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Executive Summary

MOHA Associates has been contracted by Aboumoussa & Kobakhidze LLC to develop a parcel of land at 198 Lake Street in northern White Plains, New York into a modern and complete middle school campus to serve the community of White Plains. The new Silver Lake Middle School will reduce crowding in an overburdened school district and provide an environment to middle school students conducive to learning. The proposed development consists of two buildings to be constructed on the parcel of land, including a main academic building for classrooms and offices, and a secondary gymnasium building for athletic purposes. Additionally, two recreational spaces are designed for outdoor student engagement: a community learning garden, and an outdoor grass seating area complete with picnic tables and trees for shade.

In this report, the design process and products are discussed for Silver Lake Middle School. First, from the initial site survey and general zoning information, we generate a concept plan that includes all of the required buildings and facilities, such as parking, in their proper locations. The concept plan is refined throughout the duration of the project, as changes are made to the scope and locations of features with respect to one another.

From this point, the site grading is designed, which determines the final ground surface of the development. The cut-to-fill ratio is determined from the existing and proposed elevations. Retaining walls are added to the site as needed in order to meet the existing ground surface and provide adequate swale channels for water drainage.

Then, the drainage structures, stormwater retention features, and stormwater pipes are designed and located on the site to handle the runoff that the proposed development is expected to generate. The retention pond and detention chambers are then designed to accommodate the expected volume of water generated in a 100-year storm on the site. EGL and HGL diagrams and calculations are carried out to determine the feasibility of the proposed system.

A utility plan considering both water supply and sanitary waste pipe systems is designed. Water supply analysis in EPANET is performed to ensure adequate peak demand and fire flows.

Next, the pavement and signage plan is developed, including typical cross-sections of the roadway and pedestrian sidewalk. An erosion and sediment control plan is also formulated to ensure that during sitework sediments are adequately dealt with and do not erode from the site or deposit onto the site.

Finally, the engineering estimate for the cost of the designed and proposed work is presented, along with the proposed scheduling of the project work to bring the project to fruition.
Introduction

About MOHA Associates:

MOHA Associates is a Civil Engineering firm founded in 2022 headquartered in Brooklyn, New York. As a relatively small company, MOHA Associates specializes in getting to know their clients’ needs in order to provide them with an optimal project design, schedule, and build.

MOHA Associates has expertise in a diverse array of disciplines and services that allow MOHA to deliver state of the art multi-purpose facilities, residences, and schools for New York area clients. MOHA prides itself on conscientiousness and thoroughness in design and robust yet sound engineering solutions.

Our Team:

The MOHA project team consists of lead Project Manager Shakya Amaratunga, along with project engineers Nicholas Hudanich, Joanna Mira-Villa, and Michael Osorio. The team met as undergraduate students at New York University’s Tandon School of Engineering where they each obtained a Bachelor of Science degree in Civil Engineering. MOHA Associates was created from a shared interest in improving the built environment and sound engineering, with the vision of pioneering long-lasting engineering projects for the benefit of New York. The hierarchy of the project team is represented in Figure I.1 below.

![Project Hierarchy Flowchart]

Figure I.1 Team Hierarchy Flowchart
Project Objective & Requirements:

The proposed Silver Lake Middle School (SLMS) is MOHA Associates’ latest endeavor located in the parcel of land at 198 Lake Street in the City of White Plains, New York. The objective of this project is to provide the surrounding communities with a state-of-the-art educational facility open to children between grades 6 through 8 to foster a modern and quality learning environment. The site of the proposed SLMS is currently undeveloped, and heavily wooded. The existing ground surface is steep with rocky outcrops in various locations. The site is heavily constrained by the adjacent parcels in access to Lake Street.

The proposed development follows the minimum requirements and zoning ordinances established by documents such as the White Plains Zoning Ordinances, AASHTO Green Book, and Americans with Disabilities Act, as well as other federal and state regulations pertaining to the site. In addition to these requirements, the client has imposed additional requirements to be met by the design:

- Two buildings are proposed, including a 10,820 square foot main educational building, and a 6,000 square foot ancillary gymnasium.

- Two recreational facilities are proposed, including a horticultural community garden area of 3,200 square feet, and an outdoor seating and picnic area of 3,850 square feet. These two recreational facilities are maintained at a maximum slope of 2% for walkability. The maximum slope of any landscaped area on the site does not exceed 25 percent (with a minimum of 1 percent to allow appropriate drainage).

- A setback requirement for structures on this parcel of land of 25 feet is maintained from all adjacent properties. Similarly, the retention pond is designed with a stormwater buffer area of 40 feet from the high-water edge of the pond. The pond basin is designed to a maximum slope of 33%.

- Rock outcroppings and other exposed rock are avoided and left outside of the construction footprint to minimize cost in sitework and excavation.

- A total of 64 vehicle parking spaces are provided within the property. 6 accessible spaces and 2 van accessible spaces are provided, as per ADA regulations. Parking spaces have standard dimensions of 8 ½ feet x 18 ½ feet.

- The farthest parking spaces from the building are located less than 200 feet of walking distance from the entrance to the main building.

- The minimum turning radius on parking lot access roads is 50 feet, and all roadways accommodate access for trucks, and all emergency vehicles, regardless of direction entering the site.
1.0 Concept Plan

Silver Lake Middle School intends to bring back specialized programs into public schools within the area. With extreme budget cuts in recent years within New York's public school system, a lot of learning environments are suffering from the loss of not being able to provide students with any form of arts, tech or music education. MOHA Associates is inspired to take on this project that hopes to provide early exposure to students to explore different talent based opportunities within music, visual arts and STEM.

1.1 Original Concept Plan

For the initial concept design of this project, the proposed development included two buildings to be constructed on the parcel of land, including a main academic building (of 10,823 sq-ft) for various sized classrooms and offices and a secondary building (of 6,000 sq-ft) including a small gymnasium for athletic purposes. Upon preliminary investigation into the site, MOHA had originally proposed the idea of a high school to be built on site, however, some limitations were detected that resulted in the changing of scope of the project from a high school to a middle school. Namely, high school students are old enough to drive, and thus they typically require much more parking than middle and elementary school students. Furthermore, a middle school does not typically have the same athletic facility requirements that a high school usually has, and thus the project proposal was adopted into the Silver Lake Middle School proposal.

Additionally, the outdoor recreational area was originally proposed to consist of an amphitheater that would allow for outdoor learning activities and theater-based arts education. This stage would have also had the benefit of possible use for performances and student activities. After further research into trying to adapt to the slope of the existing conditions on the site, the steep slope of the existing terrain was determined to be infeasible for constructing the amphitheater and would have required an inordinate amount of retaining walls to be constructed around the amphitheater.

The design of the pavement and parking areas in the original concept plan originally included provisions for a bus lane to the side of the parking lot, as well as much more space for a truck loading bay. However, after careful consideration and assessing the anticipated vehicular needs of the site, it was determined that for the size of the school there would be a sufficiently low volume of buses to not require a separate bus loading bay. Furthermore, the loading dock location on the building was changed to accommodate trucks more easily. Finally, in the initial concept plan, there was a very long set of crosswalks across the main entrance road at the existing street sidewalks. To mitigate this distance, a pedestrian refuge island was added to the final concept plan to make the site more walkable.
1.2 Final Concept Plan

The proposed development in the finalized concept plan for this project consists of two buildings to be constructed on the parcel of land, as well as two recreational sites, a retaining pond, and a parking lot with access roads to sufficiently accommodate anticipated demand. Figure 1.2-1 below shows an overview of the complete final concept plan.

![Figure 1.2-1. Complete Final Concept Plan](image)

One of the buildings, the main academic building, consists of a 10,823 square feet footprint, with two floors dedicated to various sized classrooms and administrative offices. In this facility, students and faculty are provided with proper space to learn and work together in a favorable and modern education environment. The other building includes a small indoor gymnasium of 6,000 square feet, including extended bathrooms with some shower facilities. This building exists for athletic and physical education purposes for the students of Silver Lake Middle School. The project buildings will conform to the White Plains and State of New York building codes. The conceptual design of the main building is detailed below in Figure 1.2-2, and the conceptual design of the secondary gymnasium building is detailed below in Figure 1.2-3.
Figure 1.2-2. Complete Final Concept Plan Main Academic Building

Figure 1.2-3. Complete Final Concept Plan Secondary Gymnasium Building
Furthermore, a total of two recreational sites are included in the project proposal. A 3,200 square feet horticultural garden area is proposed adjacent to the gymnasium. This space is dedicated for educational gardening purposes and learning about the natural environment. Resilient crop species for education such as potatoes, carrots, and Swiss chard are proposed to be some of the crops included in this horticultural facility. Additionally, a 3,850 square foot outdoor picnic area is proposed for the Northeastern end of the site development. This space, for the enjoyment of the school community, includes outdoor furniture such as wooden tables, planted hemlock and birch trees, and waste receptacles. The horticultural area and the picnic area are shown in detail in Figures 1.2-4 and 1.2-5 below, respectively.

Figure 1.2-4. Complete Final Concept Plan Horticultural Area

Figure 1.2-5. Complete Final Concept Plan Picnic Recreation Area
In the final concept plan, a parking area with a total of 64 parking spaces is proposed, which meets the minimum requirements set forth by the City of Whites Plains Zoning Ordinances. 8 of these spaces are ADA accessible spaces, including 2 ADA van accessible spaces. All ADA spaces are at least minimum standard to conform with the Americans with Disabilities Act standards and specifications. Figure 1.2-6 below shows the complete proposed parking area for the site.

Figure 1.2-6. Complete Final Concept Plan Parking Area

1.3 Geometric Roadway Design & Vehicle Tracking

For the entrance route into Silver Lake Middle School, several vehicular turning movements are simulated to appropriately design for the parking lot and access roads. Simulations in consideration for access into the parking lot include cars, emergency vehicles, delivery trucks, and standard NYS Fire trucks. The turning movement simulations provide a reliable analysis on the amount of space needed to create a safe and reliable entryway for any type of vehicle that may need access to the site.

The simulated turning movements are performed on AutoCAD and are shown below in Figures 1.3-1, 1.3-2, and 1.3-3 for a pump fire truck, aerial fire truck, and regular freight truck, respectively.
Figure 1.3-1. Pump Fire Truck Simulated Turning Movement

Figure 1.3-2. Aerial Fire Truck Simulated Turning Movement
Figure 1.3-3. SU-30 Truck Simulated Turning Movement
2.0 Grading

In this section, the proposed site grading is discussed, including the existing site survey, the proposed demolition and site grading plans, and the means and methods employed by MOHA to accomplish this design. Furthermore, the layout and design of the proposed on-site retaining walls is discussed. Finally, the total volume of cut and fill required for the proposed site grading is estimated.

2.1 Site Survey

The initial site survey for project development is provided to MOHA Associates by a surveying team. The survey includes the metes and bounds for the entire property, dictates which easements are available for this project, and contains critical topographical information on the existing site grading and elevations, the location of elements such as swales, rock, and dense brush, and which areas of the site have a shallow or steep slope, which inform the design on the location of various features. The initial site survey also contains existing structures, roadways, drainage facilities, sewers, and utility infrastructure that is considered when designing the site.

Specifically, the existing grade lines on the site survey are paramount for deciding the design of the middle school, and determining the locations of features like the parking lot, main buildings, and recreational areas depends directly on the existing topography. Parking lots and adjacent sidewalks have slightly steeper allowable slopes (up to 6 and 5 percent respectively) and thus are generally located in moderately sloped areas. Recreational areas, on the other hand, must be incredibly flat for pedestrian mobility, and as such these features are generally situated on areas of the site that are naturally flat. This school of thought also helps to minimize the total cut or fill that may be required to grade the site.

Extruding rocks are also apparent on the site survey. The locations of these rocks are an important consideration in the layout of the middle school design as the rocks must be avoided if necessary. Cutting rock is much more expensive than excavating standard soils, and as such, to minimize the cost of grading to the firm, all of the site development is placed such that it does not require the excavation of known rocks extruding from the surface.

The property and easement boundaries are equally important components of the initial site survey. These metes and bounds inform MOHA which areas cannot be developed and provide a reference from which the setbacks required by the zoning ordinances can be established. Easements dictate what land was available for developing the entrance from the road to the middle school and inform the design of the site roadways.
2.2 Demolition Plan

The first plan to prepare for the development of Silver Lake Middle School is the demolition plan. Due to the presence of existing structures on the site in locations that conflict with proposed structures, these features are first designated as to be removed before beginning site work. These features include pre-existing retaining walls, man-made fountain components, and a road surface from a former alignment of the main road. Figure 2.2-1 below shows an example of a callout from the demolition plan for an existing frame to be excavated and removed.

![Figure 2.2-1. Sample Callout from Demolition Plan for Removal of Existing Frame](image)

Thus, the demolition plan is prepared which indicates some detailed instructions as to what should be removed, such as “in entirety” or “including subbase”. Beyond this, it is to the general contractor’s discretion which means, and methods are employed to remove the features to the extent which is indicated by the demolition plan.

2.3 Grading Plan, Strategies, & Theoretical Backup

The primary process that informs the design and layout of the proposed project is site grading. There are several regulations in both the City of White Plains zoning ordinances, the
New York State Department of Environmental Protection, and the Americans with Disabilities Act that limit the slope of various components of the site, which all are considered when designing the proposed graded surface.

Specifically, there are four general space usages (vehicular, pedestrian walking spaces, pedestrian open spaces, and landscaped surfaces) on the proposed site, each with its own governing limits on the slope of each respective area.

For paved spaces, which include the site access roads, loading bay area, trash receptacle area, and parking lot, the maximum allowed slope in the direction of travel is 8 percent, with a maximum allowed cross-slope of 2 percent. Both the forward slope and cross slope have a required minimum of 1 percent to allow for proper drainage. Short sections of the roadway may travel at 15 percent slope, but these areas are classified as ramps and have their own provisions, and as such are avoided in the development of SLMS.

The slope of the parking lot is primarily constricted by the adjacent sidewalk, as pedestrian walking paths have a required maximum of 6% slope to comply with the Americans with Disabilities Act. In addition to walking paths, other pedestrian spaces are constrained to have a 2 percent maximum slope resultant for the same reasons. Both facilities still have a minimum required slope of 1 percent to allow proper drainage of the site.

Finally, any landscaped surface on the site is required to have a maximum of 25 percent slope after grading is complete. Existing slopes greater than 25 percent are allowed to be retained on the site if that area is not being developed or landscaped. This requirement applies to all grass, mulched, or other natural surfaces except for the retaining pond, which had an interior maximum slope of 33 percent. Generally, for any proposed grading scheme, the slope is checked by applying the standard slope formula, which is shown in equation 2.3.1 below, where $S$ is the slope of a linear section, $\Delta Y$ is the change in vertical elevation along that section, and $\Delta X$ is the total horizontal length of the section.

$$S = \frac{\Delta Y}{\Delta X} \quad \text{Equation 2.3.1}$$

Thus, in order to maintain a maximum of 8% slope for the roadway, for example, contour lines must always be 12.5 feet away from each other at any point on a proposed contour to any other point on any other proposed contour. Figure 2.3.1 below shows the complete graded site, where the thick and dark contours are the proposed contours, and the dashed light contours are the existing contours. Figure 2.3.2 below shows a sample close-up section of the middle of the parking lot, where the slope vector components of the proposed surface are constant.
Figure 2.3-1. Overview of Proposed Site Grading Scheme

Figure 2.3-2. Typical Parking Lot Grading Design

It is worth noting that due to the steep slopes of the natural site to the Northeast, as found by the site survey, increases in elevation and conforming to the maximum allowable slope are followed as close to Lake Street as possible to reduce the total volume of cut that is generated from the proposed grading scheme, and to minimize the usage of costly tall retaining walls on the project site.
2.4 Retaining Walls

An important feature of the proposed site with respect to grading and hydrology is the presence of retaining walls. Retaining walls are proposed by MOHA Associates in certain locations to minimize the burden on the site when grading to the required extents, and to reduce the total overall borrow that will need to be excavated from the site.

In total, there are 7 retaining walls that MOHA Associates proposes for the site, all of which are stone masonry walls at a maximum height of 3.5 feet, and all of the walls have a width of 1 foot. Figure 2.4-1 below shows an example of one of the proposed retaining walls from the project plans, located on the East side of the main academic building. Notice the callouts for the elevations of the top and bottom of the wall at each section, and the material callout for the wall.

![Figure 2.4-1. Sample Typical Stone Masonry Retaining Wall from Project Plan Documents](image)

3 of the 7 proposed retaining walls surround the community garden space due to a substantial difference between the existing slope of the ground surface and the slope of the proposed community garden space (2%), to accommodate people walking and conforming with ADA standards for that kind of space. These retaining walls vary from being 0.5 feet in height to 3.5 feet in height. It is important to note that the area outside the garden changes from being below the garden to above the garden, and thus there is an inflection point, at which the wall remains half a foot tall to serve as a barrier between the two spaces and assist in the channeling of water around the facility to a dry well, rather than into the area drain located in the community garden.

The fourth retaining wall is located between the main academic building and the ancillary gymnasium facility. It serves the purpose of ensuring that the proposed slopes of the
site can remain under a 1 to 4 ratio and ensures that stormwater from above the buildings on the site is directed away from the parking and walking facilities.

The remaining 3 retaining walls are located on the East side of the building and provide substantial increases in the site elevation without creating an excessively steep ground slope. The walls also allow rainwater to be channeled into swales for collection in dry wells.

2.5 Cut and Fill

To assess the proposed site grading scheme and calculate the total amount of earthwork needed to be performed, a cut/fill analysis is performed on the project site. First a grid is superimposed on the project plan, consisting of 50 foot by 50 foot square units. Regions near to the buildings or retaining walls are subsequently subdivided into four 25 foot by 25 foot units. A higher resolution in the estimate is obtained by decreasing the unit size in this way. In order to keep track of each section in a meaningful and coordinated way, each axis is given either a letter or number reference, with letters along the top of the project plan grid, and letters along the side. These references were placed on the 50 by 50 lines, and not as referring to the entire unit, as when units are subdivided, they can be referenced much easier.

The used naming convention for a cell is the intersection of lines in the top left corner of that cell. For example, cell A - 4 has the intersection of the A and 4 reference lines in its top left. For cells that are subdivided in four, decimal letters and numbers are used, such as G.5 - 3, which indicates the cell whose top left corner is the intersection of the G.5 and 3 reference lines, where the G.5 reference line is the subdivision between the G and H reference lines.

The final step to setting up the reference grid for calculations is to label both the existing and proposed elevations at each of the relevant reference line intersections. Figure 2.5.1 below shows the resulting reference line grid which was used to calculate cut and fill.

![Figure 2.5-1. Cut and Fill Reference Line Grid](image-url)
From this point, the means used to calculate the total change in volume of each cell utilize geometric averages of the four corners of the reference cell. Assuming an average and uniform distribution of the cut and fill across a cell is a close approximation for the total cut and fill of that cell. See Appendix A for detailed sample calculations with respect to this method, as well as the entire cut and fill quantities for each site subdivision.

Ultimately, the total cut generated from the site is calculated to be 8,023.99 cubic yards and the total fill on the site is calculated to be 2,068.60 cubic yards. However, before the cut-to-fill ratio can be calculated, two more factors are first considered, swell and shrinkage. The swell factor used on this project is 12 percent. The shrinkage factor used on this project is 8 percent.

Thus, the total volume of soil cut from the site considering swell is 8,976.87 cubic yards, and the total volume of soil filled onto the site considering shrinkage is 2,234.09 cubic yards.

Finally, the cut-to-fill ratio is calculated by the fraction of these two values, which is 4.02.
3.0 Stormwater Drainage

In this section, the design process for the proposed stormwater drainage system, its layout and strategies utilized to determine it, and theoretical backup alongside its calculations are discussed.

3.1 Layout and Strategies

The primary guiding principle for the design of the stormwater drainage system is to ensure that the runoff water on the proposed site did not exceed that of the pre-development site. Figure 3.1-1 below shows the overview of the proposed stormwater drainage plan, which is discussed further in this section.

![Figure 3.1-1. Stormwater Drainage Plan Overview](image)

Properties that are essential to consider when designing the stormwater system include the elevations, materials, slopes, and locations of each of the pipes and drainage structures in the system. It is paramount that the last outfall pipe matches the desired invert into the pond, falling slightly below the permanent pool level, and as such the design of the system is best carried out when starting at the lower end of the pond and working upwards to the required facilities. Keeping the pipes lower down with respect to the ground surface is shown to be beneficial in this method, as it allows for the easier placement of utility and sanitary utility pipes in future plans.

Catch basins are provided in areas to standardize the approximate size of watershed areas and convey a uniform amount of water into different parts of the drainage system, without creating concentrated channel flow conditions with excessive overland flow distances. Catch basins are designed to have a bottom at least a foot below the lowest out invert in order to serve as a sediment control method. HDPE pipes are utilized for the stormwater drainage system, as HDPE is corrosion resistant, more cost effective compared to other pipes, and more environmentally sustainable.
3.2 Theoretical Backup and Results

The first phase in the design process for the stormwater drainage system is to determine the watershed areas on the site by following the natural flow of water going perpendicular to the proposed grade lines and stopping at any sharp increase in elevation such as curbs or structures. From tracing these flow lines, the best locations of catch basins to prevent concentrated flow from long overland flow are determined. Watershed areas, materials, and roughness coefficients are then found cataloged, as they will be utilized in the rational method.

Next, using publicly available NOAA data, an IDF curve is developed custom for the site of the proposed project. Using this IDF curve and the time of concentration of overland flow from the most remote part of any given watershed, the rational method is used to determine the peak flow experienced by any drainage structure at its time of concentration (Discussed further below).

An iterative process for the design of each watershed area is performed until each of the watershed areas is small enough that their corresponding drainage facilities would be able to handle a design 100 year storm as per NYS DEP standards. The finalized watershed areas are shown in Figure 3.2-1 below.

In this figure, the watershed areas are color coded for easy identification; repeat colors do not represent any correlation between these watershed areas. Watershed areas are all continuous without breaks or islands.

Calculations are performed for the times of concentration, intensity, and design flow rates to ensure that selected pipe sizes and locations are to code. The tables containing all these calculated values for each stormwater drainage pipe are found in Appendix B.
First the flowrate of water that would be collected by each catch basin or water management system for each watershed is found. To do this, the areas of each watershed, both paved and unpaved, are calculated alongside the paved and unpaved lengths from the most remote areas to the inlet. The times of concentration for each watershed are calculated using the following equation from TR-55. This equation is chosen as all the areas were assumed to have sheet flow since the flow lengths never exceed 300 feet.

\[
T_t = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}S^{4}} \quad \text{Equation 3.2.1}
\]

Using the time of concentration, the intensity of the rainfall is then determined using Figure B.1 based on the rainfall information from the NOAA-generated IDF curve for the project site. Given that rainfall under five minutes may be harder to predict, the IDF curve begins at five minutes, and so any time of concentration less than that is defaulted to a minimum of five minutes. Once the intensity is obtained, the flowrate is calculated using the rational method (equation 3.2.2).

\[
Q = CiA \quad \text{Equation 3.2.2}
\]

The values for roughness coefficient C are obtained from Table B.4 for a design storm with a return period of 100 years. This is performed both for paved and unpaved areas with their respective runoff coefficients and then summed together to give the total peak runoff.

The pipes are then designed by first determining the slope and pipe length through the invert in and out elevations and then assuming a diameter for the pipe. This diameter is then used to calculate full pipe flow area and hydraulic radius. Manning’s equation (equation 3.2.3) is then used with these values to determine the design flow rate of the full pipe based on the selected diameter.

\[
Q = \frac{1.49}{n} AR^{2/3} \sqrt{S} \quad \text{Equation 3.2.3}
\]

Using the flow rates of each contributing component to any given pipe, the cumulative flow rate over the full pipe flow rate ratio is calculated. This ratio is then used in conjunction with Figure B.3 to determine the design velocity over full pipe velocity ratio. In addition to that, the \(y/D\) ratio is checked to ensure that the height of the water reaches at least half the diameter of the pipe, so the method parameters apply. The calculated full velocity is checked to verify that it reaches at least 2.5 ft/s to ensure scour velocity (scour is 1.0 ft/s) and less than 10 ft/s so that the water flowing doesn’t damage the pipes themselves or cause any need for tying the pipes down. If the pipe velocity exceeds 10 ft/s, the diameter of the pipe or the invert elevations, and therefore the slope, are adjusted. With the pipe velocity, the travel time of
water in the pipe is determined by dividing the velocity from the pipe length. This process is repeated for each pipe in the system until every pipe meets the requirements.

The finalized specifications for the pipes, containing their invert in and invert out elevations, slope, pipe length, diameter, velocity, and travel time of water are shown in *Table 3.2-1* below. For full pipe information, refer to the project plans submitted or Appendix B.

Table 3.2-1. Designed Pipes Characteristics

<table>
<thead>
<tr>
<th>Pipe</th>
<th>Slope of Pipe (ft/ft)</th>
<th>Pipe Length (ft)</th>
<th>Design Pipe Diameter (in)</th>
<th>Design Flow Rate (ft^3/s)</th>
<th>Design Velocity (ft/s)</th>
<th>Pipe Travel Time (min)</th>
<th>Time of Concentration (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Roof - CB-01</td>
<td>3.00%</td>
<td>33.43</td>
<td>8.00</td>
<td>2.51</td>
<td>7.97</td>
<td>0.07</td>
<td>5.07</td>
</tr>
<tr>
<td>CB-01 - CB-02</td>
<td>3.00%</td>
<td>54.77</td>
<td>10.00</td>
<td>3.85</td>
<td>8.88</td>
<td>0.10</td>
<td>5.17</td>
</tr>
<tr>
<td>CB-02 - CB-04</td>
<td>2.50%</td>
<td>19.68</td>
<td>12.00</td>
<td>5.31</td>
<td>8.88</td>
<td>0.04</td>
<td>5.21</td>
</tr>
<tr>
<td>CB-03 - CB-04</td>
<td>2.00%</td>
<td>85.85</td>
<td>8.00</td>
<td>0.59</td>
<td>4.54</td>
<td>0.32</td>
<td>5.00</td>
</tr>
<tr>
<td>CB-04 - CB-08</td>
<td>1.50%</td>
<td>113.93</td>
<td>14.00</td>
<td>6.80</td>
<td>7.87</td>
<td>0.24</td>
<td>5.45</td>
</tr>
<tr>
<td>CB-07 - CB-08</td>
<td>2.00%</td>
<td>42.93</td>
<td>8.00</td>
<td>0.76</td>
<td>4.85</td>
<td>0.15</td>
<td>5.00</td>
</tr>
<tr>
<td>Gym Roof - CB-05</td>
<td>3.00%</td>
<td>38.56</td>
<td>8.00</td>
<td>1.36</td>
<td>6.64</td>
<td>0.10</td>
<td>5.10</td>
</tr>
<tr>
<td>CB-05 - CB-06</td>
<td>2.00%</td>
<td>35.22</td>
<td>10.00</td>
<td>1.95</td>
<td>6.22</td>
<td>0.09</td>
<td>5.19</td>
</tr>
<tr>
<td>CB-06 - CB-08</td>
<td>2.00%</td>
<td>19.96</td>
<td>10.00</td>
<td>2.83</td>
<td>6.97</td>
<td>0.05</td>
<td>5.24</td>
</tr>
<tr>
<td>CB-08 - CB-09</td>
<td>1.00%</td>
<td>114.40</td>
<td>20.00</td>
<td>11.59</td>
<td>7.47</td>
<td>0.26</td>
<td>5.71</td>
</tr>
<tr>
<td>CB-09 - CB-10</td>
<td>1.25%</td>
<td>45.78</td>
<td>24.00</td>
<td>12.53</td>
<td>8.17</td>
<td>0.09</td>
<td>5.80</td>
</tr>
<tr>
<td>CB-10 - MH-01</td>
<td>1.25%</td>
<td>17.13</td>
<td>24.00</td>
<td>13.69</td>
<td>8.50</td>
<td>0.03</td>
<td>5.83</td>
</tr>
<tr>
<td>AD-01 - MH-01</td>
<td>3.00%</td>
<td>11.26</td>
<td>8.00</td>
<td>0.40</td>
<td>4.54</td>
<td>0.04</td>
<td>5.00</td>
</tr>
<tr>
<td>MH-01 - Pond</td>
<td>1.00%</td>
<td>74.69</td>
<td>24.00</td>
<td>14.09</td>
<td>7.90</td>
<td>0.16</td>
<td>5.99</td>
</tr>
<tr>
<td>TD-01 - DW-06</td>
<td>2.00%</td>
<td>17.84</td>
<td>8.00</td>
<td>1.48</td>
<td>5.87</td>
<td>0.05</td>
<td>5.00</td>
</tr>
</tbody>
</table>
3.3 Pipe EGL & HGL Profiles

To determine the efficiency of the pipe system and to see if it overflows, EGL and HGL values are calculated according to the procedure in the Land Development Handbook Guide, shown in Figure 3.3-1 below.

Several junction head losses such as entrance, exit, expansion, contraction and bend head loss are calculated using equation 3.3.1 below where $k$ is the loss coefficient.

$$H = k \frac{v^2}{2g} \quad \text{Equation 3.3.1}$$

Calculations start where the water elevation is known which is the invert out of the pipe connecting to the pond. When calculating the EGL values, the calculations show that both EGL and HGL lines are below the rim elevation which ensures that the system will not overflow. If it did rise above the rim elevation, surcharge occurs and the system operates as pressure flow instead of gravity flow. Figure 3.3-1 below shows a sample generated EGL/HGL profile for one of the stormwater pipes from the gym to catch basin 8.

Table 3.3-1: EGL and HGL Calculation Table

<table>
<thead>
<tr>
<th>No</th>
<th>EGL out</th>
<th>HGL</th>
<th>D</th>
<th>S</th>
<th>P</th>
<th>H</th>
<th>H</th>
<th>H</th>
<th>H</th>
<th>0.5</th>
<th>US</th>
<th>US</th>
<th>Rim</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td>22</td>
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</tr>
</tbody>
</table>

Column 1: Structure identification label
Column 2: EGL elevation at downstream side of structure = preceding column 10 + preceding column 15
Column 3: Velocity head at the discharge side of the structure = $V^2/2g$
Column 4: HGL elevation = column 2 + column 3
Column 5: Pipe diameter
Column 6: Discharge
Column 7: Discharge velocity
Column 8: Pipe length between structures
Column 9: Friction slope = $V^2/(2g)$
Column 10: Friction head loss = column 8 X column 9 (fig. 14.20-f)
Column 11: Entrainment head loss at terminal structure on US end of run (fig. 14.20-a)
Column 12: Entrance head loss at terminal structure on US end of run (fig. 14.20-b)
Column 13: Contractual head loss for overflow from a nonterminal structure = $k_d(V^2/2g)$
Column 14: Expansion head loss for inflow into nonterminal manhole = $k_e(V^2/2g)$
Column 15: Bend head loss due to deflection of flow through manhole (fig. 14.20-e)
Column 16: Total head loss through the junctions = column 13 + column 14 + column 15
Column 17: Head loss for contributing flow from surface inlet = column 16 X 1.3 if surface discharge is greater than 10% of the mainline flow
Column 18: Reduction in head loss of column 16 for inlet shaping = column 16 or column 17 X 0.5
Column 19: EGL on the upstream side of the manhole structure = column 2 + column 11 or column 16, 17, or 18
Column 20: Velocity head of the incoming pipe = $V^2/2g$
Column 21: HGL on the upstream side of the manhole structure = column 19 - column 20
Column 22: Elevation of top of manhole or inlet structure.
Figure 3.3-1: Sample EGL/HGL Profile
4.0 Stormwater Management Facilities

There are several stormwater management facilities used to manage stormwater from the proposed site, including a stormwater retention pond, dry wells, and an underground detention system, which are discussed in this section.

4.1 Dry Wells

Five dry wells are proposed on the site to manage stormwater in areas where there would be a high volume of overland runoff. The addition of dry wells detain stormwater until it gradually infiltrates into the soil, which mitigates surface pooling, water leaving the site in high volumes, and sediment erosion.

Using the New York State Stormwater Design Manual, the amount and size of the dry wells are determined. The dry wells are designated as precast dry wells with a 8-foot diameter and 5 feet deep; surrounded by 1 foot of crushed 3/4 inch stone. The surrounding crushed stone serves to help the water infiltrate the soil. The selected dry well, shown in Figure 4.1-1 below meets the requirements for storage and the manufacturer is based in New Jersey.

![Figure 4.1-1: H20 Dry 1 from Mershon Concrete](image-url)
The location of the dry wells is determined by analyzing where the runoff being directed by the proposed swells is draining towards. If the mentioned runoff would be leaving the site in a greater magnitude than the runoff from the existing conditions, a dry well is placed. The locations at which these dry wells are shown in Figures 4.1-2 and Figure 4.1-3 below for the East and the West sides of the project site, respectively.

Figure 4.1-2: Dry Wells Placed at the Right of the Site
4.2 Underground Detention System

Due to limitations in the maximum size of stormwater retention pond that is able to be
designed for this site (from natural topography and building locations), it is necessary to include
underground detention chambers beneath the parking lot in order to retain and detain all of
the runoff produced by the site.

In order for adequate design of the detention system, a method for calculating the
volume from the peak flow rate is used, in which it is assumed that the fractional ratio of all
peak discharges contributing to the pond is approximately proportional to the fractional ratio
contribution to the complete volume of runoff. While this method is not exact, it does provide
an estimate within a small percent error and is an efficient method of determining the required
storage volumes. Then, using these ratios and the total known volume of runoff generated, the volume required to be detained is determined.

Due to their large contribution to the pond and availability of adjacent space to drainage pipes that does not conflict with other systems, the water coming from the roofs of the site is determined to be most apropos to be attempted to retain. Therefore, using the Stormtech online tool merchant, a configuration for the MC-7200 series detention chamber is designed, in two units on the site (one for each roof drainage pipe). Figure 4.2-1 below shows a sample plan view of one of the detention chambers, from the main building roof out of catch basin 01.

![Figure 4.2-1. Retention Pond Detail](image)

Note that the hatched chamber is where water is initially fed into (as per Stormtech specifications) and is the isolator row for the system for sediment control, with water then infiltrating to the other chamber for percolation into the ground.

Both detention chambers are the same size and detain approximately 0.07 acre-feet of runoff, for a total detention volume of 0.14 acre-feet. The remainder of the site runoff is handled by the detention pond.

### 4.3 Stormwater Retention Pond

The proposed retention pond for this development is located on the far West of the site and holds a large volume of runoff from the site. All of the catch basins and area drains, as well as drainage from the building roofs, are channeled through the drainage system to the stormwater retention pond (with some being abstracted from the roofs to the underground
detention chambers). The pond is designed such that the maximum depth never exceeds four feet for safety, especially since this is on school property with young children.

To design the stormwater retention pond, the New York State Stormwater Management Design Manual is utilized. A forebay, aquatic bench, riser chamber with outfall pipe, and an emergency spillway are all designed into the stormwater retention pond. Figure 4.3-1 below shows a CAD model view of the graded retention pond with facilities included and called out.

![Figure 4.3-1. Graded Stormwater Retention Pond with Facilities Shown](image)

Following the New York State Stormwater Management Design Manual, the general storage requirements for the water quality volume, stream channel protection, peak control, and flood control are calculated. The results for the calculated general storage requirements are shown in Table 4.3-1 below.

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Category</th>
<th>Volume Required (ac-ft)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>WQv</td>
<td>Water Quality Volume</td>
<td>0.0128</td>
<td>Used half for permanent pool</td>
</tr>
<tr>
<td>Cpv</td>
<td>Stream Protection</td>
<td>0.1101</td>
<td>Average ED release rate is 0.029 cfs over 24 hours</td>
</tr>
<tr>
<td>Qp</td>
<td>Peak Control</td>
<td>0.1993</td>
<td>10-year</td>
</tr>
<tr>
<td>Qf</td>
<td>Flood Control</td>
<td>0.2430</td>
<td>100-year</td>
</tr>
</tbody>
</table>
Following the calculation of the required volumes, the storage capacities are calculated for each elevation. These elevations and capacities are plotted against each other to determine a curve that can estimate the elevation to set the water surface level based on the required volume. The storage elevation table which includes the elevations, their areas, depths, and volumes are shown in Table 4.3-2 below.

<table>
<thead>
<tr>
<th>Elevation MSL ft</th>
<th>Area ft^2</th>
<th>Depth ft</th>
<th>Volume ft^3</th>
<th>Cumulative Volume ft^3</th>
<th>Cumulative Volume ac-ft</th>
<th>Volume Above Permanent Pool ac-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>251</td>
<td>80</td>
<td></td>
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<tr>
<td>251.4</td>
<td>235</td>
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<td>0.002</td>
<td>0.000</td>
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<tr>
<td>251.7</td>
<td>270</td>
<td>0.3</td>
<td>81</td>
<td>175</td>
<td>0.004</td>
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<td>252</td>
<td>330</td>
<td>0.3</td>
<td>99</td>
<td>274</td>
<td>0.006</td>
<td>0.004</td>
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<td>253.6</td>
<td>988</td>
<td>0.6</td>
<td>592.8</td>
<td>1527</td>
<td>0.035</td>
<td>0.033</td>
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<td>254</td>
<td>1175</td>
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<td>470</td>
<td>1997</td>
<td>0.046</td>
<td>0.044</td>
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<td>254.6</td>
<td>1842</td>
<td>0.6</td>
<td>1105.2</td>
<td>3102</td>
<td>0.071</td>
<td>0.069</td>
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<tr>
<td>254.92</td>
<td>1998</td>
<td>0.32</td>
<td>639.36</td>
<td>3741</td>
<td>0.086</td>
<td>0.084</td>
</tr>
<tr>
<td>255</td>
<td>2015</td>
<td>0.08</td>
<td>161.2</td>
<td>3903</td>
<td>0.090</td>
<td>0.088</td>
</tr>
</tbody>
</table>

With the help of these values and the storm elevation curve, the elevations for the permanent pool, extended detention, channel protection, overbank protection, and extreme storm are calculated following the procedures from the New York State Stormwater Management Design Manual. Additionally, the type/size of controls are also calculated following the design manual. The results from these calculations are shown in Table 4.3-3 below.
### Table 4.3-3. Water Volumes Elevation Table

<table>
<thead>
<tr>
<th>Control Element</th>
<th>Type/Size of Control</th>
<th>Storage Provided (Acre-feet)</th>
<th>Elevation (MSL)</th>
<th>Discharge (CFS)</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent Pool</td>
<td></td>
<td>0.002</td>
<td>251.4</td>
<td>0</td>
<td>Part of WQv</td>
</tr>
<tr>
<td>Extended Detention (WQv-ED)</td>
<td>3” pipe, sized to 2.0” equivalent diameter</td>
<td>0.004</td>
<td>251.7</td>
<td>0.3924</td>
<td>part of WQv., vol. above perm. pool, discharge is average release rate over 24 hours</td>
</tr>
<tr>
<td>Channel Protection (Cpv-ED)</td>
<td>3” pipe, sized to 2.0” equivalent diameter</td>
<td>0.035</td>
<td>253.6</td>
<td>0.4744</td>
<td>volume above perm. pool, discharge is average release rate over 24 hours</td>
</tr>
<tr>
<td>Overbank Protection (Qp-10)</td>
<td>Use one 2ft x 1ft on a 3’ x 3’ riser, 12” barrel</td>
<td>0.071</td>
<td>254.7</td>
<td>5.9426</td>
<td>volume above perm. pool</td>
</tr>
<tr>
<td>Extreme Storm (Qf-100)</td>
<td>Use 8’ wide earth spillway</td>
<td>0.090</td>
<td>255</td>
<td>8.5985</td>
<td>volume above perm. pool</td>
</tr>
</tbody>
</table>

With respect to the design of the riser chamber, a pond drain with gate valve, a reverse extended detention (ED) pipe with gate valve, and a weir orifice are designed. An anti-seep collar is also designated in order to prevent excessive seepage from the retention pond to the outside of the embankment. A cross-section typical detail of the pond was designed and is shown in Figure 4.3-2 below.

![Figure 4.3-2. Retention Pond Detail](image-url)
Note that each of the four stormwater management volumes, including the water quality volume, stream protection volume, permanent control volume, and flood control volume, are numerically calculated to determine their relative depth in the retention pond, in addition to the standard permanent pool level. These volumes are calculated according to (SOURCE) and the elevations are calculated accordingly with respect to the bottom of the pond elevation.

Also note that places with outfall from pipe systems have loose stone underneath them to prevent erosion of these sections. Typically, the entirety of the bottom of the retention pond is also lined with this type of material.
5.0 Utilities

In this section, the design process for the utility systems for the proposed development is discussed, including water supply, and sanitary waste, as well as supporting calculations and EGL/HGL profiles.

5.1 Water Supply

The water supply system is designed by increasing the wastewater demand by 5% to account for peak conditions during the peak hour of the peak day of water demand. Utilizing EPANET, a reservoir is used to simulate for both peak flow and fire flow scenarios (Shown in Figure 5.1-1 below). The water demands for the main building and gym are 0.074 ft³/s and 0.021 ft³/s respectively. Diameters chosen for the system are 6in and 8in pipes as they are able to provide adequate pressure and velocity to the pipes in both scenarios. Peak flow pressures range from 50 psi to 60 psi. When determining the fire flow demand, the ISO equation is used (Equation 5.1.1).

\[ Q = 18 * C * \sqrt{A} \]  
\textit{Equation 5.1.1}

The type of construction coefficient C is assumed as 1 for ordinary construction. For the area of the building, A, only 30% of the area was used when accounting for sprinklers. The calculated fire flow demand for the main building and gym are 2.28 ft³/s and 1.70 ft³/s, respectively. Fire hydrant demand is taken as 250 GPM or 0.67 i. Fire flow pressures range from 28 psi - 60 psi; pressures in both scenarios are adequate for the system as the minimum and maximum pressures are 20 psi and 100 psi.
Table 5.1-1 below contains the data for the node elevation, base demand, used demand, head, and pressure for each of the nodes in the designed water supply system. Table 5.1-2 below contains the data for each pipe section, including geometric properties, flow, velocity, unit head loss, friction factor, and resulting status.

### Table 5.1-1. Node Data for Peak Flow

<table>
<thead>
<tr>
<th>Node ID</th>
<th>Elevation</th>
<th>Base Demand</th>
<th>Demand</th>
<th>Head</th>
<th>Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Junc J-8</td>
<td>262.5</td>
<td>0.021</td>
<td>0.02</td>
<td>378.53</td>
<td>50.28</td>
</tr>
<tr>
<td>Junc J-11</td>
<td>262.5</td>
<td>0</td>
<td>0</td>
<td>378.53</td>
<td>50.28</td>
</tr>
<tr>
<td>Junc J-9</td>
<td>262.5</td>
<td>0.021</td>
<td>0.02</td>
<td>378.53</td>
<td>50.28</td>
</tr>
<tr>
<td>Junc J-13</td>
<td>262.5</td>
<td>0.021</td>
<td>0.02</td>
<td>378.53</td>
<td>50.28</td>
</tr>
<tr>
<td>Junc J-10</td>
<td>262.5</td>
<td>0</td>
<td>0</td>
<td>378.53</td>
<td>50.28</td>
</tr>
<tr>
<td>Junc J-4</td>
<td>262.5</td>
<td>0.095</td>
<td>0.09</td>
<td>378.54</td>
<td>50.28</td>
</tr>
<tr>
<td>Junc J-3</td>
<td>262.5</td>
<td>0.095</td>
<td>0.09</td>
<td>378.57</td>
<td>50.29</td>
</tr>
</tbody>
</table>
Table 5.1-2. Pipe Data for Peak Flow

<table>
<thead>
<tr>
<th>Link ID</th>
<th>Length (ft)</th>
<th>Diameter (in)</th>
<th>Roughness</th>
<th>Flow (CFS)</th>
<th>Velocity (fps)</th>
<th>Unit Headloss (ft/Kft)</th>
<th>Friction Factor</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe P10</td>
<td>21.19</td>
<td>8</td>
<td>150</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.01</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P11</td>
<td>22</td>
<td>8</td>
<td>150</td>
<td>0.04</td>
<td>0.12</td>
<td>0.01</td>
<td>0.025</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P14</td>
<td>11.104</td>
<td>6</td>
<td>150</td>
<td>0.02</td>
<td>0.11</td>
<td>0.01</td>
<td>0.031</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P12</td>
<td>8.11</td>
<td>8</td>
<td>150</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.01</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P4</td>
<td>66</td>
<td>8</td>
<td>150</td>
<td>0.4</td>
<td>1.15</td>
<td>0.59</td>
<td>0.019</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P3</td>
<td>24.645</td>
<td>8</td>
<td>150</td>
<td>0.5</td>
<td>1.42</td>
<td>0.87</td>
<td>0.018</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P2</td>
<td>318.869</td>
<td>8</td>
<td>150</td>
<td>0.59</td>
<td>1.69</td>
<td>1.2</td>
<td>0.018</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P1</td>
<td>13.84</td>
<td>8</td>
<td>150</td>
<td>0.69</td>
<td>1.97</td>
<td>1.58</td>
<td>0.018</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P6</td>
<td>21.35</td>
<td>8</td>
<td>150</td>
<td>0.15</td>
<td>0.42</td>
<td>0.09</td>
<td>0.022</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P13</td>
<td>5.02</td>
<td>8</td>
<td>150</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.01</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P8</td>
<td>4.89</td>
<td>8</td>
<td>150</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0.01</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P7</td>
<td>6.48</td>
<td>8</td>
<td>150</td>
<td>0.07</td>
<td>0.21</td>
<td>0.02</td>
<td>0.022</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P5</td>
<td>5.02</td>
<td>8</td>
<td>150</td>
<td>0.31</td>
<td>0.88</td>
<td>0.35</td>
<td>0.02</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P9</td>
<td>53.93</td>
<td>8</td>
<td>150</td>
<td>0.06</td>
<td>0.18</td>
<td>0.02</td>
<td>0.025</td>
<td>Open</td>
</tr>
</tbody>
</table>

Table 5.1-3 below contains the data for the node elevation, base demand, used demand, head, and pressure for each of the nodes in the designed water supply system under fire flow conditions. Table 5.1-4 below contains the data for each pipe section, including geometric properties, flow, velocity, unit head loss, friction factor, and resulting status under fire flow conditions.
Table 5.1-3. Node Data for Fire Flow

<table>
<thead>
<tr>
<th>Node</th>
<th>Elevation (ft)</th>
<th>Base Demand (cfs)</th>
<th>Demand (cfs)</th>
<th>Head (ft)</th>
<th>Pressure (psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Junc J-8</td>
<td>262.5</td>
<td>0.021</td>
<td>0.02</td>
<td>327.98</td>
<td>28.37</td>
</tr>
<tr>
<td>Junc J-11</td>
<td>262.5</td>
<td>0.66</td>
<td>0.66</td>
<td>327.95</td>
<td>28.36</td>
</tr>
<tr>
<td>Junc J-9</td>
<td>262.5</td>
<td>0.021</td>
<td>0.02</td>
<td>327.29</td>
<td>28.07</td>
</tr>
<tr>
<td>Junc J-13</td>
<td>262.5</td>
<td>0.021</td>
<td>0.02</td>
<td>327.29</td>
<td>28.07</td>
</tr>
<tr>
<td>Junc J-10</td>
<td>262.5</td>
<td>1.7</td>
<td>1.7</td>
<td>327.04</td>
<td>27.97</td>
</tr>
<tr>
<td>Junc J-4</td>
<td>262.5</td>
<td>0.095</td>
<td>0.09</td>
<td>330.87</td>
<td>29.63</td>
</tr>
<tr>
<td>Junc J-3</td>
<td>262.5</td>
<td>0.095</td>
<td>0.09</td>
<td>338.06</td>
<td>32.74</td>
</tr>
<tr>
<td>Junc J-2</td>
<td>262.5</td>
<td>0.095</td>
<td>0.09</td>
<td>340.82</td>
<td>33.93</td>
</tr>
<tr>
<td>Junc J-1</td>
<td>240</td>
<td>0.095</td>
<td>0.09</td>
<td>377.37</td>
<td>59.52</td>
</tr>
<tr>
<td>Junc J-5</td>
<td>262.5</td>
<td>0.095</td>
<td>0.09</td>
<td>330.34</td>
<td>29.4</td>
</tr>
<tr>
<td>Junc J-6</td>
<td>262.5</td>
<td>0.074</td>
<td>0.07</td>
<td>329.99</td>
<td>29.24</td>
</tr>
<tr>
<td>Junc S-2</td>
<td>262.5</td>
<td>1.7</td>
<td>1.7</td>
<td>327</td>
<td>27.95</td>
</tr>
<tr>
<td>Junc S-1</td>
<td>262.5</td>
<td>2.28</td>
<td>2.28</td>
<td>329.92</td>
<td>29.21</td>
</tr>
<tr>
<td>Junc J-7</td>
<td>262.5</td>
<td>0.074</td>
<td>0.07</td>
<td>329.99</td>
<td>29.24</td>
</tr>
<tr>
<td>Resvr R-1</td>
<td>379</td>
<td>#N/A</td>
<td>-7.03</td>
<td>379</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 5.1-4. Pipe Data for Fire Flow

<table>
<thead>
<tr>
<th>Link ID</th>
<th>Length (ft)</th>
<th>Dia. (in)</th>
<th>Roughness</th>
<th>Flow (cfs)</th>
<th>Velocity (fps)</th>
<th>Unit Headloss</th>
<th>Friction Factor</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pipe P10</td>
<td>21.19</td>
<td>8</td>
<td>150</td>
<td>0.66</td>
<td>1.89</td>
<td>1.47</td>
<td>0.018</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P11</td>
<td>42</td>
<td>8</td>
<td>150</td>
<td>3.44</td>
<td>9.86</td>
<td>31.36</td>
<td>0.014</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P14</td>
<td>11.104</td>
<td>6</td>
<td>150</td>
<td>0.02</td>
<td>0.11</td>
<td>0.01</td>
<td>0.031</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P12</td>
<td>8.11</td>
<td>8</td>
<td>150</td>
<td>3.4</td>
<td>9.74</td>
<td>30.66</td>
<td>0.014</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P4</td>
<td>66</td>
<td>8</td>
<td>150</td>
<td>6.74</td>
<td>19.31</td>
<td>108.89</td>
<td>0.013</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P3</td>
<td>24.645</td>
<td>8</td>
<td>150</td>
<td>6.84</td>
<td>19.58</td>
<td>111.75</td>
<td>0.013</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P2</td>
<td>318.869</td>
<td>8</td>
<td>150</td>
<td>6.93</td>
<td>19.86</td>
<td>114.65</td>
<td>0.012</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P1</td>
<td>13.84</td>
<td>8</td>
<td>150</td>
<td>7.03</td>
<td>20.13</td>
<td>117.57</td>
<td>0.012</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P6</td>
<td>21.35</td>
<td>8</td>
<td>150</td>
<td>2.43</td>
<td>6.96</td>
<td>16.43</td>
<td>0.015</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P13</td>
<td>5.02</td>
<td>8</td>
<td>150</td>
<td>1.7</td>
<td>4.87</td>
<td>8.49</td>
<td>0.015</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P8</td>
<td>4.89</td>
<td>8</td>
<td>150</td>
<td>2.28</td>
<td>6.53</td>
<td>14.63</td>
<td>0.015</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P7</td>
<td>6.48</td>
<td>8</td>
<td>150</td>
<td>0.07</td>
<td>0.21</td>
<td>0.03</td>
<td>0.027</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P5</td>
<td>5.02</td>
<td>8</td>
<td>150</td>
<td>6.65</td>
<td>19.04</td>
<td>106.06</td>
<td>0.013</td>
<td>Open</td>
</tr>
<tr>
<td>Pipe P9</td>
<td>53.93</td>
<td>8</td>
<td>150</td>
<td>4.12</td>
<td>11.81</td>
<td>43.81</td>
<td>0.013</td>
<td>Open</td>
</tr>
</tbody>
</table>
5.2 Sanitary Waste

Sanitary waste demand is calculated in a similar manner to stormwater pipes, utilizing manning’s equation to determine the velocities. The wastewater demand is calculated using the table for recommended design flows from the Land Development Handbook and by calculating the peak factor (Equation 5.2.1), where \( p \) is the number of people in the building, which is estimated to be 758 people for the main building and 200 people for the gymnasium.

\[
P = \frac{14}{1 + p^{0.5}}
\]

Equation 5.2.1

The calculated design flows for the main building and gym from this equation are 0.071 \( \text{ft}^3/\text{s} \) and 0.020 \( \text{ft}^3/\text{s} \) respectively, which are summarized in Table 5.2-1 below. According to The Land Development Handbook, the appropriate required design flow is 16 GPD, taking into consideration cafeterias and showers in the respective school buildings. The pipe material chosen is PVC and the design sizes are either 4” or 8”, where the latter is only used on the pipe that connects to the sewer main in the street. A summarized table of pipe diameters and velocities are shown in Table 5.2-2 below, with a more replete and detailed version included in Appendix C.

Table 5.2-1. Sewage Design Flows for Main Building and Gym

<table>
<thead>
<tr>
<th>Building Type</th>
<th># of people</th>
<th>( p )</th>
<th>Design flow/person/day</th>
<th>Peak Factor</th>
<th>Required Design Flow GPD</th>
<th>Required Design Flow (cfs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Building</td>
<td>758</td>
<td>0.758</td>
<td>16</td>
<td>3.87</td>
<td>46988.36</td>
<td>0.071</td>
</tr>
<tr>
<td>Gym</td>
<td>200</td>
<td>0.2</td>
<td>16</td>
<td>4.15</td>
<td>13273.72</td>
<td>0.020</td>
</tr>
</tbody>
</table>

Table 5.2-2. Sewage Pipe Details

<table>
<thead>
<tr>
<th>Pipe</th>
<th>S (ft/ft) (slope of pipe)</th>
<th>Calculated Pipe Length (ft)</th>
<th>Selected Pipe Diameter (in)</th>
<th>Velocity (ft/s)</th>
<th>Time of Flow (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Building - MH-05</td>
<td>2.96%</td>
<td>143.27</td>
<td>4</td>
<td>5.43</td>
<td>0.09</td>
</tr>
<tr>
<td>Gym - MH-05</td>
<td>7.00%</td>
<td>17.41</td>
<td>4</td>
<td>8.36</td>
<td>0.14</td>
</tr>
<tr>
<td>MH-05 - MH-04</td>
<td>3.64%</td>
<td>137.33</td>
<td>4</td>
<td>6.03</td>
<td>0.10</td>
</tr>
<tr>
<td>MH-04 - MH-03</td>
<td>3.87%</td>
<td>51.69</td>
<td>4</td>
<td>6.22</td>
<td>0.10</td>
</tr>
<tr>
<td>MH-03 - MH-02</td>
<td>1.29%</td>
<td>155.01</td>
<td>8</td>
<td>5.70</td>
<td>0.09</td>
</tr>
</tbody>
</table>
6.0 Erosion and Sediment Control

In this section, the methods and considerations with respect to mitigating erosion and sedimentation on the project site are discussed.

When preparing a site for construction, after the removal of existing vegetation and the excavation of the existing ground, a project site is much more susceptible to undesired erosion effects, particularly due to rainwater. It is problematic for an inordinate amount of sediments to leave from a site, as not only is the project site losing material that it could otherwise have used, but the surrounding area is also contaminated. Uncontrolled sediments traveling to the street, for example, collect in municipal water management systems, and the city, as well as environmental agencies, will levy the blame and responsibility on the project parties.

In order to avoid financial liability for sedimentation occurring off the site, and to protect the existing material on the site, a series of erosion and sediment controls are proposed for this project. With respect to erosion control, the most impactful measure being proposed is a temporary diversion swale with check dams located around the perimeter of the site. These temporary diversions redirect water flowing towards the site into channels which travel around the site to temporary sediment basins, where the water speed and turbidity are reduced to precipitate out sediments, before the water is released.

The temporary diversions and temporary sediment basins prevent excessive water from flowing onto the project site and causing undesired erosion. Furthermore, inside of the temporary diversion, check dams are provided to further control the conveyance of sediment in that channel. Check dams are provided such that the top of one check dam is the lowest base height of the next check dam heading up the temporary diversion, with a maximum spacing of 50 feet. Figure 6.0-1 below shows an example of a temporary diversion with check dams on the East side of the project site, directing water into a temporary sediment basin.
An important method of sediment control which is employed on this project is the addition of a silt fence around the entirety of the site. A silt fence is a standard semi-permeable membrane which allows water to be conveyed through itself, while retaining sediments on the side of the fence of their origin. Silt fences serve a dual purpose, as downhill of the project site they prevent sediments from the development from contaminating the municipality, while uphill of the project site the silt fence prevents sedimentation from outside the site bringing sediment into the site that is not desired. In Figure 6.0-1 above, the silt fence is the line type with black, filled squares around the perimeter of the entire site.

Despite the presence of a silt fence around the perimeter of the site, it is still necessary to provide more provisions, as there must be a gap in this protection to facilitate access to the site by people and vehicles. At this point, a temporary construction access point is provided. The temporary construction access consists of a layer of crushed stone underneath a temporary site access road which is used to remove sediment from the wheels of construction vehicles. Furthermore, a pump truck with a hose is used to spray down the wheels and underbelly of trucks leaving the site to ensure that sediment is removed and trapped by the crushed stone. The water is able to freely move through the stone, while sediments are trapped, much like a very large filter. Figure 6.0-2 below shows the proposed temporary construction access point.
Another proposed method of sediment control for the project site is inlet protection. The drainage structures proposed for the site are installed early in the project due to the excavation requirements, and as such there is a risk of the loose soil from the construction site infiltrating the stormwater drainage structures and management system. Sedimentation in these structures is expensive to clean periodically if not managed correctly. Thus, inlet protection is provided in the project plans to ensure measures are taken to avoid sedimentation in these structures.

There are two types of inlet protection proposed for the project site, including fabric inlet protection, and concrete block inlet protection. Fabric inlet protection is typically used on flatter areas of the site which may see less overall risk of sedimentation via slower overland water flow velocities, and as such are provided on graded areas such as the parking lot and recreation areas with lower final graded slopes. Concrete block inlet protection is provided where water flow may be sufficiently powerful and carry enough sediments to damage or overpower fabric inlet protection, and as such is proposed on areas of the site with higher proposed slopes, namely natural landscaped areas, and dry wells inside of proposed swales.
Figure 6.0-3 below shows an example of a fabric inlet protection in the drawing plans, and Figure 6.0-4 below shows a similar example for a concrete block inlet protection callout location.

Figure 6.0-3. Proposed Fabric Inlet Protection

Figure 6.0-4. Proposed Concrete Block Inlet Protection
**7.0 Pavement and Signage**

In this section, the pavement and signage plan, as well as the roadway typical cross-section, sidewalk typical cross-section, and roadway profile are discussed. Roadway signage placement and geometry are designed to conform to AASHTO and NYSDOT standards. A brief demand forecast is executed to assess the required roadway strength and course characteristics.

**7.1 Pavement & Signage Plan**

A pavement plan is proposed for the project site that considers proper NYSDOT signage that complies with the Manual on Uniform Traffic Control Devices (MUTCD). Signs are provided in locations that give information to drivers about roadway geometry, conflict zones, and regulations which drivers should be aware of. MUTCD has specific and unique standards for every single sign, with exact specified dimensions and colors, referred to by the MUTCD code or identification number of the sign.

With specific respect to the proposed project, a total of 11 signs are deemed to be necessary to properly convey needed information to drivers. These signs include Stop Signs (R1-1), Yield Signs (R1-2), Accessible Reserved Parking (R7-8), Accessible Reserved Van Parking (R7-8a), One Way (R6-1), Keep Right (R4-7a), Horizontal Alignment Sign or Chevron (W1-8), Crosswalk Warning (W11-2), School Zone Warning Assembly (S1-1 & S4-3P), and School-Specific Speed Limit Warning (S5-1).

These signs are proposed in the areas in which they convey accurate and timely information to the driver, for example, chevrons on the exterior sharp curve, or speed limit warning signs entering the site. Figure 7.1-1 below shows the complete proposed signage scheme for the project.

![Figure 7.1-1. Complete Proposed Pavement Plan](image-url)
In addition to the MUTCD-conforming signage, the pavement plan also addresses appropriate pavement markings, including lane keeping markings, directional markings, parking space delineation markings, and pedestrian facility markings.

All road pavement markings comply with MUTCD standards, as well as New York State specific requirements. For example, it is required that stop bars be placed in front of crosswalks with a minimum displacement of 10 feet to allow for overrunning if a vehicle is not able to stop completely in time. All features such as stop bars, crosswalks, and parking stalls have specific dimensions that are standard across the United States, and must be conformed to at all times.

Furthermore, colors of each pavement marking have significance with respect to the usage of that area of the roadway. For example, white lane dividing lines are used when dividing travel lanes that operate in the same direction, whereas yellow lane dividing lines are used for opposing travel directions. Parking stalls are standard yellow, with the exception of accessible spaces, which have blue hatched areas and accessibility symbols as per MUTCD code.

Finally, roadway white directional arrows are provided where the driver would benefit from more information with respect to available roadway travel directions, such as at curves, or junction points. At the exit of the site, these arrows provide guidance on which direction leaving vehicles are able to turn from the site (in this case, due to low volumes on Lake Street, turning both directions is permissible).

7.2 Traffic Study & Roadway Profile and Cross-Sections

In order to determine the required material thickness of the roadway and generate a cross-section that meets strength requirements, a demand forecast is performed for the expected volumes and types of vehicles that the site is anticipated to generate. The preliminary traffic study utilizes average traffic volume data for middle schools in New England and New York, which assesses the number of needed buses and expected passenger cars for a school depending on the number of students. In order to determine the volume of trucks expected on the site, rational reasoning was used to estimate the frequency of supplies and loading trucks.

Based on preliminary traffic analysis, it is expected that the site will experience 300 car loads per day, 10 bus loads per day, and 0.25 truck loads per day over approximately 200 days a year. This study is preliminary and based on available forecasting data, and after completion a followup traffic study should be conducted to observe if vehicular volumes greatly exceed this anticipated amount.

Using the AASHTO equivalent single axle load (ESAL) method, and the general data in Table 7.2-1 below, the total ESAL for the site can be estimated, which is 17,821.
Table 7.2-1 AASHTO Equivalent Site ESAL Calculation

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>Weight (lb)</th>
<th>Forecast Volume (per year)</th>
<th>Axel 1 ESAL factor</th>
<th>Axel 2 ESAL factor (tandem)</th>
<th>Axel 3 ESAL factor (tandem)</th>
<th>Total ESAL</th>
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<tbody>
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<td>-</td>
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The calculated ESAL for the site of 17,821 is below the bounds for minimum pavement design according to AASHTO, and thus the pavement is designed to have a minimum structural number of 2.35.

In order to meet this requirement, as well as the requirements for minimum thickness of each road course, a section of 2 inch hot mix asphalt (HMA) surface course, 2 inch HMA base course, and 6 inch soil-cement subbase (10% cement), are proposed. Below the subbase, there is a proposed 6 inch crushed stone leveling course for ground stability. The structural number for these combined layers is 2.36, which meets the minimum design threshold for the road. The resulting designed road cross-section is shown in Figure 7.2-1 below.

![Figure 7.2-1 Designed Roadway Cross-Section View](image)

In addition to this cross-section, a vertical profile of the roadway from the entrance to the beginning of the parking lot is also provided in the project plans for use by the contractor.
8.0 Engineering Estimate

An engineering estimate is created for the project using only material costs. Costs are calculated according to MasterFormat 2018 from the RSMeansData website and outside sources. The current estimate includes General Requirements, Existing Conditions, Earthwork, Exterior Improvements, and Utilities. The summary table for each division is shown below while a detailed table is shown in Appendix C.

Table 8.0-1. Cost Estimate Summary

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<th>Phase</th>
<th>Cost</th>
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<td>Preconstruction</td>
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<tr>
<td>Earthwork</td>
<td>$939,744.52</td>
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<tr>
<td>Pavement and Signage</td>
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<tr>
<td>Landscaping</td>
<td>$1,851.31</td>
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<tr>
<td>Stormwater Drainage</td>
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<tr>
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<tr>
<td>Sediment and Erosion Control</td>
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<td><strong>Total</strong></td>
<td><strong>$2,053,532.16</strong></td>
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</table>

Preconstruction includes temporary fencing, site survey, demolition and boundary and survey markers required for surveying which amounts to $34,239.17. Earthwork includes excavation, grading, etc and results in a total cost of $939,744.52. Pavement and Signage includes paving, curbs, pavement symbols and more which has a total cost of $242,263.99. Landscaping costs $1,851.31 while stormwater drainage costs $622,271.10. Total water supply material amounts to $73,058.52 and wastewater costs $59,486.56. Sediment and erosion control which includes items such as silt fence, check dams and more have a total cost of $60,285.00. The total engineering estimate of the project is calculated to be $2,053,532.
9.0 Scheduling

A construction schedule was created to effectively demonstrate the timeline of the project. The schedule was created using Microsoft Excel and was divided into several components: Preconstruction, mobilization, demolition, sediment and erosion control, grading, excavation, retaining walls, drainage, pavement, landscaping, internal finishes, and post construction. Workdays include only weekdays and weekends were not considered for work activities to continue. Pre-construction starts on September 1st, 2022 with surveying existing conditions, and concludes on March 14, 2023 with obtaining necessary site permits. Other pre-construction activities include creating the design and planning documents, cost estimation, conducting a feasibility study, sending and reviewing Request For Proposals (RFP), awarding the contract, material selection and obtaining necessary permits.

The construction phase begins with site mobilization on March 15, 2023. Then certain existing components are demolished such as retaining walls and fountain components which lasts from March 30, 2023, to April 6, 2023. Sediment and erosion control measures are taken to protect the site and start on March 11, 2023, and end on May 2, 2023. Once the site is ready, grading takes place from May 3, 2023, to May 17, 2023, while excavation takes place from May 18, 2023, to May 25, 2023. Once excavation is completed, retaining walls are installed from May 26, 2023, to June 2, 2023, Drainage installed from June 3rd, 2023, to August 1, 2023, pavement installed from August 2, 2023, to August 27, 2023, landscaping done from August 28, 2023, to September 4, 2023, and internal finishes done from September 5, 2023, to September 25, 2023. Post construction phase starts on September 26, 2023, with site clean-up. Then, a punch list is generated to keep track of any work or issues remaining in the project. Once all the punch list items are completed along with a final walkthrough and inspections, a certificate of occupancy is obtained. The post construction phase concludes on October 25, 2023, with the official turnover to the project owner. The total duration of the project is 59 weeks. A summary table of the main phases is shown below with a more detailed schedule shown in Appendix D.
Table 9.0-1. Summary of Construction Schedule

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<td>Total Weeks</td>
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# Appendix A - Cut and Fill Estimation

Table A.1. Cut and Fill Estimation Data

<p>| Cell | Northwest | | Northeast | | Southwest | | Southeast | | Avg Change (ft) | | Fill (yd³) |
|------|-----------|---|-----------|---|-----------|---|-----------|---|------------|---|
| Cell | Exist (ft) | Prop (ft) | | Exist (ft) | Prop (ft) | | Exist (ft) | Prop (ft) | | Exist (ft) | Prop (ft) | | Area (ft²) | | |
| A - 3 | 259.44 | 259.44 | | 261.12 | 261.12 | | 249.80 | 249.80 | | 250.80 | 254.00 | | 2500 | 0.80 | 74.07 |
| A - 4 | 249.80 | 249.80 | | 250.80 | 254.00 | | 246.50 | 246.50 | | 249.50 | 249.50 | | 2500 | 0.80 | 74.07 |
| B - 3 | 261.12 | 261.12 | | 262.43 | 262.43 | | 250.80 | 254.00 | | 255.90 | 254.00 | | 2500 | 0.32 | 30.09 |
| B - 4 | 250.80 | 254.00 | | 255.90 | 254.00 | | 249.50 | 249.50 | | 251.31 | 250.93 | | 2500 | 0.23 | 21.30 |
| B - 5 | 249.50 | 249.50 | | 251.31 | 250.93 | | 246.90 | 246.90 | | 248.10 | 248.10 | | 2500 | -0.09 | -8.80 |
| C - 3 | 262.43 | 262.43 | | 262.31 | 262.31 | | 255.90 | 254.00 | | 257.06 | 255.57 | | 2500 | -0.85 | -78.47 |
| C - 4 | 255.90 | 254.00 | | 257.06 | 255.57 | | 251.31 | 250.93 | | 252.67 | 251.95 | | 2500 | -1.12 | -103.94 |
| C - 5 | 251.31 | 250.93 | | 252.67 | 251.95 | | 248.10 | 248.10 | | 249.44 | 248.95 | | 2500 | -0.40 | -36.81 |
| D - 3.5 | 259.05 | 259.05 | | 258.60 | 258.10 | | 257.06 | 255.57 | | 257.20 | 256.33 | | 625 | -0.72 | -16.55 |
| D.5 - 3.5 | 258.60 | 258.10 | | 258.95 | 258.95 | | 257.20 | 256.33 | | 257.30 | 257.70 | | 625 | -0.24 | -5.61 |
| D - 4 | 257.06 | 256.57 | | 257.20 | 256.33 | | 255.05 | 254.05 | | 255.70 | 254.02 | | 625 | -1.26 | -29.17 |
| D - 4.5 | 255.05 | 254.04 | | 255.70 | 254.02 | | 252.67 | 251.95 | | 253.55 | 252.45 | | 625 | -1.13 | -26.10 |
| D.5 - 4 | 257.20 | 256.33 | | 257.30 | 257.70 | | 255.70 | 254.02 | | 255.77 | 257.66 | | 625 | -0.06 | -1.50 |
| D.5 - 4.5 | 255.70 | 254.02 | | 255.77 | 257.66 | | 253.55 | 252.45 | | 254.30 | 253.50 | | 625 | -0.42 | -9.78 |
| D - 5 | 252.67 | 251.95 | | 254.30 | 253.50 | | 249.44 | 248.95 | | 252.10 | 251.84 | | 2500 | -0.57 | -52.55 |
| D - 6 | 249.44 | 248.95 | | 252.10 | 251.84 | | 249.15 | 249.15 | | 250.34 | 250.34 | | 2500 | -0.19 | -17.36 |
| E - 3 | 263.23 | 263.23 | | 263.95 | 263.95 | | 258.95 | 258.95 | | 259.60 | 259.95 | | 625 | 0.09 | 2.03 |
| E - 3.5 | 258.95 | 258.95 | | 259.60 | 259.95 | | 257.30 | 257.70 | | 257.72 | 258.22 | | 625 | 0.31 | 7.23 |
| E.5 - 3 | 263.23 | 263.23 | | 263.95 | 263.95 | | 259.60 | 259.95 | | 260.48 | 260.75 | | 625 | 0.15 | 3.59 |
| E.5 - 3.5 | 259.60 | 259.95 | | 260.48 | 260.75 | | 257.72 | 258.22 | | 258.05 | 258.63 | | 625 | 0.42 | 9.84 |
| E - 4 | 257.30 | 257.70 | | 257.72 | 258.22 | | 255.77 | 257.66 | | 256.27 | 256.08 | | 625 | 0.65 | 15.05 |
| E - 4.5 | 255.77 | 257.66 | | 256.27 | 256.08 | | 254.30 | 253.50 | | 254.85 | 255.29 | | 625 | 0.34 | 7.75 |
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| E.5 - 4.5 | 256.23 | 256.08 | | 256.86 | 255.97 | | 254.85 | 255.29 | | 255.50 | 256.98 | | 625 | 0.47 | 10.88 |
| E - 5 | 254.30 | 253.50 | | 255.50 | 256.98 | | 252.10 | 251.84 | | 253.38 | 253.73 | | 2500 | 0.19 | 17.82 |</p>
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Sample Calculation (Cell P-5)

Change in Northwest corner = 281.31 - 283.29 = -1.98
Change in Northeast corner = 286.95 - 286.95 = 0.00
Change in Southwest corner = 278.60 - 278.60 = 0.00
Change in Southeast corner = 282.00 - 282.00 = 0.00
Average Change = (-1.98 + 0 + 0 + 0 ) / 4 = -0.50
Total Fill = -0.50 ft * 2500 ft² = -1250 ft³ = -45.83 yd³
## Appendix B - Drainage Calculations

### Table B.1. Watershed Calculations

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<th>Unpaved Length From Most Remote Place(ft)</th>
<th>Time of Concentration (Paved) (hrs)</th>
<th>Time of Concentration (Unpaved) (hrs)</th>
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<td>255.65</td>
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<td>4.04</td>
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<td>0.90</td>
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<td>0.78</td>
<td>10.47</td>
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<td>249.50</td>
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<td>32.96</td>
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<td>245.40</td>
<td>245.04</td>
<td>245.04</td>
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<td>8.00</td>
<td>0.35</td>
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<td>1.48</td>
<td>0.67</td>
<td>0.92</td>
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Table B.3. Roughness Coefficients (Manning’s n) for Sheet Flow¹

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<thead>
<tr>
<th>Surface description</th>
<th>n ( \frac{1}{n} )</th>
</tr>
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<tbody>
<tr>
<td>Smooth surfaces (concrete, asphalt, gravel, or bare soil)</td>
<td>0.011</td>
</tr>
<tr>
<td>Fallow (no residue)</td>
<td>0.05</td>
</tr>
<tr>
<td>Cultivated soils:</td>
<td></td>
</tr>
<tr>
<td>Residue cover ( \leq 20% )</td>
<td>0.06</td>
</tr>
<tr>
<td>Residue cover &gt;20%</td>
<td>0.17</td>
</tr>
<tr>
<td>Grass:</td>
<td></td>
</tr>
<tr>
<td>Short grass prairie</td>
<td>0.15</td>
</tr>
<tr>
<td>Dense grasses</td>
<td>0.24</td>
</tr>
<tr>
<td>Bermudagrass</td>
<td>0.41</td>
</tr>
<tr>
<td>Range (natural)</td>
<td>0.13</td>
</tr>
<tr>
<td>Woods:</td>
<td></td>
</tr>
<tr>
<td>Light underbrush</td>
<td>0.40</td>
</tr>
<tr>
<td>Dense underbrush</td>
<td>0.80</td>
</tr>
</tbody>
</table>

¹ The \( n \) values are a composite of information compiled by Engman (1986).
² Includes species such as weeping lovegrass, bluegrass, buffalo grass, blue grama grass, and native grass mixtures.
³ When selecting \( n \), consider cover to a height of about 0.1 ft. This is the only part of the plant cover that will obstruct sheet flow.

Table B.4. Runoff Coefficients for Use in Rational Method²

<table>
<thead>
<tr>
<th>Character of surface</th>
<th>Return period (years)</th>
</tr>
</thead>
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<tr>
<td></td>
<td>2</td>
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<tr>
<td>Developed</td>
<td></td>
</tr>
<tr>
<td>Asphaltic</td>
<td>0.73</td>
</tr>
<tr>
<td>Concrete/roof</td>
<td>0.75</td>
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<tr>
<td>Grass areas (lawns, parks, etc.)</td>
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</tr>
<tr>
<td>Poor condition (grass cover less than 50% of the area)</td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.32</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.37</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.40</td>
</tr>
<tr>
<td>Fair condition (grass cover 50% to 75% of the area)</td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.25</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.33</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.37</td>
</tr>
<tr>
<td>Good condition (grass cover larger than 75% of the area)</td>
<td></td>
</tr>
<tr>
<td>Flat, 0–2%</td>
<td>0.21</td>
</tr>
<tr>
<td>Average, 2–7%</td>
<td>0.29</td>
</tr>
<tr>
<td>Steep, over 7%</td>
<td>0.34</td>
</tr>
</tbody>
</table>

¹From TR-55 https://www.hydrocad.net/pdf/TR-55%20Chapter%203.pdf
²From City of Monterey CA p18-19
https://www.co.monterey.ca.us/home/showpublisheddocument/22433/63624563303870000
Figure B.1. Log/Log IDF Curve Developed for Site Location

Figure B.2. NOAA Reference Precipitation Depth Curve
Figure B.3. Flow in Partially Full Pipes Reference Chart

Figure 10.3
Hydraulic characteristics of circular pipes flowing partly full.

\(^3\)From Linsley and Franzini, 1979
Appendix B Sample Hand Calculations

Watershed Calculations:

Catch Basin: CB-06
Total Watershed Area: 0.15 Acres  
Slope of unpaved area: 25%
Paved Watershed Area: 0.12 Acres  
Type of flow: sheet (less than 300 ft)
Unpaved Watershed Area: 0.03 Acres  
\[ P_2 = 2 \text{ yr} 24 \text{ hr rainfall} = 36 \text{ in (NOAA)} \]
Manning’s Roughness coeff. (paved): 0.011
Manning’s Roughness coeff. (unpaved): 0.15

\[ \text{Time of Concentration (paved)} = \frac{0.007(nL)^{0.8}}{(P_2)^{0.5}} \frac{0.4}{(3.6)_{0.5} (0.06)^{0.4}} = \frac{0.0118 \text{ hrs or 0.708 mins}}{0.0072 \text{ hrs or 0.432 mins}} \]

Total overland time of concentration = paved TOC + unpaved TOC
= 0.708 mins + 0.432 mins = 1.14 mins

Intensity: 10.39 (from IDF curve using minimum number of 5 mins)
Paved 100 yr storm runoff coefficient= 0.95 (from Table 15.1.3)
Unpaved 100 yr storm runoff coefficient= 0.53 (from Table 15.1.3)

Main root total flow rate= \( 2.51 \text{ ft}^3/\text{s} \)

\[ Q(\text{paved}) = CIA = (0.95) (10.39 \text{ in/hr}) (0.12 \text{ acres}) = 1.18 \text{ ft}^3/\text{s} \]

\[ Q(\text{unpaved}) = CIA = (0.53)(10.39 \text{ in/hr})(0.03 \text{ acres}) = 0.16 \text{ ft}^3/\text{s} \]

Total Q= \( Q(\text{paved}) + Q(\text{unpaved}) = 1.18 \text{ ft}^3/\text{s} + 0.16 \text{ ft}^3/\text{s} = 1.34 \text{ ft}^3/\text{s} \)

Cumulative Q= Total Q + Q from previous watershed = \( 1.34 \text{ ft}^3/\text{s} + 2.51 \text{ ft}^3/\text{s} = 3.85 \text{ ft}^3/\text{s} \)

Pipe Calculations:

Pipe CB-05— CB- 08
Inv Out: 260.94 ft
Inv In: 259.30 ft
Horizontal Pipe Length = 54.75 ft

\[ \text{Slope} = \frac{\Delta y}{\Delta x} = \left( \frac{\text{Inv Out} \times \text{Inv In}}{- - - - \text{Pipe Length}} \right) = \frac{260.94 \text{ ft} - 259.30 \text{ ft}}{54.75 \text{ ft}} = 0.02995 = 3\% \]

\[ \text{Pipe Length} = \sqrt{\text{Slope} \times \text{horizontal pipe length}}^2 \] + (horizontal pipe length)²

\[ = \sqrt{(0.03)(54.75 \text{ ft}) + (54.75 \text{ ft})} = 54.77 \text{ ft} \]

Assume Pipe Diameter
Pipe Diameter = 12 in = 1 ft
Pipe Area = \( \pi r^2 = \pi \left( \frac{1ft}{2} \right)^2 = 0.79 \text{ ft}^2 \)

Full Flow Rate = \( \frac{1.49}{n} AR^{2/3} \sqrt{5} = \frac{1.49}{0.01}(0.79 \text{ ft}^2) \left( \frac{1ft}{4} \right)^{2/3} \sqrt{0.03} = 8.04 \frac{ft^3}{s} \)

Cumulative Flow Rate = 3.85 ft/s (From rational formula or watershed area)

\[ \frac{Q}{Q_{\text{Full}}} = \frac{3.85 \text{ ft}^3/s}{8.04 \text{ ft}^3/s} = 0.48 \]

\[ \frac{V}{V_{\text{Full}}} = 0.82 \text{ (from graph)} \]

\[ V_{\text{Full}} = \frac{Q_{\text{Full}}}{\text{Pipe Area}} = \frac{8.04 \text{ ft/s}}{0.79 \text{ ft}^2} = 10.23 \text{ ft/s} \]

\[ V = 0.82(V_{\text{Full}}) = 0.82(10.23 \text{ ft/s}) = 8.39 \text{ ft/s} \]

Pipe Travel Time = \( \frac{\text{Pipe Length}}{\text{Velocity}} \) = \( \frac{54.77 \text{ ft}}{8.39 \text{ ft/s (60)}} = 0.11 \text{ mins} \)

Pipe Time of Concentration

\( = \text{Pipe travel time} + \text{prior connected times of concentration} \)

\( = 0.11 \text{ mins} + 5.07 \text{ mins} = 5.18 \text{ mins} \)
Figure B.4. Graph Used For Pipe Design Calculations
Table B.5: HGL/EGL Calculations for Network 1 (Pond-CB 03)

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<th>Struct No.</th>
<th>D/S EGL</th>
<th>HGL (D/S)</th>
<th>D (in)</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hf (ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>U/S EGL</th>
<th>Incoming V</th>
<th>hv in ft</th>
<th>U/S HGL</th>
<th>Elev</th>
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<td>0.000</td>
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<td>0.000</td>
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Table B.6: HGL/EGL Calculations for Network 2 (MH 01 - AD 01)

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<th>D (in)</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hf (ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht (ft)</th>
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<th>Incoming V</th>
<th>U/S HGL</th>
<th>Elev</th>
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Table B.7: HGL/EGL Calculations for Network 3 (MH-01-AD-01)

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<th>D/S HGL</th>
<th>D (in)</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hf (ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht (ft)</th>
<th>U/S EGL</th>
<th>Incoming V</th>
<th>U/S HGL</th>
<th>Elev</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB-08</td>
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<td>8.30</td>
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<td>0.267</td>
<td>0.374</td>
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<td>0.321</td>
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### Table B.8: HGL/EGL Calculations for Network 4 (CB-08-CB-07)

<table>
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<tr>
<th>Struct No.</th>
<th>D/S EGL</th>
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<th>D/S HGL</th>
<th>D (in)</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht ft</th>
<th>U/S EGL</th>
<th>Incoming V</th>
<th>hv in ft</th>
<th>U/S HGL</th>
<th>Elev</th>
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<tbody>
<tr>
<td>CB-08</td>
<td>254.93</td>
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<td>93</td>
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### Table B.9: HGL/EGL Calculations For Network 5 (CB-04 - Main Roof)

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<th>D/S HGL</th>
<th>D (in)</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht ft</th>
<th>U/S EGL</th>
<th>Incoming V</th>
<th>hv in ft</th>
<th>U/S HGL</th>
<th>Elev</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB-04</td>
<td>259.2</td>
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<td>8.03</td>
<td>19.68</td>
<td>2.35%</td>
<td>0.463</td>
<td>0.25</td>
<td>0.475</td>
<td>0.725</td>
<td>0.943</td>
<td>0.471</td>
<td>9.35</td>
<td>1.358</td>
<td>260.20</td>
<td>258.84</td>
<td>267.4</td>
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<td></td>
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<td></td>
<td>1.001</td>
<td>10.00</td>
<td>8.58</td>
<td>8.03</td>
<td>19.68</td>
<td>2.35%</td>
<td>0.463</td>
<td>0.25</td>
<td>0.725</td>
<td>0.943</td>
<td>0.471</td>
<td>9.35</td>
<td>1.358</td>
<td>260.20</td>
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<tr>
<td>CB-02</td>
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<td>7.34</td>
<td>9.35</td>
<td>54.77</td>
<td>3.19%</td>
<td>1.746</td>
<td>0.339</td>
<td>0.446</td>
<td>0.786</td>
<td>1.021</td>
<td>0.511</td>
<td>9.06</td>
<td>1.275</td>
<td>263.20</td>
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<td></td>
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<td>1.358</td>
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<td>7.34</td>
<td>9.35</td>
<td>54.77</td>
<td>3.19%</td>
<td>1.746</td>
<td>0.339</td>
<td>0.786</td>
<td>1.021</td>
<td>0.511</td>
<td>9.06</td>
<td>1.275</td>
<td>263.20</td>
<td>262.25</td>
<td>267.4</td>
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<td>10.00</td>
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<td>4.94</td>
<td>9.06</td>
<td>4.03%</td>
<td>1.348</td>
<td>0.319</td>
<td>0.332</td>
<td>0.650</td>
<td>0.846</td>
<td>0.423</td>
<td>7.81</td>
<td>0.947</td>
<td>264.37</td>
<td>264.19</td>
<td>268.95</td>
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<td>1.275</td>
<td>8.00</td>
<td>4.94</td>
<td>9.06</td>
<td>4.03%</td>
<td>1.348</td>
<td>0.319</td>
<td>0.332</td>
<td>0.650</td>
<td>0.846</td>
<td>0.423</td>
<td>7.81</td>
<td>0.947</td>
<td>264.37</td>
<td>264.19</td>
<td>268.95</td>
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<td>MAIN ROOF</td>
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</tr>
</tbody>
</table>

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Sample HGL/EGL Calculations For CB-04 in Network 5

\[ h_L = \frac{V^2}{2g} = \frac{8.03^2}{2 \times 32.2} = 1.00 \text{ ft} \]
\[ D/S \text{ EGL} = 258.20 + 1.00 = 259.20 \]
\[ D/S \text{ HGL} = 256.01 - 1.00 = 255.01 \]
\[ H_{\text{entr}} = 1.00 \times 1.00 = 1.00 \]
\[ H_0 = 0.25 \times 1.00 = 0.25 \]
\[ H_1 = 0.35 \times 1.358 = 0.47 \]
\[ H_T = 0.47 + 0.25 = 0.72 \]
\[ 1.3H_t = 1.3 \times 0.72 = 0.943 \]
\[ 0.5H_t = 0.5 \times 0.943 = 0.471 \]
\[ U/S \text{ EGL} = 259.20 + 1.00 = 260.20 \]
\[ \text{Incoming } v \rightarrow \text{ From Table B.2} \]
\[ h_v = \frac{V^2}{2g} = \frac{9.35^2}{2 \times 32.2} = 1.358 \text{ ft} \]
\[ U/S \text{ HGL} = 260.20 - 1.358 = 258.84 \]
# Appendix C - Sanitary Waste Utility Data

## Table C.1. Sanitary Waste Pipe Design Data

<table>
<thead>
<tr>
<th>Pipe</th>
<th>Invert Out (ft)</th>
<th>Invert In (ft)</th>
<th>Horizontal Pipe Length (ft)</th>
<th>Calculated Pipe Length (ft)</th>
<th>Selected Pipe Diameter (in)</th>
<th>Flow rate (Full Flow-Manning s) (ft^3/s)</th>
<th>Required Design Flow (ft^3/s)</th>
<th>Q/Qfull</th>
<th>V/vfull</th>
<th>Full Pipe Velocity</th>
<th>Flow Velocity (Mannings Chart) (ft/s)</th>
<th>Time of Flow (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Building - MH-05</td>
<td>254.2</td>
<td>4</td>
<td>250</td>
<td>143.21</td>
<td>2.96%</td>
<td>143.27</td>
<td>4</td>
<td>0.09</td>
<td>0.47</td>
<td>0.070</td>
<td>0.149</td>
<td>0.49</td>
</tr>
<tr>
<td>Gym - MH-05</td>
<td>253</td>
<td>8</td>
<td>17.36</td>
<td>17.41</td>
<td>7.00%</td>
<td>0.09</td>
<td>0.73</td>
<td>0.020</td>
<td>0.027</td>
<td>0.3</td>
<td>8.36</td>
<td>2.51</td>
</tr>
<tr>
<td>MH-05 - MH-04</td>
<td>249</td>
<td>8</td>
<td>244</td>
<td>137.24</td>
<td>3.64%</td>
<td>0.09</td>
<td>0.53</td>
<td>0.090</td>
<td>0.172</td>
<td>0.44</td>
<td>6.03</td>
<td>2.65</td>
</tr>
<tr>
<td>MH-04 - MH-03</td>
<td>243</td>
<td>15.66</td>
<td>241</td>
<td>51.69</td>
<td>3.87%</td>
<td>0.09</td>
<td>0.54</td>
<td>0.090</td>
<td>0.167</td>
<td>0.52</td>
<td>6.22</td>
<td>3.23</td>
</tr>
<tr>
<td>MH-03 - MH-02</td>
<td>240</td>
<td>8</td>
<td>238</td>
<td>155.00</td>
<td>1.29%</td>
<td>0.35</td>
<td>1.99</td>
<td>0.090</td>
<td>0.045</td>
<td>0.45</td>
<td>5.70</td>
<td>2.56</td>
</tr>
</tbody>
</table>

### Sample Calculations for Required Design Flow for Main Building

\[
P = \frac{14}{1 + p^{0.5}} = \frac{14}{1 + (758^{0.5})} = 3.87
\]

Design Flow/Person/Day = 16 GPD

Required Design Flow = # of people * Peak Factor * Design Flow/Person/Day

= 758 * 16 GPD * 3.87 = 46,988.36 GPD

GPD -> CFS = (46,988.36 GPD) * 0.0000015 = 0.0705 ft³/s
Sample Calculations For Main Building - MH-05 Pipe

\[
Slope = \frac{\Delta y}{\Delta x} = \left(\frac{\text{Inv Out} \times \text{Inv In}}{-\text{Pipe Length}}\right) = \frac{254.24 \text{ ft} - 250 \text{ ft}}{143.21 \text{ ft}} = 0.0296 = 2.96\% \\
\]

\[
\text{Pipe Length} = \sqrt{(\text{Slope} \times \text{horizontal pipe length})^2 + (\text{horizontal pipe length})^2} \\
\]

Selected Pipe Diameter : 4in

\[
\text{Pipe Area} = \pi \times R^2 = \pi \times (2/12)^2 = 0.09 \text{ ft}^2 \\
\]

\[
\text{Full Flow rate} = \left(\frac{1.49}{n}\right)AR^{2/3}\sqrt{S} = \left(\frac{1.49}{0.009}\right)(0.09 \text{ ft}^2)(4/4)^{2/3}(\sqrt{0.0296}) = 0.47 \text{ ft}^3/s \\
\]

\[
\frac{Q}{Q_{\text{full}}} = \frac{0.0705 \text{ ft}^3/s}{0.47 \text{ ft}^3/s} = 0.149 \\
\]

\[
\frac{V}{V_{\text{full}}} = 0.49 \rightarrow \text{From graph} \\
\]

\[
V_{\text{full}} = \frac{Q}{A} = \frac{0.47 \text{ ft}^3/s}{0.09 \text{ ft}^2/s} = 5.43 \text{ ft/s} \\
\]

\[
\text{Flow Velocity (V)} = 0.49 \times V_{\text{full}} = 0.49 \times 5.43 \text{ ft/s} = 2.66 \text{ ft/s} \\
\]

\[
\text{Time of flow} = \frac{\text{Calculated Pipe Length}}{\text{Flow Velocity}} = \frac{143.27 \text{ ft}}{2.66 \text{ ft/s}} = 53.80 \text{ s or 0.90min} \\
\]
### Table C.2: HGL/EGL Calculations for Network 1 (Sewer Main-Main Building)

<table>
<thead>
<tr>
<th>Struct No.</th>
<th>D/S HGL</th>
<th>hL out</th>
<th>D/S HGL</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hf (ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht ft</th>
<th>U/S HGL</th>
<th>Incomin g V</th>
<th>U/S HGL</th>
<th>Elev</th>
</tr>
</thead>
<tbody>
<tr>
<td>MH-02</td>
<td>238.0</td>
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<td>240.0</td>
<td>0</td>
<td>0.00</td>
<td>0.00</td>
<td>1</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>238.24</td>
<td>237.73</td>
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<td></td>
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<tr>
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<td>1.99</td>
<td>5.70</td>
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<td>0.120</td>
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<td>34577</td>
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<td>254.2</td>
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</table>

### Table C.3: HGL/EGL Calculations for Network 2 (MH-05-Gym)

<table>
<thead>
<tr>
<th>Struct No.</th>
<th>D/S HGL</th>
<th>hL out</th>
<th>D/S HGL</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hf (ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht ft</th>
<th>U/S HGL</th>
<th>Incomin g V</th>
<th>U/S HGL</th>
<th>Elev</th>
</tr>
</thead>
<tbody>
<tr>
<td>MH-05</td>
<td>251.7</td>
<td>8</td>
<td>253.0</td>
<td>0</td>
<td>1.084</td>
<td>1.084</td>
<td>1</td>
<td>0.73</td>
<td>8.36</td>
<td>1.74</td>
<td>0.271</td>
<td>0.176</td>
<td>0.447</td>
<td>72529</td>
<td>34577</td>
<td>252.87</td>
<td>251.78</td>
<td>260.1</td>
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<td></td>
</tr>
<tr>
<td>GYM</td>
<td>254.2</td>
<td>4</td>
<td>254.2</td>
<td>4</td>
<td>0.504</td>
<td>4.00</td>
<td>1</td>
<td>1.99</td>
<td>5.70</td>
<td>0.126</td>
<td>0.210</td>
<td>0.336</td>
<td>0.437</td>
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<td>254.58</td>
<td>253.98</td>
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<table>
<thead>
<tr>
<th>Struct No.</th>
<th>D/S HGL</th>
<th>hL out</th>
<th>D/S HGL</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hf (ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht ft</th>
<th>U/S HGL</th>
<th>Incomin g V</th>
<th>U/S HGL</th>
<th>Elev</th>
</tr>
</thead>
<tbody>
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<td>MH-05</td>
<td>251.7</td>
<td>8</td>
<td>253.0</td>
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<td>1.084</td>
<td>1.084</td>
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<td>8.36</td>
<td>1.74</td>
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<td>0.447</td>
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<td>GYM</td>
<td>254.2</td>
<td>4</td>
<td>254.2</td>
<td>4</td>
<td>0.504</td>
<td>4.00</td>
<td>1</td>
<td>1.99</td>
<td>5.70</td>
<td>0.126</td>
<td>0.210</td>
<td>0.336</td>
<td>0.437</td>
<td>51825</td>
<td>254.58</td>
<td>254.58</td>
<td>253.98</td>
<td>263.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Struct No.</th>
<th>D/S HGL</th>
<th>hL out</th>
<th>D/S HGL</th>
<th>Q (cfs)</th>
<th>V (fps)</th>
<th>L (ft)</th>
<th>Sf (ft/ft)</th>
<th>Hf (ft)</th>
<th>Hex (ft)</th>
<th>H entr</th>
<th>H0</th>
<th>H1</th>
<th>Hb</th>
<th>HT</th>
<th>1.3 Ht ft</th>
<th>0.5 Ht ft</th>
<th>U/S HGL</th>
<th>Incomin g V</th>
<th>U/S HGL</th>
<th>Elev</th>
</tr>
</thead>
<tbody>
<tr>
<td>MH-05</td>
<td>251.7</td>
<td>8</td>
<td>253.0</td>
<td>0</td>
<td>1.084</td>
<td>1.084</td>
<td>1</td>
<td>0.73</td>
<td>8.36</td>
<td>1.74</td>
<td>0.271</td>
<td>0.176</td>
<td>0.447</td>
<td>72529</td>
<td>34577</td>
<td>252.87</td>
<td>251.78</td>
<td>260.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>GYM</td>
<td>254.2</td>
<td>4</td>
<td>254.2</td>
<td>4</td>
<td>0.504</td>
<td>4.00</td>
<td>1</td>
<td>1.99</td>
<td>5.70</td>
<td>0.126</td>
<td>0.210</td>
<td>0.336</td>
<td>0.437</td>
<td>51825</td>
<td>254.58</td>
<td>254.58</td>
<td>253.98</td>
<td>263.3</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Appendix D - Pond Design

Table D.1: Preliminary information and Water Quality Volume

<table>
<thead>
<tr>
<th>90% Rainfall in New York State (in) (Based on Figure 4.1)</th>
<th>1.5</th>
<th>Soil Type</th>
<th>Hydrologic soil group (HSG) C</th>
<th>I (Impervious Cover)</th>
<th>77.65%</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Year 24 Hour Precipitation, P (in)</td>
<td>2.94</td>
<td>Total Developed Area (ac)</td>
<td>1.79</td>
<td>Runoff Coefficient, Rv</td>
<td>0.06</td>
</tr>
<tr>
<td>Rainfall Type (Based on Figure B-2)</td>
<td>III</td>
<td>Total Impervious Area (ac)</td>
<td>1.39</td>
<td>Water Quality Volume, WQv (ac-ft)</td>
<td>0.0128</td>
</tr>
</tbody>
</table>

Table D.2: Site Intensity and Precipitation Depth for 1, 2, 10, and 100 year storms

<table>
<thead>
<tr>
<th>24 Hour Precipitation (White Plains, NY)</th>
<th>1-year</th>
<th>2-year</th>
<th>10-year</th>
<th>100-year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Intensity (in/hr)</td>
<td>0.122</td>
<td>0.148</td>
<td>0.224</td>
<td>0.345</td>
</tr>
<tr>
<td>Precipitation Depth (in)</td>
<td>2.94</td>
<td>3.55</td>
<td>5.38</td>
<td>8.28</td>
</tr>
</tbody>
</table>

Table D.3: Pre Development Characteristics

<table>
<thead>
<tr>
<th>Storm</th>
<th>Precipitation Depth (in)</th>
<th>Runoff, Q (in)</th>
<th>Peak Runoff (ft^3/s)</th>
<th>Volume (ft^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-year</td>
<td>2.94</td>
<td>0.21</td>
<td>0.81</td>
<td>1365</td>
</tr>
<tr>
<td>2-year</td>
<td>3.55</td>
<td>0.42</td>
<td>0.94</td>
<td>2729</td>
</tr>
<tr>
<td>10-year</td>
<td>5.38</td>
<td>0.82</td>
<td>1.46</td>
<td>5328</td>
</tr>
<tr>
<td>100-year</td>
<td>8.28</td>
<td>1.22</td>
<td>2.90</td>
<td>7927</td>
</tr>
</tbody>
</table>

Table D.4: Post Development Characteristics

<table>
<thead>
<tr>
<th>Storm</th>
<th>Precipitation Depth (in)</th>
<th>Runoff, Q (in)</th>
<th>Peak Runoff (ft^3/s)</th>
<th>Volume (ft^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-year</td>
<td>2.94</td>
<td>1.15</td>
<td>4.55</td>
<td>7472</td>
</tr>
<tr>
<td>2-year</td>
<td>3.55</td>
<td>1.91</td>
<td>5.27</td>
<td>12411</td>
</tr>
<tr>
<td>10-year</td>
<td>5.38</td>
<td>2.42</td>
<td>8.66</td>
<td>15724</td>
</tr>
<tr>
<td>100-year</td>
<td>8.28</td>
<td>3.08</td>
<td>14.85</td>
<td>20013</td>
</tr>
</tbody>
</table>
### Table D.5: Curve Numbers

<table>
<thead>
<tr>
<th></th>
<th>Ia</th>
<th>Ia/P</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre Development CN (From Table 2-2c)</td>
<td>70</td>
<td>0.857</td>
</tr>
<tr>
<td>Post Development CN (From Table 2-2a)</td>
<td>92.6</td>
<td>0.1165</td>
</tr>
</tbody>
</table>

### Table D.6: Cpv, Qp10, and Extreme Flood Volume Calculations

#### Channel Protection Volume, Cpv

<table>
<thead>
<tr>
<th>qu, csm/in</th>
<th>qp</th>
<th>qO/qI</th>
<th>Vs/Vr</th>
<th>Vs, ac-ft</th>
<th>Average Release Rate, cfs</th>
</tr>
</thead>
<tbody>
<tr>
<td>650</td>
<td>5.345</td>
<td>0.030</td>
<td>0.64</td>
<td>0.1101</td>
<td>0.0555</td>
</tr>
</tbody>
</table>

#### Overland Flood Protection Volume, Qp10

<table>
<thead>
<tr>
<th>qO (ft^3/s)</th>
<th>qI (ft^3/s)</th>
<th>qO/qI</th>
<th>Vs/Vr</th>
<th>Vs, ac-ft</th>
<th>Qp10, ac-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.46</td>
<td>8.66</td>
<td>0.17</td>
<td>0.48</td>
<td>0.1733</td>
<td>0.1993</td>
</tr>
</tbody>
</table>

#### Extreme Flood Protection Volume, Qf

<table>
<thead>
<tr>
<th>qO (ft^3/s)</th>
<th>qI (ft^3/s)</th>
<th>qO/qI</th>
<th>Vs/Vr</th>
<th>Vs, ac-ft</th>
<th>Qf, ac-ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.90</td>
<td>14.85</td>
<td>0.20</td>
<td>0.46</td>
<td>0.2113</td>
<td>0.2430</td>
</tr>
</tbody>
</table>

### Table D.7: Required Volumes

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Category</th>
<th>Volume Required (ac-ft)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>WQv</td>
<td>Water Quality Volume</td>
<td>0.0128</td>
<td>Used half for permanent pool</td>
</tr>
<tr>
<td>Cpv</td>
<td>Stream Protection</td>
<td>0.1101</td>
<td>Average ED release rate is 0.029 cfs over 24 hours</td>
</tr>
<tr>
<td>Qp</td>
<td>Peak Control</td>
<td>0.1993</td>
<td>10-year</td>
</tr>
<tr>
<td>Qf</td>
<td>Flood Control</td>
<td>0.2430</td>
<td>100-year</td>
</tr>
</tbody>
</table>

### Table D.8: Percentage of Water Going into the Pond

<table>
<thead>
<tr>
<th>Category</th>
<th>Volume Required (ac-ft)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total Storage</td>
<td>0.2430402333</td>
<td>ac-ft</td>
</tr>
<tr>
<td>Pond Storage</td>
<td>0.08959044995</td>
<td>ac-ft</td>
</tr>
<tr>
<td>Detention Storage</td>
<td>0.1534497834</td>
<td>ac-ft</td>
</tr>
<tr>
<td>Ratio of water going to pond</td>
<td>30.72%</td>
<td></td>
</tr>
</tbody>
</table>
### Table D.9: Runoff Reduction Volume (RRv)

<table>
<thead>
<tr>
<th>Runoff Reduction Volume, RRv (ac-ft)</th>
<th>0.00041</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rv</td>
<td>0.057</td>
</tr>
<tr>
<td>Ai</td>
<td>0.417</td>
</tr>
<tr>
<td>Specific Reduction Factor (S)</td>
<td>0.30</td>
</tr>
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</table>

### Table D.10: Corrected Required Volumes

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Category</th>
<th>Volume Required (ac-ft)</th>
<th>Notes</th>
</tr>
</thead>
<tbody>
<tr>
<td>WQv</td>
<td>Water Quality Volume</td>
<td>0.00392</td>
<td>Used half for permanent pool</td>
</tr>
<tr>
<td>Cpv</td>
<td>Stream Protection</td>
<td>0.03381</td>
<td>Average ED release rate is 0.029 cfs over 24 hours</td>
</tr>
<tr>
<td>Qp</td>
<td>Peak Control</td>
<td>0.06121</td>
<td>10-year</td>
</tr>
<tr>
<td>Qf</td>
<td>Flood Control</td>
<td>0.07466</td>
<td>100-year</td>
</tr>
</tbody>
</table>

### Table D.11: Necessary Volumes (Based off prior table)

<table>
<thead>
<tr>
<th>Units</th>
<th>ac-ft</th>
<th>ft^3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wet Forebay</td>
<td>0.000392</td>
<td>17.1</td>
</tr>
<tr>
<td>Permanent Pool Volume</td>
<td>0.001959</td>
<td>85.3</td>
</tr>
<tr>
<td>WQv-ED</td>
<td>0.003917</td>
<td>170.6</td>
</tr>
</tbody>
</table>

### Table D.12: Volumes At Each Elevation

<table>
<thead>
<tr>
<th>Elevation MSL</th>
<th>Area</th>
<th>Depth</th>
<th>Volume</th>
<th>Cumulative Volume</th>
<th>Cumulative Volume</th>
<th>Volume Above Permanent Pool</th>
</tr>
</thead>
<tbody>
<tr>
<td>ft</td>
<td>ft^2</td>
<td>ft</td>
<td>ft^3</td>
<td>ft^3</td>
<td>ac-ft</td>
<td>ac-ft</td>
</tr>
<tr>
<td>251</td>
<td>80</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>251.4</td>
<td>235</td>
<td>0.4</td>
<td>94</td>
<td>94</td>
<td>0.002</td>
<td>0.000</td>
</tr>
<tr>
<td>251.7</td>
<td>270</td>
<td>0.3</td>
<td>81</td>
<td>175</td>
<td>0.004</td>
<td>0.002</td>
</tr>
<tr>
<td>252</td>
<td>330</td>
<td>0.3</td>
<td>99</td>
<td>274</td>
<td>0.006</td>
<td>0.004</td>
</tr>
<tr>
<td>253</td>
<td>660</td>
<td>1</td>
<td>660</td>
<td>934</td>
<td>0.021</td>
<td>0.019</td>
</tr>
<tr>
<td>253.6</td>
<td>988</td>
<td>0.6</td>
<td>592.8</td>
<td>1527</td>
<td>0.035</td>
<td>0.033</td>
</tr>
<tr>
<td>254</td>
<td>1175</td>
<td>0.4</td>
<td>470</td>
<td>1997</td>
<td>0.046</td>
<td>0.044</td>
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</table>
### Table D.13: Summary of Controls Provided

<table>
<thead>
<tr>
<th>Control Element</th>
<th>Type/Size of Control</th>
<th>Storage Provided</th>
<th>Elevation</th>
<th>Discharge</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent Pool</td>
<td></td>
<td></td>
<td>0.002</td>
<td>251.4</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Part of WQv</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extended Detention (WQv-ED)</td>
<td>3&quot; pipe, sized to 2.0&quot; equivalent diameter</td>
<td>0.004</td>
<td>251.7</td>
<td>0.3924</td>
<td>Part of WQv, vol. above perm. pool, discharge is average release rate over 24 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Channel Protection (Cpv-ED)</td>
<td>3&quot; pipe, sized to 2.0&quot; equivalent diameter</td>
<td>0.035</td>
<td>253.6</td>
<td>0.4744</td>
<td>Volume above perm. pool, discharge is average release rate over 24 hours</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Overbank Protection (Qp-10)</td>
<td>Use one 2ft x 1ft slot on a 6' x 6' riser, 12&quot; barrel</td>
<td>0.071</td>
<td>254.7</td>
<td>5.9426</td>
<td>Volume above perm. pool</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Extreme Storm (Qf-100)</td>
<td>Use 8' wide earth spillway</td>
<td>0.090</td>
<td>255</td>
<td>8.5985</td>
<td>Volume above perm. pool</td>
</tr>
</tbody>
</table>

### Orifice Calculation Tables:

#### Table D.14: WQv-ED Orifice Calculation

<table>
<thead>
<tr>
<th>WQv-ED Release Rate</th>
<th>Orifice elevation</th>
<th>Orifice elevation</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.001975</td>
<td>251.4</td>
<td></td>
</tr>
<tr>
<td>A</td>
<td>0.001059 ft^2</td>
<td>Wsel</td>
</tr>
<tr>
<td>D</td>
<td>0.44 in</td>
<td>Say</td>
</tr>
<tr>
<td>Q WQv-ED (using 2&quot;)</td>
<td>0.3923923657 ft^3/s</td>
<td>Use 3&quot; pipe with 3&quot; gate valve to achieve equivalent diameter</td>
</tr>
</tbody>
</table>
### Table D.15: CPV Orifice Calculation

<table>
<thead>
<tr>
<th>CPv Size to release Average</th>
<th>ft³/s</th>
<th>Orifice elevation</th>
<th>251.7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Head</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wsel</td>
<td></td>
<td></td>
<td>253.6</td>
</tr>
<tr>
<td>Average Release Rate Elevation</td>
<td></td>
<td>Storage at average release rate elevation</td>
<td>0.014</td>
</tr>
<tr>
<td>Average Release Rate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>CP Release Rate</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>A</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Q WQv-ED (using 2&quot;)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table D.16: CP10 Orifice Calculation

<table>
<thead>
<tr>
<th>10 year predeveloped discharge</th>
<th>Units</th>
<th>Orifice elevation</th>
<th>253.6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Post Development Inflow</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Volume Required</td>
<td></td>
<td>Average Head</td>
<td>1</td>
</tr>
<tr>
<td>Slot length</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Maximum Q</td>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>A</td>
<td></td>
<td></td>
<td>2</td>
</tr>
<tr>
<td>Q</td>
<td></td>
<td></td>
<td>6.2</td>
</tr>
<tr>
<td>Q10</td>
<td></td>
<td></td>
<td>6.2</td>
</tr>
<tr>
<td>Hw</td>
<td></td>
<td></td>
<td>4.1</td>
</tr>
<tr>
<td>Try</td>
<td></td>
<td>inch RCP</td>
<td></td>
</tr>
<tr>
<td>Discharge</td>
<td></td>
<td></td>
<td>7</td>
</tr>
</tbody>
</table>

Use 3" pipe with 3" gate valve to achieve equivalent diameter.
### Table D.17: Emergency Spillway Calculation

<table>
<thead>
<tr>
<th>Description</th>
<th>Value 1</th>
<th>Value 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hazard Class &quot;A&quot; Dam</td>
<td></td>
<td>254.6</td>
</tr>
<tr>
<td>Orifice elevation, ft</td>
<td>2.90</td>
<td>Wsel, ft</td>
</tr>
<tr>
<td>Post Developement Inflow</td>
<td>14.85</td>
<td>Average head, ft</td>
</tr>
<tr>
<td>Volume Required</td>
<td>0.0859</td>
<td>Use 8' wide earth spillway</td>
</tr>
<tr>
<td>Slot length</td>
<td>2.0019</td>
<td>ft</td>
</tr>
<tr>
<td>Maximum Q</td>
<td>24.8</td>
<td>ft^3/s</td>
</tr>
<tr>
<td>A</td>
<td>8</td>
<td>ft^3</td>
</tr>
<tr>
<td>Q</td>
<td>15.4079</td>
<td>&gt;</td>
</tr>
<tr>
<td>Emergency Spillway Flowrate</td>
<td>8.5985</td>
<td>ft^3/s</td>
</tr>
</tbody>
</table>
## Appendix E - Engineering Estimate

Table E.1 Complete Engineering Estimate

<table>
<thead>
<tr>
<th>Phase</th>
<th>Category</th>
<th>Quantity</th>
<th>Unit of Measure</th>
<th>Unit Cost</th>
<th>Total Cost</th>
</tr>
</thead>
<tbody>
<tr>
<td>Preconstruction</td>
<td>Temporary Fencing</td>
<td>3,736.73</td>
<td>L.F</td>
<td>$4.50</td>
<td>$16,815.30</td>
</tr>
<tr>
<td></td>
<td>Site Survey-Topographical Topographical</td>
<td>10.57</td>
<td>Acres</td>
<td>$18.20</td>
<td>$192.37</td>
</tr>
<tr>
<td></td>
<td>Demolition of Existing Stone Wall</td>
<td>101.62</td>
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## Appendix F - Construction Schedule

### Table F.1 Complete Construction Schedule

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**Post Construction**

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**Total**

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**Grand Total**

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