x10 wooden member. In addition to sizing the bolts it is necessary to ensure that the wood will not fail when it is in tension. The detailed drawing of the connections shown in Figures 16 & 17 ensures that there is enough wood to prevent the bolt being pulled through the wood.

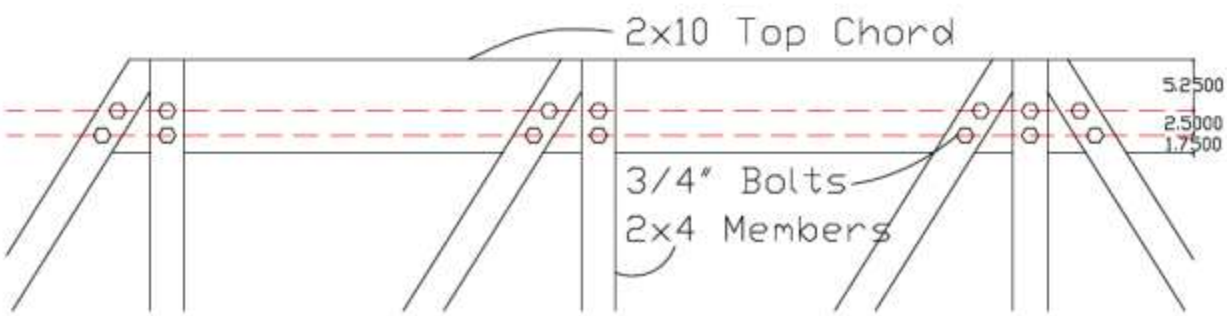



**Figure 16: Truss Top Chord Connection**

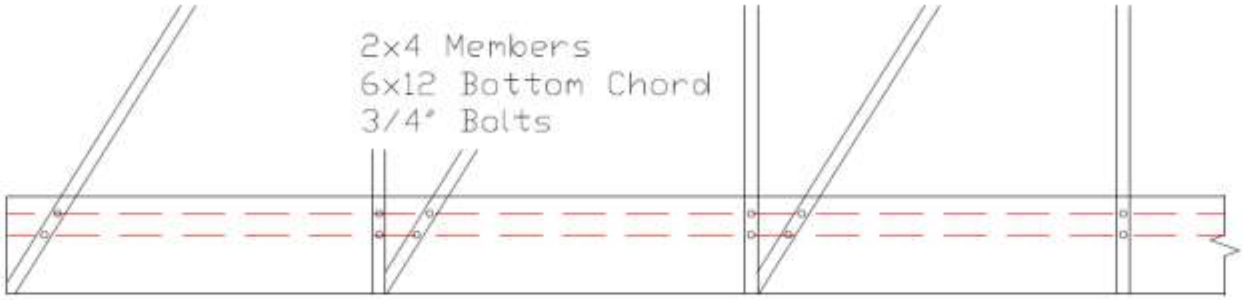



**Figure 17: Truss Bottom Chord Connection**

## Cost Estimate

Once the designs were finalized then the cost of each option was determined. The cost for the normal sized lumber were obtained from Home Depot. The large 6x12 wooden members are available at T.S. Mann Lumber. The steel I-Beams are available through Peterson Steel (Quote #305613). A list of costs of materials and the unit prices are in Tables 1 & 2. These costs do not include delivery charges. The truss design will cost about $4000, and the simple span will cost about $3500.

**Table 1: Cost Estimate for Truss Design**

| Item | Length | Unit Cost | Number | Total Cost ($) |
|---|---|---|---|---|
| W6x12 | 22' | 340 | 3 | 1020 |
| 6x12 | 22' | 367 | 2 | 734 |
| 4x8 decking | 14' | 40 | 35 | 1400 |
| 2x10 | 16' | 24 | 2 | 48 |
| 2x4 | 10' | 5 | 12 | 60 |
| 2x4 | 6' | 10 | 6 | 60 |
| 3/4" Bolts (hex head) | 8" | 5.08 | 44 | 223.52 |
| washer | | 0.75 | 44 | 33 |
| nuts | | 0.46 | 44 | 20.24 |
| 3/4" Bolts (hex head) | 4" | 2.69 | 44 | 118.36 |
| washer | | 0.75 | 44 | 33 |
| nuts | | 0.46 | 44 | 20.24 |
| 60 D Nails | 8" | 0.18 | 140 | 25.2 |
| 5 lb box of Screws | 3" | 26 | 1 | 26 |
| 1/4" Carriage Bolts | 5" | 0.46 | 210 | 96.6 |
| 1/4" Washer | | 0.06 | 210 | 12.6 |
| Nuts | | 0.06 | 210 | 12.6 |
| | | | **Total** | **3943.36** |

**Table 2: Cost Estimate for Simple Span**

| Item | Length | Unit Cost | Number | Total Cost ($) |
|---|---|---|---|---|
| W6x12 | 22' | 340 | 3 | 1020 |
| 6x12 | 22' | 367 | 2 | 734 |
| 4x8 decking | 14' | 40 | 35 | 1400 |
| 2x4 | 22' | 12 | 4 | 48 |
| 2x4 | 12' | 6 | 6 | 36 |
| 5/16" Bolts (hex head) | 8" | 1.2 | 24 | 28.8 |
| washer | | 0.08 | 24 | 1.92 |
| nuts | | 0.07 | 24 | 1.68 |
| 60 D Nails | 8" | 0.18 | 140 | 25.2 |
| 5 lb box of Screws | 3" | 26 | 1 | 26 |
| 1/4" Carriage Bolts | 5" | 0.46 | 210 | 96.6 |
| 1/4" Washer | | 0.06 | 210 | 12.6 |
| Nuts | | 0.06 | 210 | 12.6 |
| | | | **Total** | **3443.40** |

## Bridge to Foundation Transition

The bridge does not need to be fastened to the foundation because the weight of the bridge provides enough resistance from lateral forces due to wind and earthquakes. One aspect of this transition that is not simple is that the heights of the wooden and steel members are not the same. To compensate for this there needs to be notched section to accommodate for the different heights. This notch will need to be 5-1/2 inches in height as shown in Figure 14 on page31.

## Constructability

One major problem with constructing this bridge is the fact that there are large members which span the river. The steel members weigh twelve pounds per linear foot for a total of 264 pounds. The wooden members weigh 11.4 pounds per linear foot for a total of 251 pounds. The first member that crosses the span will be the most difficult. To get the first member across a front end loader connected to the member can lift and drag one side across the span. Once the first member is crossing the span then the others can be dragged across on top of it. After the spanning members are in place then the decking can be laid out and connected to the wooden members. Once this is done then the railing or truss component can be added to the structure. The bolt holes can be drilled either before the members are put into place or be drilled once in place. If they are drilled once they are in place then a ladder or some sort of staging will be necessary to make the member accessible. After the truss or railing members are attached then the top components can be connected. While this is taking place then the decking can be attached to the steel beams. With these two events occurring at the same time then the total time to complete construction is reduced.

# *Foundation Design*

## Functions

The primary function of any foundation is to support the structure above on the soil below. This purpose is important because soils can be unpredictable, they can move and become unstable, when subject to loading. The foundation acts as an interface, stabilizing the structure on a relatively unstable surface. Although that is the foundations primary purpose the foundation for the Petersham Bridge has other functions it must serve. Because the current structure failed due to exposure to water, any new structure would need to prevent this happening in the future. Separation between the soil and the wooden structure is vitally important to extending the life of the bridge. In addition, the foundation must provide or maintain the stability of the slope it is on. Because it is at the bottom of a slope stability of that slope has to be provided for by the foundation. In essence, the foundation designed will act as a retaining wall, holding back soils from the stream below in addition to its function as a foundation.

## Determination of Design Parameters

Before design could be completed a the following information is required: loads on the foundation, soil paramaters for design, and slope stability determinations.. Once these values were found a proper design of the foundation could be undertaken.

### *Loading*

The loading used in design was based on the heaviest bridge design considered with maximum loading, to provide the worst case scenario for the structure. The loading was obtained from the structural design of the bridge under maximal loading conditions and assumed to be distributed evenly across the 14 foot length of foundation. These loadings translated into a load of 1650 lb/ft.

### *Geotechnical Analysis*

One of initial processes necessary to designing the foundation is an analysis of the soil. A simplified soil analysis was all that was required for the site in order to determine its characteristics, because only rough strength values were needed for a small scale foundation and these could be attained by merely determining the soil type available. A lab based test was performed to determine the general makeup of the soil by percent of constituent size. From the content discovered soil parameters, most important shear strength and friction angle, can be estimated using tables of known values, these tables have been reproduced in the Appendix . This saves the time of a lengthy analysis process for a soil that is easy to categorize and is, based on content, likely a very good building soil.

The testing was done through a sieve analysis. The sieve analysis was conducted on a shovel sample, which was taken from the site during a site-survey trip. This sample was dried and weighed to determine its total weight, 2950 grams. The soil was then passed through a series of sieves: 3/4", 1/2", 3/8", #4, #8, #16, #30, #40, #50, #100, and #200. Once separated, calculations on the results determined that the soil was a well graded gravely sand with good amounts of silt. Clay content was found to be low, reducing chance of soil expansion. The soil therefore makes an excellent construction soil. Even given the minimum unit weight, 95 $lb/ft^3$, and an absolute minimum friction angle, 15°, for this type of soil, a suitable, shallow foundation could be designed safely. The following graph (Figure 18) shows the well-graded nature of the soil.

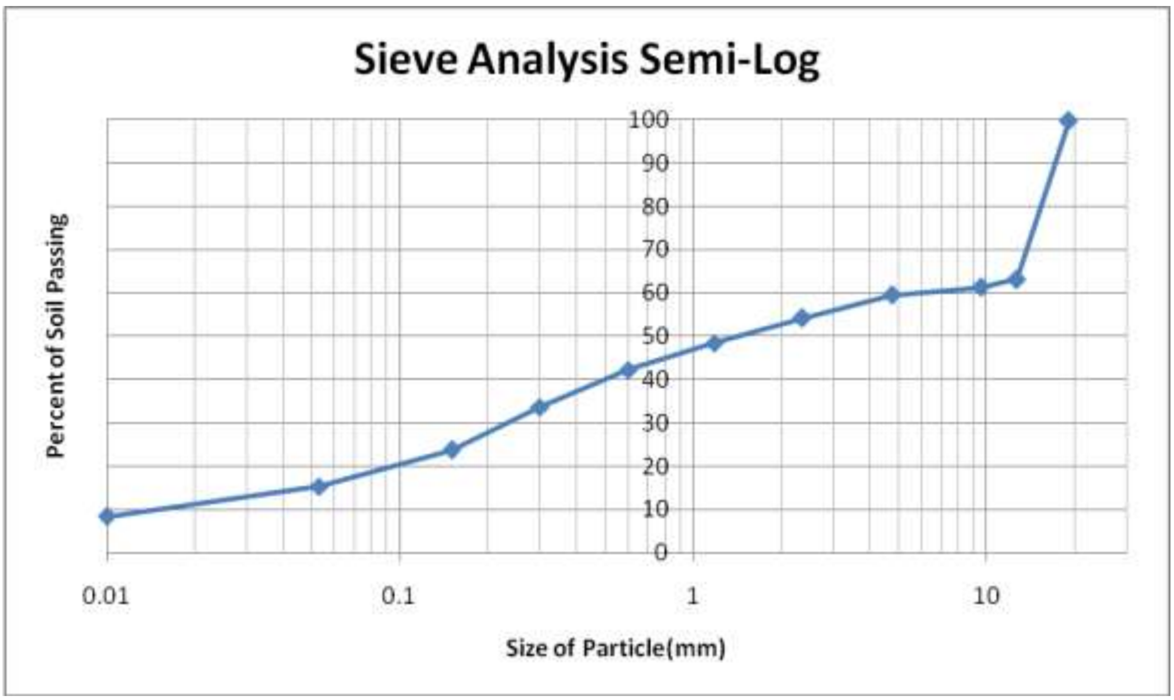



**Figure 18: Sieve Analysis Semi-Log, well-graded gravelly sand with silt**

### *Slope Stability*

After an analysis of the soils, an analysis of the stability of the current abutment system is possible and necessary in order to determine if countermeasures to any current stability issues which need to be solved. Slope stability is also determined during the design process in order to determine the stability of the slope with any new feature, a foundation or retaining wall, added. The slope stability took into account the current stone abutment as well as the load of the current bridge and soils upslope.  The nearness to water complicates the process of determining slope stability.  In order to simplify analysis, water was treated as a uniform soil mass when considering its weight..

For the determining the stability of the slope, the Swedish slip circle method was used to break up the soil into sections and compare their weights to the un-drained shear strength of the soils. This analysis took into account the large stone abutment that is currently in place.  A factor of safety of 5.41 was determined, indicating that the current slope is very stable and therefore no stability considerations needed to be made in design.

## Retaining Wall Foundation

With the completion of the geotechnical findings the design process of the foundation could be undertaken.  The first design considered was a retaining wall foundation.  The use of a retaining wall style foundation for both ends of the bridge serves a number of purposes.  As the name suggests it primarily acts as a foundation, distributing the load of the bridge to the soil beneath, as well as a retaining wall, holding back the soils from collapsing into the stream bed. Secondarily, the wall would protect against the major mode of failure for the current bridge structure.  Because the current bridge was eroded from below due to runoff being channeled into the abutment, some form of protection against this will be necessary for the longevity of the future bridge.  Erosion and water contact increased deterioration of bridge members while additionally removing necessary support from below.  A retaining wall foundation would effectively block water from eroding the bridge members and proper drainage of the wall would prevent the wall being eroded in a similar fashion to the current abutments.

### *Design*

The design of the foundation wall was broken into four major sections: external stability of the footing, internal stability of the footing, the internal stability of the stem, and the overall external stability.  First, however, limitations on the design dimensions need to be considered.  These limitations were the depth (controlled by frost heave) and width (controlled by bridge width).

This particular foundation has to be placed at a depth at least five feet below the surface due to potential uplift and cracking due to frost heave. Because the soil is very near a water source and has some silt content, it is somewhat susceptible to frost heave. This susceptibility corresponds to classification F2 in regards to frost susceptibility using U.S. Army Corps of Engineers guidelines. Since there was no reason to dig any deeper than the minimum, five feet was used as the design depth for the foundation

Also, in order to perform the secondary function of reducing water flow through the current, abutment structure a foundation the width of the bridge was necessary, this width is 12 feet, refer to Figure 18. However, 14 feet was used as the length of the foundation to provide safe coverage from runoff damage. Anything less would merely divert flow to the outside of the abutment, further eroding the original structure. These two initial dimensions, 5 foot depth and 14 foot length, had to be use in the final design.

### *Footing External Stability*

A few different specific dimensions were then determined based on stability performance of the foundations interaction with the soil. The primary requirement for external stability is whether the bearing capacity of the soil is sufficient to prevent shear failure. The footing had to be large enough to allow the soil below to hold its weight. The worst-case bearing capacity was found to be 750 lb/ft$^3$.

Initially a width of 3 feet was designed. This design turned out to be well within the factor of safety for bearing capacity; however, when internal strength was determined the reinforcing lateral steel did not have enough room to develop strength. Because 3 feet did not provide sufficient development length the dimension was increased to 4 feet.

The design of a four foot wide footing turned out to be the best design to prevent shear failure and the four feet allowed room for the lateral steel to develop. The effective depth required for the footing, based on the loads provided from the bridge turned out to be well below the minimum required, so the minimum of 6 inches was used in addition to a total minimum depth of footing of 12 inches.

### *Footing Internal Stability*

With the dimensions of the footing determined the number and size of the reinforcing steel within the footing had to be determined. In order to withstand tensile strength due to bending under load, reinforcement has to be used. Because the footing width is fairly wide compared to the stem width (1 foot) lateral steel had to be used in addition to the standard longitudinal steel. Steel analysis determined that lateral steel had to be #5 bars placed every 4 inches. The longitudinal steel was 5 # 4 bars placed every 8 inches. Due to the depth of the footing, temperature steel is not necessary for design. In all cases a minimum of 3 inches of concrete between the steel and the exterior of the concrete was necessary.

### *Stem Internal Stability*

The internal stability of the stem had to be checked to make sure that it did not fail because of excess flexure. The soils behind the wall exert force upon the wall causing it to bend. By determining the moment forces created by the soil the amount of reinforcement the concrete would need was determined. It was found that 5 #5 bars would be sufficient, because of the low amount of moment forces; these steel bars would be placed every 3.5 feet. Using a check for minimum shear steel these also accounts for the shear steel that would be needed, because shear forces against the wall are relatively low.

### *Overall External Stability*

Since this foundation is being treated as a retaining wall it was also checked for its ability to withstand overturning. Overturning is determined by using the moments about the point on the foundation it will tip over, the toe. A sum of the driving moments, those causing overturning, and resisting moments, those resisting overturning, was done and the factor of safety against overturning was determined to be 1.9. This is the lowest factor of safety determined for the design; however, it is still above the minimum required of 1.5 for overturning.

### *Constructability*

The materials for the foundation wall are fairly extensive as well as the labor required. Two excavations of 14'x5'x5', a total of 700 $ft^3$, would need to be made. This would be followed by lying of gravel then formwork after which pouring the concrete would take place. This design would require a large amount of concrete for such a remote site: 8.5 cubic yards.

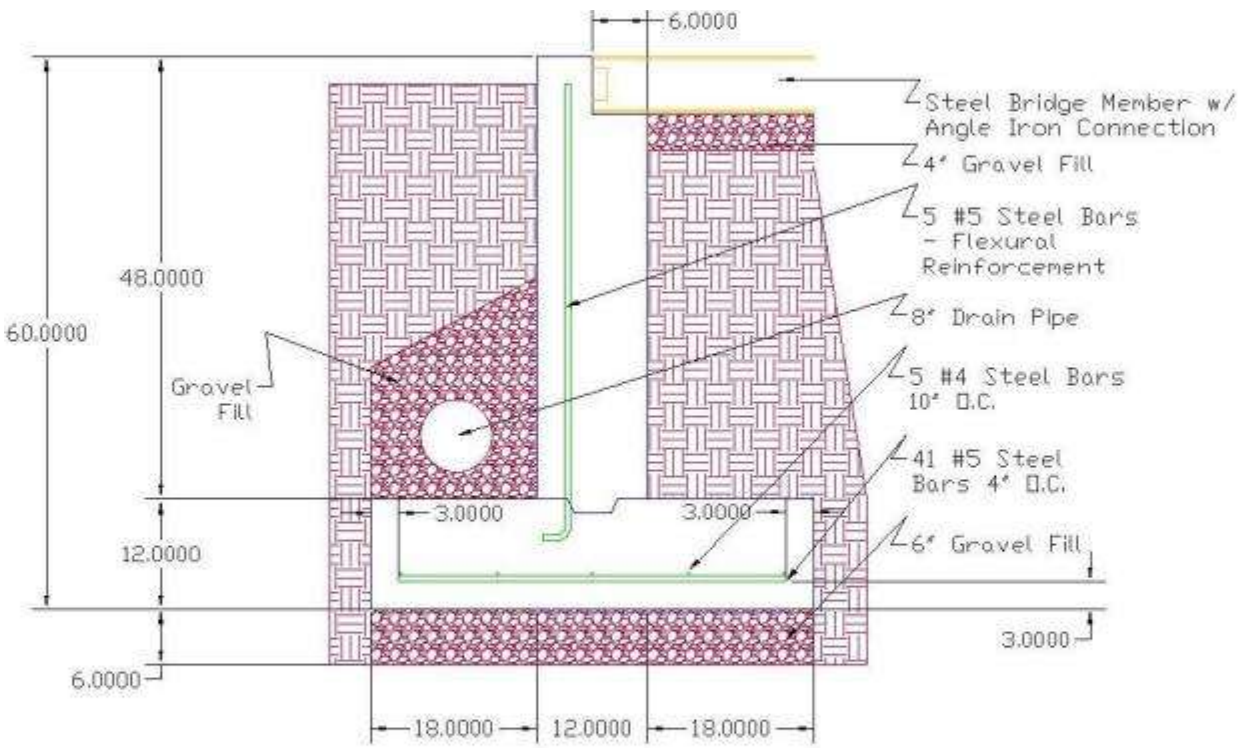



**Figure 19: Foundation Wall Cross Section**

### *Cost Estimate*

**Table 3: Cost Estimate of Retaining Wall Foundation**

| Material | Quantity or Volume | Price per Unit | Price |
|---|---|---|---|
| **Concrete** | 8.5 cubic yards | $70 | $600 |
| **Gravel** | 2.5 cubic yards | $30 | $75 |
| **Reinforcement #4 (Longitudinal)** | 70 feet | $.50 | $35 |
| **Reinforcement #4 (Lateral)** | 141.75 feet | $.50 | $70 |
| **Reinforcement Stem #4** | 20 feet | $.50 | $10 |
| | | **Total** | **$790** |

## Concrete Pour Foundation

The Concrete Pour Foundation, which is a type of cast in place footing, would be a very simple foundation for the bridge. It involves pouring concrete into the stone abutment below in order to cement the stones and prevent water passage while holding the deteriorating structure together. As stated previously, the damage caused by water flower over wood members and through the stone abutment lead to the failure of the bridge. Pouring concrete between these stones after removal of the soils between them would generate a sufficient barrier to these damages. The large concrete foundation would act as a massive gravity wall/foundation, both maintaining the soil behind it due to its large weight and supporting the structure above due to its large area. Because of the simplicity of the design stability in regards to the slope stability, sliding, overturning, and bearing capacity were the only calculations necessary. These all relate to the external stability because it is assumed that internal stability is maintained by the mass alone. The void ratio of the boulders was assumed to be around .2 due to the purposeful stacking of boulders, this assumption was used in cost estimate and weight calculations.

### *External Stability*

The first check to be done on the gravity wall is whether such a large object would begin sliding due to the weight of soil behind it. Based on calculations summing the forces resisting and driving the foundation, the concrete pour passed significantly, with a factor of safety of 6.5 against sliding.

Second was the necessity of checking against overturning. Much like the foundation wall before it overturning would occur about the lowest point down slope, the toe. Calculations showed that because of the large weight of the foundation overturning about the toe will not occur. The factor of safety against overturning was 4.4, much larger than the factor of safety for the foundation wall.

The bearing capacity of the soil was checked to see if it could withstand the additional load of concrete, reusing the same dimensions as our previous foundation for a rough estimate of size, the concrete pour foundation passed the bearing capacity test very well. Because, for this foundation, the soil is assumed to be mostly large boulders, the additional weight of the concrete does not add significantly to the already large unit weight of the boulders.

Finally, and most importantly, it was checked whether this new foundation would maintain the stability of the slope. With additional weight now driving a slide, this was the number one concern with this design. The same methods were used as with previous slope stability with the addition of the foundation as a large soil unit with a unit weight the same as concrete. Despite concerns, it once again passed slope stability analysis. Using a Swedish slip circle analysis, the factor of safety turned out to be 5.4.

***Constructability***

Normally the construction of a massive gravity foundation is very difficult due to transportation of heavy stones. However, this foundation would be built assuming that the stones were already in place, therefore avoiding this issue. However, a further problem with using this particular design is the necessity to remove soil from between the rocks. The most feasible way to do this on a large scale is to use some sort of water pump to force the soil out of the recesses. However, this could have adverse effects on the stream. Another alternative would be to remove some boulders and the soil, and then replace the boulders before pouring the concrete. This method, potentially, would not provide the depth to protect the abutment, but with proper runoff maintenance uphill this could be a feasible solution while reducing cost. The precise dimensions of the foundation are not necessary as long as a fairly significant amount of soil is removed the additional weight added by the concrete should serve its purpose as a barrier to erosion and a distributor of structural loads.

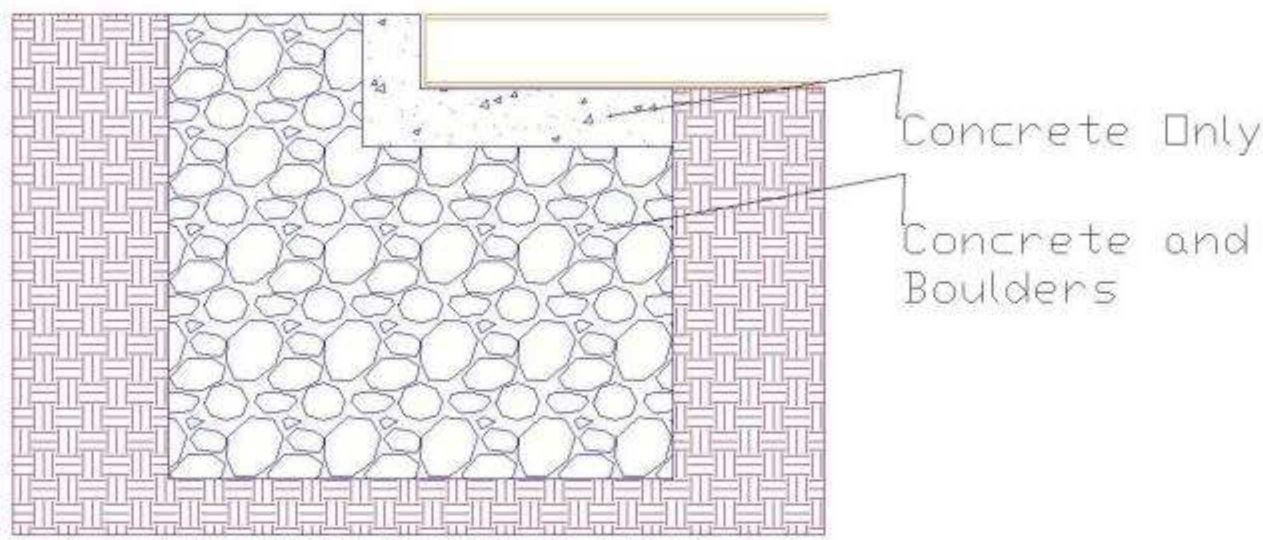



**Figure 20: Cross Section of Massive Gravity Foundation**

### *Cost Estimate*

**Table 4: Cost Estimate of Concrete Pour Foundation**

| Material | Volume or Quantity | Price Per Unit | Price |
|---|---|---|---|
| **Concrete** | 2.5 cubic yards | $70 | $175 |
| **Pump Rental** | 2 days | $50/day | $100 |
| | | **Total** | **$275** |

### *Weaknesses*

The Concrete Pour Foundation would be a fairly simple foundation and require minimal labor; however, it would require additional machinery in order to remove the soil from between the stones. As discussed in the constructability some form of hydraulic removal of the soil, using a water pump, would be feasible, or the pour could be made in the areas more clear of soil directly under the bridge. This would have the advantage of being simpler, however the concrete, if showing would detract from the rustic aesthetics of the bridge.

## *Slope Runoff Diversion Design*

As seen in the hydrologic analysis of the project site, a solution to address the amount and velocity of runoff is needed to extend the life of the new structure.

### Considerations

A large contribution to the failure of the previous bridge stems from runoff of rainfall. In order to combat this, the situation should be addressed by reducing two factors, the volume and velocity of the water when it reaches the structure. One way to counteract this problem is to re-engineer the foundation to withstand higher velocities, as well as to have less of a weakness to water itself. This is being addressed through the foundation design, as opposed to the previous structure, in which the wooden beams were placed directly on the ground and abutments. The foundation will also include the use of different materials. While the original purpose of this is to counteract the effect of frost heaves, it should also contribute to increasing the infiltration of rainfall runoff. The new soil will have larger pieces, allowing for more space for the water to infiltrate. It will also assist in slowing the water down while it is under the structure.

Uphill of the structure and foundation, another measure should be taken to further decrease velocities, and to divert some of the runoff away from the bridge.

### Possible solutions

A common method in recreational trail design to address runoff is a waterbar across the path (Ministry of Tourism, Sport and the Arts, 2001). This is a popular technique in trail design; as such, this is a good solution because trail users will likely have experience with this type of trail modification, removing the risk of injury due to surprise or inexperience with the structure.

### *Waterbar*

A trench dug at an angle and reaching completely across the path should address two of the causes of bridge failure (infiltration velocities, and runoff volumes). It should minimally effect path use, and as a method used at other locations, not disrupt the rustic feeling of the location. It can be constructed by volunteers and the only materials to be added are easy to obtain and install. Figure 21 shows a general design for this type of installation.

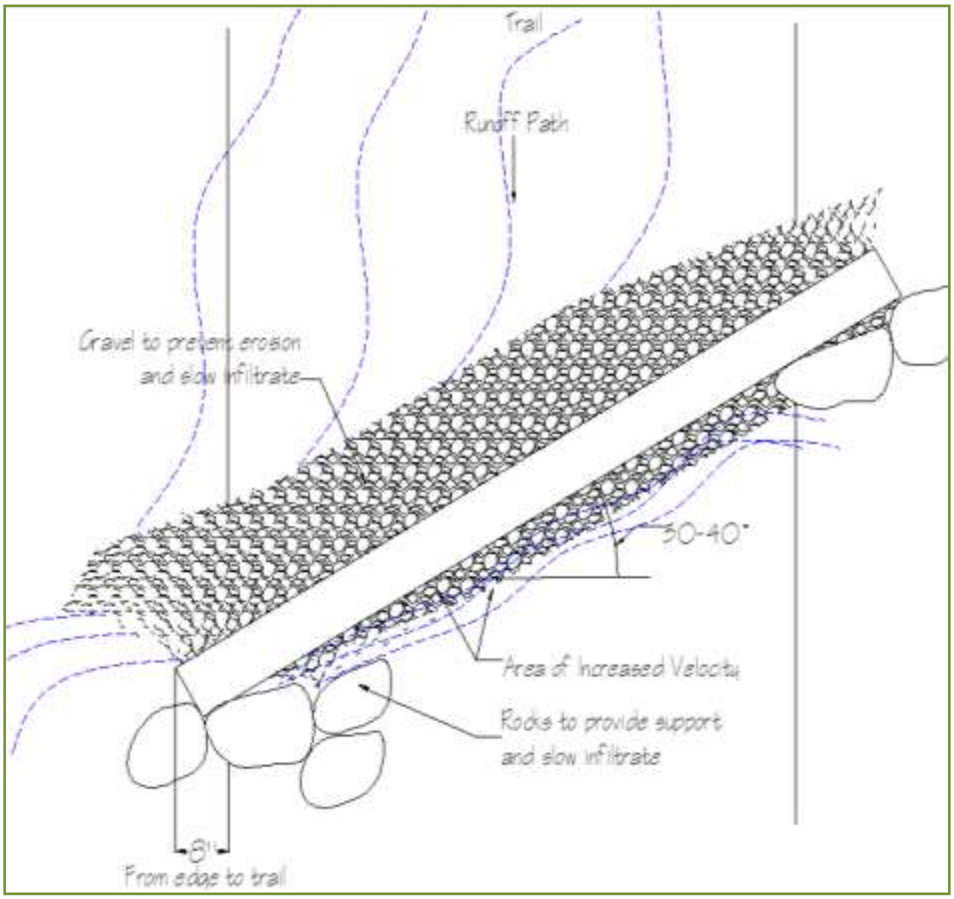



**Figure 21: Log Waterbar Plan View**

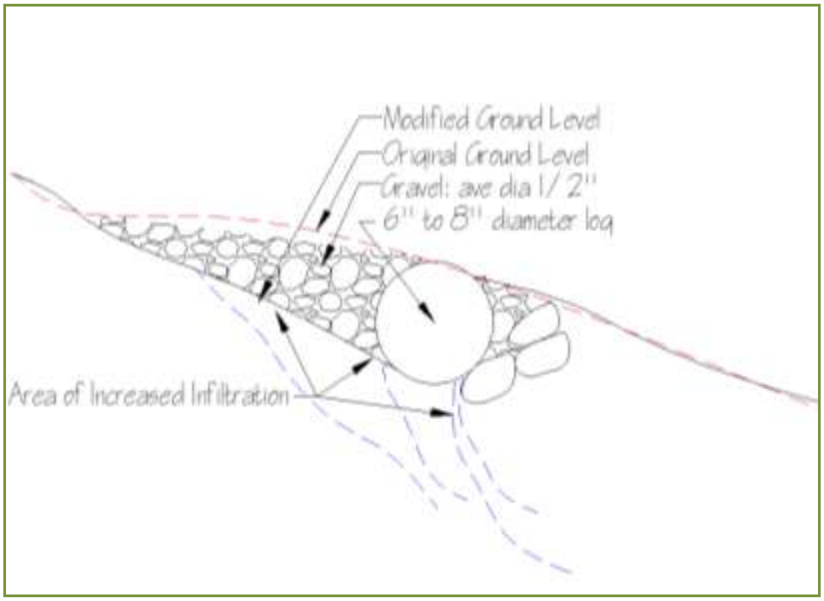



**Figure 22: Log Waterbar Cross Section**

There are two choices for waterbar construction, and either would be a good solution. The first employs a log to form the main interruption of surface runoff (Figure 22).

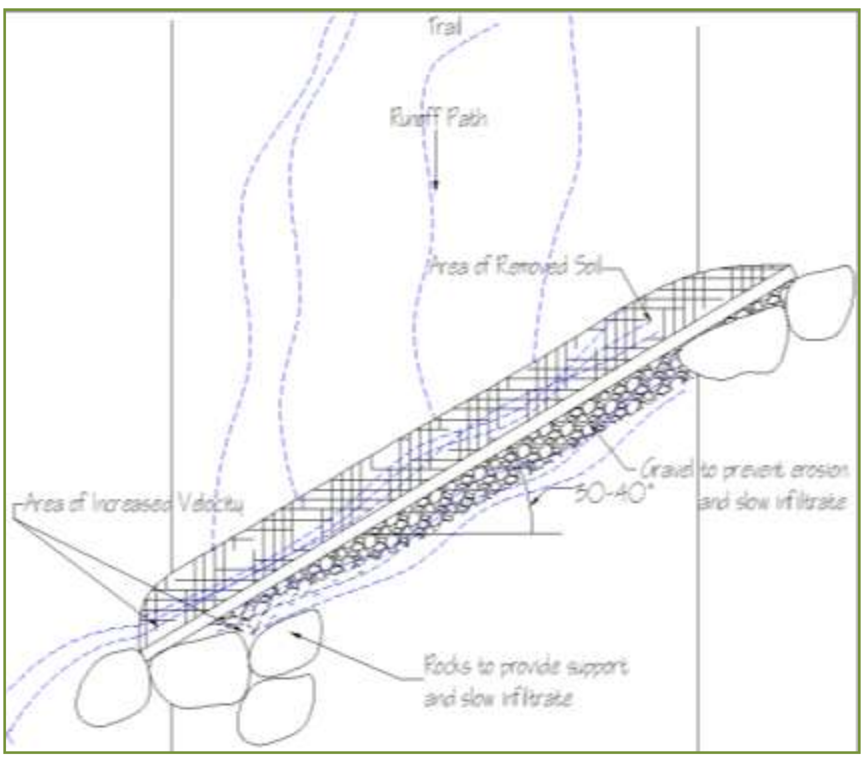



**Figure 23: Plank, Rubber, Stone Waterbar Plan View**

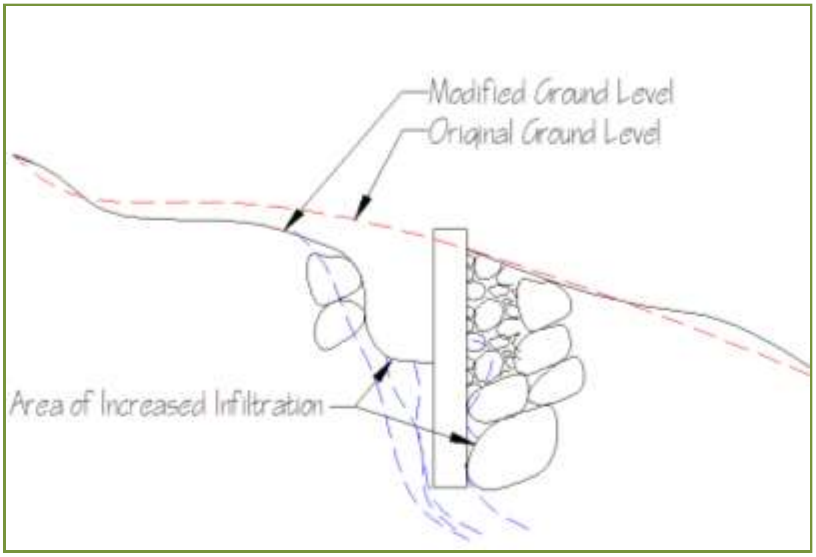



**Figure 24: Plank, Rubber, Stone Waterbar Cross Section**

The second allows for a choice of materials, either a wooden plank, rubber sheet, or stone barrier (Figures 23 & 24). These provide reinforcement to prevent erosion on the downhill side.

## Construction Materials

The choice of materials for the support piece of the project, by far the most costly and most difficult to install, is open. One choice is a 6-8" diameter log, of a length that will cross the trail on a diagonal of 30 or 40 degrees, and extend at least 8" beyond the downhill edge of the trail. The other materials should carry the same length but they may be wither a 2"x24" wooden plank, a PVC pipe of 6-8" diameter, or a collection of stones with a depth of at least 14". If the trail is 10 feet wide, the length will need to be 14'-6". See Figures 21-24 for drawings of the possibilities.

It is estimated that 2.07 cubic yards of crushed stone or gravel will be used for each installation of the waterbar. In order to maintain the appearance (or actuality) that all materials are local, effort should be made to match imported stones with those of the area. If it is found that stones of the proper size (1/4 inch to 2 inches) are found while soil is removed, these are suggested to be used in the gravel fill. The gravel should be delivered in the TTOR truck or equivalent. It may require more than one trip of multiple waterbars are being installed at the same time.

It is suggested that the construction take place while the ground is not frozen for ease of construction. There are two choices for creating the trenches required for changing the fill.

The first is to use hand shovels. While this will take longer, it is the simplest solution and a good way to involve the volunteers in the construction process.

The other option is to use the machinery that TTOR has available. This would require moving the equipment to the site, which may have a negative effect on the condition of the trail it would travel over. It would also exclude volunteers from the process, as it may be a safety concern to have people of inexperience on the site during its operation.

A level and measuring tape should be employed to ensure that the waterbar is constructed at the correct angle and is level with the water table.

The method for installing the waterbar is relatively simple. A trench should be cut at 30-40 degree angle in the slope (placement to be determined). This should be deep enough to completely cover the log. On the downward side, rocks should be placed to support the ends of the log. These will also serve to slow velocities of water that travels along the smooth backside of the log. Along this backside, gravel fill should also be placed, to encourage infiltration and slow water. On the frontside or uphill edge of the log, additional fill should be placed. This should begin approximately one foot in front of the log and be placed to a depth of 16", increasing with proximity to the waterbar itself. See AutoCAD drawings for completed plans.

## Impact on the immediate area

A trench of this nature will ultimately result in primarily moving the water to one side of the trail, not removing it from the area, so there will be consequences of taking this action. The side of the trail that the water is diverted to will erode, causing a small streambed to form. This will lead to additional sedimentation where this joins the Swift River, as the water loses velocity. It will change the look of the area, and over time, could contribute to erosion along the side of the path, if not properly constructed and attended.

# PROJECT IMPLEMENTATION

## *Permitting*

Permitting is a large part of successfully completing any construction project. To ensure that the right steps are passed on to the Trustees for completing permitting requirements, much research was completed. Town Officials are responsible for overseeing the projects occurring in their town, so a visit to the Town Office proved useful. From there, contact was made with members of the Conservation Commission, and Planning Board.

Completing the permit includes providing plans of both existing conditions and the proposed structure. A plan for construction is also necessary, since the construction process has the potential to greatly impact the surrounding area.

It was determined that a Notice of Intent (NOI) is needed for permitting on the project. The NOI is regulated under the Wetlands Protection Act (310 CMR 10.00). The Massachusetts Department of Environmental Protection Bureau of Resource Protection provides the application and instructions for completion (Massachusetts Department of Environmental Protection, 2008). The purpose of the NOI is to provide the appropriate committees with a description of the site and the proposed work. Applicants must submit the NOI in completed form along with all plans and supporting materials to both the Mass DEP and the Town of Petersham Conservation Committee. A professional engineer may need to certify the calculations and plans of the proposed project.

There is a fee for submitting the application and its review. Plans may be required to have the certification of a professional engineer before the committee will consider them.

A summary and procedure for completing the Notice of Intent have been developed and can be seen in Appendix E. This is designed to assist The Trustees of Reservations in completing the permit, in hopes of expediting the project process.

## *Cost Analysis*

Minimization of cost is a key to any project, however, due to the donation and volunteer dependent nature of The Trustees of Reservations, this minimization is especially important. The use of volunteers for the bridge construction process will reduce the final cost. Incidental costs that may appear are limited to such items as food and transportation of volunteers.

Materials represent a cost in the redesign and reconstruction process. Any new materials used will generate the highest costs and therefore recycled products are preferred. It is assumed that any recycled materials will be free of cost of acquisition, other than any transportation costs due to gas expenditure. Some areas where new materials could potentially be needed are: the wood decking and all connections and fasteners. Recycled angle iron, reused decking, and reused stones for abutments will be treated as cost free.

The energy consumption of a generator based on build time and onsite usage will be factored into the cost. Any gas consumed during the building process, from the generator or the use of vehicular transportation will be the second constituent that will play an important role in the costs of building. This includes: transportation of materials, transportation of volunteer labor, transportation of tools, and generator use. Depending upon the distance from the site to all these resources costs will vary.

## *Construction Scheduling*

The construction scheduling will be broken into four primary components. The specifics of these will depend upon the final bridge design.

The first portion of the construction project will be the safe removal of the current structure. This includes disassembly and removal of all unused materials. The process and requirements for what is to be removed will be based upon the final design. Whether recycled construction materials will be removed to an offsite location or kept onsite will ultimately depend upon the judgment of the building team.

Depending on the need for new portions to the abutments the removal of currently, structurally unsound portions will be conducted following the removal of decking and beams. This could include the removal of abutment stones from the waterway and pieces of abutment from the current gap in the interior of the North Westerly abutment, depending upon levels of reconstruction decided upon.

Any reconstruction required on the abutment will be done following this removal of materials. This stage will be desired minimal and may be decided against. Likely it will involve the use of local stones stacked back into the abutment to stabilize and strengthen it.

The construction of the new bridge will begin once all removal of materials and the rebuilding of all abutment work is finished. Depending upon the design chosen, construction may include lying of beams, on and off site construction of portions of the bridge, landscaping to provide proper foundation for bridge ends, and fastening of all bridge components as per the design specifications.

# RECOMMENDATIONS

## *Structural Design*

In order to further analyze the bridge alternatives it is necessary to eliminate some of the designs that are not reasonable to use. The all wood simple span design is not reasonable to use because timbers of the necessary dimensions are not readily available. Another option that is not reasonable to use is glulam timbers. This is because the Trustees of Reservations goal is to preserve nature and use “green” techniques and glulam timbers are not available in this region. In addition to the required lumber it is necessary for a substantial amount of glue to keep the different layers together. These members must be made in a specialized location and then shipped to the site which is a much greater impact on the environment. A truss design is a good alternative however it is necessary for there to be some support in the decking other than that which is provided on the sides due to the truss. A covered truss design is a viable option because the extended lifespan outweighs the additional cost associated with the construction. In addition to the covered bridge option the simple span is another option which should be analyzed further. A combination of steel and wooden members will maximize the strength while minimizing the cost and impact on the environment.

In order to minimize materials and therefore costs it is apparent that the simple span is a viable option. This design requires the same amount of spanning materials as the truss option. The railing component of the simple span can be varied as long as it provides enough shear and moment capacity as explained in the results section. This option is also the simplest which will make construction easier. The other design option which is feasible is the Howe Truss. This design utilizes the same spanning members as the simply supported however it also uses additional members in the truss. Both of these options are simple which will allow construction to occur at a rapid pace. These designs will utilize the existing abutment and a reconstruction of the failed side which will keep the member sizes a minimum. The reuse of the abutment will reduce the environmental impact of the project.

## *Foundation Recommendations*

Both designs are feasible designs that depend largely upon subsurface soil conditions which have not been determined. Given continuous large boulders below the surface, a poured concrete foundation would be advisable due to excess costs as well as the difficulty of removing large boulders. On the other hand, given more easily excavated soil the reinforced concrete foundation would have to be used, despite its greater cost because the poured concrete foundation requires the presence of the boulders for its strength, as well as to reduced the volume of concrete necessary.

## APPENDIX A: Profile of The Trustees of Reservations

The Trustees of Reservations are a private non-profit conservation organization formed in 1891 by Charles Eliot, a landscape architect.  They have grown since their inception to own and maintain 94 reservations in Massachusetts and currently have over 40,000 members.

The purpose of the Trustees is stated in the opening of their charter:

"*The Trustees of Reservation* shall have as its purposes acquiring, holding, arranging, maintaining and opening to the public, under suitable regulations, beautiful, historic, and ecologically significant places and tracts of land within this Commonwealth; acquiring, holding, maintaining and enforcing such conservation and preservation restrictions, easements and other interests in land, water areas and structures as it deems appropriate and in the public interest,; and educating the public with regard to natural and historic resources and their conservation and stewardship, all in the manner and to the extent permitted by law, with the powers and privileges and subject to the duties set forth in Chapter 180 and in such other general laws as now or hereafter may be in force relating to such corporations; but said corporation shall have no capital stock." (Reservations, 2008)

## APPENDIX B: Map of Brooks Woodland Preserve

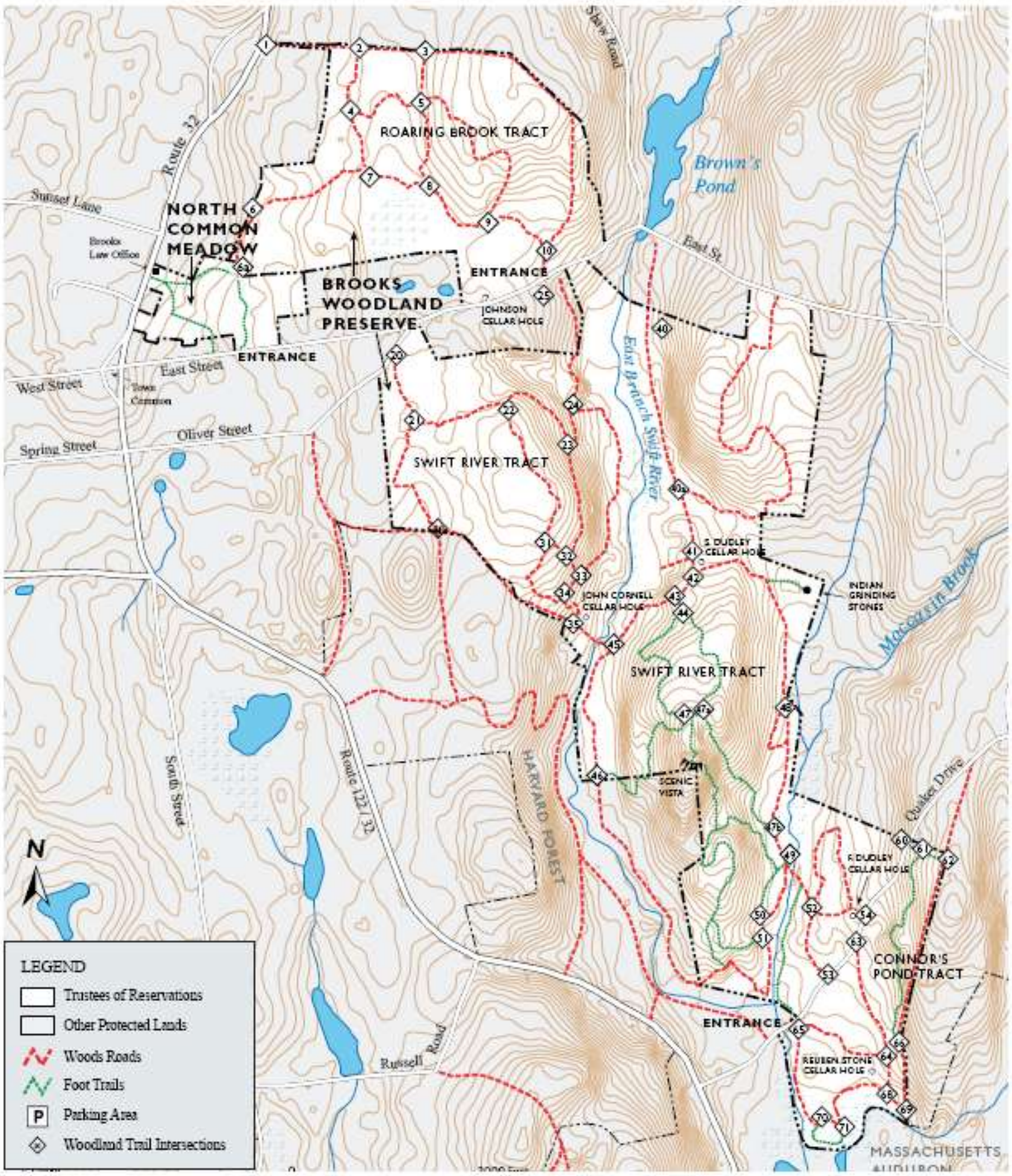
# BROOKS WOODLAND PRESERVE

*Roaring Brook Tract:* East Street ▪ Petersham, Massachusetts

*Swift River and Connor's Pond Tracts:* Quaker Drive ▪ Petersham, Massachusetts

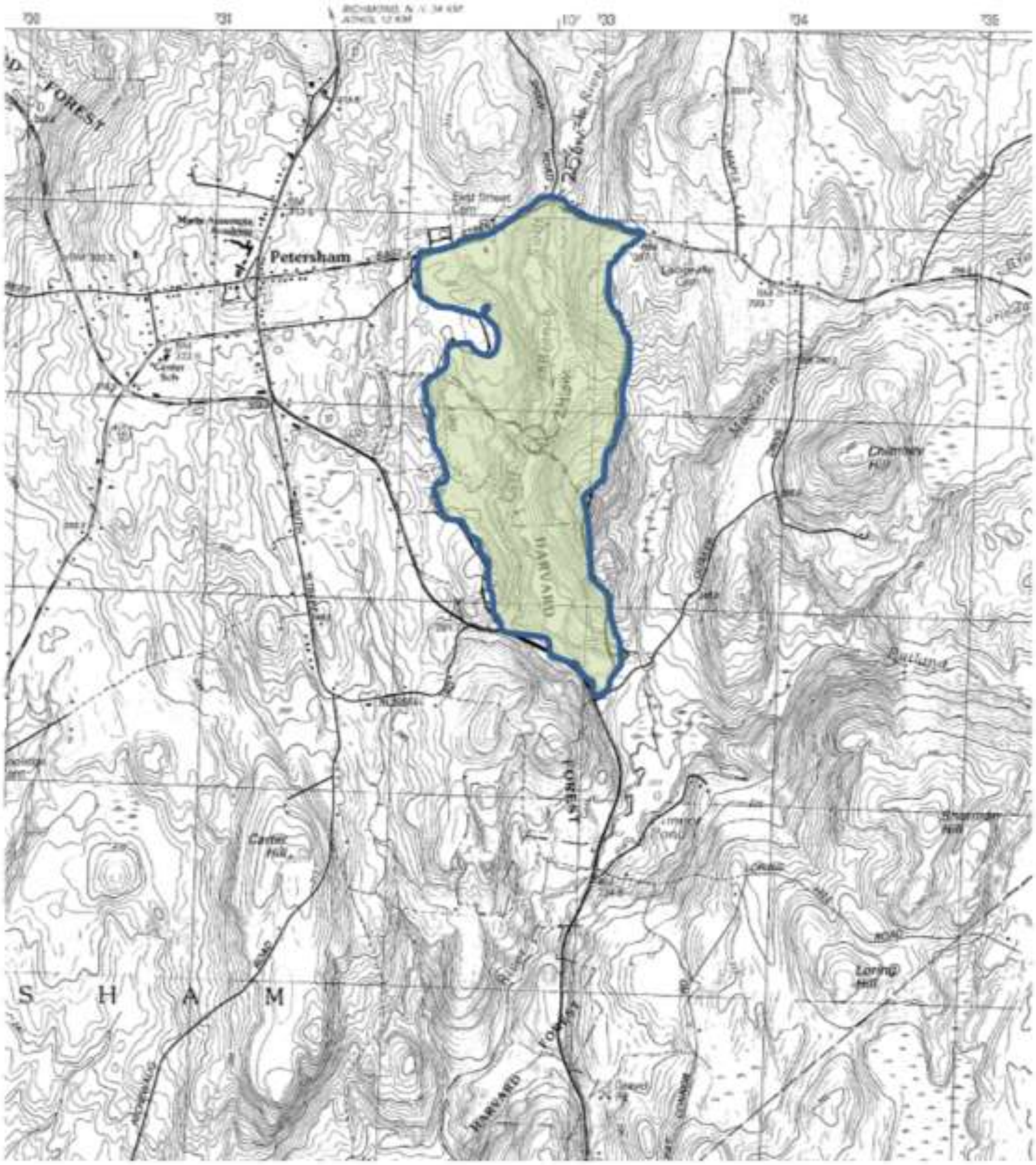
978.248.9455 ▪ central@ttor.org ▪ www.thetrustees.org

A PROPERTY OF THE TRUSTEES OF RESERVATIONS




This map is a product of the Geographic Information System of The Trustees of Reservations. Source data obtained from 1:25,000 scale USGS topographic maps, field surveys, Global Positioning System (GPS), and the Massachusetts Executive Office of Environmental Affairs, MassGIS. Boundary lines and trail locations are approximate. March 2003.

# APPENDIX C: Map of Watershed Area



## APPENDIX D: Hydrologic Calculations

### *SMADA 6.0 for Windows: Watershed Information*

| | |
|---|---|
| Watershed Total Area (acres) | :480.00 |
| Impervious Area (acres) | :1.00 |
| Time of Concentration (min) | :20.3 |
| % Impervious Directly Connected | :100.00 |
| Additional Abstraction | |
| Over Pervious Area (inches) | :0.00 |
| Over Impervious Area (inches) | :0.00 |
| Infiltration Characteristics: | |
| Max Infiltration Capacity (in) | :999.00 |
| SCS Curve Number for Pervious | :70 |
| Initial Abstraction Factor | :0.20 |

### *SMADA 6.0 for Windows: Time of Concentration Calculation Methods*

------ Izzard's equation ------
Time of concentration (minutes) = 18.7
Overland flow distance (ft) = 1340
Retardance Coefficient = .06
Rainfall intensity (in/hr) = 5
Watershed slope = .1322

------ Kerby's equation ------
Time of concentration (minutes) = 25.
Overland flow distance (ft) = 1340
Retardance Roughness Coefficient = .4
Watershed slope = .1322

------ Kirpich's equation ------
Time of concentration (minutes) = 4.3
Overland flow distance (ft) = 1340
Watershed slope = .1322

------ Kinematic equation ------
Time of concentration (minutes) = 41.8
Overland flow distance (ft) = 1340
Mannings overland roughness = .45
Rainfall intensity (in/hr) = 5

Watershed slope =  .1322

------  Bransby Williams equation ------

Time of concentration (minutes) = 8.3
Watershed area (square miles) =  .75
Overland flow distance (ft) =  1340
Watershed slope =  .1322

------  FAA equation ------

Time of concentration (minutes) = 23.9
Overland flow distance (ft) =  1340
Watershed slope =  .1322
Rational Coefficient =  .25

## APPENDIX E: Notice of Intent Procedure

A PDF of the Notice of Intent Application can be found online at
http://www.mass.gov/dep/water/approvals/wpaform3.pdf
The document is 27 pages long and has been included ahead of this section in this project.
The instructions included with the NOI application are fairly straightforward and clear. The application is broken up into sections designed to divide the permit into logical steps.

### *Section A: General Information*

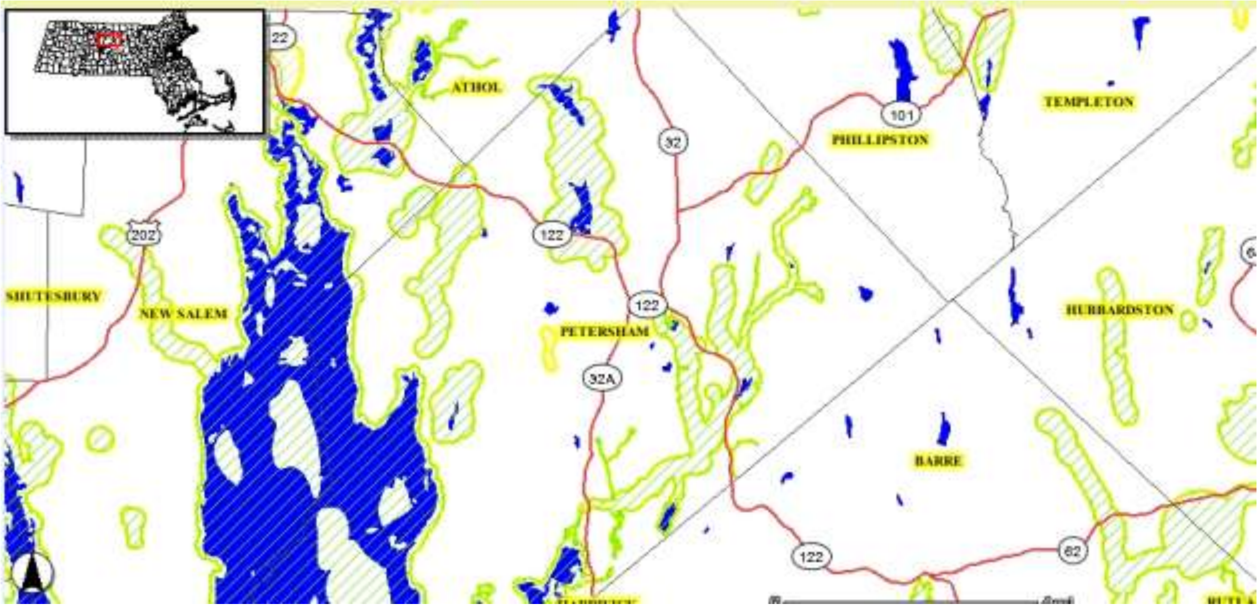
This section is simple. It asks for information on the applicant and property owner, information the Trustees of Reservations should have on hand. It also includes (if filed online) an electronic GIS locator to define the project site. The latitude and longitude coordinates of the site can be used to graphically locate it and they can then be inputted.
This section also includes confirmation that the application fee has been paid to the town and the state. The total fee is based on the type of project, for this project it should be $1450 as this is a Category 4 project (bridge).

### *Section B: Resource Area Effects*

This section should be competed using the plans, specifications and drawings provided in the project proposal. It is looking for information related to the project itself, its disturbance to the environment, and materials used.

### *Section D: Other Applicable Standards and Requirements*

The project area needs to be checked to see if it overlaps an Estimated Habitat of Rare Wildlife. Below is the Habitat Map pf State-listed Rare Wetland Wildlife. (Green outline with hatching).




## *NOI Wetland Fee Transmittal Form*

This form should be completed to send payment to the state and town for the permit application. This fee covers the cost of reviewing the project. Two payments need to be sent separately to the state and town.

## *Stormwater Management Form*

### Section A: Property Information

The proposed project is redevelopment, as it is a replacement for a current structure. The stormwater runoff is not calculated, as there is no overall increase in impervious area due to redevelopment.

### Section B: Stormwater Management Standards

The stormwater is designed so that the stormwater does not cause erosion to the wetlands or discharge point.

The peak discharge and stormwater controls were designed around a 25-year, 24-hour storm.

The amount of impervious area to be infiltrated is equal to the amount of surface area of concrete poured.  Soil group C was used in calculations of infiltration rates.  Other hydrologic numbers used in calculations can be found in Appendix D.

# APPENDIX F: Preliminary Designs

Glulam

$F'_b = F_b\, C_D C_M C_t C_L C_V C_{fu} C_c$

$= F_b \cdot .9 \cdot .8 \cdot 1 \cdot 1 \cdot 1 \cdot 1 \cdot 1$

$= .72\, F_b$

$C_V = \left(\frac{21}{L}\right)^{\frac{1}{x}} \left(\frac{12}{d}\right)^{\frac{1}{x}} \left(\frac{5.125}{b}\right)^{\frac{1}{x}} \le 1$ $\quad x = 20$ for S.P.

Simply Supported with 4 Members

$F'_v = F_v \cdot C_D \cdot C_M \cdot C_t$

$= F_v \cdot .9 \cdot .875 \cdot 1$

$= .7875\, F_v$

30' Span $\quad M = \frac{511.46 \cdot 30^2}{8}$ $\quad = 57,540$ ft-lb $\qquad V = \frac{511.46 \cdot 30}{2}$ $\quad = 7,672$ lb

An excel spreadsheet was set up to conduct repeated calculations

| Span | member size | stress Rating | F'b | F'v | Combined Shear + Moment | |
|---|---|---|---|---|---|---|
| 30' | 5×11 | 16F-1.3E | 1152 | 149.6 | | X |
| | | 24F-1.7E | 1728 | 165.4 | 1.17 | X |
| | | 24F-1.8E | 1728 | 208.7 | .998 | ✓ |
| 35' | 5×11 | 16F-1.3E | 1152 | 151.56 | | X |
| | | 20F-1.5E | 1440 | 165.37 | | X |
| | | 24F-1.8E | 1728 | 208.68 | 1.2 | X |
| | | 26F-1.9E | 1872 | 209.68 | 1.18 | X |
| | 6¾×11 | 20F-1.5E | 1440 | 165.37 | 1.128 | X |
| | | 24F-1.7E | | | 1.06 | X |
| | | 24F-1.8E | 1728 | 208.68 | .91 | ✓ |
| 40' | 6¾×11 | 24F-1.8E | 1728 | 208.7 | 1.23 | X |
| | | 26F-1.9E | 1872 | 208.7 | 1.18 | X |

# Round Timber

**Design Values from NDS**

Southern Pine $F_b = 2400$ psi
$F_v = 110$ psi

$F'_b = F_b\, C_D\, C_T\, C_F\, C_i\, C_{fu}$
$= 2400 \times .9 \times 1 \times 1.18 \times 1 \times .77$
$= 1962.57$ psi

$F'_v = F_v\, C_D\, C_T\, C_i$
$= 110 \times .9 \times 1 \times 1.18$
$= 116.82$ psi

Simply Supported With 4 members
Roof Load + Dead Load
$85\text{ psf} + 24.6\text{ psf} = 109.6\text{ psf}$
Load × Trib Area
$109.6 \times 4.666 = 511.46$ plf

$M = wL^2/8 \qquad f_b = \frac{Mc}{I}$
$V = wL/2 \qquad f_v = V/A$
$f_b/F'_b + f_v/F'_v \le 1$

**30' Span**

$M = \frac{511.46 \times 30^2}{8}$
$= 57{,}540$ ft lb

$V = \frac{511.46 \times 30}{2}$
$= 7{,}672$ lb

Try 16" Ø
$f_b = \frac{57540 \times 8 \times 12}{3217}$
$= 1717$ psi
$f_v = 7672/64\pi$
$= 38.2$ psi

$\frac{1717}{1962.57} + \frac{28.49}{116.82} = 1.11$

Too Small

Try 18" Ø
$f_b = \frac{57540 \times 9 \times 12}{5153}$
$= 1206$ psi
$f_v = 7672/81\pi$
$= 30.1$ psi

$\frac{1206}{1962.57} + \frac{30.1}{116.82} = .85$ good

**35' Span**

$M = \frac{511.46 \times 35^2}{8}$
$= 78318.33$ ft lb
$V = 511.46 \times 35/2$
$= 8950.66$ lb

Try 18" Ø
$f_b = \frac{78318.1 \times 9 \times 12}{5153}$
$= 1641.4$
$f_v = \frac{8950.66}{81\pi}$
$= 35.17$

$\frac{1641.4}{1962.52} + \frac{35.17}{116.82} = 1.13$

Too Small

Try 20" Ø
$f_b = \frac{78318.3 \times 10 \times 12}{8} = 1196.6$ psi
$f_v = 8950.66/100\pi = 28.49$ psi

$\frac{1196.6}{1962.52} + \frac{28.49}{116.82} = .85$ good

**40' Span**

$M = \frac{511.46 \times 40^2}{8}$
$= 102293.2$ ft lb

$V = \frac{511.46 \times 40}{2}$
$= 10229.3$

Try 20" Ø
$f_b = 102293.2 \times 10 \times 12/7854$
$= 1563$ psi
$f_v = \frac{10229.3}{100\pi}$
$= 32.6$ psi

$\frac{1563}{1962.52} + \frac{32.6}{116.82} = 1.07$

Too small

Try 22" Ø
$f_b = \frac{102293.2 \times 11 \times 12}{11{,}499}$
$= 1174.25$ psi
$f_v = 10229.3/100\pi$
$= 32.6$ psi

$\frac{1174.25}{1963.52} + \frac{32.6}{116.82} = .82$ good

## APPENDIX G: Simple Span Design

Simple Span Final


Loading — Forces on members (K)

| Loading | 1 | 2 | 3 | 4 | 5 |
|---|---|---|---|---|---|
| all | .12 | .424 | .447 | .424 | .12 |
| 1-2 | .145 | .201 | -.023 | .006 | -.001 |
| 1-3 | .112 | .455 | .224 | -.071 | .007 |
| 1-4 | .121 | .418 | .47 | .223 | -.026 |
| 2-3 | -.033 | .254 | .246 | -.038 | .009 |

Members 1+5

Moment = 8772.5 ft lb
Shear = 1595 lb

6×12 $f_b$ = 849 psi — combined loading = .89 < 1 good
$f_v$ = 25.1 psi

8×10 $f_b$ = 908 psi — combined loading .96 < 1 good
$f_v$ = 22.3 psi

Members 2+4

Moment = 27527.5 ft lb
Shear = 5006 lb

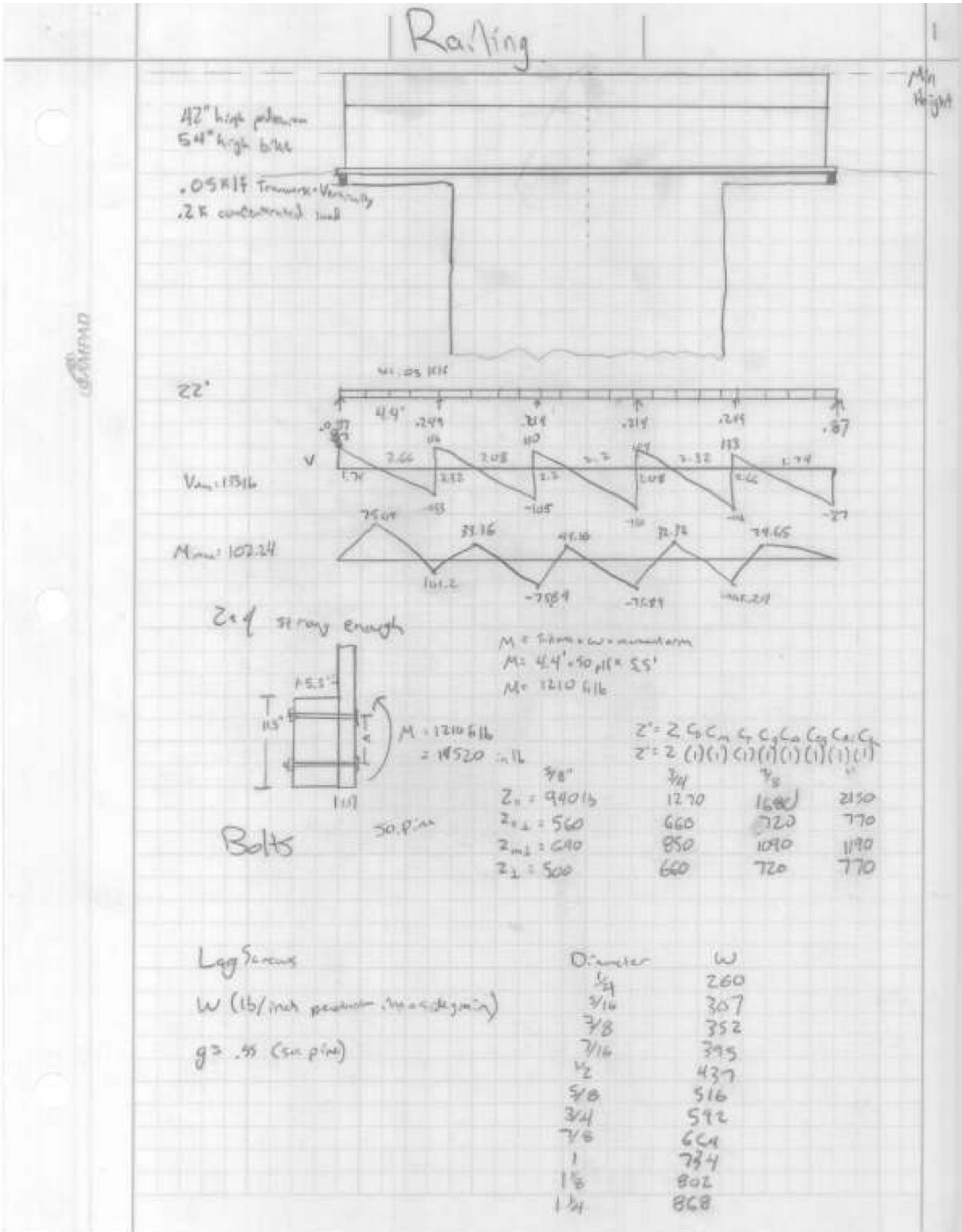
$Z = \frac{27.5 \times 12}{.9 \times 50} = 7.34$ — W6×12 z = 8.30 > 7.34 good

Member 3

Moment = 29,826.5 ft lb
Shear = 5,423 lb

$Z = \frac{29.8 \times 12}{.9 \times 50} = 7.95$ — W6×12 z = 8.30 > 7.95 good

## APPENDIX H: Railing Design


Railing

1

Min Height

42" high pedestrian
54" high bike

.05 K/ft Transverse + Vertically
.2 K concentrated load

W = .05 K/ft

22'

4.4'

V

$V_{max}$ = 133 lb

$M_{max}$ = 102.24

2x4 strong enough

M = 4.4 ft × ω × moment arm
M = 4.4' × 50 plf × 5.5'
M = 1210 ft-lb

5.5"

11.5"

M = 1210 ft-lb
= 14520 in-lb

$Z' = Z\, C_D C_M C_t C_g C_\Delta C_{eg} C_{di} C_{tn}$
$Z' = Z\,(1)(1)(1)(1)(1)(1)(1)(1)$

Bolts

| | 5/8" | 3/4 | 7/8 | 1" |
|---|---|---|---|---|
| $Z_{\parallel}$ | 940 lb | 1270 | 1680 | 2150 |
| $Z_{s\perp}$ | 560 | 660 | 720 | 770 |
| $Z_{m\perp}$ | 640 | 850 | 1090 | 1190 |
| $Z_{\perp}$ | 500 | 660 | 720 | 770 |

Lag Screws

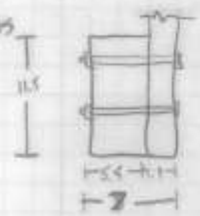
W (lb/inch penetration into side grain)

g = .55 (so. pine)

| Diameter | W |
|---|---|
| 1/4 | 260 |
| 5/16 | 307 |
| 3/8 | 352 |
| 7/16 | 395 |
| 1/2 | 437 |
| 5/8 | 516 |
| 3/4 | 592 |
| 7/8 | 664 |
| 1 | 734 |
| 1 1/8 | 802 |
| 1 1/4 | 868 |

Railing

Bolts




minimum length = 8"

grade 2 steel
min Tensile Strength 74,000 psi

$\frac{1}{4}" \times 8"$ bolts = \$.75/bolt

bolt hole = $\frac{1}{4}" + \frac{1}{8}"$
= $\frac{5}{8}"$

Area bolt hole = $\frac{5}{8}" \times 5.5"$
= 3.4375

Min x = 4"

$\omega_{max} = .145$

$M_{max} = 8.7725$ ft K
$V_{max} = 1.595$ K

6×12 $A = 63.25 - 3.4375$
$= 59.8125$
$I = 5.5 \times 11.5^3/12 - 5.5 \times 4.3125^3/12 + 5.5 \times 3.6875^3/12$
$= 697.06 - 36.75 + 22.98$
$= 683.28 \text{ in}^4$

6×12 Still strong enough w/ 4" spacing between bolts .92 ≤ 1

6×12 Still strong enough w/ 5" spacing between bolts .94 ≤ 1

Shear on wood = 50 plf × 4.4 feet
= 220 lbs

$F_v = 175$ lb/in²
$F'_v = 175 \times 1 \times .97 \times 1 \times 1 \times 1$
$= 169.75$ lb/in²

$169.75 \text{ lb/in}^2 = \frac{220 \text{ lb}}{A_{min}}$
$A_{min} = 1.296 \text{ in}^2$

2×4
Area = $1.5 \times (3.25 - \frac{1}{8})$ ✓
= 4.6875

6×12
Area = $5.5(3.25 - \frac{1}{8})$ ✓
= 17.18

## APPENDIX I: Howe Truss Member Forces

| Member # | Axial (k) | Shear (k) | Moment (ft-k) | Member Size |
|---|---|---|---|---|
| **M1** | 1.57 | 0 | 0 | 2x4 |
| **M2** | -0.835 | 0.266 | -0.244 | Bottom Chord |
| **M3** | -1.329 | 0 | 0 | 2x4 |
| **M4** | 0.835 | 0 | 0 | Top Chord |
| **M5** | 0.942 | 0 | 0 | 2x4 |
| **M6** | -1.337 | 0.266 | -0.244 | Bottom Chord |
| **M7** | -0.798 | 0 | 0 | 2x4 |
| **M8** | 1.337 | 0 | 0 | Top Chord |
| **M9** | 0.314 | 0 | 0 | 2x4 |
| **M10** | -1.504 | 0.266 | -0.244 | Bottom Chord |
| **M11** | -0.532 | 0 | 0 | 2x4 |
| **M12** | 1.337 | 0 | 0 | Top Chord |
| **M13** | 0.314 | 0 | 0 | 2x4 |
| **M14** | -1.504 | 0.266 | -0.244 | Bottom Chord |
| **M15** | -0.797 | 0 | 0 | 2x4 |
| **M16** | 0.835 | 0 | 0 | Top Chord |
| **M17** | 0.942 | 0 | 0 | 2x4 |
| **M18** | -1.337 | 0.266 | -0.244 | Bottom Chord |
| **M19** | -1.329 | 0 | 0 | 2x4 |
| **M20** | 1.57 | 0 | 0 | 2x4 |
| **M21** | -0.835 | 0.266 | -0.244 | Bottom Chord |

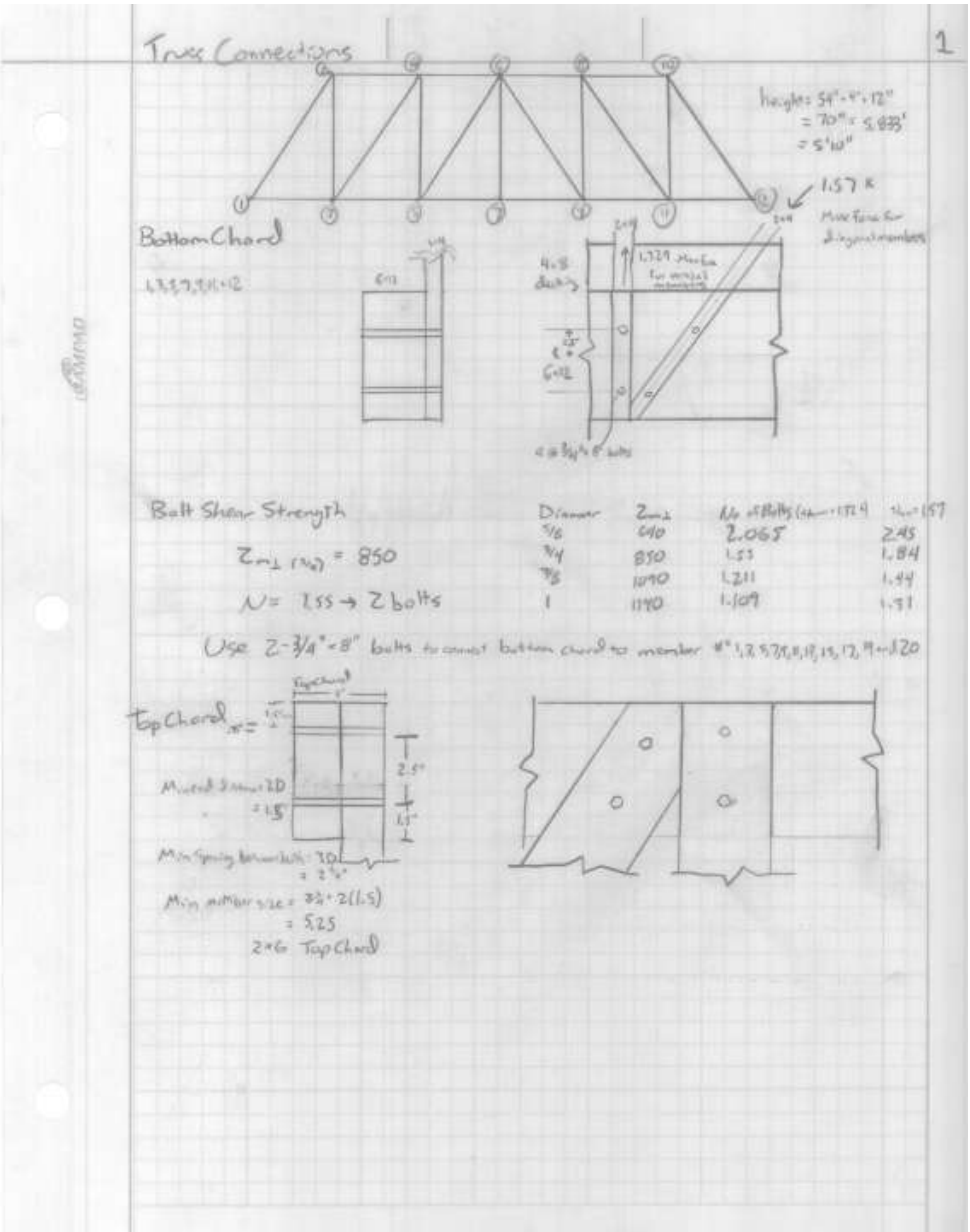

3'8" between vertical members

5'10" high

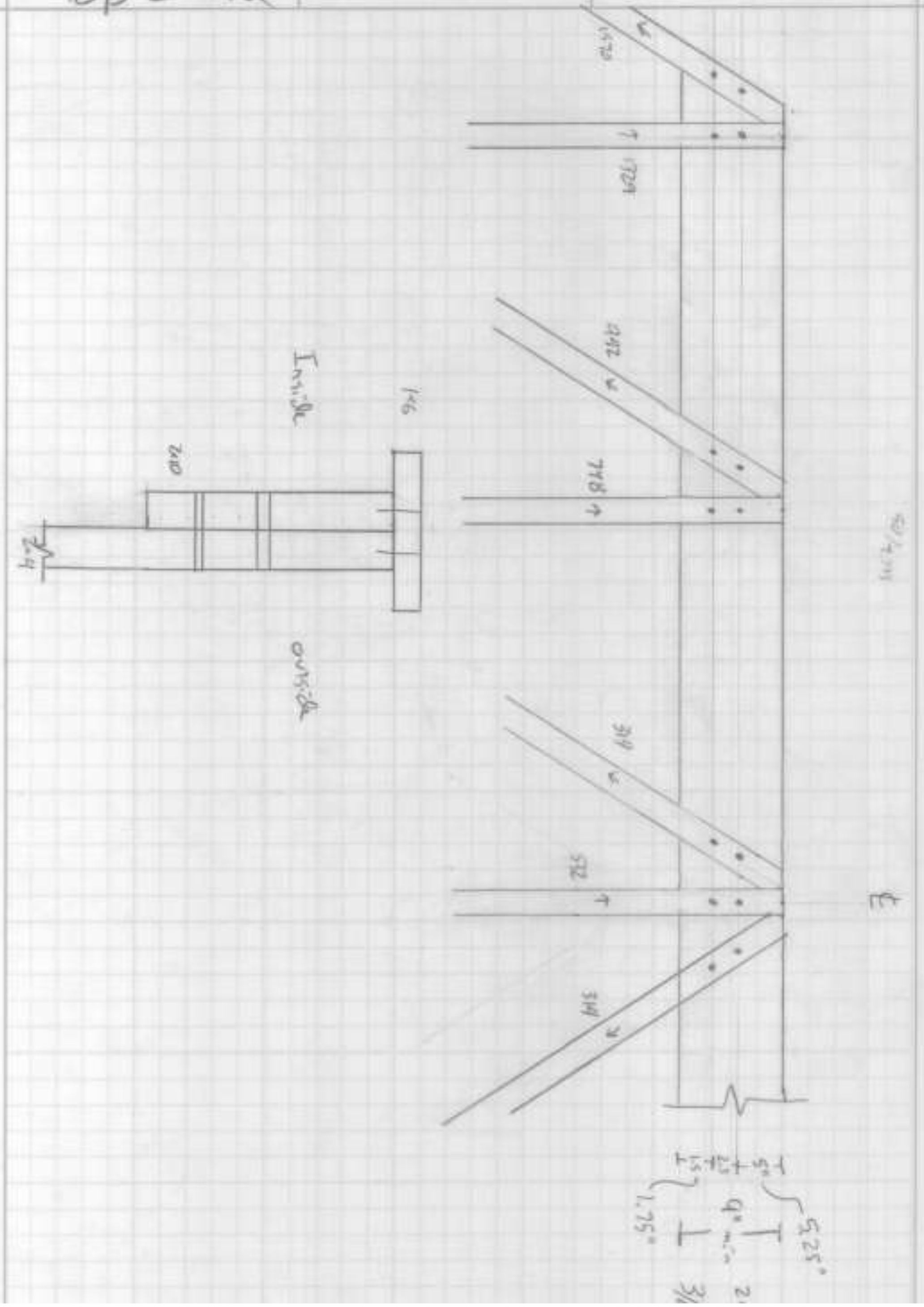
**Bottom Chord needs to be 6x12 to withstand the forces and allow connections**

**Top Chord needs to be a 2x10 to withstand the forces and allow connections**

## APPENDIX J: Truss Connections



Top Chord

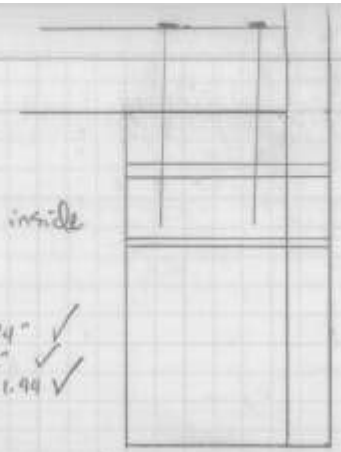


Bottom Chord

# Bottom Chord

3/4" bolts

min spacing for moment = .494" ✓
min length wood for shear = 4.8" ✓
bolt shear → min # 3/4" bolts = 1.44 ✓



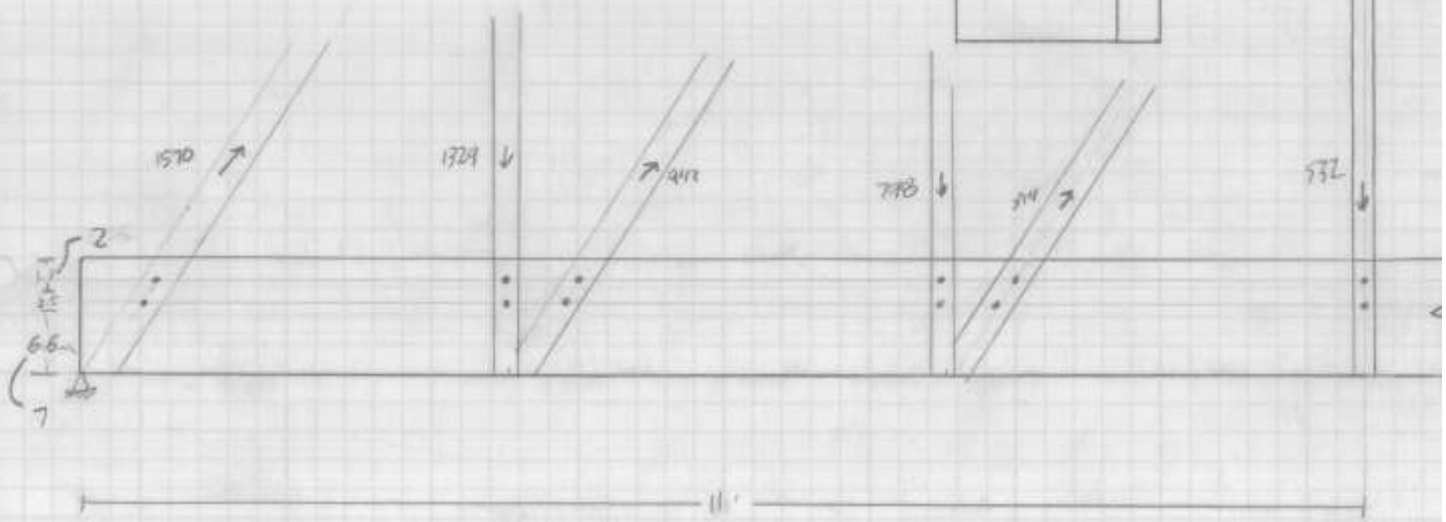
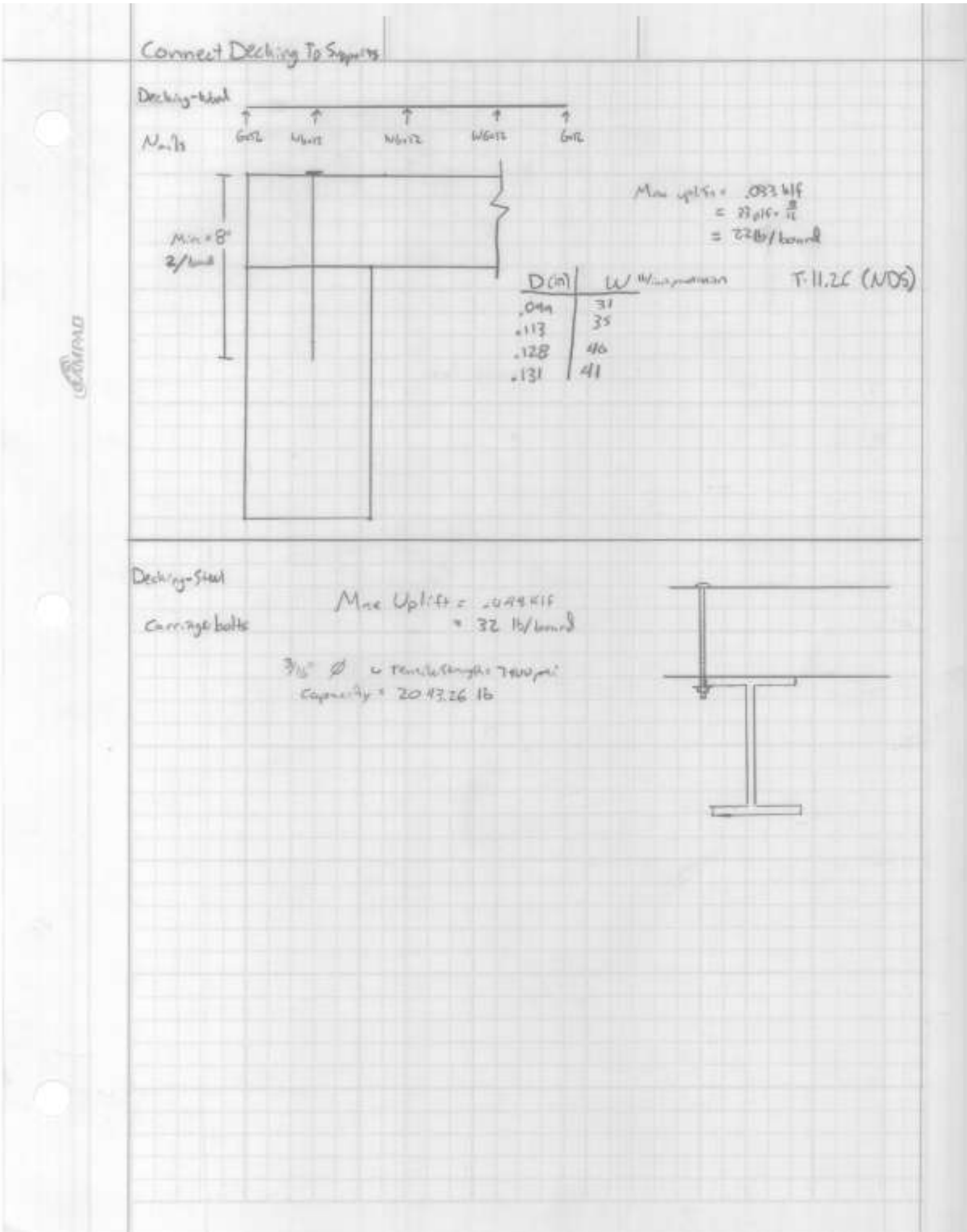



**Table 5: Shear in Wood from Connections**

| Member | Shear Force | F'v | Amin | Member Thickness | Bolt Diameter | Min Length | Min Thickness |
|---|---|---|---|---|---|---|---|
| Railing | 220 | 169.75 | 1.296023564 | 1.5 | 0.25 | 0.989 | 4.989 |
| Member 3 | 1329 | 169.75 | 7.82916053 | 1.5 | 0.75 | 5.594 | 9.594 |
| | 1329 | 169.75 | 7.82916053 | 2.5 | 0.75 | 3.507 | 7.507 |
| | 1329 | 169.75 | 7.82916053 | 3.5 | 0.75 | 2.612 | 6.612 |
| | 1329 | 169.75 | 7.82916053 | 1.5 | 0.875 | 5.657 | 9.657 |
| | 1329 | 169.75 | 7.82916053 | 2.5 | 0.875 | 3.569 | 7.569 |
| | 1329 | 169.75 | 7.82916053 | 3.5 | 0.875 | 2.674 | 6.674 |
| | 1329 | 169.75 | 7.82916053 | 1.5 | 1 | 5.719 | 9.719 |
| | 1329 | 169.75 | 7.82916053 | 2.5 | 1 | 3.632 | 7.632 |
| | 1329 | 169.75 | 7.82916053 | 3.5 | 1 | 2.737 | 6.737 |
| Member 7 | 835 | 169.75 | 4.918998527 | 1.5 | 0.75 | 3.654 | 7.654 |
| | 835 | 169.75 | 4.918998527 | 2.5 | 0.75 | 2.343 | 6.343 |
| | 835 | 169.75 | 4.918998527 | 3.5 | 0.75 | 1.780 | 5.780 |
| | 835 | 169.75 | 4.918998527 | 1.5 | 0.875 | 3.717 | 7.717 |
| | 835 | 169.75 | 4.918998527 | 2.5 | 0.875 | 2.405 | 6.405 |
| | 835 | 169.75 | 4.918998527 | 3.5 | 0.875 | 1.843 | 5.843 |
| | 835 | 169.75 | 4.918998527 | 1.5 | 1 | 3.779 | 7.779 |
| | 835 | 169.75 | 4.918998527 | 2.5 | 1 | 2.468 | 6.468 |
| | 835 | 169.75 | 4.918998527 | 3.5 | 1 | 1.905 | 5.905 |
| Member 11 | 532 | 169.75 | 3.134020619 | 1.5 | 0.75 | 2.464 | 6.464 |
| | 532 | 169.75 | 3.134020619 | 2.5 | 0.75 | 1.629 | 5.629 |
| | 532 | 169.75 | 3.134020619 | 3.5 | 0.875 | 1.333 | 5.333 |
| | 532 | 169.75 | 3.134020619 | 1.5 | 0.875 | 2.527 | 6.527 |
| | 532 | 169.75 | 3.134020619 | 2.5 | 0.875 | 1.691 | 5.691 |
| | 532 | 169.75 | 3.134020619 | 3.5 | 0.875 | 1.333 | 5.333 |
| | 532 | 169.75 | 3.134020619 | 1.5 | 1 | 2.589 | 6.589 |
| | 532 | 169.75 | 3.134020619 | 2.5 | 1 | 1.754 | 5.754 |
| | 532 | 169.75 | 3.134020619 | 3.5 | 1 | 1.395 | 5.395 |

## APPENDIX K: Decking Connections



## APPENDIX L: Soil Analysis Tables

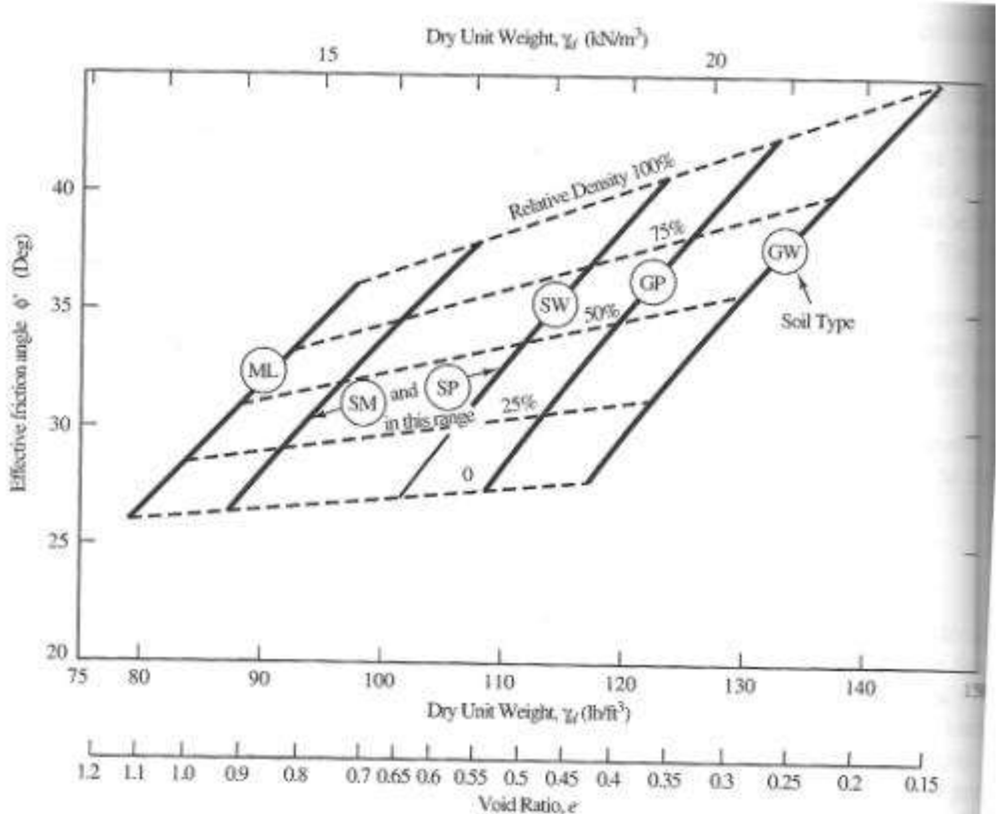


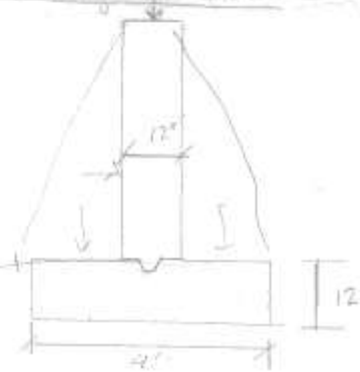

**Figure 3.18** Typical $\phi'$ values for sands, gravels, and silts without plastic fines (Adapted from U.S. Navy, 1982a)

**TABLE 3.8** RELATIONSHIP BETWEEN CONSISTENCY OF COHESIVE SOILS AND UNDRAINED SHEAR STRENGTH (Adapted from Terzaghi & Peck, 1967 and ASTM D2488-90; used with permission).

| | Undrained Shear Strength, $s_u$ | | |
|---|---|---|---|
| Consistency | (lb/ft²) | (kPa) | Visual Identification |
| Very soft | <250 | <12 | Thumb can penetrate more than 1 in (25 mm) |
| Soft | 250–500 | 12–25 | Thumb can penetrate about 1 in (25 mm) |
| Medium | 500–1000 | 25–50 | Penetrated with thumb with moderate effort |
| Stiff | 1000–2000 | 50–100 | Thumb will indent soil about 1/4 in (8 mm) |
| Very stiff | 2000–4000 | 100–200 | Thumb will not indent, but readily indented with thumbnail |
| Hard | >4000 | >200 | Indented by thumbnail with difficulty or cannot indent with thumbnail |

## APPENDIX M: Retaining Wall Design

Retaining Wall Function




External Stability

Sliding $F = \frac{\Sigma(P_R/b)}{\Sigma(P_D/b)}$ $\frac{\text{horizontal resisting}}{\text{horizontal driving}} \geq 1.5$

Coulomb's Method

$$K_A = \frac{\cos^2(\phi - \alpha)}{\cos^2\alpha \cos(\phi_w + \alpha)\left[1 + \sqrt{\frac{\sin(\phi + \phi_w)\sin(\phi - \beta)}{\cos(\phi_w + \alpha)\cos(\alpha - \beta)}}\right]}$$

$\phi = 30°$

$\alpha = 0°$

$\beta = 0°$

$\phi_w = 0°$

$$K_A = \frac{\cos^2(30°)}{\cos^2(0)\cos(0)\left[1 + \sqrt{\frac{\sin 30 \sin 30}{\cos 0 \cos 0}}\right]} = \frac{.75}{(1)(1)\left[1 + \sqrt{\frac{(.5)(.5)}{(1)(1)}}\right]} = \frac{.75}{1 + .5}$$

$K_A = 0.5$

$$P_A/b = \frac{\gamma H^2 K_A \cos\phi_w}{2} = \frac{130(5)^2(0.5)(1)}{2} = 812.5 \text{ lb/ft}$$

$V_A/b = 0$

Weight on Bottom of Footing

Stem: $(1')(4')(150\,lb/ft^3) = 600\,lb/ft$
Footing $(1')(4')(150\,lb/ft^3) = 600\,lb/ft$
Soil behind walls: $(3')(4')(130\,lb/ft^3) = 1560\,lb/ft$
Loading: $1643\,lb/ft$

$= \Sigma = P/b + W/b = 4403\,lb/ft = 4400\,lb/ft$

Shear Friction
$V_f = (P/b + W/b)\mu_a + 0.5\lambda_a D^2$

$D = 5'$

$V_f = (4400\,lb/ft)(0.367) + 0.5(234)5^2$
$1615 + 1462.5$

$\lambda_a = \gamma(\tan^2(45° + \phi/2) - \tan^2(45-\phi))$

$V_f = 3080\,lb/ft$

$\lambda_a = \gamma\left(\frac{3 - 0.3}{1.5}\right) = 234$

$u_a = \frac{\mu}{F} = \frac{0.55}{1.5} = 0.367$

$F = \frac{\Sigma(P_r/b)}{\Sigma(P_b/b)} = \frac{3080\,lb/ft}{812.5\,lb/ft} = 3.8$ ✓

Internal Stability
$V_u/b \leq 0.5\,\phi V_n/b$

$V_n/b = 2\,bwd\sqrt{f'_c}$

$V_u/b = P_u/b = 812.5\,lb/ft$

$V_n/b = 2(12\,in/ft)\,d\sqrt{4000\,lb/in^2}$
$= 1518d$

$812.5\,lb/ft \leq 0.5(0.85)(1518d)$

$812.5\,lb/ft \leq 645.15d$
$d \geq 1.3''$

$T \geq d + d_b + cover = 1.3 + 0.5 + 3 = 4.8''$ ✓ $\boxed{12''}$

Steel
$A_{vf}/b = \frac{V_u/b}{\phi f_y \mu} = \frac{812.5\,lb/ft}{0.85(60{,}000)0.6} = 0.027\,in^2/ft$

Flexural Steel

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

$$= \frac{4000 \cdot 12''}{1.176 \cdot 60{,}000}\left(1.3 - \sqrt{1.3^2 - \frac{2.353\,(2430)}{0.9\,(4000)(12)}}\right) = 0.0353\ in^2$$ ✗

Minimum Steel

$A_s \geq 0.0012\ A_g$

$\geq 0.0012\,(12)(4)$

$\geq 0.0576\ in^2/ft$ ✓

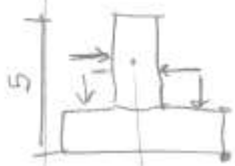
4 #5 bars sufficient @ 3.5"

Sum of Moments (Overturning)

| Source | Force (lb) | Moment Arm | $M_O/b$ ft-lb/ft | $M_R/b$ |
|---|---|---|---|---|
| $P_a/b$ | 810 | 3 ft | 2430 | |
| Stem | 600 | 2 ft ~~3 ft~~ | | 1200 ~~1800~~ |
| Footing | 600 | 2 ~~.5 ft~~ | | 1200 ~~180~~ |
| Soil | 1140 (lb) | 2 | | 2280 |
| | | | | 4680 |

$$F = \frac{\Sigma M_R/b}{\Sigma M_O/b} = \frac{4680\ ft\text{-}lb/ft}{2430\ ft\text{-}lb/ft}$$

$F = 1.9 > 1.5$ ✓




A=

Foundation Design:

Frost Susceptibility:

Soil Group: F2  10-20% fine Gravel (GW)

Information: Not highly susceptible, not least

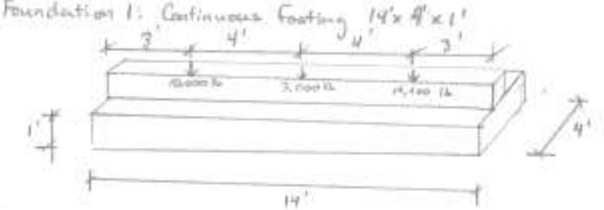
Both Conditions met for ice lense:

1. New Source of Water ✓
2. Soil is frost-susceptible ✓

Suggestions: 1. Dig foundation deep
However, due to foundation laying on a rise in the land this may not help
2. Replace some soil below with frost-free soil (sand / gravel)

Frost Penetration = 5'

Foundation 1: Continuous Footing 14' x 4' x 1'




Design for Shear:

$d = \frac{(P_u/b)(B-c)}{4\cdot 4\phi\sqrt{f'_c} + 2P_u/b}$

$P_u/b = 23{,}000\ lb/14' = 1643\ lb/ft \times$ [illegible]

$B = 48"$

$c = 12"$

$\phi = 0.85$

$f'_c = 4000\ lb/in^2 \times$ [illegible]

$d = \frac{(1643\ lb/ft)(48" - 12")}{4(0.85)4\sqrt{4000\ lb/in^2} + 2(1643\ lb/ft)}$

$d = T - 3in - d_b$

$d = 0.521$ in

Much below min at 6 in

Use $d = 6"$ or $T = 9"$ [illegible]

$d =$ 6 in

$T =$ [illegible]

[illegible]

[illegible] #4

Design of Lateral Steel:

$$\ell = \frac{B - c/2}{2} = \frac{4' - 1'/2}{2} = 1.75' \quad \text{(Table 9.2)}$$

$$M_{uc}/b = \frac{(P_u/b)\ell^2}{2B} + 0 = \frac{(643\,lb/ft)(1.75')}{2(4')} = 357.4\ lbft/ft \quad (9.21)$$

$$A_s/b = \left(\frac{f'_c\, b}{1.176 f_y}\right)\left(d - \sqrt{d^2 - \frac{2.353\, M_{uc}}{\phi f'_c\, b}}\right)$$

$$= \left[\frac{(4000\,lb/in)(48")}{1.176(60{,}000\,lb/in^2)}\right]\left(6" - \sqrt{6^2 - \frac{2.353(357.4)}{(0.9)\,4000\,lb/in^2}}\right) = \quad (9.13)$$

$$2.72\left(6" - 5.78"\right)$$

$$A_s/b = .0533\ \frac{in^2}{ft}$$


Wider than 45° frustum

Min Steel Check

$A_s = \rho B d$

$\rho$ = steel ratio = 0.0020 for grade 40 steel
$B$ = footing width $4' = 48"$
$d$ = effective depth = 9"

$A_s/b = 0.0020(48")(9") = \underline{0.864\ in^2/ft}$ Min governs

$\boxed{\#5 \text{ bars @ } 4"}$ $\#5 - 0.31\,in^2$

$3/ft = .93\,in^2/ft > .864\,in^2/ft$

Check Development Length

$$\frac{c + K_{tr}}{d_b} = \frac{12"}{0.5} = 24" \times \quad 2.5\ \text{max} \checkmark$$

$$l_{d\,sup} = \frac{B - C}{2} - 3\,in = \frac{48 - 12}{2} = 15" \checkmark > 12"$$

$$\frac{l_d}{d_b} = \frac{3}{40}\frac{f_y}{\sqrt{f'_c}}\frac{\alpha\beta\gamma\lambda}{\left(\frac{c + K_{tr}}{d_b}\right)} = \frac{3}{40}\frac{60000}{\sqrt{4000}}\frac{(1)(1)(.8)(1)}{2.5} = 22.8 \quad l_d = 22.8(0.5) = 11.4" \checkmark$$

$11.4" < 12"$

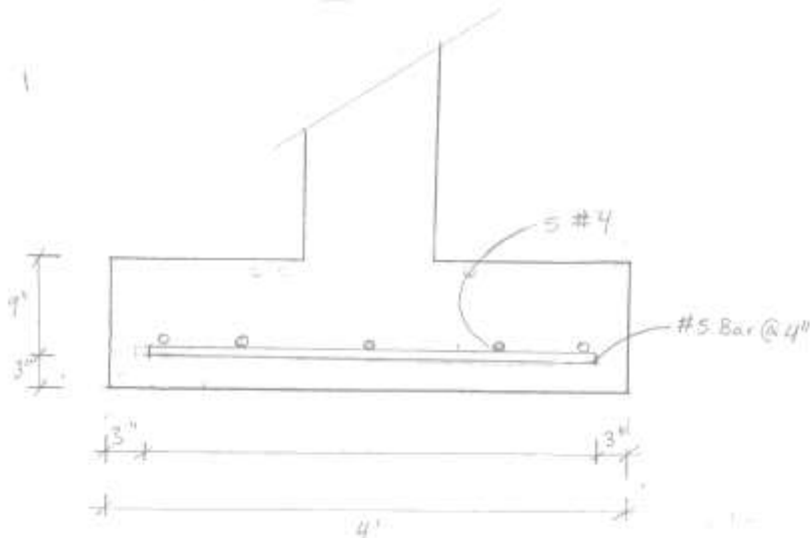
Design Longitudinal Steel

$A_s = \rho Bd = 0.0020(48'')(9'') = 0.864 in^2$

Use 5 #4 bars ($A_s = 1 in^2 > 0.864 in^2$)

add

Final Design



APPENDIX N: Concrete Pour/Gravity Wall Design

Massive Gravity Wall: External stability

<u>Sliding:</u> (Revised Calculations)

$P_a/b = 812.5$ lb/ft (from previous design)


Weight

$(5' \times 4')(150\ lb/ft^3) = 3000\ lb/ft$


<u>Shear Friction</u>

$V_f = (3,000\ lb/ft)(0.367) + 0.5(27-1)\ 5^2$

$V_f = 5126\ lb/ft$

$F = \frac{\Sigma P_r/b}{\Sigma(P_D/b)} = \frac{5126\ lb/ft}{812.5\ lb/ft} = 6.3$ ✓

<u>Overturning</u>

$F = \frac{\Sigma M_r/b}{\Sigma M_D/b}$

| | lb | arm | $M_D/b$ | $M_R/b$ |
|---|---|---|---|---|
| $P_a/b$ | 812.5 | 0.333(5) = 1.67 | 1354.17 | |
| $V_a/b$ | 0 | | | |
| Weight | 3000 lb | 0.5(4) = 2 | 6000 lb/ft → | |

$F = \frac{\Sigma M_R/b}{\Sigma M_O/b} = \frac{6000\ lb/ft}{1354.17\ lb/ft} = 4.43$ ✓

Bearing Capacity

$q_a = 748$ lb/ft² (worst case)

$B' = B - 2e = 4 - 2(.45) = 3.1$
$L' = 1'$ (continuous footing)

$q_{apply} = \frac{P + W_F}{B'L'} - u_b = \frac{3000 \text{ lb/ft}}{(3.1)(1)} = \boxed{967.7 \text{ lb/ft}^2}$ ✓

$\Sigma M_{/F} = 1355 + 3000x - 6000 = 0$

$x = 1.55$ ft

$e = \frac{4}{2} - x = .45$ ft

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