Design Of Ski Ward Hill Parking Lots:
Solving Flooding Issues With Resurfacing and Drainage Applications

A Major Qualifying Project

For Ski Ward Hill

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Abstract

A lack of a reliable drainage system has led to many issues involving water runoff, flooding and the deterioration of the Ski Ward Parking Lot. In this project we designed feasible drainage methods and parking lot improvements in efforts to solve the recurring water problems.

Authorship

Andrew Boynton:

Andrew contributed to all of the main chapters of the report. He took the lead on the design of the Asphalt Cement Resurfacing. He also contributed to the Conclusion and Recommendation sections of the report. Andrew also formatted and edited the report and the presentation board that was used during Project Presentation day.

Mack Eberhardt:

Mack contributed to all the chapters of this report. He took the leading on the design of the Catch Basin. He also contributed to the Conclusion and Recommendation sections of the report. Mack was the main point of contact between the owner of Ski Ward Hill Resort and the MQP team.

Dereck Pacheco:

Dereck contributed to all of the main chapters of the report. He took the lead on the design of the French Drain. He communicated with town engineers for various characteristics of the land (rain occurrence interval, rainfall intensities, elevations, soil characteristics, etc.). He generated an online survey to obtain a drainage class, gravel source, parent material, ponding frequency, sand source, and an overall soil map description of the whole land.

Daniel Page:

Daniel Contributed to all of the main chapters of the report. He took the lead on the design of the Gravel Resurfacing. Daniel also took the lead on finding the equations listed in section 3 which helped us evaluate environmental factors.

Acknowledgements

We would like to thank our advisor Professor Tao, for the guidance throughout this project. We would also like to thank the entire CE Department staff for preparing us for the workforce after graduation.

Executive Summary

Roadways and parking areas are consistently affected by rain, ice, snow and traffic loads. In New England, the quickly varying weather can cause structural failures to the roadways and parking areas, which creates safety hazards and unusable surfaces. The focus of this project was on the parking areas of Ski Ward Hill, a ski hill located in Shrewsbury, Massachusetts. Ski Ward Hill is privately owned and lacks the proper funding to maintain the parking areas on the property. In recent years, the parking areas have experienced water damage from storms, snow melt, runoff water from the hill and traffic loading from cars, which have resulted in multiple sections of the parking areas to be unusable. The goal of this project was to redesign the structures of the two parking areas, Lot A (main parking area) and Lot B (secondary parking area), to improve their durability and drainage. Lot A experiences widespread potholes and water puddling while Lot B experiences a single concentrated area of water build-up. These problems have lowered the amount of customers that visit the ski hill because there is simply not enough space to park. In order to understand why the problems continued to reoccur, the team first analyzed the existing site conditions by gathering information from the national weather service, local town and county land evaluation documents as well as AutoCADD drawings produced by Shrewsbury town engineers. Factors such as elevation, soil conditions and the average amount of water the parking areas encountered during a typical storm, were determined and used to develop the optimal solutions for both parking areas. The durability and drainage of the parking areas will be improved through: 1. Resurfacing the parking areas to improve their structural stability and durability; and 2. Implementing drainage systems in the parking areas to enhance drainability.

The team then established four possible designs, listed below, in which option A and B would increase durability, while option C and D would help with the drainage of the parking areas:

a. Gravel Resurfacing
b. Asphalt Cement Resurfacing
c. French Drain
d. Catch Basin

Each of these designs could be combined with one another, so as a team, we needed to evaluate the strengths and weaknesses of the proposed designs once they were developed. In order to develop the designs, the team had to research and select the materials that would be used. During this process the team had to follow MassDOT's construction standards and requirements for materials and design specifications in order for the designs to be accepted, if implemented in the future. Finally, a cost analysis of each of the designs was performed.

This project concluded that the two designs that would provide the best durability and drainage for the parking areas were option B and D. Option B, Asphalt Cement Resurfacing would be implemented for Lot A and Option D, Catch Basin, would be implemented for Lot B. The recommended designs for each of the parking areas were determined to address both issues (i.e., structural instability and water ponding) that have been undermining the performance of the parking areas of Ski Ward Hill.

Capstone Design Experience

Our capstone design consisted of furthering developing the parking areas of Ski Ward Hill. We were challenged with improving the drainage and durability of the parking areas. During our experience, this Major Qualifying Project face 4 realistic constraints, which are:

Economic

The redesign of Ski Ward Hill Parking Areas addressed the total cost of each of the designs. We determined the costs of the materials associated with each of the designs as well as the labor cost to construct them. The team did a cost analysis of the Gravel, Asphalt Concrete, French Drain and Catch Basin Designs. An example of a material cost was for the Gravel and Asphalt Concrete designs, where the main cost were the aggregates needed for each of the layers.

Constructability

The redesign of Ski Ward Hill Parking Areas allowed for the recommendations for the change of the existing conditions, to produce a more drainable and durable structure. The four designs are all examples of this because the Gravel and Asphalt Concrete resurfacing changed the durability and the French Drain and Catch Basin changed the drainage of the design.

Sustainability

One critical practice in today's society, this report discusses the importance of a sustainable parking lot design. We introduce the problems that can potential impacting a parking area and the solutions necessary to positively impact the areas long term.

Environmental

The environmental aspects are covered in two main areas of this report, French Drain and Catch Basin. The main environmental concern addressed in this report is waste water removal. Examples of this can be seen in the design of the French Drain, which requires a filter at the end

of the piping and the Catch Basin, where the owner needs licenses to remove the water from the Catch Basin.

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1 Introduction

Small, family run businesses are an example of a stereotypical “American Dream”. In these cases, families manufacture, sell, provide service etc. to citizens of the community. However, small businesses are often outpaced economically by larger companies. In order to keep a share of the market, small businesses have to maintain customers and strive for their loyalty. Returning customers and building personal relationships is an important aspect of a private business. Our project revolves around Ski Ward Hill, located on Main Street in Shrewsbury, MA, shown in Figure 1.1.

Figure 1.1: Location of Ski Ward Hill

The family run business, currently owned by the LaCroix family, is one of the oldest operating ski hills in New England and has been in operation since 1939. Since 1990, when the LaCroix’s took over ownership, they have upgraded the facilities to draw in more customers. The upgrades included a snow tubing facility, new triple chairlift, extensive lighting and snowmaking upgrades. In more recent years, Ski Ward Hill has become a year-round destination with summer

tubing, summer skiing and Slopeside Bar & Grill offering lunch, dinner and ice cream with outdoor seating.

The technical improvements to Ski Ward Hill has drawn many more customers from around New England over the years. The parking lots of Ski Ward Hill are essential aspects to the business because all available space must be accessible to support the increase of cars that can park on the property. However, after talking to the LaCroix family, our group was informed that many sections of the parking lots were inaccessible due to inadequate conditions caused by water damage. The parking areas have been consistently damaged during the winter months due to melting on the parking areas as well as the water runoff from the adjacent hill. To remove the snow from the parking areas, the LaCroix's use a plow. The plow however, destroys parts of the parking areas by ripping up the existing gravel and sandy surface. This causes uneven surfaces and allows water to build up in those areas. The parking areas are also affected by rain storms because the existing conditions do not drain the water properly. The follow figures, 1.2 and 1.3 show the parking areas as well as the existing conditions.

Figure 1.2: Lot A without water presence

Figure 1.3: Lot B without water presence

This is how the parking areas appear when there is not water present on site. However, we visited the site during a rain storm and found that the parking areas were severely impacted.

Below, figures 1.4 and 1.5, show how the parking areas are affected over time during a rainstorm.

Figure 1.4: Lot A with water

Figure 1.5: Lot B with water

Large pools of water build up on the parking areas. The LaCroix's have recognized the need for reliable solutions that will permanently prevent recurring water build-up and damage to the parking areas. The goal of this project is to redesign the structures of the two parking lots, Lot A and B, to improve the durability and drainage. In return, the new design will accommodate changes in traffic volume, efficient drain ability and resistance to storms by considering factors such as, materials, environmental impacts and cost.

2 Background

Currently, Ski Ward Hill is experiencing flooding and water damage in the two main parking lots of the property, which has contributed to a decline in customer return rates. This chapter will review the existing conditions of the property and provide an overview of the basic designs of the four options we established as top options to solve the problem at hand. These options are, Re-Gravel, Pavement, French Drain and Catch Basin.

2.1 Existing Conditions of Ski Ward Hill

There are two main parking lots located on the property of Ski Ward Hill. The following figures 2.1 and 2.2 are aerial photographs showing what we denoted “Lot A” and “Lot B”. These images were provided by a Google Earth Satellite.

Figure 2.1: Total Area, Square Feet, of Lot A, the main parking lot of Ski Ward Hill (Google Earth Images)

Figure 2.2: Total Area, Square Feet, of Lot B, second parking lot of Ski Ward Hill (Google Earth Images)

The existing soils of Lot A and B are similar to each other and are Whitman loam [1]. The existing soil conditions are rated as poor to very poor for drainage purposes, leading to the flooding and water damage the LaCroix's faced every year since gaining ownership of Ski Ward. Appendix A provides detailed descriptions of Lot A and B existing conditions. In addition to unsatisfactory existing soil conditions, Main St., a major roadway in Shrewsbury, has a higher elevation than the parking areas of Ski Ward. Main St. averages around 357 ft above sea level compared to the parking areas, averaging 351 ft above sea level. The Department of Transportation of Massachusetts sloped the roadway towards Ski Ward to decrease water puddling and slippery conditions for motor vehicles on the roadways. Consequently, the slope from Main St. allows further water runoff to enter the parking area. The owner, John Lacroix, has repeatedly attempted to decrease the flooding and water damage by covering Lot A and B

with gravel and stone. This is an unreliable solution because the same issue continues to occur year after year. Thus, the LaCroix's believe that more technical, redesign solutions are needed to solve flooding and water damage.

2.2 Parking Lot Pavement Design

Dirt and gravel roads have been the choice of mass travel since as early as 4000 BC [2]. Roads utilizing multiple layers were first pioneered by the Persians under King Darius I for his 'Royal Road' and used as courier roads in his empire. Later during the expansion of the Roman Empire the design was perfected and more widely used for the purpose for transportation for foot, cart and animal traffic [2]. The Romans used rubble stones for the bottom layer, coarse and fine aggregates for the middle layers and limestone for the top layer. Gravel surfaces are sufficient for use only when there is a proper mix of different sizes (gradation) to pack and compact itself into a dense and smooth surface. Gravel surfaces are fitting for low-traffic, light-use roads and surfaces. When properly designed and installed, gravel is able to self-drain. The different sizes of aggregate allows water to permeate through the layers and eventually spread out enough, eliminating puddling or wash-outs. However, because the top surface of a gravel-surfaced pavement is naturally loose and non-bound, this form of design is not suitable for high-traffic, or heavy duty purposes, that increased as civilizations are advanced.

As civilizations grew and transportation methods advanced, the need for a permanent, durable top surface has forced road design to progress. To strengthen the gravel surface that was once used by many societies, asphalt pavement was incorporated into standard layer design. Three of the early pavement top surfaces used to reinforce the layered design included, Tar Macadam, Sheet Asphalt and Bitulithic Pavements [2]. Tar Macadam or Tarmac, was a mixture of coal, wood and petroleum used around the U.K. during the mid to late 1800s. This pavement mixture could only withstand light traffic and was not very durable, resulting in failure within a

few years [3]. The failure of Tarmac lead to the development of Asphalt designs. Sheet Asphalt was first used in the mid 1800's in Paris, France. Sheet Asphalt mixes were a combination of a wearing course, binder course and base course.

1. Wearing Course - Composed of asphalt cement and sand produced in slab pieces
2. Binder Course - Composed of broken stone and asphalt cement
3. Base Layer - Composed of cement concrete or pavement rubble

The base layer thickness for the Sheet Asphalt depended on the weight of the traffic. High volume traffic would call for a larger thickness versus minimal traffic which would require a small thickness [3]. In contrast, Bitulithic Pavements, produced and patented in the United States in 1903, contained a mixture of bituminous (asphalt) cement and graded aggregate [3]. The bituminous cement, petroleum and fine aggregates, was created to resolve the poor durability of the Sheet Asphalt wearing or surface layer course. Since the early 1900's the paving industry has primarily used the Bitulithic Pavement mixture because of its success on roadways, sidewalks, parking lots etc. [3].

Currently, typical pavement designs are constructed with the layers of subbase, base course and surface course. Figure 2.3 shows a standard pavement design approved by the U.S. Department of Transportation.

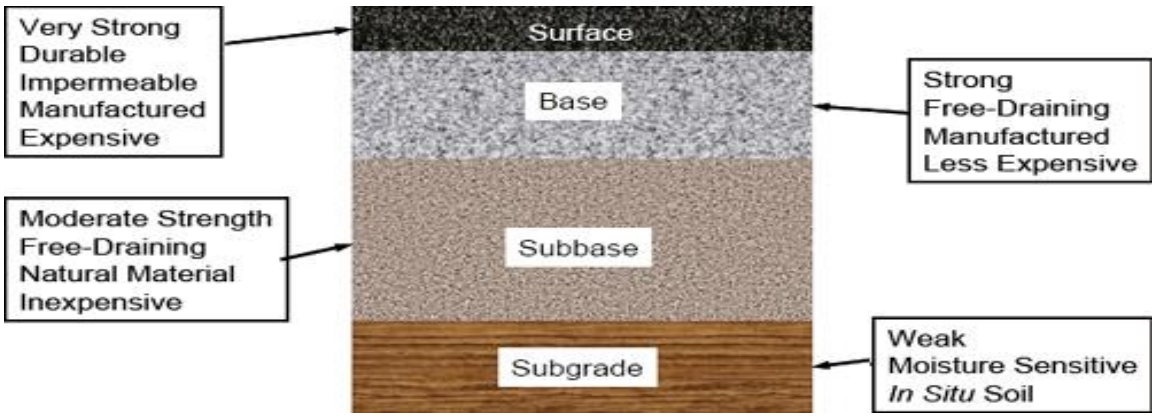


Figure 2.3: U.S. Department of Transportation Federal Highway Administration - Geotechnical Aspects of Pavements

The Subgrade layer is the existing soil or material of the location. The Subbase layer consists of both fine and coarse aggregate materials. Fine aggregates are crushed stones or sand while coarse aggregates are larger stone and gravel materials. The Base layer is composed of both fine and coarse aggregates that are compacted for increased strength. This layer provides the stable foundation needed to support the surface asphalt layer. The Surface course is the final layer and is used to provide strength and added durability to the layered pavement design. Asphalt concrete and Cement are the most common used top surface types [4]. Material selection for each of the layers is determined by the region of the country the pavement is being constructed. Materials for pavement constructed in the Northeast and Northwest must be more durable and stronger due to harsher climates. Compared to materials used in the South, where the climate change throughout the year is more moderate, requiring less durable materials [4]. For Ski Ward Hill, gravel and asphalt designs are viable options for the parking areas. The selection and structural components for the materials of these design options will be thoroughly discussed in the next section.

2.3 Parking Lot Drainage Design

Drainage techniques have been utilized since Ancient Rome, where they dug trenches for water supply, sewage transportation, and much more [5]. It wasn't until the 1700s when the first hollow-pipe drainage system was invented. The innovation of using hollow-pipe was considered a drainage milestone due to its efficiency. Storm and groundwater was able to be transported quicker and be easier manipulated [6]. As years went by, drainage designs began to evolve even more with the help of modern technology. The French drain and catch basin are simply a derivative of previous drainage techniques.

The French drain was created by agriculturalist, Henry Flagg French, in the early 1800's. Currently, this design is primarily used to prevent storm and groundwater from entering and/or

damaging foundations. Even though protecting foundations serves as its primary function, a French drain can be implemented in any situation where there needs to be water diversion. The drain is comprised of a trench filled with coarse aggregates and perforated pipe. Due to advancements in the geotextile industry, Henry's design has been improved and perfected over the years. Now, the design calls for geofabric to line the trench walls. In cases where filtration is of the utmost importance, designs also include geofabrics wrapped around the perforated pipe. Geofabrics are permeable materials that qualify as a great resource for draining.

French drains are commonly placed in areas where flooding occurs. For example, a sloped trench is dug at the base of a valley to serve as a temporary holding tank for flooding water. Geofabric is then placed along the trench prior to the first fill of gravel. When placed properly, the fabric has the ability to separate substances, prevent soil erosion, and provide adequate filtration. A small layer of gravel is dumped on top of the geofabric prior to pipe installation, so it is easier for the perforations to collect water. The perforations are uniformly distributed along the pipe at standard degrees to ensure quality input of storm and/or groundwater. The remaining coarse aggregates fill the rest of the trench to complete the drain. The function of the coarse aggregates is to hold trench walls, keep the pipe in place, and even serve as another filtration device. When coarse aggregates surround the pipe, it creates enough room for water to collect and flow through the perforations. Once the water enters the pipe, it travels down the slope of the trench to the desired location.

There are many applications French drains can be used for, but all of them achieve the same result. Figure 2.4 below depicts two kinds of French drains used in the irrigation industry. There are two common French drains, a basic one where filter cloth rests at the top and an open

one where river stone is placed in on top in addition to the filter cloth. American Irrigation Services provide these services to prevent one's yard from flooding

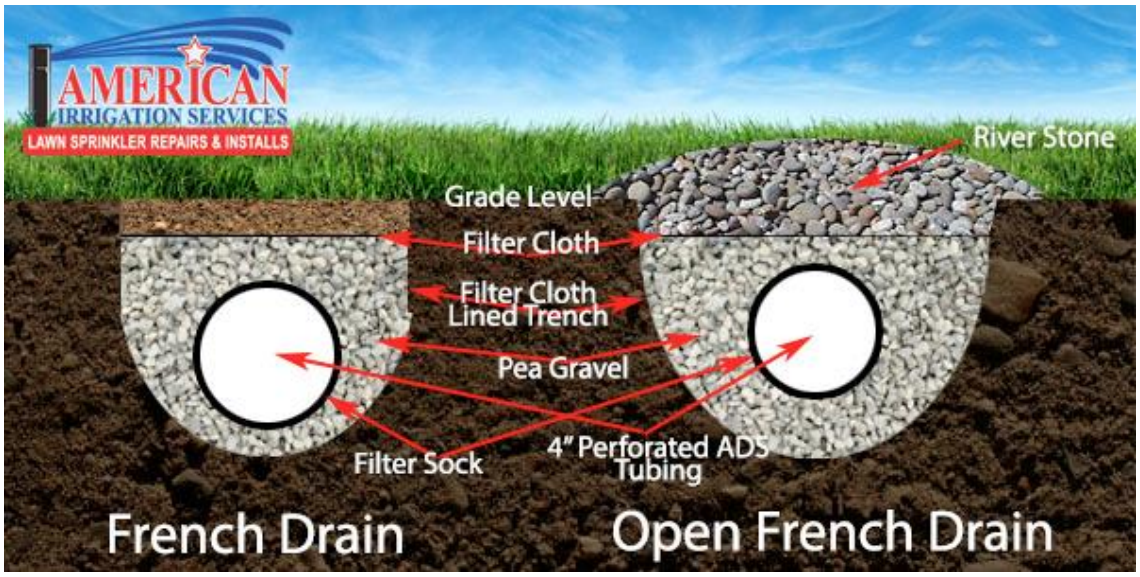


Figure 2.4 Example French Drain, American Irrigation Services
(http://lawnsprinklerstampa.com/drainage/)

. Another application can be seen in Figure 2.5. This company uses the French drain to prevent foundation walls from settling, leaning, or cracking. It even protects the wall face from stains and discoloration. The next application of a French drain is known in the transportation industry, which is why we employed it for Ski Ward Hill.

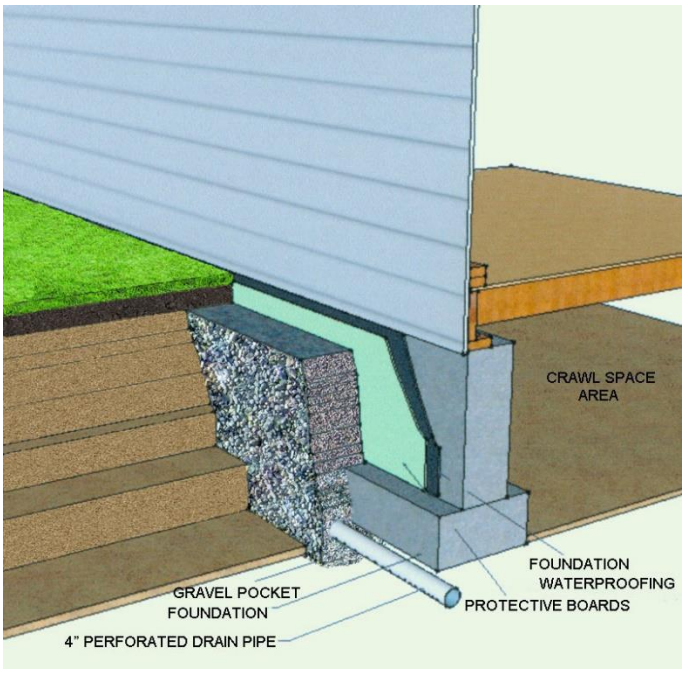


Figure 2.5: Example French Drain, Schneider Construction, Inc.
(http://www.schneiderconstructioninc.com/drainage/pages_drainage/french_drains.htm)

In addition to French Drains another drainage system widely used is Catch Basins. Catch Basins can be traced back to ancient times. Early civilizations dealt with seasonal flooding by digging irrigation canals and building catch basins to ensure crops had enough water for the remainder of the year. The manhole covers for these catch basins started off as slabs of stone or pieces of wood allowing access to covered trenches that carried sewage. These early forms of catch basins lasted from 3500 BCE through the 1750's-1850's CE until the modern manhole and catch basin was developed and patented in the early 19th century [7]. The modern catch basin is typically composed of three main parts: a grate, an outlet trap, and the basin itself which holds the storm-water. The grate is usually made from cast iron and the basin is made from precast concrete [8]. Figure 2.6 illustrates a basic catch basin design based on MassDOT Construction Standards [9].

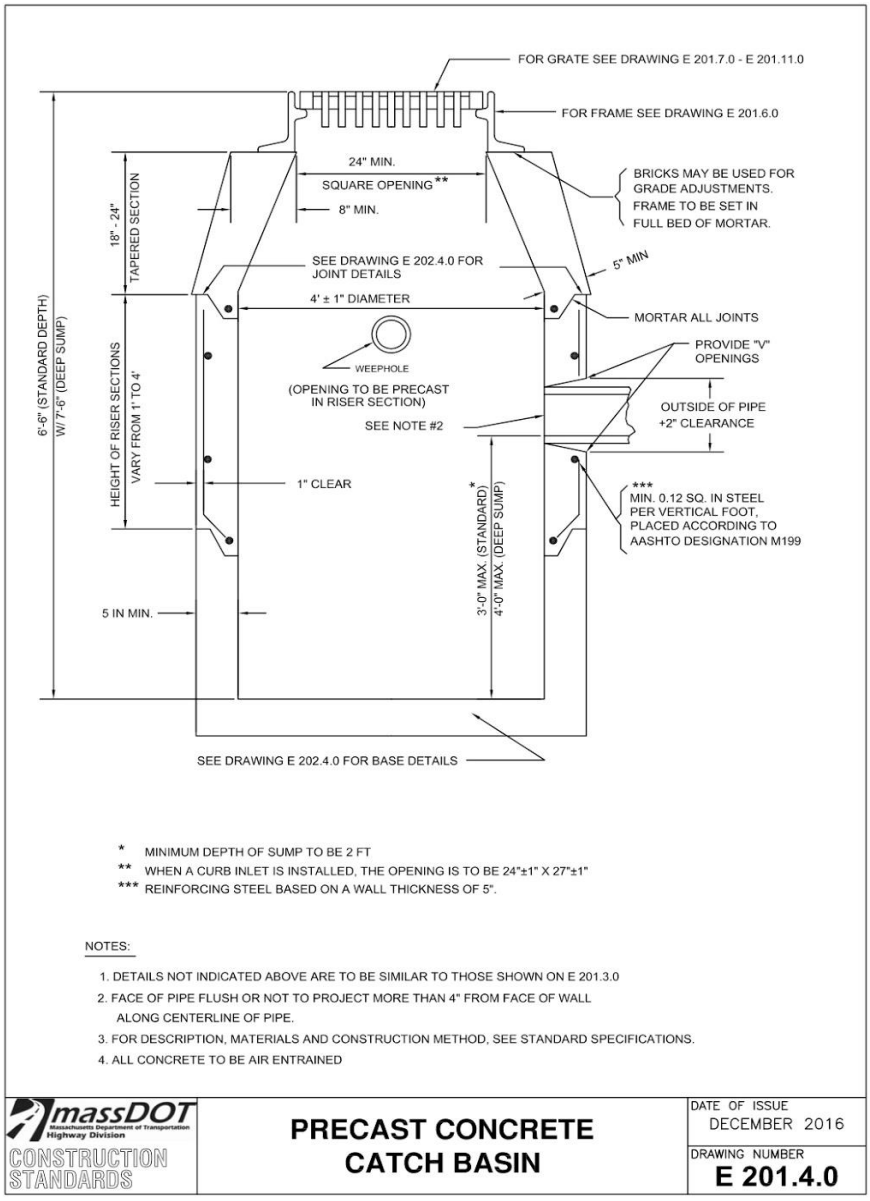


Figure 2.6: Precast Concrete Catch Basin, MassDOT Construction Standards

The purpose of a catch basin is to accept storm-water flows and catch debris that should not be transferred to local receiving waters. The storm-water and debris is then collected and stored in the receptacle, reservoir, basin, or pit beneath the grate. Catch basins minimize sewer clogging and provide basic stormwater pretreatment by trapping larger matter with an inlet grate

and allowing sediment and other smaller material to settle in a sump located below the invert elevations of all outlet pipes [8]. Figure 2.7 illustrates how a catch basin functions.

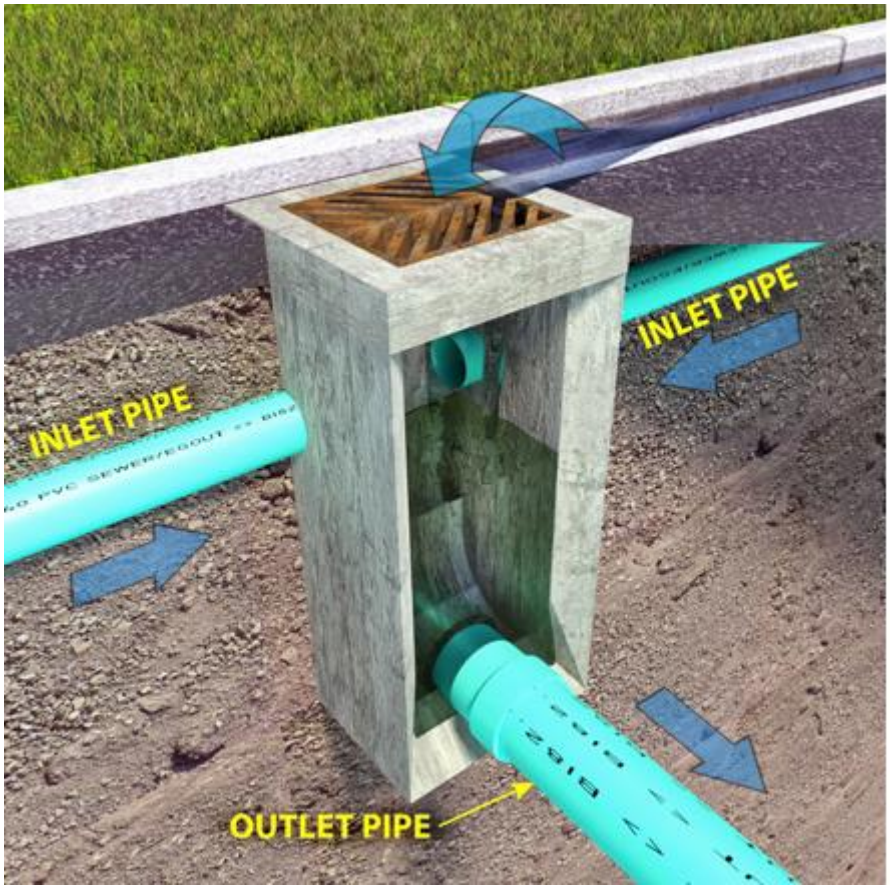


Figure 2.7: Catch Basin Functions

The primary controlled pollutants that catch basins store are coarse sediments (catch basins are not meant to trap or store hazardous material or chemical waste). Because of this, before storm-water is removed from a catch basin that is not connected to a common sewage system, it must be treated. There are a few options for storm-water treatment. A couple of these options include settling: the process of letting the sediment settle in the bottom of the sump; or infiltration: the process of storm-water infiltrating through a pervious bottom of a leaching type catch basin. Once the storm-water is properly treated, it can be removed through a number of methods. One example is through the use of a sewer vacuum attached to a vacuum truck, which

is usually offered by environmental companies. Overall, a regularly cleaned out catch basin can achieve up to approximately 50% removal of coarse sediment [10].

Since the purpose of a catch basin is to catch storm-water flow, the most effective location for one is at the lowest point of the target area. These are called drop inlet catch basins. The ideal situation here is for the storm-water to flow downhill, directly into the catch basin because of gravity. Sometimes, lots need to be regraded for this. Drop inlet catch basins are most common in parking lots [10].

2.4 Summary

Although the LaCroix's have been able to maintain the parking areas enough for customers, there are still feasible designs advancements that need to be made to finally solve the problem. Flooding and poor existing conditions have caused damage to the parking areas, leading to a decrease in customers at Ski Ward Hill. However, Gravel, Pavement, French Drain and Catch Basin designs provide the LaCroix's the ability to solve their issues with flooding, water damage and faulty drainage. In the next chapter, we will discuss how we went about designing the optimal Gravel, Pavement, French Drain and Catch Basin solutions for Ski Ward Hill parking areas.

3 Methodology

Ski Ward Hill has encountered constant flooding and water damage to the two parking areas located in between Main St. of Shrewsbury and the ski hill. Most of the flooding is caused by runoff water from the roadway, which is elevated above the parking areas, as well as water from melting snow from the hill. The parking lots are not equipped to withstand large amounts of runoff water in addition to the rainfall and snowfall that is common in the Northeast. Without a proper drainage system and poor existing surface conditions, large areas of both parking areas are closed, resulting in a decrease of customers to Ski Ward Hill. The goal of our project was to provide the LaCroix's with engineering solutions that can prevent further flooding and water damage. In order to provide potential solutions, we evaluated parking lot pavement and drainage designs. For the pavement design of the parking areas, we chose Gravel and Asphalt resurfacing, while we chose the French Drain and Catch Basin systems for the drainage. In the following section, we will analyze how we designed our four options, Gravel, Asphalt, French Drain and Catch Basin by structural and material analysis.

3.1 Parking Lot Pavement Design

This section will discuss the two options we chose as possible solutions, Gravel and Asphalt Resurfacing. We will discuss the process that was undertaken in identifying the necessary structural components and appropriate materials that would solve the problem of water damage and inadequate structural stability. The following subsections will detail the criteria and processes of design for Gravel Resurfacing and Asphalt Cement Resurfacing respectively.

3.1.1 Identify Structural and Material Components for Gravel Resurfacing

In order to design a gravel surface which would improve the existing system, we had to identify which issues were contributing to the poor performance of the current surfacing design, research appropriate solutions for those issues, then design those solutions per accepted design standards. The gravel was primarily designed for strength, drainability, and gradation (proper

mix) of layers. These design parameters correlate to proper aggregate material selection and appropriate particle size distributions of selected aggregate materials. Geosynthetics were designed in order to provide proper layer separation and strength reinforcement of the designed gravel layers. These had to be designed for durability during installation as well as functionality in everyday use. The design requirements met will be discussed later in this chapter.

Our first directive was to assess the state of the current gravel surfacing system and identify which issues contributed to that deficient state. Following a site walk and meeting with owner John Lacroix, we identified several key issues plaguing the site. The design team found issues of ponding (standing water), potholes, and a parking lot surface uneven and difficult to walk on. After the team identified these issues, contributing factors to such issues were investigated. The Federal Highway Administration (FHWA) Gravel Design Handbook, Koerner's Designing with Geosynthetics, Rollings et al. Geotechnical Materials in Construction, and published lectures from geosynthetics researchers regarding proper gravel design were all consulted [11][12]. Factors contributing to such issues of ponding, drainage, and surface damage were identified. This gave direction as to how to design properly engineered solutions for those issues. The contributing factors identified are laid out in the following paragraphs.

Ponding is a result of poor drainage in the lot, an issue which directly correlates to poor surface sloping and poorly designed surface and subsurface aggregate layers. An absence of drainage layers combined with the use of aggregate mixes with low permeability (low water flow through the material) contribute to this issue. Research showed that potholing present in the lot is the result of poorly supported surface layers in the gravel mix. Poor support results from a poorly prepared subbase and a lack of supplemental support for the soil. This leads to settling and compacting during use, allowing potholes to form. The uneven surface and presence of large

and small aggregate on the surface were found to be the result of aggregate settling and the intermixing of different aggregate materials. This mixing of different materials is called sacrificial aggregate. Without proper separation of subbase, base layers, and the surface layer, smaller and larger stones will intermix, compromising drainage capacity and creating a surface that is difficult and uncomfortable to walk on. Per FHWA, the team found that proper design of aggregate layering as well as the proper design and implementation of geosynthetic materials for support and separation would mitigate the issues we found plaguing the site.

Once the team understood the necessary solutions to for the gravel design, the team engineered those solutions to sufficiently serve the gravel system. The first step of design was the setup of gravel layering. Two layers were designed for proper gravel design a surface layer and base layer. These can be seen in Figure 3.1.

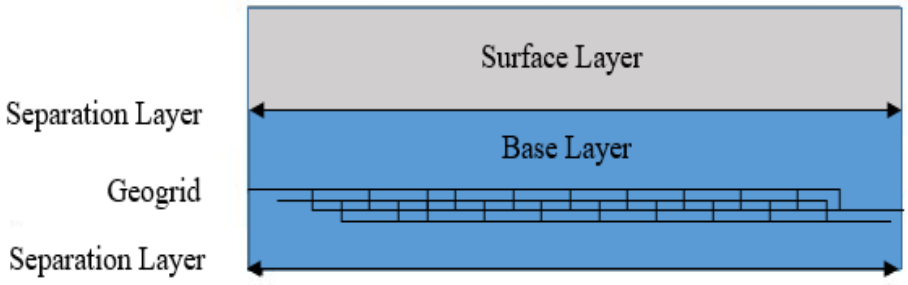


Figure 3.1 Cross section of proper gravel layering and components

With each layer's different purpose, there were specific criteria that had to be met. The base was to be designed for strength and drainability. It must be designed for strength because it is the primarily load-bearing layer in a proper gravel layering design. It must also be designed with high-enough water flow capacity to act as a proper drainage layer, preventing the ponding issues identified in the existing gravel system. A plasticity index of ~0 (no fine particles, i.e., those passing through No. 200 sieve) was necessary to allow for proper drainage, meaning there could be minimal fines (small particles, silt, and loam) in this aggregate mix. As a primary load-

bearing layer, the aggregate selected had to also adhere to MassDOT standards for strength properties, as according to MassDOT base layer material standard M02.01.1. In the Table 3.1.1 the standard requirements for the aggregate are outlined.

Table 3.1: Coarse Aggregate Sieve Analysis Size and Allowable % Passing for Base Layer

MassDOT Distribution Requirements Standard M02.01.1				
Sieve Identification	Standard Sieve Sizes (in.)	Lower Limit Percent Passing	Upper Limit Percent Passing	
2	2.00			
1 -1/2	1.50	95.00	100.00	
1	1.00	35.00	70.00	
3/4	0.75	0.00	25.00	

The surface layer was designed to be compactable into a dense and smooth surface for use and not become muddy in rainy conditions. Per FHWA standards, a surface material with a specified plasticity index of ~ 3 (being moderate clayey) and a gradation with even size distributions of some fines, medium, and larger size gravel was necessary to achieve this. Table 3.2 outlines that requirements for the surface gravel material, as per MassDOT standard M.02.01.7.

Table 3.2: Surface Material specified by MassDOT requirements for Surface Layer

MassDOT Distribution Requirements Standard M.02.01.7				
Sieve Identification	Standard Sieve Sizes (in.)	Lower Limit Percent Passing	Upper Limit Percent Passing	
2	2.00	100.00	100.00	
1 -1/2	1.50	70.00	100.00	
3/4	0.75	50.00	85.00	
No. 4	0.19	30.00	55.00	
100	0.01	8.00	24.00	
	0.00	3.00	10.00	

Another critical part of the design was the implementation of geosynthetics for both separation and reinforcement within the layers. Published lectures regarding the proper use of

geosynthetics were consulted for selection of types of geosynthetics, as well as Rolling's' and Koerner's Geotechnical Materials in Construction and Designing with Geosynthetics, respectively [13] [14]. Geofabric was chosen as the proper material for separation and geogrids were selected as the proper material for reinforcement. The proper geofabric was selected in accordance with standards set out by Koerner [13] for tearing, puncture, burst, and tear resistance which would result from the selected aggregate, using equations 3.1.1, 3.1.2, 3.1.3, and 3.1.4, respectively, shown below.

$$T_{req'd} = p' * (d_v)^2 * f(\varepsilon) \qquad \text{Eq. 3.1.1}$$

Where,

$T_{req'd}$ = required tensile force for tearing (lb)

p' = applied pressure (psi)

$d_v = .33\, d_a$ = maximum void diameter (in)

d_a = average stone diameter (in)

$f(\varepsilon) = .52$ @ 33% strain ~ standard allowance

$$F_{req'd} = p' * (d_a)^2 * S_1 * S_2 * S_3 \qquad \text{Eq. (3.1.2)}$$

Where,

$F_{req'd}$ = required vertical force resisted for puncture (lb)

p' = applied pressure (psi)

d_a = average stone diameter (in)

$S_1 = \frac{h_h}{d_a}$ where $h_h \leq d_a$ protrusion factor

$S_2 = \frac{0.31}{d_a}$

S_3 = shape factor of aggregate

$$T_{req'd} = \frac{1}{2} * p' * (d_v)^2 * f(\varepsilon) \qquad \text{Eq. (3.1.3)}$$

Where,

$T_{req'd}$ = required tensile force resisted for burst (lb)

p' = applied pressure (psi)

$d_v = .33\, d_a$ = maximum void diameter (in)

d_a = average stone diameter (in)

$f(\varepsilon)$ = .52 @ 33% strain ~ standard allowance

$$E = 85 * (d_a)^3 * h \qquad \text{Eq. (3.1.4)}$$

Where,

E = energy released from falling object to be withstood (psf)

d_a = average stone diameter (ft)

h = height of object dropped to surface (ft)

The appropriate geogrid was selected according to design parameters for tensile strength and aperture size. These were calculated according to equations 3.1.5 and 3.1.6.

$$T_{req'd} = \sigma_z R\Omega \qquad \text{Eq. (3.1.5)}$$

Where,

$T_{req'd}$ = tensile force required for reinforcement (lb/ft)

$\sigma_z = 2 * \gamma_{ave} * R$ = vertical stress

γ_{ave} = unit weight of covering soil

R = radius of differential settlement (ft)

$\Omega = 0.25\left[\frac{2y}{R} + \frac{B}{2y}\right]$

y = depth of settlement void

B = width of settlement void

$$B_{GG} \geq 3.5 * D_{50} \qquad \text{Eq. (3.1.6)}$$

Where,

B_{GG} = required aperture minimum width (in)

D_{50} = diameter of 50% passing sieve test of aggregate

The designed geofabric was also vetted per the AASHTO M-288-06 standards published by the American Association of Highway and Transportation Officials, shown in Appendix B. This standard lays out necessary strength criteria for geosynthetics used in construction, depending upon their intended workload. After these calculations were performed, geosynthetics providers were contacted and proper geosynthetics were selected according to these parameters. Cost estimates were then prepared also. They can be found in the results section of this report.

3.1.2 Identify Environmental Parameters and Materials for Asphalt Concrete Pavement Resurfacing

In order to design the optimal model of Asphalt Concrete Pavement for the parking areas of Ski Ward Hill, we first needed to develop the design criteria for our proposed Asphalt Concrete Pavement. The two most important factors we established were based of the problems the existing parking areas faced, which included inadequate drainage and poor durability from car loading and environmental conditions. The design required us to find materials that not only drained water properly but were also strong enough to structurally support traffic loads.

We first analyzed the environmental conditions of our location to determine the peak runoff water and rainfall intensities that were common for Ski Ward Hill's location. These values were important because we needed to know which materials would be able to successfully drain the water without it puddling up on the surface. We contacted a town engineer to determine the recurrence of intense rainfalls which occurred in Shrewsbury as well as the time duration of the

rainfalls. With those parameters, we were able to determine the amount, in inches, of rainfall for the average storm. We calculated the rainfall intensity using the following formula provided by the NOAA National Weather Service [15].

$$I = D/T \quad \text{Eq.} \qquad (3.1.7)$$

Where,

I = rainfall intensity
D = Depth of Rainfall (inch)
T=Time (hr)

We calculated the runoff water with the Rational Equation (U.S. Department of Transportation).

$$Q = CIA \qquad \text{Eq. } 3.1.8$$

Where,
Q = stormwater runoff (gal/min)
C = rational runoff coefficient
I = rainfall intensity (in/hr)
A = drainage area (sqft)

The Rational Runoff Coefficient is a constant that is based on the soil conditions and drainage slope of the area being evaluated. We used the existing conditions of Ski Ward Hills parking areas to determine the specific coefficient needed for the formula above. The solved for Rainfall Intensity was used for variable I. We then converted the total square feet of parking Lot A and B to acres. The total Runoff was calculated for the two parking lots separately because the areas were different, which resulted in different Runoff peaks. Once these factors were found we began the process of selecting the materials that would allow for the necessary drainability and durability.

We based our design off of a common structural design for Asphalt Cement Pavement, illustrated in Figure 2.3, in which there are three layers, surface (finish), base and subbase. In

order to identify the proper thickness and materials needed in each of the layers, we reviewed the Massachusetts Department of Transportation (MassDOT) Specifications for Highway Design. Within this handbook, specification 32 12 00, Appendix C outlined the materials and thickness for each layer.

Table 3.3: MassDOT's Requirements for the Materials of the Layers in an Asphalt Cement Pavement Structure

Layer	Material (s)	Thickness
Finish Layer	• Asphalt Cement Concrete A. Fine Aggregate B. Coarse Aggregate C. Cement Mix D. Admixtures	• 1 inch
Base	• Binder • Fine Aggregate • Coarse Aggregate	• 2 inches
Subbase	• Coarse Aggregate	• 12 inches (1ft)

Shown in Table 3.3 each layer was composed of either fine or coarse aggregate. Fine aggregate are smaller sized, primarily sand or stone dust that are used to fill the space between the gaps created by the coarse aggregate's jagged structure, as reinforced strength. In contrast, coarse aggregates are much larger and provide strength to the layers. Gaps between the coarse aggregate materials allow for water to drain through. Determining the correct balance between these types of aggregates was important for increasing the drainability and durability of the design.

To determine the specific types of aggregates that would be used for each layer, a Gradation Distribution test was performed for each type of aggregate that was required within a particular layer. We evaluated the Division III Material Specifications of MassDOT's Highway and Design handbook, to identify the aggregates acceptable physical requirements that would satisfy each layer of the parking lot design. This section of the handbook provided the sieve analysis data of

the materials, which was the determination of the size of the particles of a given fine or coarse aggregate sample. The materials we would eventually select had to follow these size requirements because if the particles were too small or there was no variance in the distribution of the sizes of the particles, the design would not be nearly as strong or able to drain water as we would have liked. In the following tables, the data from the sieve analysis's of the aggregates for the Finish, Base and Subbase layers are provided.

1. Finish Layer: Required both fine and coarse aggregates

Table 3.4: Fine Aggregate Sieve Analysis Size and % Passing for Fine Aggregates of the Finish Layer

MassDot Standard Distribution Requirements				
Sieve Identification	Standard Sieve Sizes (in.)	Lower Limit Percent Passing	Upper Limit Percent Passing	Acceptable Percent Passing
3/8	0.375	100%		100.0%
No. 4	0.187	95%	100%	97.5%
No. 16	0.0469	55%	80%	67.5%
No. 50	0.0117	10%	25%	17.5%
No. 100	0.0059	2%	8%	5.0%
No. 200	0.0029	0%	3%	1.5%

Table 3.5: Coarse Aggregate Sieve Analysis Size and % Passing for Fine Aggregates of the Finish Layer

MassDOT Standard Distribution Requirements				
Sieve Identification	Standard Sieve Sizes (in.)	Lower Limit Percent Passing	Upper Limit Percent Passing	Acceptable Percent Passing
1/2	1.5			
3/4	0.75	.90	1.00	.95
1/2	0.50			
3/8	0.38	.20	.50	.35
No. 4	0.187	0	.10	0.05
No. 8	0.0937	0	.05	0.03

2. Base Layer: Required both fine and coarse aggregates

Table 3.6: Fine Aggregate Sieve Analysis and % Passing for Fine Aggregates of the Base Layer

MassDOT Standard Distribution Requirements				
Sieve Identification	Standard Sieve Sizes (in.)	Lower Limit Percent Passing	Upper Limit Percent Passing	Acceptable Percent Passing
3/8	0.375	0.95	1.00	97.50%
No. 8	0.0937	0.70	0.95	82.5%
No. 50	0.0117	0.20	0.40	30%
No. 200	0.0029	0.02	0.16	9.00%

Table 3.7: Coarse Aggregate Sieve Analysis and % Passing for Coarse Aggregates of the Base Layer

MassDOT Standard Distribution Requirements				
Sieve Identification	Standard Sieve Sizes (in.)	Lower Limit Percent Passing	Upper Limit Percent Passing	Acceptable Percent Passing
2	2			
1	1	1.00		100%
3/4	0.75	0.80	1.00	90%
5/8	0.625			
1/2	0.5	0.55	0.75	65%
3/8	0.375			
No. 4	0.187	0.28	0.50	39%
No. 8	0.0937	0.20	0.38	29%
No. 16	0.0469			
No. 30	0.0234	0.08	0.22	15%
No. 50	0.0117	0.05	0.15	10%
No. 100	0.0059			
No. 200	0.0029	0.00	0.05	2.50%

3. Subbase Layer: Required on coarse aggregate

Table 3.8: Coarse Aggregate Sieve Analysis and % Passing for Coarse Aggregates of the Subbase Layer

MassDot Standard Distribution Requirements				
Sieve Identification	Standard Sieve Sizes (in.)	Lower Limit Percent Passing	Upper Limit Percent Passing	Acceptable Percent Passing
3	3	N/A	N/A	
2	2	N/A	N/A	
1 1/2	1 1/2	0.70	1.00	85.00%
1/4	1/4	0.50	0.85	67.50%
No. 4	0.187	0.30	0.60	45.00%
No. 200	0.0029	0.00	0.10	5.00%

As shown in the following tables, 3.3-3.8, the size (in.) of the sieve layers directly connected to the percent passing of the particles that went through the individual layers of the sieve. The larger sieve sizes collected very few amounts of the particles, higher percent passing, causing the other sieve layers to have more particles. The lower and upper limits of the percent passing of the particles was provided to show the minimum and maximum percent passing of an acceptable sample. Each aggregate sample would have to meet these limits in order to satisfy the design requirements. We used these values compared to local aggregate distribution center's gradation summaries of their aggregates on hand. We were able to clearly see if any aggregates would meet the distribution requirements set by MassDOT. After finding a local aggregate distributor that had aggregates which adhered to MassDOT's gradation distribution requirements, we determined that there would be acceptable drainability and durability. In the Results chapter, graduation distribution graphs of each of these tables will be provided to show how the chosen aggregates for the design compared and successfully met MassDOT's requirements.

After the evaluation of the different types of aggregates for each of the layers of the design was completed, we had to determine the type of cement mix and admixtures for the Finish

layer and the type of binder for the Base layer. We researched cement distributors around Massachusetts and found one company that had mixes approved by MassDOT, which included the required fine and coarse aggregates we chose for the finish layer. As for the Base layer, the binder type and binder content were provided in MassDOT's Division III Materials Specifications, Appendix D.

3.2 Drainage

This section will describe the drainage system and the methods used to create the optimal system for the parking areas. We chose the French Drain and Catch Basin Systems as the two options that would provide the necessary drainage for the type of conditions present at Ski Ward Hill.

3.2.1 French Drain

When designing our French drain, there are many factors that need to be taken into consideration. First of all, determining site conditions like topography, soil characteristics, and even annual rainfall were some essential steps prior to working out the design. Once site conditions were determined, the research of state/town drainage regulations commenced. Here we began gathering information that served as criterion or constraints for the final design. After there was enough criterion to build on, the design portion of the project was introduced. This included the research of sustainable materials that can provide adequate drainability, withstand current site conditions, and be as efficient as possible in the process. The final steps included obtaining final cost estimates and probable lead time to make this job possible. Due to budgeting constraints, two separate French drain designs were prepared for this project. The following flowchart depicts the process used for both designs.

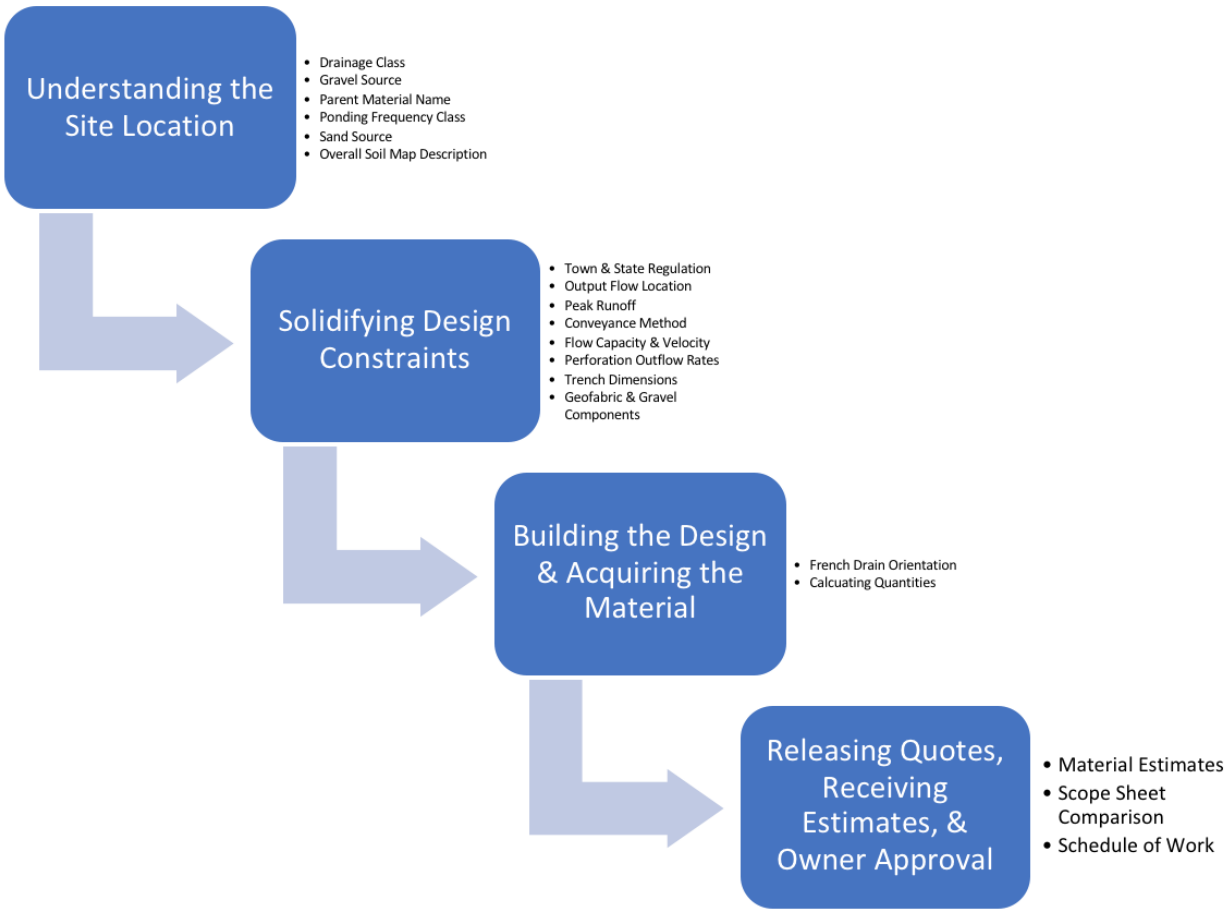


Figure 3.2: This figure shows the process used for the design of both French Drain options

3.2.2 Location of the French Drain

The first few weeks were dedicated to understanding the project site. Our team travelled to Ward Hill Ski Resort post rain storm and thoroughly examined its condition. Sketches of where flooding occurred were taken, along with other the physical characteristics of the land.

In addition to site visits, we contacted the Town of Shrewsbury's Engineering Department and acquired some relevant information about the land. Brad Stone [16], a conservation specialist in the department, provided the team with a link to Web Soil Survey (WSS). WSS provides soil data and information produced by the National Cooperative Soil Survey. It is operated by the United States Department of Agriculture (USDA) Natural

Resources Conservation Service, holds 95% of the country's soil maps and data, and is the single authoritative source of all soil survey information [17]. We used WSS to find the following:

- **Drainage Class -** Refers to the frequency and duration of wet periods under conditions similar to those under which the soil formed. The seven classes include:
    a. recognized excessively drained
    b. somewhat excessively drained
    c. well drained
    d. moderately well drained
    e. somewhat poorly drained
    f. poorly drained
    g. very poorly drained
- **Gravel Source -** Soils are rated "good," "fair," or "poor." A rating of "good" or "fair" means that the source material is likely to be in or below the soil.
- **Parent Material Name -** A term used for the general physical, chemical, and mineralogical composition of the unconsolidated material, mineral or organic, in which the soil forms.
- **Ponding Frequency Class -** Ponding frequency classes are based on the number of times that ponding occurs over a given period. Frequency is expressed as "none," "rare," "occasional," or "frequent."
- **Sand Source -** Soils are rated "good," "fair," or "poor." A rating of "good" or "fair" means that the source material is likely to be in or below the soil.
- **Overall Soil Map Description -** Provides general understanding of the land. Includes slope percentages, elevation levels, mean annual precipitation, mean annual air

temperature, frost free period, depth to water table, depth to restrictive feature, and so on [17].

**NOTE: Please see Appendix [E] for further information on all descriptions for WSS results, as well as a table of values with a color coded aerial view of the land surveyed.*

The next characteristic we needed to know about the land was its elevation status. FreeMapTools.com [18] provided us with elevation tools to gage the slope throughout the lot.

3.2.3: Solidifying Design Constraints

With the relevant soil characteristics obtained, the next step was to solidify design criteria through the guidance of town and state regulations. One of the biggest issues faced with drainage systems is managing the output flow. There are countless rules and regulations that prohibit drainage discharge into waters and wetlands without the proper equipment. We used Massachusetts Stormwater Handbook, Vol. 1 [19] along with a few tips from town engineer, Brad Stone, to determine a location for our design's output flow. Not only did we need to find a location, but we also needed to take into consideration town policies on discharge rates. Allowable discharge rates are inquired to avoid erosion caused by outflow.

Once the output location and allowable discharge rates were found, we were able to start configuring pipe materials and layment grade. ADS Pipe is one of the most well-known and reliable manufacturers in the U.S. and Canada. We utilized this manufacturer because they supply all products necessary to complete the design, as well as their strict policy to abide by all ASTM and AASHTO standards.

In order for us to gauge a pipe size, calculations for peak runoff were completed using Eq. 3.1.8. After conversing with Brad Stone, our group decided to implement designs that accommodated a 25 year storm. Having the constraint of using a 25 year storm, and with the help of National Oceanic and Atmospheric Administration's (NOAA's) National Weather Service, we

were able to acquire the site's average rainfall intensity [20]. The rational runoff coefficient was determined using surface characteristics obtained from WSS and a runoff coefficient table [21], see Appendix F. Using the measurement application in google maps formulated the acreage in both drainage areas, see Appendix G. Peak runoff was finally calculated in cubic feet per second (cfs).

Collecting peak runoff then allowed our team to utilize the Conveyance Method. "Conveyance provides a convenient means of selecting a variety of pipe options that will satisfy a project's flow requirements" [22]. Minimum and maximum velocity requirements were also taken into consideration for pipe selection. If flow velocity is too high, it can cause durability problems over time as well as produce possible erosion at the discharge point. If flow velocity is too low, then the pipe could get easily clogged and ruin a smooth flow. ADS provided the following equations in their ADS, Inc. Drainage Handbook, Appendix H.

$$Q=1.486*A*R2/3*S1/2n \qquad \text{Eq. (3.2.1)}$$

$$R=AP \qquad \text{Eq. (3.2.2)}$$

$$V=Q/A \qquad \text{Eq. (3.2.3)}$$

Where,

Q= Pipe flow capacity (cfs)
n=Manning's value (roughness coefficient)
A=Cross-sectional flow area of the pipe (ft^2)
R=Hydraulic radius (ft)
P=Wetted perimeter (ft); Pipe inside circumference, for full flowing pipe conditions.
S=Pipe slope (feet/foot)
V=Velocity of flow

**NOTE: Further information on each variable can be found in ADS, Inc. Drainage Handbook.*

Once the pipe's flow capacity was determined, it subsequently became a design constraint. The pipe size that was chosen needed to have a higher flow rate capacity than the runoff rate acquired by a 25 year storm. This ensures the system will not overflow and flood the area.

Now that a pipe size has been chosen, the next step was to calculate the free outflow from the perforations in the pipe. Understanding the flowrate through each perforation helped determine how much water is able to exit and enter the pipe at a given time. ADS Pipe provided the following equation used on their products.

$$Q_p = C_d * A * 2gH \qquad \text{Eq. (3.2.4)}$$

Where,

Q_p=free outfall flow rate through one perforation (ft3/s)
C_d=Coefficient of discharge (given)
A=Cross sectional area of one perforation (ft2)
g=Gravimetric constant (given)
H=Height of water above perforation, head (ft)

The final design constraints then moved toward the excavation of the trench. Areas of consideration when making a trench include knowing the appropriate dimensions for the specific pipe, making sure the depth to the water table is within regulation, and even frost factors. This ensures that the drainage system will be able to withstand the elements post installation. The report previously generated from WSS gave us both water table and frost depth factors. ADS pipe supplied the remaining information through a table that includes recommended trench widths based off pipe size, see Appendix I. Acquiring trench dimensions then gave us material quantities for both the geofabric and gravel component of the design.

Geofabric, specifically non-woven, are designed to filter soil particles from drainage systems. They have high resistance to any kind of puncture or tear, and increases the life of

roadways [23]. Geofabric is essential to have in this design so soil particles will not enter the perforations in the pipe, causing it to clog. The gravel component is also pretty essential because not only does it serve as another filter, but it also serves as reinforcement for the trench walls. Aggregates will push the fabric up against the wall and tightly seal every crevice of soil particles in the trench, while providing a load against the walls at the same time. The size of the gravel component needs to be bigger than the perforation openings in the pipe to ensure no aggregates slip through, and cause a disturbance in the flow of water.

3.2.4: Building the Design & Acquiring the Material

Securing all lot characteristics and design constraints, led our team to begin acquiring material and building designs. The point slopes of the lot as well as flood patterns seen during site visits helped us determine how both designs should be oriented along the lot. The recommended trench width provided by ADS pipe, and the soil characteristics from WSS led us create a trench with the best possible dimensions. Linear footage was then calculated through google earth for each design, giving us all necessary quantities for material selection. Excavation quantities (ie. dirt removal by cubic yard) was considered for the trench simply by multiplying the trench's area by its linear footage. The same calculation was done to determine the amount of gravel. However, in order to calculate the allowable area in the trench with the pipe, the external area of the pipe had to be subtracted. Certain areas for pipe elbows were taken into consideration when coming up with the final list of materials.

Since we already knew ADS pipe was going to be our manufacturer for materials, acquiring it was fairly easy. Brad Stone, as well as ADS Sales Representor John Stelmokas [24], assisted our team by solidifying constraints and describing what the standard French drain consists of. Their advice, as well as our previous research handed us all the proper materials

needed to fulfill the job in accordance with town and state regulations. All products with the exception of coarse aggregates were chosen via ADS pipe Catalog [25]. Worcester Sand & Gravel was the supplier chosen to provide the aggregates. Since gravel a key component to each design, the pipe material chosen had to be able to withstand the load on the walls. A list of materials for each design was then recorded and released to the suppliers for estimates.

3.2.5: Releasing Quotes, Receiving Estimates, Owner Approval

In addition to quoting for materials, our team also contacted local contractors as well a landscaping companies to get a final price for labor. The owner of Ski Ward Hill informed our team that he owns an excavator, which gave us to the option to exclude labor cost for trench excavation. Due to this, separate estimates that included installation with and without excavation were asked from each prospective company. In addition, each prospective subcontractor was asked to provide lead times and a schedule for how long it would take them to finish the job. Once all parties extended their estimates, scope documents were made for comparison. The only step that was left was to get one of the designs approved by the owner and begin construction.

3.3 Catch Basin

Once a drainage system design was developed for Lot A, our team decided to use a catch basin design for Lot B. As opposed to Lot A, where there was a continuous slope across the whole lot, Lot B had two significant depressions in the landscape which were determined to be most easily drained through the use of catch basins. As you can see in Figure 3.2, ponding naturally occurs as storm water flows to these locations in Lot B.

Figure 3.3: Lot B Depressions

3.3.1 Catch Basin Design

Once our team determined that the best way to drain Lot B would be with catch basins, we had to design the actual catch basin. Figure 3.4 is a flowchart that illustrates the step-by-step process we followed when designing the catch basin for each half of the lot.

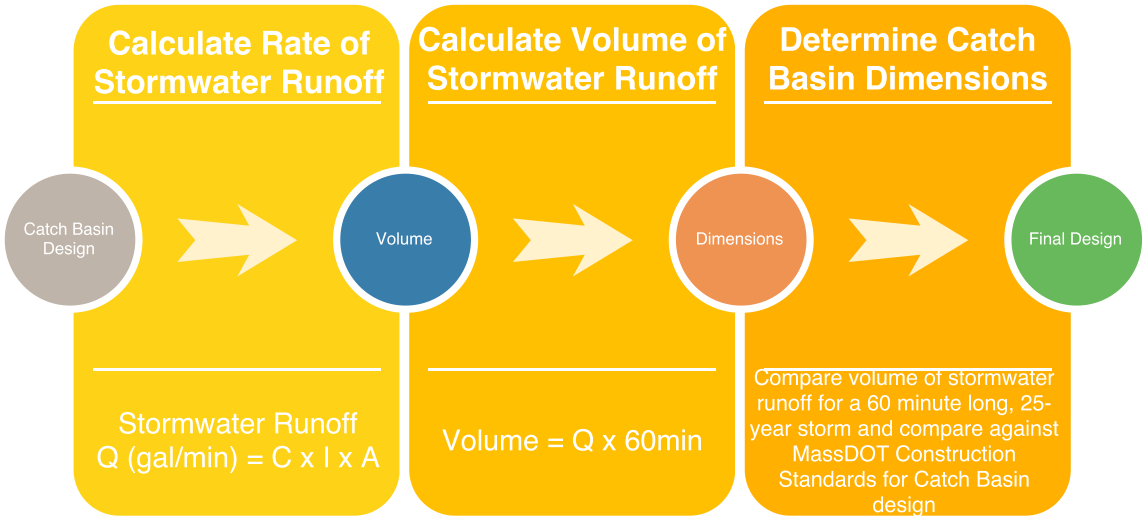


Figure 3.4: Catch Basin Design Process

Step 1: Calculate Stormwater Runoff

The first step of this process was determining the stormwater runoff for a 25-year storm. Equation 3.1.8 was used to determine this value. The variable "C" – the coefficient of runoff – is dependent on the surface material of the lot, which affects the runoff. Figure 3.4 is a table provided by NDS, which lists these values based on the surface material [26].

Coefficient of Runoff (C)= Runoff / Rainfall			
Soil Texture	**C**	**Soil Texture**	**C**
Concrete, Asphalt, Roof	1.00	Loam – Bare	0.60
Gravel – Compact	0.70	Loam – Light Vegetation	0.45
Clay – Bare	0.75	Loam – Dense Vegetation	0.35
Clay – Light Vegetation	0.60	Sand – Bare	0.50
Clay – Dense Vegetation	0.50	Sand – Light Vegetation	0.40
Gravel – Bare	0.65	Sand – Dense Vegetation	0.30
Gravel – Light Vegetation	0.50	Grass Area	0.35
Gravel – Dense Vegetation	0.40		

Figure 3.5: NDS Coefficient of Runoff Values

The variable "I" – rainfall intensity – is determined by using Equation 3.1.7. Finally, the variable "A" – drainage zone area – was determined through a Google Earth mapping process. An outline of Lot B was traced on Google Earth and thus produced a square footage of area in Lot B. An excel spreadsheet was then prepared to account for the different possible designs that we considered for Lot B's surface, Appendix J, and several stormwater runoff values were produced.

Step 2: Calculate Total Volume of Stormwater Runoff

The next step of the process was to determine the volume of water that would accumulate in Lot B based on the rainfall intensity of a 25-year storm for a duration of 60 minutes. Equation 3.2.5 was used to determine this volume and was incorporated into the excel sheet mentioned earlier, Appendix E.

$$V = Q * 60 \text{ minutes} \qquad \text{Eq. (3.2.5)}$$

where,

V = stormwater volume (gal)

Q = stormwater runoff (gal/min)

Since several "Q" values were calculated, this resulted in several volume values as well.

Step 3: Determine Catch Basin Dimensions

Once the potential volumes were calculated for a 60-minute, 25-year storm on different surfaces, four catch basins were designed to contain the stormwater values based on MassDOT construction standards. Equation 3.2.6 was used to calculate the capacity of each catch basin.

$$V = Pi * r^2 * h \qquad \text{Eq. (3.2.6)}$$

where,

V = Volume of catch basin (gal)

r = radius of catch basin tub (ft.)

h = height of catch basin tub (ft)

MassDOT construction standards for precast concrete catch basins specify the following [9].

- Diameter must be greater than or equal to 4 feet

- Height to inlet pipe cannot exceed 3 feet for a standard sump and must be a minimum height of 2 feet
- Height to inlet pipe cannot exceed 4 feet for a deep sump

Based on these specifications, catch basin dimensions were calculated by cross-referencing the volumes that would result from the 25-year storm on different surface material and the MassDOT standards.

3.3.2 Grate Design

Once the body of the catch basin was designed, the next process was grate design. There are several factors that go into grate design including debris and gravel considerations, pedestrian and biker safety, and grate loading conditions [27]. However, the first factor we needed to consider before any of the others was the stormwater flow rate. The stormwater flow rate was used to determine the grate opening dimensions. The grate opening dimensions were calculated first because the rest of the factors for consideration in the design process were dependent on knowing these dimensions. Once the dimensions of the grate openings were determined, the rest of the factors were then checked to ensure they passed regulations. Figure 3.5 illustrates the process for our grate design and how all the other considerations come after stormwater flow rate.

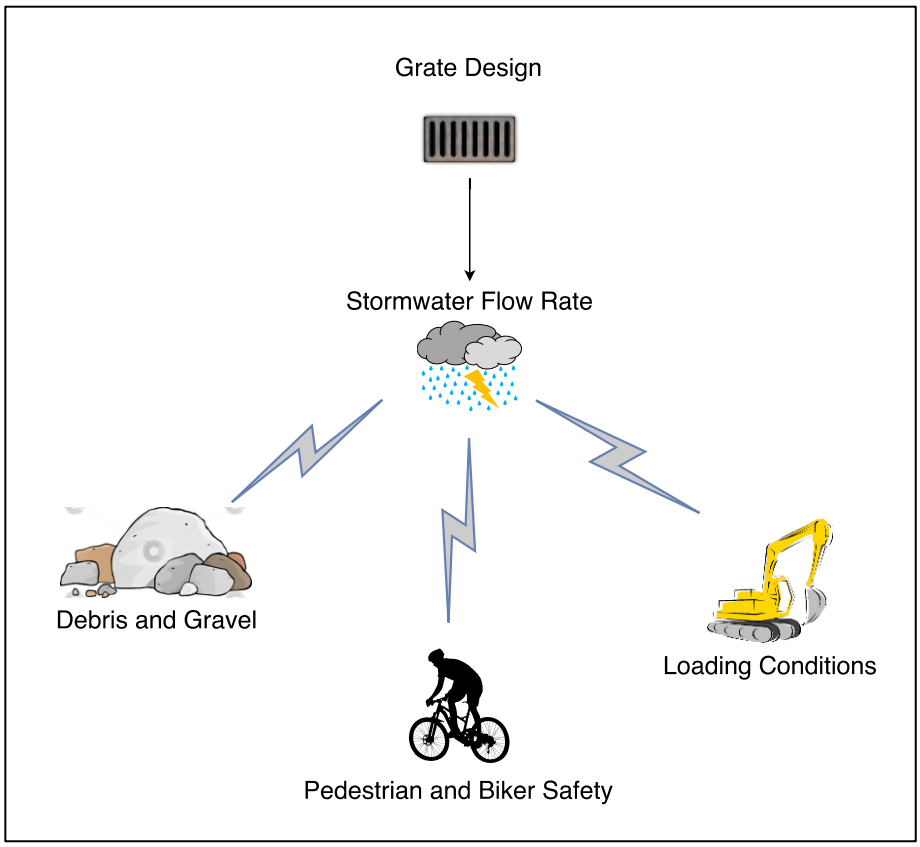


Figure 3.6: Grate Design Process

Step 1: Determine Grate Opening Based on Stormwater Runoff Rate

In order to design the dimensions of the grate openings, Equation 3.2.4 was used. Based on this equation for flow rate, "C" is a constant with a value of 0.6, "A" is the area of open grate in square feet, "g" is the value for gravity, and "h" is the value for ponding depth. In order to ensure an adequate flow rate capacity for the grate, various values of "A" and "h" were tested to determine the final design for the grate openings. The flow rate capacities that were calculated for various values of "A" at different depths of ponding were compared to the flow rate of the 25-year storm and as long as the flow rate of the grate was larger than the flow rate of the 25-year storm, the grate design would work.

Step 2: Check Remaining Factors for Compliance

Debris and Gravel

Once we confirmed that there was an adequate amount of open area in the grate design to allow for fluid flow, the next thing we needed to think about was debris and gravel considerations. The goal of designing an effective grate is to let water in but keep debris and gravel out. Once we knew how much area needed to be left open on the grate, we were able to design the length and width of these openings. Our dimensions for the grate openings were determined by considering the size of gravel that would be used in our surface design and ensuring that the diameter was larger than the width of the grate openings. We also took into consideration the average size of common trash and garbage.

Pedestrian and Biker Safety

The third consideration for our grate design was pedestrian and biker safety. According to ACO Drainage Design Standards user safety is a requirement for grate application [27]. In order to do this, we researched the average bike tire widths for different types of bikes. We ensured that the width of the grate openings was small enough that even the smallest bike tire would not have a chance of getting stuck or falling into the grate. We also made sure that the orientation of the grate was acceptable according to ACO Drainage Design standards.

Loading Conditions

The final consideration for the grate design was applicable loading conditions. According to ACO Drainage Design Standards, load class B is for sidewalks and small private parking lots [27]. Based on this classification, our team had to make sure the grate was strong enough to withstand the loads associated with load class B.

3.3.3 Pump Design

Once the catch basin body and grate design were complete, our team had to consider ideas for draining the catch basin itself. Since there were no local sewage drains in the surrounding area, we could not design a catch basin with an outlet pipe to drain on its own. The catch basin was designed to capture and hold stormwater with manual drainage. We decided that there were two options:

1. Hire a licensed professional to drain the catch basin
2. Design a pump with the capacity to drain the catch basin

Since the first option is a paid service, we decided to also design for a self-sufficient service by using a pump (despite a license requirement for treatment and disposal of stormwater). According to the Design Standards of the Bureau of Engineering, 10 days after a storm the available volume of the catch basin must equal that of the volume of a 10-year storm. The standards also state that 14 days after a storm the total available volume must be restored. This means that the pump had to be designed to drain the volume of a 25-year storm to the point where there is enough available capacity in the catch basin to withhold the potential stormwater volume of a 10-year storm. Once the 10-day mark passes, the pump must be able to drain the rest of the remaining stormwater from the catch basin in 4 days [27].

In order to design for these specifications, our team researched the rainfall intensity of a 10-year storm, then used Equation 6 to calculate the volume of water this storm would produce. We then subtracted the volume of a 10-year storm from the volume of a 25-year storm to find how much stormwater must be pumped in the first 10 days and determined the rate at which the pump must be able to work. We then divided the remaining volume by 4 days to determine the rate at which the pump must be able to work in the final 4 days. The larger rate of work then governed the capacity at which the pump must be designed for.

4 Results and Analysis

The goal of this project was to provide Ski Ward Hill with redesign options that would improve the durability and drainage of the two parking lots, Lot A and B. This chapter will focus on the components, materials and the final designs of the potential solutions we chose. Each section will follow the step by step procedure outlined in the methodology chapter.

4.1. Identify Structural and Material Components for Gravel Resurfacing

In accordance with necessary methods outlined in the Methodology section, 3.1.1, base and surface layers of the gravel were designed, followed by the proper geosynthetics for separation and reinforcement. The base layer was designed to act as a strength bearing and drainage layer. Local aggregate suppliers were consulted, and Delta Sand aggregate provider in Sunderland, MA was selected for use. Per their aggregate summary, 2" Double-Washed Stone was found to be appropriate and selected. The summary can be found in Appendix K. As it can be seen in the aggregate summary in Table 3.1, 1-1/2" Double-Washed stone met MassDOT requirements. Figure 4.1 shows the 1-1/2" Double-Washed stone compared to MassDOT's requirements.

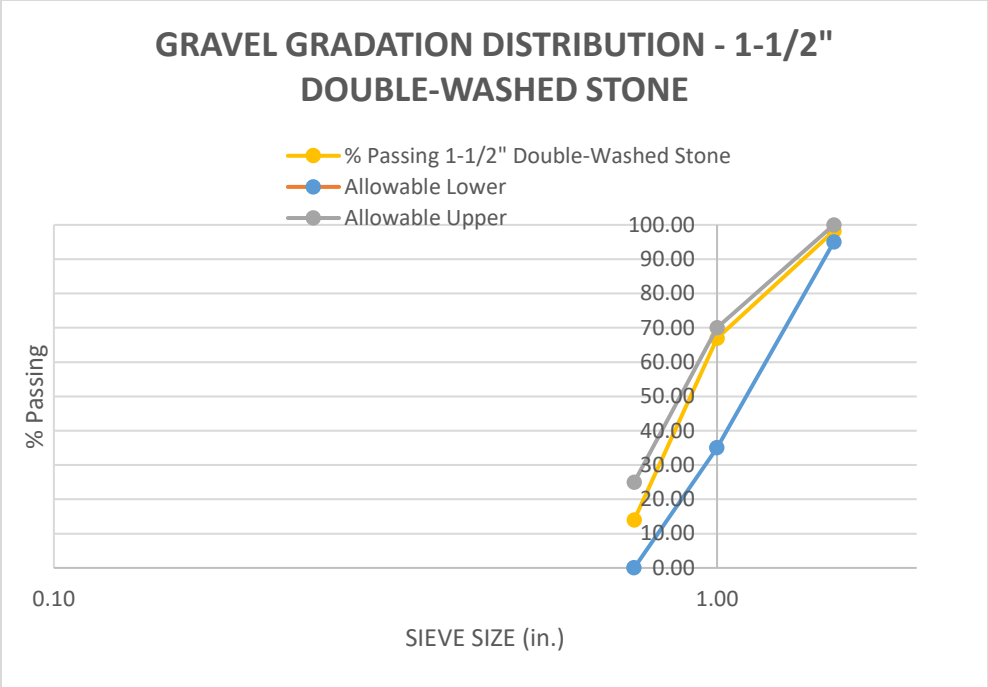
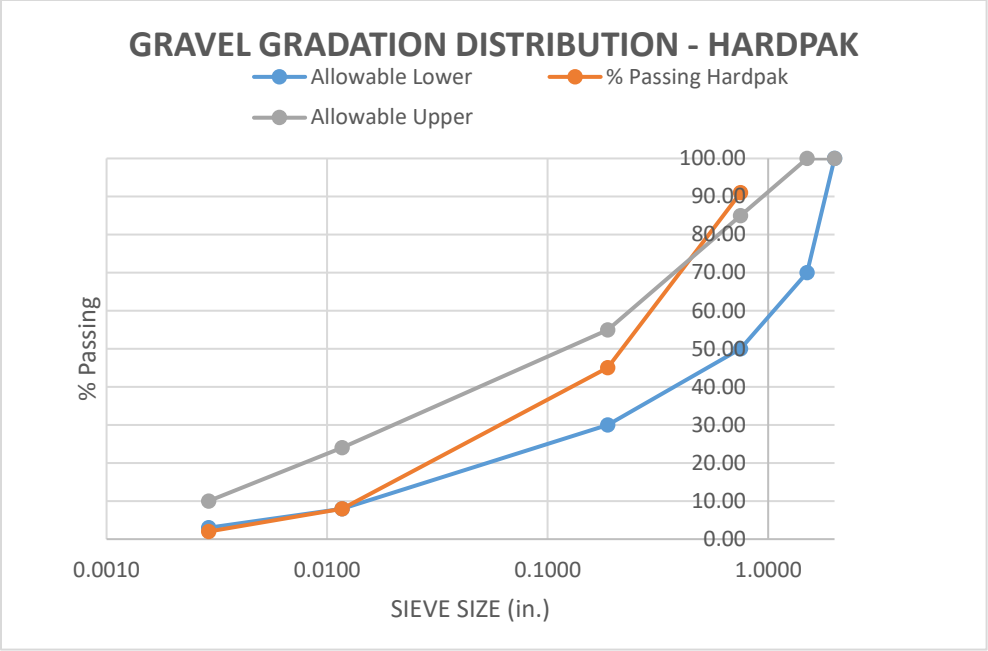


Figure 4.1: This plot shows the relationship between the 1-1/2" Double-Washed Stone and the % passing requirements of the base layer

Per FHWA regulations, the surface layer had to be smoothed surfaced, not muddy in rainy situations and not to produce excessive dust in dry situations, and be properly sloped to encourage natural water runoff. The materials list from Delta Sand was consulted and Hardpak™ was chosen. Table 3.2 showed the aggregate requirements of MassDOT. Figure 4.2 shows that the Hardpack materials successfully met these standards.



4.2: This plot shows the relationship between Hardpack and the % passing requirements for the surface layer

A general summary of design of the layers can be seen in Table 4.1. Cost analysis can be seen in Table 4.2.

Table 4.1 Aggregate design requirements

Design Purpose	**Material Chosen**	**Thickness**	**Plasticity Index**	**Slope of Surface**
Surface Layer	Hardpak	6”	~3	4%
Base Layer	1-1/2” Double-Washed Stone	10”	~0	None – subbase must be flat

Table 4.2: Aggregate cost estimates

	Area (sq. yd)	**Surface Layer @ 6” (cu. yd.)**	**Base Layer @ 10” (cu. yd.)**	**Surface Price @ $11.95/cu. yd.**	**Base Price @ $13.95/cu. yd.**	**Total Price**
Lot 1	**4720**	87	146	$1039.65	$2036.70	$3076.35
Lot 2	**3670**	68	114	$812.60	$1590.30	$2402.90
Total	8390	155	260	$1852.25	$3627	**$5479.25**

To support the proper layers and ensure they would continuously operate at their designed intent, proper geosynthetics were designed to be used in separation and support capacities. Using the equations outlined in the methods section, the geofabric and the geogrid were designed. Per Koerner’s *Designing with Geosynthetics*, the geofabric used for separation should be designed with specified minimum allowable tensile stress resistance, tearing resistance, puncture resistance, and burst resistance allowances. These allowances create proper durability for geosynthetics to continue to be effective after sustaining normal wear and tear involved in installation as well as from the service loading associated with everyday use. Allowable tensile stress was calculated using equation 3.1.1 and found to be 13 lb/foot lengthwise and widthwise. According to equation 3.1.2 required puncture resistance was found to be 32 lb. Required burst resistance was calculated to be 13 psi according to equation 3.1.3. Per equation 3.1.4 required tear resistance was calculated to be .664 psf. These calculations can be found in Appendix L.

Per Koerner’s *Designing with Geosynthetics*, the geogrid was designed for reinforcement to have necessary strength, and aperture size requirements. The strength requirement would

ensure the grid is able to resist stretching due to stresses from the service load imposed on the parking lot. The geogrid must be able to resist these stresses both lengthwise and widthwise. If it operates as intended by design, the geogrid will provide reinforcement to reduce presence of potholing and unevenness in the parking lot. The minimum aperture size requirement ensures the grid will be able to create enough friction to hold itself in place and provide support to the layers above it. These requirements were calculated according to equations 3.1.5 and 3.1.6. Depth for the geogrid was found to be 10" [9]. Necessary tensile strength for the geogrid was found to be 376.9 lb/ft, both lengthwise and widthwise. Necessary minimum aperture size for use with the base aggregate selected was calculated to be .2881 inches in diameter.

We contacted a supplier who was within deliverable distance to the project site and selected geofabric and geogrids from their product list which met the engineered design criteria. National geosynthetics provider US Fabrics was contacted and the proper geosynthetics were selected. US Fabrics' US 200 Geotextile was selected to be used as the appropriate geofabric. As shown in table 4.3, it can be seen that the selected geofabric meets the necessary criteria for use in separation. Geogrid, US Fabrics' BaseGrid 12 was chosen as the appropriate geogrid material to be used. As shown in table 4.4, it is also shown that this material meets the necessary engineered specifications for reinforcement and aperture opening sizes.

Table 4.3: Geofabric requirements versus performance standards

Geofabric	**Tensile Strength (lb)**	**Burst Resistance (psi)**	**Puncture Resistance (lb)**	**Tear Resistance (psf/ psi)**
Design Requirement	13	13	32	.664 psf
US Fabrics US 200	200	400	90	400 psi
Factor of Safety	15.4	30.7	3	>100

Table 4.4: Geogrid requirements versus performance standards

Geogrid	Tensile Strength (lb/ft)	Aperture Size (in)	Placement Depth
Design Requirement	376.9	.2881 (minimum)	10”
US Fabrics BaseGrid 12	1310	1 x 1.3	
Factor of Safety	3.47		

The selected geofabric is in compliance with AASHTO M-288-06, a design criterion set out by the American Association of State Highway and Transportation Officials. Using the two parking area’s dimensions we were able to calculate the cost for geosynthetic materials, summarized in table 4.5.

Figure 4.3: Lot A Dimensions

Figure 4.4: Lot B Dimensions

Table 4.5: Geosynthetics Cost Estimation

	Sq. Yd. Covered	**Sq. Yd./roll**	**Cost /roll**	**Rolls Needed**	**Flat Rate Delivery Fee**	**Cost**
GeoFabric Lot 1	4720	600	$360	8	$250 (for both)	$610
GeoFabric Lot 2	3670	600	$360	7	N/A	$360
GeoGrid Lot 1	4720	239	$358.50	20	$200 (for both)	$558.50
GeoGrid Lot 2	3670	239	$358.50	16	N/A	$358.50
Total						$1887

Finally, our final design is below, outlined in figure 4.5

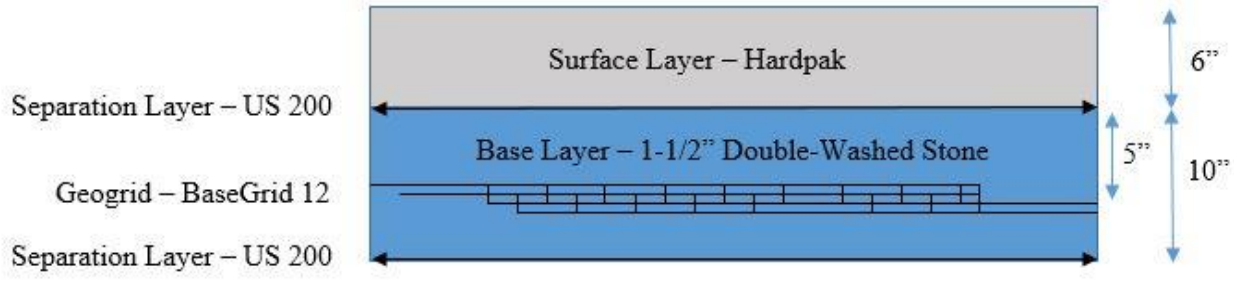


Figure 4.5: Cross Section of Gravel layer design, with base and surface layers and designed geosynthetics

4.2 Identify Environmental Parameters and Materials for Asphalt Cement Pavement Resurfacing

After talking to the town engineer of Shrewsbury, we were informed that the average recurrence of intense rainfalls in the town of Shrewsbury occurred in 25 year intervals at 60 min durations. Using this information, the NOAA provided the amount of average rainfall, inches, during the 60 min duration. On average, 2.01 inches fall during this type of storm. Using equation 7 in section 3.2, we found the Rainfall Intensity to be 0.0335 in/min. In order to determine the Runoff the parking areas endured during a storm, we first needed to determine the Rational Runoff Coefficient, c, from equation in section 3.2. The existing conditions of the parking lots were found to be sand and gravel. Both of these areas had not been improved, resulting in a Runoff Coefficient of .2. We then converted the area of both of the parking areas to acres and found that Lot A and B were 1.18 and .76 respectively. We used the previously solved for Rainfall Intensity value in equation 8 and were able to calculate the Runoffs. Lot A endured a Runoff of 0.008 in/min*acre. Lot B endured a Runoff of 0.0051 in/min*acre. These values were then used to determine the types of aggregates needed in the design to withstand these Runoff rates.

The values solved for above were critical for designing our structure of the asphalt cement pavement. As previously stated before, our design called for better drainability and durability. Once we reviewed MassDOT's material requirements and established the acceptable percent passing for an aggregate sample, we were able to choose a local aggregate distributor. We identified Delta Sand as a local aggregate distributor that could provide the necessary aggregate for our design. Appendix K shows the gradation distribution, particle percent passing through the sieve layers, of the materials Delta Sand has at their location. We used their

aggregate summary to compare the distributions of the materials to the required distribution of MassDOT, shown in tables 3.3 to 3.8. The following graphs identify the selected material and the gradation distribution of that material is compared to the lower, upper and acceptable distributions of MassDOT.

1. Finish Layer:

    a. Fine Aggregate – Concrete Sand
    b. Coarse Aggregate – ¾" Blend Crushed Trap Rock

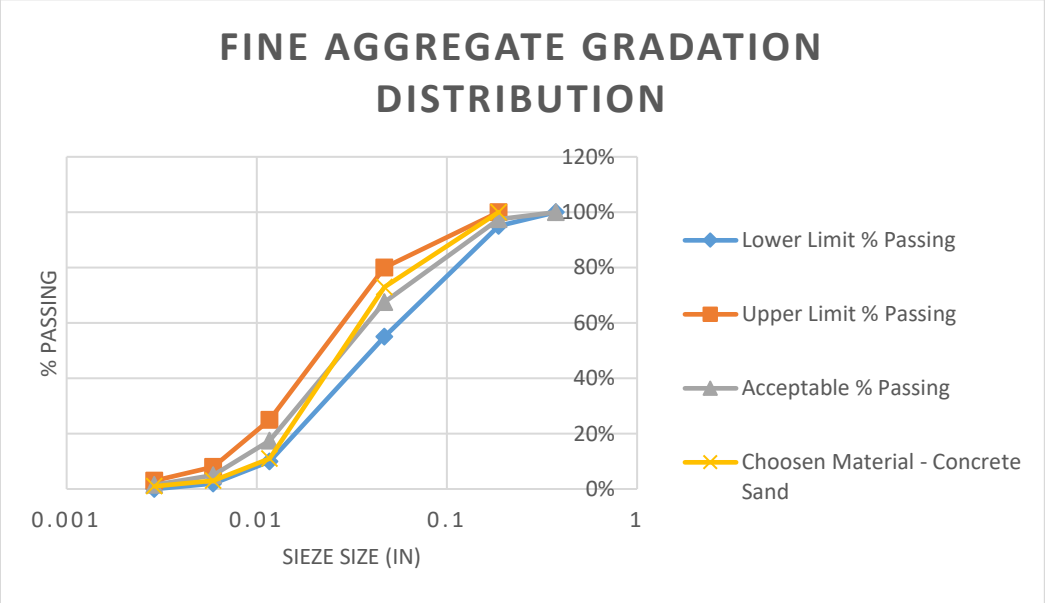


Figure 4.6: This plot shows the gradation distribution of Concrete Sand compared to MassDOT's requirements for fine aggregates of the finish layer.

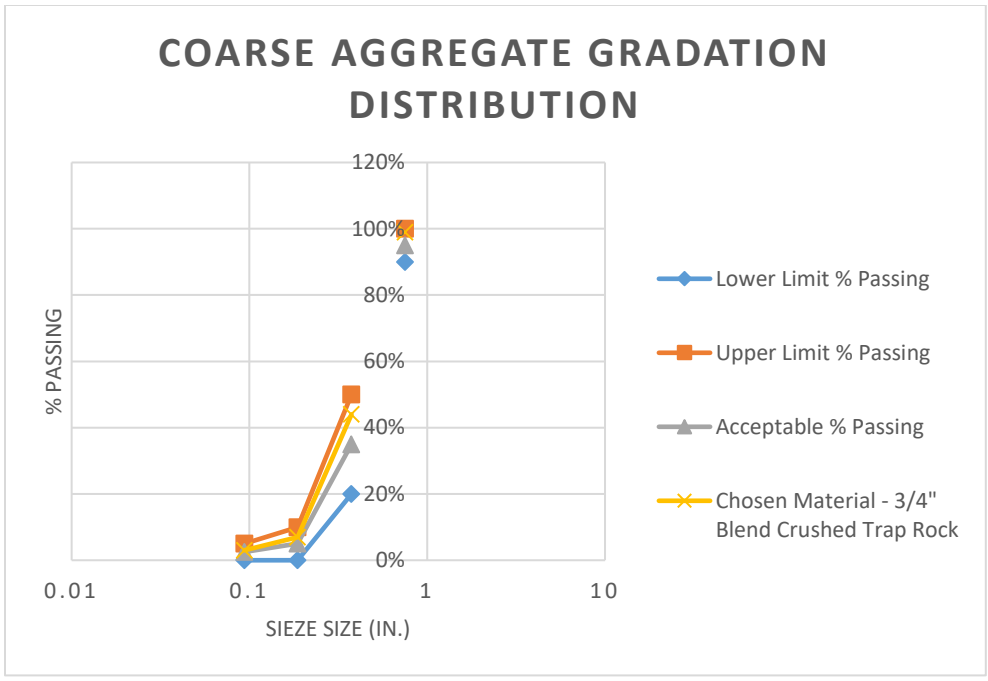


Figure 4.7: This plot shows the gradation of the ¾" Blend Crushed Trap Rock compared to MassDOT's requirements for coarse aggregates of the finish layer.

2. Base Layer
    a. Fine Aggregate – Gravel Stone Dust
    b. Coarse Aggregate – Processed Quarry Rock (Dense Grade)

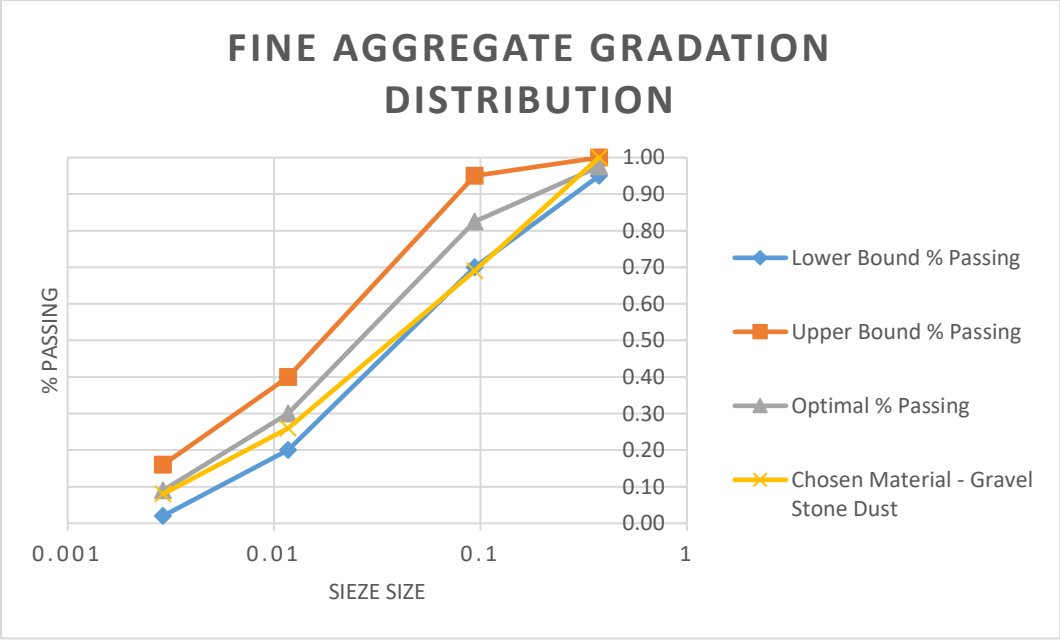


Figure 4.8: This plot shows the gradation of the Gravel Stone Dust compared to MassDOT's requirements for fine aggregates of the base layer.

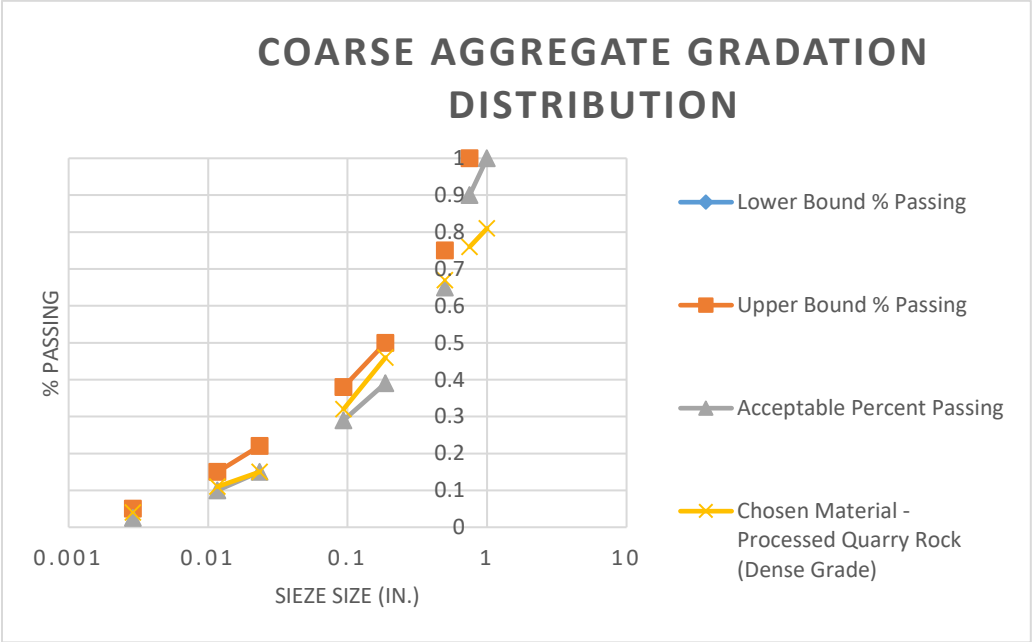


Figure 4.9: This plot shows the gradation of the Processed Quarry Rock (Dense Grade) compared to MassDOT's requirements for coarse aggregates of the base layer.

3. Subbase:
    a. Coarse Aggregate – 1.5" Minus Crushed Gravel

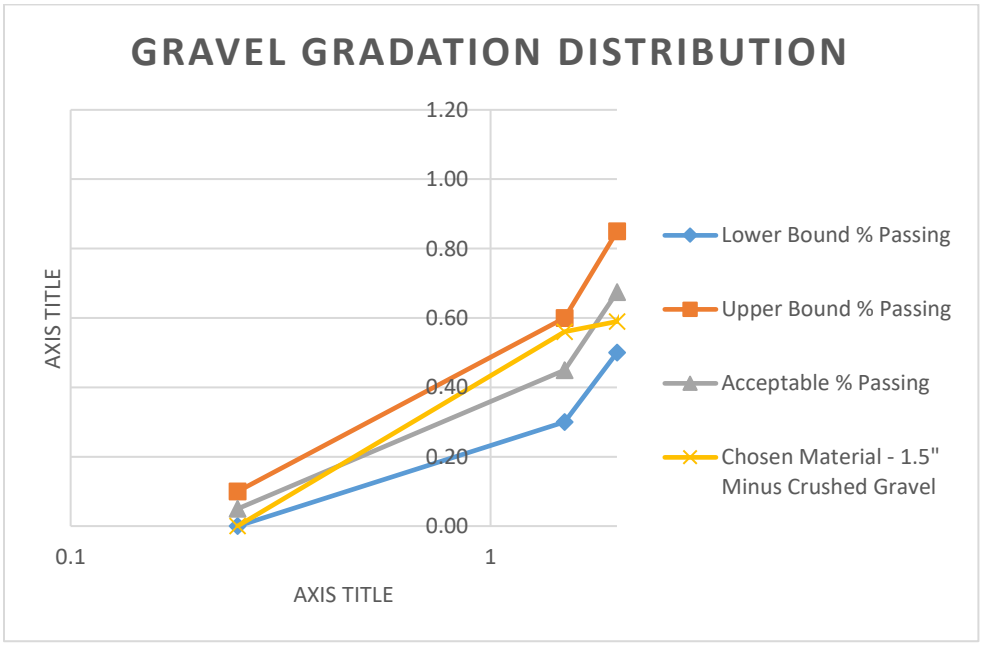
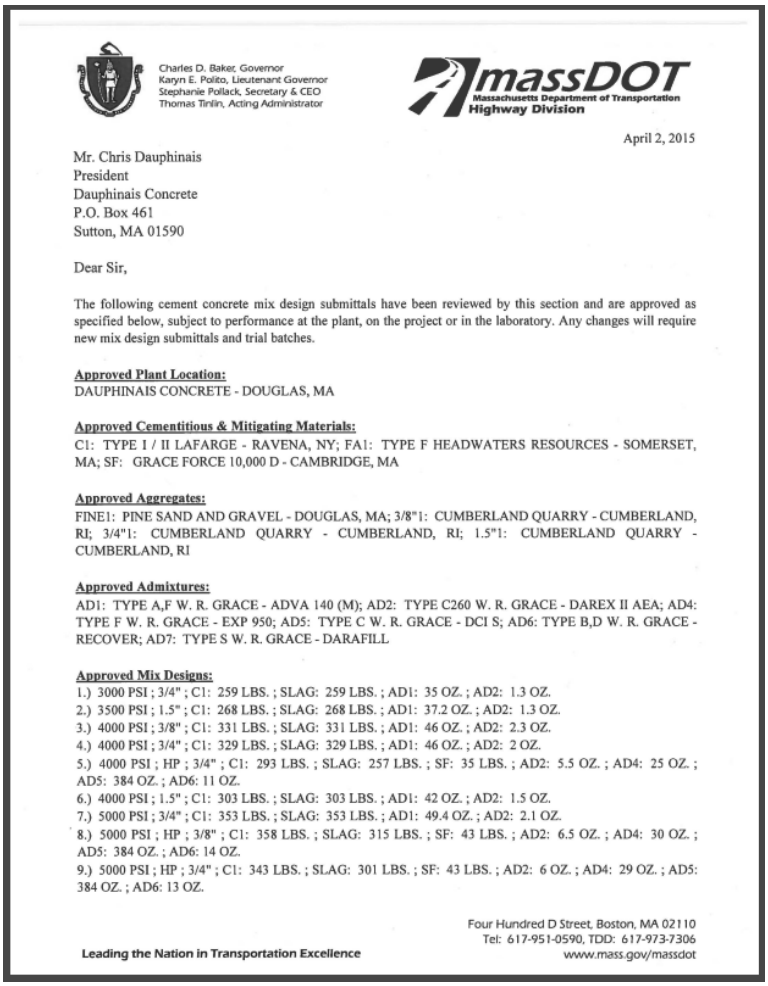


Figure 4.10: This plot shows the gradation of the 1.5" Minus Crushed Gravel (Dense Grade) compared to MassDOT's requirements for coarse aggregates of the base layer.

Figures 4.6 - 4.10 support our decision in choosing the selected aggregates because each one of the materials met the gradation requirements, therefore producing a structural sound design.

In order to complete the design we needed to find MassDOT approved cement mixtures for the finish layer. We identified Dauphinais Concrete of Worcester County as the company that could provide the appropriate mix for our design. This company had previously received certification from MassDOT for their mixes for highway and parking lots in Worcester County. We checked their available mixes and found that mix #7 in the following figure 4.9, incorporated the ¾" Blend Crushed Trap Rock coarse aggregate and Concrete Sand fine aggregate.



Charles D. Baker, Governor
Karyn E. Polito, Lieutenant Governor
Stephanie Pollack, Secretary & CEO
Thomas Tinlin, Acting Administrator

MassDOT
Massachusetts Department of Transportation
Highway Division

April 2, 2015

Mr. Chris Dauphinais
President
Dauphinais Concrete
P.O. Box 461
Sutton, MA 01590

Dear Sir,

The following cement concrete mix design submittals have been reviewed by this section and are approved as specified below, subject to performance at the plant, on the project or in the laboratory. Any changes will require new mix design submittals and trial batches.

Approved Plant Location:
DAUPHINAIS CONCRETE - DOUGLAS, MA

Approved Cementitious & Mitigating Materials:
C1: TYPE I / II LAFARGE - RAVENA, NY; FA1: TYPE F HEADWATERS RESOURCES - SOMERSET, MA; SF: GRACE FORCE 10,000 D - CAMBRIDGE, MA

Approved Aggregates:
FINE1: PINE SAND AND GRAVEL - DOUGLAS, MA; 3/8"1: CUMBERLAND QUARRY - CUMBERLAND, RI; 3/4"1: CUMBERLAND QUARRY - CUMBERLAND, RI; 1.5"1: CUMBERLAND QUARRY - CUMBERLAND, RI

Approved Admixtures:
AD1: TYPE A,F W. R. GRACE - ADVA 140 (M); AD2: TYPE C260 W. R. GRACE - DAREX II AEA; AD4: TYPE F W. R. GRACE - EXP 950; AD5: TYPE C W. R. GRACE - DCI S; AD6: TYPE B,D W. R. GRACE - RECOVER; AD7: TYPE S W. R. GRACE - DARAFILL

Approved Mix Designs:
1.) 3000 PSI ; 3/4" ; C1: 259 LBS. ; SLAG: 259 LBS. ; AD1: 35 OZ. ; AD2: 1.3 OZ.
2.) 3500 PSI ; 1.5" ; C1: 268 LBS. ; SLAG: 268 LBS. ; AD1: 37.2 OZ. ; AD2: 1.3 OZ.
3.) 4000 PSI ; 3/8" ; C1: 331 LBS. ; SLAG: 331 LBS. ; AD1: 46 OZ. ; AD2: 2.3 OZ.
4.) 4000 PSI ; 3/4" ; C1: 329 LBS. ; SLAG: 329 LBS. ; AD1: 46 OZ. ; AD2: 2 OZ.
5.) 4000 PSI ; HP ; 3/4" ; C1: 293 LBS. ; SLAG: 257 LBS. ; SF: 35 LBS. ; AD2: 5.5 OZ. ; AD4: 25 OZ. ; AD5: 384 OZ. ; AD6: 11 OZ.
6.) 4000 PSI ; 1.5" ; C1: 303 LBS. ; SLAG: 303 LBS. ; AD1: 42 OZ. ; AD2: 1.5 OZ.
7.) 5000 PSI ; 3/4" ; C1: 353 LBS. ; SLAG: 353 LBS. ; AD1: 49.4 OZ. ; AD2: 2.1 OZ.
8.) 5000 PSI ; HP ; 3/8" ; C1: 358 LBS. ; SLAG: 315 LBS. ; SF: 43 LBS. ; AD2: 6.5 OZ. ; AD4: 30 OZ. ; AD5: 384 OZ. ; AD6: 14 OZ.
9.) 5000 PSI ; HP ; 3/4" ; C1: 343 LBS. ; SLAG: 301 LBS. ; SF: 43 LBS. ; AD2: 6 OZ. ; AD4: 29 OZ. ; AD5: 384 OZ. ; AD6: 13 OZ.

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Figure 4.11: Dauphinais Concrete Finish Layer mixes approved by MassDOT. Mix #7 will is used in the final design.

Finally, figure 4.12 shows the final structural design of the asphalt cement pavement and table 4.6.A and 4.7.B, outlines the cost analysis for each parking area.

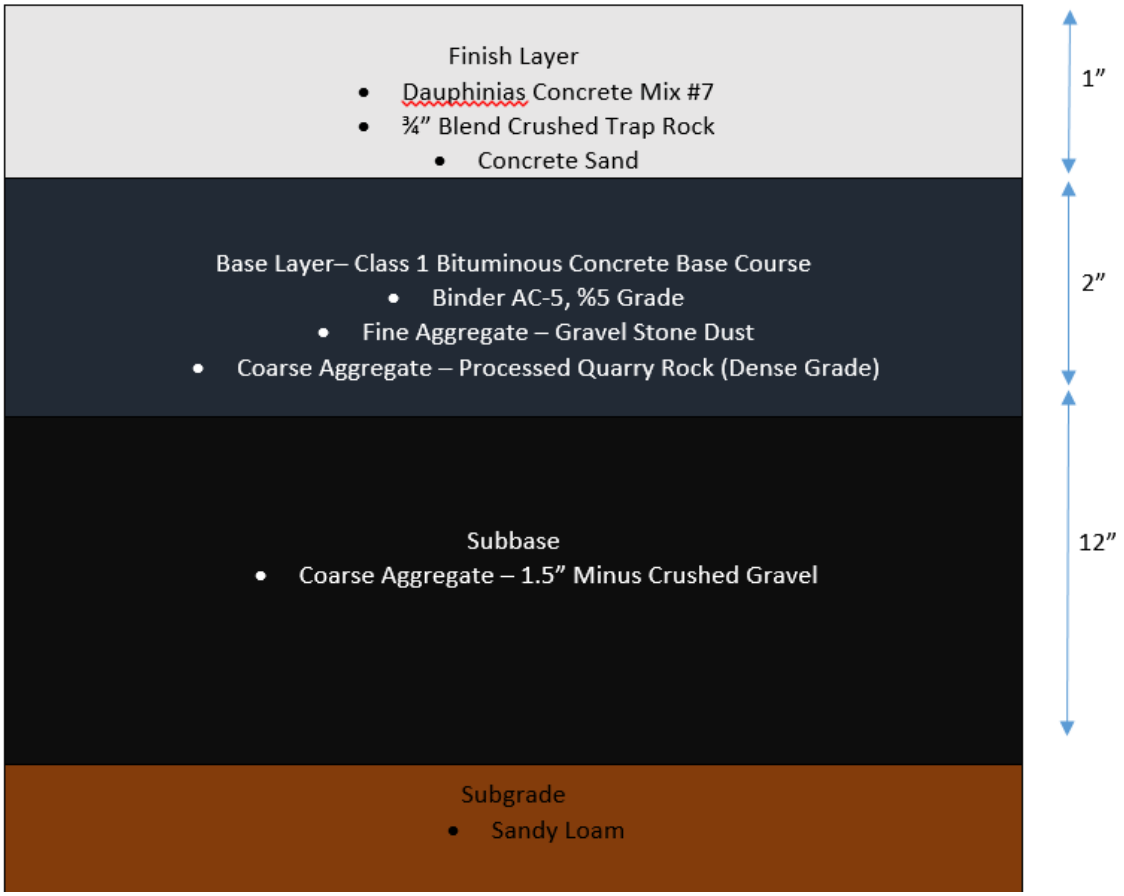


Figure 4.12: Asphalt Concrete Pavement Resurfacing Final Design

Table 4.6: Lot A

Lot A Cost Anaylsis of Asphalt Cement Resurfacing				
Layer	Type of Material (s)	Price per Cubic Yard	Total Cubic Yards	Cost
Finish Course	Dauphinias Cement Concrete Mix Design # 7		1916.3	$210,793
Binder Course	Processed Quarry Rock Dense Grade	$27.93	319.4	$8,920.84
	Gravel Stone Dust	$19.81	319.4	$6,327.31
Subbase	1.5 " Minus Crushed Gravel	$14.35	1916.3	$27,498.91
Total				$253,540.06

Table 4.7: Lot B

Lot B Cost Anaylsis of Asphalt Cement Resurfacing				
Layer	**Type of Material (s)**	**Price per Cubic Yard**	**Total Cubic Yards**	**Cost**
Finish Course	Dauphinias Cement Concrete Mix Design # 7		1221.3	$134,343
Binder Course	Processed Quarry Rock Dense Grade	$27.93	203.55	$5,685.15
	Gravel Stone Dust	$19.81	203.55	$4,032.33
Subbase	1.5 " Minus Crushed Gravel	$14.35	1221.3	$17,525.66
Total				$161,586.13

4.3 French Drain Design

As previously explained, the site was broken up into two lots “A” and “B”. It was agreed on that Lot A would house the French drain. The figure below shows a one of the contributors to the flooding problem at Ski Ward Hill. As you can see, not only is Lot A receiving direct rainfall from above, but it’s receiving unwanted runoff from Main Street as well. Other underlying issues revolved around the topography of the site. Depressions in the land make it easy for puddles to form. These are revealed in figure 4.13.

Figure 4.13: This figure shows Lot A (right), and how it rests at a lower elevation than Main St (left). Main St. runs parallel along the top of Lot A and disposes stormwater runoff down the sloped hill and onto Lot A of Ski Ward Hill.

After running the online survey for Lot A through WSS, the results were broken into two zones see Figure 4.14. This was due to their difference in drainage characteristics. Below is a short summary of the results from WSS. More information can be found in Appendix E.

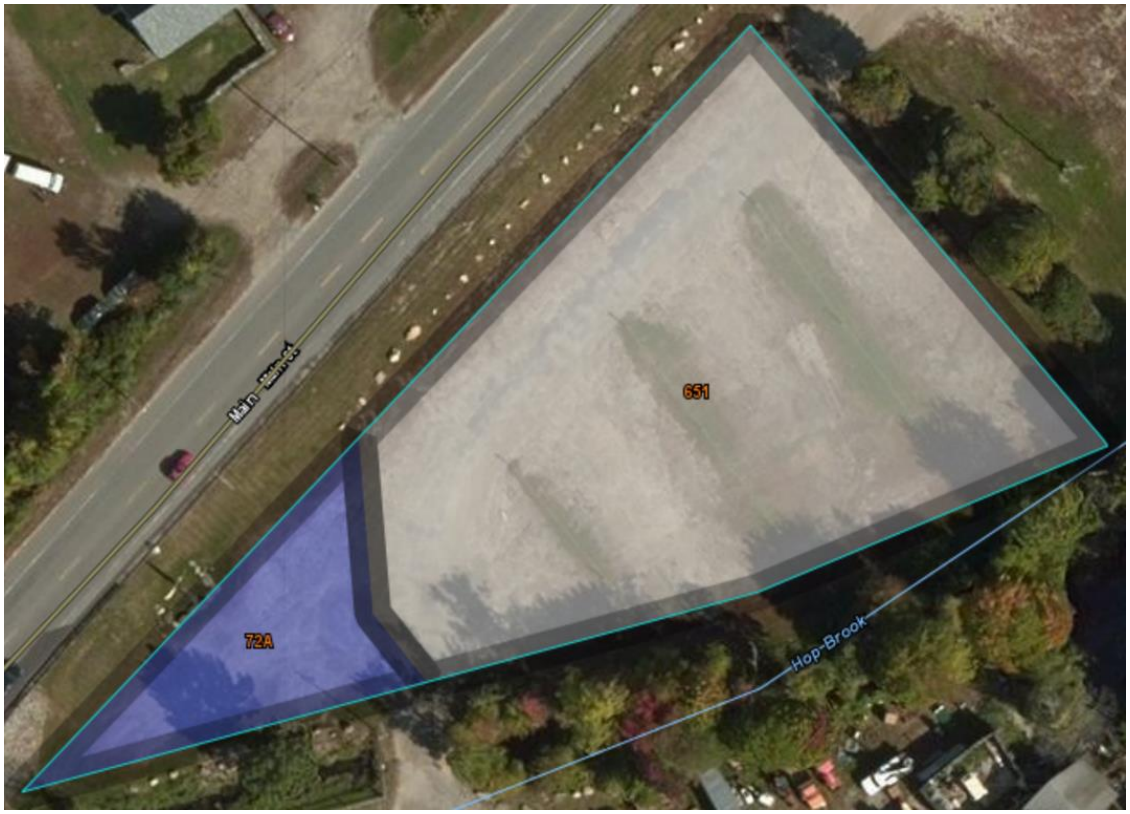


Figure 4.14: This figure shows an aerial view of Lot A along with the two separate zones used in the Web Soil Survey for Ski Ward Hill (Zone 72A & 651).

- **Drainage Class**
  - *Zone 72A* - very poorly drained
  - *Zone 651* - not rated (null)
- **Gravel Source**
  - *Zone 72A* - poor
  - *Zone 651* - not rated (null)
- **Parent Material Name**
  - *Zone 72A* - friable coarse-loamy eolian deposits over dense coarse-loamy lodgement till derived from granite and gneiss
  - *Zone 651* - made land over firm loamy basal till
- **Ponding Frequency Class**
  - *Zone 72A* - frequent
  - *Zone 651* - none
- **Sand Source**
  - *Zone 72A* - fair
  - *Zone 651* - not rated (null)
- **Overall Soil Map Description**
  - *Zone 72A*
    a. Slopes = 0-3%
    b. Whitman soils = 70%, Minor Components = 30%
    c. Typical profile = 0-60 inches fine sandy loam
    d. Depth to restrictive feature = 18” to densic material

e. Depth to water table = 6”
f. Frequency of ponding = Frequent

- *Zone 651*

. Slopes = 0-3%
a. Udorthent soils = 80%, Urban Land = 20%
b. Depth to restrictive feature = 80+”
c. Depth to water table = 80+”
d. Frequency of ponding = None

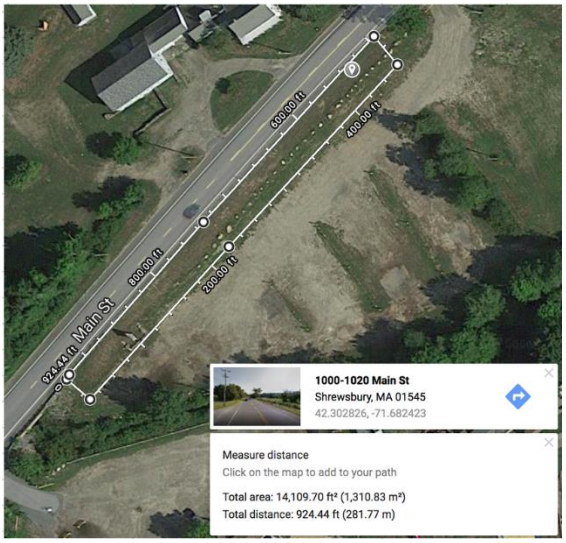
In addition to the slope of the lot, the slope from Main street to Lot A also had to be taken into consideration. Refer back to figure 4.13 above, and notice how the road is significantly higher than Lot A. Using FreeMapTools.com (FMT), we were able to recognize that the elevation of the road was 2-3 feet higher than the elevation of Lot A. Figure 4.15 below displays a point elevation test provided by FMT, followed by a table for reference.

Figure 4.15: This figure shows an aerial view of Lot A along with specific points chosen for elevation data. This figure was key in the design phase when determining the orientation of the French Drain.

Table 4.8: This table depicts all relevant elevation points in relation to Figure 4.15

Point	Elevation (ft.)
1	353.6
2	355.2
3	355.7
4	355.3
5	351.3
6	351.3
7	351.3
8	348.0
9	347.5
10	347.2

Figures 4.16 and 4.17 below will show you the two areas of Lot A where slope is significantly different. As you can see, Google Earth also calculated the area in sq. ft.



Figures 4.16: Area from Main Street to Lot A, Google Earth

Figure 4.17: Area of Lot A, Google Earth

4.3.1 Solidifying Design Constraints

The square footage from figures 4.14 and 4.15 above let us calculate the peak runoff rates during a 25 year storm. Figure 4.14 had a runoff rate of 0.12 cubic feet per second, while Figure 4.15 had a runoff rate of 1.5 cubic feet per second, see Appendix M for calculations. In conclusion, we found the chosen pipe needed to be able to support a flow rate of 1.5 cfs. After going back and forth with Brad Stone, the discharge location was found at Hop Brook, a source of water that ran through the property and into a reservoir. Massachusetts Stormwater Handbook, Vol. 1, "No new stormwater conveyances (e.g. outfalls) may discharge untreated stormwater directly to or cause erosion in wetlands or waters of the Commonwealth" [19]. This standard basically allows stormwater to be discharged if the water is adequately treated. In this instance, our design needed a filter that had to be able to remove sediment and other contaminants to protect nearby water sources. The filter that was chosen was ADS High-Density Polyethylene (HDPE) Stormwater Quality Unit acquiring a treated flow rate of 1.5 cfs, which accommodates the system's flow rate constraint, see Appendix N. HDPE pipe is one of the most durable materials on the market and is widely used for drainage. It is resistant to the effects of chemicals, abrasion, hot soils, and effluent [28]. The unit itself removes 80% of all total suspended solids (TSS) as well as 80% of all grease and oil substances. This capability allowed our team to further examine discharging into the brook. Massachusetts Stormwater Handbook, Vol. 1 also states "Stormwater management systems shall be designed so that post-development peak discharge rates do not exceed pre-development peak discharge rates" [19]. Using equation 3.1.8, our team found that pre-development discharge rates into Hop Brook was the same as the treated flow rate of the ADS filter, see Appendix M for calculations. Since this value does not exceed the pre-development rate, it allows our design to be discharged into Hop Brook.

Following the output location and choice of filter, the rest of the design was in order. We chose HDPE pipe for the rest of the design not only because of its durability, but also because of its compatibility with the filter. Specifically, our design calls for ADS N-12 high-density polyethylene (HDPE) Dual Wall Perforated Pipe. The dual wall consists of a corrugated exterior surface for extra strength and durability, and a smooth interior surface that offers exceptional hydraulics [22]. Small perforations surround the pipe every 120 degrees, so that water may be collected and discharged. Equations 3.2.1, 3.2.2, and 3.2.3 were used to find the best possible slope of the pipe so that its outflow rate that is capable of moving runoff 1.5 cfs. It was calculated that an 8" dual wall pipe at a slope of 1.5% would have a flow rate of 1.608 cfs and a velocity of 4.61 ft/s. Both values agree to the maximum flow rate requirements of the system, as well as the minimum and maximum velocity requirements acquired from ADS, Inc. Drainage Handbook, see Appendix 0. Using equation 3.2.4, our team was able to calculate the flow rate through each perforation in the pipe. The maximum flow rate was found to be .023 cfs through each perforation. The perforations are located at every valley of corrugations (2 ⅛"), which means the maximum flow rate through perforations are about 2.69 cfs per foot of pipe. Since this value is more than the systems runoff rate of 1.5 cfs, means that the perforations are more than capable to collect all stormwater without flooding. Further materials including gravel and geofabric were added to the design as another filter component. The gravel size chosen was 1 ½ in. crushed stone from the local provider, Worcester Sand & Gravel. The only constraint in choosing this size was making sure aggregate size was larger than slot size in the perforated pipe. The geofabric was chosen with the help of ADS Sales rep, John Stelmokas. After further discussion on the project's specifications, John was able to recommend a non-woven geofabric. It is a widely used product and is made specific for drainage applications like the one used in our

design. Specifically, we used a 6 oz. Hancor geofabric material that will cover the interior perimeter of the trench, and then some.

4.3.2 Building the Design & Acquiring the Material

The following figures below shows how each design option is oriented in Lot A.

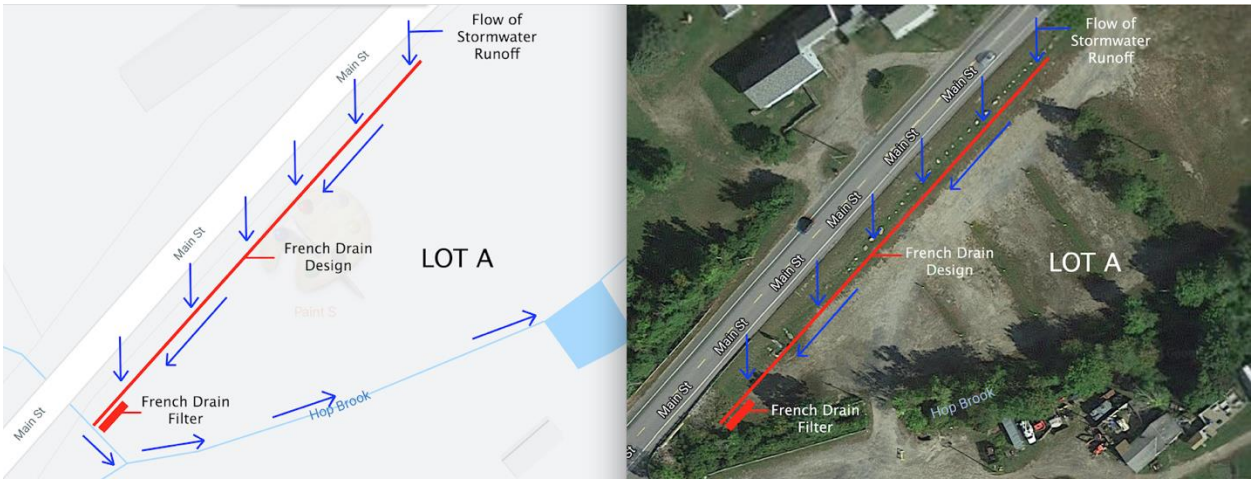


Figure 4.18: This figure depicts the orientation for option 1 of the French Drain design. Both map & satellite images are included to show how runoff will enter Hop Brook. The direction of flow along with the filter and trench visual are labeled (Note: Image is not to scale).

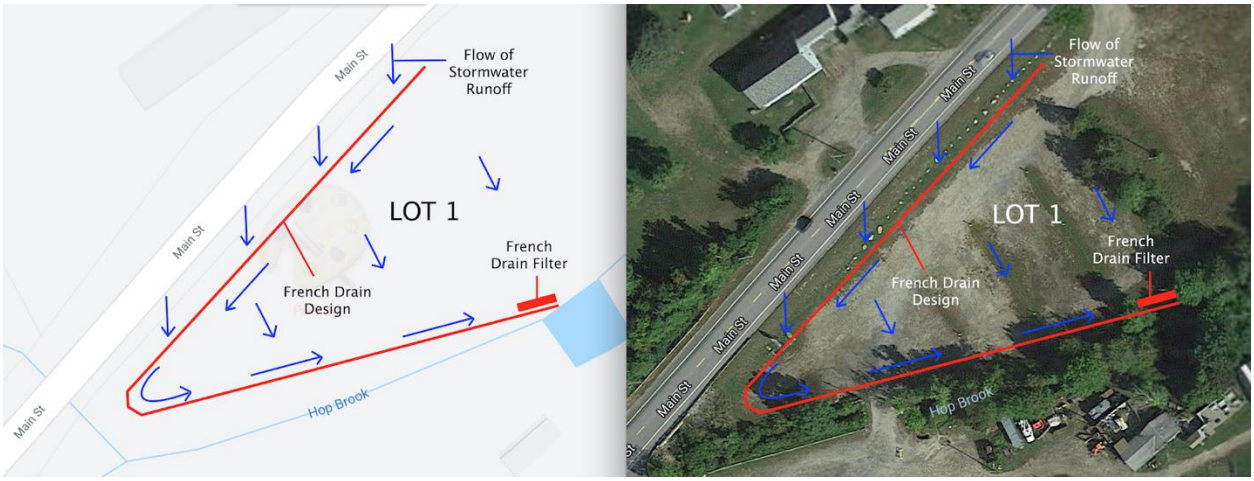


Figure 4.19: This figure depicts the orientation for option 2 of the French Drain design. Both map & satellite images are included to show how runoff will enter Hop Brook. The direction of flow along with the filter and trench visual are labeled (Note: Image is not to scale).

When the linear footage for each option was set in stone, the following list of materials as well as their quantities were chosen.

Option 1

- 8 in. HDPE Dual Wall Hancor Pipe N-12 ST IB (450 ft.)
- 6 oz. Non-woven Hancor Geofabric (5,700 sq. ft.)
- 65 cubic yds of 1 ½ in. crushed stone
- (1) ADS Stormwater Quality Unit (1.5 cfs)
- (2) ADS Soiltight Couplers (8 in. → 10 in.)

Option 2

- 8 in. HDPE Dual Wall Hancor Pipe N-12 ST IB (900 ft.)
- (3) ADS Dual Wall Fabricated 45° Bends
- 6 oz. Non-woven Hancor Geofabric (11,400 sq. ft.)
- 130 cubic yds of 1 ½ in. crushed stone
- (1) ADS Stormwater Quality Unit (1.5 cfs)
- (2) ADS Soiltight Couplers (8 in. → 10 in.)

4.3.3 Releasing Quotes, Receiving Estimates, Owner Approval

A list of materials were sent out for estimates to both John Stelmokas of ADS and a sales rep at Worcester Sand & Gravel. The following figure below shows the response from each supplier. Note that the unit quantity of gravel is in cubic yards.

Ski Ward Hill Parking Lot Materials Estimate

Lot 1

Option 1

Product	Qty.	Rate	Cost	Source
8" Hancor HDPE Dual Wall Pipe N-12 ST IB (NOTE: 20' lengths)	23	$ 5.01	$ 115.23	ADS Pipe
6 oz. Hancor Geofabric (NOTE: 500 sq. yd. per roll)	2	$ 425.00	$ 850.00	ADS Pipe
Storm Water Quality Unit (1.5cfs)	1	$ 8,200.00	$ 8,200.00	ADS Pipe
Soiltight Couplers (8" → 10")	2	$ 25.00	$ 50.00	ADS Pipe
1 1/2" Gravel Component (NOTE: $85-$125 delivery fee)	65	$ 15.00	$ 975.00	Worcester Sand & Gravel
		TOTAL	**$ 10,190.23**	NOTE: Gravel delivery fee not included

Option 2

Product	Qty.	Rate	Cost	Source
8" Hancor HDPE Dual Wall Pipe N-12 ST IB (NOTE: 20' lengths)	45	$ 5.01	$ 225.45	ADS Pipe
6 oz. Hancor Geofabric (NOTE: 500 sq. yd. per roll)	3	$ 425.00	$ 1,275.00	ADS Pipe
8" Dual Wall Fabricated 45 deg. bend	3	$ 50.00	$ 150.00	ADS Pipe
Storm Water Quality Unit (3.66cfs)	1	$ 20,000.00	$ 20,000.00	ADS Pipe
Soiltight Couplers (8" → 12")	2	$ 25.00	$ 50.00	ADS Pipe
1 1/2" Gravel Component (NOTE: $85-$125 delivery fee)	130	$ 15.00	$ 1,950.00	Worcester Sand & Gravel
		TOTAL	**$ 23,650.45**	NOTE: Gravel delivery fee not included

Figure 4.20: This figure depicts a full material estimate for both options of the French Drain design. ADS Pipe and Worcester Sand & Gravel were the only two sources used for the job.

Option 1 received a material estimate of $10,190.23, while option 2 came in at an astounding $23,650.45. Our team then reached out to Lynch Landscaping Company (LLC) and New England Dry Basements (NEDB) for labor estimates. Not only were excavation quantities considered for the trench, an installation estimate was also infered. Once our project team received these estimates, a scope document was created to show comparison. Figures 4.21 and 4.22 shows their responses for both design options.

Ski Ward Hill Resort Lot 1 Scope Sheet Comparison

Option 1

Product	Source	Qty.	Rate	Cost	Notes
8" Hancor HDPE Dual Wall Pipe N-12 ST IB ***NOTE:** 20' lengths	ADS Pipe	23	$5.01	$115.23	
6 oz. Hancor Geofabric ***NOTE:** 500 sq. yd. per roll	ADS Pipe	2	$425.00	$850.00	
Stormwater Quality Unit (1.5cfs)	ADS Pipe	1	$8,200.00	$8,200.00	
Soiltight Couplers (8" → 10")	ADS Pipe	2	$25.00	$50.00	
1 1/2 " Gravel Component ($/yd) ***NOTE:** $85 - $125 delivery fee	Worcester Sand & Gravel	65	$15.00	$975.00	
Labor	Total Materials Cost			**$10,190.23**	‣Gravel delivery fee not included
Labor w/ Excavation (man hrs) 3-4 workers	Lynch Landscaping Co. (LLC)	275	$65.00	$19,675.00	‣Includes $1,800 equipment fee ‣$650 - $800 trucking fee not
Labor w/o Excavation (man hrs) 3-4 workers	Lynch Landscaping Co. (LLC)	200	$65.00	$13,000.00	
Labor w/o Excavation ($/person/day) *NOTE: 5 Days of work, 3 workers	New England Dry Basements (NEDB)	15	$700.00	$10,500.00	
Total Materials & Labor w/ Excavation (LLC)				**$29,865.23**	‣Gravel Delivery Fee Not Included
Total Materials & Labor w/o Excavation (LLC)				**$23,190.23**	‣Gravel Delivery Fee Not Included
Total Materials & Labor w/o Excavation (NEDB)				**$20,690.23**	‣Gravel Delivery Fee Not Included

Figure 4.21: The following figure shows a comparison of scope between Lynch Landscaping, Co. and New England Dry Basements for design option 1.

Option 2

Product	Source	Qty.	Rate	Cost	Notes
8" Hancor HDPE Dual Wall Pipe N-12 ST IB ***NOTE:** 20' lengths	ADS Pipe	45	$5.01	$225.45	
6 oz. Hancor Geofabric ***NOTE:** 500 sq. yd. per roll	ADS Pipe	3	$425.00	$1,275.00	
8" HDPE Dual Wall Fabricated 45° Bend	ADS Pipe	3	$50.00	$150.00	
Stormwater Quality Unit (3.66cfs)	ADS Pipe	1	$20,000.00	$20,000.00	
Soiltight Couplers (8" → 12")	ADS Pipe	2	$25.00	$50.00	
1 1/2 " Gravel Component ($/yd) ***NOTE:** $85 - $125 delivery fee	Worcester Sand & Gravel	65	$15.00	$975.00	
Labor	Total Materials Cost			**$22,675.45**	‣Gravel delivery fee not included
Labor w/ Excavation (man hrs) 3-4 workers	Lynch Landscaping Co. (LLC)	415	$65.00	$28,775.00	‣Includes $1,800 equipment fee ‣$650 - $800 trucking fee not
Labor w/o Excavation (man hrs) 3-4 workers	Lynch Landscaping Co. (LLC)	265	$65.00	$17,225.00	
Labor w/o Excavation ($/person/day) *NOTE: 10 Days of work, 3 workers	New England Dry Basements (NEDB)	30	$700.00	$21,000.00	
Total Materials & Labor w/ Excavation (LLC)				**$51,450.45**	‣Gravel Delivery Fee Not Included
Total Materials & Labor w/o Excavation (LLC)				**$39,900.45**	‣Gravel Delivery Fee Not Included
Total Materials & Labor w/o Excavation (NEDB)				**$43,675.45**	‣Gravel Delivery Fee Not Included

Figure 4.22: The following figure shows a comparison of scope between Lynch Landscaping, Co. and New England Dry Basements for design option 1.

For Option 1, Lynch handed a final cost estimate with materials of $29,865.23, which included excavation. The same company estimated $23,190.23 without excavation. New England Dry Basements came in lower with an estimate of $20,690.23 without excavation. For Option 2, Lynch had a final cost estimate with materials that was $51,450.45, which included excavation.

The same company estimated $39,900.45 without excavation. New England Dry Basements came in lower with an estimate of $43,675.45 without excavation.

Next, schedule and typical lead times were obtained from the same contractors to see how long it would take them to complete the job. Oddly enough, both contractors came in with the same lead times for the task at hand. The following table shows these lead times.

Ski Ward Hill Resort Lot 1 Schedule

Task	Day 1	Day 2	Day 3	Day 4	Day 5	Day 6	Day 7	Day 8	Day 9	Day 10	Day 11	Day 12	Day 13	Day 14
Option 1														
Excavation of 450' long trench with 2% slope (24"depth x 26" width)														
Install base layer of 6oz. geofabric and 1 1/2" gravel along trench length														
Install 8" HDPE Corrugated pipe sections along trench length														
Install stormwater quality unit at the end base of the trench below grade (directed to Hop Brook)														
Backfill trench with remaining 1 1/2" gravel & overlay excess 6oz. Geofabric along trench length														
Truck remaining dirt to an offsite location (OPTIONAL)														
Option 2														
Excavation of 900' long trench with 2% slope (24"depth x 26" width)														
Install base layer of 6oz. geofabric and 1 1/2" gravel along trench length														
Install 8" HDPE Corrugated pipe sections along trench length w/ 45° bends														
Install stormwater quality unit at the end base of the trench below grade (directed to Hop Brook)														
Backfill trench with remaining 1 1/2" gravel & overlay excess 6oz. Geofabric along trench length														
Truck remaining dirt to an offsite location (OPTIONAL)														

Figure 4.23: This figure shows a schedule for each option's task. Each task as well as their lead times are displayed.

Based on figure 4.23, option 1 was estimated to take about 8 days to complete, while option 2 would only take 14 days even though it is double the work.

4.4 Catch Basin Design

Once the drainage design for Lot A was complete, we developed the final design for the catch basins in Lot B. Ultimately we chose catch basins as the drainage system for Lot B due to the natural depressions in the landscape. The figures below illustrate the site profile we created for the Lot.

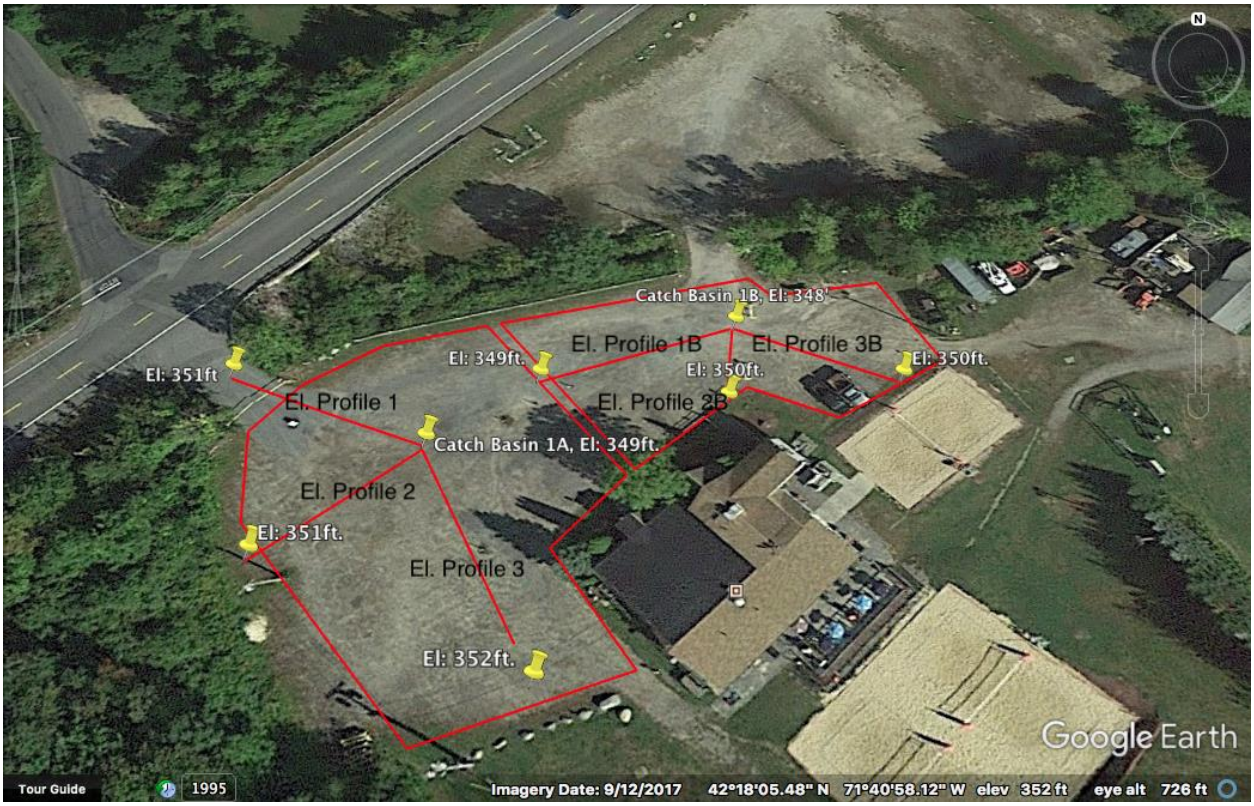


Figure 4.24: Location of Prospective Catch Basins and Surrounding Elevation Points

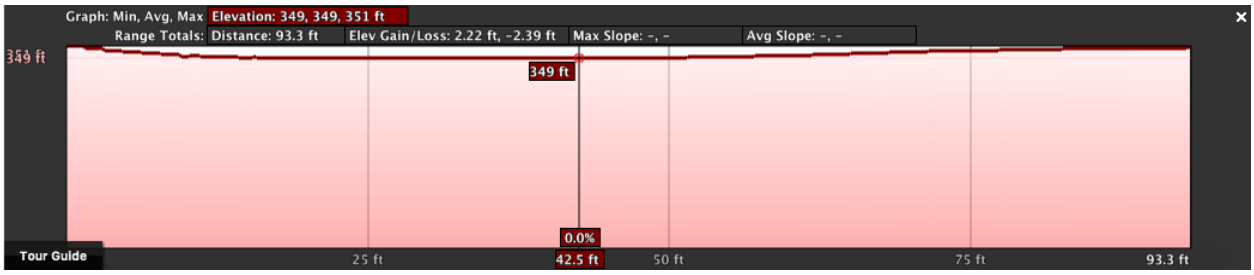


Figure 4.25: Elevation Profile 1

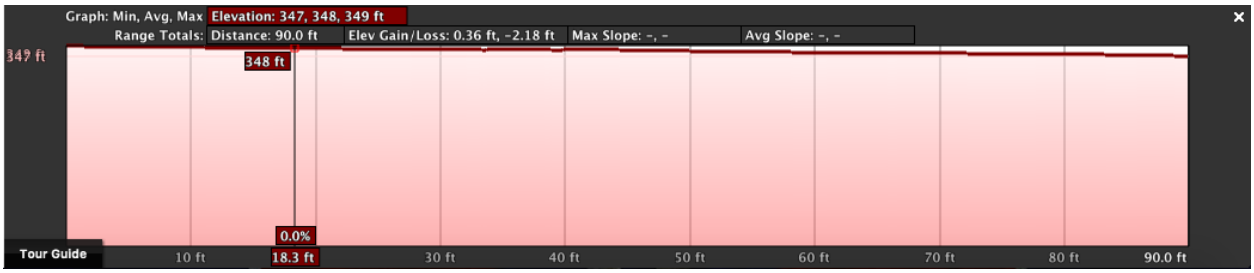


Figure 4.26: Elevation Profile 2

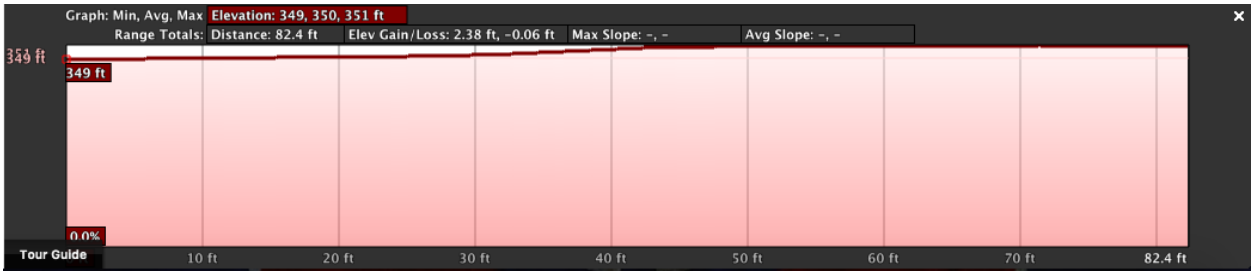


Figure 4.27: Elevation Profile 3

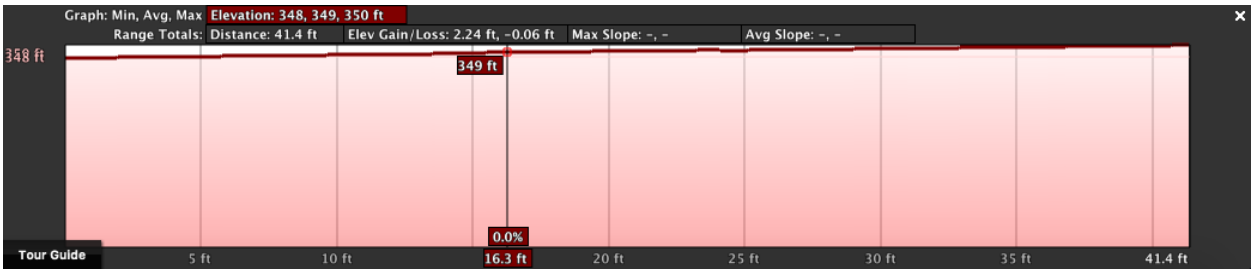


Figure 4.28: Elevation Profile 1B

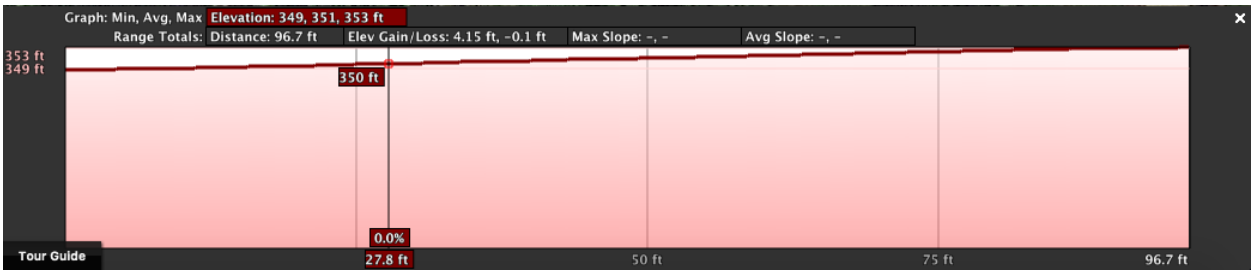


Figure 4.29: Elevation Profile 2B

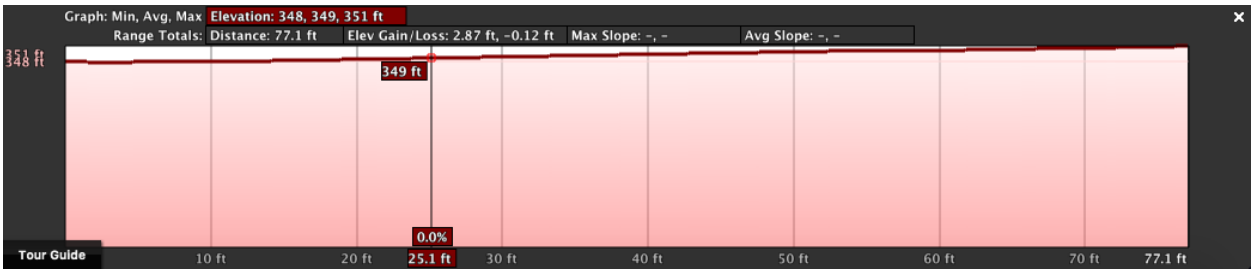


Figure 4.30: Elevation Profile 3B

The pins in Figure 4.24 were dropped into the two depressions within the lot and along the perimeter of the lot using Google Earth in order to illustrate the elevation difference from the outer edge to the center of the depressions. Each pin has a respective elevation labeled beside it. The red lines that connect these pins represent the downward slopes that were traced from each outer elevation pin to the catch basin pins and can be viewed in the correlating figures 4.26 through 4.30. These figures give a 2-dimensional, sectional view of the downward slope that corresponds to each red line in Figure 4.24 and are labeled "Elevation Profile 'X'" in order to show the relationship between the figures. Each red line had about a 1% slope in declination over the distance from elevation pin to catch basin pin. Based on this site profile, we decided that the best locations for the catch basins would be at the following grid coordinates.

Table 4.9: Exact Location of Prospective Catch Basins

	Latitude	Longitude	Elevation (ft)
Catch Basin 1A	42°18'5.94"N	71°40'59.88"W	349
Catch Basin 1B	42°18'6.56"N	71°40'58.12"W	348

Based on the location of each catch basin, our team split up the area of the entire lot so that each catch basin was responsible for roughly half of the lot. You can see in Table 4.9 that there are two perimeter outlines with a catch basin located in the middle of each, labeled "Catch Basin 1A" and "Catch Basin 1B." Since each of these catch basins were responsible for their own halves of the lot, their respective drainage zone areas were different, which resulted in

differing Q values for each. Catch Basin 1A was responsible for draining an area of 19,500sqft and Catch Basin 1B was responsible for draining an area of 11,370sqft. The Q values were then used to find the volume of stormwater produced in each area. Table 4.10 gives the respective Q values and stormwater volumes for each catch basin based on surface material.

Table 4.10: Stormwater Runoff and Volume Values for Each Catch Basin

	Catch Basin 1A		**Catch Basin 1B**	
Surface Material	Gravel-Bare	Concrete/Asphalt	Gravel-Bare	Concrete/Asphalt
Stormwater Runoff (Q)	4.6gal/min	7.1gal/min	2.7gal/min	4.1gal/min
Stormwater Volume (V)	275gal	423gal	160gal	247gal

Based on the volume values in Table 4.10 above, we designed four catch basins with the following dimensions for each surface material.

Table 4.11: Dimensions of Catch Basins for Each Surface Design

	Catch Basin 1A		**Catch Basin 1B**	
	Gravel-Bare	**Concrete/Asphalt**	**Gravel-Bare**	**Concrete/Asphalt**
Height (h)	3ft	4.5ft	2ft	2.75ft
Radius (r)	4ft	4ft	4ft	4ft
Total Volume (V)	285gal	420gal	190gal	255gal

As you can see by comparing Table 4.10 and Table 4.11, the final dimensions for each catch basin design can hold more than the calculated stormwater runoff volumes for a 25-year storm. AutoCAD drawings for our final catch basin designs can be referenced in Appendices P.

The grate design was the next step in finalizing the catch basin design. In order to determine the required area for grate opening, we chose an arbitrary square footage value and tested it in Equation 3.2.4 by checking the flow rate capacity against several ponding depths. Ultimately, the final dimensions chosen were 0.5in x 4.5in rectangular openings which result in a final area of 0.7sqft. This area was applied to Equation 3.2.4 and applied to ponding depths of up

to 1ft. For each ponding depth scenario, the grate had an excess flow rate capacity compared to the flow rate of the stormwater runoff for a 25-year storm. The following table provides the flow rate capacity for our grate design.

Table 4.12: Flow Rate Capacity of Grate vs. Various Ponding Depth

Ponding depth (h)	Flow Rate Capacity of Grate (Qgrate)
.2ft	678gal/min
.4ft	959gal/min
.6ft	1174gal/min
.8ft	1357gal/min
1.0ft	1517gal/min

Compared against Figure above, each of these values for grate flow rate capacity is substantially larger than each value of Q for a 25-year storm.

The grate opening sizes were also compared against the size of the material in our gravel design and the average bike tire widths used on different types of bikes. Our gravel design includes gravel with a minimum diameter of 1.5in. so it would not be able to clog the 0.5in. wide grate openings. The gravel aggregates we use can be referenced in Appendix K. Based upon research, we found that even the thinnest tire Table 4.13 would not get caught in the grate slits.

Table 4.13: Average Bike Tire Widths (BikeTiresDirect.com)

Bike Style	**Tire Width (inches)**
Road Riding	.90
Self-Supported Touring	1.10
Touring/Hybrid	1.85
Mountain Biking	2.0 – 2.5

As for the width and thickness of the grate, MassDOT Construction Standards specify that the grate must be a square frame and have a minimum of 24in width [9]. The thickness of

the grate from top to bottom was determined to be 3in. and the thickness of the grating bars was 1in. A full set of grate drawings can be referenced in Appendix Q.

Once the dimensions were determined, we needed to choose the materials. MassDOT Construction Standards specify that a catch basin for these dimensions must be composed of a precast concrete based and concrete blocks that sit on top to adjust for grade. Each of these components needs to have a strength of f'c=4000psi. MassDOT also specifies that the grate must be made of cast iron with no allowance for black asphalt coating.

Based on these specifications we checked the grate material and design dimensions against applicable load conditions stated in Load Class B of the ACO Design Manual. In order to do this, we calculated the required number of bearing bars, the moment of inertia, and the final deflection based on these loading conditions [9]. Table 4.14 contains these results.

Table 4.14: Deflection of Our Design for a Cast Iron Grate Under Class B Loading Conditions

Grate Material – Cast Iron Steel (E=29,000,000psi)	
Required number of bearing bars	16
Moment of Inertia	1.33in^3
Concentrated Load for Load Class B	28,100lb
Deflection	**0.21in**

The final step of our design process for the drainage system in Lot B was the pump design. Using Equations 3.1.8 and 3.2.5, we calculated the total volume of stormwater runoff that would result from a 10-year and 25-year storm in order to determine the necessary volume of water that must be drained. Table 4.15 contains the results for these calculations.

Table 4.15: Stormwater Runoff Volumes for 10-Year and 25-Year Storms

	Lot 1A		Lot 1B	
	Gravel-Bare	**Concrete/Asphalt**	**Gravel-Bare**	**Concrete/Asphalt**
10 Year Storm Volume (gal)	227.4193	349.8759	132.6029	204.0045
25 Year Storm Volume (gal)	275.0378	423.1351	160.3682	246.7203
Amount to be Pumped in 10 Days (gal/day)	22.7419	34.9876	13.2603	20.4005
Amount to be Pumped in Final 4 Days (gal/day)	11.9046	18.3148	6.9413	10.6789

Based on our calculations, we determined an adequate pump design for each half of Lot A – denoted as "Lot 1A" and "Lot 1B" - based on surface material used. For each case – in terms of flow rate capacity – the required volume of water to be pumped in 10 days governed over the required volume of water to be pumped in the final 4 days. For each design, the minimum pump flow rate capacity would depend on the amount of time spent each day pumping water. Based on Table 4.15, If the owner only wanted to spend 1 hour per day pumping, then the minimum flow rate capacity of a pump for each design would be 25gal/hour for Lot 1A gravel design, 35gal/hour for Lot 1A concrete design, 15gal/hour for Lot 1B gravel design, and 25 gal/hour for Lot 1B concrete design.

Based on the dimensions and material selection of our final design, we received a cost estimate from a professional civil engineering company. Table 4.16 contains the expenses of the materials, delivery, and installation for each catch basin and subsequent grate along with it. The total cost includes the sum of the costs for two catch basins and two grates two account for each half of Lot 1.

Table 4.16: Final Cost Analysis for Catch Basin Drainage Option

	Catch Basin Costs (per basin)	**Grate Costs (per grate)**
Supply with Delivery (including fabrication and material costs)	$1,000.00	$450.00
Installation (Labor and Equipment)	$3,000.00	$250.00
Totals	$4,000.00	$700.00
Total Cost of Project	**$9,400.00**	

5.0 Conclusion and Final Recommendations

In this project, our team developed four possible solutions to improve the drainage and usage of Ski Ward Hill's parking lots:

1. Gravel Resurfacing
2. Asphalt Cement Resurfacing
3. French Drain Implementation
4. Catch Basin Implementation

These solutions were designed in accordance with MassDOT construction standards, the Standard Specifications for Highways and Bridges, and other relevant research. The team utilized official town documentation, the national weather service, AutoCAD drawings produced by Shrewsbury town engineers, as well as several other online resources in our research and analysis of the lots. Once we determined the final dimensions and materials for each of the designs we were able to successful design the solutions in AutoCAD. After evaluating each solution and the needs of the parking lots, the team has proposed the following recommendations for Ski Ward Hill in regard to the design of this project.

5.1 Lot A

5.1.1 Structural Design

For Lot A our team determined that the most effective solution to increase the durability would be to resurface the entire area with the Asphalt Concrete. Based on our analysis of MassDOT Design and Construction documents, we recommend the design be composed of 3 layers; Finish, Base and Subbase. The Finish Layer is critical because it is the strongest layer of the mix design. This layer has to be impermeable so that runoff water and snow melt do not penetrate and the affect this layer. If water is able to permeate into the Finish layer, then cracking and potholing can occur. The Base and Subbase layers are used to provide support to the Finish layer. In our design we recommend the Finish layer is 1 in., Base Layer is 2 in. and Subbase Layer is 12 in.. These were the standard layer depths that MassDOT required for parking areas, so we wanted to strictly follow those requirements to adhere to the State's regulations.

5.1.2 Material Design

The materials we chose for this design were primarily based of gradation distribution tests. The aggregates chosen for the design had to be within the upper and lower % passing limits

established by MassDOT. This meant that the aggregate samples had to be comprised of the right sized particles. For our layers of the design, The Finish layer called for Coarse and Fine aggregates, the Base layer called for Coarse and Fine aggregates and the Subbase layer called for Coarse aggregates. We had to find a local aggregate distributor whose aggregate samples sizes would meet MassDOT's gradation distribution requirements. We chose Delta Sand as the aggregate distributor. After performed gradation distribution analysis of the materials, we chose the following aggregate for the layers.

- Finish Layer
  - Coarse – ¾" Blend Crushed Trap Rock
  - Fine – Concrete Sand
- Base Layer
  - Coarse – Processed Quarry Rock
  - Fine – Gravel Stone Dust
- Subbase Layer
  - Coarse – 1.5" Minus Crushed Gravel

The two layers that required other materials besides aggregates were the Finish layer and the Base Layer. The Finish layer required a concrete mix that included admixtures that would prevent damage from freezing and thawing of ice. We researched concrete mix companies and found that Dauphinias Concrete provided MassDOT approved mixes that incorporated the aggregates we chose as well as the correct admixtures. Finally, for the Base layer, we needed to select a binder material. MassDOT provided 4 possible binders that could be used so we chose the AC-5 binder material, which has high strength, needed to support the consistent car traffic.

5.2 Lot B

5.2.1 Structural Design

For Lot B, our team determined that the most effective solution to increase drainage and usage of the area would be to install two separate, precast concrete catch basins. Based on our analysis, we recommend that each catch basin be placed at the following coordinates:

	Latitude	Longitude	Elevation (ft)
Catch Basin 1A	42°18'5.94"N	71°40'59.88"W	349
Catch Basin 1B	42°18'6.56"N	71°40'58.12"W	348

Having two catch basins at these locations decreases the amount of stormwater runoff that will saturate into the soil before draining into the catch basin and provides for a smaller square footage area that each catch basin is responsible for draining. Based on our calculations and analysis, we recommend that catch basin 1A have a height of 3 feet and a diameter of 4 feet and that catch basin 1B have a height of 2 feet and a diameter of 4 feet in order to have a volume capacity large enough to withstand the rainfall of a 25-year storm. The advantage of implementing these catch basins is that they are installed underground and do not have any negative environmental impacts, but the owner should be responsible in the treatment and disposal of the water from the catch basin. We recommend that the owner acquire a license for stormwater treatment and disposal, but if he cannot then he should hire a professional service to do so for him. The catch basins are also relatively easily maintainable, but the owner should regularly inspect the catch basins for debris and overall cleanliness in order to avoid costly repairs.

Our team also recommends that each catch basin be capped with a 24x24.5in., cast iron steel grate. The cast iron steel grate should have four rows of fifteen 0.5x4.5in. openings that allow for adequate flow rate capacity in order to drain stormwater runoff without clogging and causing ponding. We recommend that the grate be oriented so that the lengthwise openings on

the grate are parallel to the direction of vehicular travel and perpendicular to the most likely direction of foot travel. Orienting the grate in this way creates less of a chance that a person walking over the grate will get their foot caught in an opening. Similarly, to the catch basin, we recommend that the owner regularly inspects the grates for any debris that may clog the openings in order to avoid decreased flow capacity and other damage to the grates.

5.2.2 Material Design

For the material of the catch basins, we recommend precast concrete with a strength of f'c = 4000psi. This is in accordance with MassDOT construction standards and provides a strength that is able to withstand the loading that corresponds to the loading class of Ski Ward Hill's parking lot, which is Load Class B. Using precast concrete also allows for ease of installation since the catch basin will be delivered to the site prefabricated, which will result in a lower cost. Our team also determined that the material for the grate should be cast iron steel. Cast iron steel is in accordance with MassDOT construction standards and can withstand the heaviest concentrated load in Load Class B (28,100lb) with a deflection of only 0.21in.

5.3 Final Recommendations

Our team's final recommendation to Ski Ward Hill is to consider the long-term cost-benefit ratio. The advantages of spending more money for a longer-term solution outweigh the advantages of spending less money for an immediate solution. Going with the easy, cheap, short-term solution will most likely result in repeated expenses associated with maintenance and inefficient drainage as well as a loss in profits due to customer dissatisfaction. If the owner of Ski Ward Hill decides to pursue our recommended solution, he will most likely save money in the long run due to the durability and effectiveness of our recommendation.

Professional Licensure Statement

In order to ensure that a project has been properly designed, engineering firms are required to have a Professional Engineer (PE) sign off on the project. Being a PE indicates that one has developed strong capabilities in engineering design. This role is quite important, since a PE takes responsibility for a project in its entirety by signing off on it. To become a PE, one must first graduate from an accredited engineering program. The individual also has to have taken and passed the Fundamentals of Engineering (FE) Exam to become an Engineer in Training (EIT). After working in professional practice for four years as an EIT, the individual must pass the Principles and Practices of Engineering (PE) exam to receive professional licensure in his or her given state. Professional licensure is important on both an individual and community-wide basis. Individually, passing the PE is an important step in one's engineering career. It signifies that one has reached a high level of expertise in engineer design. Communities that hire engineering firms benefit from having a professional engineer sign off on the finished project as it signifies that the project has reached high levels of health and safety standards through the design, review, and supervision of professional practice. The proposed alternative stormwater management designs and developer template would require a stamp of a licensed PE in order to be implemented. These deliverables are preliminary and would require further review by a PE in order to ensure that they comply with state engineering standards.

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Appendices

Appendix A

Lot A

Tables — Sand Source — Summary By Map Unit

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Map unit symbol	Map unit name	Rating	Component name (percent)	Rating reasons (numeric values)	Acres in AOI	Percent of AOI
72A	Whitman loam, 0 to 3 percent slopes	Fair	Whitman (70%)	Bottom layer (0.00)	0.2	15.8%
				Thickest layer (0.02)		
651	Udorthents, smoothed	Not rated	Udorthents (80%)		0.9	84.2%
			Urban land (20%)			
Totals for Area of Interest					**1.1**	**100.0%**

Table — Sand Source — Summary by Rating Value

Summary by Rating Value

Summary by Rating Value

Rating	Acres in AOI	Percent of AOI
Fair	0.2	15.8%
Null or Not Rated	0.9	84.2%
Totals for Area of Interest	**1.1**	**100.0%**

Lot B

Tables — Sand Source — Summary By Map Unit

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Map unit symbol	Map unit name	Rating	Component name (percent)	Rating reasons (numeric values)	Acres in AOI	Percent of AOI
72A	Whitman loam, 0 to 3 percent slopes	Fair	Whitman (70%)	Bottom layer (0.00)	0.7	84.3%
				Thickest layer (0.02)		
651	Udorthents, smoothed	Not rated	Udorthents (80%)		0.1	15.7%
			Urban land (20%)			
Totals for Area of Interest					**0.8**	**100.0%**

Table — Sand Source — Summary by Rating Value

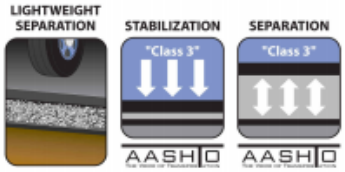
Summary by Rating Value

Summary by Rating Value

Rating	Acres in AOI	Percent of AOI
Fair	0.7	84.3%
Null or Not Rated	0.1	15.7%
Totals for Area of Interest	**0.8**	**100.0%**

Appendix B

Construction Geosynthetics



US 200

NTPEP APPROVED - GTX-2016-01-086. US 200 is a woven geotextile made of 100% polypropylene slit film yarns. US 200 resists ultraviolet and biological deterioration, rotting, naturally encountered basics and acids. Polypropylene is stable within a pH range of 2 to 13. US 200 will satisfy the requirements as outlined in AASHTO M-288-06 for Class 3 Stabilization & Separation applications and meets the following M.A.R.V. values except where noted:

Property	Test Method	English	Metric
Weight - Typical	ASTM D-5261	4.0 oz/sy	136 g/sm
Tensile Strength	ASTM D-4632	200 lbs	890 N
Elongation @ Break	ASTM D-4632	15%	15%
Mullen Burst*	ASTM D-3786*	400 psi	2,756 kPa
Puncture Strength*	ASTM D-4833*	90 lbs	400 N
CBR Puncture	ASTM D-6241	700 lbs	3,115 N
Trapezoidal Tear	ASTM D-4533	75 lbs	334 N
Apparent Opening Size	ASTM D-4751	40 US Sieve	0.43 mm
Permittivity	ASTM D-4491	0.05 Sec-1	0.05 Sec-1
Water Flow Rate	ASTM D-4491	5 g/min/sf	203 l/min/sm
UV Resistance @ 500 Hours	ASTM D-4355	70%	70%

Roll Size	Roll Diameter	Area	Weight
12.5' x 432'	12.0 in	600 sys	200 lbs
15' x 360'	12.0 in	600 sys	200 lbs
17.5' x 309'	12.0 in	600 sys	200 lbs

* Historical averages (current values not available): Mullen Burst Strength ASTM D3786 is no longer recognized by ASTM D-35 on Geosynthetics as an acceptable test method. Puncture Strength ASTM D4833 is not recognized by AASHTO M288 and has been replaced with CBR Puncture ASTM D6241.

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Appendix C

DESIGN AND CONSTRUCTION GUIDELINES AND STANDARDS
DIVISION 32 • EXTERIOR IMPROVEMENTS

32 12 00 • ASPHALT PAVING

ROADS & SIDEWALKS

DESIGN

Recommended course thickness for roadways and parking lots:

- 12 inch processed gravel or reclaimed paving base course
- 2 inch binder course
- 1 inch finish course

or:

- 12 inch processed gravel or reclaimed paving base course
- 1-1/2 inch binder course
- 1-1/2 inch finish course

Recommended course thickness for sidewalks:

- 8 inch processed gravel base (No reclaimed material)
- 1-1/2 inch binder course
- 1 inch finish course

Mix designs should be provided as part of the design submittal process during construction. All mixtures delivered to the job site shall be accompanied with a certificate of compliance provided by the asphalt batching plant and countersigned by the paving contractor.

Two finish courses are not acceptable because the materials are too similar and will not adhere.

MATERIALS

Materials must comply with the Standard Specifications for Highways and Bridges, latest edition, of the Department of Public Works of the Commonwealth of Massachusetts. Consult the local DPW to determine whether their requirements are more stringent than state regulations.

Subgrade – subgrade shall be either Type 1, 2, 3, or 4 material in accordance with related specifications.

Sub-base – sub-base shall be type 6 screened gravel material in accordance with related specifications.

Binder Course – binder course shall be Class 1 Bituminous Concrete Base Course Type I-1 per the Massachusetts State Highway Specifications, current edition.

Finish Course – Finish course shall be Class 1 Bituminous Concrete Pavement per the Mass. Highway Specifications, current edition.

Curbs – curbs may be vertical granite or Cape Cod bituminous asphalt. (See figure 1 ands 2)

Vertical bituminous curbs will only be permitted in to match existing curbing.

Appendix D

M3.01.0 Asphalt Cement.

Materials of this type shall conform to AASHTO-M226, Table 2, with the additional requirement for Ductility at 60 °F.

Table 2
Specification Requirements for Asphalt Cement
Viscosity Graded at 140 °F (60 °C)

	Viscosity Grade							
	AC-5		AC-10		AC-20		AC-40	
Tests	Min	Max	Min	Max	Min	Max	Min	Max
Viscosity, 140 °F (60 °C), poises	500 ± 100		1000 ± 200		2000 ± 400		4000 ± 800	
Viscosity, 275 °F (135 °C), Cs	175		250		300		400	
Penetration, 77 °F (25 °C), 100g, 5 Sec.	140		80		60		40	
Flash Point, COC, °F	350		425		450		450	
Solubility in trichloroethylene %	99.0		99.0		99.0		99.0	
Tests on residue — Thin Film Oven Test								
Loss on Heating %		1.00		0.50		0.50*		0.50
Ductility, 60 °F (15.5 °C) 5 cm/min. cm	100		40		20			
Ductility, 77 °F (25 °C) 5 cm/min. cm	100		75		50		25	
Maximum viscosity, 140 °F (60 °C), poises		2000		4000		8000		16000

Ductility test temperature will be specified by the Research and Materials Engineer.

*Up to 1 % if approved by the Engineer.

Appendix E

Description — Drainage Class

"Drainage class (natural)" refers to the frequency and duration of wet periods under conditions similar to those under which the soil formed. Alterations of the water regime by human activities, either through drainage or irrigation, are not a consideration unless they have significantly changed the morphology of the soil. Seven classes of natural soil drainage are recognized-excessively drained, somewhat excessively drained, well drained, moderately well drained, somewhat poorly drained, poorly drained, and very poorly drained. These classes are defined in the "Soil Survey Manual."

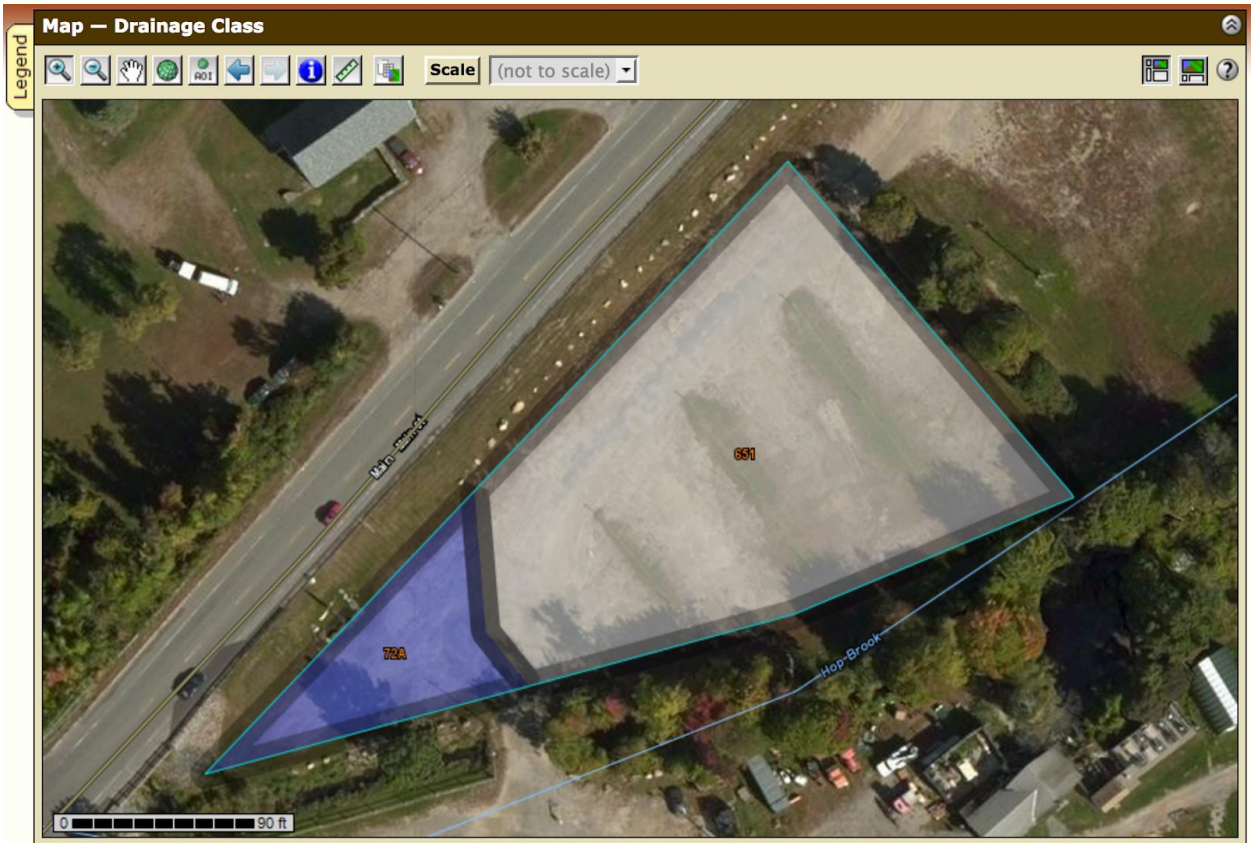
Tables — Drainage Class — Summary By Map Unit

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
72A	Whitman loam, 0 to 3 percent slopes	Very poorly drained	0.2	15.8%
651	Udorthents, smoothed		0.9	84.2%
Totals for Area of Interest			**1.1**	**100.0%**

Map — Drainage Class



Description — Gravel Source

Gravel consists of natural aggregates (2 to 75 millimeters in diameter) suitable for commercial use with a minimum of processing. It is used in many kinds of construction. Specifications for each use vary widely. Only the probability of finding material in suitable quantity is evaluated. The suitability of the material for specific purposes is not evaluated, nor are factors that affect excavation of the material.

The properties used to evaluate the soil as a source of gravel are gradation of grain sizes (as indicated by the Unified classification of the soil), the thickness of suitable material, and the content of rock fragments. If the bottom layer of the soil contains gravel, the soil is considered a likely source regardless of thickness. The assumption is that the gravel layer below the depth of observation exceeds the minimum thickness. The ratings are for the whole soil, from the surface to a depth of about 6 feet. Coarse fragments of soft bedrock, such as shale and siltstone, are not considered to be gravel.

The soils are rated "good," "fair," or "poor" as potential sources of gravel. A rating of "good" or "fair" means that the source material is likely to be in or below the soil. The bottom layer and the thickest layer of the soils are assigned numerical ratings. These ratings indicate the likelihood that the layer is a source of gravel. The number 0.00 indicates that the layer is a poor source. The number 1.00 indicates that the layer is a good source. A number between 0.00 and 1.00 indicates the degree to which the layer is a likely source.

The map unit components listed for each map unit in the accompanying Summary by Map Unit table in Web Soil Survey or the Aggregation Report in Soil Data Viewer are determined by the aggregation method chosen. An aggregated rating class is shown for each map unit. The components listed for each map unit are only those that have the same rating class as listed for the map unit. The percent composition of each component in a particular map unit is presented to help the user better understand the percentage of each map unit that has the rating presented.

Other components with different ratings may be present in each map unit. The ratings for all components, regardless of the map unit aggregated rating, can be viewed by generating the equivalent report from the Soil Reports tab in Web Soil Survey or from the Soil Data Mart site. Onsite investigation may be needed to validate these interpretations and to confirm the identity of the soil on a given site.

Tables — Gravel Source — Summary By Map Unit

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Map unit symbol	Map unit name	Rating	Component name (percent)	Rating reasons (numeric values)	Acres in AOI	Percent of AOI
72A	Whitman loam, 0 to 3 percent slopes	Poor	Whitman (70%)	Bottom layer (0.00)	0.2	15.8%
				Thickest layer (0.00)		
651	Udorthents, smoothed	Not rated	Udorthents (80%)		0.9	84.2%
			Urban land (20%)			
Totals for Area of Interest					**1.1**	**100.0%**

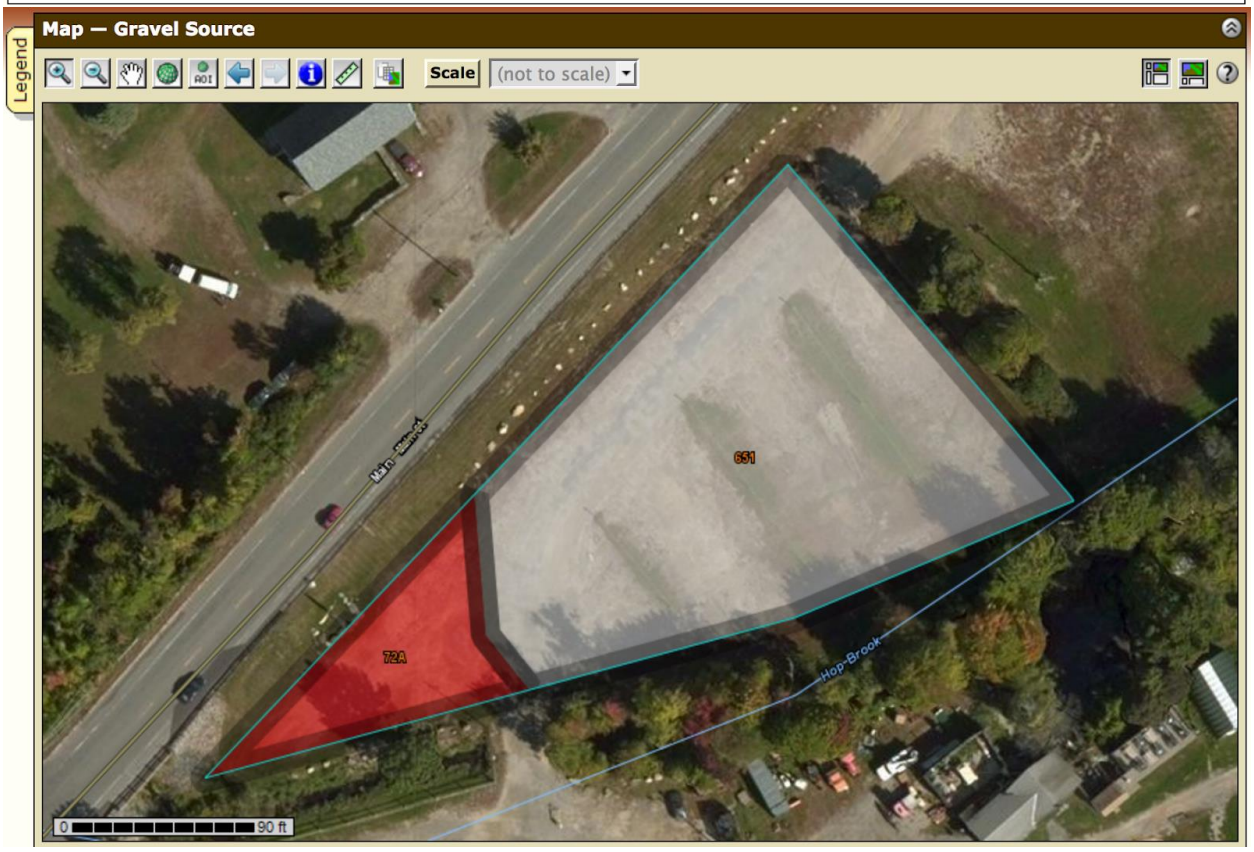
Table — Gravel Source — Summary by Rating Value

Summary by Rating Value

Summary by Rating Value

Rating	Acres in AOI	Percent of AOI
Poor	0.2	15.8%
Null or Not Rated	0.9	84.2%
Totals for Area of Interest	**1.1**	**100.0%**

Map — Gravel Source



Description — Parent Material Name

Parent material name is a term for the general physical, chemical, and mineralogical composition of the unconsolidated material, mineral or organic, in which the soil forms. Mode of deposition and/or weathering may be implied by the name.

The soil surveyor uses parent material to develop a model used for soil mapping. Soil scientists and specialists in other disciplines use parent material to help interpret soil boundaries and project performance of the material below the soil. Many soil properties relate to parent material. Among these properties are proportions of sand, silt, and clay; chemical content; bulk density; structure; and the kinds and amounts of rock fragments. These properties affect interpretations and may be criteria used to separate soil series. Soil properties and landscape information may imply the kind of parent material.

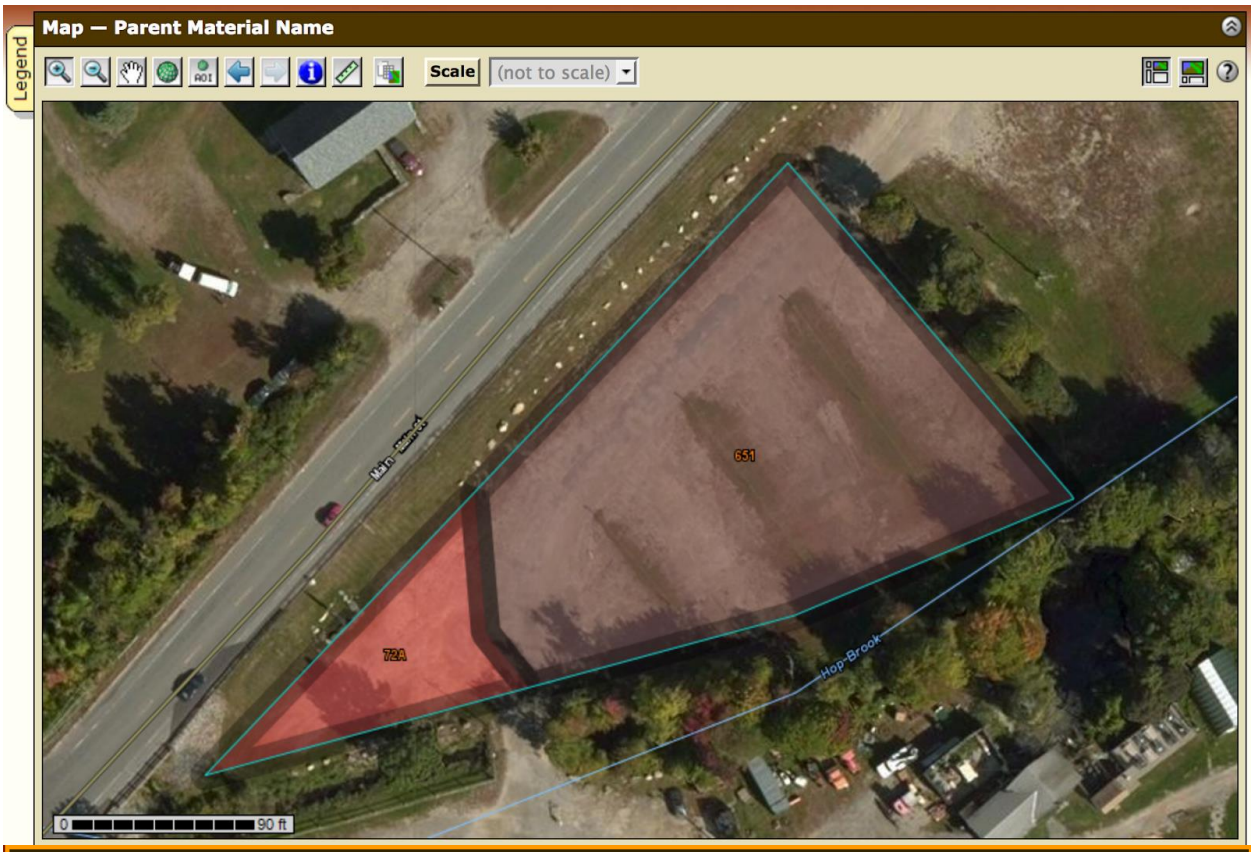
For each soil in the database, one or more parent materials may be identified. One is marked as the representative or most commonly occurring. The representative parent material name is presented here.

Tables — Parent Material Name — Summary By Map Unit

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
72A	Whitman loam, 0 to 3 percent slopes	friable coarse-loamy eolian deposits over dense coarse-loamy lodgment till derived from granite and gneiss	0.2	15.8%
651	Udorthents, smoothed	made land over firm loamy basal till	0.9	84.2%
Totals for Area of Interest			**1.1**	**100.0%**



Description — Ponding Frequency Class

Ponding is standing water in a closed depression. The water is removed only by deep percolation, transpiration, or evaporation or by a combination of these processes. Ponding frequency classes are based on the number of times that ponding occurs over a given period. Frequency is expressed as none, rare, occasional, and frequent.

"None" means that ponding is not probable. The chance of ponding is nearly 0 percent in any year.

"Rare" means that ponding is unlikely but possible under unusual weather conditions. The chance of ponding is nearly 0 percent to 5 percent in any year.

"Occasional" means that ponding occurs, on the average, once or less in 2 years. The chance of ponding is 5 to 50 percent in any year.

"Frequent" means that ponding occurs, on the average, more than once in 2 years. The chance of ponding is more than 50 percent in any year.

Tables — Ponding Frequency Class — Summary By Map Unit

Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

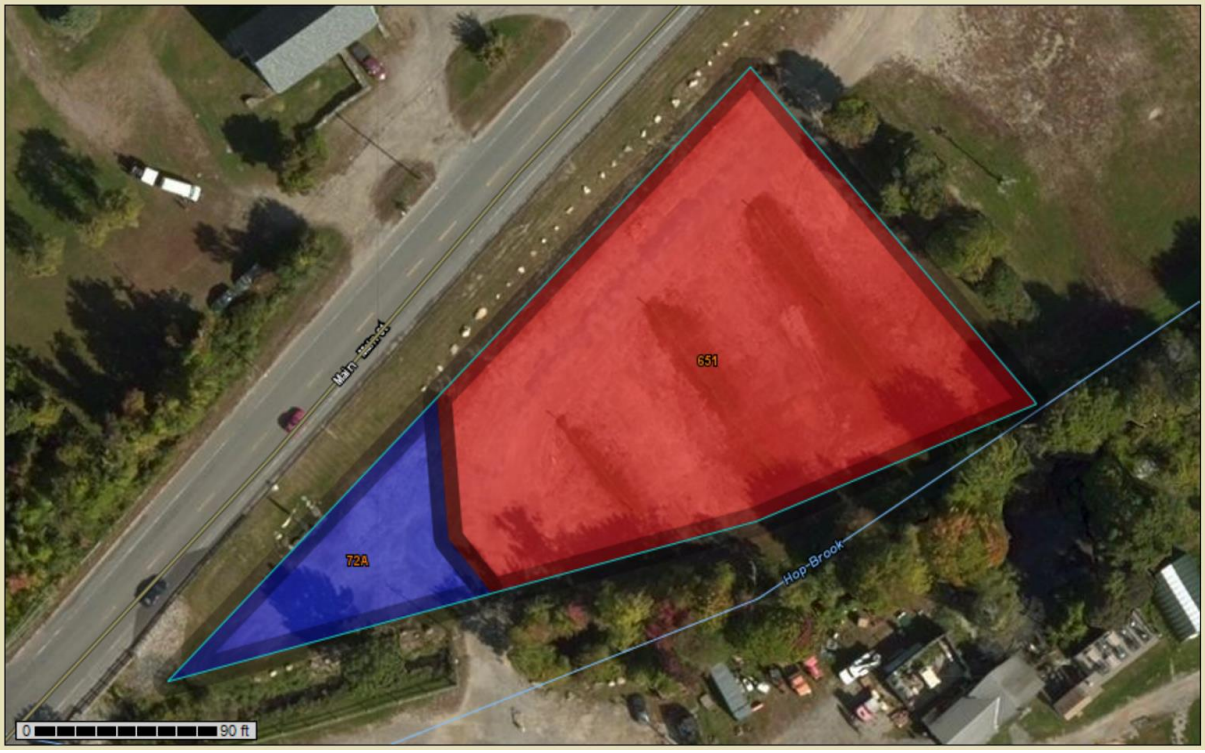
Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Map unit symbol	Map unit name	Rating	Acres in AOI	Percent of AOI
72A	Whitman loam, 0 to 3 percent slopes	Frequent	0.2	15.8%
651	Udorthents, smoothed	None	0.9	84.2%
Totals for Area of Interest			**1.1**	**100.0%**

Map — Ponding Frequency Class

Legend

Scale (not to scale)



Description — Sand Source

Sand is a natural aggregate (0.05 millimeter to 2 millimeters in diameter) suitable for commercial use with a minimum of processing. It is used in many kinds of construction. Specifications for each use vary widely. Only the probability of finding material in suitable quantity is evaluated. The suitability of the material for specific purposes is not evaluated, nor are factors that affect excavation of the material.

The properties used to evaluate the soil as a source of sand are gradation of grain sizes (as indicated by the Unified classification of the soil), the thickness of suitable material, and the content of rock fragments. If the bottom layer of the soil contains sand, the soil is considered a likely source regardless of thickness. The assumption is that the sand layer below the depth of observation exceeds the minimum thickness. The ratings are for the whole soil, from the surface to a depth of about 6 feet.

The soils are rated "good," "fair," or "poor" as potential sources of sand. A rating of "good" or "fair" means that sand is likely to be in or below the soil. The bottom layer and the thickest layer of the soil are assigned numerical ratings. These ratings indicate the likelihood that the layer is a source of sand. The number 0.00 indicates that the layer is a "poor source." The number 1.00 indicates that the layer is a "good source." A number between 0.00 and 1.00 indicates the degree to which the layer is a likely source.

The map unit components listed for each map unit in the accompanying Summary by Map Unit table in Web Soil Survey or the Aggregation Report in Soil Data Viewer are determined by the aggregation method chosen. An aggregated rating class is shown for each map unit. The components listed for each map unit are only those that have the same rating class as listed for the map unit. The percent composition of each component in a particular map unit is presented to help the user better understand the percentage of each map unit that has the rating presented.

Other components with different ratings may be present in each map unit. The ratings for all components, regardless of the map unit aggregated rating, can be viewed by generating the equivalent report from the Soil Reports tab in Web Soil Survey or from the Soil Data Mart site. Onsite investigation may be needed to validate these interpretations and to confirm the identity of the soil on a given site.

Tables — Sand Source — Summary By Map Unit

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Summary by Map Unit — Worcester County, Massachusetts, Northeastern Part (MA613)

Map unit symbol	Map unit name	Rating	Component name (percent)	Rating reasons (numeric values)	Acres in AOI	Percent of AOI
72A	Whitman loam, 0 to 3 percent slopes	Fair	Whitman (70%)	Bottom layer (0.00)	0.2	15.8%
				Thickest layer (0.02)		
651	Udorthents, smoothed	Not rated	Udorthents (80%)		0.9	84.2%
			Urban land (20%)			
Totals for Area of Interest					**1.1**	**100.0%**

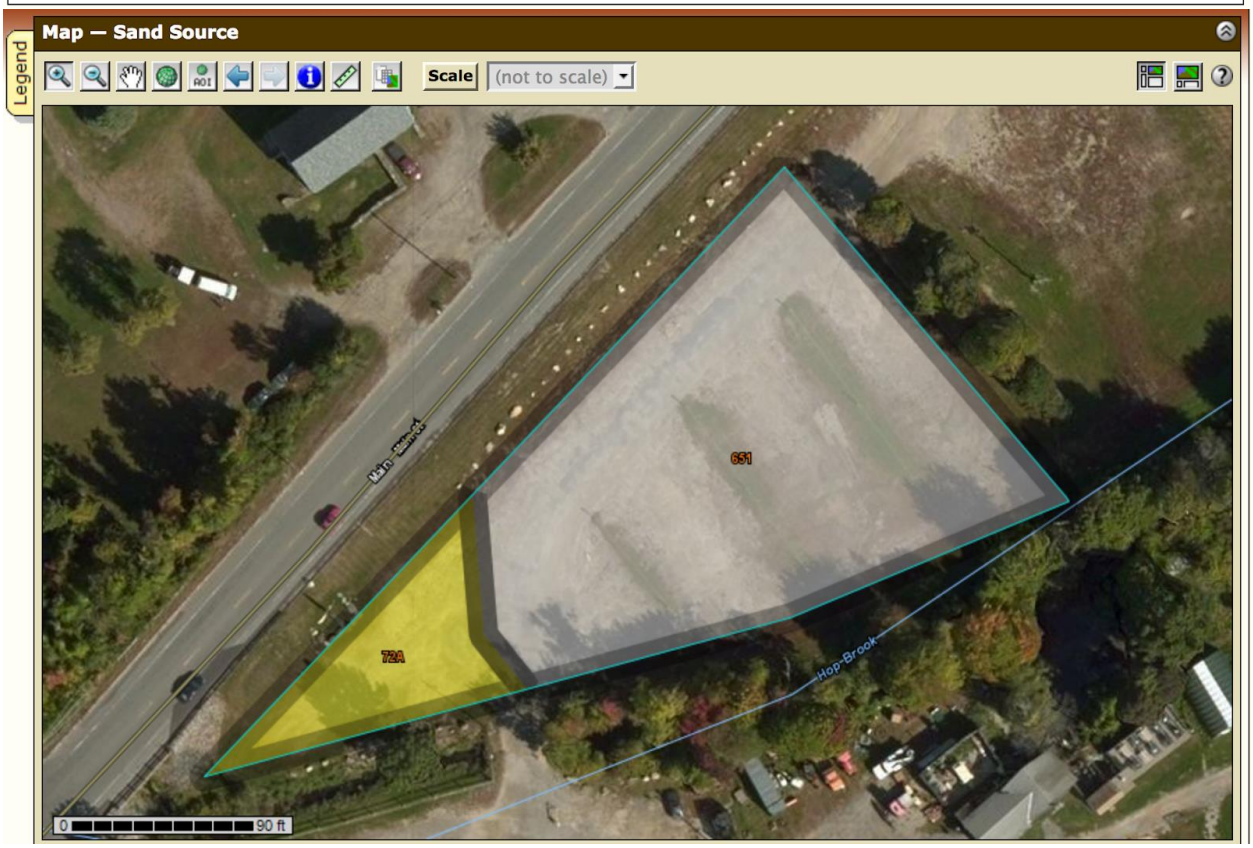
Table — Sand Source — Summary by Rating Value

Summary by Rating Value

Summary by Rating Value

Rating	Acres in AOI	Percent of AOI
Fair	0.2	15.8%
Null or Not Rated	0.9	84.2%
Totals for Area of Interest	**1.1**	**100.0%**

Map — Sand Source



Appendix F

Character of Surface	Range of Runoff Coefficients	Recommended Value*
Pavement		
Asphaltic and Concrete	0.70–0.95	0.85
Brick	0.75–0.85	0.80
Roofs	0.75–0.95	0.85
Lawns, sandy soil		
Flat, 2%	0.05–0.10	0.08
Average, 2 to 7%	0.10–0.15	0.13
Steep, 7%	0.15–0.20	0.18
Lawns, heavy soil		
Flat, 2%	0.13–0.17	0.15
Average, 2 to 7%	0.18–0.22	0.20
Steep, 7%	0.25–0.35	0.30

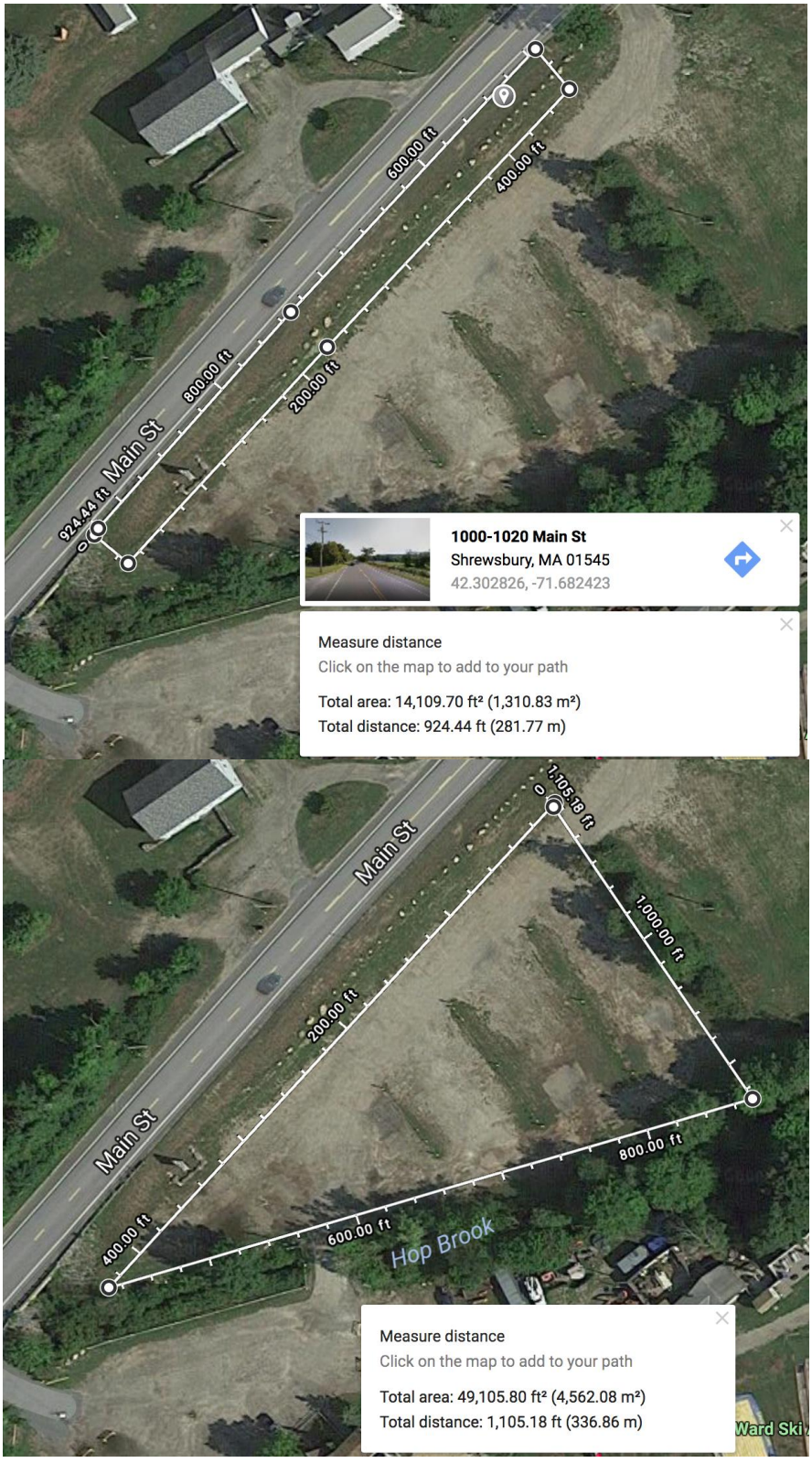
The coefficients in these two tabulations are applicable for storms of 5- to 10-year frequencies. Less frequent, higher intensity storms will require the use of higher coefficients because infiltration and other losses have a proportionally smaller effect on runoff. The coefficients are based on the assumption that the design storm does not occur when the ground surface is frozen.

*Recommended value not included in original source.

Source: Design and Construction of Sanitary and Storm Sewers, American Society of Civil Engineers, New York, p. 332, 1969.

Figure 2: Typical composite Rational method runoff coefficients, after McCuen (2004).

Appendix G


600.00 ft
400.00 ft
800.00 ft
200.00 ft
924.44 ft
Main St
1000-1020 Main St
Shrewsbury, MA 01545
42.302826, -71.682423
Measure distance
Click on the map to add to your path
Total area: 14,109.70 ft² (1,310.83 m²)
Total distance: 924.44 ft (281.77 m)
1,105.18 ft
1,000.00 ft
200.00 ft
800.00 ft
600.00 ft
400.00 ft
Main St
Hop Brook
Measure distance
Click on the map to add to your path
Total area: 49,105.80 ft² (4,562.08 m²)
Total distance: 1,105.18 ft (336.86 m)
Ward Ski

Appendix H

3-3 THE CONVEYANCE METHOD

Conveyance provides a convenient means of selecting a variety of pipe options that will satisfy a project's flow requirements. Conveyance factors are based on a greatly simplified version of the Manning's equation shown in Equation 3-1 and 3-1 (a). In the following discussion, example problems and subsequent sections, the pipe is assumed to be flowing full. This assumption typically allows for a simplified, yet accurate analysis of the given conditions. Each project should be evaluated on a case-by-case basis to determine the best, most-representative design method.

$$Q = \frac{(1.486)(A)(R^{2/3})(S^{1/2})}{n}$$ Equation 3-1

Where:

Q = pipe capacity, (cfs)
n = Manning's "n"
A = cross-sectional flow area of the pipe ($ft.^2$)
R = hydraulic radius (ft.):

$$R = \frac{A}{P}$$

P = Wetted perimeter (ft); Pipe inside circumference, or (π)(inside diameter) for full flowing pipe conditions

S = pipe slope (feet/foot)

Or, in metric units:

$$Q = \frac{(A)(R^{2/3})(S^{1/2})}{n}$$ Equation 3-1(a)

Where:

Q = pipe capacity, m^3/s
n = Manning's "n"
A = cross-sectional flow-area of the pipe (m^2)
P = Wetted perimeter (ft); Pipe circumference, or (π)(diameter) for full flowing pipe conditions

R = hydraulic radius (m),

$$R = \frac{A}{P}$$

S = pipe slope (m/m)

3-4 MINIMUM VELOCITY CONSIDERATIONS

Sediment can reduce the capacity of a stormwater pipe over time. In some installations, it may render the pipe useless until the system can be cleaned. This is an expensive, time-consuming undertaking so preventative measures should be taken during design. Sedimentation is of great concern in sewer applications since large, heavy grit may be present.

To minimize potential problems, flow should be maintained at a minimum, or self-cleansing, velocity. A commonly accepted self-cleansing velocity for storm and sanitary sewers is 3 fps (0.9 m/s). In each design, a final check should be performed to compare the expected velocity with the self-cleansing velocity. The design velocity for full-flowing pipes can be *approximated* with Equation 3-5:

$$V = \frac{Q}{A}$$ Equation 3-5

The potential for settling is determined by the specific gravity and diameter of particle, its cohesive properties, flow velocity, and the roughness of the pipe interior. For further discussion on the complexities and variables associated with determining the self-cleansing velocity for a specific pipe diameter and material, refer to ASCE publication No. 60, "Gravity Sanitary Sewer Design and Construction." In some specialized installations where sediment is a known problem it may be wise to perform a soil analysis prior to final drainage design.

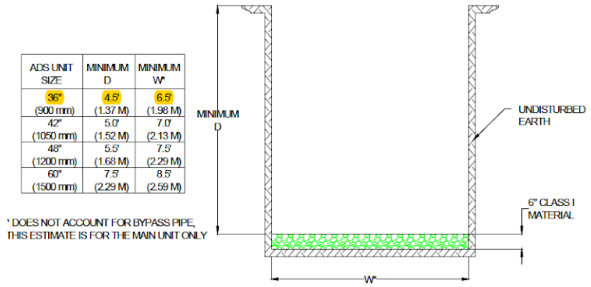
Appendix I

Trench Construction

- Trench or ditch should be just wide enough to place and compact backfill around the entire pipe. A minimum width of OD + 36" but no greater than OD + 72" is recommended. *Trench width does not account for the bypass pipe, this estimate is for the main unit only.*
- As with any pipe, groundwater or seasonal high water tables may impede installation. De-watering is necessary for safe, efficient installation.

ADS UNIT SIZE	MINIMUM D	MINIMUM W*
36" (900 mm)	4.5' (1.37 M)	6.5' (1.98 M)
42" (1050 mm)	5.0' (1.52 M)	7.0' (2.13 M)
48" (1200 mm)	5.5' (1.68 M)	7.5' (2.29 M)
60" (1500 mm)	7.5' (2.29 M)	8.5' (2.59 M)

* DOES NOT ACCOUNT FOR BYPASS PIPE, THIS ESTIMATE IS FOR THE MAIN UNIT ONLY

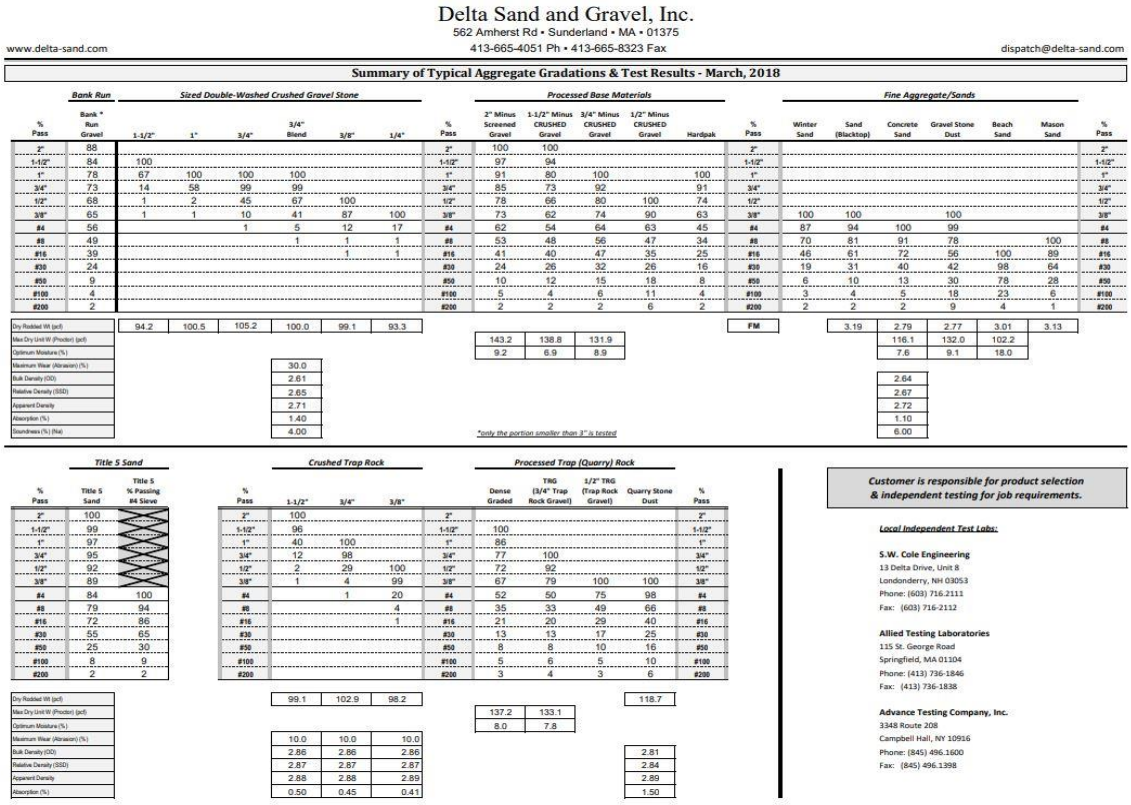


Appendix J

	Lot 1A - 25 Year Storm		Lot 1B - 25 Year Storm	
	Gravel-Bare	Concrete/Asphalt	Gravel-Bare	Concrete/Asphalt
Stormwater Runoff (gal/min)	4.5840	7.0523	2.6728	4.1120
Volume (gal per 60 min duration)	275.0378	423.1351	160.3682	246.7203

Gravel-Bare	C=.65
Concrete/Asphalt	C=1
25 Year Storm	I=.0335 in/min
Lot 1A	A=19,500 sqft
Lot 1B	A=11370sqft

Appendix K



Delta Sand and Gravel, Inc.
562 Amherst Rd • Sunderland • MA • 01375
www.delta-sand.com 413-665-4051 Ph • 413-665-8323 Fax dispatch@delta-sand.com

Summary of Typical Aggregate Gradations & Test Results - March, 2018

	Bank Run	Sized Double-Washed Crushed Gravel Stone							Processed Base Materials						Fine Aggregate/Sands						
% Pass	Bank Run Gravel	1-1/2"	1"	3/4"	3/4" Blend	3/8"	1/4"	% Pass	2" Minus Screened Gravel	1-1/2" Minus CRUSHED Gravel	3/4" Minus CRUSHED Gravel	1/2" Minus CRUSHED Gravel	Hardpak	% Pass	Winter Sand	Sand (Blacktop)	Concrete Sand	Gravel Stone Dust	Beach Sand	Mason Sand	% Pass
2"	88							2"	100	100				2"							2"
1-1/2"	84	100						1-1/2"	97	94				1-1/2"							1-1/2"
1"	78	67	100	100	100			1"	91	80	100		100	1"							1"
3/4"	73	14	58	99	99			3/4"	85	73	92		91	3/4"							3/4"
1/2"	68	1	2	45	67	100		1/2"	78	66	80	100	74	1/2"							1/2"
3/8"	65	1	1	10	41	87	100	3/8"	73	62	74	90	63	3/8"	100	100		100			3/8"
#4	56			1	5	12	17	#4	62	54	64	63	45	#4	87	94	100	99			#4
#8	49				1	1	1	#8	53	48	56	47	34	#8	70	81	91	78		100	#8
#16	39					1	1	#16	41	40	47	35	25	#16	46	61	72	56	100	89	#16
#30	24							#30	24	26	32	26	16	#30	19	31	40	42	98	64	#30
#50	9							#50	10	12	15	18	8	#50	6	10	13	30	78	28	#50
#100	4							#100	5	4	6	11	4	#100	3	4	5	18	23	6	#100
#200	2							#200	2	2	2	6	2	#200	2	2	2	9	4	1	#200
Dry Rodded Wt (pcf)		94.2	100.5	105.2	100.0	99.1	93.3							FM		3.19	2.79	2.77	3.01	3.13	
Max Dry Unit W (Proctor) (pcf)									143.2	138.8	131.9						116.1	132.0	102.2		
Optimum Moisture (%)									9.2	6.9	8.9						7.6	9.1	18.0		
Maximum Wear (Abrasion) (%)					30.0																
Bulk Density (SSD)					2.61												2.64				
Relative Density (SSD)					2.65												2.67				
Apparent Density					2.71												2.72				
Absorption (%)					1.40												1.10				
Soundness (%) (Na)					4.00												6.00				

*only the portion smaller than 3" is tested

	Title 5 Sand		Crushed Trap Rock				Processed Trap (Quarry) Rock				
% Pass	Title 5 Sand	Title 5 % Passing #4 Sieve	% Pass	1-1/2"	3/4"	3/8"	% Pass	Dense Graded	TRG (3/4" Trap Rock Gravel)	1/2" TRG (Trap Rock Gravel)	Quarry Stone Dust	% Pass
2"	100		2"	100			2"					2"
1-1/2"	99		1-1/2"	96			1-1/2"	100				1-1/2"
1"	97		1"	40	100		1"	86				1"
3/4"	95		3/4"	12	98		3/4"	77	100			3/4"
1/2"	92		1/2"	2	29	100	1/2"	72	92			1/2"
3/8"	89		3/8"	1	4	99	3/8"	67	79	100	100	3/8"
#4	84	100	#4		1	20	#4	52	50	75	98	#4
#8	79	94	#8			4	#8	35	33	49	66	#8
#16	72	86	#16			1	#16	21	20	29	40	#16
#30	55	65	#30				#30	13	13	17	25	#30
#50	25	30	#50				#50	8	8	10	16	#50
#100	8	9	#100				#100	5	6	5	10	#100
#200	2	2	#200				#200	3	4	3	6	#200
Dry Rodded Wt (pcf)				99.1	102.9	98.2					118.7	
Max Dry Unit W (Proctor) (pcf)								137.2	133.1			
Optimum Moisture (%)								8.0	7.8			
Maximum Wear (Abrasion) (%)				10.0	10.0	10.0						
Bulk Density (SSD)				2.86	2.86	2.86					2.81	
Relative Density (SSD)				2.87	2.87	2.87					2.84	
Apparent Density				2.88	2.88	2.89					2.89	
Absorption (%)				0.50	0.45	0.41					1.50	

Customer is responsible for product selection & independent testing for job requirements.

Local Independent Test Labs:

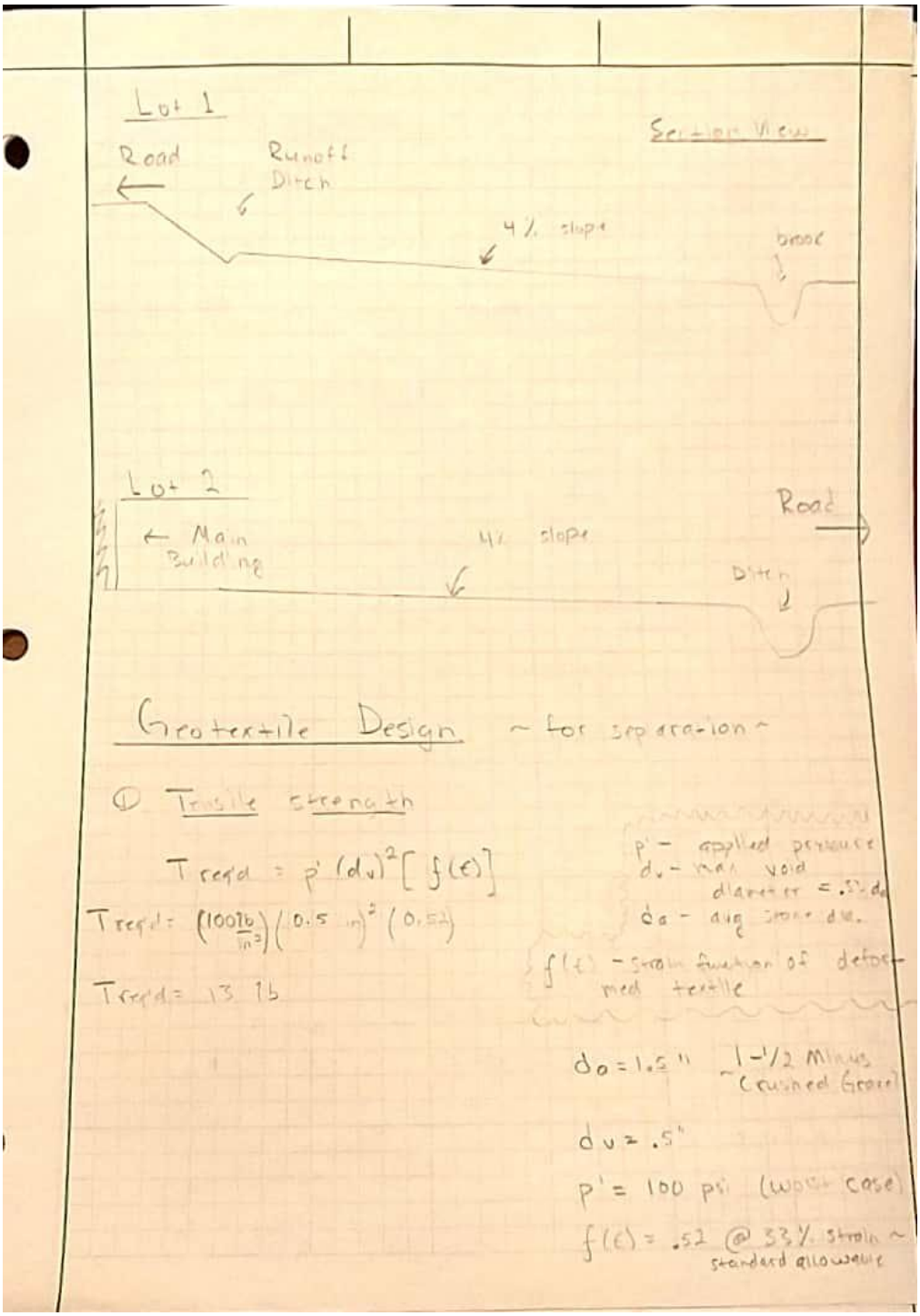
S.W. Cole Engineering
13 Delta Drive, Unit 8
Londonderry, NH 03053
Phone: (603) 716.2111
Fax: (603) 716-2112

Allied Testing Laboratories
115 St. George Road
Springfield, MA 01104
Phone: (413) 736-1846
Fax: (413) 736-1838

Advance Testing Company, Inc.
3348 Route 208
Campbell Hall, NY 10916
Phone: (845) 496.1600
Fax: (845) 496.1398

Delta offers many more materials than are shown on this summary sheet. Refer to the current price sheet.

Appendix L


Lot 1
Section View
Road
Runoff Ditch
4% slope
brook
Lot 2
Main Building
4% slope
Road
Ditch
Geotextile Design ~ for separation ~
① Tensile strength
$T_{reqd} = p'(d_v)^2[f(\epsilon)]$
p' - applied pressure
d_v - max void diameter = .5 d_o
d_o - avg stone dia.
f(ε) - strain function of deformed textile
$T_{reqd} = \left(100\frac{lb}{in^2}\right)(0.5\ in)^2(0.52)$
$T_{reqd} = 13$ lb
$d_o = 1.5$" ~ 1-1/2 Minus Crushed Gravel
$d_v = .5$"
p' = 100 psi (worst case)
f(ε) = .52 @ 52% strain ~ standard allowable

Burst Resistance

$$T_{req'd} = \frac{1}{2} p' d_v [f(\epsilon)]$$

$$T_{req'd} = \frac{1}{2}\left(\frac{100 lb}{in^2}\right)(0.5)(0.52)$$

$$T_{req'd} = 13 \frac{lb}{in^2}$$

$p' = 100$ psi
$d_v = 0.5$"
$f(\epsilon) = 0.52$

Puncture Resistance

$$F_{req'd} = p' d_a^2 S_1 S_2 S_3$$

$$F_{req'd} = \left(100 \frac{lb}{in^2}\right)(1.5in)^2(1)(.2067)(0.7)$$

$F_{req'd} = 32 lb$

$F_{req'd}$ - required vertical force resisted

d_a - average stone diameter

S_1 - protrusion factor: $\frac{h_k}{d_a}$ where $h_k \leq d_a$

S_2 - $0.31/d_a$

S_3 - shape factor

$d_a = 1.5$" $S_1 = 1$

$p' = 100 \frac{lb}{in^2}$ $S_2 = .2067$

$S_3 = 0.7$ - 1-1/2" minus Gravel

Impact (Tear) Resistance

$$E = \frac{\pi d_a^3}{6}(62.4)(2.60)(h)$$

$$E = 85(d_a)^3 h$$

$$E = 85\left(\frac{1.5}{12}\right)^3(4)$$

$$E = .664 \text{ ft-lb}_f$$

$d_a = 1 - 1/2"$

$h = 4' - 4\frac{1}{2}'$ ~ height when dumping tar off

E - energy released from falling object to be withstood

h - height of object fall

d_a - average particle diameter ~ max used to calculate max allowable

Geogrid Design

- for Reinforcement -

- $T_{reqd} = \sigma_z R \Omega$

- $\sigma_z = 2 \gamma_{ave} R$

σ_z = vertical stress

R = radius of differential settlement

$\Omega = 0.25\left[\frac{2y}{B} + \frac{B}{2y}\right]$

y = depth of settlement void

B = width of settlement void

γ_{ave} = unit wt. of soil

$\gamma = 134 \frac{lb}{ft^3}$

R = 1.5' (R of average pothole)

$\sigma_z = 2 \gamma_{ave} R$

$\sigma = 2\left(134 \frac{lb}{ft^3}\right)(1.5 ft)$

$\sigma_z = 402 \frac{lb}{ft^2}$

$T_{reqd} = \sigma_z R \Omega$

$= \left(402 \frac{lb}{ft^2}\right)(1.5 ft)(.625)$

$T_{reqd} = 376.875 \frac{lb}{ft}$

$\Omega = .25\left(\frac{2(2)}{3} + \frac{3}{2(.75)}\right)$

$= .625$

Geogrid Design

- Aperture Size -

$B_{req} \geq 3.5\ d_{50}$

$B_{req} = 3.5(0.0937)$

$\boxed{B_{req} \geq 0.2811}$

B_{req} - 7-1/2" Minus Crushed Gravel

d_{50} - # 8 sieve or 0.0937 in.

B_{req} - required aperture minimum width

- Factor of Safety -

$$FS = \frac{T_{allow}}{T_{reqd}}$$

$$T_{allow} = T_{ult}\left[\frac{1}{FS_{ID} \cdot FS_{cr} \cdot FS_{cr} \cdot FS_{BD}}\right]$$

$T_{allow} = 1310\ lb/ft\left(\frac{1}{(1.1)(1.5)(1.1)(1.1)}\right)$

$T_{allow} = 656\ lb/ft$

$$F.S = \frac{656\ lb/ft}{377\ lb/ft} = 1.7$$

$\boxed{F.S = 1.7}$

- FS_{ID} - F.S for installation damage
- FS_{cr} - F.S for creep over time
- FS_{cr} - F.S for chemical degradation
- FS_{BD} - F.S for Biological degradation

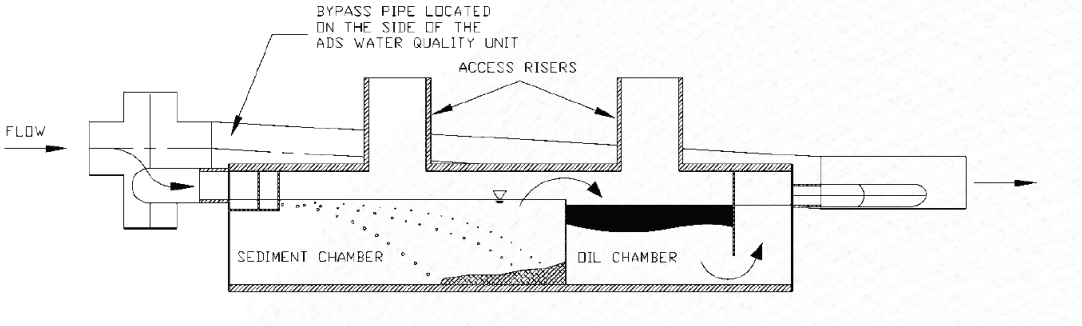
Appendix M

Lot 1		
	25 year storm	
C	0.65	
I	2.01	in/hr
A	1.15	acres
Q (ft^3/hr)	**1.50**	**ft^3/s**
Q (gal/min)	**673.09**	**gal/min**

Road to Lot		
	25 year storm	
C	0.18	
I	2.01	in/hr
A	0.34	acres
Q (ft^3/hr)	**0.12**	**ft^3/s**
Q (gal/min)	**55.92**	**gal/min**

Appendix N

ADS STORM WATER QUALITY UNIT



Appendix O

3-4 MINIMUM VELOCITY CONSIDERATIONS

Sediment can reduce the capacity of a stormwater pipe over time. In some installations, it may render the pipe useless until the system can be cleaned. This is an expensive, time-consuming undertaking so preventative measures should be taken during design. Sedimentation is of great concern in sewer applications since large, heavy grit may be present.

To minimize potential problems, flow should be maintained at a minimum, or self-cleansing, velocity. A commonly accepted self-cleansing velocity for storm and sanitary sewers is 3 fps (0.9 m/s). In each design, a final check should be performed to compare the expected velocity with the self-cleansing velocity. The design velocity for full-flowing pipes can be *approximated* with Equation 3-5:

$$V = \frac{Q}{A}$$ Equation 3-5

The potential for settling is determined by the specific gravity and diameter of particle, its cohesive properties, flow velocity, and the roughness of the pipe interior. For further discussion on the complexities and variables associated with determining the self-cleansing velocity for a specific pipe diameter and material, refer to ASCE publication No. 60, "Gravity Sanitary Sewer Design and Construction." In some specialized installations where sediment is a known problem it may be wise to perform a soil analysis prior to final drainage design.

3-5 MAXIMUM VELOCITY CONSIDERATIONS

High flow velocity can also create problems if not properly taken into consideration. High velocity is usually considered to be approximately 12 fps (3.7 m/s) but can vary depending on the specific site conditions.

The preferred method of contending with high velocity is to look for opportunities to minimize it, such as reducing the slope of the pipe. If that is not feasible, and many times it is not, the velocity must simply be managed the best way possible.

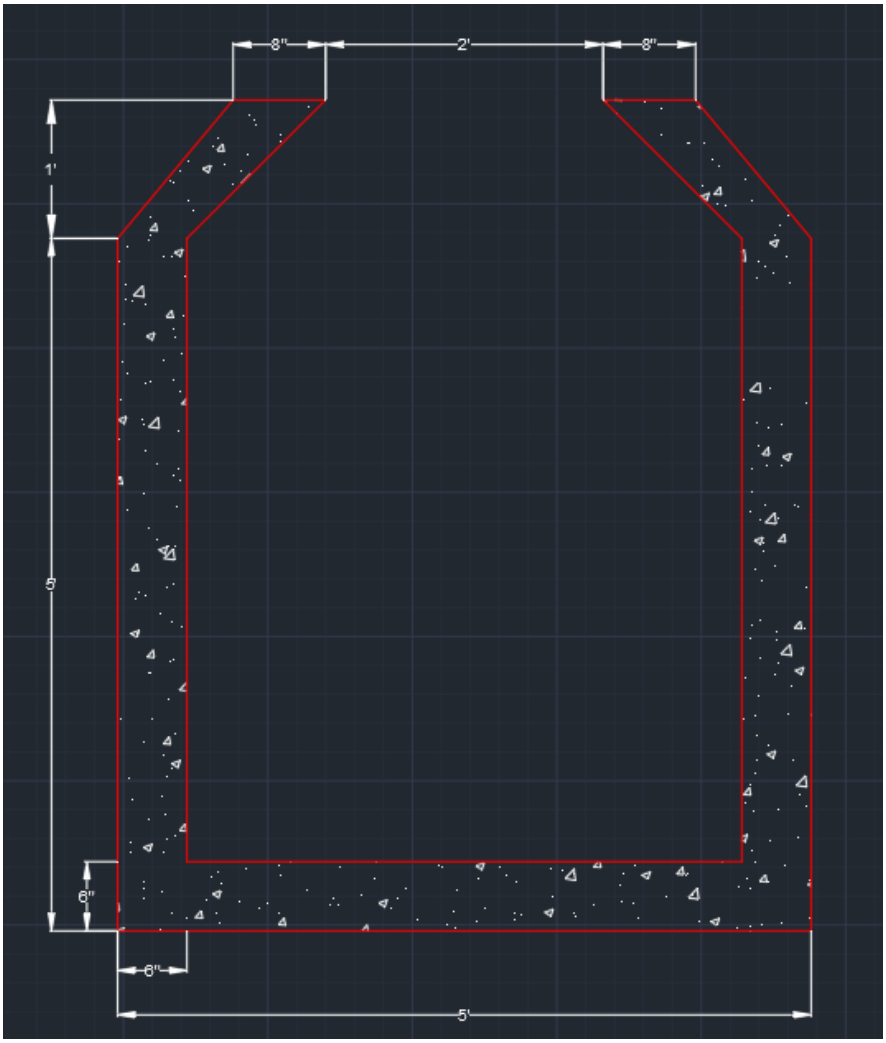
High velocity, especially if it carries an abrasive effluent, can present durability problems. Over time, the invert of the pipe can wear prematurely. Thermoplastics resist the effects of these rigorous conditions better than many other traditional pipe materials. Additional information specific to the effects of abrasives on many types of pipe materials is provided in the *Durability* section (Section 4) of the Drainage Handbook.

Special consideration should also be given to the conditions at the pipe outlet. High flow velocity can erode the channel where the flow is deposited. Erosion management methods, such as rip-rap, should be considered in these areas.

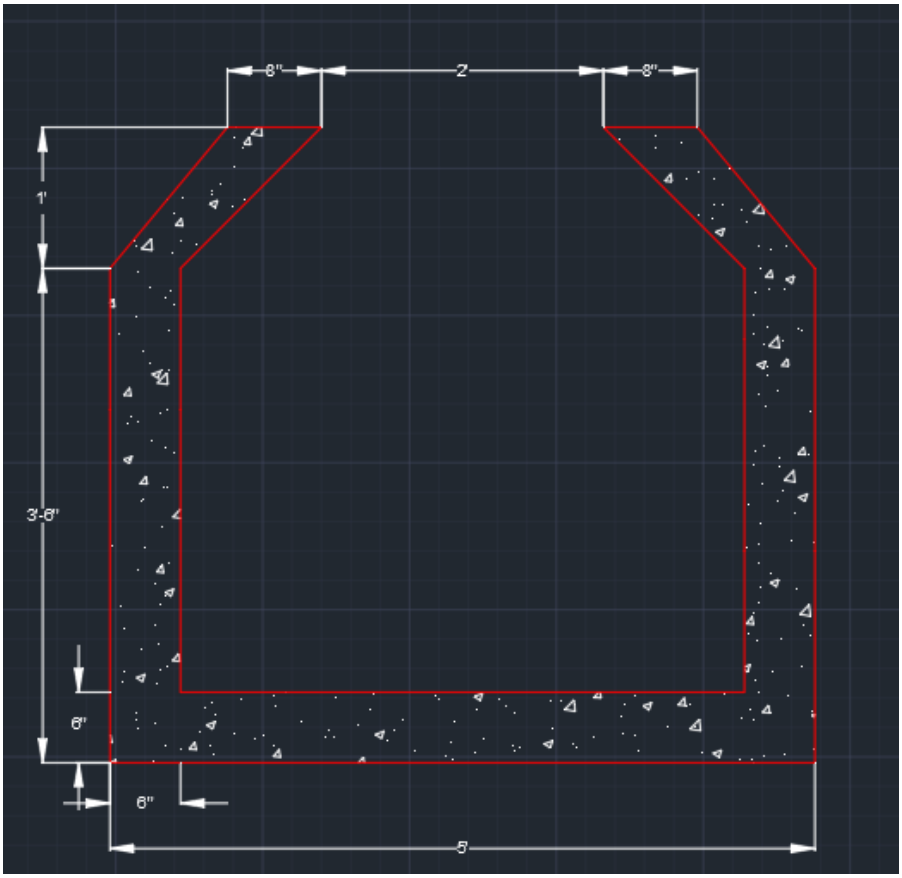
Another consideration in high velocity applications is managing the momentum of the flow. Changes in flow direction will result in large forces that can cause pipe movement, especially if the pipe size is large, velocity is very high, and the native soil has a low bearing strength. Concrete thrust blocks positioned at areas of flow direction change and sized specifically for the site conditions can be used to control the effects of momentum.

Anchoring systems may also need to be considered when the velocity is high or the slope that the pipe is installed is fairly steep. Anchors keep the pipe from moving down the slope while it is being installed and later due to the energy of the flow. They are an especially important consideration if the native soil is subject for movement or instability. ADS does not produce anchoring systems, but can provide additional information on companies who are experts in this area. For further discussion of steep slope applications, refer to “Steep Slope Installations” in the *Installation* section (Section 5) of the Drainage Handbook.

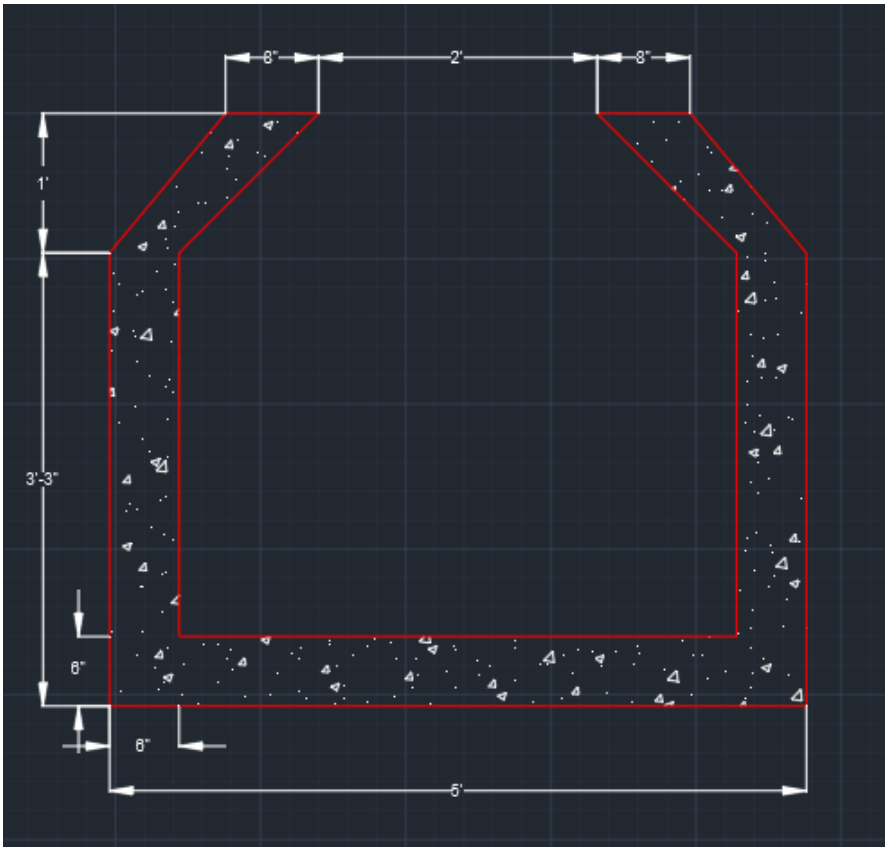
Appendix P



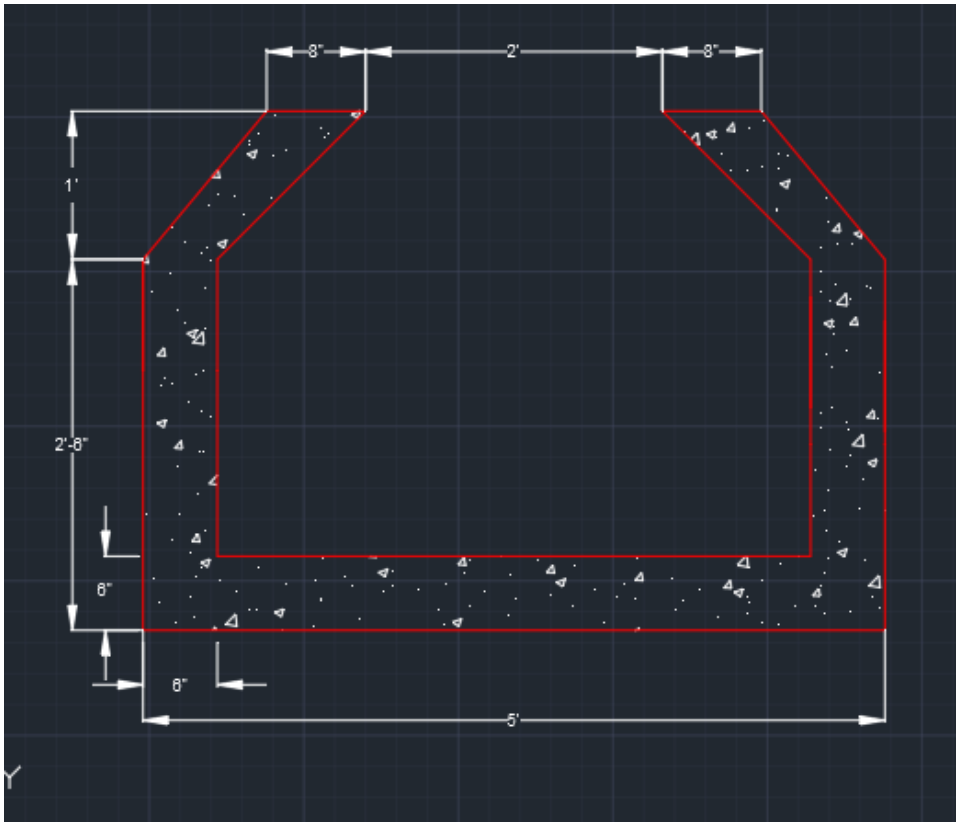
Appendix 2: Lot 1A Asphalt Surface Catch Basin Design



Appendix 3: Lot 1A Gravel Surface Catch Basin Design

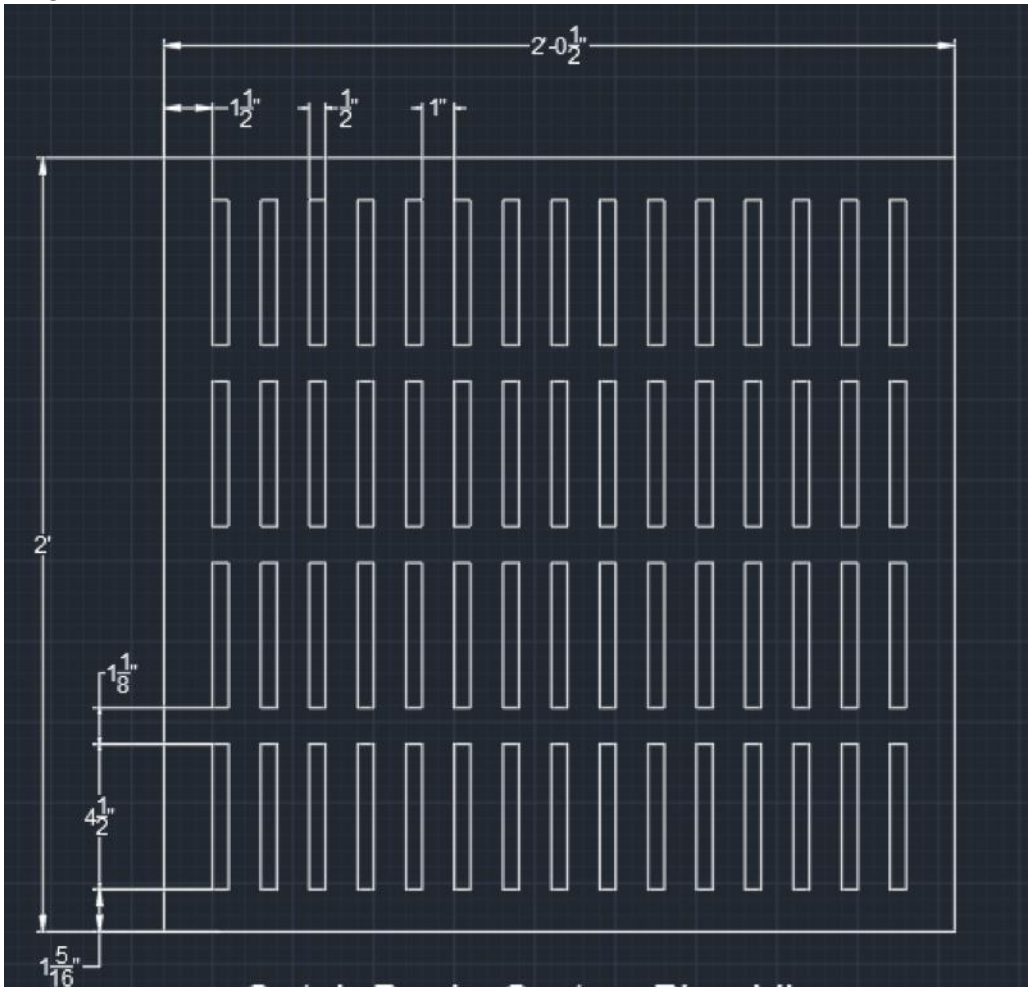


Appendix 4: Lot 1B Asphalt Surface Catch Basin Design

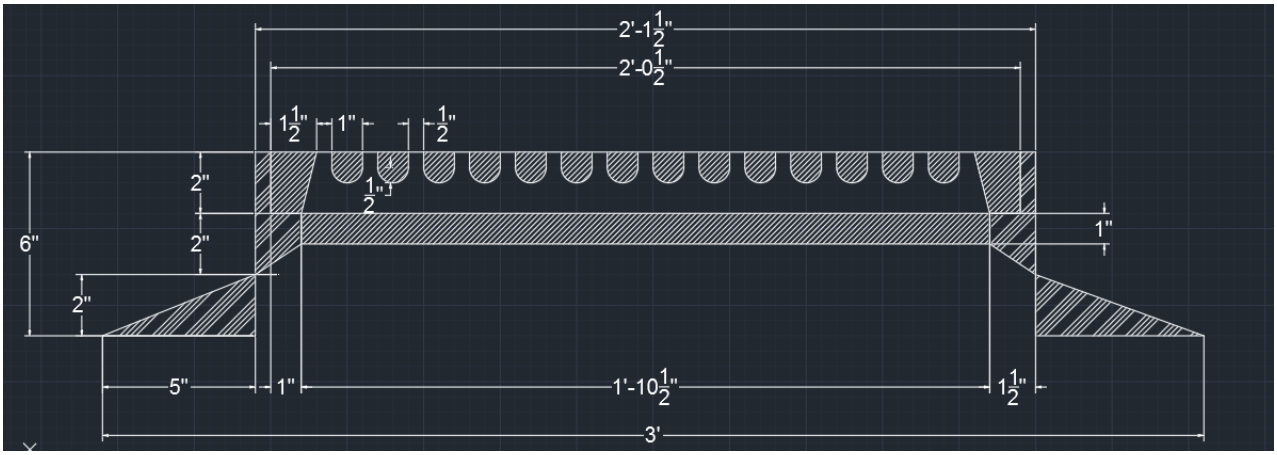


Appendix 5: Lot 1B Gravel Surface Catch Basin Design

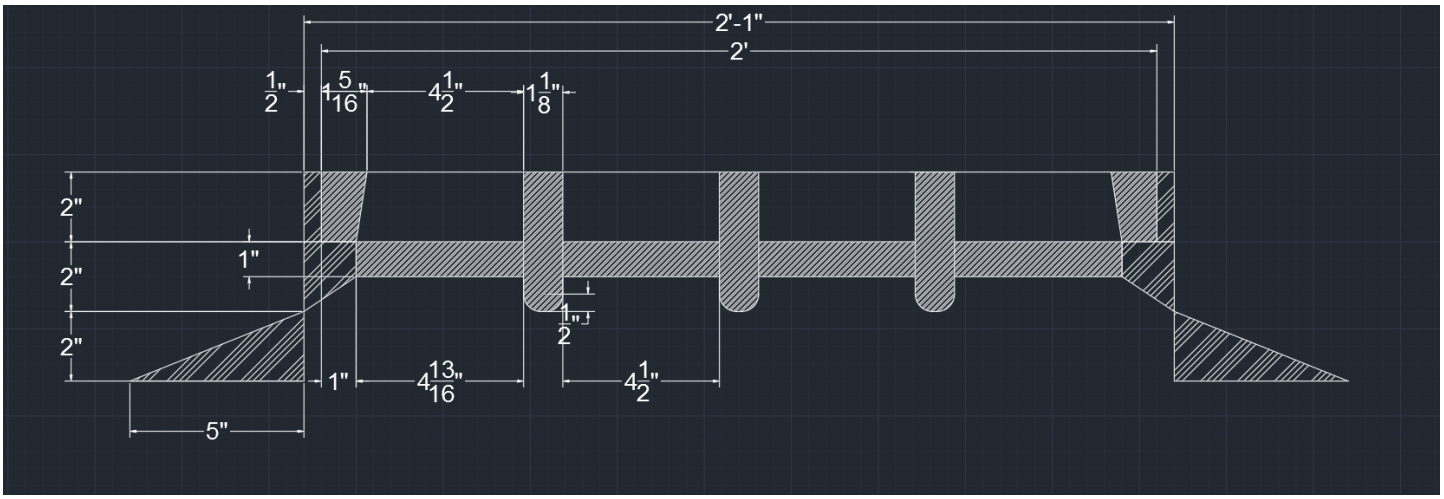
Appendix Q



Appendix 6: Grate Design – Plan View



Appendix 7: Grate Design – Section View 1



Appendix 8: Grate Design – Section View 2