Preliminary Design Report, Wastewater Treatment Facility Flood Protection and Mitigation Design, Warwick, Rhode Island

Prepared for:

Warwick Sewer Authority
Warwick, Rhode Island

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AECOM

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A. Background

The Warwick Sewer Authority (WSA) owns and maintains a 7.7 million gallons per day (mgd) Advanced Wastewater Treatment Facility (WWTF) which discharges into the Pawtuxet River, a major tributary to Narragansett Bay. After repetitive flood damage to the WWTF in the 1960s and 1970s, the City constructed a protective berm, or levee, in the mid-1980s to protect the WWTF from future damages. The City's Animal Shelter is also located within the confines of the levee. The levee was designed to protect to the 100-year flood level (26.3 NAVD), plus three feet of freeboard (29.3 NAVD). For all but a few days per year, treatment plant effluent flows by gravity to the Pawtuxet River. When the river elevation is high, pumping is needed to convey the final effluent to the river.

In addition to the levee, the existing flood protection system includes the interior drainage system and groundwater seepage control. The interior drainage system collects stormwater and groundwater from the toe drain system and transports it to the drainage junction chamber to the east of the effluent pump station by gravity. According to the levee design files, the system was designed to prevent flooding up to the 100-year storm. Groundwater seepage under and through the levee is collected in the toe drain system on the inboard side of the levee. Controlling groundwater through the toe drain system is important to the overall flood protection scheme as it alleviates high hydrostatic pressure within the levee.

In March of 2010, record rainfall in Rhode Island caused the Pawtuxet River to crest to the highest levels ever recorded at the United States Geological Survey (USGS) gauge on the Pawtuxet River at the Warwick-Cranston line. On March 31, 2010, the gauge recorded a river crest of 28 NAVD (28.8 NGVD). High water at the WWTF site reached 31.5 NAVD (32.3 NGVD), 3 feet above the top of the dike, flooding the plant with over 10 feet of water in a matter of hours. This report evaluates options to increase flood protection to prevent the flood protection system from being overwhelmed in the future as it no longer protects against the 100-year flood.

There are three modes of levee failure: overtopping, levee failure due to seepage, and breakthrough of groundwater on the dry side of the levee during an extended period of high floodwaters. Flooding is also possible via overloading of the internal storm drainage system. Each will be addressed separately.

B. Levee Overtopping

AECOM recommends that the top-of-levee be designed to protect against the 500 year flood by increasing the top of the levee to elevation 35 NAVD (35.8 NGVD), or 7.7 feet higher than the current levee elevation. AECOM recommends this elevation because it meets the Federal Emergency Management Agency (FEMA) levee accreditation standards (100 year flood and 3' free board), provides protection against the March 2010 flood of record (El. 31.5 NAVD), and the USGS's current estimate of the 500-year flood of 33.1 feet NAVD (33.9 NGVD) with approximately 2' of free board. While the final 500 year flood elevation is still being finalized by USGS, minor adjustments to this elevation can be made at later stages of design should USGS revise its 500-year flood elevation estimates.

Three potential levee alignments for raising the elevation were evaluated from outboard (outside the levee) construction alignments to inboard (inside the levee) alignments.

AECOM recommends raising the levee height by expanding the levee inward with a wall system such as gabion walls or sheeting. AECOM believes that it is the most feasible because it can be implemented quickly and presents fewer regulatory hurdles and environmental impacts such as construction activities within wetlands and the need for compensatory flood storage and wetland mitigation.

Using an inboard levee alignment, four design alternatives were then developed for raising the levee height:

1. A 100-year flood protection alternative using earth and gabion walls;

2. A 500-year protection alternative consisting of an earthen levee with gabion walls near structures;

3. A 500-year protection alternative consisting of vinyl sheeting flood walls, and;

4. A 500-year protection alternative with a combination of earthen walls and vinyl sheeting.

After considering the identified advantages, disadvantages, and estimated construction cost, AECOM recommends moving forward with Alternative 4 – 500 Year Earthen Levee & Flood Wall Combination. Alternative 4 provides a cost effective means to increase the flood protection at the WWTF to protect against the March 2010 flood and 500-year flood while also preserving inboard space for future structures, minimizing impact on wetlands and flood plain, reducing structural impacts to existing structures, eliminating constructability concerns and preserving space for the sixth secondary clarifier, and providing repair access for the main plant wastewater pipeline.

C. Groundwater Seepage & Breakthrough Evaluation

Groundwater seepage through the existing levee is controlled by an existing toe drain system. The toe drains are intended to prevent seepage from channeling or “piping” through the levee and weakening its integrity and stability.

AECOM developed a preliminary groundwater model of the area immediately around the existing levee system to estimate the expected seepage during the 500-year flood event and to estimate the effectiveness of the existing toe drain system. The modeling found that the existing toe drain appears effective in maintaining the stability and integrity of the exiting levee system at the 500-year flood elevation. The modeling also showed that groundwater breakthrough via the ground surface was possible during the 500-year flood event though the extent and rate of the breakthrough were not able to be quantified with the data available.

Based on the results obtained, AECOM recommends making no changes to the existing toe drain system other than extending the cleanouts to the grade level of the raised levee. AECOM also recommends including the cost of installing a second deeper drain system that would operate independently of the toe drain system to prevent groundwater breakthrough throughout the site. If necessary and as currently envisioned, this system would only operate during severe storm events and would require the use of mobile pumps to transfer the groundwater over the levee. It should be noted that additional field investigations and analysis will be performed during the design to confirm the preliminary model results for seepage and groundwater breakthrough and to estimate the actual flow and pumping needs.

D. Interior Drainage Evaluation

A preliminary hydrologic / hydraulic modeling was used to evaluate the ability of the existing effluent pump station and interior drainage system to transport and process the plant's maximum day wastewater flow and the storm discharge (off site and on site) under current and future situations. The model results were checked against two historical storms. The preliminary model results were then compared to the capacity of the effluent pump station to estimate whether the pump station had capacity to pump the flow generated and if flooding is projected to occur, under which scenarios, and its location, volume and duration.

Based on the model results, an area of flooding near the chlorine contact tanks may occur during the 100-year storm as a result of the pump station capacity being exceeded. The flooding does not last for more than an hour, is less than two feet in depth, does not affect plant equipment, and is able to be mitigated for a reasonable cost by raising the Chlorine Contact Tank walls. Large capital expenditures to prevent interior drainage flooding during a storm event by adding storage or increasing pumping capacity are not recommended. AECOM recommends conducting more extensive hydrologic / hydraulic modeling prior to final design to better define the onsite flooding resulting from the 100-year storm event. The mapping of this interior flooding will need to be included with the FEMA Certification for the new levee system. Once the extent of flooding is better defined, flood protection measures can be finalized and included in the final design, which could include raising the height of the walls on the chlorine contact tanks as required by TR-16 standards.

AECOM recommends that the interior drainage system be accredited for a 100-year storm. AECOM does not believe that the benefits of protecting against a 500-year storm for the interior drainage system justify the cost of larger infrastructure of piping and pumps. The 500-year event is extremely rare and the volume and duration of stormwater generated from this storm inside the levee is identical in volume and slightly shorter in duration as the 100-year event provided all pumps in the effluent pump station are operational. Additionally, the 100-year design level is consistent with FEMA's regulations for accreditation provided the flooded area is properly mapped.

Based on the interior drainage evaluation, AECOM recommends that:

1. The I-95 drainage swales discharging to the WWTF site be redirected to outside of the levee thereby reducing the amount of drainage the internal plant drainage system will need to address;

2. That the piped I-95 drainage system located on the eastern side of the plant property be modified to exclude highway drainage from overtopping two existing manholes within the plant site during a flood event;

3. That flooding be allowed inside the plant site near the Chlorine Contact Tanks and that mitigation measures for the flooding be implemented.

E. Recommended Approach

AECOM's recommended approach follows:

- Continue to monitor USGS revisions to their flood models for updates to the 100- and 500-year flood elevations and refine the levee height should USGS finalize their flood models within the timeline of the project,

- Protect against the 500-year flood by raising the top-of-levee elevation to a maximum of 35 NAVD (35.8 NGVD), by expanding inward using a combination of earthen levee and flood walls where existing structures prevent the expansion of the earthen levee,

- Gather additional data and conduct additional groundwater analysis before making any additions to the groundwater seepage control system,

- Gather additional data and conduct additional hydraulic and hydrologic modeling to map the extent of flooding on-site that is expected from the 100-year rainfall event to satisfy FEMA requirements and make modifications to affected structures to protect against future flooding; and

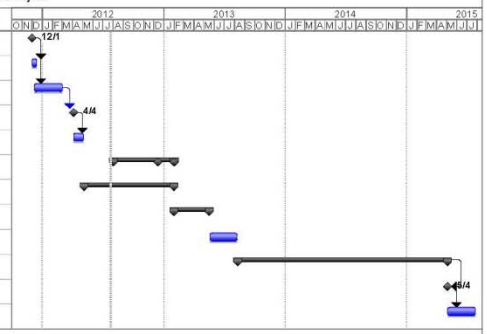
- Redirect I-95 drainage to the Pawtuxet River and modify two existing drainage manholes to prevent I-95 flooding to overtop.

The probable total capital cost for the levee improvements is estimated to be $4,000,000. Implementation of these improvements should be combined with the Phosphorus Removal Project as shown in Figure I-1. Because the levee improvements will be in part an earthen levee, the potential for re-use of the excavated material from the Phosphorus Removal Building for the levee is possible and would result in some cost savings. In addition, mobilization costs, and other administrative costs (both engineering & WSA) would be reduced. It is recommended that WSA pursue grant funding for the portion of this project that will be associated with the levee.

Figure I-1: Implementation Schedule

Warwick Sewer Authority
Phosphorus Removal Upgrade / Flood Protection
Implementation Schedule with Combined Construction Projects

ID	Task Name	Duration	Start	Finish	Predecessors
1	Submit Draft Facilities Plan Amendment	1 day	Thu 12/1/11	Thu 12/1/11	
2	Submit Intergovernmental Review Documents	10 days	Fri 12/2/11	Thu 12/15/11	1
3	RIDEM Review	3 mons	Fri 12/9/11	Thu 3/1/12	1
4	Public Hearing	1 day	Wed 4/4/12	Wed 4/4/12	3FS+15 days
5	Respond to Comments / Submit Final Facility Plan	22 days	Thu 4/5/12	Fri 5/4/12	4
6	Levee Design	131 days	Fri 8/3/12	Fri 2/1/13	
32	Phosphorus Removal Design	194 days	Mon 5/7/12	Fri 2/1/13	
64	RIDEM Review / Order of Approval	76 days	Fri 2/1/13	Fri 5/17/13	
73	Bidding Services + Contract Execution	60 days	Mon 5/20/13	Fri 8/9/13	72
74	Flood Protection & Mitigation and Phosphorus Removal Construction	451 days	Mon 8/12/13	Mon 5/4/15	
81	Substantial Completion	1 day	Mon 5/4/15	Mon 5/4/15	74FF
82	Punch List and Closeout	60 days	Tue 5/5/15	Mon 7/27/15	81



Project: Implementation Schedule
Date: Tue 7/24/12

Task	Milestone	External Tasks
Split	Summary	External Milestone
Progress	Project Summary	Deadline

II. INTRODUCTION

The Warwick Sewer Authority (WSA) owns and maintains a 7.7 mgd Advanced Wastewater Treatment Facility (WWTF) which discharges into the Pawtuxet River, a major tributary to Narragansett Bay. The original facility was constructed in 1965 and it has undergone several upgrades and modifications since then. The WSA also owns and maintains 48 wastewater pump stations and over 250 miles of sewers.

The WWTF is located in a meander of the Pawtuxet River. The river wraps around the WWTF on three sides, with Interstate Route 95 bordering the WWTF to the east (refer to Figure II-1). After repetitive flood damage to the WWTF in the 1960s and 1970s, the City constructed a flood levee in the mid-1980s to protect the WWTF from future damages. The City's Animal Shelter is also located within the confines of the levee. The levee was designed to protect to the 100-year flood level plus three feet of freeboard in accordance with Federal Emergency Management Agency (FEMA) accreditation requirements. For all but a few days per year, treatment plant effluent flows by gravity to the Pawtuxet River. When the river elevation is high, pumping is needed to convey the final effluent to the river.

Figure II-1: Aerial of Warwick WWTF

In March of 2010, record rainfall in Rhode Island caused the Pawtuxet River to crest to the highest levels ever recorded at the United States Geological Survey (USGS) gauge on the Pawtuxet River at the Warwick-Cranston line. Between March 12 and March 15, 2010, Warwick received 3.7 inches of rain. Warwick saw localized flooding throughout the collection system, the loss of three pump stations, and a new record high river level. Refer to Figure II-2. The WSA sustained approximately $50,000 in damages during this time period.

Figure II-2: Western Side of Levee at Warwick WWTF, March 15, 2010

Photo by Patrick Doyle

Less than two weeks later heavy rain began again. WSA activated its emergency plans on Sunday, March 28th based on forecasts of severe flooding. On Tuesday morning (March 30th), although it continued to rain hard, the National Oceanographic and Atmospheric Agency (NOAA) were still predicting Pawtuxet River elevations below the elevation of the WWTF's levee. During the course of that morning, the WWTF received flows that were five times the average, exceeding the WWTF peak flow rating. The facility experienced intermittent but frequent power losses. The storm drain system for Interstate Route 95, which runs through the WWTF, began backing up.

At approximately 1:15 pm, the Pawtuxet River breached the western side of the levee and began flooding the treatment facility. The water quickly filled up the approximately 18 acres located within the confines of the levee. Essential staff remained to remove and/or secure as many pieces of equipment and documents as possible but had to evacuate at about 1:45 pm on March 30th.

When the storm ended on March 31, 2010 the City had received a total of 8.8 inches of rain. The measured water level at the WWTF site was 31.5 NAVD (32.3 NGVD), approximately 4 feet above the top of the levee. The plant was flooded with over 10 feet of water in a matter of hours. The WWTF campus was filled with an estimated 75 million gallons of stormwater and wastewater. The flood also completely inundated six (6) pumping stations located along the banks of the Pawtuxet River. The month of March set two new records for river level and was the wettest month on record.

Figure II-3: Aerial Photo of Warwick WWTF, April, 2010

The primary objective of this project is to increase flood protection to help prevent a flood at the Warwick WWTF similar to that of the March 2010 event. This report will summarize work to date in that effort, including the analysis of the primary components of the flood protection system, including levee design, interior drainage, and groundwater seepage control in order to identify the necessary improvements to provide 500-year protection, as requested by the WSA. This report reviews design improvement alternatives for each flood protection component and provides a recommended preliminary design approach. Lastly, it reviews the required permits and includes an estimate of probable construction cost and an implementation schedule for the recommended flood control improvements.

III. EXISTING FLOOD PROTECTION SYSTEM

This section summarizes the existing flood protection system, which includes the levee, interior drainage system, and groundwater seepage control.

A. Levee

The main component of the flood protection system is the levee, which extends roughly 2,200 linear feet around the treatment facility to the north, south, and west. The levee's original crest elevation was 28.7 NAVD (29.5 NGVD). The current crest elevation varies from 27.3 to 29 NAVD (28.1 to 29.8 NGVD) and ties into the elevated access road and highway embankment adjacent to and east of the WWTF. The levee was originally designed to protect to the 100-year flood level, plus three feet of freeboard, in accordance with FEMA accreditation requirements.

The levee is constructed of rolled compacted fill, an impervious layer, and rip rap along the western boundary to protect the levee from damage. Along the northern and southern bounds, the side slope is 2:1 and transitions to a 3:1 slope on the western bound. The surface of the levee is mostly covered in grass.

The levee is penetrated at three locations for pipelines having 30, 48, and 18-inch diameters. I-95 protects the east side of site up to El.31.7 NAVD (32.5 NGVD).

B. Interior Drainage

The existing interior drainage system collects stormwater and groundwater from the toe drain system and transports it to the drainage junction chamber to the east of the effluent pump station by gravity. From there, the water can be directed by gravity through the levee to the river or, during times of high river levels, by gravity to the effluent pump wet well where it can be pumped to the river.

The system was designed to prevent flooding up to the 100-year storm. The 100-year storm used for design had a peak flow of 11.4 mgd according to the design files that were found.

Drainage swales from I-95 direct water that falls on the roadway onto the site into the wooded area adjacent to the access road. Water from this area is collected in the plant's interior drainage system. There is also a drainage pipe from I-95 that travels through the site and through the levee. There are two manholes on the site associated with this pipe.

C. Groundwater Seepage Control

Groundwater seepage under and through the levee is collected in the toe drain system on the inboard side of the levee. This water is transported to the plant drainage system at eight manhole locations. The toe drains consist of an area of compacted filter material surrounded by filter cloth. Within the filter material is a perforated 10-inch pipe that collects water and transports it to the drainage system.

Controlling groundwater through the toe drain system is important to the overall flood protection scheme as it alleviates high hydrostatic pressure within the levee. This high pressure can create instability and result in situations where water breaks out of the levee, carrying away soil and reducing the effectiveness of the levee. It should be noted that the existing toe drain system is designed to control groundwater seepage through the levee only to maintain the levee’s stability and overall integrity. It is not designed to prevent groundwater breakthrough throughout the site.

IV. GEOTECHNICAL EVALUATION

A. Review of Existing Information

AECOM has reviewed available subsurface information from prior investigations conducted at the site. These include the following:

- CE Maguire 1963 subsurface investigation for the original plant facilities
- CE Maguire June 1981 Flood Protection Facilities Report
- CE Maguire 1982 subsurface investigation for design of the flood control levees
- Paul B Aldinger & Associates Inc. (PBA) Geotechnical Data Report prepared in December 1999 for Beta Engineering in support of Contract 71A-D

In their 1981 Flood Protection Facilities Report, CE Maguire provided a Generalized Soil Profile based on a limited number of shallow borings and observation wells as follows:

- Loam, topsoil or fibrous peat from zero and two feet below grade
- Medium dense, fine to medium sand, some gravel, trace silt from 2 to 15 feet
- Medium dense, fine sand, some silt to end of borings (approximately 40 ft. below ground surface)

Based on their review of all of the subsurface information listed above, PBA described the following generalized subsurface conditions at the plant site in their 1999 Geotechnical Data Report:

- Topsoil/loam between zero and three feet below grade
- Granular Fill adjacent to existing structures which was likely placed as backfill following construction of existing facilities
- Outwash plain deposits consisting mostly of medium dense stratified sand and silt with various amounts of gravel underlying the topsoil or ground surface. The outwash plain deposits are relatively sandy to depths of up to 100 feet, and are comprised primarily of silt at greater depths. The deepest borings terminated in glacial till and the thickness of the outwash plain deposits ranges from 148.5 to 159.3 feet below grade.
- The maximum groundwater level is anticipated at 1.5 to 2 feet below ground surface.

B. Subsurface Investigation

AECOM conducted a supplementary geotechnical investigation to support the evaluation of alternatives and conceptual design for raising the flood levee. Four test borings were drilled to depths ranging from 22 to 52 feet. Of these, two were drilled from the levee crest to check that the make-up of the levees is consistent with CE Maguire 1983 design drawings and to investigate foundation conditions. A third test boring was drilled along the toe of the levee on the river side and the last boring within the plant to provide full cross sections for stability evaluation and seepage analyses for conceptual design. All boreholes were converted to observation wells to monitor the groundwater level. Grain size distribution tests were conducted on selected samples to check the visual soil descriptions included in the boring log. A Geotechnical Data Report presenting the information gathered during this investigation is included as Appendix A.

C. Findings and Recommendations

AECOM borings drilled within the levee did not encounter the 2.5 ft. thick layer of compacted impervious fill shown in the CE Maguire April 1983 Dike Cross Sections. The levee embankment was found to consist of medium dense to very dense sand and gravel. Beneath the levee, subsurface conditions are consistent with the generalized geologic profile described in PBA 1999 Geotechnical Data Report except that 10 feet of loose sands were encountered in boring B-2 immediately beneath the levee (i.e., approximately within the top 10 feet beneath original ground surface).

Overall, the levee and foundation soils are deemed to have adequate shear strength to allow for raising the dike up to 8 feet while maintaining the existing 2H:1V side slopes. Detailed stability analyses are recommended at the next design stage based on the following soil parameters:

1. Levee embankment: total unit weight of soil of 120 pounds per cubic foot (pcf); effective angle of friction of 34 degrees with zero cohesion.

2. Foundation soils: total unit weight of soil of 110 pcf; effective angle of friction of 32 degrees with zero cohesion.

The generalized soil profile from the 1981 CE Maguire report suggests that the sand within the top 15 feet beneath the levees is more gravelly and, hence, more pervious than the underlying more silty sand. This generalization is not supported by the soil's grain size distributions from CE Maguire laboratory gradation tests which show significantly higher percentage of fines and less gravel than indicated by the visual soil descriptions in their boring logs. AECOM encountered similar difficulties with field soil descriptions in the recent geotechnical investigation but corrected the boring logs to reflect the more accurate soil classifications from laboratory gradation tests. From all of the boring logs and gradation tests that are available, there is too much inconsistency in the percentage of silt or gravel in the sand to establish a simplified stratification as portrayed in the CE Maguire 1981 report. For seepage analysis at the conceptual level and based on these findings, AECOM modeled the outwash sand deposits as having a 100 foot depth with a hydraulic conductivity in the rage of .01 cm/sec. A more refined investigation is required to determine the composition and hydraulic conductivity of the outwash sand deposits for analysis and final design of drainage provisions to intercept seepage beneath the levees.

V. LEVEE AND GROUNDWATER SEEPAGE ALTERNATIVE DEVELOPMENT

This section addresses the levee and groundwater seepage together because these issues are closely connected. The section includes recommendations for levee height, levee alignment, levee design as well as modifications to the groundwater seepage system.

A. Levee Height

As a result of the March 2010 flooding, FEMA asked the USGS to re-compute the flood elevations in many watersheds in Rhode Island, including the Pawtuxet River. USGS is performing that work now and it is not yet complete. Thus, the 100- and 500-year flood elevations may change, which will have a fundamental effect on the design of the levee.

Top-of-levee elevation is one of the primary factors in determining levee alignment. Levee height directly impacts the levee geometry, along with the cost and scale of the project. The WSA has requested that the new flood protection system protect against the 500-year flood level. The following table provides key elevation data for the levee.

Table V-1: Key Levee Elevation Data

Pertinent Elevations	Feet, NAVD(1)	Feet, NGVD
Current 100-year flood elevation	26.3	27.1
Estimated 100-year flood elevation as reported by USGS(2)	27.7	28.5
Current 500-year flood elevation as reported by Kent County FIS	35	35.8
Estimated 500-year flood elevation as reported by USGS(2)(3)	33.1	33.9
Flood elevation during the March 2010 flood event	31.5	32.3
Minimum top-of-levee elevation (western crest)	27.3	28.1

Notes:

1. The elevations are reported in NAVD to be consistent with the results in the Kent County Flood Insurance Study and the results being reported by USGS. This datum is different than and should not be confused with NGVD. Many of the design records for the Warwick WWTF are in NGVD. NAVD 88 elevations were converted to NGVD 29 Datum by adding 0.83 to the NAVD 88 elevation.
2. The most recent estimates (September 2011) for the 100-year and 500-year events are based on a discharge estimate at the USGS gage 0116500 on the Pawtuxet River of 8,370 cfs and 14,000 cfs, respectively. This compares with the currently effective flood insurance study estimates of 6,650 and 19,600 cfs, respectively. The estimate from the USGS represents an increase in the 100-year discharge and elevation and a decrease in the 500-year discharge and elevation.
3. It is important to note that USGS's study is on-going, and their results could change. USGS has provided this information in the interest of helping WSA with its decision making but has clearly stated that its results are not final.

1. Potential Top-of-Levee Elevations

To meet FEMA certification requirements, the top-of-levee elevation must be 3-feet higher than the 100-year elevation. To satisfy this requirement, the top-of-levee would need to be at built at Elevation 30.7 NAVD (31.5 NGVD) or higher based on USGS's current projections. However, this height would not have protected the facility from the flood in March 2010. To protect against the flood-of-record and provide 3-feet of freeboard, the top of levee elevation would have to have been 34.5 NAVD (35.3 NGVD).

To build to the design criteria designated by the WSA (500-year), the top-of-levee elevation would be 33.1 NAVD (33.9 NGVD) or higher, based on the most recent USGS estimate. Any recommended freeboard would be added to this elevation. Potential freeboard options are reviewed below.

2. 500-year Freeboard Requirements:

Freeboard is a factor of safety that compensates for many unknown factors that could contribute to flood heights and often includes wave action. Wave action is not expected to be a major factor for this levee because of the limited fetch available for the wind to generate waves.

For FEMA accreditation purposes, there is no requirement to protect to the 500-year flood level. Therefore, as long as the top-of-levee is 3-feet above the 100-year, it will be certifiable. Building the levee top-of-levee elevation to the 500-year level is in of itself a factor of safety above and beyond FEMA's levee certification requirements.

Freeboard is only potentially required over and above the 500-year elevation if the facility is designated as a critical facility. To date, the WWTF has not been designated a critical facility, therefore any regulations pertinent to critical facilities do not yet apply.

It is unknown if the WWTF will be designated as a critical facility, and the guidance is ambiguous. Equally ambiguous are the requirements regarding freeboard should the WWTF be designated as a critical facility. The Glossary of Terms in "FEMA543, Design Guide for Improving Critical Facility Safety from Flooding and High Winds: Providing Protection to People and Buildings" states, "A freeboard of 1 to 3 feet is often applied to critical facilities". It appears that appropriate freeboard is based on site specific circumstances at the critical facility, subject to engineering judgment.

Therefore, any freeboard recommended is based on engineering judgment, not regulatory requirements.

3. Recommended Top-of-Levee Elevation

AECOM recommends that preliminary design of the top-of-levee be based on 35 NAVD (35.8 NGVD) for the following reasons:

- Meets FEMA levee accreditation requirements (100-year flood + 3' freeboard) and is well above the 100-year flood elevation (+7.3-feet).

- Provides over 3-feet of freeboard for the March 2010 flood-of-record (+3.5-feet).

- Provides almost 2-feet of freeboard above the current estimate of the 500-year flood level (El. 35.0 – El. 33.1 NAVD).

AECOM believes that minor adjustments to this elevation can be made at later stages of design should the facility be designated a critical facility or should USGS revise its 500-year flood elevation estimates upward.

B. Levee Alignment

The geometry and level of impact of the conceptual alignment alternatives presented below assume 35 NAVD (35.8 NGVD) to be the design elevation. This will result in approximately 6 to 7.7 feet of height added to the existing levee (existing top-of-levee elevation: 27.3 – 29 NAVD). In addition to the levee height, the location of existing and future facilities was considered when determining alignment feasibility. AECOM is currently working with the WSA to develop a Facility Plan that includes a conceptual design of future upgrades through the year 2030. At present, the conceptual design alternatives include possible upgrades or expansion in the area of the previously proposed North Final Clarifier, possibly in the area north of the Animal Shelter, and to the Primary Settling Tanks. Each levee alignment alternative was screened for its potential impact to these areas. Refer to Appendix B for a review of the various permits that could apply to these options.

The following sections provide an overview of three conceptual levee alignment alternatives to improve flood protection at the WWTF. Refer to Figure V-1 for a conceptual schematic of each alternative.

1. Levee Alignment Alternative 1 – Outward Expansion

Alternative 1 involves moving the levee alignment outward and away from the existing treatment facilities. This will allow for future expansion within the levee bounds and limit impacts to existing facilities.

Building on the outboard side of the existing levee will require construction activity within the wetland and floodplain located along the Pawtuxet River. The following regulatory requirements will be required for this design approach:

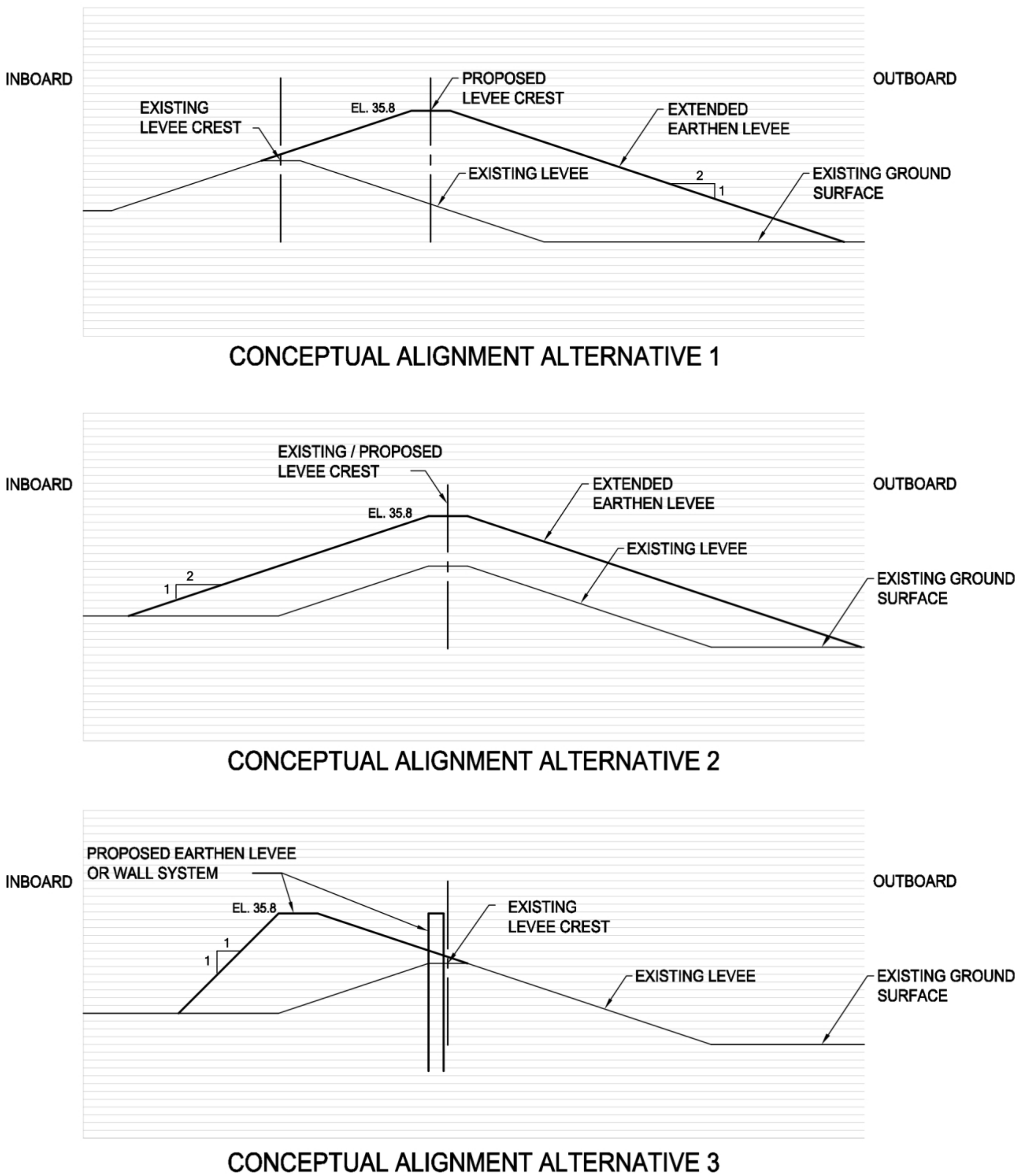
- **United Stated Army Corps of Engineers (USACE)**: Individual Permit for New Fill / Excavation Discharges

- **Rhode Island Department of Environmental Management (RIDEM)**: Application to Alter Freshwater Wetlands, Individual Water Quality Certification (WQC), Construction General Permit, and Rhode Island Pollutant Discharge Elimination System (RIPDES) Stormwater Discharge Associated with Industrial Activity (if modifications to drainage system)

- **City of Warwick**: Soil Erosion and Sediment Control Plan

In addition, compensatory wetlands and flood storage will likely be required due to the increased size of the levee eliminating floodplain area.

Figure V-1: Conceptual Levee Alignments



2. Levee Alignment Alternative 2 – Maintain Existing Levee Alignment

This alternative would maintain the existing alignment and earthen design. If the existing alignment and slopes are maintained, the footprint of the levee will increase approximately 22.5 feet on both the inboard and outboard sides of the northern and southern slopes (existing slope ~ 3H:1V) and approximately 15 feet on the inboard and outboard sides of the western slopes (existing slope ~ 2H:1V).

Based on the estimated footprint expansion, the following regulatory requirements have been identified:

- **United Stated Army Corps of Engineers (USACE)**: Individual Permit for New Fill / Excavation Discharges

- **Rhode Island Department of Environmental Management (RIDEM)**: Request for Preliminary Determination, or Application to Alter Freshwater Wetlands (depending on level of impact), Individual Water Quality Certification (WQC), Construction General Permit, and RIPDES Stormwater Discharge Associated with Industrial Activity (if modifications to drainage system)

- **City of Warwick**: Soil Erosion and Sediment Control Plan

AECOM referred to the Record Drawings to determine if expanding the levee footprint is feasible on the developed side of the levee. Significantly expanding the footprint on the western and southwestern sides of the property is not feasible because the existing treatment facilities and roadway are already located relatively close to the existing levee. The expanded footprint has minimal impacts in the northeastern portion of the property where there is undeveloped space and there are no known plans for future facilities.

In addition, compensatory wetlands and flood storage will likely be required due to the increased size of the levee eliminating floodplain area.

3. Levee Alignment Alternative 3 – Inward Expansion with Wall System

Alternative 3 involves shifting the alignment inward so that the levee improvements do not impact the surrounding wetland. The existing earthen levee design will be implemented where possible with an increased inboard slope to minimize expansion. In developed areas, primarily in the western and southern portions of the site, a sloped earthen design will not be feasible because the expanded footprint will encroach on existing and future facilities. In these areas, AECOM proposes the use of a temporary wall system, parapet wall, mechanically stabilized earth (MSE) such as a gabion wall, or wall / MSE combination to gain vertical height while minimizing horizontal expansion.

This design approach will shift the current levee alignment approximately 15 to 20 feet inward in areas where inboard expansion is possible. Where inboard expansion is not possible, the MSE or walled levee system will be located within the existing levee footprint.

The following minimum regulatory requirements have been identified for this design approach:

- **United Stated Army Corps of Engineers (USACE)**: Covered under RI General Permit for New Fill / Excavation Discharges Category 1 (non-reporting) or Category 2 (reporting) review process*

* Please note that the Category 1 and Category 2 review process is dependent on the extent of impact to wetland areas. If there is significant construction activity within the wetland, including staging areas or access roads, an Individual WQC and/or RIDEM Application to Alter Freshwater Wetlands may be required. This information would be determined during design.

- **Rhode Island Department of Environmental Management (RIDEM)**: Request for Preliminary Determination (insignificant alteration to freshwater wetlands), WQC conditionally granted under USACE Category 1 or 2 review process, Construction General Permit, and RIPDES Stormwater Discharge Associated with Industrial Activity (if modifications to drainage system)

- **City of Warwick**: Soil Erosion and Sediment Control Plan

Regulatory requirements are contingent on the impacts to the surrounding wetlands. At this time, it is uncertain whether this design approach will require the contractor to access the levee from the outboard side of the property. If outboard access is necessary approximately 0.8 acres of wetland will be disturbed due to construction activities.

4. Analysis and Recommendation

Of the three alignment alternatives discussed above, Alternative 3 - Inward Expansion with Wall System is recommended as AECOM believes that it is the most feasible to be implemented quickly. Outward expansion of the levee, as described in Alternative 1 and Alternative 2, present numerous regulatory hurdles and environmental impacts including permanent impacts to the wetland and the need for compensatory flood storage and wetland as noted below.

While technically feasible, Alignment Alternative 1 is not recommended due to the environmental impacts and the regulatory hurdles associated with the design. This Alternative would likely face a long permitting cycle before construction to determine where compensatory wetlands could be located, the amount needed to off-set construction, and the environmental impact due to construction in the flood plain.

Alternative 2 has similar environmental and regulatory impacts as Alternative 1. These impacts will be slightly less than Alternative 1 because half of the expansion will be on the in-board side of the levee.

Alignment Alternative 3 minimizes impact to the adjacent wetlands and minimizes expansion on the inboard side of the existing levee. This alignment will require the use of mechanically stabilized earth such as gabion walls or a walled flood protection system on the western and southern bounds so that the expanded footprint does not encroach upon existing or future facilities. These levee design details will be evaluated in subsequent sections.

C. Levee Design Alternatives

Using levee alignment Alternative 3, four design alternatives were developed for raising the levee to increase flood protection. Each levee design minimizes disturbance to the surrounding wetlands. The expansion of the levee footprint and permanent construction is limited to the inboard side of the existing levee crest. In addition to the “500-year” protection alternatives, AECOM has included a “100-year” protection option as a baseline alternative. This alternative represents the minimum improvements that should be implemented by WSA to meet FEMA accreditation requirements.

Once again, it is important to note that USGS’s study of flood elevations on the Pawtuxet River is on-going and their results could change. USGS has provided information in the interest of helping WSA with its decision making but has clearly stated that its results are not final. AECOM believes that minor adjustments to the levee elevation can be made at later stages of design based on the outcome of the USGS study.

Note that Alternatives 2 through 4 below both contain costs for improving the drainage from I-95. I-95 drainage along the southern property line is currently directed into two 18-inch storm drains that traverse beneath the WWTF grounds before discharging into the wetland west of the site. During the March 2010 storm events, stormwater flow exceeded the drainage system capacity and discharged through two manholes on the WWTF property. To prevent future discharge from this source, the manholes will be armored, raised, or a combination of the two. Armoring would involve strengthening the upper portion of the manhole with concrete to allow for pressurized conditions. By raising the manhole, the capacity is increased to prevent discharge.

1. Alternative 1 – 100 Year Earthen Levee

Alternative 1 involves elevating the levee to elevation 31.5 NGVD to meet FEMA accreditation requirements based on new 100 year flood elevations after the March 2010 storm. The existing earthen levee design will be maintained by extending the existing slope and reestablishing the 5-foot wide crest at elevation 31.5 NGVD. Although FEMA recommends a 10-foot wide crest, this alternative mimics the original design and maintains a 5-foot wide crest. Similar to the original design, the levee will consist of general fill, an impervious layer, and rip rap armor along the western boundary. The levee alignment will shift inward slightly. To minimize conflict with existing utilities, a gabion wall is proposed to be constructed along a portion of the inboard slope. The adjusted alignment and typical levee sections are shown Appendix C.

During construction, a temporary access road along the outboard side of the levee may be required where existing facilities limit access along the inboard slope. The proposed 25-foot wide access road will result in approximately 0.8 acres of temporary disturbance.

Please note that Alternative 1 would not have provided adequate protection during the March 2010 storm, which had a flood elevation of 31.5 NAVD (32.3 NGVD). AECOM considered incorporating an additional temporary option that could be implemented by WSA if a major storm event is predicted such as a Hesco Wall, PortaDam, or Muscle Wall. These systems can potentially provide an additional 5 to 10 feet of height. However, during preliminary analysis, this approach proved to be too difficult to implement at the Warwick WWTF and therefore is not considered to be a feasible option. Due to the length of the levee, installing the temporary system would require a major mobilization of staff and equipment prior to the predicted storm event. In addition, WSA would have to arrange to store the system on site or keep the temporary system in place permanently once installed.

2. Alternative 2 – 500 Year Earthen Levee

Alternative 2 involves elevating the earthen levee to elevation 35 NAVD (35.8 NGVD) to provide 500-year protection. The existing earthen levee design will be maintained by extending the existing slope and reestablishing the 5-foot wide crest at elevation 35 NAVD (35.8 NGVD). Although FEMA recommends a 10-foot wide crest, this alternative mimics the original design and maintains a 5-foot wide crest. A 2H:1V slope will be established on the inboard slope. A minimum 5-foot clearance is proposed between the expanded levee footprint and existing facilities. This will result in the need for extended stretches of gabion walls. Similar to Alternative 1, a temporary access road along the outboard side of the levee may be required where existing facilities limit access. The adjusted alignment and typical sections are presented in Appendix C.

This option will consist of the placement of general fill, an impervious layer, and rip rap armor along the western boundary. Testing will be conducted during design to verify the presence of the impervious layer. If the original impervious layer is not encountered remedial steps such as replacement of the layer or placement of a new layer will be designed to provide for an appropriate cutoff to seepage. No remedial steps are planned at this time because the existing levee has worked as designed.

The increased footprint will impose extra lateral pressure on the adjacent foundations and underground structures. The record structural drawings indicate that the South Pump Station and Clarifiers are surrounded by 2.5-feet wide reinforced concrete secant walls extending 47.5-feet below the ground surface, while the North Clarifier has a 25-feet deep mat foundation with a 1.33-feet thick wall. This increased pressure and stability of the foundations and underground walls needs to be further evaluated by a structural engineer during final design. If the increase in lateral pressure requires additional stability, it would likely be mitigated through the installation of a retaining wall or other means to distribute pressure on structures.

It should also be noted that the existing conditions survey shows a portion of the I-95 boundary below the 500-year flood elevation. To provide full protection around the entire WWTF perimeter, AECOM recommends installing a floodwall to elevation 35 NAVD (35.8 NGVD) along the I-95 boundary, as shown on Appendix C. A sheet pile floodwall is proposed in this area to minimize the levee footprint and impact along I-95. This floodwall system is further described in Alternative 3. A catch basin and drainage pipe would also be required to collect runoff that collects behind the proposed wall. The pipe would direct flow under Arthur W. Devine Boulevard and into the adjacent wetland. To prevent backflow, a duckbill valve would be installed on the outlet.

3. Alternative 3 – 500 Year Floodwall

Alternative 3 involves the installation of a flood wall driven into the existing earthen levee with a top of wall elevation of 35 NAVD (35.8 NGVD) to provide 500-year protection. AECOM proposes the use of vinyl sheet pile for this flood wall system. The feasibility of both concrete and vinyl sheet pile walls was considered. Vinyl is a less expensive alternative than concrete ‘I-type’ walls but is typically limited by height restrictions of approximately 15 to 20 feet above grade. The maximum height requirement at the WWTF, 7.7 feet, is within the limitations of the vinyl sheet pile system. Figure V-2 shows an example of a vinyl sheet pile wall. Note that vinyl sheet piles also come in more aesthetically pleasing alternatives. The details of this system would be decided during final design should it be chosen as the design approach.

Figure V-2: Example Vinyl Sheet Pile Wall

It is anticipated that the sheet pile will be driven through the center of the existing levee crest to a depth of approximately 14. Portions of the existing earthen levee disturbed during construction will require restoration. Narrow portions of the existing levee may need to be widened and rip rap removed to allow the contractor to drive the piles. Testing will be conducted during design to verify the presence of the impervious layer. If the original impervious layer is not encountered remedial steps such as replacement of the layer or placement of a new layer will be designed to provide an appropriate cutoff to seepage. No remedial steps are planned at this time because the existing levee has worked as designed. These areas would be restored after the wall is installed. These areas will remain loam or rip-rap to match the existing design. In areas where there are utility crossings below the levee, the vinyl sheets will be installed so as not to impact the utility lines and cut to match adjacent piles at the top, which is common practice for vinyl sheet pile walls. The structural impact of installation to adjacent facilities is considered negligible. The proposed alignment and typical sections are shown in Appendix C.

Similar to Alternative 2, AECOM recommends installing a floodwall to elevation 35 NAVD (35.8 NGVD) along the I-95 boundary and stormwater drainage, as shown in Appendix C to provide full protection around the entire WWTF perimeter. No wetland compensation is necessary with this alternative.

4. Alternative 4 – 500 Year Earthen Levee & Floodwall Combination

Alternative 4 combines Alternative 2 and 3 and involves both elevating the earthen levee to elevation 35 NAVD (35.8 NGVD) and a flood wall driven into the existing earthen levee with a top of wall elevation of 35 NAVD (35.8 NGVD) to provide 500-year protection. The extended stretches of gabion walls required under Alternative 2 have been replaced in this alternative with vinyl flood wall described in Alternative 3. The proposed alignment and typical sections are presented in Appendix C. This alternative eliminates the impact the secondary clarifiers and maximizes space for future structures while maximizing the amount of the less expensive earthen levee.

Similar to Alternatives 2 and 3, AECOM recommends installing a floodwall to elevation 35 NAVD (35.8 NGVD) along the I-95 boundary and stormwater drainage, as shown in Appendix C to provide full protection around the entire WWTF perimeter.

5. Analysis and Recommendation

The estimated levee construction costs are presented below in Table V-2 along with various qualitative evaluation criteria. An itemized breakdown of the cost of each levee alternative is presented in Appendix D

Table V-2: Levee Design Alternative Evaluation

	Alternative 1 – 100 Year Earthen Levee	Alternative 2 – 500 year Earthen Levee	Alternative 3 – 500 Year Floodwall	Alternative 4 – 500 Year Earthen Levee & Floodwall Combination
Estimated Construction Cost (Levee Only)	$330,000	$2,200,000	$3,500,000	$2,600,000
Benefit:Cost Analysis	Benefits greater than cost. Beneficial to grant funding.	Benefits likely greater than cost.	Benefits likely greater than cost.	Benefits likely greater than cost.
Flood Protection	Protects to 100-year storm plus 3 feet of free board to comply with FEMA certification guidelines. Does not protect to March 2010 flood level	Protects to 500-year storm plus approximately 2 feet of free board. Exceeds FEMA certification guidelines. Provides 3.5 feet of protection on top of the March 2010 flood level.	Protects to 500-year storm plus approximately 2 feet of free board. Exceeds FEMA certification guidelines. Provides 3.5 feet of protection on top of the March 2010 flood level.	Protects to 500-year storm plus approximately 2 feet of free board. Exceeds FEMA certification guidelines. Provides 3.5 feet of protection on top of the March 2010 flood level.
Impact to WWTF	Reduces access around Secondary Clarifiers.	Reduces access and adds structural load around Secondary Clarifiers. Structural impact will need to be evaluated during design. Future construction of Secondary Clarifier #6 will be more challenging.	No impact to WWTF.	Limits access to the little used southern and northern sides of plant.
Construction Risk	Low. Location of work should not have large impact on existing structures.	Low to Moderate. Small risk of damage to existing structures due to proximity.	Moderate. Potential for encountering soils unsuitable for vinyl sheeting.	Moderate. Potential for encountering soils unsuitable for vinyl sheeting.
Coordination with Phosphorus Removal Project	Likely more cost effective to combine projects since excavation from new structure may be able to be used for levee.	Likely more cost effective to combine projects since excavation from new structure may be able to be used for levee.	Combining projects could result in lower overall cost to WSA due to mainly savings on General Conditions.	Combining projects could result in lower overall cost to WSA due to mainly savings on General Conditions.
Aesthetics	Views out of the WWTF site will be reduced by approximately 1.5 feet	Views out of the WWTF site will be reduced by over 7 feet due to higher levee	Views out of the WWTF site will be reduced by over 7 feet due to higher levee.	Views out of the WWTF site will be reduced by over 7 feet due to higher levee.

	Alternative 1 – 100 Year Earthen Levee	Alternative 2 – 500 year Earthen Levee	Alternative 3 – 500 Year Floodwall	Alternative 4 – 500 Year Earthen Levee & Floodwall Combination
	due to higher levee height. Combination of earth levee and gabion walls could negatively impact aesthetics on site.	height. Combination of earth levee and gabion walls could negatively impact aesthetics on site.	Vinyl sheeting wall over entire length of is expected to be the most aesthetically pleasing alternative.	Combination of earth levee and vinyl sheeting wall is expected to be an aesthetically pleasing alternative.

Alternative 1 meets the requirements of FEMA certification but does not provide protection from a flood comparable to that which occurred in March 2010 and is therefore not recommended. Alternative 1 represents the minimum WSA would have to spend to meet FEMA accreditation requirements based on the new 100 year flood level. Alternatives 2, 3, and 4 meet the requirements for FEMA certification and protects to the 500-year flood and the March 2010 flood of record. Alternative 2 is $400,000 less expensive than Alternative 4 and $1,200,000 less expensive than Alternative 3. However, it requires a larger footprint of the levee which will reduce the limited space on site and place increased lateral pressure on existing structures. A structural analysis of these structures would need to be completed to find out if any modifications would need to be made. It also makes construction of the sixth secondary clarifier more difficult and more expensive. Alternative 3 has a very small impact to the site access and onsite structures, provides the most aesthetically pleasing product, and can be implemented more quickly than Alternative 2. Although Alternative 4 decreases inboard space along the south and north sides of the plant, it minimizes the impact to the secondary clarifiers along the west side and eliminates any concerns about additional structural load on existing structures and constructability of the sixth secondary clarifier. Additionally, it can be more aesthetically pleasing and is only 18% more expensive than Alternative 2.

After considering the identified advantages, disadvantages, and estimated construction cost, AECOM recommends moving forward with Alternative 4 – 500 Year Earthen Levee & Flood Wall Combination. Alternative 4 provides a cost effective means to increase the flood protection at the WWTF to withstand the March 2010 flood and 500-year flood while also preserving inboard space, reducing structural impacts to existing structures, and eliminating constructability concerns and preserving space for the sixth secondary clarifier. Although it provides the same level of protection as Alternative 4, it is AECOM's opinion that the construction cost savings afforded with Alternative 2 are out-weighed by the qualitative benefits of Alternative 4.

D. Groundwater Seepage Control Alternatives

There are three modes of levee failure: overtopping, piping failure due to seepage, and breakthrough of groundwater on the dry side of the levee during an extended period of high floodwaters. Protection against overtopping is being addressed by raising the existing levee. Groundwater seepage through the existing levee is controlled by an existing toe drain system. The toe drains are intended to control seepage from channeling or piping through the levee and weakening its integrity and stability. It does not appear that specific measures were put in place during the construction of the original levee to protect against breakthrough other than what would be captured by the toe drain system.

Preliminary groundwater modeling was utilized to estimate flow within the toe drain system under the levee. As part of the model, the 500-year levee design was utilized to contain a 500-year flood elevation of 33.1 NAVD (33.9 NGVD). The model yielded both preliminary drain flow rates and the approximate day at which groundwater would "breakthrough" the ground surface on the inboard side of the levee, at which time groundwater would directly contribute to flooding at the WWTF.

It should be noted that this model assumes that the river elevation is held at a constant elevation, 33.1 NAVD (33.9 NGVD), for the entire flood period. Therefore, the breakthrough estimates are conservative. The model runs performed were developed to provide a basis for conceptual design, as well as to determine primary data collection needs. Based on the sensitivity results, the primary data needs include soil hydraulic conductivity and historic storm/flood timelines with which to establish design-basis assumptions during modeling.

In addition to modeling the existing toe drain system, the model was run with a new drain installed 5-feet below the inboard ground surface elevation and run with a new drain installed 8-feet below the ground surface elevation. A scenario with 60-feet deep wells installed every 30 feet along the toe drain to direct additional seepage into the drain system was also considered. Appendix E includes additional details on the groundwater modeling completed for this analysis. Table V-3 shows the resulting flow rates with the addition of the new drain and wells.

Table V-3: Seepage Analysis

Scenario	Basic Description	Day of "break through" at WWTF	Drain Flow @ 0.5 Days (gpm)	Drain Flow Prior to "breakthrough" (gpm)
1	Existing Toe Drain – 2 ft. deep	Day 2	780	1,060
2	Toe Drain – 5 ft. deep	Day 5	1,769	1,646
3	Toe Drain – 8 ft. deep	Day 7	2,303	1,831
4	Toe Drain – 5 ft. deep; install wells every 30 ft., 60 feet deep	> 7 Days	3,244	2,834

Because of the preliminary nature of the model, further data and analysis is needed to determine whether or not groundwater control improvements beyond the existing toe drain system are necessary. It is possible that the flooding from the existing toe drain system is manageable. However, if groundwater control improvements are necessary, Table V-3 demonstrates that a new 5-foot deep toe drain system with wells installed every 30 feet shown in Scenario 4 represents the most effective method for capturing seepage. However, it is also the most expensive. The existing toe drain system is approximately 1,800 feet in length, which would result in the need for approximately 60 wells. Depending on well diameter, the cost of the well system alone is estimated to be between $720,000 and $1,440,000. This cost does not include installing the toe drain at a lower depth or upgrading system capacity. This scenario was therefore excluded due to cost.

The remaining three scenarios demonstrate that breakthrough will be delayed as the toe drain is moved deeper. Similar to Scenario 4, Scenario 3 will prevent groundwater from breaking through for approximately 7 days. This provides more protection than the existing toe drain system, which would allow breakthrough within 2 days. The estimated cost of the toe drain installed at 8 feet is approximately $300,000.

At this time, AECOM recommends making no changes to the existing toe drain system other than extending the clean outs to the grade level of the raised levee. The existing system was designed for the 100-year flood and has functioned without incident since construction. Preliminary modeling suggests that the existing toe drain system continues to provide seepage control and “breakthrough” prevention during the 500-year flood. Maintaining the location of the existing toe drain system as the levee is raised and moved inward will position it in a more central location below the levee and will improve seepage and piping control.

Additional data will be collected and analysis conducted during the next stages of the project to get better input data for the model and determine if additional groundwater control systems are necessary. At this time, AECOM recommends carrying the cost of installing a deeper drain system that would operate independently of the toe drain system to prevent breakthrough throughout the site. If necessary and as currently envisioned, this system would be deeper than the existing toe drain and only operate during severe storm events. During severe storm events, the groundwater would be transported to a wet well where portable pumps could be used to remove the water from the site. An additional, permanent pump station for groundwater control requiring annual exercise and maintenance is not recommended. AECOM recommends carrying $300,000 for a deep groundwater drain system as contingency in case subsequent results show that the existing toe drain is not adequate.

Moving forward, the proposed model refinements include the following:

- Expand model grid extents to limit influence of no-flow model boundaries;
- Review model in the area of the highway and how water may potentially enter area outside of drain influence (potentially add drains in this area);
- Adjust ground surface elevations in WWTF. This will actually provide additional water storage;
- Account for reduced storage below ground where tanks/foundations exist;
- Adjust existing drain location and elevations for accuracy;
- Apply any field data collected to adjust hydraulic conductivity;
- Rather than applying a continuous new drain, model a realistic scenario which likely is constructed in sections; and
- Review historic storm and flood timeline data, as well as precipitation data, to refine in model.

VI. INTERIOR DRAINAGE EVALUATION

This section addresses the interior drainage system designed to minimize flooding from direct rainfall on the WWTF grounds.

A. Computer Simulations

To evaluate the interior drainage system, three design events, the 10-, 100-, and 500-year design storms, were simulated along with historic events from October 2005 and March 2010 using the U.S. Army Corps of Engineer's HEC-HMS Rainfall-Runoff computer simulation model. Rainfall statistics for these events are presented in Table VI-1 and Table VI-2. Since the levee was built there have been no 100-year 24-hour events. Detailed results and discussion of these preliminary model simulations are included in Appendix F.

Table VI-1: Rainfall Characteristics

Event	Maximum 1-hour rainfall (inches)	Maximum 3-hour rainfall (inches)	Maximum 12-hour rainfall (inches)	Maximum 24-hour rainfall (inches)	Maximum 2-day rainfall (inches)
October 2005	0.82	2.09	5.75	6.38	6.38
March 2010	0.55	1.46	4.31	6.86	8.83
10-year	1.51	2.57	3.88	4.73	5.12
100-year	2.65	4.63	6.91	8.31	9.21
500-year	3.95	6.99	10.34	12.23	13.94

Table VI-2: Peak Discharges and Storm Volumes

Storm	Peak Discharge		Storm Volume	
	cfs	mgd	acre-feet	million gallons
October 2005	13.3	8.6	6.4	2.1
March 2010	8.4	5.4	9.9	3.2
10-year	17.7	11.4	2.2	0.7
100-year	44.0	28.4	5.7	1.9
500-year	79.9	51.6	10.2	3.3

The October 2005 event was simulated as a check on modeling results. Prior to March 2010, the October 2005 storm was the largest on record since the levee was built. The model simulation of the October 2005 storm resulted in no flooding, consistent with what actually happened. The March 2010 event was simulated to see if there would have been any interior flooding on site had the levee not overtopped and the I-95 storm drain did not surcharge. The March 2010 simulation also demonstrated that there would have been no interior flooding if the levee did not overtop based on pump station capacity.

The model was then used to evaluate the ability of the existing effluent pump station to transport and process the plant's maximum day wastewater flow and the storm discharge under current and future situations for these two historical storms as well as the 10-, 100-, and 500-year storm events. The results of this evaluation are presented below in Table VI-3. As seen in the table, the pump station is adequately sized for both the October 2005 and March 2010 storms during either the present or future maximum day wastewater flows. The pump station is adequate to handle the 10-year storm under present day conditions however under 2030 conditions the firm capacity, i.e., the capacity with one pump out of service, will be exceeded. If all pumps remain operable during this scenario however, there will be no flooding. The capacity of the pump station is exceeded during the 100-year and 500-year storm events during present and future day scenarios.

Table VI-3: Pump Station Capacity during Various Storm Scenarios

Storm	Plant Design	Maximum Daily Wastewater Flow (MGD)	Peak Storm Discharge (MGD) (including 2 MGD toe drainage)	Peak Inflow to Pumping Station (MGD)	Firm Pump Station Capacity at 25' TDH (MGD)	Total Pump Station Capacity at 25' TDH (MGD)
March 2010	Present Day	8.7	7.4	16.1	24	32
March 2010	Future (2030)	13.3	7.4	20.7	24	32
October 2005	Present Day	8.7	10.6	19.3	24	32
October 2005	Future (2030)	13.3	10.6	23.9	24	32
10-year	Present Day	8.7	13.4	22.1	24	32
10-year	Future (2030)	13.3	13.4	26.7	24	32
100-year	Present Day	8.7	30.4	39.1	24	32
100-year	Future (2030)	13.3	30.4	43.7	24	32
500-year	Present Day	8.7	53.6	62.3	24	32
500-year	Future (2030)	13.3	53.6	66.9	24	32

It is important, however, not to lose sight of the length of time that the pump station is overwhelmed. The peak discharge rates of these design storms do not last long and as shown in Table VI-4 the length of time that the pump station is overwhelmed is relatively short and does not necessarily result in large volumes of flooding. For example, for the 10-year 2030 condition, the flow rate exceeds the firm pump station capacity for only 19 minutes, and the flood volume is relatively insignificant. For the 100-year design storm, the volume and duration of flooding is still relatively minor. Only under 500-year design storm conditions does the flooding become more significant if one pump is out of service. The extent and depth of flooding (in feet) under the 100-year storm events are shown in Figure VI-1 through Figure VI-4.

Table VI-4: Flood Volumes (acre-feet) and Durations (minutes) for Design Storms

Results Assuming Firm Pump Capacity (24 mgd – one pump out of service)				
	Present		Future (2030)	
	Volume (acre-feet)	Duration (minutes)	Volume (acre-feet)	Duration (minutes)
10-year	-	-	0.07 ac-ft.	19 minutes
100-year	0.71 ac-ft.	40 minutes	1.2 ac-ft.	61 minutes
500-year	2.48 ac-ft.	71 minutes	3.38 ac-ft.	116 minutes
Results Assuming Total Pump Capacity (32 mgd – all pumps operational)				
	Present		Future (2030)	
	Volume (acre-feet)	Duration (minutes)	Volume (acre-feet)	Duration (minutes)
10-year	-	-	-	-
100-year	0.21 ac-ft.	22 minutes	0.47 ac-ft.	32 minutes
500-year	0.86 ac-ft.	31 minutes	1.21 ac-ft.	39 minutes

Figure VI-1: 100-year Present Day Flood Area – Firm Capacity

Figure VI-2: 100-year Present Day Flood Area – Total Capacity

Figure VI-3: 100-year Future Day Flood Area – Firm Capacity

Figure VI-4: 100-year Future Day Flood Area – Total Capacity

B. Conclusions and Recommendations

Based on the results of the modeling, the existing interior drainage system cannot contain the 100-year storm event rainfall inside the levee without flooding an area near the chlorine contact tanks because the pump station capacity is exceeded. It is likely that the 100-year storm event changed since the storm drain system was installed. Based on the results shown in Table VI-4 and Figure VI-1 through Figure VI-4, the flooding is considered minor and of short duration. The duration and depth of flooding will increase in the future under anticipated 2030 design conditions, but will still be minor.

AECOM recommends that the interior drainage system be accredited for a 100-year storm. AECOM does not believe that the benefits of protecting against a 500-year storm for the interior drainage system justify the cost of larger infrastructure of piping and pumps. The 500-year event is extremely rare and the volume and duration from this storm inside the levee is similar in volume and duration as the 100-year event provided all pumps in the effluent pump station are operational. Additionally, the 100-year design level is consistent with FEMA's regulations for accreditation provided the flooded area is properly mapped.

The WSA has three viable options regarding interior drainage design and FEMA accreditation of the interior drainage system:

- WSA can choose not to seek accreditation – The site will remain accredited until FEMA revokes the accreditation. At that time, flood insurance may be required on buildings inside the levee on the northwest portion of the site. Under this option, no improvements (beyond those required to accommodate the floodwall footprint and I-95 drainage) to the interior drainage system are needed. The interior drainage system has served the site adequately for 27 years.

- WSA can make no drainage system improvements and seek accreditation by showing the extent of anticipated 100-year flooding – The floodplain inside the levee would need to be shown, small improvements may be needed to protect existing structures, and buildings within the floodplain may be required to purchase flood insurance if no improvements are made. Like the first option, no drainage systems improvements beyond the floodwall footprint and I-95 drainage improvements would be required. Model results show that under 100-year present conditions, the 100-year flood volume is .21 acre-feet and the flood duration is 22 minutes. The approximate floodplain associated with this is located in a relatively small area on the northwest portion of the site near the effluent pump station, and has a maximum depth less than one foot. This floodplain is based on all pumps in service at the pump station.

- WSA can improve the drainage system to eliminate interior flooding from the 100-year event, in which case no flood insurance would be required. This could be accomplished by on-site storage, increased pumping capacity, or a combination of both.

Large capital expenditures to prevent interior drainage flooding during a storm event by adding storage or increasing pumping capacity are not recommended because of the short duration of the flooding and shallow depths. AECOM recommends conducting more extensive hydrologic / hydraulic modeling prior to final design to better define the onsite flooding resulting from the 100-year storm event. The mapping of this interior flooding will need to be included with the FEMA certification for the new levee system. Once the extent of flooding is better defined, flood protection measures can be finalized and included in the final design. These flood protection measure likely include raising the height of the walls on the chlorine contact tanks as required by TR-16 standards. AECOM recommends carrying $100,000 for construction for this task. Based on the interior drainage evaluation, AECOM recommends that:

1. The I-95 drainage swales be redirected to outside of the levee thereby reducing the amount of drainage the internal plant drainage system will need to address;

2. That the piped I-95 drainage system located on the eastern side of the plant property be modified to exclude highway drainage from overtopping two existing manholes within the plant site;

3. That flooding be allowed inside the plant site and that mitigation measures for the flooding be implemented.

VII. RECOMMENDED APPROACH

A. Capital Cost

Table VII-1 presents the capital costs for the flood protection alternatives considered. The values for groundwater seepage and interior drainage remain the same across all alternatives because they apply to all alternatives. The percentages used to estimate professional services and program contingency vary depending on the option and the perceived risk associated with that option.

Table VII-1: Capital Cost Estimates

	Alternative 1 - 100 Year Earthen Levee	Alternative 2 - 500 Year Earthen Levee	Alternative 3 - 500 Year Floodwall	Alternative 4 - 500 Year Earthen Levee & Floodwall Combination
Estimated Construction Cost (2012)				
Levee	$330,000	$2,200,000	$3,500,000	$2,600,000
Groundwater Seepage	$0	$300,000	$300,000	$300,000
Interior Drainage	$100,000	$100,000	$100,000	$100,000
Total Estimated Construction Cost (2012	$430,000	$2,600,000	$3,900,000	$3,000,000
Professional Services & Program Contingency	$220,000	$780,000	$975,000	$900,000
Total Capital Cost (2012)	$650,000	$3,380,000	$4,875,000	$3,900,000
Escalated Capital Cost (2013)	$670,000	$3,490,000	$5,030,000	$4,020,000

B. Levee and Groundwater Seepage

1. Levee

AECOM recommends that preliminary design of the top-of-levee be based on 35 NAVD (35.8 NGVD) and that the levee alignment be moved inward to prevent disturbance to the floodplain and avoid regulatory and permitting challenges.

AECOM recommends moving forward with a 500 Year Earthen Levee and Flood Wall Combination. Alternative 4 provides a cost effective means to increase the flood protection at the WWTF to withstand the March 2010 flood and 500-year flood while also preserving inboard space, reducing structural impacts to existing structures, and eliminating constructability concerns and preserving space for the sixth secondary clarifier, and providing repair access for the main plant wastewater pipeline. Although it provides the same level of protection as Alternative 4, it is AECOM’s opinion that the construction cost savings afforded with Alternative 2 are out-weighed by the qualitative benefits of Alternative 4.

2. Groundwater Seepage

At this time, AECOM recommends making no changes to the existing toe drain system other than extending the clean outs to the grade level of the raised levee. This system was designed for the 100-year flood and has functioned without incident since construction.

AECOM also recommends carrying the cost of installing a deeper drain system that would operate independently of the toe drain system to prevent breakthrough throughout the site. If necessary and as currently envisioned, this system would be deeper than the existing toe drain and only operate during severe storm events.

C. Interior Drainage System

AECOM recommends that the interior drainage system be designed for the 100-year storm. This recommendation is made in part because AECOM does not believe that the benefits of protecting against a 500-year storm for the interior drainage system justify the cost. The 500-year event is extremely rare (a 0.2% chance of occurring in any given year), and the volume and duration from this storm inside the levee is similar in volume and duration as the 100-year event provided all pumps in the effluent pump station are operational. Additionally, the 100-year design level is consistent with FEMA's regulations for accreditation provided the flooded area in properly mapped.

Hydraulic modeling showed that the existing stormwater system is somewhat undersized for the 100-year storm event. Upgrading the existing system to a 100-year storm level through large capital expenditures such as adding storage or increasing pumping capacity for the small chance and small time frame of flooding is not recommended. Based on the interior drainage evaluation, AECOM recommends that:

1. The I-95 drainage swales be redirected to outside of the levee thereby reducing the amount of drainage the internal plant drainage system will need to address;

2. That the piped I-95 drainage system located on the eastern side of the plant property be modified to exclude highway drainage from overtopping two existing manholes within the plant site;

3. That flooding be allowed inside the plant site and that mitigation measures for the flooding be implemented.

D. Next Steps

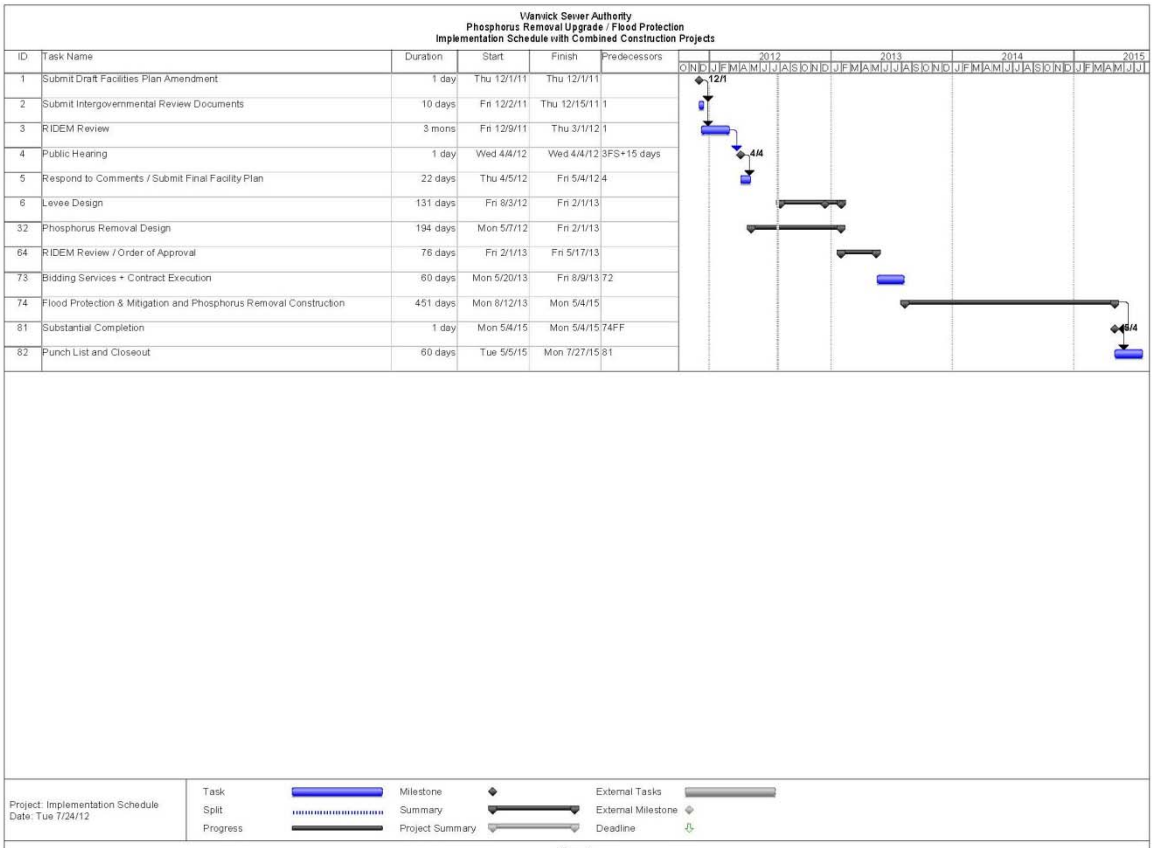
After completion of this report and as part of the design phase, an additional geotechnical evaluation will be conducted to refine the existing groundwater model to further quantify flow under the levee and through the toe drain system to determine if a deeper drain system is necessary. Additional analysis is also necessary to refine the extent of flooding from the 100-year storm due to the interior drainage system being slightly undersized for FEMA certification purposes. Lastly, upon completion of the USGS study, a final levee height must be decided upon and the levee improvements designed.

E. Implementation Schedule

The proposed implementation schedule is shown below. AECOM recommends bundling this project with the Phosphorus Removal Project as it will likely reduce the overall cost to the WSA through a savings on the contractor's General Conditions and the ability to reuse excavated material as part of the levee. It is expected that the specifications will require a levee completion date well before the completion of the Phosphorus Removal Project so that it is protected against any flooding while under construction.

Although the completion of this report will lag the beginning of the design of the Phosphorus Removal Project, there will not be a delay to the Phosphorus Removal Project because the flood protection design work is less complicated and has a shorter design timeline.

Figure VII-1: Implementation Schedule



F. Benefit Cost (B/C) Ratio and Funding Opportunities

This project is eligible for funding under the Mitigation Grant Program administered by FEMA. To evaluate eligibility, FEMA looks at the benefit to cost ratio – a benefit to cost ratio greater than one indicates that a project is cost effective and would benefit from additional flood protection, and would therefore be eligible. Many factors affect the outcome when determining this ratio including how long the plant was out of service, cost of repair, critical facility designation, number of people serviced, and many other factors. Also, the determination of this ratio is subject to interpretation and acceptance of assumptions that go into the model including estimates of time out of service, current and future damage costs.

AECOM believes that the B/C ratio is greater than one based on our interpretation of the FEMA requirements and recommend that this avenue of funding be pursued for this project.

Appendix A
Geotechnical Data Report

AECOM
701 Edgewater Drive
Wakefield, MA 01880

www.aecom.com

781-246-5200 tel
781-224-6546 fax

Memorandum

File:	60219451
Date:	March 29, 2012
To:	Doug Gove
From:	Geotechnical Department
Subject:	**Geotechnical Data Report (GDR)** **Warwick Wastewater Treatment Plant Dike Improvement** **Warwick, RI**
Discipline:	Geotechnical
Copies:	J. Ramos; M. Washington

INTRODUCTION

A subsurface exploration program was conducted to support the evaluation and design of Warwick Wastewater Treatment Plant Dike Improvement project in Warwick, RI. The exploration program consisted of drilling and sampling four (4) test borings, and converting all the test borings to observation wells. The test borings were drilled by New England Boring Contractors of Glastonbury, CT. The work was performed from August 31 to September 2, 2011.

EXPLORATION PROGRAM

The four test borings ranged in depth from 22 to 52 feet and were drilled using 4-inch HW casing. Split-spoon soil samples were collected at ground surface and at 5-foot intervals thereafter to the termination of boreholes. An AECOM representative was present to log and collect each split-spoon sample.

All four test borings were converted to monitoring wells at approximately 20 ft depth below the ground surface. The monitoring wells were constructed using 2-inch diameter Schedule 40 screen and riser, a sand pack and bentonite seal, and flushmount roadway protective boxes or a standup steel protective pipe that were concreted into place.

The test boring locations are shown on the attached site plan. Geologic logs from the exploration program are provided in Attachment 1.

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GROUNDWATER MEASUREMENTS

Depth to groundwater table was measured during drilling and noted on the boring logs. Table 1 shows the monitoring records of all observation well readings. Groundwater levels may fluctuate with precipitation, season, construction activities, run-off controls, and other factors. As a result, water levels during construction may vary from those observed during the subsurface investigation.

Table 1 Groundwater Reading Table (Depth in ft from the ground surface.)

Reading Date \ Well No.	B-1	B-2	B-3	B-4
9/9/2011				2.71
9/12/2011	11.9	0.5	11.6	3.7
9/19/2011	12.45	1.38	12.5	4.22
9/26/2011	12.23	1.23	12.29	4.02
10/3/2011	12	0.6	11.72	3.71
10/17/2011	12.18	1.0	12.15	4.0
10/31/2011	11.46	(+0.6)	10.41	2.86
11/14/2011	11.86	1.0	11.59	3.65
11/28/2011	11.84	0.45	11.38	3.53
12/15/2011	11.67	0.33	10.90	3.39
1/9/2012	12.26	1.36	12.54	4.19
1/11/12	12.40	1.41	12.64	4.22
1/12/12	11.82	0.60	12.15	3.80
2/2/12	11.95	0.75	11.80	3.88

LABORATORY TESTING RESULTS

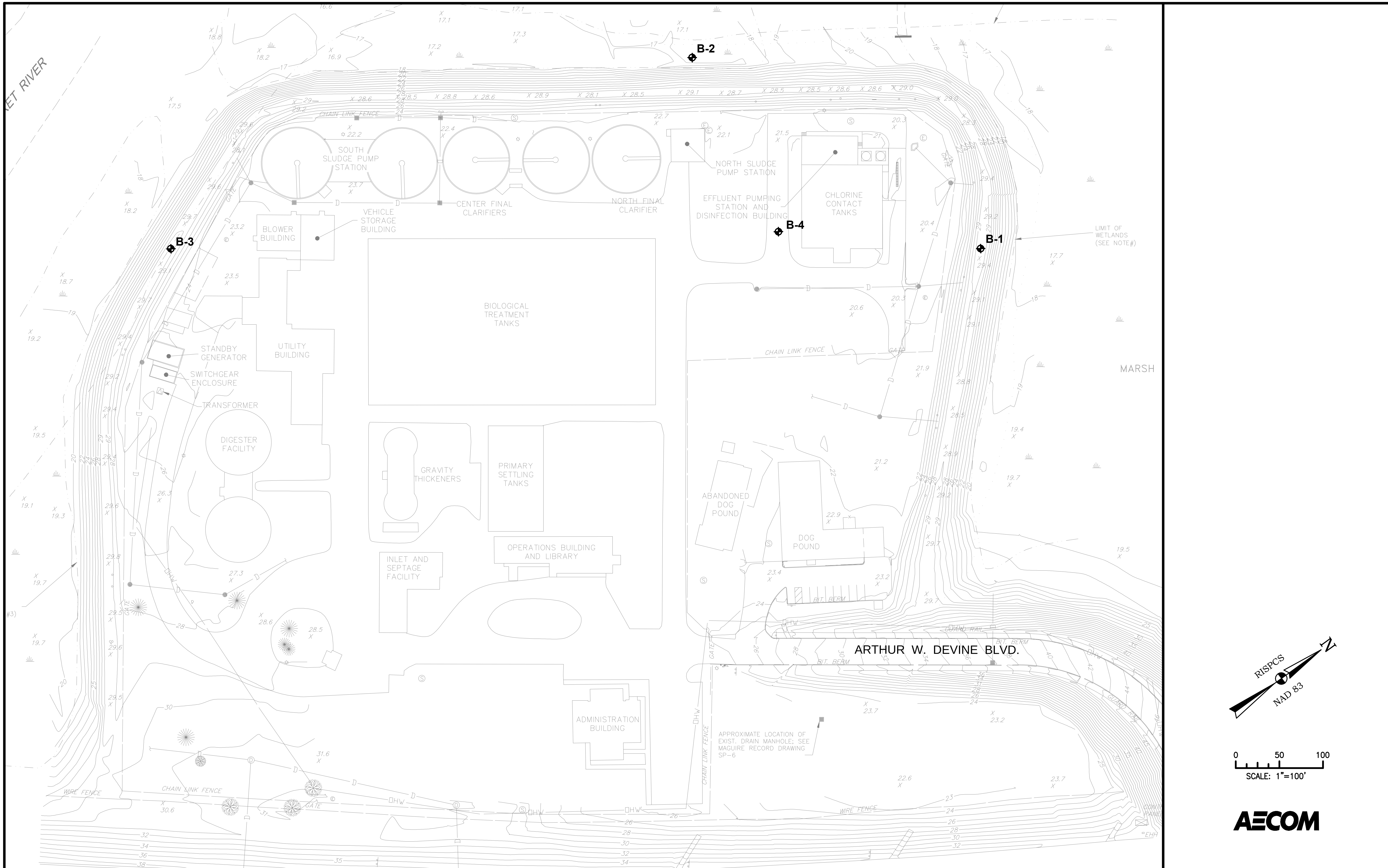
A laboratory testing program consisting of twelve (12) Grain Size Analyses was performed by GeoTesting Express, Inc. of Boxborough, MA. The submitted report is provided as Attachment 2.

ATTACHMENTS

Boring Location Plan

Attachment 1 – Boring Logs

Attachment 2 – Laboratory Testing Results



B-1
B-2
B-3
B-4
SOUTH SLUDGE PUMP STATION
CENTER FINAL CLARIFIERS
NORTH FINAL CLARIFIER
NORTH SLUDGE PUMP STATION
EFFLUENT PUMPING STATION AND DISINFECTION BUILDING
CHLORINE CONTACT TANKS
VEHICLE STORAGE BUILDING
BLOWER BUILDING
BIOLOGICAL TREATMENT TANKS
STANDBY GENERATOR
SWITCHGEAR ENCLOSURE
TRANSFORMER
UTILITY BUILDING
DIGESTER FACILITY
GRAVITY THICKENERS
PRIMARY SETTLING TANKS
OPERATIONS BUILDING AND LIBRARY
INLET AND SEPTAGE FACILITY
ADMINISTRATION BUILDING
ABANDONED DOG POUND
DOG POUND
CHAIN LINK FENCE
WIRE FENCE
GATE
BIT. BERM
GUARD RAIL
ARTHUR W. DEVINE BLVD.
MARSH
LIMIT OF WETLANDS (SEE NOTE#)
APPROXIMATE LOCATION OF EXIST. DRAIN MANHOLE; SEE MAGUIRE RECORD DRAWING SP-6
RISPCS
NAD 83
N
0
50
100
SCALE: 1"=100'
AECOM

Attachment 1 – Boring Logs

GEOLOGIC LOG

AECOM

PROJECT : Warwick WWTP Dike Improvement		SHEET 1 of 3	BORING NO. B-1
SITE LOCATION: Service Road Warwick, RI	JOB NO.: 60219451		
	LOCATION: N: E:	Elevation: 29.57	Total Depth: 52 ft
DRILL CONTRACTOR : New England Boring	ENG/GEO : W. Song		Aug. 31, 2011
DRILL RIG : Mobile B-53	DRILLER : O. Cone	FINISHED :	Sept. 1, 2011
Hole Size : ~4"	Weather : Bright, Sunny, ~80s	Ground Water (Depth) : 14 ft	
Drilling Method : 4" HW Cased	Drilling Fluid : Potable Water	Top of Rock (Depth) : Not Encountered	

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-1 0-2'	27	3-12-15-17	20"	Top 10": dark brown SILT AND SAND mixture, topsoil with grass roots. Bottom 10": light yellowish brown Silty SAND , few fine to coarse gravel, moist.		
5	SS-2 5'-7'	131	55-64-67-53	20"	Mottle colored SAND AND GRAVEL, fine to coarse sand, mostly gravel, dry.		SAND & GRAVEL
10	SS-3 10'-12'	40	22-22-18-35	20"	Top 16": SAND AND GRAVEL, fine to coarse sand, fine to coarse gravel. Bottom 4": Reddish brown fine SAND AND SILT mixture, some sand.		11'8"
							SAND & SILT
15							15'
	SS-4 15'-17'	60	25-33-27-32	16"	Mottle colored SAND AND GRAVEL, fine to coarse sand, fine to coarse gravel, wet.		SAND & GRAVEL
							20'

SAMPLE TYPES:		SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	trace 0 to 5%			Oct. 11, 11 WS
SS=SPLIT SPOON	few 5 to 10%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	
ST=SHELBY TUBE	little 15 to 25%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	some 30 to 45%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	
	mostly >50%			

GEOLOGIC LOG

AECOM

PROJECT : Warwick WWTP Dike Improvement		SHEET	BORING NO.
SITE LOCATION:	JOB NO.: 60219451	2 of 3	B-1
Service Road Warwick, RI	LOCATION: N: E:	Elevation: 29.57	Total Depth: 52 ft

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-5 20'-22'	26	10-12-14-13	4"	Light brown fine to coarse SAND, bottom 2": SANDY GRAVEL, coarse gravel, wet.		
25	SS-6 25'-27'	19	4-10-9-9	10"	Brown medium to coarse SAND, trace fine sand, trace coarse gravel, wet.		
30	SS-7 30'-32'	13	5-7-6-10	10"	Brown SILTY fine SAND, some silt, wet.		SAND
35	SS-8 35'-37'	45	12-20-25-28	14"	Blackish and reddish brown fine to medium SAND, few silt, wet.		
40	SS-9 40'-42'	26	2-10-16-19	20"	Gray fine SAND, trace medium sand, few silt, wet.		

SAMPLE TYPES:	trace 0 to 5%	SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few 5 to 10%			
SS=SPLIT SPOON	little 15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	Oct. 11, 11 WS
ST=SHELBY TUBE	some 30 to 45%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly >50%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

GEOLOGIC LOG

AECOM

PROJECT : Warwick WWTP Dike Improvement				SHEET	BORING NO.
SITE LOCATION:	JOB NO.: 60219451			3 of 3	B-1
Service Road Warwick, RI	LOCATION: N:	E:		Elevation: 29.57	Total Depth: 52 ft

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-10 45'-47'	10	11-5-5-6	10"	Light brown fine SAND, little silt, wet.		SAND
50	SS-11 50'-52'	18	6-8-10-11	14"	Brown fine to medium SAND, wet.		
							EOB @ 52 ft.
55							
60							
65							

SAMPLE TYPES:	trace 0 to 5%	SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few 5 to 10%			
SS=SPLIT SPOON	little 15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	Oct. 11, 11 WS
ST=SHELBY TUBE	some 30 to 45%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly >50%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

MONITORING WELL INSTALLATION LOG

PROJECT: Warwick WWTP Dike Improvement	JOB NO. 60219451	WELL NO. B-1
DRILLING CONTRACTOR: New England Boring	COORDINATES:	
BEGUN: 9/1/2011	GEO/ENG: W. Song	WATER LEVEL (Depth.) 14 ft.
FINISHED: 9/1/2011	DRILLER: O. Cone	

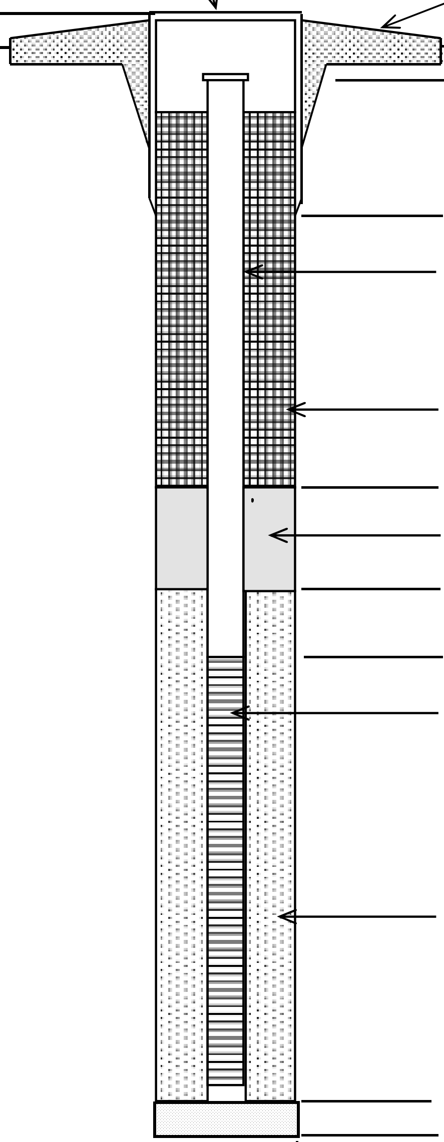
FLUSHMOUNT SURFACE CASING:

DIA.: 6"

TYPE: 10" Steel Skirt w/ Pentagon Key Cover

GENERALIZED GEOLOGIC LOG

Refer to Boring Log



	DEPTH BGS HEIGHT AGS (FT)	ELEV. (FT) NGVD DATUM
CONCRETE		
GROUND SURFACE	0.0	
TOP OF PVC RISER	2"	
BOTTOM OF PROTECTIVE PIPE	10"	
PVC RISER CASING: SCH.: 40 DIAM.: 2-in.		
BACKFILL TYPE: Cuttings & Stone		
TOP OF ANNULAR SEAL	6.0	
ANNULAR SEAL: TYPE: Bentonite Chips		
TOP OF FILTER PACK	8.0	
TOP OF WELL SCREEN	10.0	
PVC SCREEN: SCH.: 40 DIAM.: 2-in. SLOT SIZE: 0.010-in.		
FILTER PACK TYPE: Holliston SIZE: #2 Sand		
BOTTOM OF SCREEN	20.0	
BOTTOM OF HOLE*	52.0	

4-in.

(DIAM.)

AECOM

GEOLOGIC LOG

AECOM

PROJECT : **Warwick WWTP Dike Improvement**		SHEET 1 of 2	BORING NO. **B-2**
SITE LOCATION: Service Road Warwick, RI	JOB NO.: **60219451**		
	LOCATION: N: E:	Elevation: 17.49	Total Depth: 42 ft.
DRILL CONTRACTOR : New England Boring	ENG/GEO : W. Song	BEGUN : Sept. 2, 2011	
DRILL RIG : Skid D-25	DRILLER : T. Carpenter	FINISHED : Sept. 2, 2011	

Hole Size : ~4"	Weather : Bright, Sunny, ~80s	Ground Water (Depth) : 1.5 ft.
Drilling Method : 4" HW Cased	Drilling Fluid : Potable Water	Top of Rock (Depth) : Not Encountered

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-1 0-2'	14	3-7-7-8	18"	Top 2": Topsoil. Brown SAND AND GRAVEL, fine to coarse sand, fine to carse gravel, little silt, occasional cobbles.		
5	SS-2 5'-7'	19	10-9-10-11	18"	Brown SAND AND GRAVEL, mostly fine to carse sand, some fine to coarse gravel, few silt.		SAND & GRAVEL
10	SS-3 10'-12'	20	11-9-11-11	16"	Brown SAND AND GRAVEL, fine to coarse sand, fine to coarse gravel.		
15							15'
	SS-4 15'-17'	5	2-2-3-3	20"	Brown silty fine SAND, trace medium sand, some silt.		SAND

SAMPLE TYPES:	trace 0 to 5%	SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few 5 to 10%			
SS=SPLIT SPOON	little 15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	Oct. 11, 11 WS
ST=SHELBY TUBE	some 30 to 45%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly >50%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

GEOLOGIC LOG

AECOM

PROJECT : Warwick WWTP Dike Improvement		SHEET	BORING NO.
SITE LOCATION:	JOB NO.: 60219451	2 of 2	**B-2**
Service Road Warwick, RI	LOCATION: N: E:	Elevation: 17.49	Total Depth: 42 ft

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-5 20'-22'	6	3-3-3-4	20"	Brown fine SAND, few medium sand, few silt.		SAND
25							
	SS-6 25'-27'	15	4-7-8-12	12"	Brown fine SAND, little silt, wet.		
30							
	SS-7 30'-32'	9	4-4-5-6	10"	Brown fine SAND, few medium sand, little silt, wet.		
35							
	SS-8 35'-37'	8	2-4-4-5	12"	Brown fine to medium SAND, trace silt, wet.		
40							
	SS-9 40'-42'	9	4-4-5-5	14"	Brown fine to medium SAND, trace silt, wet.		
							EOB @ 42 ft.

SAMPLE TYPES:		SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few 5 to 10%			Oct. 11, 11 WS
SS=SPLIT SPOON	little 15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	
ST=SHELBY TUBE	some 30 to 45%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly >50%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

trace 0 to 5%

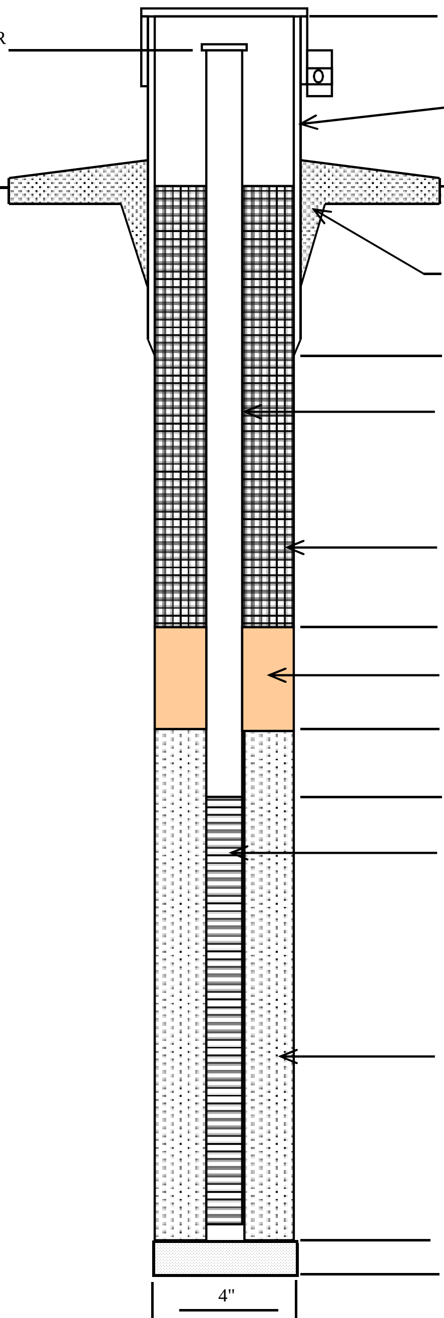
MONITORING WELL INSTALLATION LOG

PROJECT:	JOB NO.	WELL NO.
Warwick WWTP Dike Improvement	**60219451**	**B-2**

DRILLING CONTRACTOR: New England Boring	COORDINATES:
BEGUN: 9-2-11	GEO/ENG: W. Song
FINISHED: 9-2-11	DRILLER: T. Carpenter

WATER LEVEL (Depth): 1.5 ft

GENERALIZED GEOLOGIC LOG



		DEPTH BGS / HEIGHT AGS (FT)	ELEV. (FT) NGVD DATUM
TOP OF STEEL CASING		2'	
TOP OF PVC RISER		1.8'	
SURFACE CASING:	DIA.: ; TYPE:		
GROUND SURFACE		0'	
CONCRETE			
BOTTOM OF PROTECTIVE PIPE			
PVC RISER CASING:	SCH.: 40; DIAM.: 2-in.		
BACKFILL TYPE	Grout		
TOP OF ANNULAR SEAL		4'	
ANNULAR SEAL: TYPE:	Bentonite Chips		
TOP OF FILTER PACK		6'	
TOP OF WELL SCREEN		8'	
PVC SCREEN:	SCH.: 40; DIAM.: 2-in.; SLOT SIZE: 0.010-in.		
FILTER PACK	TYPE: Sand		
BOTTOM OF SCREEN		18'	
BOTTOM OF HOLE		42'	

4" (DIAM.)

AECOM

GEOLOGIC LOG

AECOM

PROJECT : Warwick WWTP Dike Improvement		SHEET	BORING NO.
SITE LOCATION: Service Road, Warwick, RI	JOB NO.: 60219451	1 of 3	B-3
	LOCATION: N: E:	Elevation: 29.43	Total Depth: 52 ft
DRILL CONTRACTOR : New England Boring	ENG/GEO : W. Song	BEGUN : Sept. 1, 2011	
DRILL RIG : Mobile B-53	DRILLER : O. Cone	FINISHED : Sept. 2, 2011	

Hole Size : ~4"	Weather : Bright, Sunny, ~80s	Ground Water (Depth) : 15 ft.
Drilling Method : 4" HW Cased	Drilling Fluid : Potable Water	Top of Rock (Depth) : Not Encountered

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-1 0-2'	25	3-10-15-10	24"	Top 4": Topsoil with grass roots. Bottom 20": brown SILTY fine to coarse SAND, few fine to coarse gravel, moist.		
5	SS-2 5'-7'	15	3-3-12-14	20"	Brown SILTY fine to coarse SAND, few fine to coarse gravel.		SAND
10	SS-3 10'-12'	35	21-19-16-23	20"	Top 4": brown fine to medium sand, little fine to coarse gravel. Bottom 16": Reddish brown SAND AND GRAVEL, fine to coarse sand, fine to coarse gravel, moist.		10'8" SAND & GRAVEL
15	SS-4 15'-17'	18	4-9-9-10	24"	Top 6": brown fine to medium SAND, few fine gravel. Bottom 18": reddish brown fine to medium SAND, few silt, wet.		15' SAND

SAMPLE TYPES:	trace 0 to 5%	SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few 5 to 10%			Oct. 11, 11 WS
SS=SPLIT SPOON	little 15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	
ST=SHELBY TUBE	some 30 to 45%	5-9 Loose, 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly >50%	30-49 Dense, 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

GEOLOGIC LOG

AECOM

PROJECT : Warwick WWTP Dike Improvement		SHEET	BORING NO.
SITE LOCATION:	JOB NO.: 60219451	2 of 3	B-3
Service Road Warwick, RI	LOCATION: N: E:	Elevation: 29.43	Total Depth: 52 ft

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-5 20'-22'	18	6-10-8-7	12"	Brown fine to medium SAND, few silt.		
25							
	SS-6 25'-27'	10	3-4-6-7	12"	Brown fine to medium SAND, few to little silt, wet.		
30							
	SS-7 30'-32'	12	3-5-7-10	20"	Top 12": similar brown fine to medium SAND, trace silt, wet. Bottom 8": reddish brown fine to medium SAND, trace to few silt.		SAND
35							
	SS-8 35'-37'	9	3-4-5-8	8"	Brown fine to medium SAND, few silt, trace coarse gravel, wet.		
40							
	SS-9 40'-42'	19	5-10-9-13	8"	Brown fine to medium SAND, few silt, wet.		

SAMPLE TYPES:			SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few	5 to 10%			Oct. 11, 11 WS
SS=SPLIT SPOON	little	15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	
ST=SHELBY TUBE	some	30 to 45%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly	>50%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

(trace 0 to 5%)

GEOLOGIC LOG

AECOM

PROJECT: Warwick WWTP Dike Improvement		SHEET	BORING NO.
SITE LOCATION:	JOB NO.: 60219451	3 of 3	B-3
Service Road	LOCATION:	Elevation:	Total Depth:
Warwick, RI	N: E:	29.43	52 ft

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-10 45'-47'	15	3-6-9-9	14"	Brown fine to medium SAND, trace silt, wet.		SAND
50	SS-11 50'-52'	17	2-6-11-17	20"	Dark brown fine to medium SAND, trace silt, wet.		
							EOB @ 52 ft.
55							
60							
65							

SAMPLE TYPES:		SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few 5 to 10%			Oct. 11, 11 WS
SS=SPLIT SPOON	little 15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	
ST=SHELBY TUBE	some 30 to 45%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly >50%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

trace 0 to 5%

MONITORING WELL INSTALLATION LOG

PROJECT: Warwick WWTP Dike Improvement	JOB NO. 60219451	WELL NO. B-3
DRILLING CONTRACTOR: New England Boring	COORDINATES:	
BEGUN: 9/2/2011	GEO/ENG: W. Song	WATER LEVEL (Depth.)
FINISHED: 9/2/2011	DRILLER: O. Cone	15 ft.

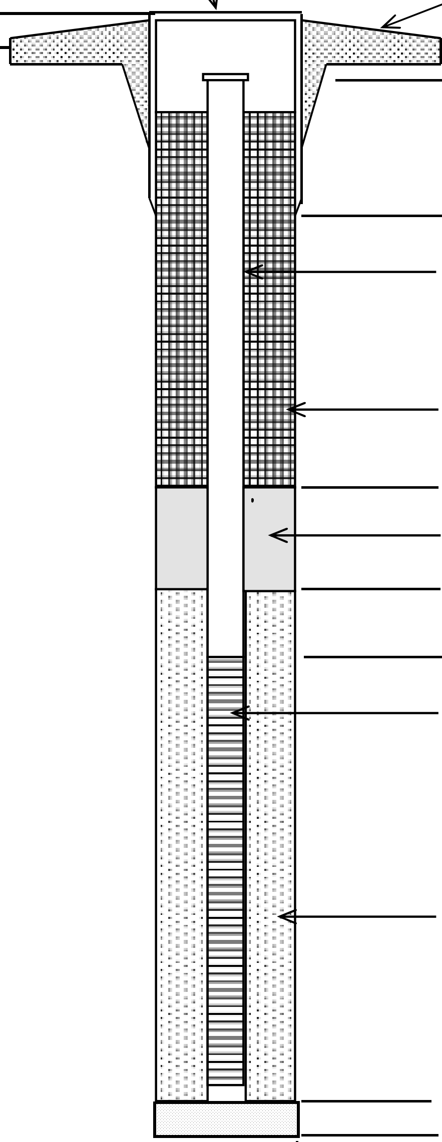
FLUSHMOUNT SURFACE CASING:

DIA.: 6"

TYPE: 10" Steel Skirt w/ Pentagon Key Cover

GENERALIZED GEOLOGIC LOG

Refer to Boring Log



	DEPTH BGS HEIGHT AGS (FT)	ELEV. (FT) NGVD DATUM
CONCRETE		
GROUND SURFACE	0.0	
TOP OF PVC RISER	2"	
BOTTOM OF PROTECTIVE PIPE	10"	
TOP OF ANNULAR SEAL	6.0	
TOP OF FILTER PACK	8.0	
TOP OF WELL SCREEN	10.0	
BOTTOM OF SCREEN	20.0	
BOTTOM OF HOLE*	52.0	

PVC RISER CASING:
SCH.: 40
DIAM.: 2-in.

BACKFILL TYPE: Cuttings & Stone

ANNULAR SEAL: TYPE: Bentonite Chips

PVC SCREEN:
SCH.: 40
DIAM.: 2-in.
SLOT SIZE: 0.010-in.

FILTER PACK
TYPE: Holliston
SIZE: #2 Sand

4-in.
(DIAM.)

AECOM

GEOLOGIC LOG

AECOM

PROJECT : **Warwick WWTP Dike Improvement**			SHEET 1 of 1	BORING NO. **B-4**
SITE LOCATION: Service Road Warwick, RI	JOB NO.: **60219451**			
	LOCATION: N:	E:	Elevation: 21.0	Total Depth: 22 ft
DRILL CONTRACTOR : New England Boring	ENG/GEO : W. Song		BEGUN : Aug. 31, 2011	
DRILL RIG : Mobile B-53	DRILLER : O. Cone		FINISHED : Aug. 31, 2011	
Hole Size : ~4"	Weather : Bright, Sunny, ~80s		Ground Water (Depth) : 5 ft	
Drilling Method : 4" HW Cased	Drilling Fluid : Potable Water		Top of Rock (Depth) : Not Encountered	

Depth (ft)	Sample Type/No.	N Value	Blow Count (per 6 in.) or Drilling Rate(min/ft)	Sample Recovery or REC & RQD	SAMPLE DESCRIPTION	ASTM Class.	STRATIGRAPHIC DESCRIPTION
	SS-1 0-2'	23	3-10-13-16	20"	Top 6": Topsoil. Middle 10": dark brown SANDY GRAVEL, coarse gravel, fine to coarse SAND. Bottom 4": brown fine to medium SAND, moist.		SAND
5							5'
	SS-2 5'-7'	32	10-14-18-16	8"	Brown with mottle colored SAND with silt and gravel, fine to coarse sand, little coarse gravel, little silt.		SAND & GRAVEL
10							10'
	SS-3 10'-12'	22	9-11-11-11	8"	Light brown fine to medium SAND, trace coarse gravel, wet.		
15							SAND
	SS-4 15'-17'	10	4-5-5-6	14"	Light brown fine to medium SAND, little silt, wet.		
							20'
	SS-5 20'-22'	8	4-4-4-5	14"	Light brown SANDY SILT, some fine sand, mostly silt, wet.		SANDY SILT EOB @ 22 ft.

SAMPLE TYPES:			SPT Resistance		Approve/Date
S3=3" SPLIT SPOON	few	5 to 10%			Oct. 11, 11 WS
SS=SPLIT SPOON	little	15 to 25%	Cohesionless Density: 0-4 Very Loose	Cohesive Consistency: 0-2 Very Soft	
ST=SHELBY TUBE	some	30 to 45%	5-9 Loose 10-29 Med. Dense	3-4 Soft, 5-8 M/Stiff, 9-15 Stiff	
R=ROCK CORE	mostly	>50%	30-49 Dense 50+ Very Dense	16-30 V-Stiff, 31+ Hard	

trace 0 to 5%

MONITORING WELL INSTALLATION LOG

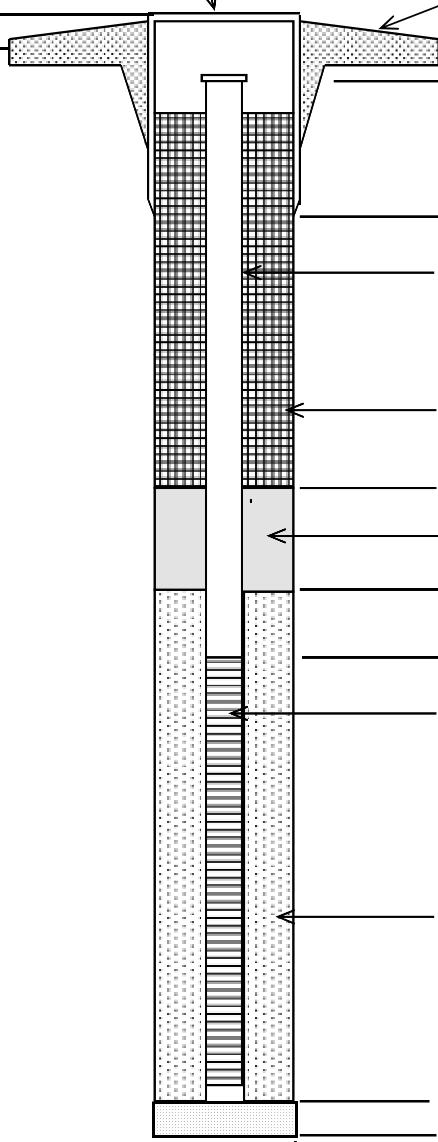
PROJECT: **Warwick WWTP Dike Improvement**	JOB NO. **60219451**	WELL NO. **B-4**
DRILLING CONTRACTOR: New England Boring	COORDINATES:	
BEGUN: 8/31/2011	GEO/ENG: W. Song	WATER LEVEL (Depth.)
FINISHED: 8/31/2011	DRILLER: O. Cone	5 ft

FLUSHMOUNT SURFACE CASING:
DIA.: 6"
TYPE: 10" Steel Skirt w/ Pentagon Key Cover

CONCRETE

GENERALIZED GEOLOGIC LOG

Refer to Boring Log



4-in.
(DIAM.)

PVC RISER CASING:
SCH.: 40
DIAM.: 2-in.

BACKFILL TYPE: Cuttings & Stone

ANNULAR SEAL: TYPE: Bentonite Chips

PVC SCREEN:
SCH.: 40
DIAM.: 2-in.
SLOT SIZE: 0.010-in.

FILTER PACK
TYPE: Holliston
SIZE: #2 Sand

	DEPTH BGS HEIGHT AGS (FT)	ELEV. (FT) NGVD DATUM
GROUND SURFACE	0.0	
TOP OF PVC RISER	2"	
BOTTOM OF PROTECTIVE PIPE	10"	
TOP OF ANNULAR SEAL	6.0	
TOP OF FILTER PACK	8.0	
TOP OF WELL SCREEN	10.0	
BOTTOM OF SCREEN	20.0	
BOTTOM OF HOLE*	22.0	

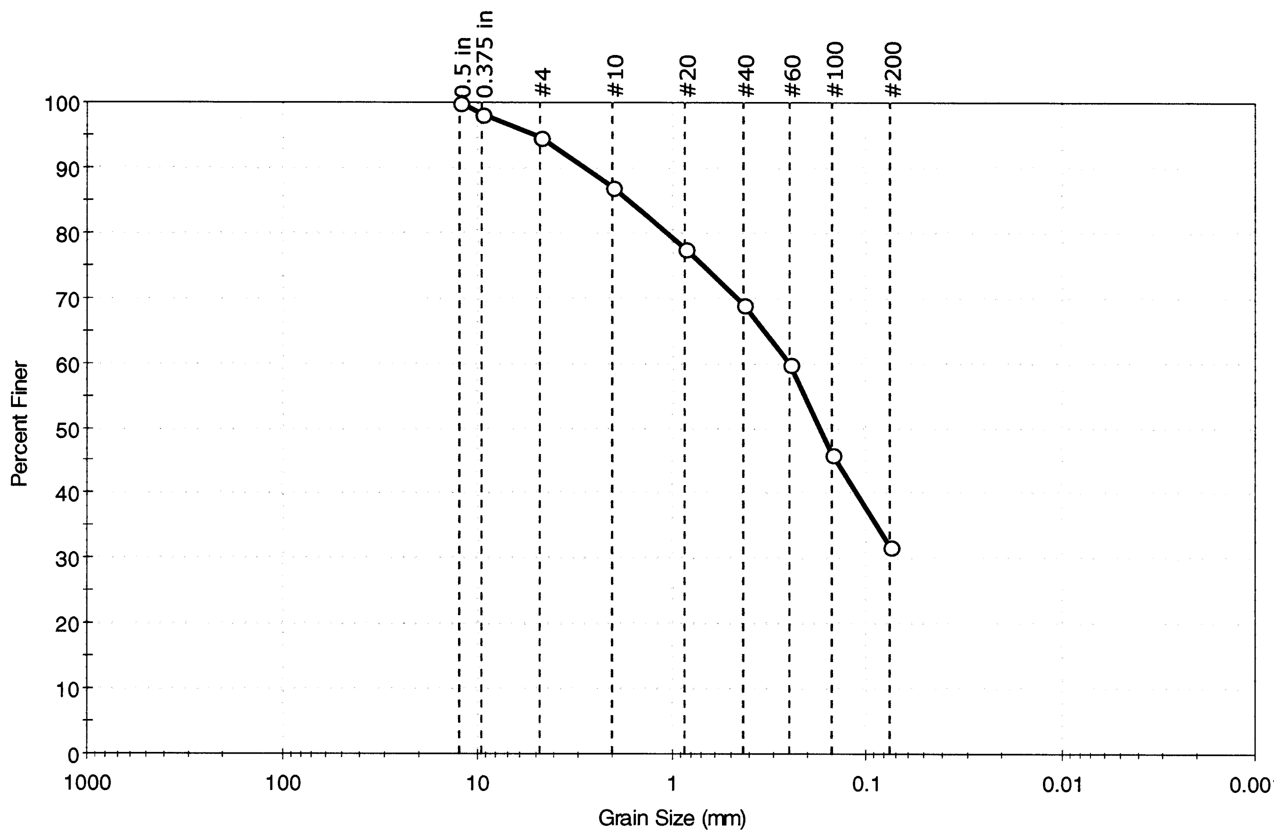
AECOM

Attachment 2 – Laboratory Testing Results

GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-1	Sample Type: jar	Tested By:	jbr
Sample ID:SS-1	Test Date: 10/05/11	Checked By:	jdt
Depth : 0-2 ft	Test Id: 219495		
Test Comment: ---			
Sample Description: Moist, light yellowish brown silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	5.3	62.7	32.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	95		
#10	2.00	87		
#20	0.85	77		
#40	0.42	69		
#60	0.25	60		
#100	0.15	46		
#200	0.075	32		

Coefficients

D_{85} = 1.6621 mm D_{30} = N/A
D_{60} = 0.2498 mm D_{15} = N/A
D_{50} = 0.1738 mm D_{10} = N/A
C_u = N/A C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

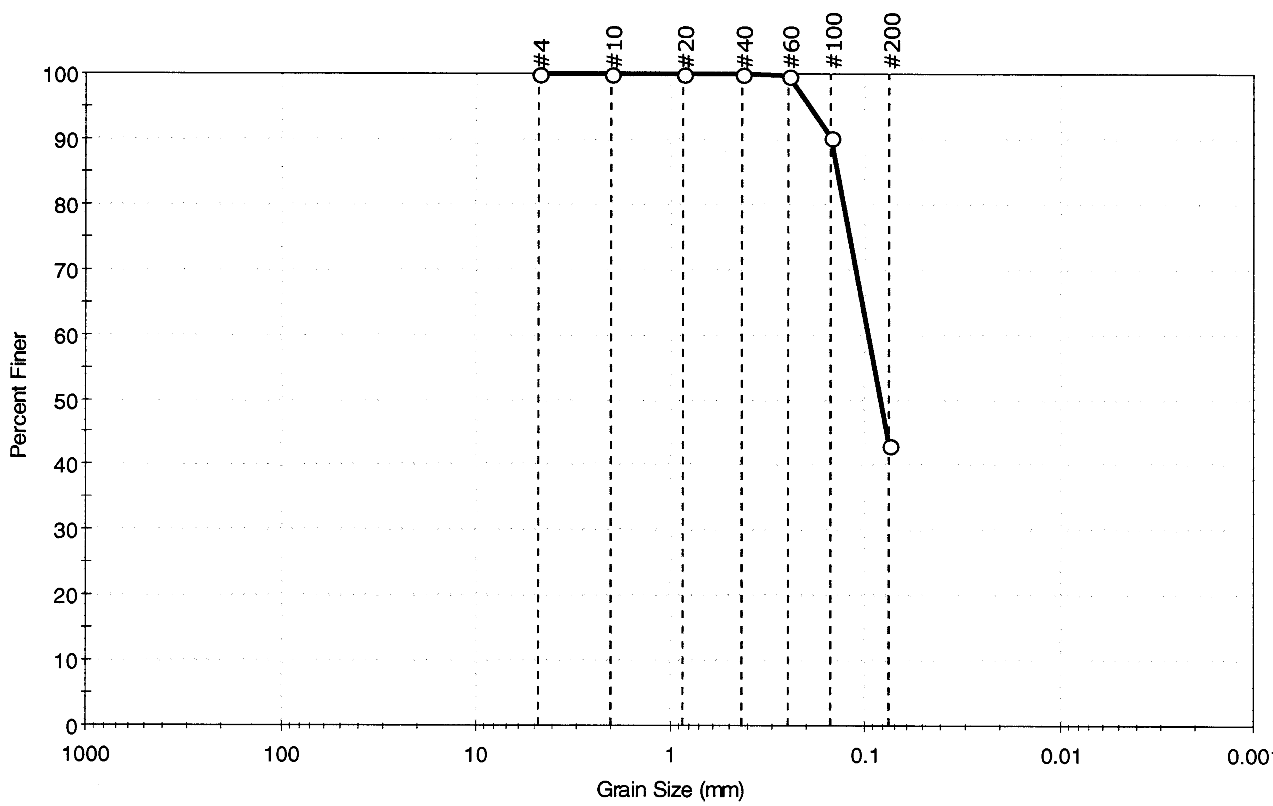
Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-1	Sample Type: jar	Tested By:	jbr
Sample ID:SS-7	Test Date: 10/07/11	Checked By:	jdt
Depth : 30-32 ft	Test Id: 219496		
Test Comment: ---			
Sample Description: Moist, light olive brown silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	57.1	42.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	90		
#200	0.075	43		

Coefficients

D_{85} =0.1386 mm　　D_{30} =N/A

D_{60} =0.0962 mm　　D_{15} =N/A

D_{50} =0.0831 mm　　D_{10} =N/A

C_u =N/A　　C_c =N/A

Classification

ASTM　N/A

AASHTO　Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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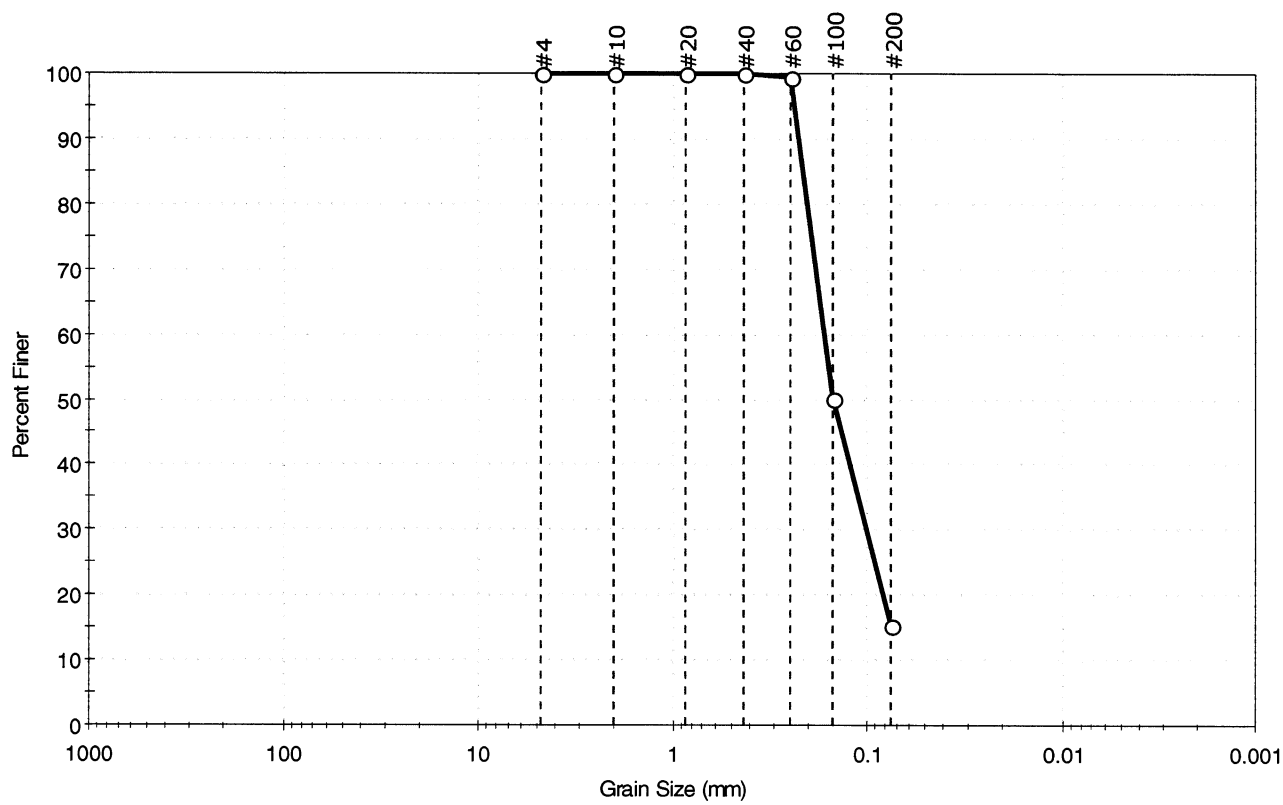
GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-1	Sample Type: jar	Tested By:	jbr
Sample ID:SS-10	Test Date: 10/05/11	Checked By:	jdt
Depth : 45-47 ft	Test Id: 219497		

Test Comment: ---
Sample Description: Moist, olive silty sand
Sample Comment: ---

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	84.6	15.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	50		
#200	0.075	15		

Coefficients

D_{85} =0.2153 mm	D_{30} =0.1003 mm
D_{60} =0.1662 mm	D_{15} =N/A
D_{50} =0.1496 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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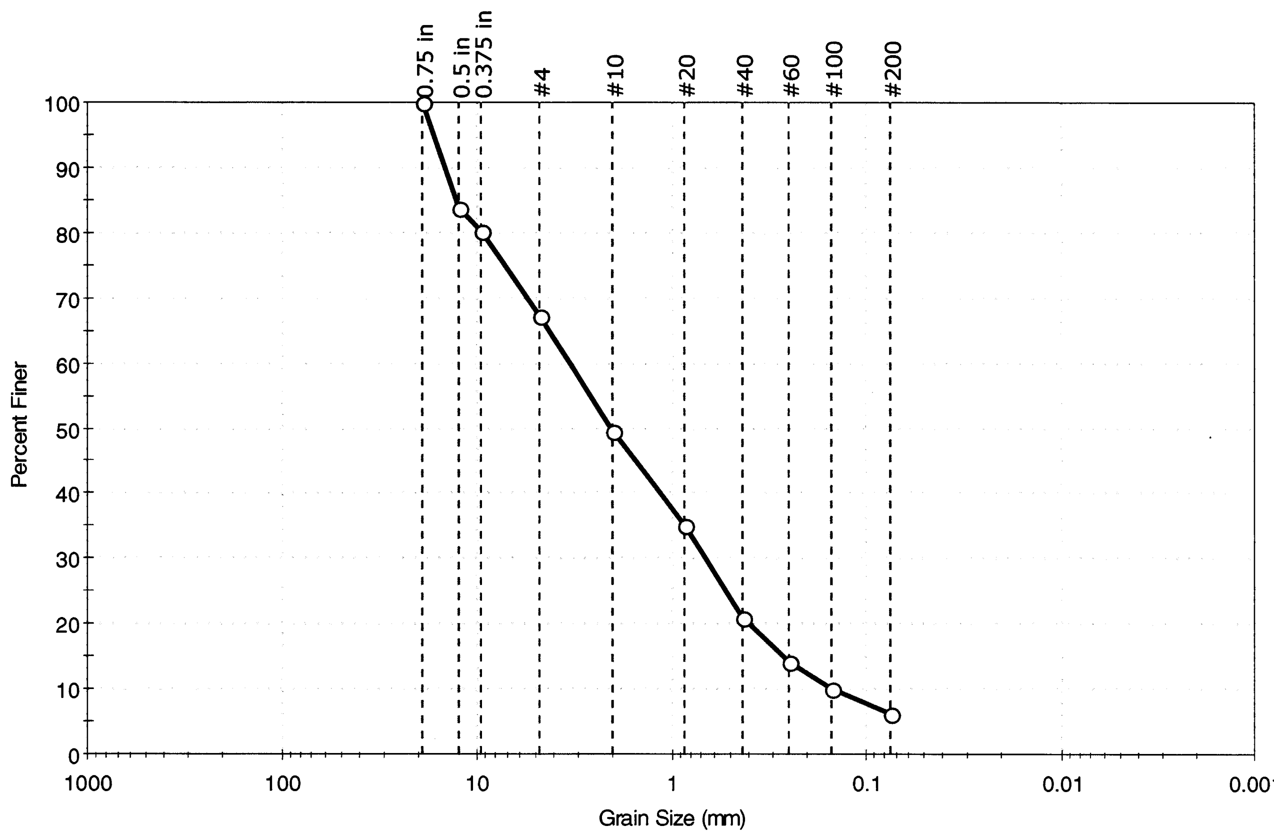
Client:	AECOM		
Project:	Warwick WWTP Dike Improvement		
Location:	Warwick, RI	Project No:	GTX-11197
Boring ID: B-2	Sample Type: jar	Tested By:	jbr
Sample ID:SS-2	Test Date: 10/05/11	Checked By:	jdt
Depth : 5-7 ft	Test Id: 219498		
Test Comment:	---		
Sample Description:	Moist, dark grayish brown sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	32.7	61.0	6.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	84		
0.375 in	9.50	80		
#4	4.75	67		
#10	2.00	50		
#20	0.85	35		
#40	0.42	21		
#60	0.25	14		
#100	0.15	10		
#200	0.075	6		

Coefficients

D_{85} =12.8576 mm D_{30} =0.6593 mm

D_{60} =3.3219 mm D_{15} =0.2692 mm

D_{50} =2.0400 mm D_{10} =0.1498 mm

C_u =22.176 C_c =0.874

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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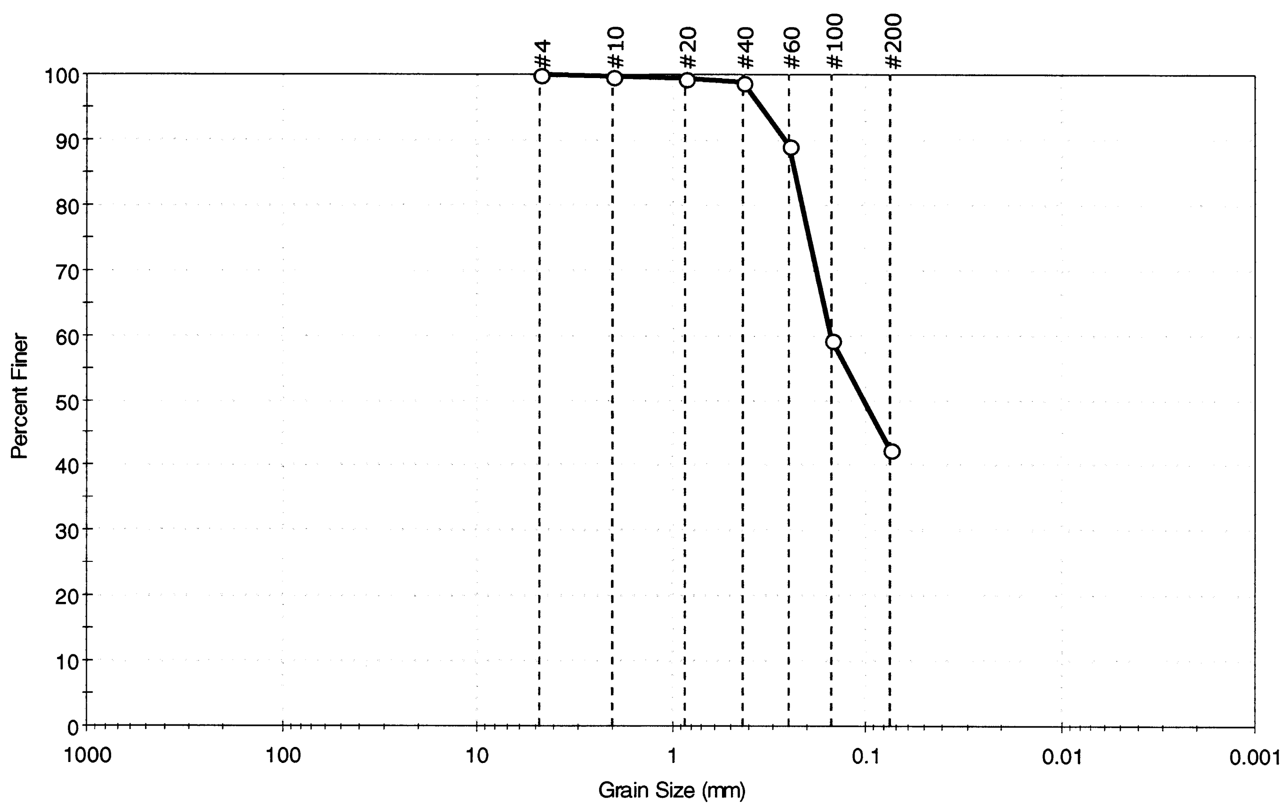
GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-2	Sample Type: jar	Tested By:	jbr
Sample ID:SS-4	Test Date: 10/05/11	Checked By:	jdt
Depth : 15-17 ft	Test Id: 219499		
Test Comment: ---			
Sample Description: Moist, olive silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	57.6	42.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	99		
#60	0.25	89		
#100	0.15	59		
#200	0.075	42		

Coefficients

D_{85} =0.2327 mm　D_{30} =N/A

D_{60} =0.1521 mm　D_{15} =N/A

D_{50} =0.1027 mm　D_{10} =N/A

C_u =N/A　C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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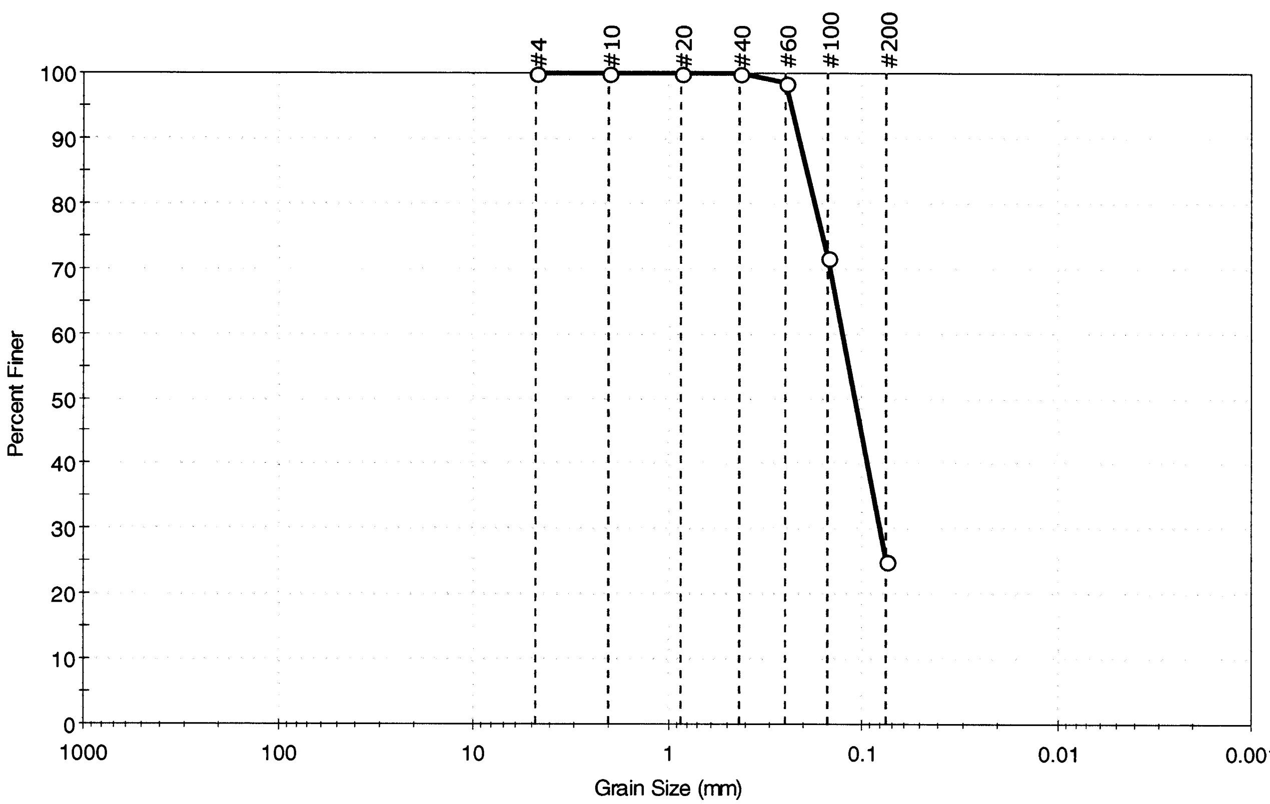
GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-2	Sample Type: jar	Tested By:	jbr
Sample ID:SS-7	Test Date: 10/07/11	Checked By:	jdt
Depth : 30-32 ft	Test Id: 219500		
Test Comment: ---			
Sample Description: Moist, pale olive silty sand			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	—	74.9	25.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	98		
#100	0.15	72		
#200	0.075	25		

Coefficients

D_{85} =0.1936 mm D_{30} =0.0806 mm

D_{60} =0.1261 mm D_{15} =N/A

D_{50} =0.1086 mm D_{10} =N/A

C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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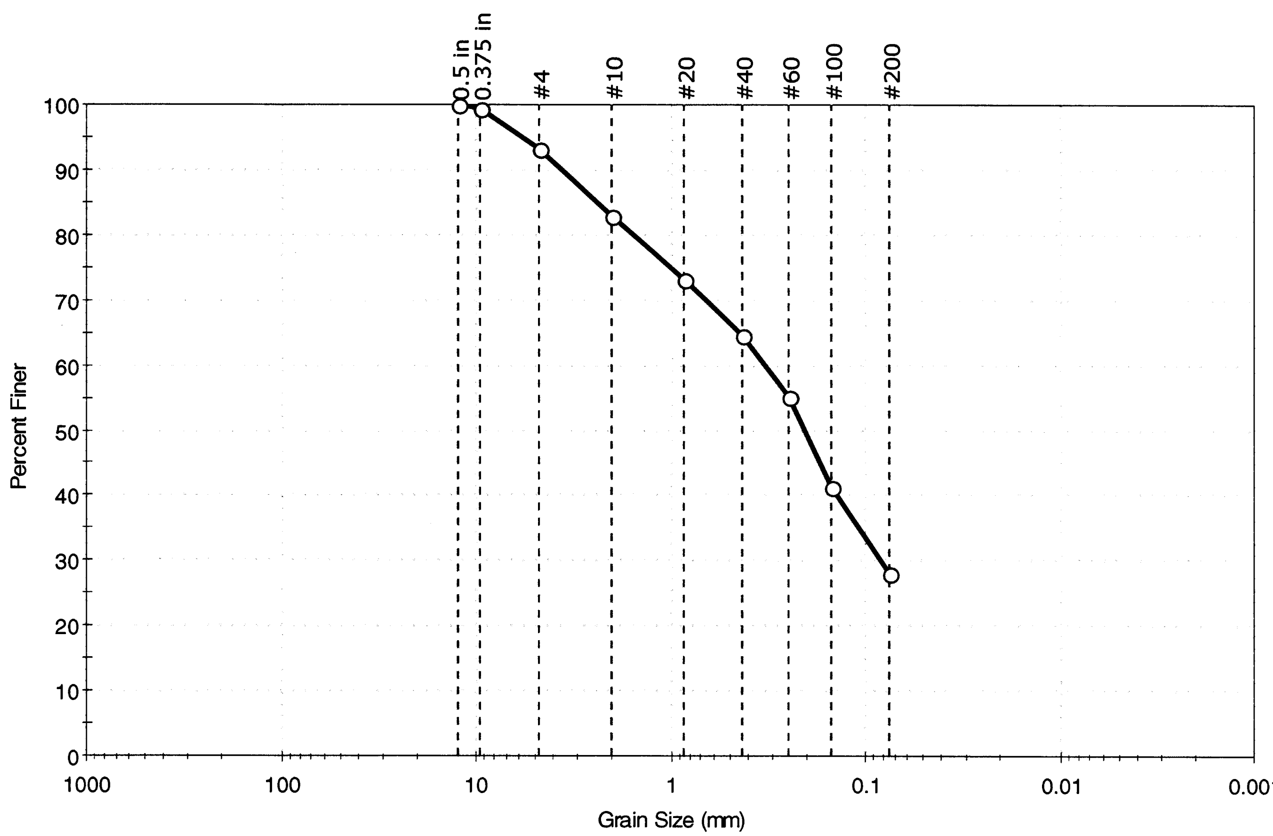
GeoTesting EXPRESS

Client:	AECOM				
Project:	Warwick WWTP Dike Improvement				
Location:	Warwick, RI			Project No:	GTX-11197
Boring ID:	B-3	Sample Type:	jar	Tested By:	jbr
Sample ID:	SS-1	Test Date:	10/05/11	Checked By:	jdt
Depth :	0-2 ft	Test Id:	219501		

Test Comment: ---
Sample Description: Moist, light olive brown silty sand
Sample Comment: ---

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	6.9	65.0	28.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	99		
#4	4.75	93		
#10	2.00	83		
#20	0.85	73		
#40	0.42	65		
#60	0.25	55		
#100	0.15	41		
#200	0.075	28		

Coefficients

D_{85} = 2.4003 mm | D_{30} = 0.0827 mm
D_{60} = 0.3286 mm | D_{15} = N/A
D_{50} = 0.2066 mm | D_{10} = N/A
C_u = N/A | C_c = N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED
Sand/Gravel Hardness : HARD

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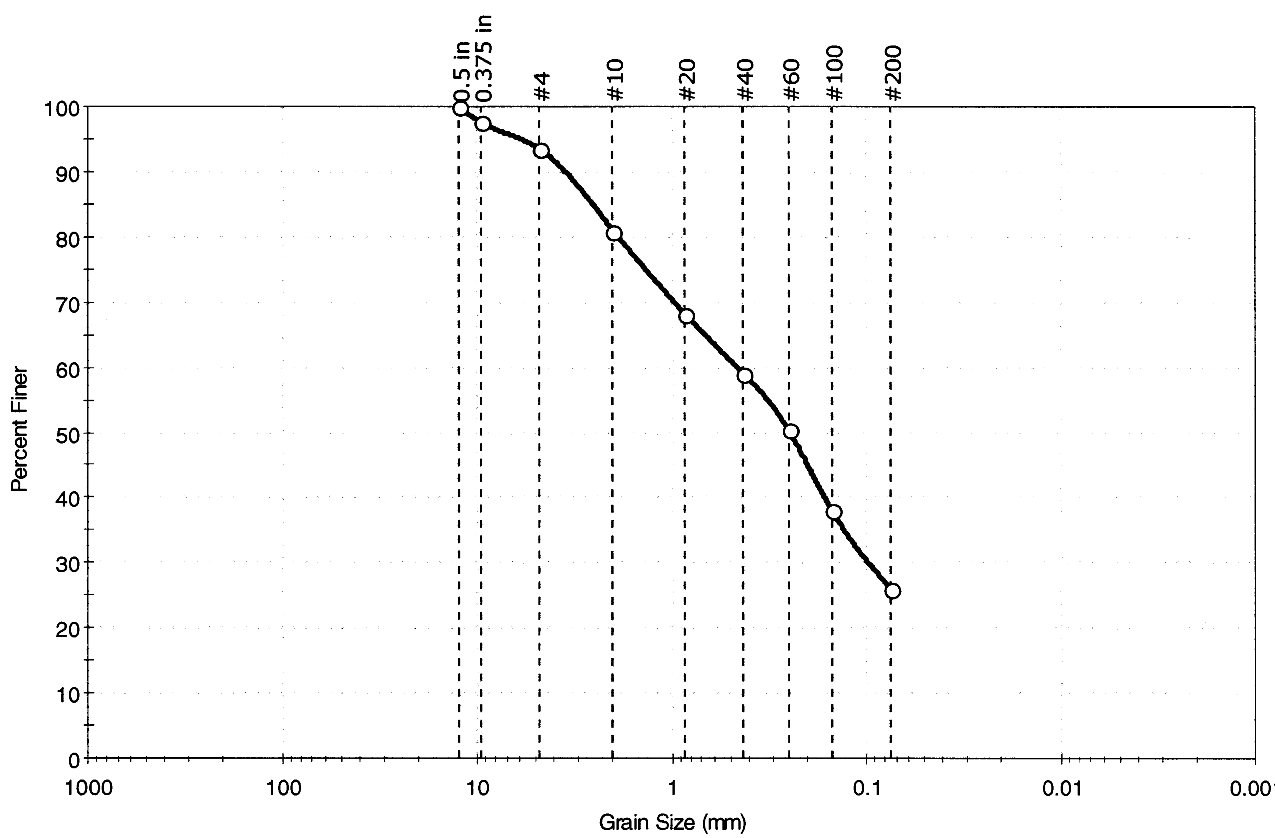
GeoTesting EXPRESS

Client:	AECOM				
Project:	Warwick WWTP Dike Improvement				
Location:	Warwick, RI			Project No:	GTX-11197
Boring ID:	B-3	Sample Type:	jar	Tested By:	jbr
Sample ID:	SS-2	Test Date:	10/05/11	Checked By:	jdt
Depth :	5-7 ft	Test Id:	219502		
Test Comment:	---				
Sample Description:	Moist, light yellowish brown silty sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	6.4	67.5	26.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	98		
#4	4.75	94		
#10	2.00	81		
#20	0.85	68		
#40	0.42	59		
#60	0.25	50		
#100	0.15	38		
#200	0.075	26		

Coefficients

D_{85} =2.6466 mm D_{30} =0.0943 mm
D_{60} =0.4586 mm D_{15} =N/A
D_{50} =0.2456 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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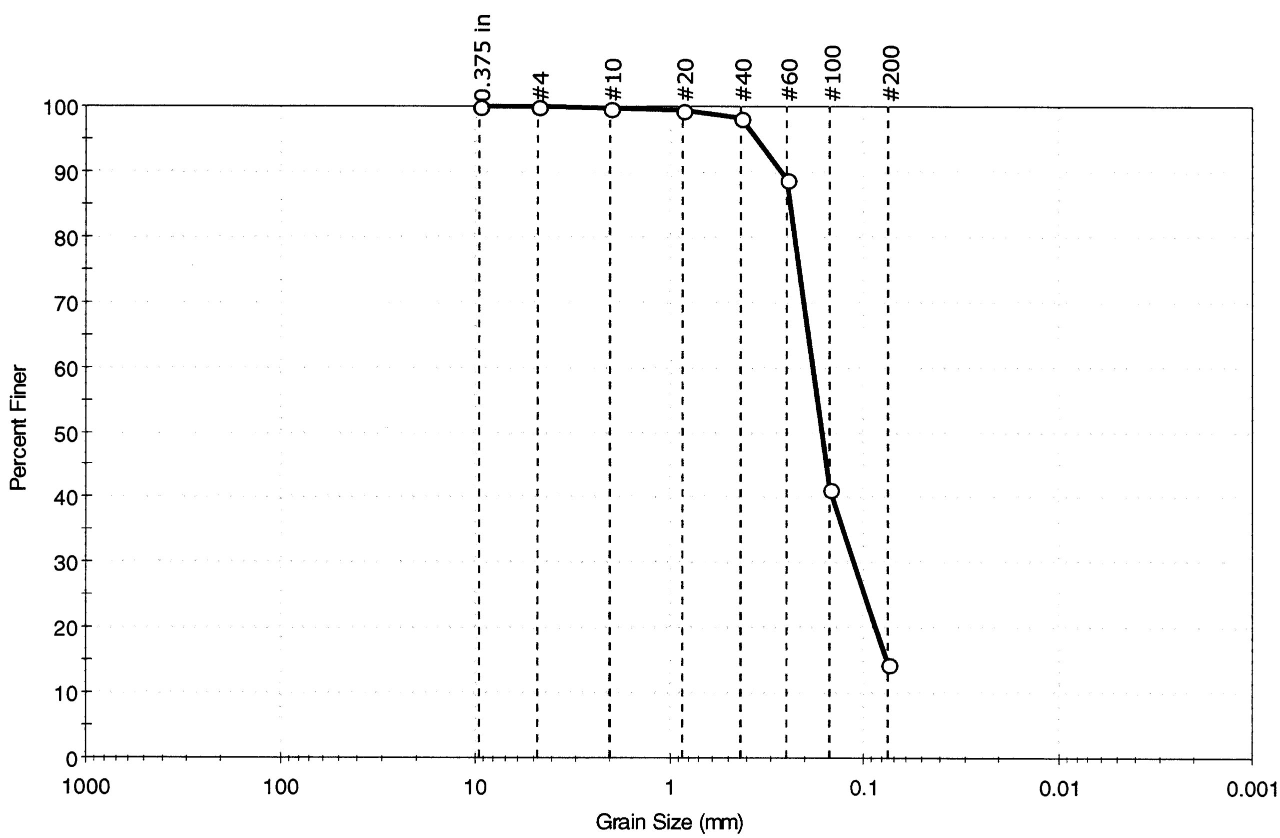
GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-3	Sample Type: jar	Tested By:	jbr
Sample ID:SS-6	Test Date: 10/07/11	Checked By:	jdt
Depth : 25-27 ft	Test Id: 219503		

Test Comment: ---
Sample Description: Moist, olive yellow silty sand
Sample Comment: ---

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.1	85.6	14.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	98		
#60	0.25	89		
#100	0.15	41		
#200	0.075	14		

Coefficients

D_{85} =0.2401 mm | D_{30} =0.1121 mm
D_{60} =0.1833 mm | D_{15} =0.0763 mm
D_{50} =0.1646 mm | D_{10} =0.0672 mm
C_u =N/A | C_c =N/A

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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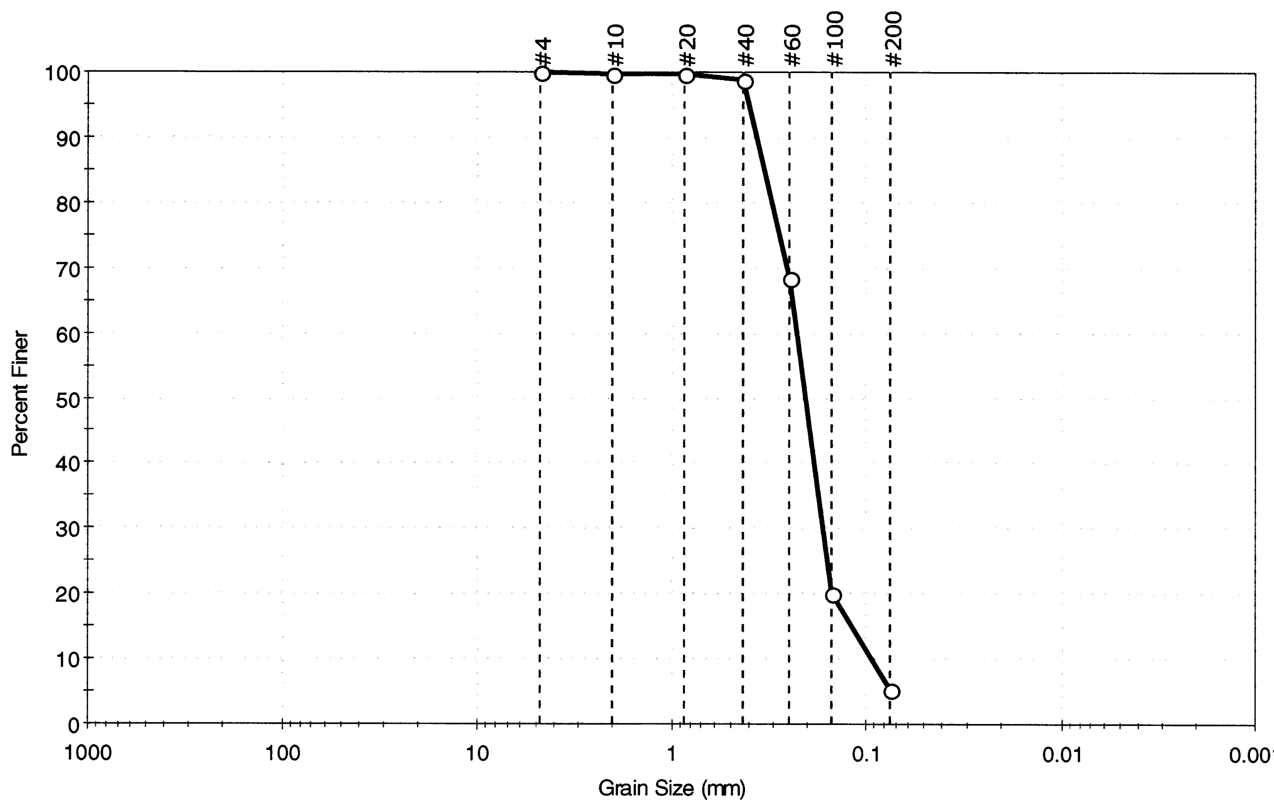
GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-3	Sample Type: jar	Tested By:	jbr
Sample ID:SS-8	Test Date: 10/07/11	Checked By:	jdt
Depth : 35-37 ft	Test Id: 219504		

Test Comment: ---
Sample Description: Moist, light yellowish brown sand with silt
Sample Comment: ---

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	94.6	5.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	99		
#60	0.25	69		
#100	0.15	20		
#200	0.075	5		

Coefficients

D_{85} =0.3331 mm
D_{30} =0.1666 mm
D_{60} =0.2285 mm
D_{15} =0.1182 mm
D_{50} =0.2056 mm
D_{10} =0.0933 mm
C_u =2.449
C_c =1.302

Classification

ASTM N/A

AASHTO Fine Sand (A-3 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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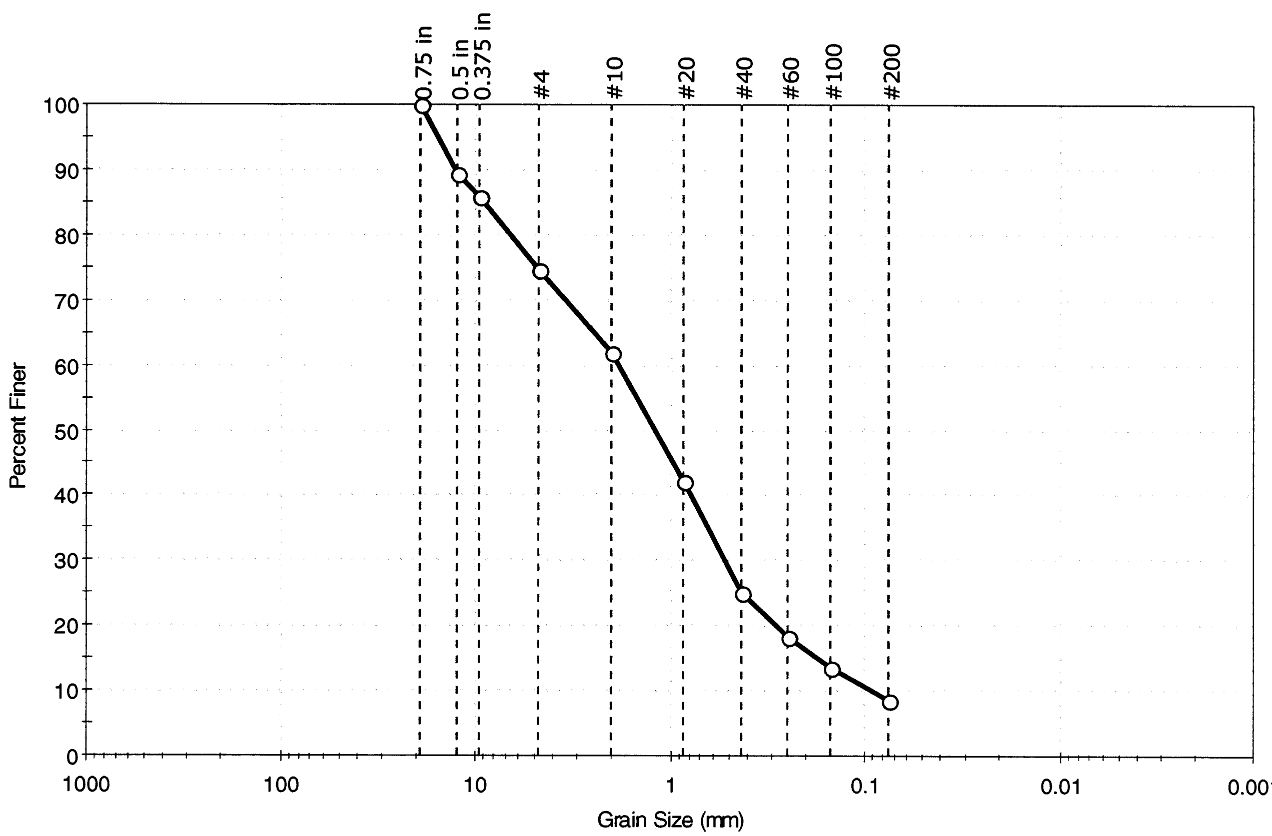
GeoTesting EXPRESS

Client: AECOM			
Project: Warwick WWTP Dike Improvement			
Location: Warwick, RI		Project No:	GTX-11197
Boring ID: B-4	Sample Type: jar	Tested By:	jbr
Sample ID:SS-2	Test Date: 10/05/11	Checked By:	jdt
Depth : 5-7 ft	Test Id: 219505		
Test Comment: ---			
Sample Description: Moist, dark grayish brown sand with silt and gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	25.3	66.1	8.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	89		
0.375 in	9.50	86		
#4	4.75	75		
#10	2.00	62		
#20	0.85	42		
#40	0.42	25		
#60	0.25	18		
#100	0.15	13		
#200	0.075	9		

Coefficients

D_{85} =9.0030 mm D_{30} =0.5170 mm
D_{60} =1.8320 mm D_{15} =0.1765 mm
D_{50} =1.1892 mm D_{10} =0.0920 mm
C_u =19.913 C_c =1.586

Classification

ASTM N/A

AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD

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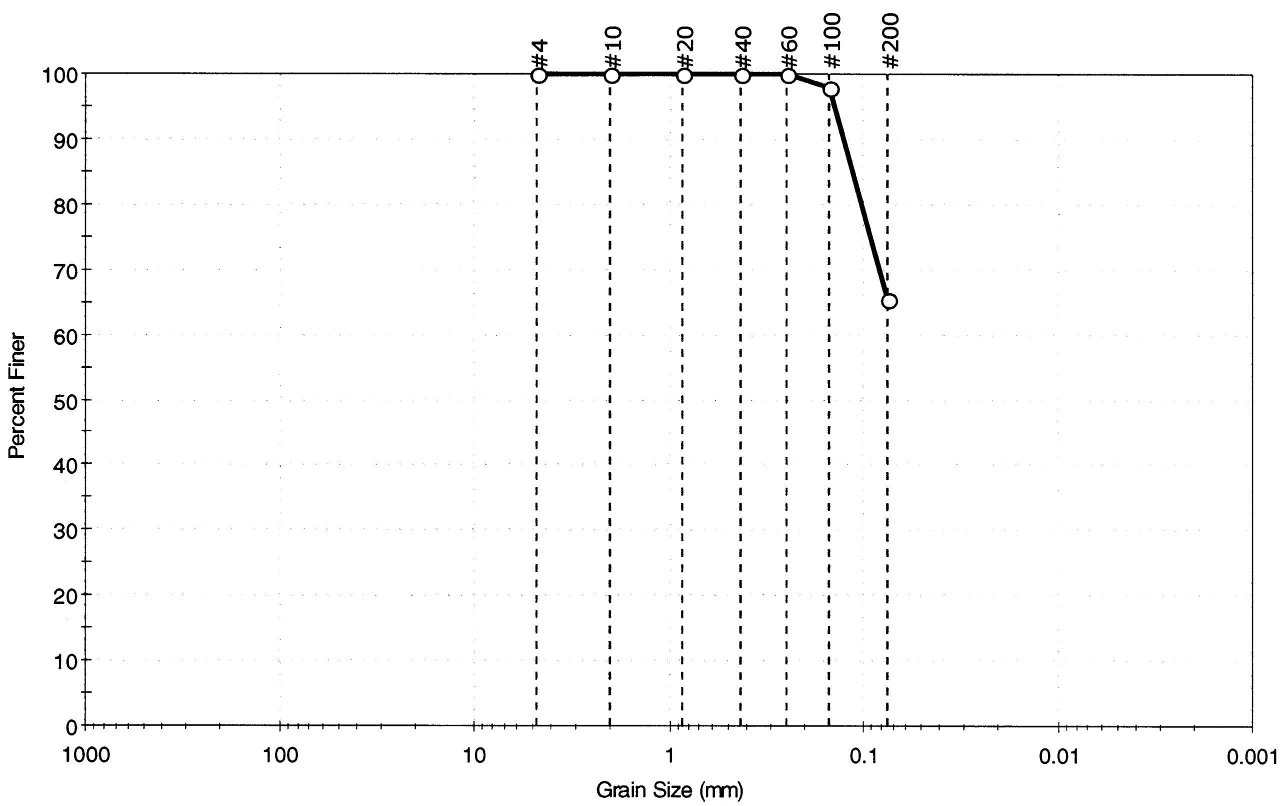
GeoTesting EXPRESS

Client:	AECOM				
Project:	Warwick WWTP Dike Improvement				
Location:	Warwick, RI			Project No:	GTX-11197
Boring ID:	B-4	Sample Type:	jar	Tested By:	jbr
Sample ID:	SS-5	Test Date:	10/05/11	Checked By:	jdt
Depth :	20-22 ft	Test Id:	219506		
Test Comment:	---				
Sample Description:	Moist, olive sandy silt				
Sample Comment:	---				

Particle Size Analysis - ASTM D 422-63 (reapproved 2002)



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	34.5	65.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	98		
#200	0.075	65		

Coefficients

D_{85} =0.1136 mm	D_{30} =N/A
D_{60} =N/A	D_{15} =N/A
D_{50} =N/A	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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Appendix B
Permitting Matrix

AECOM

Project Name:	Warwick Sewer Authority Flood Protection	Date:	September 9, 2011
Location:	Arthur W. Devine Blvd., Warwick, RI	Received By:	Meredith Washington
Person Completing Desktop Evaluation:	Karen Hanecak	Project Number:	60219451

Project Scope:

The purpose of the Project is to elevate and improve flood protection measures surrounding the Warwick Waste Water Treatment Facility (WWTF). For the purposes of this review, AECOM has assumed the final alignment of the flood control levee will follow the existing levee- alignment with some improvements and modifications. AECOM has assumed that improvements to the levee will extend toward the developed portion of the site, extending away from adjacent wetland areas. The sections below provide a summary of environmental resources and permitting requirements based solely on a desktop review of available published resources. AECOM recommends conducting a detailed field review of the site in order to further refine the Project site environmental resources, Project impacts, and permitting requirements.

Desktop Evaluation Checklist (indicate if Yes):

Freshwater Wetlands/Waterbodies w/in 100 feet of proposed work site:	Yes. The existing flood control levee is bounded by wetlands on all sides (except the southern portion of the site adjacent to Interstate 95). A portion of the flood control levee extends into the 200-foot Riverbank wetland of the Pawtuxet River.
Coastal Resources or Freshwater Wetlands in the vicinity of the coast w/in 200 feet of the proposed work site:	No. The Project is not located within RI Coastal Resources Management Council Jurisdiction (RIDEM Environmental Resource Map).
Rare, Threatened, Endangered Species Habitat in proximity of proposed work site:	No Federally-listed rare, threatened, or endangered species are listed in the City of Warwick (USFWS online project review, http://www.fws.gov/newengland/EndangeredSpec-Consultation_Project_Review.htm) No State-designated Natural Heritage areas are located in vicinity of the Project (RIDEM Environmental Resource Map).
FEMA Floodplain designations w/in 100 feet of proposed work site:	Yes. The developed portion of the WWTF itself is located in Zone X. The existing levee demarcates the transition between Zone X and Zone AE (100 year floodplain) to the northeast and southwest of the site. The floodway of the Pawtuxet River runs adjacent to the existing level to the west. (FEMA Map Panel 44003C0127G).
Designated Recreation/Conservation Area, Parkland, Open Space w/in 100 feet of proposed work site:	No (RIDEM Environmental Resource Map).
Historic Site or District located w/in 100 feet of proposed work site:	No (RIDEM Environmental Resource Map).

US EPA Regulated Facilities & Waste Management Sites:	No (RIDEM Environmental Resource Map).
US / State Highways:	Project site is located adjacent and to the west of Interstate 95.
Railroads:	No.
Local Erosion and Sediment Control Ordinances:	Yes. Chapter 68 of the City of Warwick Code of Ordinances. All plans for projects undertaken by the City through private contractors shall include in the specifications and in the contract documents the requirements of Chapter 68, Soil Erosion and Sediment Control.

Preliminary Field Review Conducted (indicate if Yes):

No

Permitting Requirements:

The extent of Project impacts to wetlands, water resources and floodplains is currently unknown. As the Project design continues to be developed and defined, impacts to these resources will be determined. Based on the above desktop review and extent of environmental resources on or adjacent to the site, the following permitting may be required for the Project:

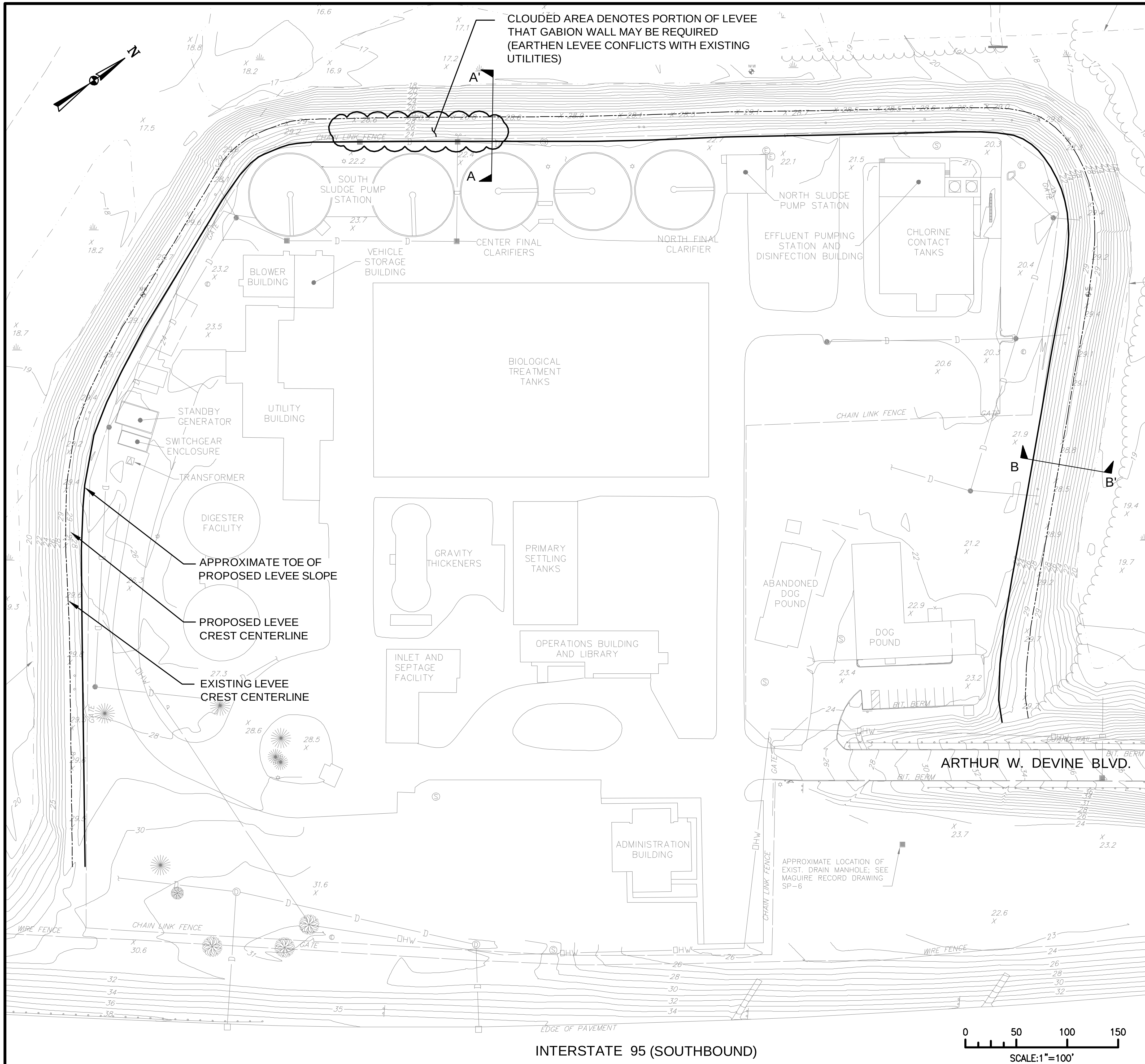
- Rhode Island Department of Environmental Management (RIDEM) Freshwater Wetlands – depending upon the extent and nature of the impact to freshwater wetlands, the following levels of permitting may be required for the Project:
  - Request for a Preliminary Determination for permit issuance for an insignificant alteration of freshwater wetlands. Insignificant alteration is a proposed alteration that is limited in scope, area or duration, which results in no more than a minimal change or modification to the characteristics, functions or values of any freshwater wetlands and is not random, unnecessary or undesirable.
  - Application to Alter Freshwater Wetlands to obtain a permit for a significant alteration of freshwater wetlands. Significant alteration is one that appears to present more than a minimal change or modification to the characteristics, functions or values of any freshwater wetlands; may be detrimental to the basic natural capabilities or values associated with any freshwater wetland; or appears to be random, unnecessary or undesirable.
- United States Army Corps of Engineers (USACE) permitting for work in wetlands and waters of the United States. New Fill / Excavation Discharges:
  - Category 1 – non reporting – <5,000 square feet of waterway/wetland fill and secondary impacts.
  - Category 2 – reporting – 5,000 square feet to 1 acre waterway/wetland fill and secondary impacts.
  - Individual Permit – ≥1 acre waterway/wetland fill and secondary impacts.
- RIDEM Water Quality Certification (WQC)
  - Section 401 WQC conditionally granted under the USACE Category 1 and 2 processes. If the Project falls within these permitting categories no Individual WQC is required.
  - If ≥1 acre Individual WQC may be required.

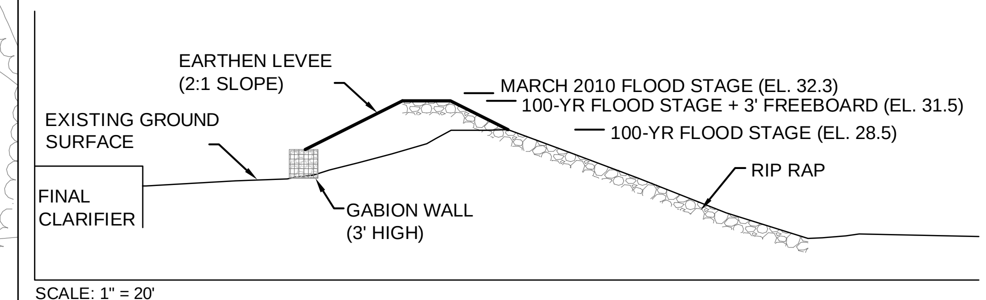
- RIDEM Rhode Island Pollutant Discharge Elimination System (RIPDES)
  - Construction General Permit - Construction activities which disturb one (1) or more acres of land and where storm water runoff is directed, via a point source, into a separate storm sewer system or into the waters of the State, are required to seek coverage under a RIPDES storm water permit.
  - RIPDES Stormwater Discharge Associated with Industrial Activity depending upon the nature of modifications (if any) to the WWTF drainage system. Assumed facility already has a RIPDES/NPDES Permit for Industrial Activity and discharge as a point source to the Pawtuxet River.
- City of Warwick – development of a Soil Erosion and Sediment Control Plan.

Findings / Recommendations:

- Compensatory wetland and flood storage mitigation will be required if impacts to these resources are not considered De Minimus.
- Rhode Island Department of Transportation (RIDOT) - should the levee improvements and construction activities extend into RIDOT property, additional permitting and coordination may be required.

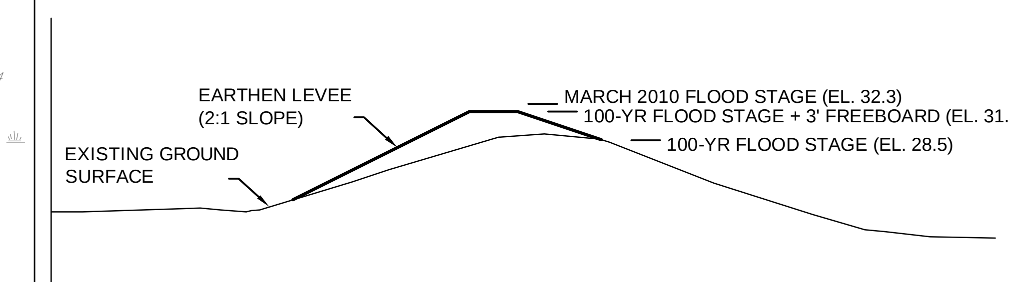
Appendix C
Adjusted Alignment and Typical Levee Sections





SCALE: 1" = 20'

SECTION A - A'
TYPICAL EARTHEN LEVEE WITH GABION WALL



SCALE: 1" = 20'

SECTION B - B'
TYPICAL EARTHEN LEVEE

NOTES:

1. FLOOD STAGE ELEVATIONS (NGVD) ARE AS FOLLOWS:

100 YEAR FLOOD STAGE:	28.5	(PRELIMINARY ESTIMATE BY USGS)
100 YEAR FLOOD STAGE + 3 FEET:	31.5	
MARCH 2010 FLOOD STAGE:	32.3	(AS REPORTED BY WSA)

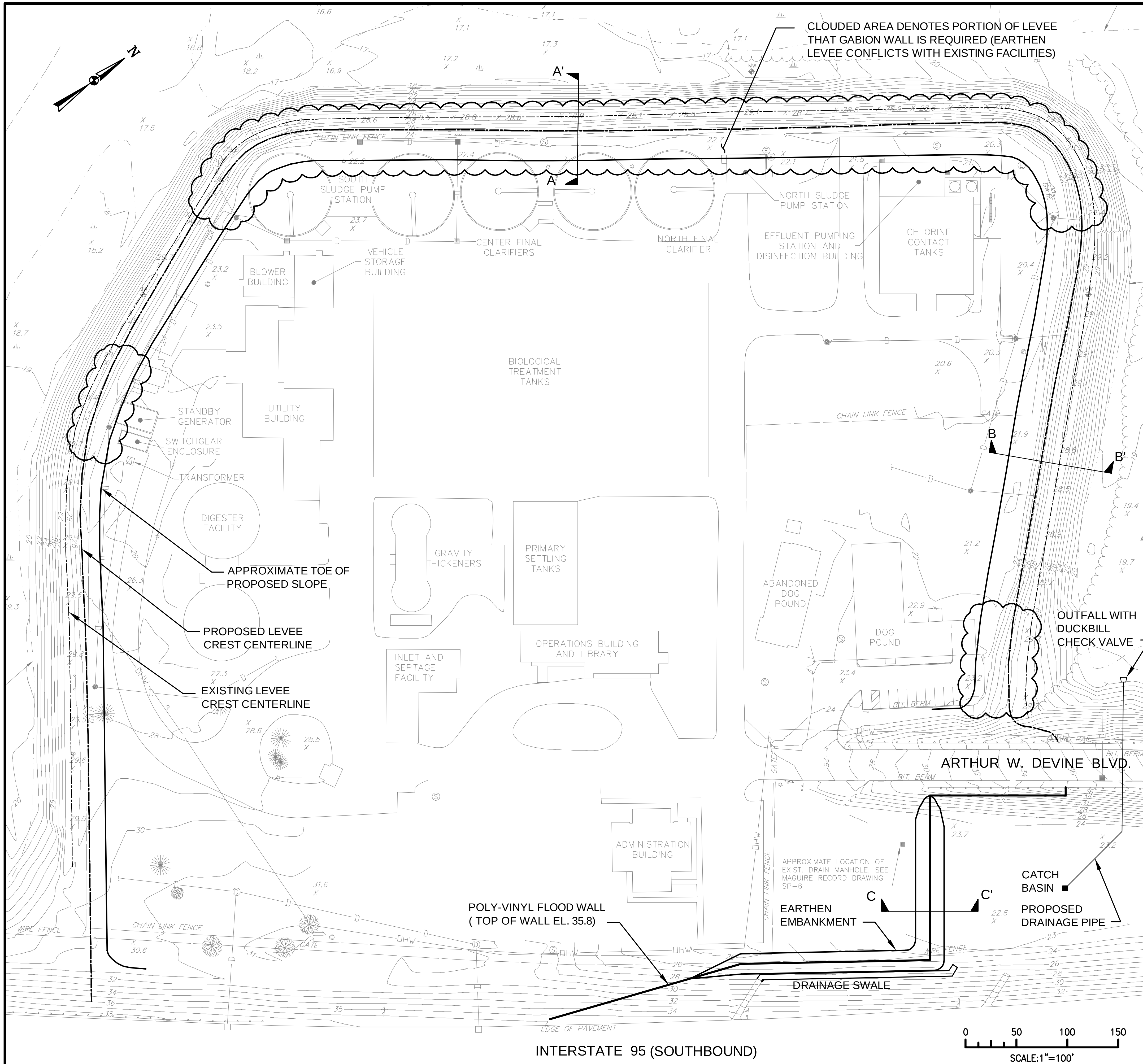
2. VERTICAL DATUM: NGVD 29; HORIZONTAL DATUM: RHODE ISLAND STATE PLANE - NAD 83.
3. EXISTING CONDITIONS SURVEY PERFORMED BY GAROFALO AND ASSOCIATES IN NOVEMBER 2011.

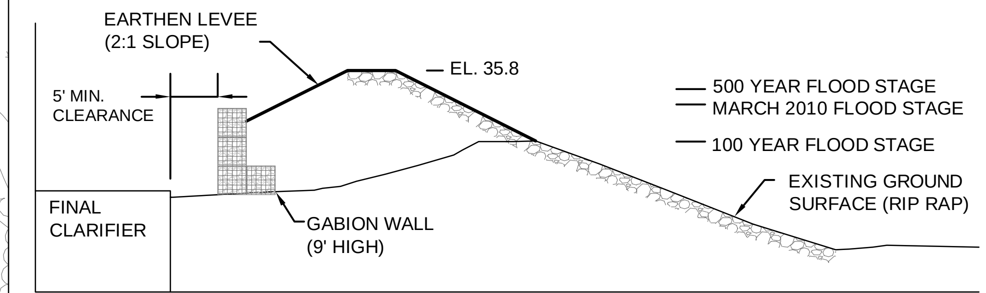
FIGURE C-1

ALTERNATIVE 1 CONCEPTUAL DESIGN
100 YEAR EARTHEN LEVEE

AECOM

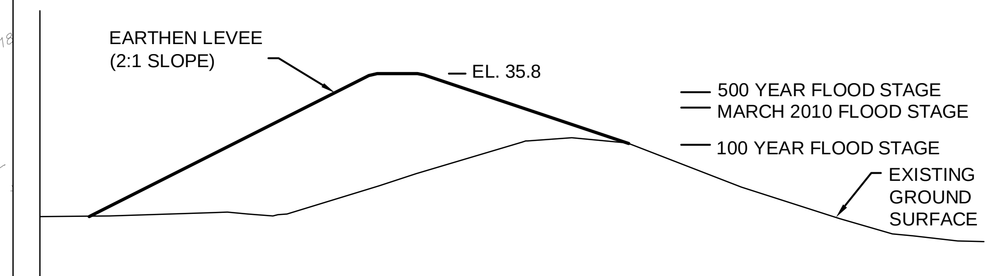
JULY 2012





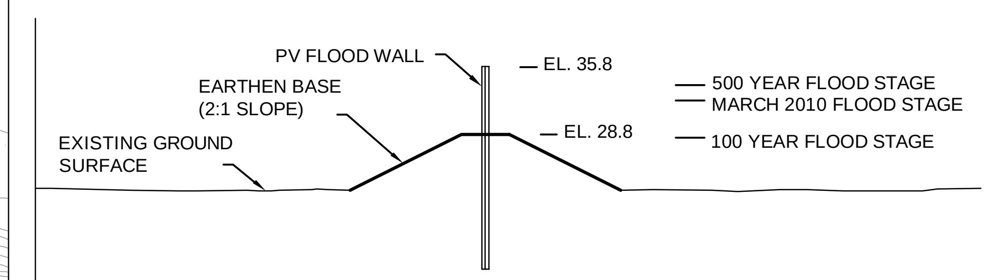
SCALE: 1" = 20'

SECTION A - A'
TYPICAL EARTHEN LEVEE WITH GABION WALL



SCALE: 1" = 20'

SECTION B - B'
TYPICAL EARTHEN LEVEE



SCALE: 1" = 20'

SECTION C - C'
TYPICAL FLOOD WALL

NOTES:

1. FLOOD STAGE ELEVATIONS (NGVD) ARE AS FOLLOWS:

100 YEAR FLOOD STAGE:	28.5	(PRELIMINARY ESTIMATE BY USGS)
MARCH 2010 FLOOD STAGE:	32.3	(AS REPORTED BY WSA)
500 YEAR FLOOD STAGE:	33.9	(PRELIMINARY ESTIMATE BY USGS)

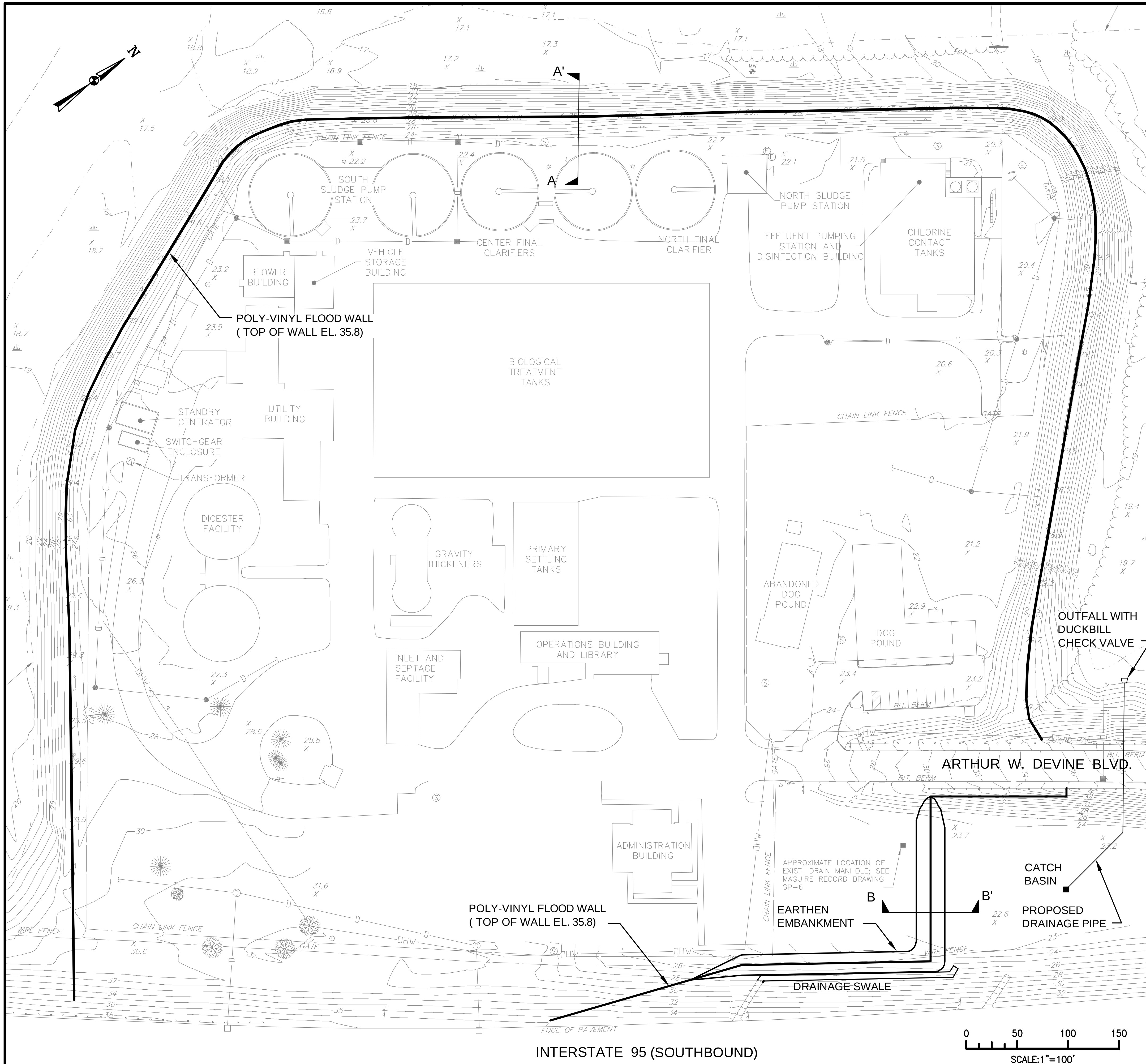
2. VERTICAL DATUM: NGVD 29; HORIZONTAL DATUM: RHODE ISLAND STATE PLANE - NAD 83.
3. EXISTING CONDITIONS SURVEY PERFORMED BY GAROFALO AND ASSOCIATES IN NOVEMBER 2011.

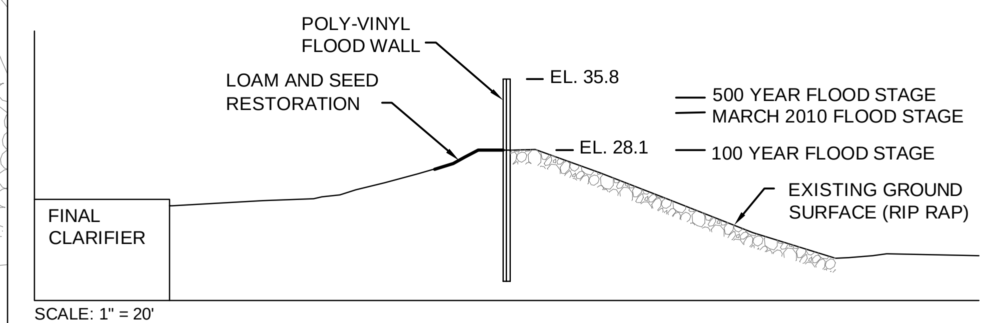
FIGURE C-2

ALTERNATIVE 2 CONCEPTUAL DESIGN
500 YEAR EARTHEN LEVEE

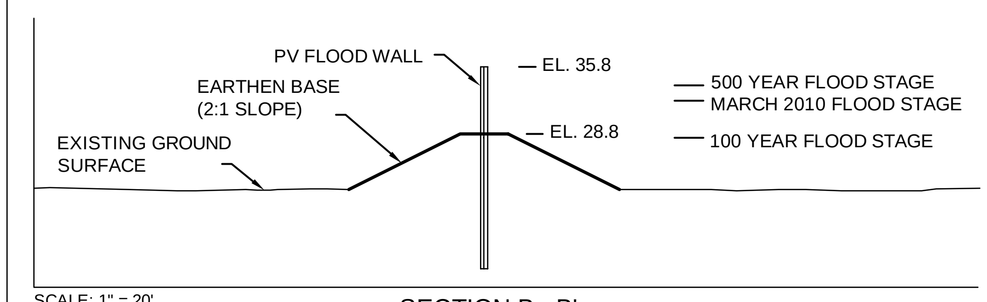
AECOM

JULY 2012





SECTION A - A'
TYPICAL FLOOD WALL SECTION



SECTION B - B'
TYPICAL FLOOD WALL SECTION

NOTES:

1. FLOOD STAGE ELEVATIONS (NGVD) ARE AS FOLLOWS:

100 YEAR FLOOD STAGE:	28.5	(PRELIMINARY ESTIMATE BY USGS)
MARCH 2010 FLOOD STAGE:	32.3	(AS REPORTED BY WSA)
500 YEAR FLOOD STAGE:	33.9	(PRELIMINARY ESTIMATE BY USGS)

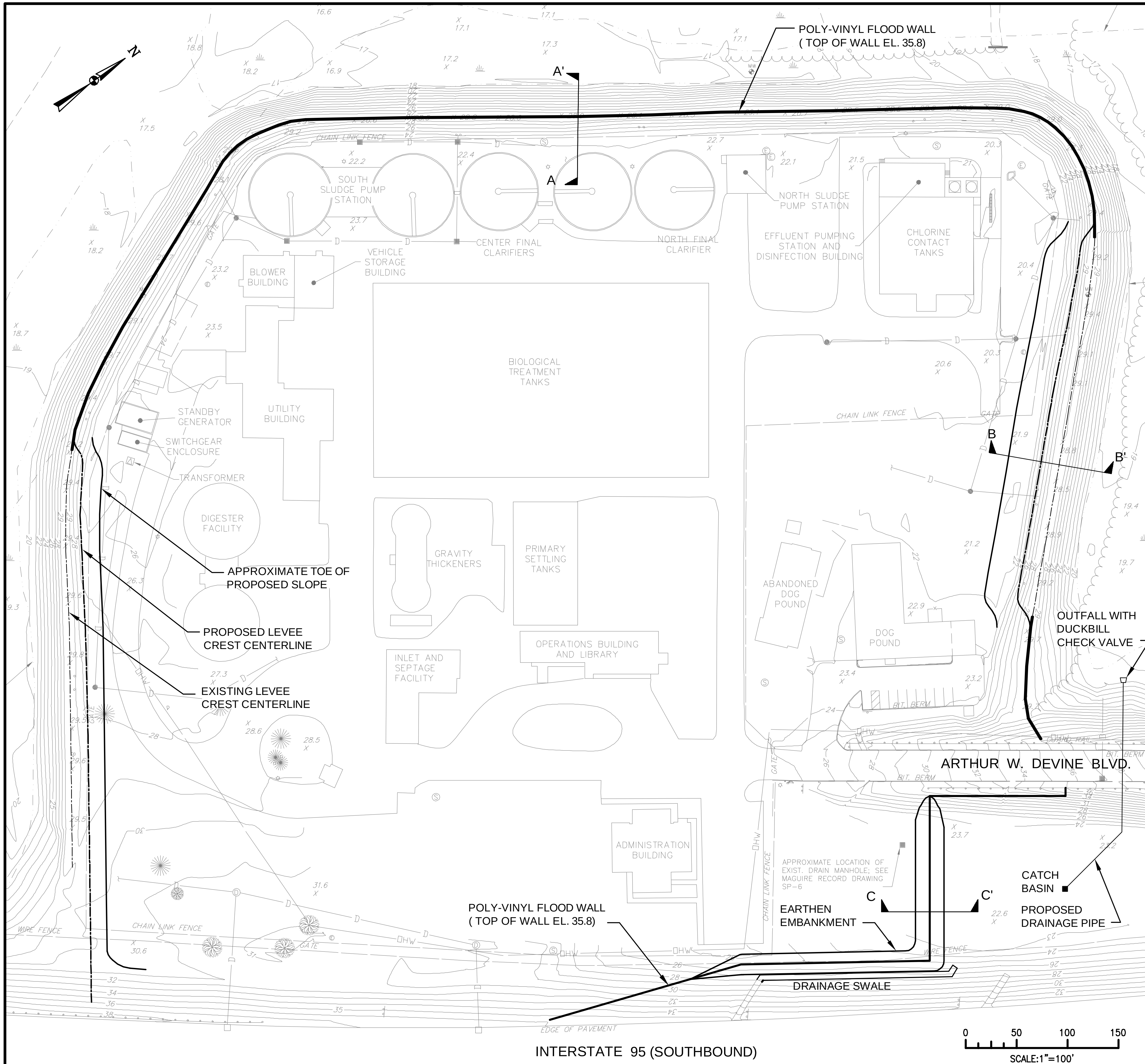
2. VERTICAL DATUM: NGVD 29; HORIZONTAL DATUM: RHODE ISLAND STATE PLANE - NAD 83.
3. EXISTING CONDITIONS SURVEY PERFORMED BY GAROFALO AND ASSOCIATES IN NOVEMBER 2011.

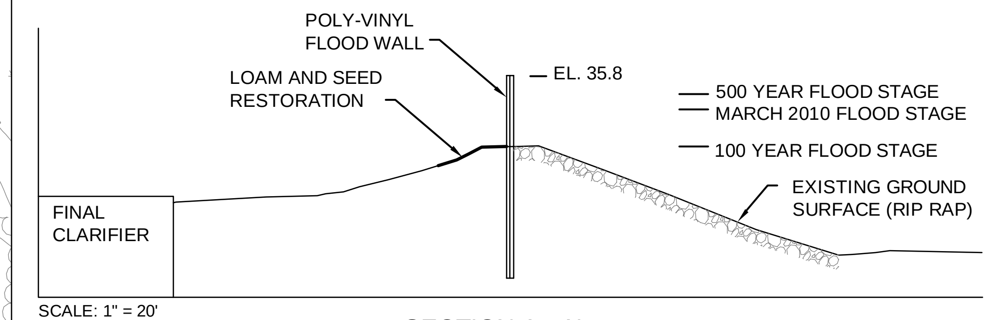
FIGURE C-3

ALTERNATIVE 3 CONCEPTUAL DESIGN
500 YEAR POLY-VINYL FLOOD WALL

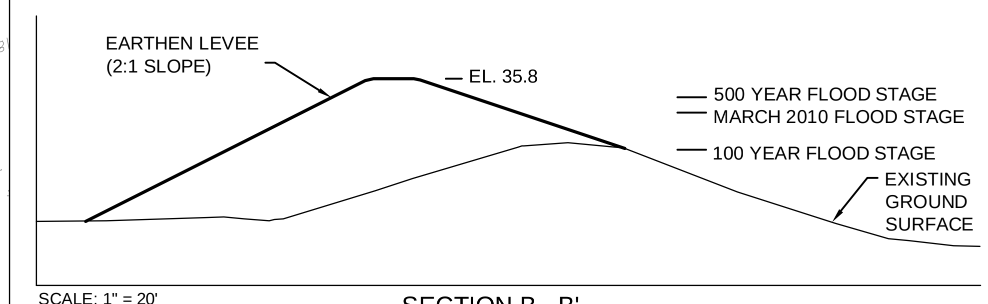
AECOM

JULY 2012

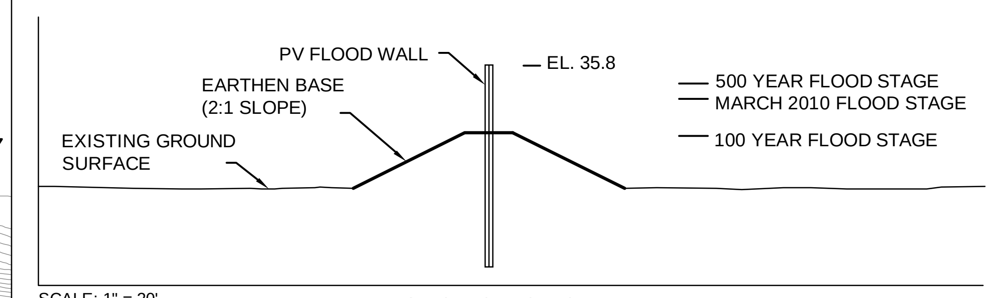




SECTION A - A'
TYPICAL FLOOD WALL SECTION



SECTION B - B'
TYPICAL EARTHEN LEVEE



SECTION C - C'
TYPICAL FLOOD WALL SECTION

NOTES:

1. FLOOD STAGE ELEVATIONS (NGVD) ARE AS FOLLOWS:

100 YEAR FLOOD STAGE:	28.5	(PRELIMINARY ESTIMATE BY USGS)
MARCH 2010 FLOOD STAGE:	32.3	(AS REPORTED BY WSA)
500 YEAR FLOOD STAGE:	33.9	(PRELIMINARY ESTIMATE BY USGS)

2. VERTICAL DATUM: NGVD 29; HORIZONTAL DATUM: RHODE ISLAND STATE PLANE - NAD 83.
3. EXISTING CONDITIONS SURVEY PERFORMED BY GAROFALO AND ASSOCIATES IN NOVEMBER 2011.

FIGURE C-4

ALTERNATIVE 4 CONCEPTUAL DESIGN
500 YEAR EARTHEN LEVEE WITH FLOOD WALL

AECOM

JULY 2012

Appendix D
Detailed Levee Construction Estimates

10:20 AM
JOB #: 60219451
DATE: April 17, 2012
LOCATION: Warwick, RI
PREPARED BY: R. Mastrogiacomo/M. Washington

AECOM
Construction Cost Estimate
10% Opinion of Cost
Flood Protection and Mitigation Design

CLIENT : WSA
PROJECT : Flood Protection and Mitigation Design
ACCURACY: ± 25 %
ENR. INDEX: 9173

ACCOUNT NO.	DESCRIPTION	QUAN	UN	MANHOURS MHR/ UNIT	TOTAL MH	MATERIAL UNIT COST	TOTAL MATL	LABOR WAGE RATE	TOTAL LABOR	EQUIPMENT UNIT RATE	TOTAL EQUIP	TOTAL DIRECT COST
-1-	**Alternative 1 - 100 Year Earthen Levee**											
	Levee											
	Loam (6")	915	CY	0.10	92	23.00	21,045	$64.75	5,924	2.00	1,830	$28,799
	Seeding	55	MSF	0.2	11	12.00	660	$64.75	712	4.00	220	$1,592
	General Backfill	1,170	CY	0.15	176		0	$64.75	11,363	1.60	1,872	$13,235
	Impervious Backfill	1,950	CY	0.200	390		0	$64.75	25,251	2.00	3,900	$29,151
	Crushed Stone	275	CY	0.200	55	15.00	4,125	$64.75	3,561	2.00	550	$8,236
	F&I Rip Rap	210	CY	0.30	63	30.00	6,300	$64.75	4,079	12.00	2,520	$12,899
	Remove and Reinstall Existing Rip Rap	485	CY	0.50	243		0	$64.75	15,701	20.00	9,700	$25,401
	Gabions (3' x3' x 5')	100	EA	4.30	430	165.00	16,500	$64.75	27,841	60.00	6,000	$50,341
	Geotextile Fabric	1,450	SY	0.01	15	2.00	2,900	$64.75	939		0	$3,839
	Temporary Access Road											
	Gravel (12" Deep)	1,300	SY	0.008	10	8.55	11,115	$64.75	673	0.50	650	$12,438
	Haybales & Silt Fence	2,400	LF	0.02	48	7.00	16,800	$64.75	3,108	0.10	240	$20,148
	Restoration (Seed)	35	MSF	0.2	7	12.00	421	$64.75	455	4.00	140	$1,016
	SUBTOTAL DIRECT COSTS				1,538		79,866		99,606		27,622	$207,100
	CONTINGENCY	30.00%										$62,130
	SUBTOTAL GENERAL CONTRACTOR											$269,200
	GENERAL CONTRACTOR OVERHEAD&PROFIT	20.00%										$53,840
	TOTAL CONSTRUCTION COST											**$330,000**

10:20 AM

JOB #: 60219451
DATE: April 17, 2012
LOCATION: Warwick, RI
PREPARED BY: R. Mastrogiacomo/M. Washington

AECOM
Construction Cost Estimate
10% Opinion of Cost
Flood Protection and Mitigation Design

CLIENT : WSA
PROJECT : Flood Protection and Mitigation Design
ACCURACY: ± 25 %
ENR. INDEX: 9173

ACCOUNT NO.	DESCRIPTION	QUAN	UN	MANHOURS MHR/ UNIT	TOTAL MH	MATERIAL UNIT COST	TOTAL MATL	LABOR WAGE RATE	TOTAL LABOR	EQUIPMENT UNIT RATE	TOTAL EQUIP	TOTAL DIRECT COST
-1-	**Alternative 2 - 500 Year Earthen Levee**											
	Levee											
	Loam (6")	1,800	CY	0.10	180	23.00	41,400	$64.75	11,654	2.00	3,600	$56,654
	Seeding	72	MSF	0.2	14	12.00	864	$64.75	932	4.00	288	$2,084
	General Backfill	7,950	CY	0.15	1,193		0	$64.75	77,209	1.60	12,720	$89,929
	Impervious Backfill	4,410	CY	0.200	882		0	$64.75	57,106	2.00	8,820	$65,926
	Crushed Stone	565	CY	0.200	113	15.00	8,475	$64.75	7,316	2.00	1,130	$16,921
	F&I Rip Rap	1,100	CY	0.30	330	30.00	33,000	$64.75	21,366	12.00	13,200	$67,566
	Remove and Reinstall Existing Rip Rap	990	CY	0.50	495		0	$64.75	32,049	20.00	19,800	$51,849
	Gabion Wall (3' x 3' x 6')	740	EA	4.30	3,182	165.00	122,100	$64.75	206,021	60.00	44,400	$372,521
	Geotextile Fabric	3,280	SY	0.01	33	2.00	6,560	$64.75	2,124		0	$8,684
	Relocate Existing Drain Manhole	1	EA	40.00	40		0	$64.75	2,590	500.00	500	$3,090
	12" DI Drain	50	LF	0.38	19	50.00	2,500	$64.75	1,230	6.00	300	$4,030
	Remove and Reinstall Existing Chainlink Fence	2,400	LF	0.20	480		0	$64.75	31,078	2.00	4,800	$35,878
	Chainlink Fence	50	LF	0.13	7	44.00	2,200	$64.75	431	1.38	69	$2,700
	Temporary Access Road											
	Gravel (12" Deep)	1,300	CY	0.008	10	8.55	11,115	$64.75	673	0.50	650	$12,438
	Haybales & Silt Fence	2,400	LF	0.02	48	7.00	16,800	$64.75	3,108	0.10	240	$20,148
	Restoration (Seed)	35	MSF	0.2	7	12.00	421	$64.75	455	4.00	140	$1,016
	I-95 Protection											
	PVC Flood Wall (21' feet height)	685	LF									$476,000
	Fill	623	CY	0.15	93	0.00	0	$64.75	6,050	1.60	997	$7,047
	Loam (6")	137	CY	0.10	14	23.00	3,152	$64.75	887	2.00	274	$4,313
	Seeding	7	MSF	0.2	1	12.00	89	$64.75	96	4.00	30	$214
	Armor 2 Manholes	2	EA									$25,000
	12" Drainage Pipe	80	LF	0.38	30	50.00	4,000	$64.75	1,968	6.00	480	$6,448
	Catch Basin (6' Deep)	1	EA	8.00	8	5,000.00	5,000	$64.75	518	12.00	12	$5,530
	Trench Excavation	89	CY	0.15	13	0.00	0	$64.75	863	1.60	142	$1,005
	Backfill	89	CY	0.15	13	0.00	0	$64.75	863	1.60	142	$1,005
	Pipe Jacking Mobilization	1	LS	0.00	0	0.00	0	$64.75	0	0.00	0	$6,000
	Pipe Jacking	150	LF	0.50	75	305.00	45,750	$64.75	4,856	12.00	1,800	$52,406
	Duckbill Outlet	1	EA	4.00	4	3,000.00	3,000	$64.75	259	2.00	2	$3,261
	SUBTOTAL DIRECT COSTS				7,285		306,426		471,703		114,536	$1,399,700
	CONTINGENCY	30.00%										$419,910
	SUBTOTAL GENERAL CONTRACTOR											$1,819,600
	GENERAL CONTRACTOR OVERHEAD&PROFIT	20.00%										$363,920
	TOTAL CONSTRUCTION COST											**$2,200,000**

10:20 AM
JOB #: 60219451
DATE: April 17, 2012
LOCATION: Warwick, RI
PREPARED BY: R. Mastrogiacomo/M. Washington

AECOM
Construction Cost Estimate
10% Opinion of Cost
Flood Protection and Mitigation Design

CLIENT : WSA
PROJECT : Flood Protection and Mitigation Design
ACCURACY: ± 25 %
ENR. INDEX: 9173

ACCOUNT NO.	DESCRIPTION	QUAN	UN	MANHOURS		MATERIAL		LABOR		EQUIPMENT		TOTAL DIRECT COST
				MHR/ UNIT	TOTAL MH	UNIT COST	TOTAL MATL	WAGE RATE	TOTAL LABOR	UNIT RATE	TOTAL EQUIP	
-1-	**Alternative 3 - 500 Year Flood Wall**											
	Flood Wall											
	PVC Flood Wall (21' feet height)	2,350	LF									$1,604,000
	Loam (6")	676	CY	0.10	68	23.00	15,546	$64.75	4,376	2.00	1,352	$21,274
	Seeding	37	MSF	0.2	7	12.00	443	$64.75	478	4.00	148	$1,068
	Reinstall Existing Rip Rap	555	CY	0.20	111	12.00	6,660	$64.75	7,187	12.00	6,660	$20,507
	I-95 Protection											
	PVC Flood Wall (21' feet height)	685	LF									$476,000
	Fill	623	CY	0.15	93	0.00	0	$64.75	6,050	1.60	997	$7,047
	Loam (6")	137	CY	0.10	14	23.00	3,152	$64.75	887	2.00	274	$4,313
	Seeding	7	MSF	0.2	1	12.00	89	$64.75	96	4.00	30	$214
	Armor 2 Manholes	2	EA									$25,000
	12" Drainage Pipe	80	LF	0.38	30	50.00	4,000	$64.75	1,968	6.00	480	$6,448
	Catch Basin (6' Deep)	1	EA	8.00	8	5,000.00	5,000	$64.75	518	12.00	12	$5,530
	Trench Excavation	89	CY	0.15	13	0.00	0	$64.75	863	1.60	142	$1,005
	Backfill	89	CY	0.15	13	0.00	0	$64.75	863	1.60	142	$1,005
	Pipe Jacking Mobilization	1	LS	0.00	0	0.00	0	$64.75	0	0.00	0	$6,000
	Pipe Jacking	150	LF	0.50	75	305.00	45,750	$64.75	4,856	12.00	1,800	$52,406
	Duckbill Outlet	1	EA	4.00	4	3,000.00	3,000	$64.75	259	2.00	2	$3,261
	SUBTOTAL DIRECT COSTS				439		83,640		28,402		12,038	$2,235,100
	CONTINGENCY	30.00%										$670,530
	SUBTOTAL GENERAL CONTRACTOR											$2,905,600
	GENERAL CONTRACTOR OVERHEAD&PROFIT	20.00%										$581,120
	TOTAL CONSTRUCTION COST											**$3,500,000**

10:20 AM
JOB #: 60219451
DATE: April 17, 2012
LOCATION: Warwick, RI
PREPARED BY: R. Mastrogiacomo/M. Washington

AECOM
Construction Cost Estimate
10% Opinion of Cost
Flood Protection and Mitigation Design

CLIENT : WSA
PROJECT : Flood Protection and Mitigation Design
ACCURACY: ± 25 %
ENR. INDEX: 9173

				MANHOURS		MATERIAL		LABOR		EQUIPMENT		TOTAL
ACCOUNT NO.	DESCRIPTION	QUAN	UN	MHR/ UNIT	TOTAL MH	UNIT COST	TOTAL MATL	WAGE RATE	TOTAL LABOR	UNIT RATE	TOTAL EQUIP	DIRECT COST
-1-	**Alternative 4 - 500 Year Earthen Levee & Flood Wall Combination**											
	Flood Wall											
	PVC Flood Wall (21' feet height)	1,323	LF									$918,034
	Loam (6")	381	CY	0.10	38	23.00	8,752	$64.75	2,464	2.00	761	$11,977
	Seeding	21	MSF	0.2	4	12.00	249	$64.75	269	4.00	83	$601
	Reinstall Existing Rip Rap	555	CY	0.20	111	12.00	6,660	$64.75	7,187	12.00	6,660	$20,507
	Levee											
	Loam (6")	900	CY	0.10	90	23.00	20,700	$64.75	5,827	2.00	1,800	$28,327
	Seeding	49	MSF	0.2	10	12.00	588	$64.75	635	4.00	196	$1,419
	General Backfill	1,850	CY	0.15	278		0	$64.75	17,967	1.60	2,960	$20,927
	Impervious Backfill	1,910	CY	0.200	382		0	$64.75	24,733	2.00	3,820	$28,553
	Crushed Stone	115	CY	0.200	23	15.00	1,725	$64.75	1,489	2.00	230	$3,444
	F&I Rip Rap	90	CY	0.30	27	30.00	2,700	$64.75	1,748	12.00	1,080	$5,528
	Remove and Reinstall Existing Rip Rap	200	CY	0.50	100		0	$64.75	6,475	20.00	4,000	$10,475
	Geotextile Fabric	600	SY	0.01	6	2.00	1,200	$64.75	388		0	$1,588
	Relocate Existing Drain Manhole	1	EA	40.00	40		0	$64.75	2,590	500.00	500	$3,090
	12" DI Drain	50	LF	0.38	19	50.00	2,500	$64.75	1,230	6.00	300	$4,030
	Remove and Reinstall Existing Chainlink Fence	1,000	LF	0.20	200		0	$64.75	12,949	2.00	2,000	$14,949
	Chainlink Fence	50	LF	0.13	7	44.00	2,200	$64.75	431	1.38	69	$2,700
	I-95 Protection											
	PVC Flood Wall (21' feet height)	685	LF									$476,000
	Fill	623	CY	0.15	93	0.00	0	$64.75	6,050	1.60	997	$7,047
	Loam (6")	137	CY	0.10	14	23.00	3,152	$64.75	887	2.00	274	$4,313
	Seeding	7	MSF	0.2	1	12.00	89	$64.75	96	4.00	30	$214
	Armor 2 Manholes	2	EA									$25,000
	12" Drainage Pipe	80	LF	0.38	30	50.00	4,000	$64.75	1,968	6.00	480	$6,448
	Catch Basin (6' Deep)	1	EA	8.00	8	5,000.00	5,000	$64.75	518	12.00	12	$5,530
	Trench Excavation	89	CY	0.15	13	0.00	0	$64.75	863	1.60	142	$1,005
	Backfill	89	CY	0.15	13	0.00	0	$64.75	863	1.60	142	$1,005
	Pipe Jacking Mobilization	1	LS	0.00	0	0.00	0	$64.75	0	0.00	0	$6,000
	Pipe Jacking	150	LF	0.50	75	305.00	45,750	$64.75	4,856	12.00	1,800	$52,406
	Duckbill Outlet	1	EA	4.00	4	3,000.00	3,000	$64.75	259	2.00	2	$3,261
	SUBTOTAL DIRECT COSTS				1,587		108,265		102,742		28,338	$1,664,400
	CONTINGENCY	30.00%										$499,320
	SUBTOTAL GENERAL CONTRACTOR											$2,163,700
	GENERAL CONTRACTOR OVERHEAD&PROFIT	20.00%										$432,740
	TOTAL CONSTRUCTION COST											**$2,600,000**

Appendix E
Groundwater Modeling Results

Memorandum

Date: November 22, 2011

To: Jose Ramos

From: Sean Czarniecki

Subject: Warwick WWTF - Modeling of Seepage Under Dike And Potential Collection Options

Distribution: Warren Diesl Project File

Groundwater modeling was utilized to estimate potential seepage flows under a proposed dike at the Wastewater Treatment Facility (WWTF) in Warwick, Rhode Island. Historic flooding has overtopped the dike, resulting in the need for improvements to reduce the potential for flooding in the WWTP facility. As part of the conceptual design, a proposed top of dike elevation (35.8 ft) was utilized to contain a 500-year flood elevation of 33.9 ft. All elevations are NGVD 29.

To quickly estimate conservative impacts of a flood surrounding the WWTF, a 1660 ft x 1660 ft model grid was established which assumed ground surface inside the WWTF at 20 ft, the lowest elevation in the WWTF. Groundwater starting elevation was assumed to be at 19 ft and the flood elevation of 33.9 ft was established surrounding three sides of the WWTF. A 10-ft grid spacing was utilized, along with a total of 11 layers: the top layer represents the flooded zone surrounding the dike, while the lower 10 layers (each 10-ft thick), represent the groundwater aquifer (see Figures 1 and 2).

Due to limited existing data, the horizontal hydraulic conductivity of the soil in the area was assumed to be 30 ft/day (approximately 10^{-2} cm/sec). Vertical hydraulic conductivity was assumed to be 1/10 of the horizontal conductivity; a typical assumption based on how soils were typically deposited historically.

An existing drain at the toe of the existing berm was placed at elevation 18 ft, which is 2 ft below the assumed ground surface (see Figure 3).

A proposed deeper drain was placed further away from the berm to determine if this would limit water from breaking the ground surface inside of the WWTF (see Figure 3). The drain was modeled at various depths. As the existing data are limited, other sensitivity runs were performed, changing the horizontal conductivity, the ratio of vertical to horizontal conductivity, as well as adding recharge (from precipitation during the flooding events) to the WWTF area. Model results were generated every half day for up to 7 days, even though the maximum flood elevation would not be expected to be maintained for that duration. Table 1 presents model results, showing the total groundwater flow rate removed by the drains at that moment in time. Visual presentation of an example model run is shown in Figures 4 (plan view) and 5 (cross-sectional view).

As it is not likely that a continuous drain inside the berm will be possible, an additional model run was performed with drains at 5-ft below ground and vertical wells spaced approximately every 30 feet connected to the drains. Although the model maintains a contiguous drain, its effectiveness is limited due to the shallower depth and the model run is primarily to observe the potential effectiveness of the vertical wells. As these wells would not contain pumps, they were modeled by increasing the hydraulic conductivity in single cells to 10000 ft/day. Vertical conductivity was assumed to be the same as the horizontal conductivity. These results are also presented in Table 1 and a cross-sectional view is presented in Figure 6.

Proposed Model Refinements and Data Collection

The model runs performed were developed to provide a basis for conceptual design, as well as to determine primary data collection needs. Based on the sensitivity results presented, the primary data needs include soil hydraulic conductivity and historic storm/flood timelines with which to establish design-basis assumptions during modeling.

Proposed model refinements include the following:

- Refine the model grid cell size for modeling of wells to more accurately simulate well size;
- Expand model grid extents to limit influence of no-flow model boundaries;
- Review model in area of highway and how water may potentially enter area outside of drain influence (potentially add drains in this area);
- Adjust ground surface elevations in WWTF. This will actually provide additional water storage;
- Account for reduced storage below ground where tanks/foundations exist;
- Adjust existing drain location and elevations for accuracy;
- Apply any field data collected to adjust hydraulic conductivity;
- Rather than applying a continuous new drain, model a realistic scenario which likely is constructed in sections; and
- Review historic storm and flood timeline data, as well as precipitation data, to refine in model.

TABLE 1. WARWICK WWTF CONCEPTUAL DESIGN GROUNDWATER MODEL RESULTS

Scenario	Basic description	Kh (ft/d)	Kh/Kv	Recharge (in/d)	Day that water exceeds El. 20 ft in WWTF	Drain Flow (gpm)				Model Run
						Day 0.5	Day 3	Prior to "breakthrough" (day)		
1	Existing Drains @ El. 18 ft (2 ft deep)	30	10	0	Day 2	780	1243	1060	(Day 1.5)	[07]
2	Add New Drains @ El. 12 ft (8 ft deep)	30	10	0	Day 7	2303	1710	1831	(Day 6.5)	[08]
3	Similar to Scenario 2; change Kh	100	10	0	Day 2.5	6128	6248	6030	(Day 2)	[09]
4	Similar to Scenario 2; change Kh/Kv	30	5	0	Day 6	2754	2333	2423	(Day 5.5)	[10]
5	Similar to Scenario 2; change Kh/Kv	30	1	0	> 7 days	4730	4344	4360	(Day 7)	[11]
6	Similar to Scenario 2; add recharge	30	10	2	Not clear due to influence of model boundaries	2443	1994	1827	(Day 1.5)	[12]
7	Similar to Scenario 2; raise drain (5 ft deep)	30	10	0	Day 5	1769	1563	1646	(Day 4.5)	[13]
8	Similar to Scenario 7; add "wells" every 30 ft, 60 ft deep	30	10	0	> 7 days	3244	2780	2834	(Day 7)	[14]

Notes
Kh = horizontal hydraulic conductivity of soil
Kv = vertical hydraulic conductivity of soil
Elevations in NGVD 29
WWTF = wastewater treatment facility

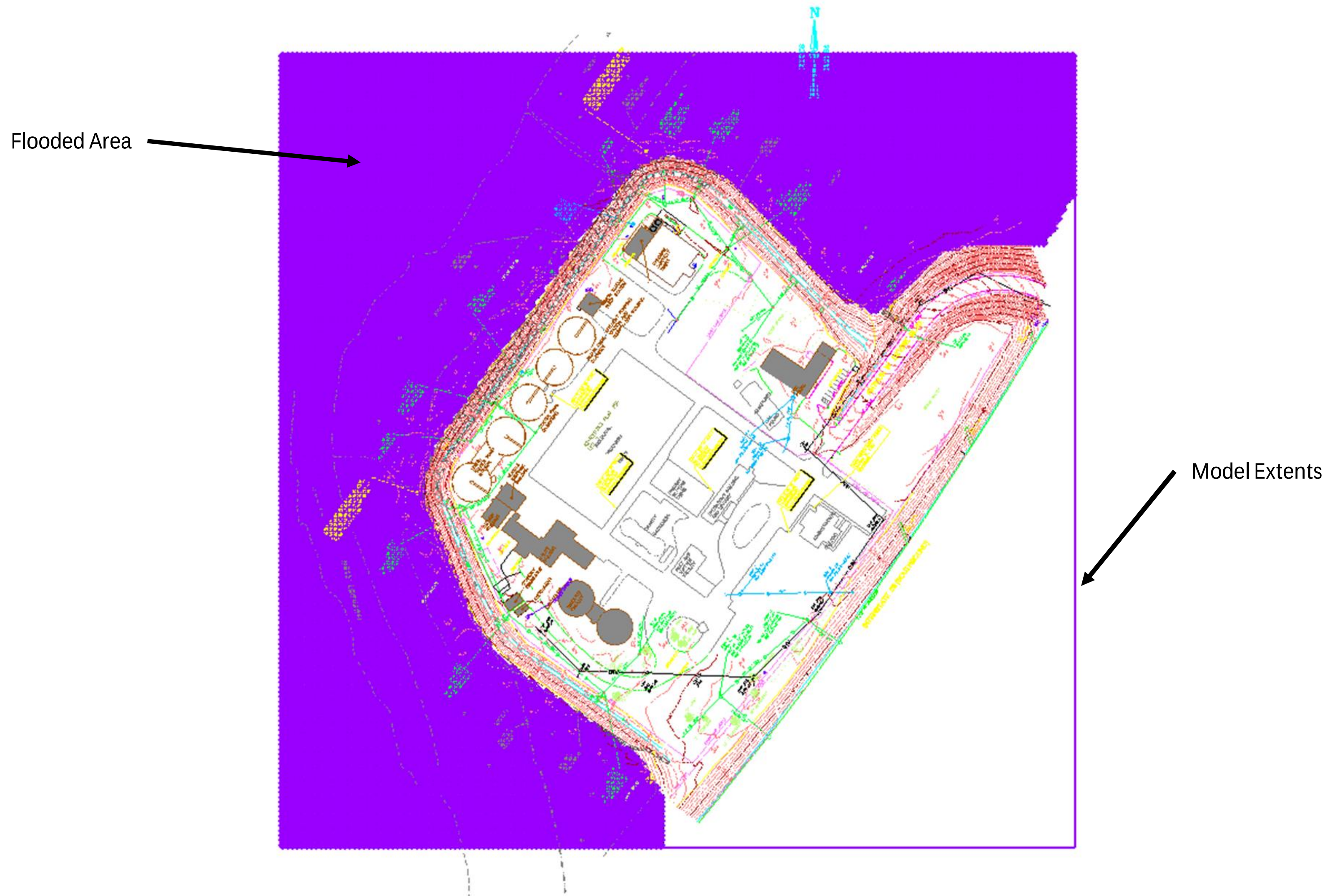


Figure 1. Model Extents and Flooded Area

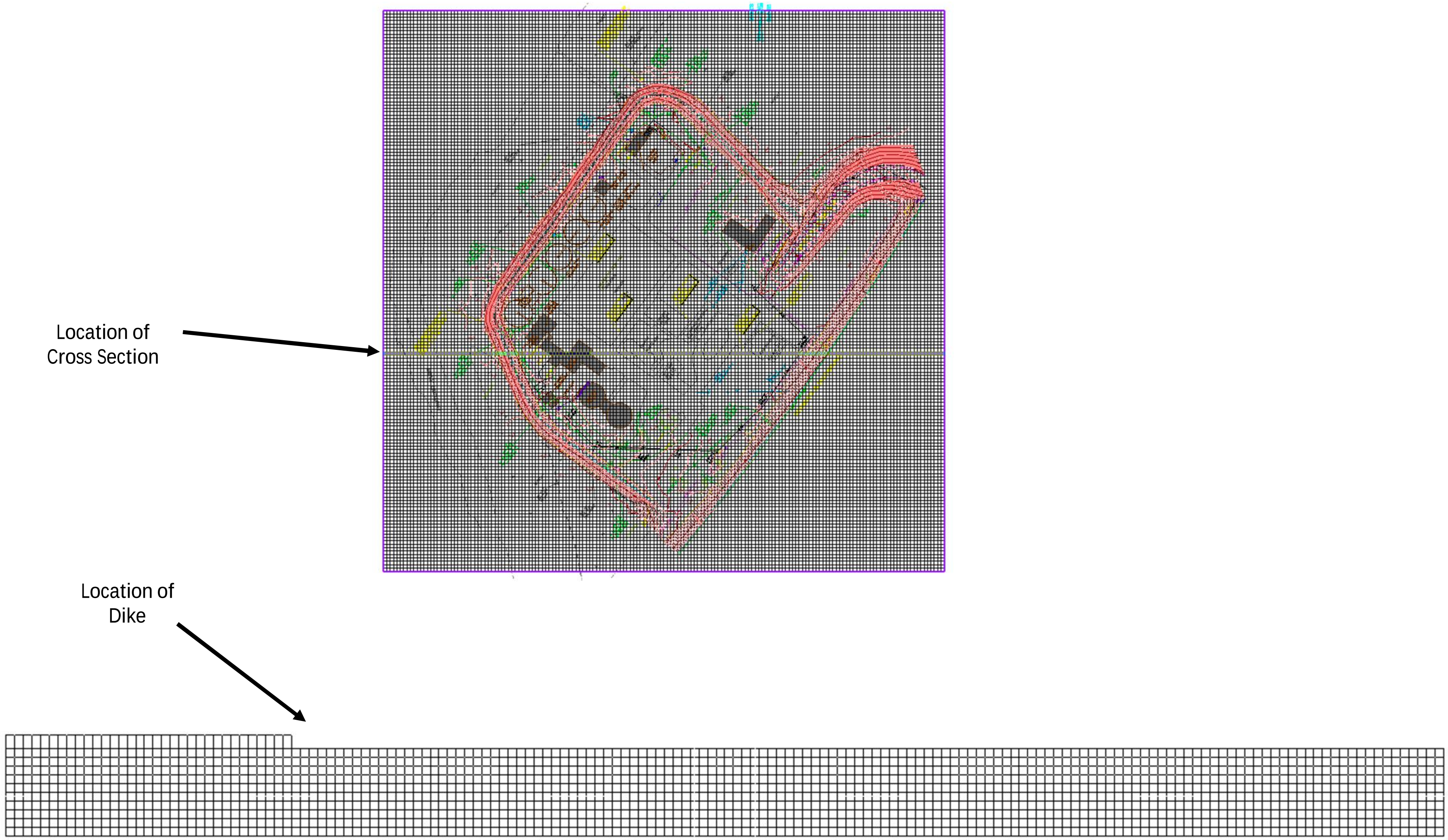


Figure 2. Model Grid Design

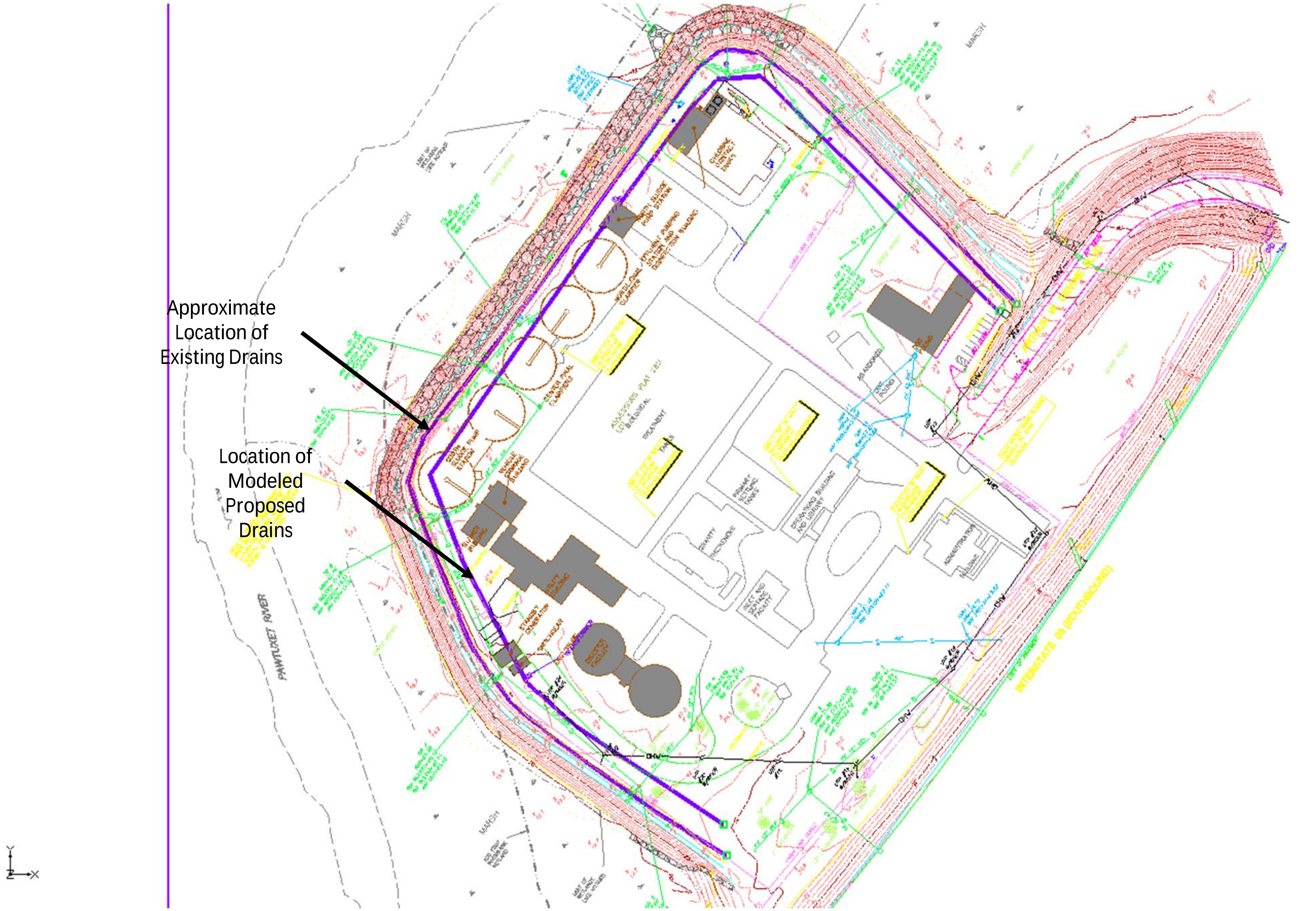


Figure 3. Drain locations

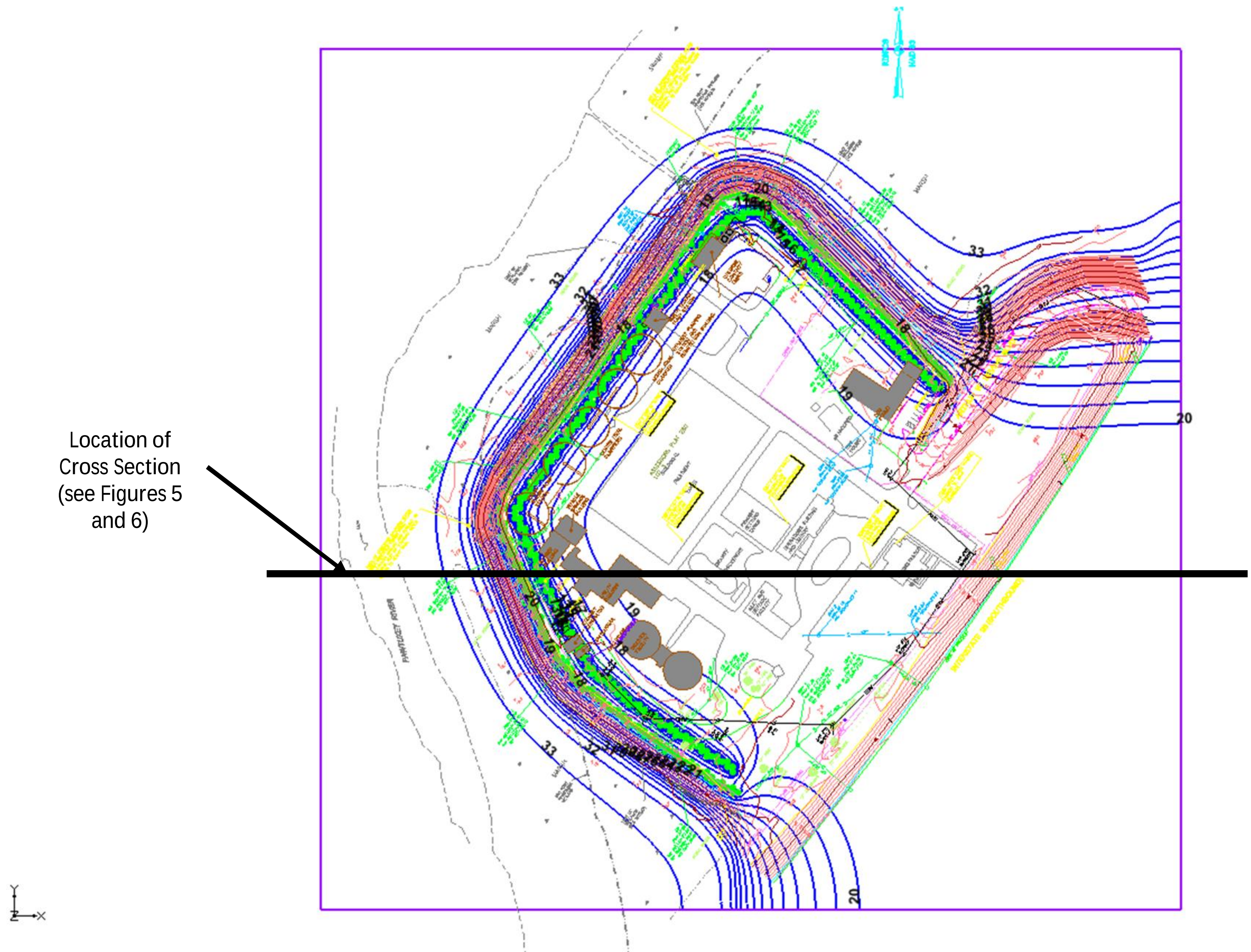


Figure 4. Example model results (Scenario 2, Layer 2 – Day 3)

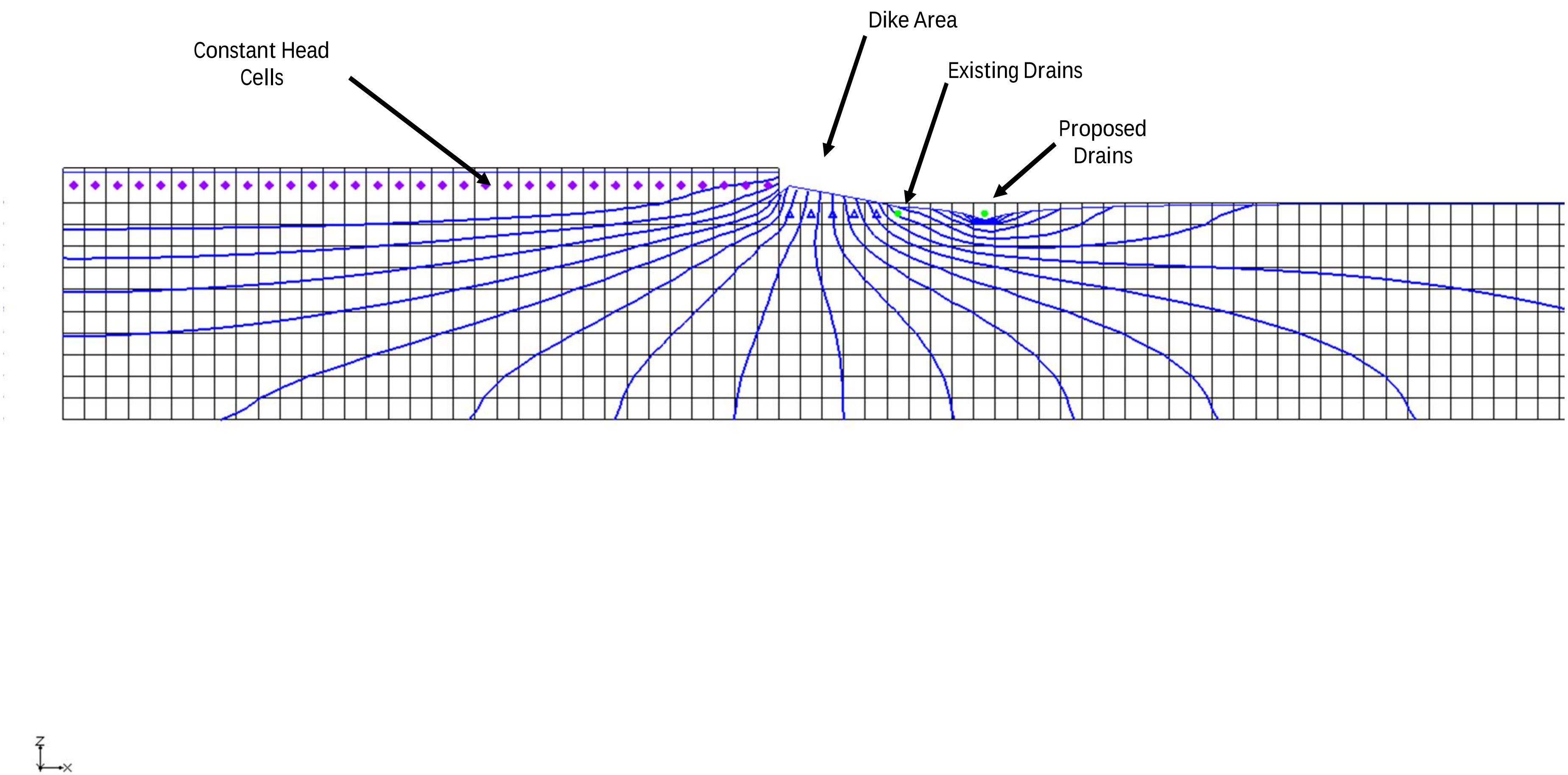


Figure 5. Example model results cross section (Scenario 2 – Day 3)

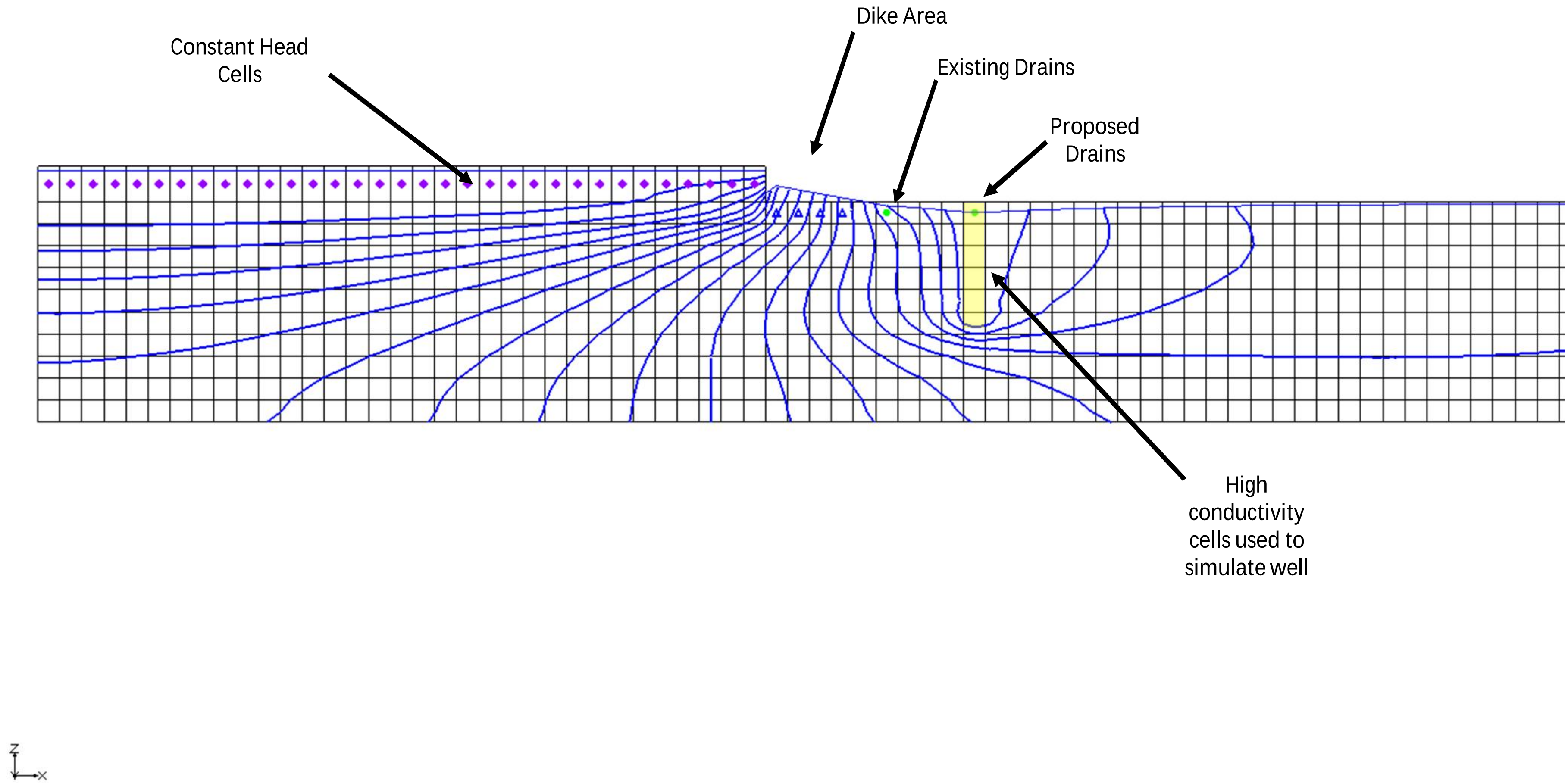


Figure 6. Model results cross section (Scenario 7 – Day 3)

Appendix F
Rainfall Modeling Results

Memorandum

To	Doug Gove, Meredith Washington, Erik Meserve, Dennis Setzo
CC	
Subject	Hydrology – Warwick WWTP Interior Drainage
From	Brent McCarthy, Dave Markwood
Date	April 2, 2012

Purpose of Memorandum

The March 2010 flooding at the Warwick WWTF occurred when the stage on the Pawtuxet River exceeded the elevation of the levee surrounding the facility. To minimize the chances of future flooding from the Pawtuxet River, AECOM is investigating alternatives focussing on raising the height of the levee. The selected levee elevation will exceed the height from the 0.2% annual chance event ("500-year" storm).

Flooding on the grounds of the WWTF can occur from two sources, from the Pawtuxet River overtopping the levee, or from direct rainfall on the WWTF grounds within the levee. This memorandum addresses the interior drainage system designed to minimize flooding from direct rainfall on the WWTF grounds.

The WWTF is subject to FEMA's levee regulations. The levee was accredited by FEMA, meaning the levee was considered high enough to prevent the Pawtuxet River from flooding during the 1% annual chance event ("100-year" storm) **and** the interior drainage system was considered sufficient to prevent interior flooding from a 1% annual chance event. Since the system was accredited, there was no need to purchase flood insurance for the buildings inside the levee.

The purpose of this memorandum is to establish:

- The appropriate design level for the interior drainage system (given that the levee is being designed to accommodate a 0.2% a "500-year" storm)
- If the interior drainage system as currently designed can accommodate this design event

Findings

AECOM recommends that the interior drainage system be designed compared against the 100-year storm. The benefits of a 500-year design for the interior drainage system do not justify the cost. The 500-year event is extremely rare (a 0.2% chance of occurring in any given year), and the volume and duration of a 500-year flood inside the levee would be significant but an order of magnitude less than if the levee is overtopped by the Pawtuxet River. The 100-year design level is also consistent with FEMA's regulations for accreditation

Without improvements, the existing interior drainage system cannot contain the 100-year storm event rainfall inside the levee without flooding. The flooding is considered relatively minor and of short duration. The duration and depth of flooding will increase in the future under anticipated 2030 design conditions, but will still be relatively minor. FEMA will still accredit the levee system as long as the area flooded is properly mapped. This finding was not expected given that FEMA had previously accredited the levee system, without showing flooding inside the levee.

The WSA has three viable options regarding interior drainage design and FEMA accreditation:

- WSA can choose not to seek accreditation – The site will remain accredited until FEMA revokes the accreditation. At that time, flood insurance may be required on buildings inside the levee on the northwest portion of the site. Under this option, no improvements (beyond those required to accommodate the floodwall footprint and I-95 drainage) to the interior drainage system are needed. The interior drainage system has served the site adequately for 27 years.
- WSA can seek accreditation and show the extent of anticipated 100-year flooding – The floodplain inside the levee would need to be shown, and buildings within the floodplain may be required to purchase flood insurance. Like the first option, no improvements beyond the floodwall footprint and I-95 drainage improvements would be required. Model results show that under 100-year present conditions, the 100-year flood volume is .21 acre-feet and the flood duration is 22 minutes (contrasted with over 150 acre-feet for many days during the March 2010 event). The approximate floodplain associated with this (as shown in Figure 11 for present conditions) is located in a relatively small area on the northwest portion of the site near the pump station, and has a maximum depth less than one foot. This floodplain is based on all pumps in service at the pump station.
- WSA can improve the drainage system to eliminate interior flooding from the 100-year event, in which case no flood insurance would be required. This could be accomplished by on-site storage, increased pumping capacity, or a combination of both.

In all cases, the likelihood of flooding will increase as the facility ramps up its anticipated Year 2030 wastewater treatment rates. For example, compared with present day, AECOM expects the volume of flooding for a 100-year storm in 2030 to increase from 0.21 acre-feet to 0.47 acre-feet and the flood duration to increase from

22 to 32 minutes. The approximate floodplain for future conditions is shown in Figure 12.

AECOM will make its final recommendation after discussing these findings with the WSA.

Introduction

The Warwick Advanced Wastewater Treatment Facility (WWTF) is enclosed on three sides by a levee and on the fourth side by I-95. All the drainage on the interior side of the levee, including that from the levee toe drain, discharges to the Pawtuxet River. When the stage on the Pawtuxet River is high and site drainage cannot flow to the river by gravity, the drainage is diverted to the wastewater effluent pumping station and is pumped to the Pawtuxet River along with the wastewater effluent.

This memo examines:

- The appropriate design level for the interior drainage system.
- Whether the WWTF would have flooded anyway from the rain that fell inside the levee during the March 2010 event. The largest event since the levee was built before March 2010 was October 2005. This event was also examined.
- Whether the WWTF will flood during standard design conditions. Three design storms, the 10-year, 100-year, and 500-year events, were examined for their impact on the WWTF site within the levee. The 10-year storm event was examined because it is often considered an appropriate level of design for minor storm drainage facilities. The 100-year storm event was examined because of the high risk associated with flooding at this site, and because of FEMA accreditation requirements. It is a design level often used for major drainage facilities where risk of failure is great. The 500-year storm event was examined since the levee elevation will be built to that level to protect the site from the Pawtuxet River during events as high and higher than the March 2010 event. The design storms were evaluated under present conditions, and under anticipated Year 2030 conditions, when wastewater flows are expected to be higher.

The existing levee was installed in 1985, 26 years ago. In that time, there have been no reports of interior flooding on the plant site, except in March 2011 when the levee was overtopped. The lack of reported flooding points to the fact that drainage system design was sufficient to handle all except the March 2010 event. Computer simulations presented below will investigate if drainage was sufficient had the levee not been overtopped in March 2010.

The proposed levee alignment reduces the drainage area inside the levee. The wooded area near the administration building between Interstate 95 and the access road will no longer be part of the interior drainage system. The current drainage area of 18.9 acres will be reduced to 16.2 acres. This area excludes open tank areas inside the levee that do not contribute to stormwater runoff. For establishing the impact of past storms (October 2005 and March 2010), 18.9 acres was used. For establishing the impact of design storms after the levee is raised, 16.2 acres is used.

Computer Simulation Modelling

Hydrologic Parameters

AECOM performed the drainage analysis using the U.S. Army Corps of Engineer's HEC-HMS Rainfall-Runoff computer simulation model. Rainfall transformation into runoff was based on the "Soil Conservation Service (SCS) Unit Hydrograph" approach. (The SCS is now the National Resource Conservation Service). The hydrologic parameters important to computing runoff from rainfall using this approach are drainage area, Initial Abstraction, Ia (the rainfall that is intercepted or immediately infiltrates into the ground and does not run off), Curve Number, CN (an indicator of how much rainfall infiltrates into the ground once runoff begins), and subbasin travel time. CN is further dependent on antecedent runoff conditions (ARC) which are characterized by the degree of soil saturation. For the March and October storms, AECOM assumed relatively saturated conditions. For the design storms, AECOM used more typical or average conditions, under the assumption that a design storm rainfall will cause design storm runoff under otherwise average watershed conditions. The SCS method defines saturated conditions as "ARCIII" and average conditions as "ARCII".

The plant area inside the levees was divided into 3 subbasins, East, North, and West, as shown in Figure 1. Each basin was assigned a Runoff Curve Number, CN. CN is based on soil type and land use, and is used to establish the portion of rainfall on a site that becomes runoff. The appropriate values were selected from Table 2-2a of SCS (now NRCS) TR-55, Urban Hydrology for Small Watersheds.

Ia and CN are related to one another. Ia is typically computed as 20% of potential maximum retention in a given soil. Recent studies have indicated that Ia is seldom that significant, and more typically assumes 5% of potential maximum retention.

Figure 1 – Subbasin Delineation

Travel time (or time of concentration) is the time it takes for runoff to travel from the hydraulically most distant point of the subbasin to its outlet. Typically, each subbasin has an overland flow, gutter flow, and channel (pipe) flow component that is summed to establish the total time. Tables 1 and 2 summarize the input parameters used for the site.

Table 1 – Hydrologic Input Parameters – Drainage Area and Runoff Loss

		CN and Initial Abstraction, Ia			
Subbasin	**Area (ac.)**	**CN $_{ARCII}$**	**CN $_{ARCIII}$**	**Ia $_{ARCII\ (in.)}$**	**Ia $_{ARCIII\ (in.)}$**
East	6.9	51	71	0.88	0.34
East (after levee is raised)	4.3	54	73	0.79	0.30
North	4.2	62	79	0.55	0.21
West	7.7	63	80	0.50	0.19

Table 2 – Hydrologic Input Parameters – Travel Times

Subbasin	Travel Time (Time of Concentration) (min.)
East	32
East (after levee is raised)	30
North	21
West	27
At junction chamber	30

<u>Rainfall Events Simulated</u>

Two historic events, October 2005 and March 2010, were simulated. The October 2005 event was simulated as a check on modelling results. Prior to March 2010, the October 2005 storm was the largest on record since the levee was built. As seen below, the model simulation of the October 2005 storm resulted in no flooding, consistent with what actually happened. The March 2010 event was simulated to see if there would have been any interior flooding on site had the levee not overtopped (and the I-95 storm drain did not surcharge). The March 2010 simulation demonstrates that there would have been no interior flooding if the levee did not overtop. Figure 2 shows a plot of the October 2005 and March 2010 rainfall events at the Providence T. F. Green Airport rain gage, Station No. 376698. The total rainfall from these events was 6.38 inches in 20 hours for October 2005 and 8.83 inches over two days, with a maximum 24-hour total of 6.86 inches for the March 2010 event.

Three design events, the 10-, 100-, and 500-year design events, were also simulated.

Figure 3 shows the Northeast Regional Climate Center Rainfall Distribution Curves for Warwick for the 10-, 100-, and 500-year events, respectively, each with a 24-hour duration. These curves were dynamically calculated on the Climate Center website (precip.eas.cornell.edu) for Warwick.

As noted by the Climate Center, the highest peak discharges from small watersheds like the area inside the levee are caused by intense, brief rainfalls that may occur as distinct events or as part of a longer storm. One common practice in rainfall-runoff analysis is to develop a synthetic rainfall distribution to use in lieu of actual storm events. This distribution includes maximum rainfall intensities for the selected design frequency arranged in a sequence. This approach is designed to capture the expected peak discharge for a given design storm that is critical for producing peak runoff. Rainfall statistics for these events are presented in Table 3. Since the levee was built there have been no 100-year 24-hour events.

Table 3 – Rainfall Characteristics

Event	Maximum 1-hour rainfall (inches)	Maximum 3-hour rainfall (inches)	Maximum 12-hour rainfall (inches)	Maximum 24-hour rainfall (inches)	Maximum 2-day rainfall (inches)
October 2005	0.82	2.09	5.75	6.38	6.38
March 2010	0.55	1.46	4.31	6.86	8.83
10-year	1.51	2.57	3.88	4.73	5.12
100-year	2.65	4.63	6.91	8.31	9.21
500-year	3.95	6.99	10.34	12.23	13.94

The rainfall statistics from the Climate Center's database are based on a significantly longer record (over 50 years longer) than previous estimates, which were generally based on the National Weather Service' "Technical Paper 40". This paper, published in the 1960's, was commonly used until recently, and may have been used when the levee was first built. Estimates of the 10-year storm using TP 40 are 1.2, 2.8, 4.1, and 4.9 inches for the 1-, 3-, 12-, and 24-hour durations, higher than in Table 3. TP 40 estimates for the 100-year storm were 2.9, 4.0, 6.0, and 7.1 inches, considerably less than the Table 3 values. This may help explain why the interior drainage system cannot now accommodate the 100-year storm.

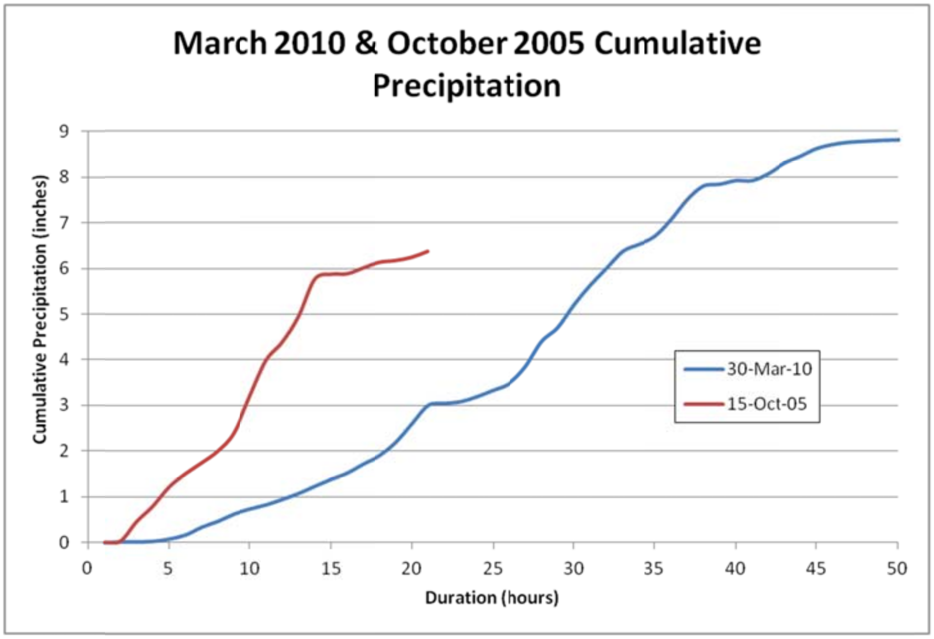


Figure 2- March 2010 Event Precipitation

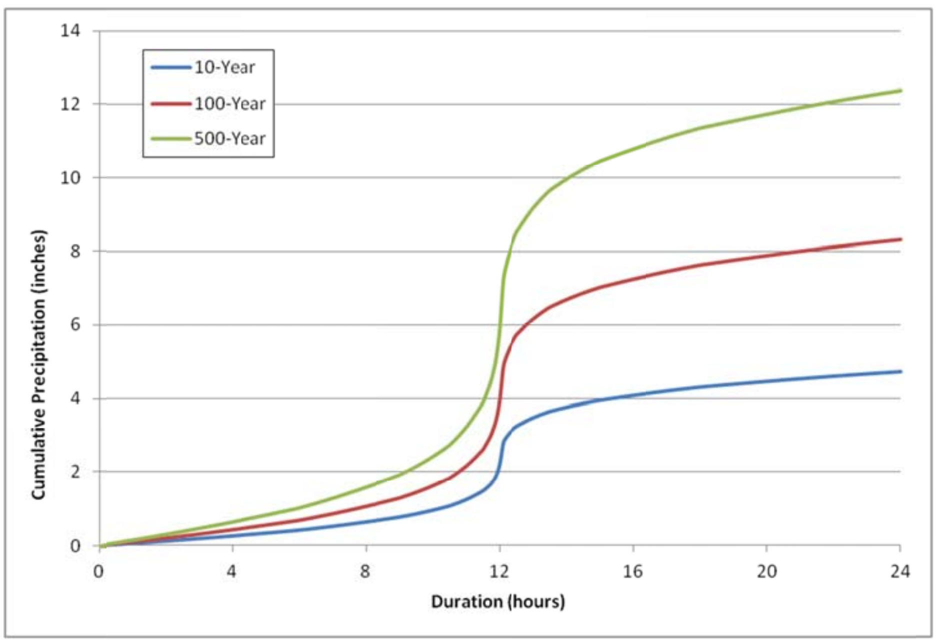


Figure 3 - Northeast Regional Climate Center Extreme Precipitation, 10-, 100-, and 500-year Events

Table 4 presents peak discharges and storm volumes for the events. Figures 4 through 8 present the corresponding hydrographs.

Table 4 - Peak Discharges and Storm Volumes

Storm	Peak Discharge		Storm Volume	
	cfs	mgd	acre-feet	million gallons
October 2005	13.3	8.6	6.4	2.1
March 2010	8.4	5.4	9.9	3.2
10-year	17.7	11.4	2.2	0.7
100-year	44.0	28.4	5.7	1.9
500-year	79.9	51.6	10.2	3.3

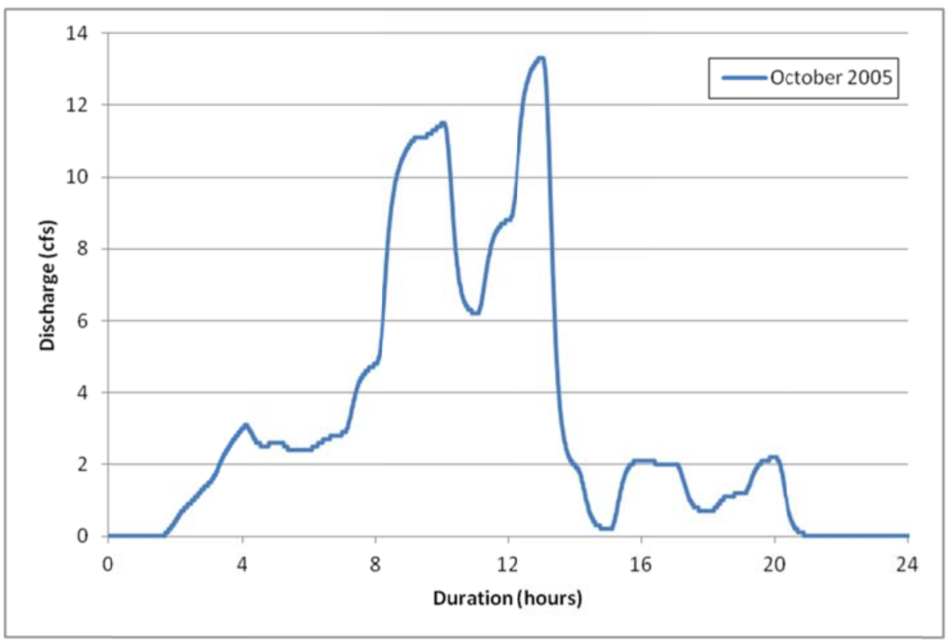


Figure 4- October 2005 Event Hydrograph

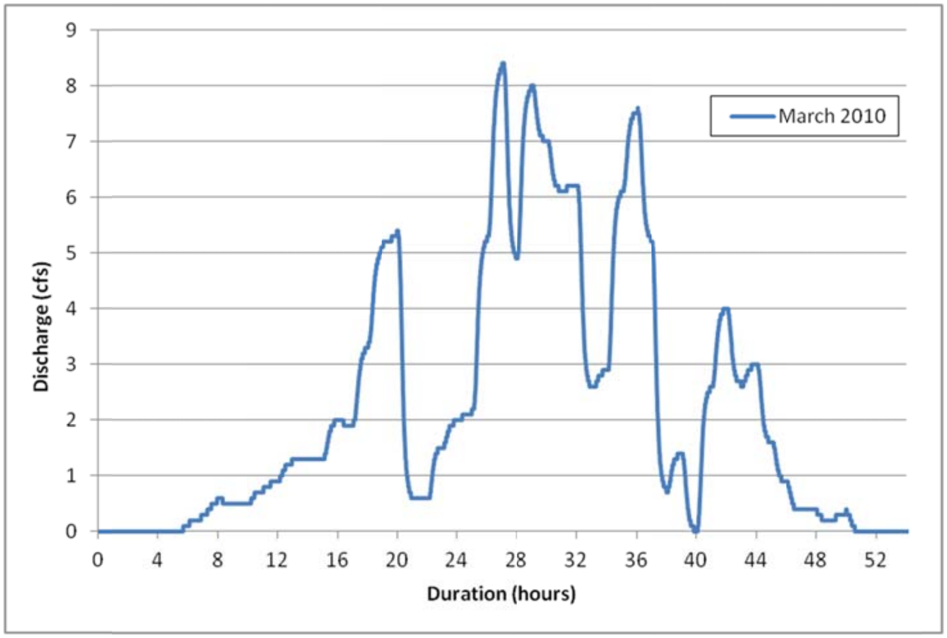


Figure 5- March 2010 Event Hydrograph

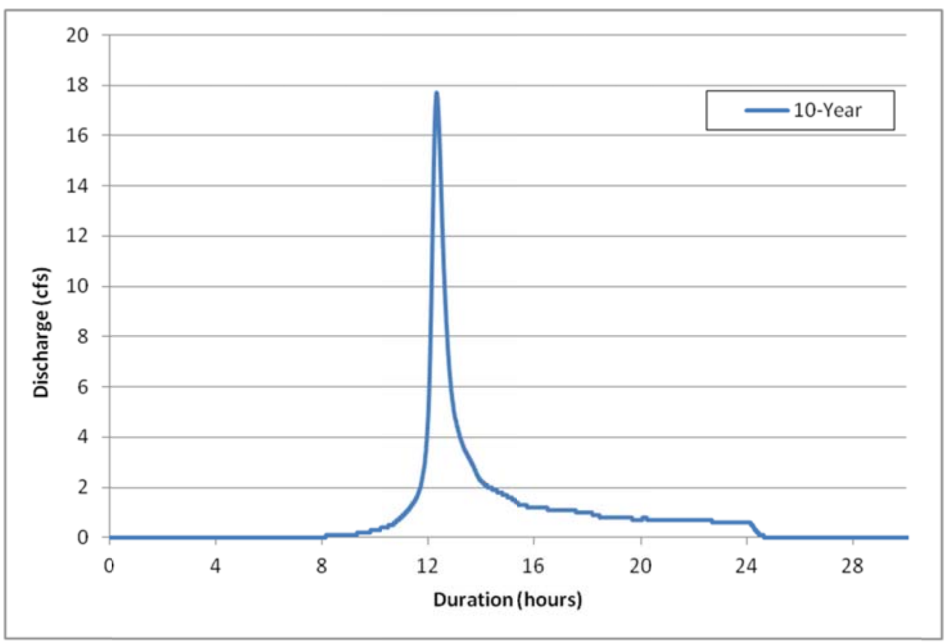


Figure 6 - 10 Year Event Hydrograph

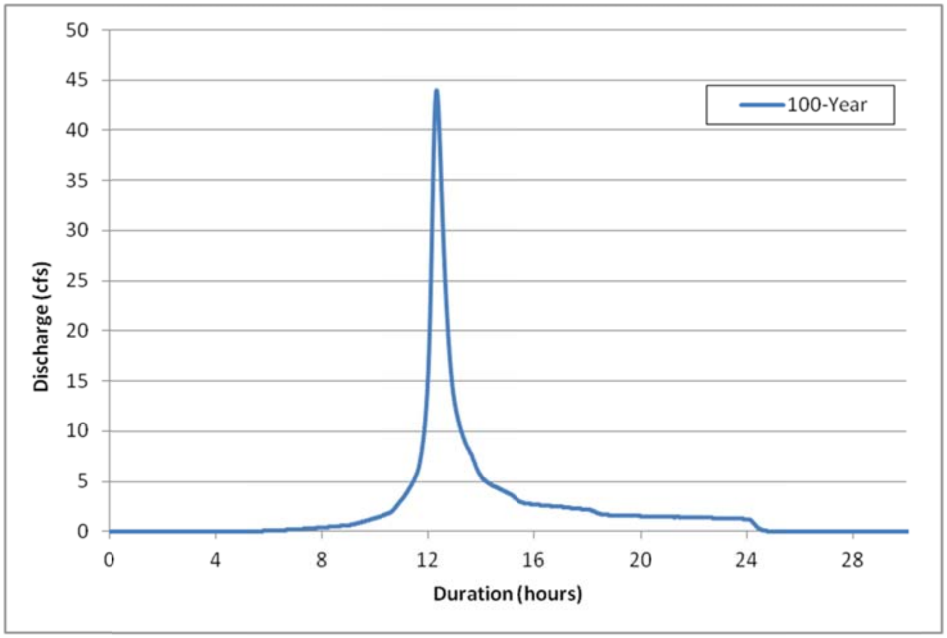


Figure 7- 100 Year Event Hydrograph

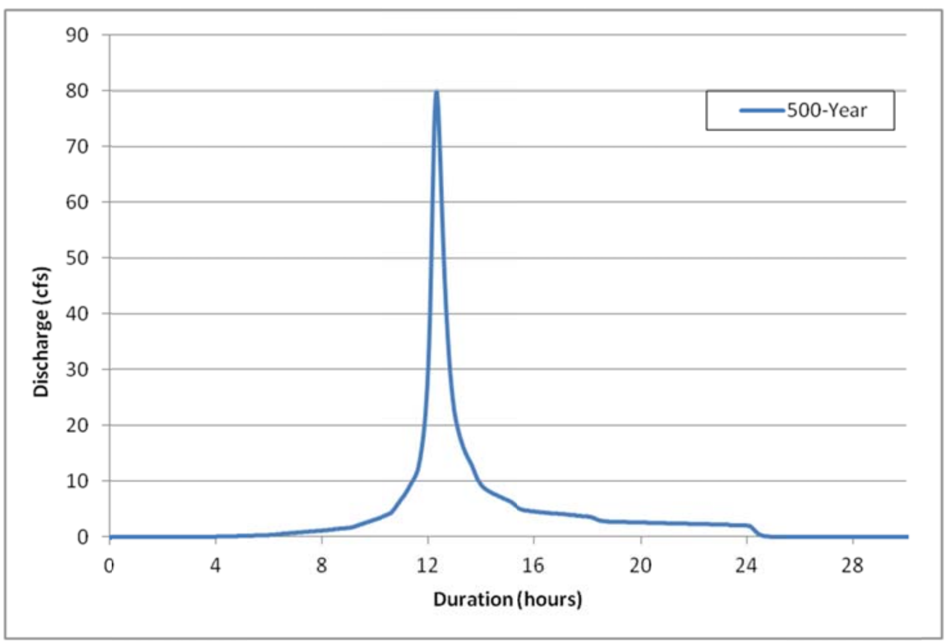


Figure 8- 500 Year Event Hydrograph

As a check on the peak discharges from the modelling, the results were compared with the peak discharges from an alternative methodology, the Rational Equation. The

Rational Equation provides a peak discharge only, and does not provide a hydrograph of the storm volume. The Rational Equation is:

$Q = CiA$, where:

Q = peak discharge (cfs)

C = runoff coefficient

i = rainfall intensity for the time of concentration (inches per hour)

A = area (acres)

The runoff coefficient for light industrial sites typically varies from 0.5 to 0.8 and for heavy industrial sites from 0.6 to 0.9. A value of 0.7 was selected for the treatment plant site. As shown previously in Table 2, the time of concentration at the junction chamber is approximately 30 minutes. The corresponding 10-, 100-, and 500-year rainfall intensities based on Northeast Regional Climate Center data for Warwick are 2.7, 4.5, and 6.4 inches per hour. The area is 18.9 acres. The 10-, 100-, and 500-year discharges computed using the Rational Equation are 35 cfs, 60 cfs, and 85 cfs, respectively. The comparison between the two methods shows the estimates from the Rational Equation are higher than modelling results. Because of the conservative nature of the Rational Equation, this is within expectations and demonstrates that the modelling results are not overly conservative.

Comparing the 10- and 100-year design storms to the October 2005 and March 2010 storms, the design storms have higher peak discharges and smaller runoff volumes. The higher peak discharges are attributable to the much more intense rainfall bursts than in the October 2005 and March 2010 storm. For example, the maximum 15-minute rainfall that actually occurred at the rainfall gage during the March storm was 0.55 inches per hour. For comparison, the 10-year 15-minute maximum is 2.72 inches per hour and the 100-year 15-minute maximum is 4.8 inches per hour. The design storms' lower runoff volumes are attributable to the slightly lower total rainfall and to the selection of average rather than wet antecedent conditions for the design events. Only the 500-year storm runoff volume exceeds the October and March events.

Levee Toe Drainage

Besides rainfall, drainage from toe drains running the length of the levee contributes to the storm drainage system. The amount of flow contributed by the toe drains is estimated at 2 MGD when the Pawtuxet River is at a 500-year stage. Though this is dependent on the head difference along the levee, 2 MGD was used as the levee toe drainage for 10-year and 100-year storms as well.

Plant Characteristics

The last component of flow at the WWTF is the wastewater itself. Currently the average daily flow is 5 MGD. Present day and planned future (2030) maximum daily and maximum hourly flow rates at the plant are as follows in Table 5:

Table 5 – WWTP Flow Characteristics

	Maximum Daily Flow		**Maximum Hourly Flow**	
	MGD	CFS	MGD	CFS
Present Day	8.7	13.4	13.3	20.5
Planned Future (2030)	13.3	20.6	20.5	31.7

The current effluent pump station, which serves both the wastewater plant and the drainage system, has a firm capacity of 16,500 gpm (24 MGD, 37 cfs) and a total capacity with no units out of service of 22,000 gpm (32 MGD, 49 cfs) at 25 feet Total Design Head.

March 2010 Storm with a higher levee

Had a higher levee been in place and the WWTF not flooded, the current pump station would have been required to pump the following flows:

> Present Day Max Daily Flow – 8.7 MGD (It is likely that there will be considerable I/I at the time of peak stormwater runoff, but it is unlikely that maximum hourly flow and peak stormwater runoff will be coincident. Therefore, maximum daily flow was used.)
>
> Toe drain flow – 2.0 MGD
>
> Peak March 2010 Runoff – 5.4 MGD
>
> Total – 16.1 MGD

Given the firm pump station capacity of 24 MGD at 25 Feet TDH, the pump station would have successfully kept up with the runoff storm event, and there would have been no flooding on the interior of the plant site. This also assumes there would have been no flooding from I-95 storm drainage.

If the March 2010 flood were to re-occur at future planned plant treatment rates, it would need to pump the following flow streams:

> Planned Future Max Day Flow – 13.3 MGD
>
> Toe Drain Flow – 2 MGD
>
> Peak March 2010 Runoff – 5.4 MGD

Total – 20.7 MGD

Should the March 2010 storm re-occur under Year 2030 conditions, the pump station will still be adequate to prevent flooding on the interior of the levee.

Summary of Simulated Conditions

This information for the March 2010 storm is tabulated in Table 6 along with the results from the October 2005 storm and from the 10-, 100-, and 500-year design storms. The table is based on the assumption that peak stage on the Pawtuxet River, peak runoff from the treatment plant site, and maximum daily flow are coincident. On large river basins the peak site runoff may precede the peak river stage, as the flood wave moves downstream on the river. It is unlikely that this would be the case for the Pawtuxet. The table compares the maximum inflow to the influent pumping station under present day and future planned conditions to firm and total pump station capacity.

Table 6 – Comparing Pump Station Peak Inflow to Pump Station Capacity

Storm	Plant Design	Maximum Daily Wastewater Flow (MGD)	Peak Storm Discharge (MGD) (including 2 MGD toe drainage)	Peak Inflow to Pumping Station (MGD)	Firm Pump Station Capacity at 25' TDH (MGD)	Total Pump Station Capacity at 25' TDH (MGD)
March 2010	Present Day	8.7	7.4	16.1	24	32
March 2010	Future (2030)	13.3	7.4	20.7	24	32
October 2005	Present Day	8.7	10.6	19.3	24	32
October 2005	Future (2030)	13.3	10.6	23.9	24	32
10-year	Present Day	8.7	13.4	22.1	24	32
10-year	Future (2030)	13.3	13.4	26.7	24	32
100-year	Present Day	8.7	30.4	39.1	24	32
100-year	Future (2030)	13.3	30.4	43.7	24	32
500-year	Present	8.7	53.6	62.3	24	32

	Day					
500-year	Future (2030)	13.3	53.6	66.9	24	32

From the table, the October 2005 storm would have stressed the interior drainage system more than the March storm, but would not have flooded the interior of the plant. This is in keeping with the actual conditions experienced at the plant. Under 2030 conditions, the pump station will still be able to handle October 2005 flow rates and volumes within its firm capacity.

For the 10-year design storm, the pump station is currently adequate to handle the 10-year storm. Under 2030 conditions, the firm capacity will be exceeded, but if all pumps remain operable, there will be no flooding. The peak discharge rates for Present Day conditions do not last long and therefore do not necessarily result in large volumes of flooding, as shown below in Table 7. For example, for the 10-year 2030 condition, the flow rate exceeds the firm pump station capacity for only 19 minutes, and the flood volume is relatively insignificant.

For the 100-year design storm, pump station capacity is exceeded under all conditions. However, the volume and duration of flooding is still relatively minor. Only under 500-year design storm conditions does the flooding become more widespread.

Table 7 – Flood Volumes (acre-feet) and Durations (minutes) for Design Storms

Results Assuming Firm Pump Capacity (24 mgd)				
	Present		**Future (2030)**	
	Volume (acre-feet)	**Duration (minutes)**	**Volume (acre-feet)**	**Duration (minutes)**
10-year	-	-	0.07 ac-ft	19 minutes
100-year	0.71 ac-ft	40 minutes	1.2 ac-ft	61 minutes
500-year	2.48 ac-ft	71 minutes	3.38 ac-ft	116 minutes
Results Assuming Total Pump Capacity (32 mgd)				
	Present		**Future (2030)**	
	Volume (acre-feet)	**Duration (minutes)**	**Volume (acre-feet)**	**Duration (minutes)**
10-year	-	-	-	-
100-year	0.21 ac-ft	22 minutes	0.47 ac-ft	32 minutes
500-year	0.86 ac-ft	31 minutes	1.21 ac-ft	39 minutes

Figures 9 – 12 show the expected extent of flooding from the 100-year design storm.


100-Year FIRM Capacity
Present Daily Max WW Demand
Legend
100 Present
0 - 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1 - 1.25


100-Year Total Capacity
Present Daily Max WW Demand
Legend
100 Present
0 - 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1


100-Year FIRM Capacity
Future Daily Max WW Demand
Legend
100 Future
0 - 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1
1 - 1.25
1.25 - 1.5


100-Year Total Capacity
Future Daily Max WW Demand
Legend
100 Future
0 - 0.1
0.1 - 0.25
0.25 - 0.5
0.5 - 0.75
0.75 - 1