South Shore of Staten Island Validation Report

Appendix E – Geotechnical Data

January 2024

Change in Geotechnical requirements

Feasibility Stage: At the Feasibility stage 14 soil borings and accompanying laboratory testing were performed in October 2002 by Matrix Environmental (see attachment for location). The boring depths ranged from 24 to 30 feet. To supplement these borings, between Feasibility stations 50+00 and 105+00, which includes the Oakwood Beach Wastewater Treatment Plant to about Tysens Lane, an additional 20 test borings, located primarily in the wetland areas, were obtained from New York City Department of Environmental Protection (NYCDEP). The borings were performed by various New York City agencies between 1949 and 1966*,* some predating construction of the wastewater treatment plant, and generally were limited to between 32 to 42 feet deep with three borings in the southwest corner of the wastewater treatment plant site extending down between 67 and 105 feet deep. These supplemental test borings were concentrated at the wastewater treatment plant site. From these borings a continuous six-foot thick clay layer, varying between 1 to 10 feet below the present ground surface, was identified commencing a short distance south of Grayson Avenue (Sta. 30+00+-) to just west of Kissam Avenue (Sta. 80+00).

About 70% of this continuous identified clay layer is located within either the levee segment or floodwall segment with only about 30% or 1500 feet falling beneath the seawall segment. In addition three localized clay layers were identified along the line of protection; one two foot thick layer eight to nine feet below grade near Tysens Avenue (Sta 105+00 +-); a second area with a maximum nine foot thick layer about 15 feet below grade near Naughton Avenue, estimated to taper from no clay at Sta.190+00+- to its full thickness near Sta. 200+00+- and then back down to no clay near station 210+00; and the final localized area identified during the Feasibility stage is centered about 1600 feet east of Seaview Avenue and was estimated to taper up from no clay at Sta. 230+00 to a six foot thick clay layer at Sta. 234+00 and then back down to no clay at Sta. 240+00 and is about 25 feet below grade.

At that time, the remainder of the project was assumed to consist of coarse to fine sand with varying amounts of clay, silt, and gravel. The intermediate and deep clay layers were generally not identified at this time due to the limited depth of the borings. Analysis performed at that time, for typical sections, included slope stability of the levee and buried rock seawall; seepage analysis for the levee, floodwall and buried rock seawall; and settlement analysis for the Levee. At that time the extent of contamination and radioactive waste was not as well known, and as with all civil works projects, were considered to be the responsibility of the non-federal partner to provide a clean site prior to the project being implemented. Since the floodwall was to be constructed on a pile foundation that penetrated through the identified clay layer and the buried rock seawall was to be primarily constructed in an area thought to be comprised of coarse sand and gravel, settlement was not a major concern and thus analysis was not performed for these features.

In accordance with USACE design manuals EM 1110-2-1913 and EM 1110-2-1902, slope stability analyses were performed for Levee and Buried Seawall sections, along the line of protection for four loading conditions as follows:

1. Case I, end of construction (land side slope),
2. Case II, steady-state seepage from full flood stage (land side slope),
3. Case III, sudden drawdown (water side slope), and,
4. Case IV, earthquake (land side slope).

A commercially available computer program, SLOPE/W©, was used to perform the slope stability analyses. SLOPE/W© is a general-purpose slope stability program that uses limit equilibrium methods to compute the factor of safety (FOS) for a given slope geometry and loading conditions. Spencer's Procedure for the method of slices - for circular failure surface - was used to evaluate the slope stability as this procedure satisfies the complete static equilibrium for each slice. SLOPE/W© automatically searches for the circular shear (failure) surface associated with the minimum FOS, which is considered the critical or controlling shear surface. The pore pressures within the embankments for the Case II loading condition were obtained from the phreatic surfaces developed using the transient and/or steady state seepage analyses using SEEP/W©. Since SEEP/W© and SLOPE/W© are companion programs, pore pressures obtained from the SEEP/W© analysis can be automatically transferred to the corresponding SLOPE/W© stability analysis. For Case III (sudden drawdown) loading condition, because of the instantaneous drawdown, it was assumed that pore pressures within the embankment remain the same before and after the drawdown.

The slope stability analyses results are presented for the Buried Seawall/Armored Levee and Earthen Levee. The slope stability analyses of buried seawalls for the Case II loading condition were performed using pore pressures obtained from transient seepage analyses. However, for the earth embankment levees the slope stability analyses for the Case II loading condition were performed using conservative pore pressures obtained from the steady-state seepage analyses. As per EM 1110-2-1913, slope stability analyses were performed for all four loading conditions. The results are presented in Table 1, along with the corresponding minimum acceptable factors of safety.

Table 1: Summary of Factor-of-Safety Resulting from Slope Stability Analyses*

Slope	Design Condition	Minimum Acceptable Factor of Safety	Buried Seawall	Levee
Land Side	Case I: End of Construction	1.3	1.4	1.7
Land Side	Case II: Seepage from maximum flood level	1.4	1.4	1.5
Water Side	Case III: Sudden drawdown	1.0	1.2	1.2
Land Side	Case IV: Earthquake	1.0	1.0	1.2

***Results assumed removal of organic material and back fill with competent soils**

Remarks:

1. Table 1 presents the factors of safety after the soft organic soils have been excavated and the compact fill added. The end-of-construction (Case 1) factor of safety values were originally 0.9 and 1.1 for a Levee and Buried Seawall/Armored Levee founded on soft organic soils, respectively. It should be noted that these factor of safety values are less than the minimum acceptable value of 1.3. Therefore, the feasibility study determined wherever encountered the soft organic soils located close to the surface should be removed and replaced with compacted sandy fill. It was recommended to remove soft organic soils and/or portions of soils with lumps of soft organics to a depth at least 6-inches deeper than bottom surface of soft organic soils. Also, the removal should be extended to at least 10 feet beyond the toe of the levee slopes or beyond the structure. This recommendation did not consider the contamination issues nor the deeper clay layers identified during the PED effort.

2. The Case II factor of safety value was determined to be 0.8 for sacrificial cover layer of the buried seawall under steady seepage condition. However, steady seepage condition will most likely not occur during the anticipated storms. Furthermore, considering that shear surface corresponding to factor of safety of 0.8 is within the sacrificial cover, even if steady seepage condition develops only sacrificial cover layer will likely to be impacted.

3. The final remark at the time of the feasibility study was that additional test borings should be performed within the wetland area and within the remainder of the alignment during the design stage to completely characterize the subsurface conditions along the LOP. These borings should be drilled to a depth that can be used to confirm the pile design capacities and drivability.

Because of the low probability of earthquakes coinciding with severe storm events, stability analyses for the Case IV (earthquake) loading condition were performed assuming no water above the ground surface. Pseudo-static coefficient of 0.16g was assumed for the earthquake loading case.

Seepage analyses were performed using the commercially available finite-element method (FEM) software program SEEP/W©. In order to perform the seepage analyses, a representative cross-section was selected for each type of structure. These representative sections were conservatively selected at maximum height locations.

Table 2 Summary of Seepage Analyses Results

Reach No.	Type of Structure	Length (ft)	Total Seepage Quantity		Exit Hydraulic Gradient
			ft^3/sec (cfs)	Gallons/min (gpm)	
A-1 and A-2	Levee	3,430	<1	20	0.25
A-3	Flood wall	1,826	< 1	20	0.05
A-4	Buried Seawall/Armored Levee	22,705	< 1	95	0.01

It should be noted that the pore pressures obtained from the seepage analyses were used for the Case II slope stability analyses as described in the Slope Stability section above. Total seepage quantity is per one foot of levee run, flood wall, *etc.*

Settlement analysis performed for the levee section assumed the soft organic clay layers were removed and back filled since as discussed above this is necessary to achieve the necessary factor of safety for slope stability. This assumption was predicated on the clay layers being near the surface and did not consider the presence of the contaminated materials. The immediate settlement values were estimated as per EM 1110-1-1904 predicated on that assumption due to the limited soil information at the time. Accordingly, the estimated immediate settlement values approximately range from ½ inches to 1½ inches. Since most of the estimated immediate settlement is likely to occur during construction, it was determined at the time of the feasibility study long-term primary compression (consolidation) settlement should not be a concern after removing any soft organic soil layer that could be present near the ground surface because the subsurface soils are generally sandy soils - based on the soil data available at that time.

For the floodwall it was determined necessary to support the foundation on piles due to the lateral wave and hydrodynamic forces and thus the surface organic clays near the surface were not critical and settlement was not an issue. Based on the subsurface conditions and Driven pile capacity analyses, it was recommended that HP14x89 friction piles be driven to the sandy stratum. The recommended pile lengths and corresponding estimated pile capacities are as follows:

Allowable Compression and Uplift Capacity (tons)	**Estimated Length for Compression Capacity (ft)**	**Estimated Length for Uplift Capacity (ft)**
35	70	80
50	80	95
70	95	115

PED Stage: For the PED design the various project segments have been advanced to various levels of design over a period of time as follows:

Segment	**Design Level Last Submission**	**Date Submitted**
Levee – North of Hylan Blvd. Closure Gate	Revised 30%	1/20/20
Levee - South of Hylan Blvd & Area A Tide Gate	100%	3/19/21
Floodwall	100%	4/2/21
Buried Rock Seawall (BRS)- OBWWTP to Miller Field	60%	2/7/21
Buried Rock Seawall (BRS)- Midland Beach to Ft. Wadsworth	10%	4/22/21
Double Row Sheetpile Seawall (DSP)- OBWWTP to Miller Field	10%	4/22/21
Double Row Sheetpile Seawall (DSP)- Midland Beach to Ft. Wadsworth	10%	4/22/21

Additional subsurface investigations were conducted between August of 2018 and August of

2020. The additional field data includes 38 additional Drill Holes; 51 SCPTus; and 27 DMTs. The collected data is distributed as follows:

1. Levee Segment: Four DHs (a.k.a. boreholes), depths 36 to 134 ft.; 13 SCPTus, depths 12-119 ft. and generally over 90 ft deep; 2 DMTs 50.5 ft. deep.

2. Floodwall Segment: Five DHs, depths 81.5 - 134 ft.; 4 SCPTus, depths 82-94 ft.; no DMTs.

3. Seawall OBWWTP - Miller Field Segment: 13- DHs, majority at 36 ft deep with three at or greater than 135 ft. deep; 13- SCPTus, generally 42 to 55 ft. deep, and one at 129 ft. deep; 12-DMTs, depths 36 ft.- majority at 50 ft..

4. Seawall Midland Beach - Ft. Wadsworth: Sixteen DHs, depths 36 ft. with one at 46 ft.; 21 SCPTus, generally 50- 96 ft. deep with two over 100 ft. deep; 13 DMTs generally 50 ft. deep.

The added geotechnical data refined the soil profiles for the various construction segments, and the added borings/ CPTs clarified (thickness and strength) the soft clay strata near the surface that presented short- and long-term settlement concerns. In addition, during the time between the feasibility stage and the PED stage, the contamination of Great Kills Park became better defined. As a result, changes to the original design were made to limit excavation within the contaminated area of Great Kills Park known as Operable Unit 1 as well as steps were taken to address the settlement concerns along the project alignment associated with the soft clays encountered.

Addressing the settlement concerns for the levee segment was accomplished by increasing the elevation of the constructed levee by 0.8 feet to allow for long-term post-construction settlement while maintaining the design height over the project life. This was also combined with the use of staged construction of the earthen levee using between one and three lifts (stages) with time allowed (3 months) between stages; this would allow the subsoils (soft clays) to gain strength to avoid a shear failure before the next stage would be placed.

In addition, to minimize excavation due to contaminated materials and to rapidly stabilize an access way for construction of the tide gate where some of the soft clay layers occurred near the surface, Deep Mixing Methods (DMM) have been incorporated into the design. The DMM panels are 3 ft in diameter with 9-inch overlap running perpendicular to the alignment at 7.5 ft on-center under the levee from a point about 162 feet north of the tide gate and extending south of the tide gate 611 feet to almost the floodwall, for a total of about 770 feet. Due to the potential for HTRW contamination it was determined that - rather than removing the soft clay soils near the surface and back filling with competent materials - the use of DMM would provide a stable access way to construct the tide gate while minimizing the excavation of potentially contaminated soils. The DMM also allows the earthen levee to be constructed in one stage without a concern for shear failure after the tide gate is complete.

For the floodwall segment: The wall is constructed on friction piles that would penetrate through soft clay layers and no special treatment was required.

For the seawall segments the primary design change between the Feasibility stage and PED stage was the need to address settlement issues associated with the soft clay layers.

Oakwood Beach to Miller Field. For the buried rock seawall from Oakwood Beach to Miller

Field a 60% Design was performed for settlement. The settlement was estimated at the center and at the toe of buried rock seawall, using data from Standard Penetration test (SPT) DH-10 (Station 81+87) and laboratory consolidation tests from boring logs DH-7 (Sta. 58+00) and DH-8 (Sta. 63+00) which were in the area of the proposed flood wall and approximately 1,000 feet from the beginning of the seawall. These were the only borings that had consolidation tests performed. On logs of boring DH-10, which is one of the two deep borings in this area, an upper and a lower clay layer were identified; the second deep boring is DH-11 on whose logs only a deeper clay layer was identified. The other borings within this area (B29, B28, B27, DH-11A, DH-12A, B-24, DH-13, DH-15, DH-16, DH-19 and DH-19A) are not deep enough to identify these clay layers. In addition, SCPTu's were performed through this area. Three of the SCPTu tests, or probings, were deep enough to identify if the clay layer existed. SCPTu-18 (Sta. 117+16) encountered a clay layer at an elevation of -105 ft; SCPTu -24A (Sta. 167+56) encountered a clay layer at an elevation of approximately -55.0 ft to -80 feet; and SCPTu -23 (Sta. 82+20) was advanced to 80 feet and did not encounter any clay layers. The remainder of the SCPTu's that were advanced in this area were shallow (SCPTu 6, 7, 7A, 9, 19, 19C, 19D, 9A, 10, 10A and 24) and were not deep enough to confirm the depth or thickness of the clay layer. Please see attached soil profile which shows soil borings and SCPTu's and the soil strata that were encountered.

Estimated Settlement. This section presents a discussion of the estimated settlement that will occur at the proposed Buried Seawall and Promenade from Oakwood Beach to Miller Field at South Shore of Staten Island. The settlement will be a result of immediate settlement and primary-compression settlement – caused by the consolidation process of clayey soils - that will occur due the construction of the proposed Seawall. As the promenade is constructed at grade along the top of the buried rock seawall, it is important to limit the settlement caused by primary and secondary compression to avoid cracking and trip hazards. The settlement analysis was based on soil boring DH-10, which is considered a conservative approach for the entire length because there was insufficient deep-soil information. The following parameters and depths were used:

Soil Layer	Depth	Unit Wt.	Effective Friction Angle	Compression Index	Initial Void Ratio	Coefficient of Consolidation	Secondary Compression Index
		g	φ'	C_c	e_o	c_v	C_α'
	(ft)	(pcf)	(deg.)			(ft^2/day)	
Silty Sand (Upper)	0 - 47	125	32	-----	--	---	-----
Clay (Upper)	47 - 79	120	---	0.212	0.785	0.337	0.0015
Sand (Lower)	79 - 121	130	35	-----	--	---	-----
Clay (Lower)	121 - 136	120	---	0.15	0.751	0.337	0.0011

The following are the anticipated settlement of the seawall once built to full height:

Summary of Total Settlement			
Item	Unit	Center of Seawall	Toe of Seawall
Elastic Settlement Above Elev. -32.5 ft (1)	(inch)	1.5	0.8
Elastic Settlement below Elev. -32.5 ft.	(inch)	0.9	0.5
Primary Compression Settlement	(inch)	8.1	3.3
Secondary Compression Settlement (50 Yrs)	(inch)	0.6	0.6
Total Settlement	(inch)	11.1	5.3
Note: For settlement, this reflects that the upper 5 feet of subgrade will be over-excavated and replaced with competent material, despite 11.3 inch of settlement predicted based on DMT-4 log that was provided by USACE.			

If the seawall is constructed and allowed to sit with no ground improvements, the elastic settlement will occur during the construction of the seawall, and it will take approximately 24 months for 90% of the primary compression (resulting from the consolidation process) to be completed. This would leave 0.8 inches and 0.3 inches of primary compression (consolidation) to occur at the center and toe of seawall, respectively, after the 24-month period.

GROUND IMPROVEMENT PROGRAM. The ground improvement method, which was selected to expedite settlement, was a wick drain program. Two options were looked at: Spacing of wick drains at 5 feet and at 7 feet. Construction of the sea wall will be performed in 2 stages in accordance with the following:

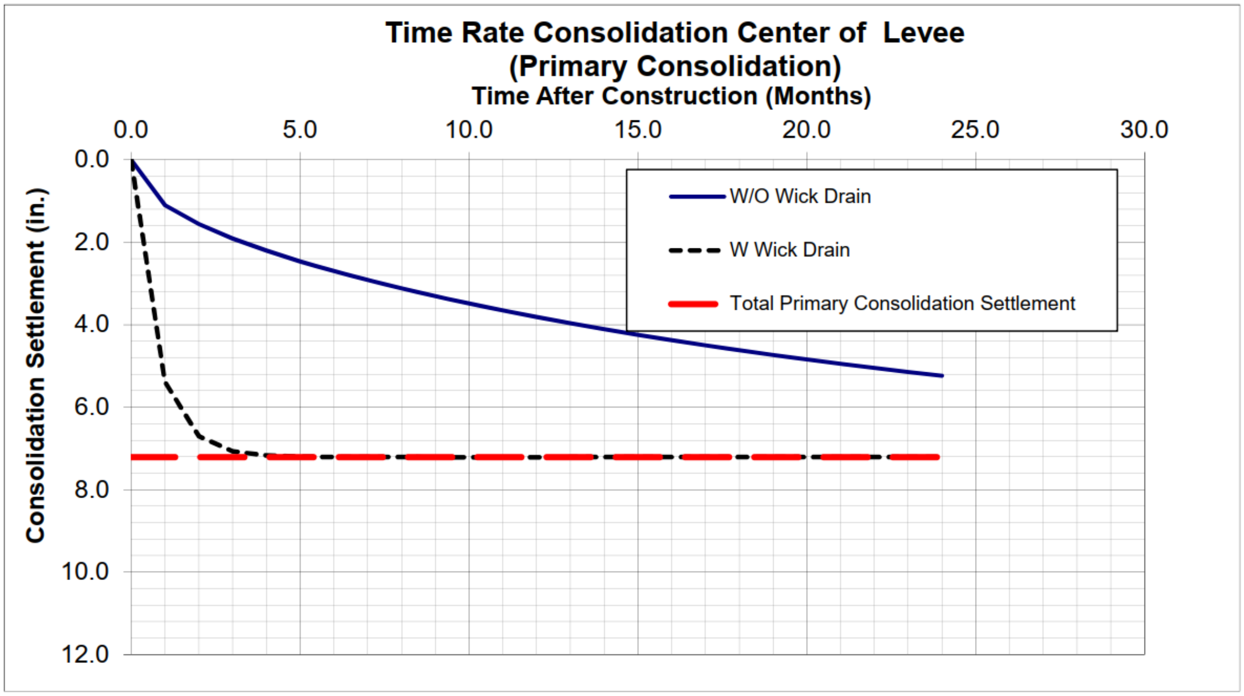
Stage 1: Drive the sheeting and construct the buried rock seawall to Elevation 17.4 feet (wall height of 15.5 feet) and allow the seawall to "sit" for one month.
Stage 2: Construct the buried rock seawall from Elevation 17.4 feet to Elevation 21.4 feet and allow the seawall to "sit" for two months. After settlement install promenade.

Based on the analysis that was performed, the findings are summarized below:

To minimize the remaining primary compression (consolidation) settlement, the recommended wick drain spacing is 5.0 feet. The remaining primary compression (consolidation) settlement after Stage 2 is 0.9 inches. The secondary compression (consolidation) settlement is 0.6 inches for a service life of 50 years.

A sand blanket of at least three feet thick will be placed prior to wick drains operation to serve as a drainage blanket for water control and to distribute the loads evenly.

The figure below shows the difference in time for primary compression (consolidation) to occur at the center of the seawall using wick drains at a 5 foot spacing and not using any wick drains. There would be a time saving of about 21 months with the use of wick drains. A sample calculation of the settlement analysis is provided in attachment 1.



For the alternative seawall design using the double row of sheet pile wall, where a 10% design has been completed, it was assumed that a similar ground improvement program would be used, and no updated settlement analysis was performed. The lateral extent of the wick drains was considered to be extended to cover the footprint of the 17-foot wide Double Sheet Pile Seawall (DSP) and the 27-foot wide Double Sheet Pile Seawall cross-sections. The surcharge would be accomplished using a 20.5-foot high pile of sand covering the footprint width of the DSP wall including the armor stone crest width and then sloping down to grade at a 3H:1V slope. Once (most of) the consolidation has occurred, the sand is to be partially removed down to the underside of the final access surface and the double row of sheet files are installed. Then, except between the two rows of sheetpile, more of the sand cap is removed to allow installation of the geotextile, bedding stone, underlayer and armor stone of the rock revetment along the waterside and the splash apron along the land side; this will be followed by final grading of a sand cover for aesthetics. The area between the sheet piles is to be filled with sand so the surcharge at that location remains in place.

Prior to advancing the design it is anticipated that some additional deep borings will be performed and “undisturbed” cohesive-soil samples will have consolidation test run on them; this is needed to better identify the clay layers and obtain consolidation parameters - for these layers - to minimize the limits where wick drains will be required.

Seawall from Midland Beach to Fort Wadsworth. This segment has been limited to a 10% design. The soil borings in this area consisted of no deep borings; however, various borings did encounter clay at shallower depths (DH-23, DH-25, DH-25a, DH-26, DH-27, DH-28, DH-29, and

DH-30) of the seawall. The following borings did not encounter any clay: DH-19A, DH-20, DH-21, DH-22, DH-24, DH-31, and DH-32; these borings were not deep enough to identify if there were deeper clay layers. In addition, SCPTu's were performed through this area. There were six deep SCPTu tests that were performed to identify the deep soil strata. SCPTu-25(Sta. 186+55) encountered a clay layer at an elevation between -45 and -68 feet; SCPTu -20 (Sta. 217+06) encountered clay layers at an elevation of approximately -14 to-22 feet, a second layer from -28 to -36 feet, and a third layer from -75 to -111 feet; SCPTu -26 (Sta. 246+49) encountered a clay layer at an elevation of -8 feet to an elevation of -14 feet and from -49 feet to the end of the SCPTu; SCPTu -21 (Sta. 267+32) encountered a clay layer at an elevation of -49 feet to the end of the SCPTu; SCPTu -22B (Sta. 265+76) encountered a clay layer at an elevation of -45 feet to the end of the SCPTu; and SCPTu -27 (Sta. 289+06) encountered a clay layer at an elevation between -84 and -90 feet. The remainder of the SCPTu's that were advanced in this area were shallow (SCPTu-11, 24B, 12, 13, 14, 15, 16, and 17) and were not deep enough to confirm the depth or thickness of any deeper clay layer(s). Please see attached soil profile which shows soil borings and SCPTu's and the soil strata that were encountered.

The soil parameters provided for settlement of the Promenade reach were based on the subsurface conditions developed for the buried rock seawall 60% DDR. The generalized descriptions of the subsurface conditions are based on boring logs and laboratory test results.

GROUND IMPROVEMENT PROGRAM. No new settlement analyses were performed for this 10 percent submittal and the proposed ground improvement program is largely based on work done as part of the detailed design for the buried rock seawall in the Oakwood Beach to Miller Field contract reach (i.e., Promenade reach). Other assumptions include:

- The surcharge will address primary-compression settlement (consolidation).
- The surcharge height is the same as previously developed for the Oakwood Beach to Miller Field buried rock seawall 60% design.
- The spacing and length of the wick-drains are the same as calculated for the Oakwood Beach to Miller Field buried rock seawall 60% design.
- The only adjustment to the surcharge was the surcharge footprint which is based on each of the proposed cross-sections as listed below:
  - Buried Rock Seawall (Boardwalk reach)
  - Double Sheet Pile Seawall 38-ft wide (Boardwalk reach)
- For the Buried Rock Seawall, it was assumed that the surcharge is to be accomplished in two stages: (1) Constructing both stages using the final geotextiles, bedding stone and armor stone, except for the first stage when only one layer of the armor stone across the crest of the stone (elevation 17.9) will be installed; (2) The second stage is complete the rock and fill up to elevation 21.4 feet.
- For the Double Sheet Pile Seawall, the surcharge will be accomplished using sand as discussed above. In both cases, the surcharge will be left in place for the final construction and no evaluation of fill cell size is included nor needed.
- Short-term settlement (elastic) would take place during construction and is not considered an issue for the final design.

The proposed solutions presented in this section are subject to refinement once additional subsurface information is obtained, and additional detailed calculations are performed for each of the proposed cross-sections.

Attachment 1 –

Settlement Analysis Excerpts From

60% Design Buried Rock Seawall Design Documentation Report

South Shore of Staten Island, New York
Coastal Storm Risk Management Project

Buried Seawall and Promenade from Oakwood Beach to Miller Field, Tidal Wetland, and Interior Drainage Area B

Design Documentation Report (DDR)
60% Submission

Appendix C - Geotechnical

February 2020

Prepared Under
Contract # W912DS18D0001
Delivery Order #W912DS19F0008
Design Task Order #3A

US Army Corps of Engineers
New York District

LIST OF ATTACHMENTS

3 SELECTION OF CRITICAL SECTIONS FOR GEOTECHNICAL ANALYSIS

A set of criteria was developed to select the critical sections for the geotechnical analysis of the buried rock seawall. This set included four performance categories: i) seepage, ii) slope stability, iii) settlement of buried rock seawall, and iv) high erosive area and narrow beach width. Each category had several specific criteria associated with the relevant soil and topographic conditions.

A scoring system was introduced for each criterion to make a quantitative evaluation. A lower score reflects a higher concern of the criteria and categories for the seawall section under evaluation, whereas a higher score indicates the opposite situation. The evaluation was performed on several cross sections from the end of the proposed floodwall at the Oakwood Beach Treatment Plant to Miller Field (Feasibility Study sta. 65+00 to sta.159+00).

3.1 Criteria to Select Critical Sections

3.1.1 Category – 1: Seepage

From a seepage perspective, three criteria impact the selection of the critical sections: i) elevation of the existing grade, ii) thickness of permeable soil layers, and iii) slope of the ground toward the waterside.

Elevation of the existing grade (maximum score = 20)

The exit hydraulic gradient of the buried rock seawall depends on the total head difference between the protected (landside) and unprotected (waterside) side of the seawall. Where the protected landside existing grade is lower, the total head difference between the storm still water elevation and grade becomes greater, which will increase the exit hydraulic gradient. It is estimated that the existing grade along the profile of the seawall line of protection varies from a low elevation of 2.08 feet (NAVD88) to a high elevation of 13.1 feet (NAVD88). A ranking score of zero was assigned for areas with an existing grade less than elevation 3 feet (NAVD88), whereas the maximum score of 20 was assigned for areas where the existing grade is higher than elevation 12 feet (NAVD88). Scores were proportioned for areas with grades between these extremes.

Thickness of permeable soil layer (maximum score = 10)

Permeable soil layer allows higher exit gradient and flowrate through the ground. Therefore, the maximum score of 10 was assigned for the thinner sand layer near the surface of the ground.

Slope to waterside (maximum score = 10)

Sloping ground to the waterside will increase the exit gradient by reducing the seepage path. The minimum (zero) score was assigned for the section with the steepest slope from the seawall line of protection to the waterside.

3.1.2 Category – 2: Slope Stability

From a slope stability perspective, three criteria impact the selection of the critical sections: i) slope of the existing ground, ii) thickness of cohesionless soil, and iii) thickness of cohesive soil.

Slope of the existing ground (maximum score = 20)

The slope of the existing ground will exacerbate the global slope stability. Along the seawall line of protection, the slope of the existing ground was investigated. The slopes vary from -5% (negative means a slope toward the waterside) to 5% (positive means a slope to the landside). The minimum score (zero) was assigned for the topographic condition with 5% slope, whereas the maximum score was assigned for the flat ground.

Thickness of cohesionless soil (maximum score = 10)

It can be assumed that the cohesionless soil layer is favorable for the global slope stability. Therefore, where the thickness of the sand layer near the ground surface is the greatest, the maximum score was assigned. For a continuous surface sand layer of 20 feet in depth or more a score of 10 was assigned.

Thickness of cohesive soil (maximum score = 10)

Generally, the presence of a cohesive soil also exacerbates the global slope stability. Therefore, a score of zero was used for a layer of cohesive soil of 10 ft thick, and more. The maximum score of ten was used for a thickness of less than 2 ft.

3.1.3 Category – 3: Settlement

From a settlement perspective, two criteria impact the selection of the critical sections: i) thickness of cohesionless soil, and ii) thickness of cohesive soil.

Thickness of cohesionless soil (maximum = 10)

Cohesionless soil was encountered near the ground surface along the seawall line of protection. Although such soils contribute least to long-term settlement, their presence generates short-term settlement. Therefore, the maximum score of ten was assigned for cohesionless soil layers with thickness of 9 ft and more.

Thickness of cohesive soil (maximum = 10)

Cohesive soil, where it was encountered in the vicinity of the seawall line of protection, is generally at elevations in excess of 50 feet below grade. However, such presence is still a significant contributor to long-term settlement. Therefore, the maximum score of 10 was assigned for the thinnest cohesive soil layer.

3.1.4 Category – 4: Erosion Area and Beach Width

The sections located at New Dorp Beach area from approximately sta. 125+00 to approximately 138+00 (per the feasibility study stationing) was given a score of -10, because the area was considered as narrow and highly erosive beach area.

The scoring was performed at an interval of 100 feet. The detailed criteria and scoring systems for each seawall section are summarized in Table 3-1 and Table 3-2.

Table 3-1: Summary of Criteria to Select Critical Sections (1)

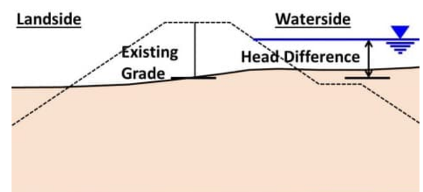
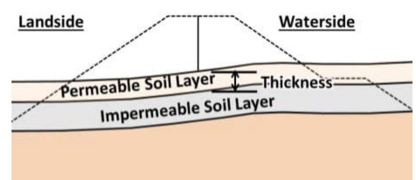
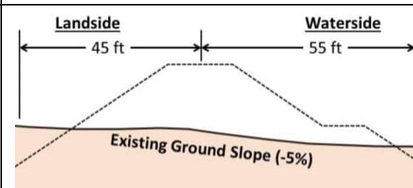
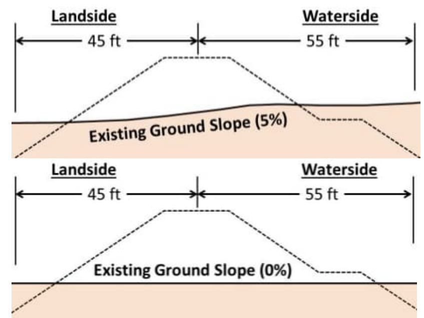
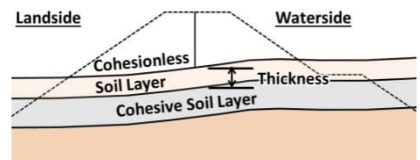
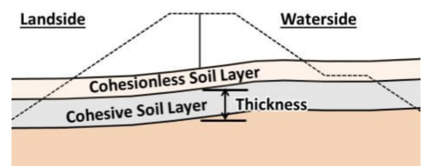
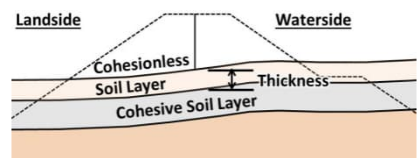
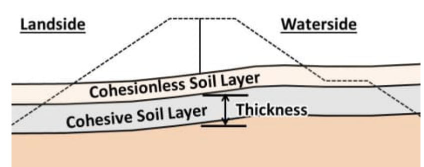
Categories	Criteria	Max. Score	Scoring System	Remarks
Seepage	Elevation of the existing grade	20	EL.2 ~ 3ft = Score 0 EL.3 ~ 4ft = Score 2 EL.4 ~ 5 ft = Score 4 EL.5 ~ 6 ft = Score 6 EL.6 ~ 7 ft = Score 8 EL.7 ~ 8 ft = Score 10 EL.8 ~ 9 ft = Score 12 EL.9 ~ 10 ft = Score 14 EL.10 ~ 11 ft = Score 16 EL.11 ~12 ft = Score 18 EL.12 ~ 14 ft = Score 20	Schematic figure to show the terms used in the criteria 
	Thickness of permeable soil (sand) layer	10	0 ~ 2 ft = Score 10 2 ~ 4 ft = Score 8 4 ~ 6 ft = Score 6 6 ~ 8 ft = Score 4 8 ~ 10 ft = Score 2 10 ~ 20 ft = Score 0	
	Slope of the existing ground to waterside	10	-5% ~ -4% = Score 0 -4% ~ -3% = Score 2 -3% ~ -2% = Score 4 -2% ~ -1% = Score 6 -1% ~ 0% = Score 8 More than 0% = Score 10	

Table 3-2: Summary of Criteria to Select Critical Sections (2)				
Categories	**Criteria**	**Max. Score**	**Scoring System**	**Remarks**
Global Slope Stability	Slope of the existing ground	20	-5% ~ -4% = Score 0 -4% ~ -3% = Score 4 -3% ~ -2% = Score 8 -2% ~ -1% = Score 12 -1% ~ 0% = Score 16 0% ~ 1% = Score 20 1% ~ 2% = Score 16 2% ~ 3% = Score 12 3% ~ 4% = Score 8 4% ~ 5% = Score 4	
	Thickness of cohesion-less soil	10	0 ~ 2 ft = Score 0 2 ~ 4 ft = Score 2 4 ~ 6 ft = Score 4 6 ~ 8 ft = Score 6 8 ~ 10 ft = Score 8 10 ~ 20 ft = Score 10	
	Thickness of cohesive soil	10	0 ~ 2 ft = Score 10 2 ~ 4 ft = Score 8 4 ~ 6 ft = Score 6 6 ~ 8 ft = Score 4 8 ~ 10 ft = Score 2 10 ~ 20 ft = Score 0	
Settlement	Thickness of cohesion-less soil	10	0 ~ 1 ft = Score 1 1 ~ 2 ft = Score 2 2 ~ 3 ft = Score 3 3 ~ 4 ft = Score 4 4 ~ 5 ft = Score 5 5 ~ 6 ft = Score 6 6 ~ 7 ft = Score 7 7 ~ 8 ft = Score 8 8 ~ 9 ft = Score 9 More than 9 ft = Score 10	
	Thickness of cohesive soil	10	0 ~ 1 ft = Score 10 1 ~ 2 ft = Score 9 2 ~ 3 ft = Score 8 3 ~ 4 ft = Score 7 4 ~ 5 ft = Score 6 5 ~ 6 ft = Score 5 6 ~ 7 ft = Score 4 7 ~ 8 ft = Score 3 8 ~ 9 ft = Score 2 More than 9 ft = Score 1	
Erosion / Narrow Beach Width		-10	-10 from Sta. 125+00 to Sta.138+00	

3.1.5 Additional Considerations

The current and planned structures along the seawall line of protection were considered in the selection of the critical sections. For example, the section at Sta.82+00 was initially included as a critical section with respect to seepage. However, a Combined Truck and Pedestrian Access Ramp was planned at this location. Therefore, critical section at sta. 82+50 was selected, instead.

If multiple sections located in a project reach were "critical" and had similar topographic and geologic characteristics, only one section was selected for detailed analysis. For example, the section at Sta. 127+00 was deemed representative of the sections from Sta.125+00 to Sta. 129+50, which was designated as a region number eight (8). The extent and characteristics of the representative regions for the critical sections selected are presented in Table 3-3 and Table 3-4.

Additionally, the section at Sta. 144+00 was considered as the most critical in Miller Field area. Therefore, Sta. 144+00 was selected as the representative section for the Miller field area.

Table 3-3: Characteristics of Regions (1)

Region No	From	To	Length (ft)	Characteristics
1	65+40	68+50	310	The region extends from the reach of task order 3A to Tarlton street. In geography, the elevation of existing grade varies from approximately 4 feet to 9 feet (NAVD88), and the ground is sloping to the waterside.
2	68+50	73+00	450	The region is located in Oakwood beach (west side) area, and the elevation of existing grade varies from approximately 3 feet to 8 feet (NAVD88). The seawall line of protection is located on the relatively flat ground, and a cohesive soil (Silty Clay to Sandy Silt) layer of 8 ft thick overlies the silty sand layer.
3	73+00	87+00	1,400	The region is located in Oakwood beach (west side) area, and the elevation of existing grade is approximately 2 feet (NAVD88) for all reaches in the region, the seawall line of protection is located on the flat ground.
4	87+00	103+00	1,600	The region is located in Oakwood beach (East side) area, and the elevation of existing grade is approximately 3 feet (NAVD88). Cohesive soil (Clayey sand) of 5 ft thick overlies the silty sand layer.

Table 3-4: Characteristics of Regions (2)				
Region No	**From**	**To**	**Length (ft)**	**Characteristics**
5	103+00	108+00	500	The region is located in Oakwood beach (East) area, and the elevation of existing grade is approximately 5 feet (NAVD88). The thickness of the cohesive soil (Clayey sand) overlying the silty sand is less than 2 feet, and the ground is mostly sloping to the landside.
6	108+00	119+00	1,100	The region is located in the lower part of Cedar Grove beach area, and the seawall line of protection is located on the ground with approximately 2% through 4% slope to the landside.
7	119+00	125+00	600	The region is located in the upper part of Cedar Grove beach area, and the seawall line of protection is located on the relatively flat ground.
8	125+00	129+50	450	The region is located in the lower part of New Dorp beach area, and the elevation of the existing grade is approximately 7 feet (NAVD88). The seawall line of protection is located on the sloping ground with approximately 1 through 5% slope.
9	129+50	142+00	1,250	The region is located in the upper part of New Dorp beach area, and the seawall line of protection is located on a narrow and highly erosive beach.
10	142+00	159+00	1,700	The region is located in Miller Field area.

3.2 Selection of Critical Sections

As presented in Table 3-5 and using the criteria discussed above, ten critical sections for geotechnical analyses have been identified and are recommended for detailed analysis.

Table 3-5: Ten Locations of the Critical Sections

Region No	From	To	Characteristics of Region	Locations of Critical Sections	Scores	Major Concerns
1	65+40	68+50	Prior to Tarlton Street	66+00	50	Slope stability
2	68+50	73+00	Oakwood Beach (west) Area	69+00	38	Settlement
3	73+00	87+00	Oakwood Beach (west) Area	82+50	50	Seepage
4	87+00	103+00	Oakwood Beach (East) Area	98+00	47	Settlement
5	103+00	108+00	Oakwood Beach (East) Area	106+00	49	Slope stability, and seepage
6	108+00	119+00	Cedar Grove Beach Area	117+00	52	Slope stability, and seepage
7	119+00	125+00	Cedar Grove Beach Area	122+00	41	Slope stability
8	125+00	129+50	New Dorp Beach Area	127+00	35	Slope stability
9	129+50	142+00	New Dorp Beach Area	130+00	54	Slope stability and the most critical in the erosive area
10	142+00	159+00	Miller Field	144+00	55	The most critical in Miller field area

The critical sections are presented on the alignment of the seawall line of protection, which is shown in Figure 3-1. The detailed evaluation and scores for each criterion are presented in Table 3-6.

Figure 3-1: Ten Critical Sections on Alignment of the Line of Protection

Table 3-6: Detailed Evaluation and Scores for each Criterion

No	Station*	Existing Grade** (ft)	Cohesionless soil thickness***	Cohesive Soil thickness	Δ (%) for Slope****	Scores					Rank	Structure	Evaluation
						Seepage	Slope	Settlement	Erosion	Total			
1	6500	9.22	8.74	8.17	4.94	26.0	14	11.00	0.0	51	19		
2	6600	7.11	7.30	8.00	3.46	24.0	16	10.00	0.0	50	16		Critical (slope stability)
3	6700	5.12	4.92	7.85	0.87	22.0	28	8.00	0.0	58	48		
4	6800	3.63	2.25	7.73	0.30	20.0	26	6.00	0.0	52	21		
5	6900	3.54	0.63	7.67	-1.40	18.0	16	4.00	0.0	38	2		Critical (settlement)
6	7000	4.90	0.61	7.61	-1.14	20.0	16	4.00	0.0	40	3		
7	7100	8.46	3.46	7.24	-0.20	28.0	22	7.00	0.0	57	44		
8	7200	4.57	0.00	6.26	-0.59	22.0	20	5.00	0.0	47	10		
9	7300	3.27	0.00	4.90	-0.19	20.0	22	7.00	0.0	49	14		
10	7400	2.80	0.00	4.74	0.08	20.0	26	7.00	0.0	53	25		
11	7500	2.27	0.84	3.91	0.00	20.0	28	8.00	0.0	56	39	74+90 Structure	
12	7600	2.68	2.14	3.67	0.00	16.0	26	10.00	0.0	52	22	76+00 Structure	
13	7700	2.78	3.33	3.23	0.00	16.0	26	11.00	0.0	53	26		
14	7800	2.08	3.71	2.68	0.00	18.0	30	12.00	0.0	60	53	78+00 Structure	
15	7900	2.15	4.48	2.30	0.00	14.0	28	13.00	0.0	55	33		
16	8000	2.08	4.63	2.11	0.00	14.0	28	13.00	0.0	55	34		
17	8100	2.16	6.29	0.34	0.00	14.0	36	17.00	0.0	67	74		
18	8200	2.83	4.27	5.75	0.00	14.0	26	10.00	0.0	50	17		Critical (seepage) Adjust to Sta.82+50
19	8300	2.28	3.91	4.40	0.00	18.0	28	10.00	0.0	56	40		
20	8400	2.29	4.62	1.79	0.00	16.0	34	14.00	0.0	64	67		
21	8500	2.27	20.00	0.00	0.00	8.0	36	20.00	0.0	64	68		
22	8600	2.25	20.00	0.00	0.00	10.0	40	20.00	0.0	70	83		
23	8700	2.19	20.00	0.00	-0.46	8.0	36	20.00	0.0	64	69		
24	8800	2.22	20.00	0.00	-0.22	8.0	36	20.00	0.0	64	70		
25	8900	2.40	20.00	0.00	0.19	10.0	40	20.00	0.0	70	84		
26	9000	2.29	20.00	0.00	0.88	10.0	40	20.00	0.0	70	85		
27	9100	3.06	20.00	0.00	1.50	12.0	36	20.00	0.0	68	76	90+70 Structure	
28	9200	3.14	1.73	1.35	0.82	22.0	30	11.00	0.0	63	65	92+10 Structure	
29	9300	2.20	1.76	4.54	0.14	20.0	26	8.00	0.0	54	28		
30	9400	2.92	3.53	7.44	0.34	18.0	26	7.00	0.0	51	20	94+10 Structure	
31	9500	3.71	3.06	6.36	0.31	20.0	26	8.00	0.0	54	29		
32	9600	3.64	0.98	4.89	0.30	22.0	26	7.00	0.0	55	35		
33	9700	2.54	0.00	3.64	0.24	20.0	28	8.00	0.0	56	41		
34	9800	2.35	0.00	4.14	0.00	18.0	22	7.00	0.0	47	11		Critical (settlement)
35	9900	2.41	0.00	4.16	0.00	18.0	22	7.00	0.0	47	12		
36	10000	2.83	0.57	3.25	0.51	20.0	28	8.00	0.0	56	42		
37	10100	3.51	1.10	2.71	0.82	22.0	28	10.00	0.0	60	54		
38	10200	4.04	1.51	2.60	1.32	24.0	24	10.00	0.0	58	49		
39	10300	3.10	0.55	2.52	0.73	22.0	28	9.00	0.0	59	50		
40	10400	5.46	2.97	2.14	1.95	24.0	26	11.00	0.0	61	59		
41	10500	6.74	4.35	1.39	1.47	24.0	30	14.00	0.0	68	77	105+00	
42	10600	4.33	2.07	0.73	-2.36	16.0	20	13.00	0.0	49	15		Critical (slope, seepage) Adjust to Sta.105+70
43	10700	4.01	1.89	0.63	-1.64	20.0	22	12.00	0.0	54	30		
44	10800	10.13	8.09	1.14	-3.59	20.0	22	18.00	0.0	60	55		
45	10900	11.49	9.49	1.94	-2.97	24.0	26	19.00	0.0	69	81		
46	11000	11.92	9.85	2.76	-2.81	24.0	24	18.00	0.0	66	72		
47	11100	12.14	9.91	3.38	-1.90	28.0	28	17.00	0.0	73	89		
48	11200	7.40	4.88	3.65	-2.77	20.0	20	12.00	0.0	52	23		
49	11300	8.53	5.61	3.58	-1.93	24.0	24	13.00	0.0	61	60		
50	11400	8.59	5.23	3.30	-2.01	22.0	20	13.00	0.0	55	36		
51	11500	10.48	6.69	2.94	-2.32	24.0	22	15.00	0.0	61	61		
52	11600	13.06	8.92	2.61	-2.44	26.0	24	17.00	0.0	67	75		
53	11700	7.90	3.53	2.46	-2.89	22.0	18	12.00	0.0	52	24		Critical (slope, seepage)
54	11800	8.08	3.61	2.52	-1.74	26.0	22	12.00	0.0	60	56		
55	11900	9.32	4.85	2.75	-2.66	24.0	20	13.00	0.0	57	45	118+90	
56	12000	10.09	5.62	3.10	0.36	32.0	32	13.00	0.0	77	92		
57	12100	6.19	1.70	3.54	2.63	28.0	20	9.00	0.0	57	46		
58	12200	5.36	0.76	4.01	-2.30	20.0	14	7.00	0.0	41	4		Critical (slope)
59	12300	6.41	1.63	4.45	-1.83	24.0	18	8.00	0.0	50	18		
60	12400	5.18	0.23	4.82	0.11	26.0	26	7.00	0.0	59	51		
61	12500	6.11	1.10	5.04	-0.92	26.0	22	7.00	-10.0	45	8		
62	12600	7.18	2.31	5.06	-1.58	24.0	20	8.00	-10.0	42	5		
63	12700	7.23	2.77	4.81	-4.96	18.0	8	9.00	-10.0	25	1		Critical (slope)
64	12800	7.49	3.73	4.24	-1.76	24.0	20	10.00	-10.0	44	6		
65	12900	6.44	3.64	3.31	-1.35	22.0	22	11.00	-10.0	45	9		
66	13000	7.54	5.74	2.12	-2.25	20.0	20	14.00	-10.0	44	7		Critical (slope, erosion)
67	13100	5.65	4.37	1.13	-1.91	18.0	26	14.00	-10.0	48	13		
68	13200	7.61	5.84	0.80	-0.16	24.0	30	16.00	-10.0	60	57		
69	13300	7.78	4.75	1.03	0.01	26.0	34	14.00	-10.0	64	71		
70	13400	7.30	2.76	1.53	-0.76	26.0	28	12.00	-10.0	56	43		
71	13500	8.57	2.77	2.08	0.42	30.0	30	11.00	-10.0	61	62		
72	13600	8.23	1.75	2.54	-0.47	30.0	24	10.00	-10.0	54	31		
73	13700	8.15	1.82	2.84	-0.22	30.0	24	10.00	-10.0	54	32		
74	13800	8.06	2.51	2.96	2.09	30.0	22	11.00	-10.0	53	27		
75	13900	6.90	2.53	2.89	1.15	26.0	26	11.00	0.0	63	66	139+30 Outfall	
76	14000	7.85	4.83	2.67	-1.91	22.0	24	13.00	0.0	59	52		
77	14100	8.19	6.53	2.35	0.72	26.0	34	15.00	0.0	75	91		
78	14200	7.52	6.91	2.18	1.09	24.0	30	15.00	0.0	69	82		
79	14300	11.47	11.69	2.03	-3.19	20.0	22	18.00	0.0	60	58	Miller filed	
80	14400	10.86	11.52	1.99	-4.47	16.0	20	19.00	0.0	55	37		Critical (Miller field)
81	14500	12.55	13.18	2.08	-3.23	22.0	22	18.00	0.0	62	64		
82	14600	12.13	12.33	2.23	-2.12	24.0	26	18.00	0.0	68	78		
83	14700	12.33	11.93	2.32	-2.29	24.0	26	18.00	0.0	68	79		
84	14800	12.01	11.04	2.21	-2.60	24.0	26	18.00	0.0	68	80		
85	14900	9.85	8.43	1.86	-1.06	22.0	30	18.00	0.0	70	86		
86	15000	11.04	9.23	1.42	-0.77	28.0	34	19.00	0.0	81	95		
87	15100	10.13	7.85	1.07	-1.06	26.0	28	17.00	0.0	71	88		
88	15200	12.16	9.24	1.00	-1.49	28.0	30	20.00	0.0	78	94		
89	15300	11.03	7.23	1.22	-3.40	24.0	20	17.00	0.0	61	63		
90	15400	11.86	6.93	1.66	-2.52	26.0	24	16.00	0.0	66	73		
91	15500	12.61	6.44	2.10	-0.21	32.0	30	15.00	0.0	77	93		
92	15600	7.54	0.14	2.57	-1.45	26.0	20	9.00	0.0	55	38		
93	15700	10.29	2.50	2.48	-2.05	28.0	18	11.00	0.0	57	47		
94	15800	9.90	2.72	1.92	-0.03	30.0	28	12.00	0.0	70	87		
95	15900	10.28	4.20	1.35	-0.08	30.0	30	14.00	0.0	74	90		

* Stationing is based on the Feasibility Study ** Existing grade is based on NAVD88 *** Cohesionless soil thickness = permeable soil thickness

**** Δ (%) = Slope of ground, which means that the difference of existing grade of the end of 100 ft wide line of protection
= positive (sloping to landside), negative (sloping to waterside)

5 SETTLEMENT CALCULATIONS

This section presents the results of the analysis to verify the consolidation and immediate settlement that will occur due the construction of the proposed Buried Seawall and Promenade from Oakwood Beach to Miller Field at South Shore of Staten Island, NY. It also discusses the solutions to mitigate the settlement of the clay layers

The settlement was estimated at the center and at the toe of buried rock seawall, as shown in Figure 5-1, using data from Standard Penetration test (SPT) DH-10 and laboratory consolidation tests from boring logs DH-7 and DH-8.

5.1 Construction Sequence

The buried rock seawall construction is planned to follow the sequence listed below:

1. Excavate to structure toe elevation.
2. Install wick drains.
3. Install sheetpile.
4. Place sand fill (core of structure). Reuse excavated material if suitable.
5. Place Geotextile.
6. Place Bedding layer.
7. Place Underlayer.
8. Place bottom half of armor layer to elevation +17.4 ft (approximately).
9. Place sand cover up to elevation +17 ft (approximately).
10. Finish construction of 9,400 ft line of protection (LOP) using this "short" section
11. Install Concrete Cap
12. Start at the beginning and "top off" the short section by placing top half of armor layer to design crest elevation. This assumes most of the settlement has already occurred (see timing below)
13. Construct promenade

The detailed construction sequence, including sketches, is presented in Attachment C.1.

For the settlement calculations it was assumed that the construction sequence listed above will happen in three stages, and each stage can have one or two construction fronts. The detailed construction stages are listed below and presented in Figure 5-1:

- **Stage 1** This stage includes the construction steps 1 through 11, from the list above. During this stage the buried rock seawall will be constructed until elevation 17.4 ft (wall height of 15.5 ft), which will take approximately 20 months assuming one construction front or 10 months assuming two construction fronts.
- **Stage 2** This stage includes construction step 12. During this stage the buried rock seawall will be raised from Elevation 17.4 ft to Elevation 21.4 ft. The 1 ft thick sand cover layer was also considered in the calculations , so the final elevation for this stage was El. 22.4 ft,

resulting in a wall 20.5 ft high. This stage is expected to take approximately 4 months considering one construction front or 2 months assuming two construction fronts.

- **Stage 3** This stage refers to the construction of the promenade (step 13). No geotechnical calculation is needed for this step.

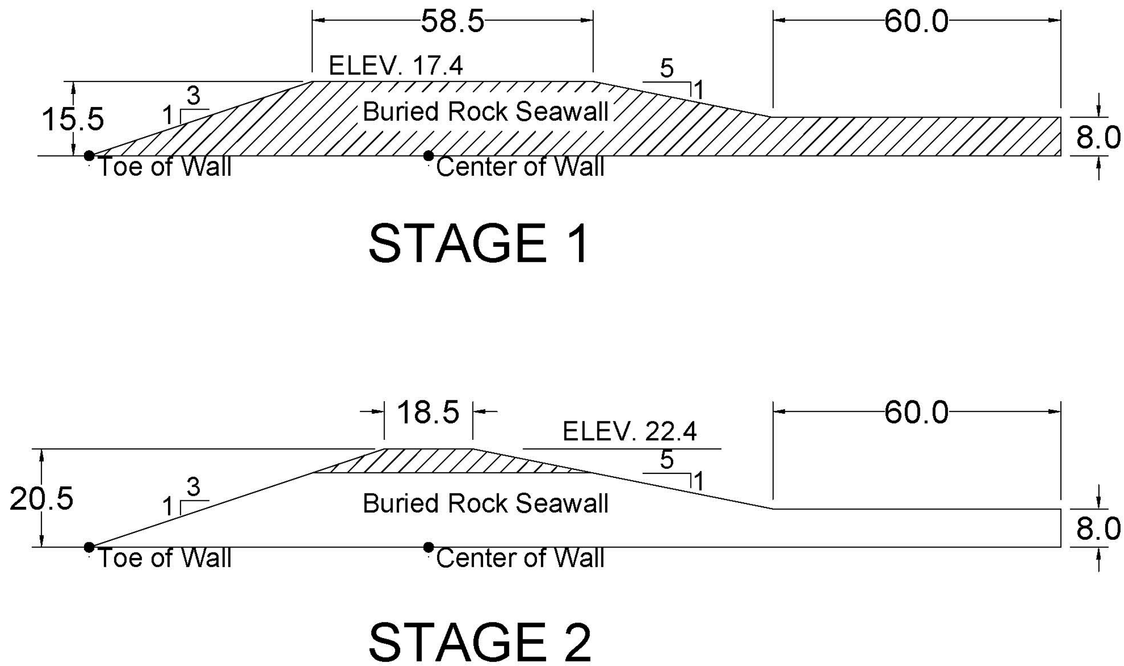


Figure 5-1: Locations of Calculated Settlement in Rock Buried Seawall (Unit in Feet)

5.2 Settlement analysis

The calculated primary consolidation settlement at the center and the toe of seawall due to the loads of Stage 1 construction is 7.2 inch and 3.1 inch, respectively. Refer to Attachment C.2 for the detailed calculations.

Table 5-1 summarizes settlement at both center and toe of seawall at final condition, after completion of stage 2 (refer to Attachment C.2). From the analysis, there is a differential settlement of 5.8 inch between the toe and center of the Buried Rock Seawall.

Table 5-1: Summary of Total Settlement (After completion of Stage 2)

Item	Unit	Center of Seawall	Toe of Seawall
Elastic Settlement Above Elev. -32.5 ft (1)	(inch)	1.5	0.8
Elastic Settlement below Elev. -32.5 ft.	(inch)	0.9	0.5
Primary Consolidation Settlement	(inch)	8.1	3.3
Secondary Consolidation Settlement (50 Yrs)	(inch)	0.6	0.6
Total Settlement	**(inch)**	**11.1**	**5.3**
Note: For settlement at center of levee, this reflects that the upper 5 ft of subgrade will be over-excavated and replaced with competent material, despite 11.3 inch present at DMT-4 log, provided by USACE.			

Based on the construction steps described in Attachment C.1, the length for the Oakwood Beach to Miller Field Contract is 9,400 feet. The consolidation settlement of Stage 2 is considered after the conclusion of this stage. Both one construction front and two construction fronts are analyzed for stage 2 construction. The analyses indicated that the number of construction fronts is not expected to impact the primary consolidation settlement. All the calculations are detailed in Attachment C.2.

One Construction Front:

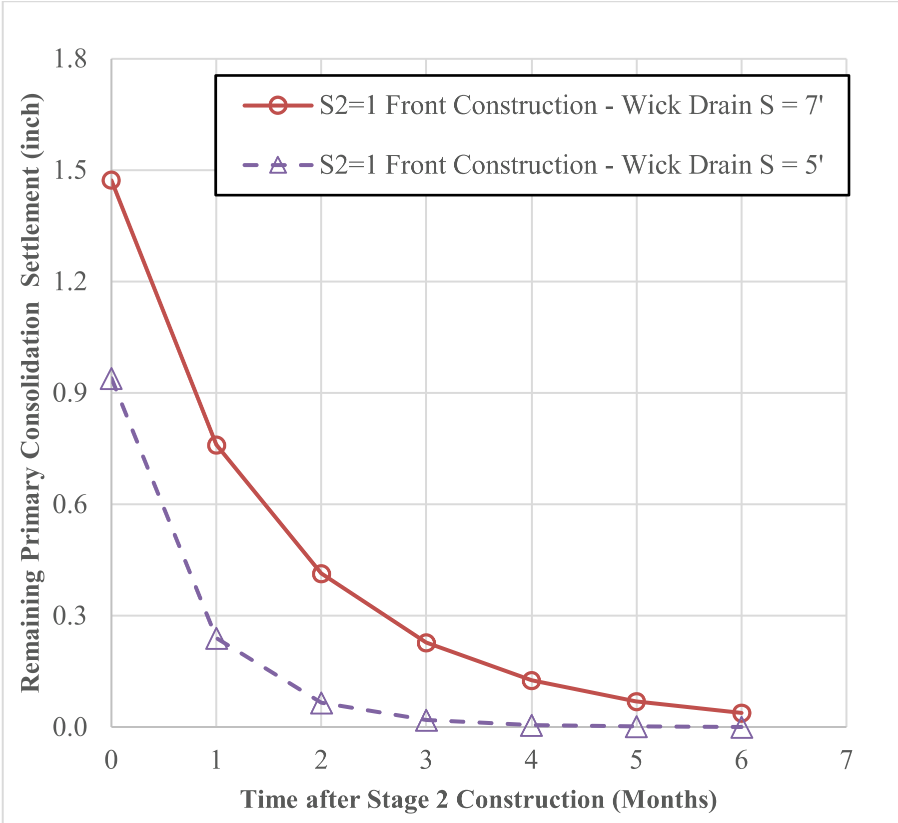


Figure 5-2: Time Rate Primary Consolidation Settlement after One Front Construction Stage 2

The construction of Stage 2 will resume in four (4) months after the construction of Stage 1 is completed. Figure 5-2 presents the remaining primary consolidation settlement at the center of the seawall versus time after the completion of construction of Stage 2 assuming different wick drain spacings. As indicated in the figure, the remaining primary consolidation settlements at the end of construction at wick drain spacing of 7.0 feet and 5.0 feet are 1.5 inch and 0.9 inch, respectively. The remaining settlements decrease to 0.8 inch and 0.2 inch, respectively one month after the conclusion of Stage 2.

Two Construction Fronts:

The construction of Stage 2 will end two (2) months after the completion of Stage 1. Figure 5-3 presents the remaining primary consolidation settlement at the center of the seawall versus time after the completion of construction Stage 2 at different wick drain spacings. As indicated in the figure, the remaining primary consolidation settlements at the end of construction at wick drain spacing of 7.0 feet and 5.0 feet are 2.8 inch and 1.4 inch, respectively. The remaining settlements decrease to 1.5 inch and 0.4 inch, respectively one month after the conclusion of stage 2.

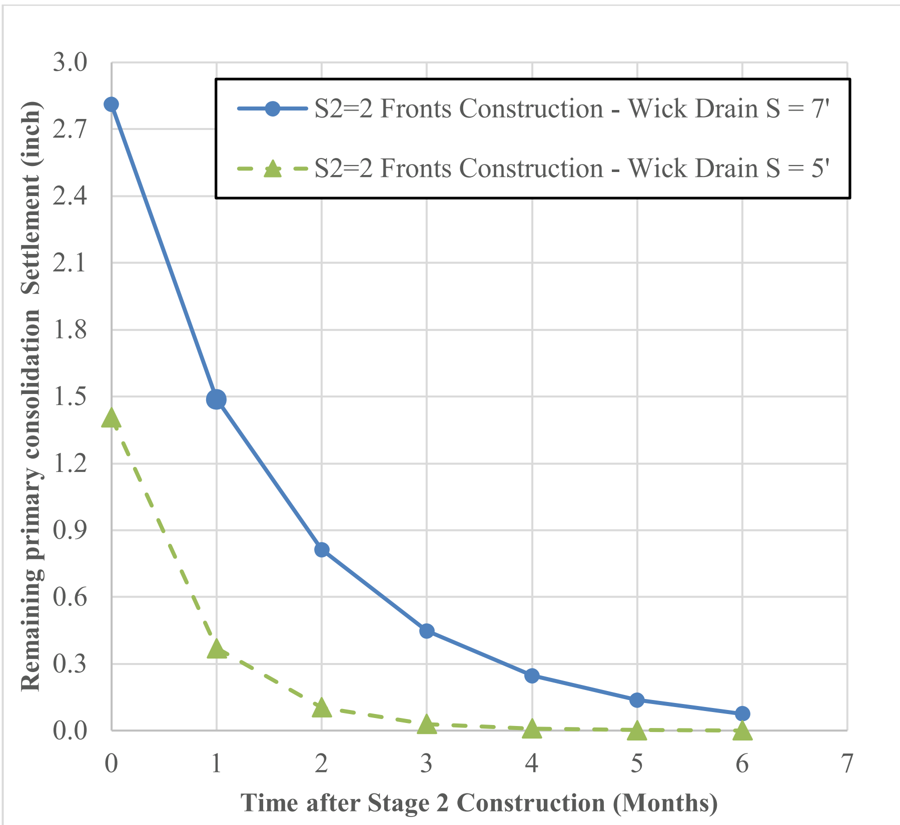


Figure 5-3: Time Rate Primary Consolidation Settlement after Two Fronts Construction Stage 2

Based on the settlement calculations the conclusions are summarized below:

- To minimize the remaining primary consolidation settlement the recommended wick drain spacing is 5.0 feet. The remaining primary consolidation settlement after 1 month from the conclusion of stage 2 construction is 0.4 inch when assuming two construction fronts and 0.2 inch assuming one construction front. The total secondary consolidation settlement is 0.6 inch for service life of 50 years.
- A sand blanket of at least three feet thick should be placed after the wick drains installetion to serve as a drainage blanket for water control and to distribute the loads evenly.

Attachment C.2

Settlement Calculation

Summary of Primary Consolidation Stettlement

Const. Front(s)	Time after completion of Stage 1 constrution	Time after completion of Stage 2 constrution	Wick drain spacing	Total Settlement due Stage 1 load	Total Settlement due Stage 2 load	Remaining Settlement of Stage 1 (After completion of Stage 2 Construction)	Remaining Settlement of Stage 2 (After completion of Stage 2 Construction)	Total Remaining Settlement
(--)	(Months)	(Months)	(feet)	(inch)	(inch)	(inch)	(inch)	(inch)
1	4	0	7.0	7.2	0.9	0.6	0.9	1.5
		1	7.0	7.2	0.9	0.3	0.4	0.8
		2	7.0	7.2	0.9	0.2	0.2	0.4
		3	7.0	7.2	0.9	0.1	0.1	0.2
		4	7.0	7.2	0.9	0.1	0.1	0.1
		5	7.0	7.2	0.9	0.0	0.0	0.1
		6	7.0	7.2	0.9	0.0	0.0	0.0
	4	0	5.0	7.2	0.9	0.0	0.9	0.9
		1	5.0	7.2	0.9	0.0	0.2	0.2
		2	5.0	7.2	0.9	0.0	0.1	0.1
		3	5.0	7.2	0.9	0.0	0.0	0.0
		4	5.0	7.2	0.9	0.0	0.0	0.0
		5	5.0	7.2	0.9	0.0	0.0	0.0
		6	5.0	7.2	0.9	0.0	0.0	0.0
2	2	0	7.0	7.2	0.9	1.9	0.9	2.8
		1	7.0	7.2	0.9	1.0	0.4	1.5
		2	7.0	7.2	0.9	0.6	0.2	0.8
		3	7.0	7.2	0.9	0.3	0.1	0.4
		4	7.0	7.2	0.9	0.2	0.1	0.2
		5	7.0	7.2	0.9	0.1	0.0	0.1
		6	7.0	7.2	0.9	0.1	0.0	0.1
	2	0	5.0	7.2	0.9	0.5	0.9	1.4
		1	5.0	7.2	0.9	0.1	0.2	0.4
		2	5.0	7.2	0.9	0.0	0.1	0.1
		3	5.0	7.2	0.9	0.0	0.0	0.0
		4	5.0	7.2	0.9	0.0	0.0	0.0
		5	5.0	7.2	0.9	0.0	0.0	0.0
		6	5.0	7.2	0.9	0.0	0.0	0.0

Project No.:	60594417		AECOM Clifton, NJ
Subject :	SSSI Task Order 3A (H = 15.5 ft S = 5 ft)		
	- Settlement Calculation Summary		
Computed By: QH	Checked By: LC	Date: 10/21/2019	
Computed By:	Checked By:	Rev.	

Problem Statement:

Summary of the settlement calculation results at Station 29+00.

Reference:

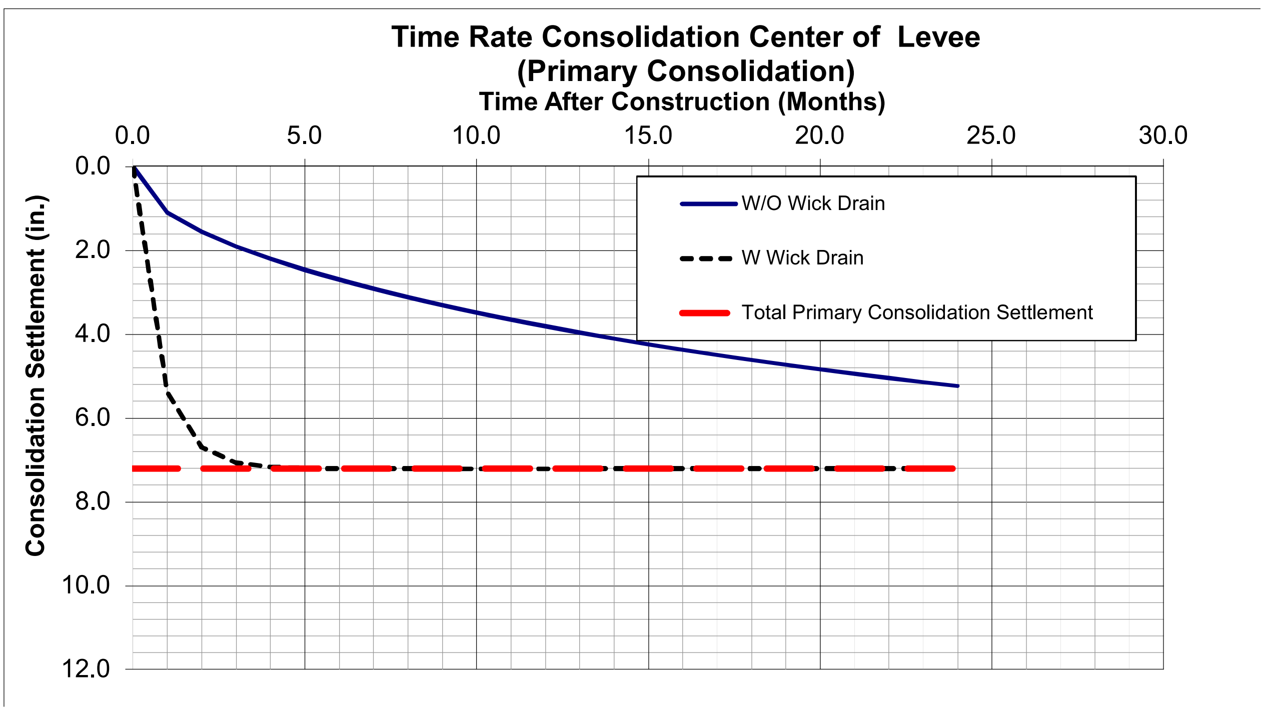
1. Das, B. M., *Principles of Foundation Engineering, Nelson, Ontario, Canada, 750 p 7th Edition, Dated 2007*
2. Naval Facilities Engineering Command (NAFAC), *Soil Mechanics Design Manual 7.01* , Dated September 1986. (DM 7.01)
3. AASHTO, *LRFD Bridge Specification 7th Edition* , 2014
4. Poulos, H. and Davis, E. *Elastic Solutions for Soil and Rock Mechanics* . John Wiley & Sons, Inc. (1974)

Summary

- Station: 29+00
- Embankment Height: 15.5 ft
- Embankment Slope at land side: 3 H: 1V
- Boring used in the model: DH-10
- Settlement Summary

	Item	Unit	Center of Levee	Toe of Levee
4.	Primary Consolidation Settlement	(inch)	**7.2**	**3.1**

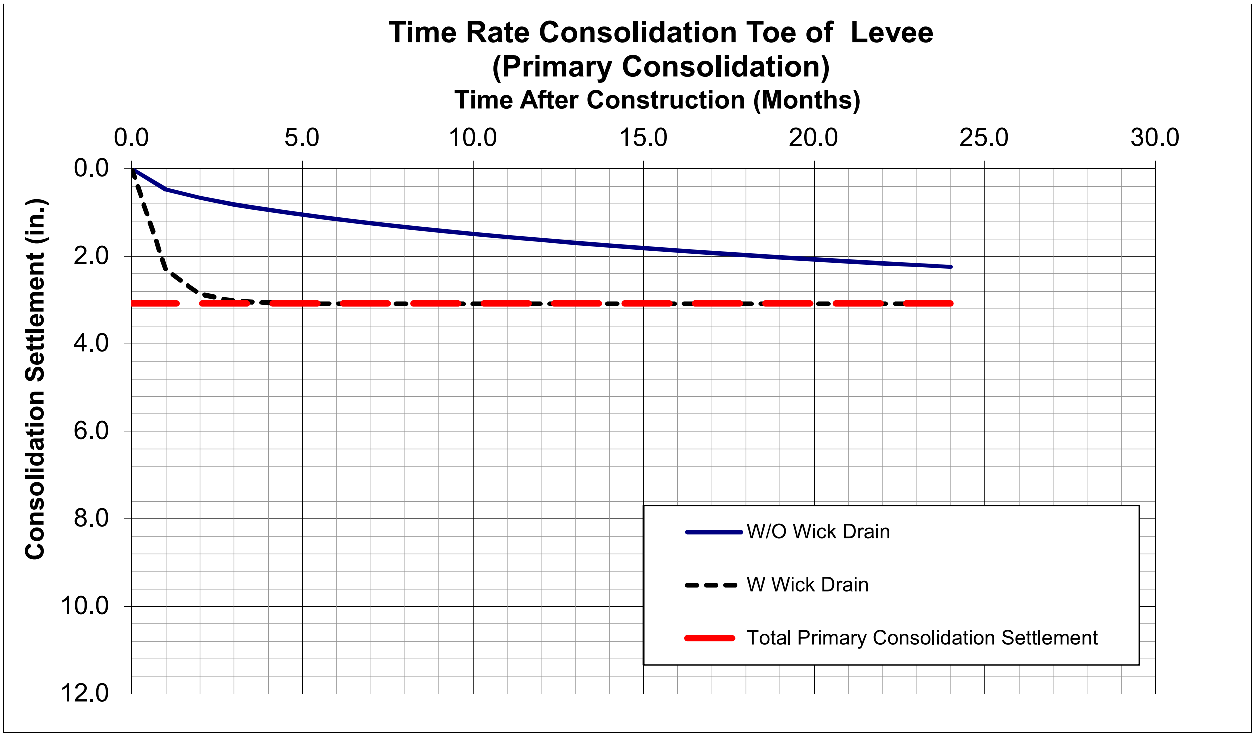
- Drain Pattern	Triangular		Assumed
Drain Spacing, S =	5.00	feet	Assumed
Wick Drain Width =	4	inch	Assumed
Wick Drain Thickness =	0.13	inch	Assumed



Project No.:	60594417
Subject :	SSSI Task Order 3A (H = 15.5 ft S = 5 ft)
	- Settlement Calculation Summary

AECOM
Clifton, NJ

Computed By:	QH	Checked By:	LC	Date:	10/21/2019
Computed By:		Checked By:		Rev:	



Parameters:

- Boring DH-10 was used in the analysis.
- Equivalent unit of weight of embankment is 130 pcf
- The following parameters were used in the calculation:

Layer No.	Soil Description	Total Unit Weight	Layer Thickness
		(pcf)	(ft)
1	Clayey Sand 1	120	2
2	Silty Sand 1	125	45
3	Lean clay 1	120	32
4	Silty Sand 2	130	42
5	Lean clay 2 (1)	120	15

Note:

1. Thickness of this layer is based on termination of boring.

Clay Layer	Compression Index	Secondary Compression Ratio	Void Ratio	Coef. Of Consolidation
	C_c	C_α'	e_0	C_v
	(---)	(---)	(---)	(ft^2/Day)
1	0.212	0.0015	0.785	0.337
2	0.150	0.0011	0.751	0.337

Project No.: 60594417		AECOM Clifton, NJ
Subject : SSSI Task Order 3A (H = 15.5 ft S= 5 ft) - Consolidation Settlement		
Computed By: QH	Checked By: LC	Date: 10/21/2019
Computed by:	Checked By:	Rev:

Problem Statement:

Estimate the secondary consolidation settlement .

Reference:

1. Das, B. M., *Principles of Foundation Engineering, Nelson, Ontario, Canada, 750 p 7th Edition, Dated 2007*
2. Naval Facilities Engineering Command (NAFAC), *Soil Mechanics Design Manual 7.01* , Dated September 1986. (DM 7.01)

Assumptions:

- The following subsurface profile was used in the calculation

Layer No.	Soil Description	Total Unit Weight	Layer Thickness
		(pcf)	(ft)
1	Clayey Sand 1	120	2
2	Silty Sand 1	125	45
3	Lean clay 1	120	32
4	Silty Sand 2	130	42
5	Lean clay 2	120	15

- Current Ground Surface El.	2.2	ft
Groundwater Table El.	-3.5	ft
Ground Water Table from Ground Surface	5.7	ft

- The following EXCEPT C_α' were adopted from Laboratory Results:

Clay Layer	Compression Index	Secondary Compression Index	Void Ratio	Coef. Of Consolidation
	C_c	C_α'	e_0	C_v
	(---)	(---)	(---)	(ft^2/Day)
1	0.212	0.0015	0.785	0.337
2	0.150	0.0011	0.751	0.337

- Based on laboratory results, both clays are normally consolidated

Project No.: 60594417
Subject : SSSI Task Order 3A (H = 15.5 ft S= 5 ft)
- Consolidation Settlement

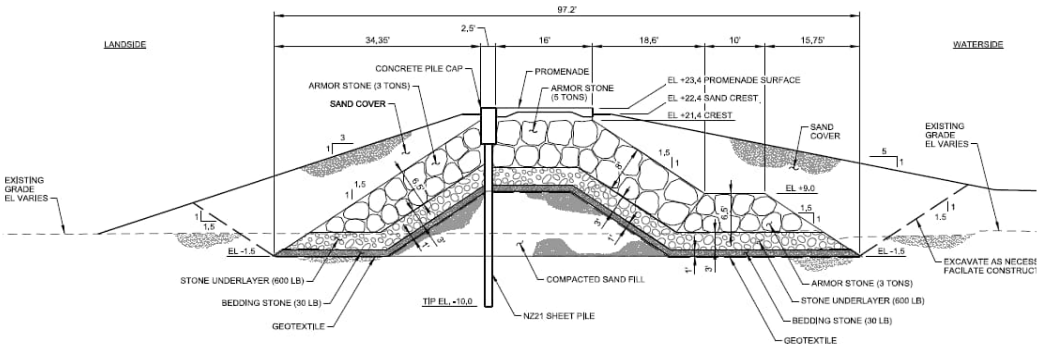
AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019
Computed by: Checked By: Rev:

Calculations:

1. *Embankment Loading*

- The proposed embankment is shown below:



2. *Consolidation Settlement*

Total Thickness of Clay used for primary consolidation, H_{clay} = **47** ft

Unit Weight of Sea Water, γ_w = 64 pcf

2a. Primary Settlement under A. Final Condition and B. permanent load and surcharge load

Clay Layer	From Ground Surface to Mid Point of Layer	Layer Thickness	Initial Effective Vertical Stress	Increase of Effective Vertical Stress At Final Condition (Center of Levee)	Increase of Effective Vertical Stress At Final Condition (Toe of Levee)	Final Effective Vertical Stress At Final Condition (Center of Levee)	Final Effective Vertical Stress At Final Condition (Toe of Levee)	Compression Index	Void Ratio	Primary Consolidation Settlement at Final Condition (Center of Levee)		Primary Consolidation Settlement at Final Condition (Toe of Levee)	
	z	H	σ_{v0}'	p_{fc}	p_{fT}	σ_{fC}'	σ_{fT}'	C_c	e_0	δ_f		δ_f	
	(ft)	(ft)	(psf)	(psf)	(psf)	(psf)	(psf)	(---)	(---)	(ft)	(inch)	(ft)	(inch)
	(1)	(2)	(3)	(4)	(4)	(6)	(6)	(8)	(9)	(10)		(10)	
1	63	32	4119.1	1564.9	572.4	5684.0	4691.5	0.212	0.785	0.53	6.4	0.215	2.6
2	128.5	15	8207.1	1086.9	651.7	9294.0	8858.7	0.15	0.751	0.07	0.8	0.043	0.5
										0.6	**7.2**	**0.3**	**3.1**

Note:

3. For Clay Layer 1, $\sigma_{v0}' = \gamma 1\, H1 + \gamma 2 * (Hw - H1) + (\gamma 2 - Yw) * (H1 + H2 - Hw) + (\gamma 3 - Yw) * H3 / 2$

   For Clay Layer 2, $\sigma_{v0}' = \gamma 1\, H1 + \gamma 2 * (Hw - H1) + (\gamma 2 - Yw) * (H1 + H2 - Hw) + (\gamma 3 - Yw) * H3 + (\gamma 4 - Yw) * H4 + (\gamma 5 - Yw) * H5 / 2$

4. p_f see attached calculation

6. $\sigma_f' = \sigma_{v0}' + p_f$

10. $\delta_f = C_c * H / (1+e_0) * \text{Log}(\sigma_f' / \sigma_{v0}')$ Ref. 1 Eq. 5.81

Project No.:	60594417
Subject :	SSSI Task Order 3A (H = 15.5 ft S= 5 ft) - Consolidation Settlement

AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019
Computed by: Checked By: Rev:

Radial drainage (wick drains)

d_e = effective zone from which the radial drainage will be directed toward the wick drain

r_w = equivalent diameter of wick drain radius

a =width of wick drain= 4 in

b =thickness of wick drain= 0.13 in

T_r =Time factor with radial drainage

C_v = C_{vr} = Coefficient of Consolidation

*No soil smear assumed

Drain Spacing, S = 5.00 feet

Placement Pattern Triangular

d_e = 5.25 feet = 1.05 * S for Triangular Pattern, = 1.13 * S for Square Pattern

Drainage Path Double

2b. Degree of Consolidation with wick drain

Below is the calculation show the degree of consolidation, primary consolidation settlement versus time after construction.

t							Double Drainage						Center of Levee		Toe of Levee	
		C_v	d_e	r_w	n	m	T_r	U_r	T_v	H_{dr}	U_v	$U_{v,r}$	$S_{iv}c$	$S_{iv,r}c$	S_{ivT}	$S_{iv,rT}$
(Months)	(Days)	(ft^2/Day)	(ft)	(ft)	-	-	-	-	-	ft	-	-	(in.)	(in.)	(in.)	(in.)
(1)		(2)	(3)	(4)	(5)	(6)	(6)	(8)	(10)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
0.0	0	0.3372	5.25	0.11	23.95	2.43	0.00	0.00	0.000	23.5	0.000	0.000	0.0	0.0	0.0	0.0
1.0	30	0.3372	5.25	0.11	23.95	2.43	0.37	0.70	0.018	23.5	0.153	0.747	1.1	5.4	0.5	2.3
2.0	60	0.3372	5.25	0.11	23.95	2.43	0.73	0.91	0.037	23.5	0.216	0.930	1.6	6.7	0.7	2.9
3.0	90	0.3372	5.25	0.11	23.95	2.43	1.10	0.97	0.055	23.5	0.265	0.980	1.9	7.1	0.8	3.0
4.0	120	0.3372	5.25	0.11	23.95	2.43	1.47	0.99	0.073	23.5	0.305	0.994	2.2	7.2	0.9	3.1
5.0	150	0.3372	5.25	0.11	23.95	2.43	1.84	1.00	0.092	23.5	0.341	0.998	2.5	7.2	1.1	3.1
6.0	180	0.3372	5.25	0.11	23.95	2.43	2.20	1.00	0.110	23.5	0.374	1.000	2.7	7.2	1.2	3.1
7.0	210	0.3372	5.25	0.11	23.95	2.43	2.57	1.00	0.128	23.5	0.404	1.000	2.9	7.2	1.2	3.1
8.0	240	0.3372	5.25	0.11	23.95	2.43	2.94	1.00	0.147	23.5	0.432	1.000	3.1	7.2	1.3	3.1
9.0	270	0.3372	5.25	0.11	23.95	2.43	3.30	1.00	0.165	23.5	0.458	1.000	3.3	7.2	1.4	3.1
10.0	300	0.3372	5.25	0.11	23.95	2.43	3.67	1.00	0.183	23.5	0.483	1.000	3.5	7.2	1.5	3.1
11.0	330	0.3372	5.25	0.11	23.95	2.43	4.04	1.00	0.201	23.5	0.507	1.000	3.7	7.2	1.6	3.1
12.0	360	0.3372	5.25	0.11	23.95	2.43	4.40	1.00	0.220	23.5	0.529	1.000	3.8	7.2	1.6	3.1
13.0	390	0.3372	5.25	0.11	23.95	2.43	4.77	1.00	0.238	23.5	0.550	1.000	4.0	7.2	1.7	3.1
14.0	420	0.3372	5.25	0.11	23.95	2.43	5.14	1.00	0.256	23.5	0.569	1.000	4.1	7.2	1.8	3.1
15.0	450	0.3372	5.25	0.11	23.95	2.43	5.51	1.00	0.275	23.5	0.588	1.000	4.2	7.2	1.8	3.1
16.0	480	0.3372	5.25	0.11	23.95	2.43	5.87	1.00	0.293	23.5	0.607	1.000	4.4	7.2	1.9	3.1
17.0	510	0.3372	5.25	0.11	23.95	2.43	6.24	1.00	0.311	23.5	0.624	1.000	4.5	7.2	1.9	3.1
18.0	540	0.3372	5.25	0.11	23.95	2.43	6.61	1.00	0.330	23.5	0.641	1.000	4.6	7.2	2.0	3.1
19.0	570	0.3372	5.25	0.11	23.95	2.43	6.97	1.00	0.348	23.5	0.657	1.000	4.7	7.2	2.0	3.1
20.0	600	0.3372	5.25	0.11	23.95	2.43	7.34	1.00	0.366	23.5	0.672	1.000	4.8	7.2	2.1	3.1
21.0	630	0.3372	5.25	0.11	23.95	2.43	7.71	1.00	0.385	23.5	0.686	1.000	4.9	7.2	2.1	3.1
22.0	660	0.3372	5.25	0.11	23.95	2.43	8.07	1.00	0.403	23.5	0.700	1.000	5.0	7.2	2.2	3.1
23.0	690	0.3372	5.25	0.11	23.95	2.43	8.44	1.00	0.421	23.5	0.713	1.000	5.1	7.2	2.2	3.1
24.0	720	0.3372	5.25	0.11	23.95	2.43	8.81	1.00	0.440	23.5	0.726	1.000	5.2	7.2	2.2	3.1

Note:

4. $r_w = 2\ (a + b) / \pi / 2$ Ref. 1 Eq. 14.33

5. $n = d_e / 2\ r_w$ Ref. 1 Eq. 14.21

6. $m = \left(\frac{n^2}{n^2 - 1}\right)\ln(n) - \frac{3n^2 - 1}{4n^2}$ Ref. 1 Eq. 14.25 for non-smear case.

8. $U_v = (4 * T_v / \pi)^{0.5}$ U_v in Decimal If T_v <= 0.283 Ref. 1 Eq. (1.74)

U_v = 1- 10^ [(T_v + 0.085)/(-0.933)] If T_v > 0.283 Ref. 1 Eq. (1.75)

9. $U_{v,r} = 1 - (1 - U_v)(1 - U_r)$

10. $\delta_{iv} = \delta_f * U_v$

11. $\delta_{iv,r} = \delta_f * U_{v,r}$

Project No.:	60594417
Subject :	SSSI Task 3A (H = 15.5 ft S= 5 ft)
	- Induced Stress under Embankment Load (Center of Levee)
Computed By: QH	Checked By: LC	Date: 10/21/2019
Computed By:	Checked By:	Rev:

AECOM
Clifton, NJ

Problem Statement:

Calculate the stress at different depths of layers under permanent embankment load. See Figure 1 on NEXT SHEET for shape of embankment.

Note: The entire embankment is shown in Figure 1. b is the width of embankment. b is NOT half embankment width.

Reference:

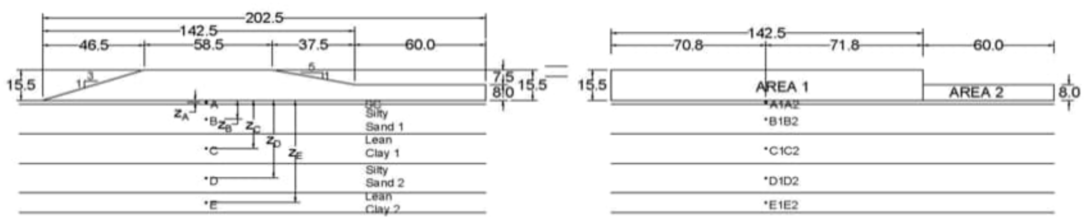
- Poulos, H. and Davis, E. (1974) Elastic Solutions for Soil and Rock Mechanics. John Wiley & Sons, Inc. p. 40.

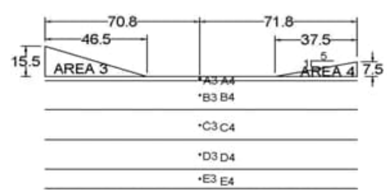
Assumptions:

- Longitudinal dimension (y) of the embankment is much greater than transverse dimension (x). Thus, plane strain applies.

Input and Results:

For consolidation settlement, Depth of Point C=	63.0	ft	Depth below Previous Existing Ground Surface
For consolidation settlement, Depth of Point E=	128.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point A=	1.0	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point B=	24.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point D=	100.0	ft	Depth below Previous Existing Ground Surface
Increase of Vertical Effective Stress due to Embankment at Point A=	**2.015**	ksf	$=\sigma_Z$ at A1 + A2 - A3 - A4
Increase of Vertical Effective Stress due to Embankment at Point B=	**1.933**	ksf	$=\sigma_Z$ at B1 + B2 - B3 - B4
Increase of Vertical Effective Stress due to Embankment at Point C=	**1.565**	ksf	$=\sigma_Z$ at C1 + C2 - C3 - C4
Increase of Vertical Effective Stress due to Embankment at Point D=	**1.262**	ksf	$=\sigma_Z$ at D1 + D2 - D3 - D4
Increase of Vertical Effective Stress due to Embankment at Point E=	**1.087**	ksf	$=\sigma_Z$ at E1 + E2 - E3 - E4





Project No.:	60594417		AECOM Clifton, NJ
Subject :	SSSI Task 3A (H = 15.5 ft S= 5 ft)		
	- Induced Stress under Embankment Load (Center of Levee)		
Computed By: QH	Checked By: LC	Date: 10/21/2019	
Computed By:	Checked By:	Rev:	

Point No. (1)	Input								Results			
	Material Properties			Embankment Geometry			Point of Interest		Stress			Strain
		Subgrade Soil										
	Emkmnt Unit Wt.	Young's Modulus	Poisson's Ratio	(See Figure 1 on Sheet 7)			Coordinates					
	$\gamma_{emb.}$	E	ν	a	b	H	x	z	σ_z	σ_x	σ_y	ε_z
	(kcf)	ksf		(ft)	(ft)	(ft)	(ft)	(ft)	(ksf)	(ksf)	(ksf)	
A1	0.130		0.2	0.00	142.50	15.50	70.75	1.00	2.0	2.0	0.8	
B1	0.130		0.2	0.00	142.50	15.50	70.75	24.50	2.0	1.2	0.6	
D1	0.130		0.2	0.00	142.50	15.50	70.75	100.00	1.4	0.2	0.3	
A2	0.130		0.2	0.00	60.00	8.00	-71.75	1.00	0.0	0.0	0.0	
B2	0.130		0.2	0.00	60.00	8.00	-71.75	24.50	0.0	0.1	0.0	
D2	0.130		0.2	0.00	60.00	8.00	-71.75	100.00	0.1	0.1	0.0	
A3	0.130		0.2	46.50	46.50	15.50	-24.25	1.00	0.0	0.0	0.0	
B3	0.130		0.2	46.50	46.50	15.50	-24.25	24.50	0.0	0.2	0.0	
D3	0.130		0.2	46.50	46.50	15.50	-24.25	100.00	0.2	0.1	0.0	
A4	0.130		0.2	37.50	37.50	7.50	-34.25	1.00	0.0	0.0	0.0	
B4	0.130		0.2	37.50	37.50	7.50	-34.25	24.50	0.0	0.1	0.0	
D4	0.130		0.2	37.50	37.50	7.50	-34.25	100.00	0.1	0.0	0.0	
C1	0.130		0.2	0.00	142.50	15.50	70.75	63.00	1.7	0.4	0.4	
E1	0.130		0.2	0.00	142.50	15.50	70.75	128.50	1.2	0.1	0.3	
C2	0.130		0.2	0.00	60.00	8.00	-71.75	63.00	0.1	0.1	0.0	
E2	0.130		0.2	0.00	60.00	8.00	-71.75	128.50	0.1	0.1	0.0	
C3	0.130		0.2	46.50	46.50	15.50	-24.25	63.00	0.2	0.1	0.1	
E3	0.130		0.2	46.50	46.50	15.50	-24.25	128.50	0.2	0.0	0.0	
C4	0.130		0.2	37.50	37.50	7.50	-34.25	63.00	0.1	0.0	0.0	
E4	0.130		0.2	37.50	37.50	7.50	-34.25	128.50	0.1	0.0	0.0	

Note:
1. Point A1, A2, A3, A4, A5 are the same point at point A BUT for different embankment loads (see previous calculation for model) .

Project No.: 60594417

Subject : SSSI Task 3A (H = 15.5 ft S= 5 ft)

- Induced Stress under Embankment Load (Center of Levee)

AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019

Computed By: Checked By: Rev:

Calculations:

Point No.	Intermediate Results						Results			
	Max. Surcharge	Distance and Angle (See Figure 1 on Sheet 7)					Stress			Strain
	p [1] (ksf)	R_0 [2] (ft)	R_1 [3] (ft)	R_2 [4] (ft)	α [5] (rad.)	β [6] (rad.)	σ_z [7] (ksf)	σ_x [8] (ksf)	σ_y [9] (ksf)	ε_z [10]
A1	2.015	70.8	70.8	71.8	0.00	3.11	2.0	2.0	0.8	
B1	2.015	74.9	74.9	75.8	0.00	2.48	2.0	1.2	0.6	
D1	2.015	122.5	122.5	123.1	0.00	1.24	1.4	0.2	0.3	
A2	1.040	71.8	71.8	131.8	0.00	0.01	0.0	0.0	0.0	
B2	1.040	75.8	75.8	134.0	0.00	0.15	0.0	0.1	0.0	
D2	1.040	123.1	123.1	165.4	0.00	0.30	0.1	0.1	0.0	
A3	2.015	24.3	70.8	70.8	0.03	0.00	0.0	0.0	0.0	
B3	2.015	34.5	74.9	74.9	0.46	0.00	0.0	0.2	0.0	
D3	2.015	102.9	122.5	122.5	0.38	0.00	0.2	0.1	0.0	
A4	0.975	34.3	71.8	71.8	0.02	0.00	0.0	0.0	0.0	
B4	0.975	42.1	75.8	75.8	0.29	0.00	0.0	0.1	0.0	
D4	0.975	105.7	123.1	123.1	0.29	0.00	0.1	0.0	0.0	
C1	2.015	94.7	94.7	95.5	0.00	1.69	1.7	0.4	0.4	
E1	2.015	146.7	146.7	147.2	0.00	1.01	1.2	0.1	0.3	
C2	1.040	95.5	95.5	146.0	0.00	0.27	0.1	0.1	0.0	
E2	1.040	147.2	147.2	184.0	0.00	0.29	0.1	0.1	0.0	
C3	2.015	67.5	94.7	94.7	0.48	0.00	0.2	0.1	0.1	
E3	2.015	130.8	146.7	146.7	0.32	0.00	0.2	0.0	0.0	
C4	0.975	71.7	95.5	95.5	0.35	0.00	0.1	0.0	0.0	
E4	0.975	133.0	147.2	147.2	0.25	0.00	0.1	0.0	0.0	

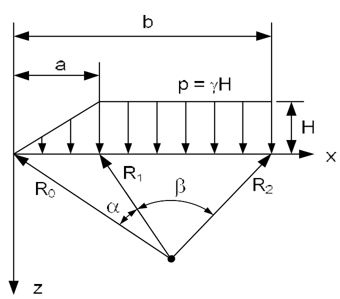


Figure 1

Notes:

1 $p = \gamma_{emb.} * H$

2 $R_0 = \sqrt{x^2 + z^2}$

3 $R_1 = \sqrt{(x-a)^2 + z^2}$

4 $R_2 = \sqrt{(b-x)^2 + z^2}$

5 $\alpha = \operatorname{atan}\left(\frac{x}{z}\right) - \operatorname{atan}\left(\frac{x-a}{z}\right)$

6 $\beta = \operatorname{atan}\left(\frac{x-a}{z}\right) + \operatorname{atan}\left(\frac{b-x}{z}\right)$

7 $\sigma_z = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} - \frac{z}{R_2^2}(x-b)\right]$

8 $\sigma_x = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} + \frac{z}{R_2^2}(x-b) + \frac{2z}{a}\log_e\frac{R_1}{R_0}\right]$

9 $\sigma_y = \upsilon[\sigma_x + \sigma_z]$

Project No.:	60594417
Subject :	SSSI Task 3A (H = 15.5 ft S=5 ft)
	- Induced Stress under Embankment Load (Toe of Levee)
Computed By: QH	Checked By: LC	Date: 10/21/2019

AECOM
Clifton, NJ

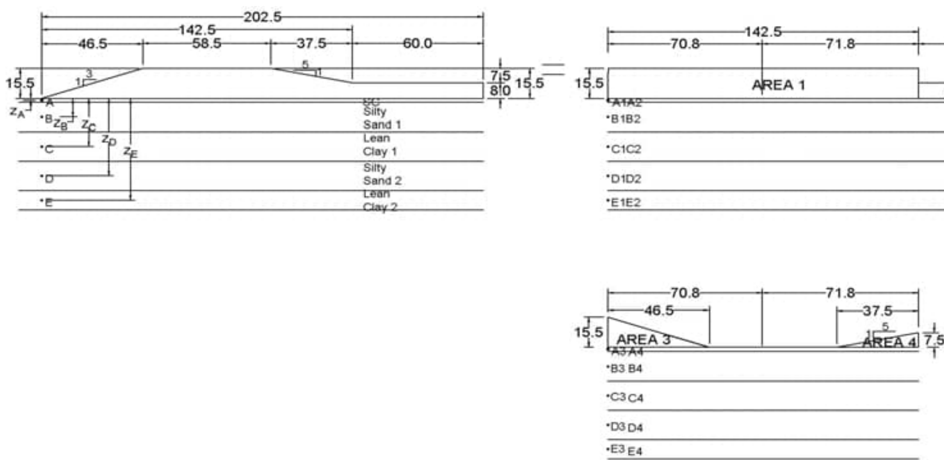
Problem Statement:
Calculate the stress at different depths of layers under permanent embankment load. See Figure 1 on NEXT SHEET for shape of embankment.
Note: The entire embankment is shown in Figure 1. b is the width of embankment. b is NOT half embankment width.

Reference:
- Poulos, H. and Davis, E. (1974) Elastic Solutions for Soil and Rock Mechanics. John Wiley & Sons, Inc. p. 40.

Assumptions:
- Longitudinal dimension (y) of the embankment is much greater than transverse dimension (x). Thus, plane strain applies.

Input and Results:

For consolidation settlement, Depth of Point C=	63.0	ft	Depth below Previous Existing Ground Surface
For consolidation settlement, Depth of Point E=	128.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point A=	1.0	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point B=	24.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point D=	100.0	ft	Depth below Previous Existing Ground Surface
Increase of Vertical Effective Stress due to Embankment at Point A=	**0.013**	ksf	$=\sigma_Z$ at A1 + A2 - A3 - A4
Increase of Vertical Effective Stress due to Embankment at Point B=	**0.309**	ksf	$=\sigma_Z$ at B1 + B2 - B3 - B4
Increase of Vertical Effective Stress due to Embankment at Point C=	**0.572**	ksf	$=\sigma_Z$ at C1 + C2 - C3 - C4
Increase of Vertical Effective Stress due to Embankment at Point D=	**0.648**	ksf	$=\sigma_Z$ at D1 + D2 - D3 - D4
Increase of Vertical Effective Stress due to Embankment at Point E=	**0.652**	ksf	$=\sigma_Z$ at E1 + E2 - E3 - E4



Project No.: 60594417
Subject : SSSI Task 3A (H = 15.5 ft S=5 ft)
- Induced Stress under Embankment Load (Toe of Levee)

AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019

Point No. (1)	Input								Results			
	Material Properties			Embankment Geometry			Point of Interest		Stress			Strain
		Subgrade Soil		(See Figure 1 on Sheet 7)			Coordinates					
	Emkmnt Unit Wt.	Young's Modulus	Poisson's Ratio									
	$\gamma_{emb.}$	E	ν	a	b	H	x	z	σ_z	σ_x	σ_y	ε_z
	(kcf)	ksf		(ft)	(ft)	(ft)	(ft)	(ft)	(ksf)	(ksf)	(ksf)	
A1	0.130		0.2	0.00	142.50	15.50	0.00	1.00	1.0	1.0	0.4	
B1	0.130		0.2	0.00	142.50	15.50	0.00	24.50	1.0	0.8	0.4	
D1	0.130		0.2	0.00	142.50	15.50	0.00	100.00	0.9	0.3	0.2	
A2	0.130		0.2	0.00	60.00	8.00	-142.50	1.00	0.0	0.0	0.0	
B2	0.130		0.2	0.00	60.00	8.00	-142.50	24.50	0.0	0.0	0.0	
D2	0.130		0.2	0.00	60.00	8.00	-142.50	100.00	0.0	0.1	0.0	
A3	0.130		0.2	46.50	46.50	15.50	46.50	1.00	1.0	0.9	0.4	
B3	0.130		0.2	46.50	46.50	15.50	46.50	24.50	0.7	0.2	0.2	
D3	0.130		0.2	46.50	46.50	15.50	46.50	100.00	0.3	0.0	0.1	
A4	0.130		0.2	37.50	37.50	7.50	-105.00	1.00	0.0	0.0	0.0	
B4	0.130		0.2	37.50	37.50	7.50	-105.00	24.50	0.0	0.0	0.0	
D4	0.130		0.2	37.50	37.50	7.50	-105.00	100.00	0.0	0.0	0.0	
C1	0.130		0.2	0.00	142.50	15.50	0.00	63.00	1.0	0.5	0.3	
E1	0.130		0.2	0.00	142.50	15.50	0.00	128.50	0.9	0.2	0.2	
C2	0.130		0.2	0.00	60.00	8.00	-142.50	63.00	0.0	0.1	0.0	
E2	0.130		0.2	0.00	60.00	8.00	-142.50	128.50	0.0	0.1	0.0	
C3	0.130		0.2	46.50	46.50	15.50	46.50	63.00	0.4	0.0	0.1	
E3	0.130		0.2	46.50	46.50	15.50	46.50	128.50	0.2	0.0	0.0	
C4	0.130		0.2	37.50	37.50	7.50	-105.00	63.00	0.0	0.0	0.0	
E4	0.130		0.2	37.50	37.50	7.50	-105.00	128.50	0.0	0.0	0.0	

Note:
1. Point A1, A2, A3, A4, A5 are the same point at point A BUT for different embankment loads (see previous calculation for model) .

Project No.: 60594417
Subject : SSSI Task 3A (H = 15.5 ft S=5 ft)
- Induced Stress under Embankment Load (Toe of Levee)

AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019

Calculations:

Point No.	Intermediate Results						Results			
	Max. Surcharge Pressure	Distance and Angle (See Figure 1 on Sheet 7)					Stress			Strain
	$p^{(1)}$ (ksf)	$R_0^{(2)}$ (ft)	$R_1^{(3)}$ (ft)	$R_2^{(4)}$ (ft)	$\alpha^{(5)}$ (rad.)	$\beta^{(6)}$ (rad.)	$\sigma_z^{(7)}$ (ksf)	$\sigma_x^{(8)}$ (ksf)	$\sigma_y^{(9)}$ (ksf)	$\varepsilon_z^{(10)}$
A1	2.015	1.0	1.0	142.5	0.00	1.56	1.0	1.0	0.4	
B1	2.015	24.5	24.5	144.6	0.00	1.40	1.0	0.8	0.4	
D1	2.015	100.0	100.0	174.1	0.00	0.96	0.9	0.3	0.2	
A2	1.040	142.5	142.5	202.5	0.00	0.00	0.0	0.0	0.0	
B2	1.040	144.6	144.6	204.0	0.00	0.05	0.0	0.0	0.0	
D2	1.040	174.1	174.1	225.8	0.00	0.15	0.0	0.1	0.0	
A3	2.015	46.5	1.0	1.0	1.55	0.00	1.0	0.9	0.4	
B3	2.015	52.6	24.5	24.5	1.09	0.00	0.7	0.2	0.2	
D3	2.015	110.3	100.0	100.0	0.44	0.00	0.3	0.0	0.1	
A4	0.975	105.0	142.5	142.5	0.00	0.00	0.0	0.0	0.0	
B4	0.975	107.8	144.6	144.6	0.06	0.00	0.0	0.0	0.0	
D4	0.975	145.0	174.1	174.1	0.15	0.00	0.0	0.0	0.0	
C1	2.015	63.0	63.0	155.8	0.00	1.15	1.0	0.5	0.3	
E1	2.015	128.5	128.5	191.9	0.00	0.84	0.9	0.2	0.2	
C2	1.040	155.8	155.8	212.1	0.00	0.11	0.0	0.1	0.0	
E2	1.040	191.9	191.9	239.8	0.00	0.17	0.0	0.1	0.0	
C3	2.015	78.3	63.0	63.0	0.64	0.00	0.4	0.0	0.1	
E3	2.015	136.7	128.5	128.5	0.35	0.00	0.2	0.0	0.0	
C4	0.975	122.4	155.8	155.8	0.12	0.00	0.0	0.0	0.0	
E4	0.975	165.9	191.9	191.9	0.15	0.00	0.0	0.0	0.0	

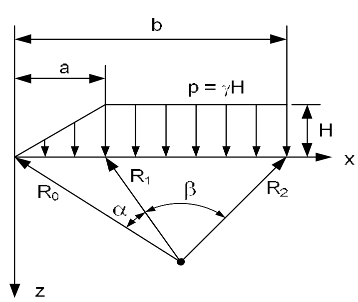


Figure 1

Notes:

1 $p = \gamma_{emb.} * H$

2 $R_0 = \sqrt{x^2 + z^2}$

3 $R_1 = \sqrt{(x-a)^2 + z^2}$

4 $R_2 = \sqrt{(b-x)^2 + z^2}$

5 $\alpha = \operatorname{atan}\left(\frac{x}{z}\right) - \operatorname{atan}\left(\frac{x-a}{z}\right)$

6 $\beta = \operatorname{atan}\left(\frac{x-a}{z}\right) + \operatorname{atan}\left(\frac{b-x}{z}\right)$

7 $\sigma_z = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} - \frac{z}{R_2^2}(x-b)\right]$

8 $\sigma_x = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} + \frac{z}{R_2^2}(x-b) + \frac{2z}{a}\log_e\frac{R_1}{R_0}\right]$

9 $\sigma_y = \upsilon[\sigma_x + \sigma_z]$

Project No.:	60594417				
Subject :	SSSI Task Order 3A (H = 15.5 ft, S=7 ft)				
	- Settlement Calculation Summary				
Computed By:	QH	Checked By:	LC	Date:	10/21/2019
Computed By:		Checked By:		Rev:	

AECOM
Clifton, NJ

Problem Statement:

Summary of the settlement calculation results at Station 29+00.

Reference:

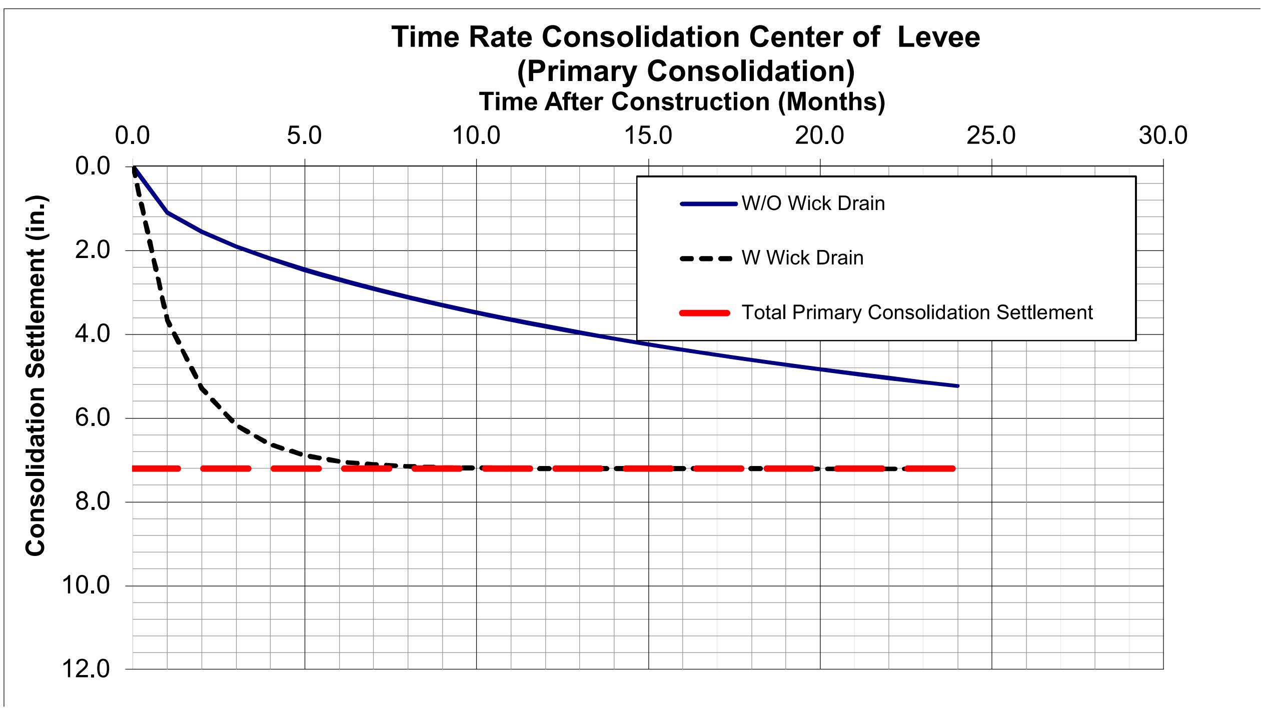
1. Das, B. M., *Principles of Foundation Engineering, Nelson, Ontario, Canada, 750 p 7th Edition, Dated 2007*
2. Naval Facilities Engineering Command (NAFAC), *Soil Mechanics Design Manual 7.01* , Dated September 1986. (DM 7.01)
3. AASHTO, *LRFD Bridge Specification 7th Edition* , 2014
4. Poulos, H. and Davis, E. *Elastic Solutions for Soil and Rock Mechanics* . John Wiley & Sons, Inc. (1974)

Summary

- Station: 29+00
- Embankment Height: 15.5 ft
- Embankment Slope at land side: 3 H: 1V
- Boring used in the model: DH-10
- Settlement Summary

	Item	Unit	Center of Levee	Toe of Levee
4.	Primary Consolidation Settlement	(inch)	**7.2**	**3.1**

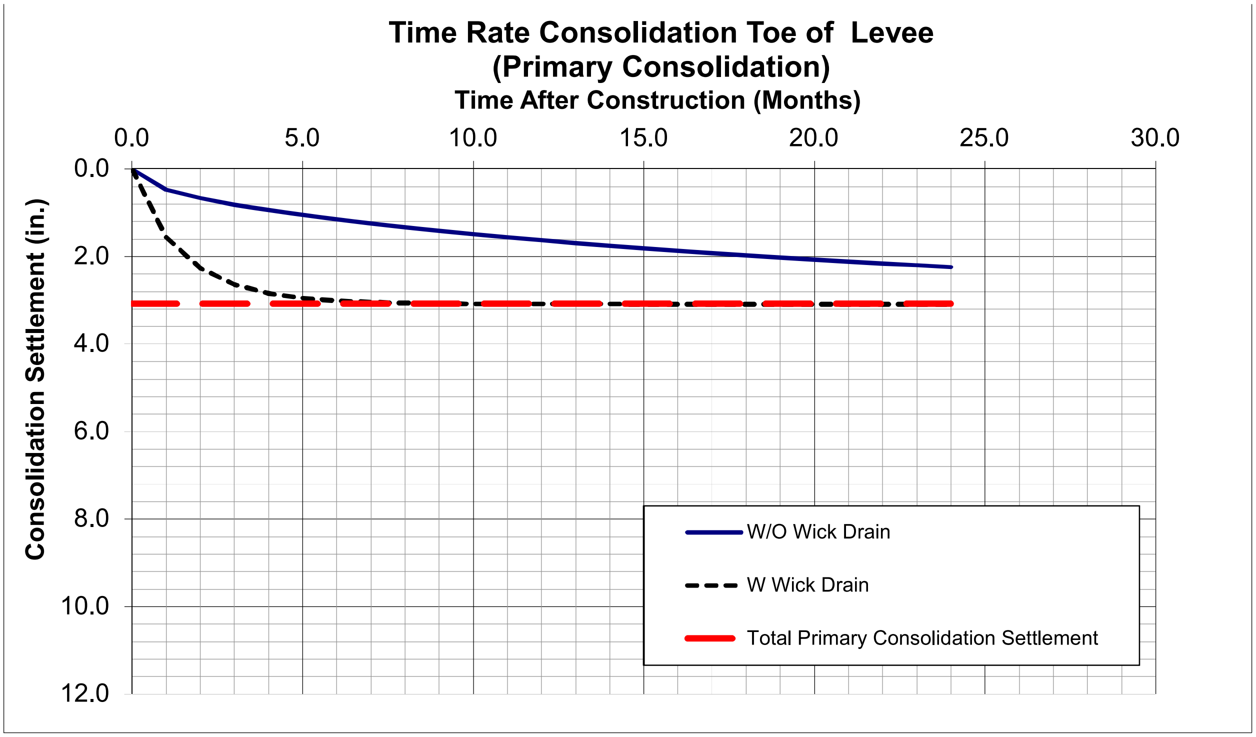
- Drain Pattern	Triangular		Assumed
Drain Spacing, S =	7.00	feet	Assumed
Wick Drain Width =	4	inch	Assumed
Wick Drain Thickness =	0.13	inch	Assumed



Project No.:	60594417
Subject :	SSSI Task Order 3A (H = 15.5 ft, S=7 ft)
	- Settlement Calculation Summary

AECOM
Clifton, NJ

Computed By:	QH	Checked By:	LC	Date:	10/21/2019
Computed By:		Checked By:		Rev:	



Parameters:

- Boring DH-10 was used in the analysis.
- Equivalent unit of weight of embankment is 130 pcf
- The following parameters were used in the calculation:

Layer No.	Soil Description	Total Unit Weight (pcf)	Layer Thickness (ft)
1	Clayey Sand 1	120	2
2	Silty Sand 1	125	45
3	Lean clay 1	120	32
4	Silty Sand 2	130	42
5	Lean clay 2 (1)	120	15

Note:

1. Thickness of this layer is based on termination of boring.

Clay Layer	Compression Index	Secondary Compression Ratio	Void Ratio	Coef. Of Consolidation
	C_c	C_α'	e_0	C_v
	(---)	(---)	(---)	(ft^2/Day)
1	0.212	0.0015	0.785	0.337
2	0.150	0.0011	0.751	0.337

Project No.:	60594417	AECOM Clifton, NJ
Subject :	SSSI Task Order 3A (H = 15.5 ft, S=7 ft) - Consolidation Settlement	

Computed By:	QH	Checked By:	LC	Date:	10/21/2019
Computed by:		Checked By:		Rev:	

Problem Statement:

Estimate the secondary consolidation settlement .

Reference:

1. Das, B. M., *Principles of Foundation Engineering, Nelson, Ontario, Canada, 750 p 7th Edition, Dated 2007*
2. Naval Facilities Engineering Command (NAFAC), *Soil Mechanics Design Manual 7.01* , Dated September 1986. (DM 7.01)

Assumptions:

- The following subsurface profile was used in the calculation

Layer No.	Soil Description	Total Unit Weight (pcf)	Layer Thickness (ft)
1	Clayey Sand 1	120	2
2	Silty Sand 1	125	45
3	Lean clay 1	120	32
4	Silty Sand 2	130	42
5	Lean clay 2	120	15

- Current Ground Surface El.	2.2	ft
Groundwater Table El.	-3.5	ft
Ground Water Table from Ground Surface	5.7	ft

- The following EXCEPT C_α' were adopted from Laboratory Results:

Clay Layer	Compression Index	Secondary Compression Index	Void Ratio	Coef. Of Consolidation
	C_c	C_α'	e_0	C_v
	(---)	(---)	(---)	(ft^2/Day)
1	0.212	0.0015	0.785	0.337
2	0.150	0.0011	0.751	0.337

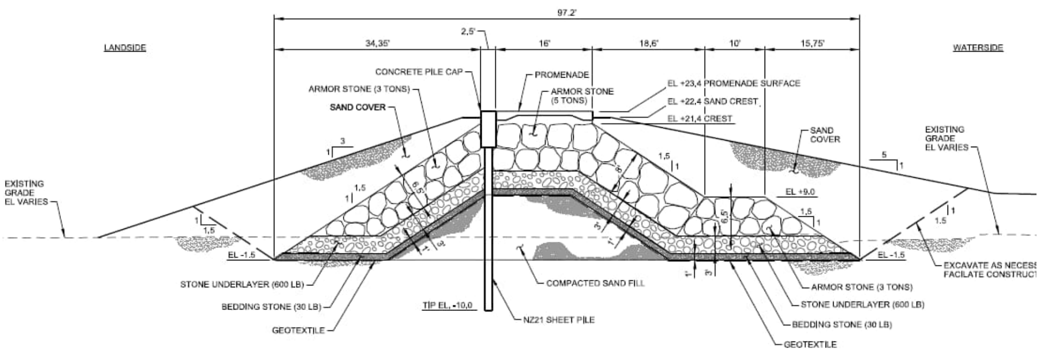
- Based on laboratory results, both clays are normally consolidated

Project No.:	60594417	AECOM Clifton, NJ
Subject :	SSSI Task Order 3A (H = 15.5 ft, S=7 ft)	
	- Consolidation Settlement	
Computed By: QH	Checked By: LC	Date: 10/21/2019
Computed by:	Checked By:	Rev:

Calculations:

1. *Embankment Loading*

- The proposed embankment is shown below:



2. *Consolidation Settlement*

Total Thickness of Clay used for primary consolidation, H_{clay} = **47** ft

Unit Weight of Sea Water, γ_w = 64 pcf

2a. Primary Settlement under A. Final Condition and B. permanent load and surcharge load

Clay Layer	From Ground Surface to Mid Point of Layer	Layer Thickness	Initial Effective Vertical Stress	Increase of Effective Vertical Stress At Final Condition (Center of Levee)	Increase of Effective Vertical Stress At Final Condition (Toe of Levee)	Final Effective Vertical Stress At Final Condition (Center of Levee)	Final Effective Vertical Stress At Final Condition (Toe of Levee)	Compression Index	Void Ratio	Primary Consolidation Settlement at Final Condition (Center of Levee)		Primary Consolidation Settlement at Final Condition (Toe of Levee)	
	z	H	σ_{v0}'	p_{fc}	p_{fT}	σ_{fC}'	σ_{fT}'	C_c	e_0	δ_f		δ_f	
	(ft)	(ft)	(psf)	(psf)	(psf)	(psf)	(psf)	(---)	(---)	(ft)	(inch)	(ft)	(inch)
	(1)	(2)	(3)	(4)	(4)	(6)	(6)	(8)	(9)	(10)		(10)	
1	63	32	4119.1	1564.9	572.4	5684.0	4691.5	0.212	0.785	0.53	6.4	0.215	2.6
2	128.5	15	8207.1	1086.9	651.7	9294.0	8858.7	0.15	0.751	0.07	0.8	0.043	0.5
										0.6	**7.2**	**0.3**	**3.1**

Note:

3. For Clay Layer 1, $\sigma_{v0}' = \gamma 1\, H1 + \gamma 2 * (Hw - H1) + (\gamma 2 - Yw) * (H1 + H2 - Hw) + (\gamma 3 - Yw) * H3 / 2$

   For Clay Layer 2, $\sigma_{v0}' = \gamma 1\, H1 + \gamma 2 * (Hw - H1) + (\gamma 2 - Yw) * (H1 + H2 - Hw) + (\gamma 3 - Yw) * H3 + (\gamma 4 - Yw) * H4 + (\gamma 5 - Yw) * H5 /2$

4. p_f see attached calculation

6. $\sigma_f' = \sigma_{v0}' + p_f$

10. $\delta_f = C_c * H/ (1+e_0) * \text{Log} (\sigma_f '/ \sigma_{v0}')$ Ref. 1 Eq. 5.81

Project No.:	60594417		AECOM Clifton, NJ
Subject :	SSSI Task Order 3A (H = 15.5 ft, S=7 ft) - Consolidation Settlement		
Computed By: QH	Checked By: LC	Date: 10/21/2019	
Computed by:	Checked By:	Rev:	

Radial drainage (wick drains)

d_e = effective zone from which the radial drainage will be directed toward the wick drain

r_w = equivalent diameter of wick drain radius

a =width of wick drain= 4 in

b =thickness of wick drain= 0.13 in

T_r =Time factor with radial drainage

$C_v = C_{vr}$ = Coefficient of Consolidation

*No soil smear assumed

Drain Spacing, S = 7.00 feet

Placement Pattern Triangular

d_e = 7.35 feet = 1.05 * S for Triangular Pattern, = 1.13 * S for Square Pattern

Drainage Path Double

2b. Degree of Consolidation with wick drain

Below is the calculation show the degree of consolidation, primary consolidation settlement versus time after construction.

t							Double Drainage						Center of Levee		Toe of Levee	
		C_v	d_e	r_w	n	m	T_r	U_r	T_v	H_{dr}	U_v	$U_{v,r}$	$S_{iv}c$	$S_{iv,r}c$	S_{ivT}	$S_{iv,rT}$
(Months)	(Days)	(ft²/Day)	(ft)	(ft)	-	-	-	-	-	ft	-	-	(in.)	(in.)	(in.)	(in.)
(1)		(2)	(3)	(4)	(5)	(6)	(6)	(8)	(10)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
0.0	0	0.3372	7.35	0.11	33.52	2.77	0.00	0.00	0.000	23.5	0.000	0.000	0.0	0.0	0.0	0.0
1.0	30	0.3372	7.35	0.11	33.52	2.77	0.19	0.42	0.018	23.5	0.153	0.507	1.1	3.7	0.5	1.6
2.0	60	0.3372	7.35	0.11	33.52	2.77	0.37	0.66	0.037	23.5	0.216	0.735	1.6	5.3	0.7	2.3
3.0	90	0.3372	7.35	0.11	33.52	2.77	0.56	0.80	0.055	23.5	0.265	0.855	1.9	6.2	0.8	2.6
4.0	120	0.3372	7.35	0.11	33.52	2.77	0.75	0.89	0.073	23.5	0.305	0.920	2.2	6.6	0.9	2.8
5.0	150	0.3372	7.35	0.11	33.52	2.77	0.94	0.93	0.092	23.5	0.341	0.956	2.5	6.9	1.1	3.0
6.0	180	0.3372	7.35	0.11	33.52	2.77	1.12	0.96	0.110	23.5	0.374	0.976	2.7	7.0	1.2	3.0
7.0	210	0.3372	7.35	0.11	33.52	2.77	1.31	0.98	0.128	23.5	0.404	0.987	2.9	7.1	1.2	3.0
8.0	240	0.3372	7.35	0.11	33.52	2.77	1.50	0.99	0.147	23.5	0.432	0.993	3.1	7.2	1.3	3.1
9.0	270	0.3372	7.35	0.11	33.52	2.77	1.69	0.99	0.165	23.5	0.458	0.996	3.3	7.2	1.4	3.1
10.0	300	0.3372	7.35	0.11	33.52	2.77	1.87	1.00	0.183	23.5	0.483	0.998	3.5	7.2	1.5	3.1
11.0	330	0.3372	7.35	0.11	33.52	2.77	2.06	1.00	0.201	23.5	0.507	0.999	3.7	7.2	1.6	3.1
12.0	360	0.3372	7.35	0.11	33.52	2.77	2.25	1.00	0.220	23.5	0.529	0.999	3.8	7.2	1.6	3.1
13.0	390	0.3372	7.35	0.11	33.52	2.77	2.43	1.00	0.238	23.5	0.550	1.000	4.0	7.2	1.7	3.1
14.0	420	0.3372	7.35	0.11	33.52	2.77	2.62	1.00	0.256	23.5	0.569	1.000	4.1	7.2	1.8	3.1
15.0	450	0.3372	7.35	0.11	33.52	2.77	2.81	1.00	0.275	23.5	0.588	1.000	4.2	7.2	1.8	3.1
16.0	480	0.3372	7.35	0.11	33.52	2.77	3.00	1.00	0.293	23.5	0.607	1.000	4.4	7.2	1.9	3.1
17.0	510	0.3372	7.35	0.11	33.52	2.77	3.18	1.00	0.311	23.5	0.624	1.000	4.5	7.2	1.9	3.1
18.0	540	0.3372	7.35	0.11	33.52	2.77	3.37	1.00	0.330	23.5	0.641	1.000	4.6	7.2	2.0	3.1
19.0	570	0.3372	7.35	0.11	33.52	2.77	3.56	1.00	0.348	23.5	0.657	1.000	4.7	7.2	2.0	3.1
20.0	600	0.3372	7.35	0.11	33.52	2.77	3.75	1.00	0.366	23.5	0.672	1.000	4.8	7.2	2.1	3.1
21.0	630	0.3372	7.35	0.11	33.52	2.77	3.93	1.00	0.385	23.5	0.686	1.000	4.9	7.2	2.1	3.1
22.0	660	0.3372	7.35	0.11	33.52	2.77	4.12	1.00	0.403	23.5	0.700	1.000	5.0	7.2	2.2	3.1
23.0	690	0.3372	7.35	0.11	33.52	2.77	4.31	1.00	0.421	23.5	0.713	1.000	5.1	7.2	2.2	3.1
24.0	720	0.3372	7.35	0.11	33.52	2.77	4.49	1.00	0.440	23.5	0.726	1.000	5.2	7.2	2.2	3.1

Note:

4. $r_w = 2\ (a + b) / \pi / 2$ Ref. 1 Eq. 14.33
5. $n = d_e / 2\ r_w$ Ref. 1 Eq. 14.21
6. $m = \left(\frac{n^2}{n^2-1}\right)\ln(n) - \frac{3n^2-1}{4n^2}$ Ref. 1 Eq. 14.25 for non-smear case.
8. $U_v = (4 * T_v / \pi)^{0.5}$ U_v in Decimal If T_v <= 0.283 Ref. 1 Eq. (1.74)
   U_v = 1- 10^ $[(T_v + 0.085)/(-0.933)]$ If T_v > 0.283 Ref. 1 Eq. (1.75)
9. $U_{v,r}$ = 1- (1 - U_v)(1- U_r)
10. $\delta_{iv} = \delta_f * U_v$
11. $\delta_{iv,r} = \delta_f * U_{v,r}$

Project No.:	60594417				
Subject :	SSSI Task 3A (H = 15.5 ft, S = 7 ft)				
	- Induced Stress under Embankment Load (Center of Levee)				
Computed By:	QH	Checked By:	LC	Date:	10/21/2019
Computed By:		Checked By:		Rev:	

AECOM
Clifton, NJ

Problem Statement:

Calculate the stress at different depths of layers under permanent embankment load. See Figure 1 on NEXT SHEET for shape of embankment.

Note: The entire embankment is shown in Figure 1. b is the width of embankment. b is NOT half embankment width.

Reference:

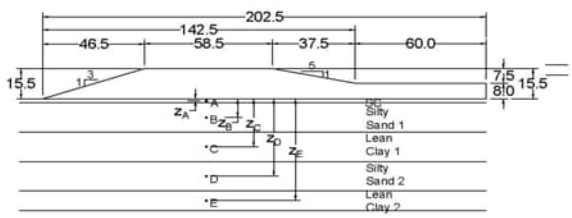
- Poulos, H. and Davis, E. (1974) Elastic Solutions for Soil and Rock Mechanics. John Wiley & Sons, Inc. p. 40.

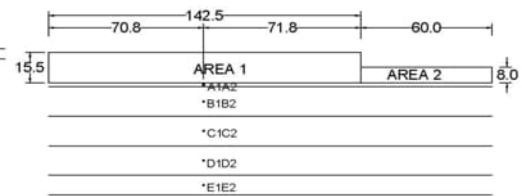
Assumptions:

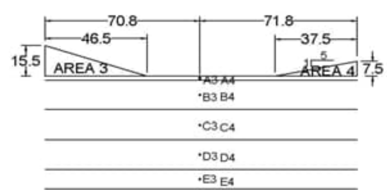
- Longitudinal dimension (y) of the embankment is much greater than transverse dimension (x). Thus, plane strain applies.

Input and Results:

For consolidation settlement, Depth of Point C=	63.0	ft	Depth below Previous Existing Ground Surface
For consolidation settlement, Depth of Point E=	128.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point A=	1.0	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point B=	24.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point D=	100.0	ft	Depth below Previous Existing Ground Surface
Increase of Vertical Effective Stress due to Embankment at Point A=	**2.015**	ksf	$=\sigma_Z$ at A1 + A2 - A3 - A4
Increase of Vertical Effective Stress due to Embankment at Point B=	**1.933**	ksf	$=\sigma_Z$ at B1 + B2 - B3 - B4
Increase of Vertical Effective Stress due to Embankment at Point C=	**1.565**	ksf	$=\sigma_Z$ at C1 + C2 - C3 - C4
Increase of Vertical Effective Stress due to Embankment at Point D=	**1.262**	ksf	$=\sigma_Z$ at D1 + D2 - D3 - D4
Increase of Vertical Effective Stress due to Embankment at Point E=	**1.087**	ksf	$=\sigma_Z$ at E1 + E2 - E3 - E4







Project No.: 60594417
Subject : SSSI Task 3A (H = 15.5 ft, S = 7 ft)
- Induced Stress under Embankment Load (Center of Levee)

AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019
Computed By: Checked By: Rev:

Point No. (1)	Input								Results			
	Material Properties			Embankment Geometry			Point of Interest		Stress			Strain
		Subgrade Soil		(See Figure 1 on Sheet 7)			Coordinates					
	Emkmnt Unit Wt.	Young's Modulus	Poisson's Ratio									
	$\gamma_{emb.}$	E	ν	a	b	H	x	z	σ_z	σ_x	σ_y	ε_z
	(kcf)	ksf		(ft)	(ft)	(ft)	(ft)	(ft)	(ksf)	(ksf)	(ksf)	
A1	0.130		0.2	0.00	142.50	15.50	70.75	1.00	2.0	2.0	0.8	
B1	0.130		0.2	0.00	142.50	15.50	70.75	24.50	2.0	1.2	0.6	
D1	0.130		0.2	0.00	142.50	15.50	70.75	100.00	1.4	0.2	0.3	
A2	0.130		0.2	0.00	60.00	8.00	-71.75	1.00	0.0	0.0	0.0	
B2	0.130		0.2	0.00	60.00	8.00	-71.75	24.50	0.0	0.1	0.0	
D2	0.130		0.2	0.00	60.00	8.00	-71.75	100.00	0.1	0.1	0.0	
A3	0.130		0.2	46.50	46.50	15.50	-24.25	1.00	0.0	0.0	0.0	
B3	0.130		0.2	46.50	46.50	15.50	-24.25	24.50	0.0	0.2	0.0	
D3	0.130		0.2	46.50	46.50	15.50	-24.25	100.00	0.2	0.1	0.0	
A4	0.130		0.2	37.50	37.50	7.50	-34.25	1.00	0.0	0.0	0.0	
B4	0.130		0.2	37.50	37.50	7.50	-34.25	24.50	0.0	0.1	0.0	
D4	0.130		0.2	37.50	37.50	7.50	-34.25	100.00	0.1	0.0	0.0	
C1	0.130		0.2	0.00	142.50	15.50	70.75	63.00	1.7	0.4	0.4	
E1	0.130		0.2	0.00	142.50	15.50	70.75	128.50	1.2	0.1	0.3	
C2	0.130		0.2	0.00	60.00	8.00	-71.75	63.00	0.1	0.1	0.0	
E2	0.130		0.2	0.00	60.00	8.00	-71.75	128.50	0.1	0.1	0.0	
C3	0.130		0.2	46.50	46.50	15.50	-24.25	63.00	0.2	0.1	0.1	
E3	0.130		0.2	46.50	46.50	15.50	-24.25	128.50	0.2	0.0	0.0	
C4	0.130		0.2	37.50	37.50	7.50	-34.25	63.00	0.1	0.0	0.0	
E4	0.130		0.2	37.50	37.50	7.50	-34.25	128.50	0.1	0.0	0.0	

Note:
1. Point A1, A2, A3, A4, A5 are the same point at point A BUT for different embankment loads (see previous calculation for model) .

Project No.:	60594417
Subject :	SSSI Task 3A (H = 15.5 ft, S = 7 ft)
	- Induced Stress under Embankment Load (Center of Levee)

AECOM
Clifton, NJ

Computed By:	QH	Checked By:	LC	Date:	10/21/2019
Computed By:		Checked By:		Rev:	

Calculations:

Point No.	Intermediate Results						Results			
	Max. Surcharge	Distance and Angle (See Figure 1 on Sheet 7)					Stress			Strain
	$p^{(1)}$ (ksf)	$R_0^{(2)}$ (ft)	$R_1^{(3)}$ (ft)	$R_2^{(4)}$ (ft)	$\alpha^{(5)}$ (rad.)	$\beta^{(6)}$ (rad.)	$\sigma_z^{(7)}$ (ksf)	$\sigma_x^{(8)}$ (ksf)	$\sigma_y^{(9)}$ (ksf)	$\varepsilon_z^{(10)}$
A1	2.015	70.8	70.8	71.8	0.00	3.11	2.0	2.0	0.8	
B1	2.015	74.9	74.9	75.8	0.00	2.48	2.0	1.2	0.6	
D1	2.015	122.5	122.5	123.1	0.00	1.24	1.4	0.2	0.3	
A2	1.040	71.8	71.8	131.8	0.00	0.01	0.0	0.0	0.0	
B2	1.040	75.8	75.8	134.0	0.00	0.15	0.0	0.1	0.0	
D2	1.040	123.1	123.1	165.4	0.00	0.30	0.1	0.1	0.0	
A3	2.015	24.3	70.8	70.8	0.03	0.00	0.0	0.0	0.0	
B3	2.015	34.5	74.9	74.9	0.46	0.00	0.0	0.2	0.0	
D3	2.015	102.9	122.5	122.5	0.38	0.00	0.2	0.1	0.0	
A4	0.975	34.3	71.8	71.8	0.02	0.00	0.0	0.0	0.0	
B4	0.975	42.1	75.8	75.8	0.29	0.00	0.0	0.1	0.0	
D4	0.975	105.7	123.1	123.1	0.29	0.00	0.1	0.0	0.0	
C1	2.015	94.7	94.7	95.5	0.00	1.69	1.7	0.4	0.4	
E1	2.015	146.7	146.7	147.2	0.00	1.01	1.2	0.1	0.3	
C2	1.040	95.5	95.5	146.0	0.00	0.27	0.1	0.1	0.0	
E2	1.040	147.2	147.2	184.0	0.00	0.29	0.1	0.1	0.0	
C3	2.015	67.5	94.7	94.7	0.48	0.00	0.2	0.1	0.1	
E3	2.015	130.8	146.7	146.7	0.32	0.00	0.2	0.0	0.0	
C4	0.975	71.7	95.5	95.5	0.35	0.00	0.1	0.0	0.0	
E4	0.975	133.0	147.2	147.2	0.25	0.00	0.1	0.0	0.0	

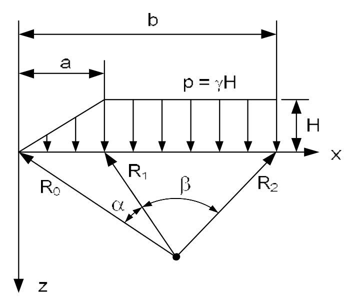


Figure 1

Notes:

1 $p = \gamma_{emb.} * H$

2 $R_0 = \sqrt{x^2 + z^2}$

3 $R_1 = \sqrt{(x-a)^2 + z^2}$

4 $R_2 = \sqrt{(b-x)^2 + z^2}$

5 $\alpha = \operatorname{atan}\left(\frac{x}{z}\right) - \operatorname{atan}\left(\frac{x-a}{z}\right)$

6 $\beta = \operatorname{atan}\left(\frac{x-a}{z}\right) + \operatorname{atan}\left(\frac{b-x}{z}\right)$

7 $\sigma_z = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} - \frac{z}{R_2^2}(x-b)\right]$

8 $\sigma_x = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} + \frac{z}{R_2^2}(x-b) + \frac{2z}{a}\log_e\frac{R_1}{R_0}\right]$

9 $\sigma_y = \upsilon[\sigma_x + \sigma_z]$

Project No.:	60594417
Subject :	SSSI Task 3A (H = 15.5 ft S= 7 ft)
	- Induced Stress under Embankment Load (Toe of Levee)

AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019

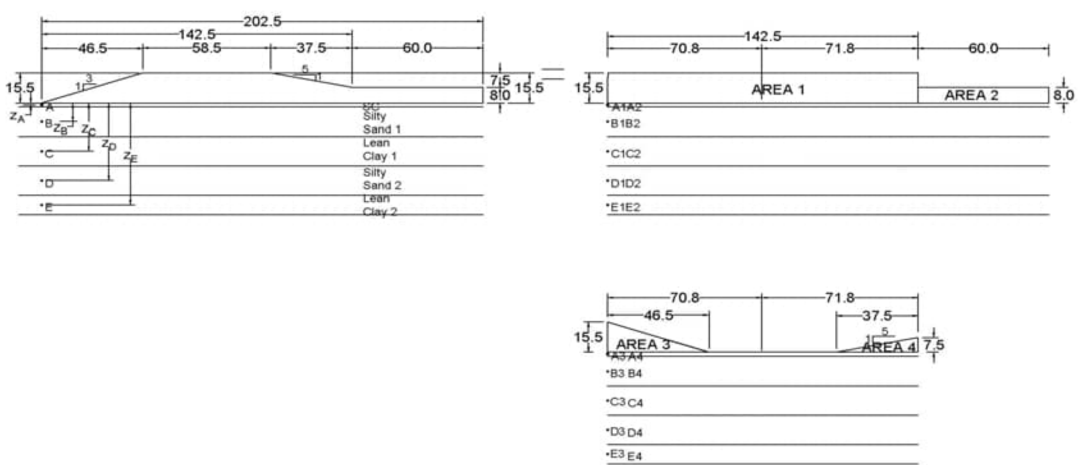
Problem Statement:
Calculate the stress at different depths of layers under permanent embankment load. See Figure 1 on NEXT SHEET for shape of embankment.
Note: The entire embankment is shown in Figure 1. b is the width of embankment. b is NOT half embankment width.

Reference:
- Poulos, H. and Davis, E. (1974) Elastic Solutions for Soil and Rock Mechanics. John Wiley & Sons, Inc. p. 40.

Assumptions:
- Longitudinal dimension (y) of the embankment is much greater than transverse dimension (x). Thus, plane strain applies.

Input and Results:

For consolidation settlement, Depth of Point C=	63.0	ft	Depth below Previous Existing Ground Surface
For consolidation settlement, Depth of Point E=	128.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point A=	1.0	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point B=	24.5	ft	Depth below Previous Existing Ground Surface
For elastic settlement, Depth of Point D=	100.0	ft	Depth below Previous Existing Ground Surface
Increase of Vertical Effective Stress due to Embankment at Point A=	**0.013**	ksf	$=\sigma_Z$ at A1 + A2 - A3 - A4
Increase of Vertical Effective Stress due to Embankment at Point B=	**0.309**	ksf	$=\sigma_Z$ at B1 + B2 - B3 - B4
Increase of Vertical Effective Stress due to Embankment at Point C=	**0.572**	ksf	$=\sigma_Z$ at C1 + C2 - C3 - C4
Increase of Vertical Effective Stress due to Embankment at Point D=	**0.648**	ksf	$=\sigma_Z$ at D1 + D2 - D3 - D4
Increase of Vertical Effective Stress due to Embankment at Point E=	**0.652**	ksf	$=\sigma_Z$ at E1 + E2 - E3 - E4



Project No.: 60594417
Subject : SSSI Task 3A (H = 15.5 ft S= 7 ft)
- Induced Stress under Embankment Load (Toe of Levee)

AECOM
Clifton, NJ

Computed By: QH Checked By: LC Date: 10/21/2019

Point No. (1)	Input								Results			
	Material Properties			Embankment Geometry			Point of Interest		Stress			Strain
		Subgrade Soil		(See Figure 1 on Sheet 7)			Coordinates					
	Emkmnt Unit Wt.	Young's Modulus	Poisson's Ratio									
	$\gamma_{emb.}$	E	ν	a	b	H	x	z	σ_z	σ_x	σ_y	ε_z
	(kcf)	ksf		(ft)	(ft)	(ft)	(ft)	(ft)	(ksf)	(ksf)	(ksf)	
A1	0.130		0.2	0.00	142.50	15.50	0.00	1.00	1.0	1.0	0.4	
B1	0.130		0.2	0.00	142.50	15.50	0.00	24.50	1.0	0.8	0.4	
D1	0.130		0.2	0.00	142.50	15.50	0.00	100.00	0.9	0.3	0.2	
A2	0.130		0.2	0.00	60.00	8.00	-142.50	1.00	0.0	0.0	0.0	
B2	0.130		0.2	0.00	60.00	8.00	-142.50	24.50	0.0	0.0	0.0	
D2	0.130		0.2	0.00	60.00	8.00	-142.50	100.00	0.0	0.1	0.0	
A3	0.130		0.2	46.50	46.50	15.50	46.50	1.00	1.0	0.9	0.4	
B3	0.130		0.2	46.50	46.50	15.50	46.50	24.50	0.7	0.2	0.2	
D3	0.130		0.2	46.50	46.50	15.50	46.50	100.00	0.3	0.0	0.1	
A4	0.130		0.2	37.50	37.50	7.50	-105.00	1.00	0.0	0.0	0.0	
B4	0.130		0.2	37.50	37.50	7.50	-105.00	24.50	0.0	0.0	0.0	
D4	0.130		0.2	37.50	37.50	7.50	-105.00	100.00	0.0	0.0	0.0	
C1	0.130		0.2	0.00	142.50	15.50	0.00	63.00	1.0	0.5	0.3	
E1	0.130		0.2	0.00	142.50	15.50	0.00	128.50	0.9	0.2	0.2	
C2	0.130		0.2	0.00	60.00	8.00	-142.50	63.00	0.0	0.1	0.0	
E2	0.130		0.2	0.00	60.00	8.00	-142.50	128.50	0.0	0.1	0.0	
C3	0.130		0.2	46.50	46.50	15.50	46.50	63.00	0.4	0.0	0.1	
E3	0.130		0.2	46.50	46.50	15.50	46.50	128.50	0.2	0.0	0.0	
C4	0.130		0.2	37.50	37.50	7.50	-105.00	63.00	0.0	0.0	0.0	
E4	0.130		0.2	37.50	37.50	7.50	-105.00	128.50	0.0	0.0	0.0	

Note:
1. Point A1, A2, A3, A4, A5 are the same point at point A BUT for different embankment loads (see previous calculation for model) .

Project No.:	60594417	**AECOM**
Subject :	SSSI Task 3A (H = 15.5 ft S= 7 ft)	**Clifton, NJ**
	- Induced Stress under Embankment Load (Toe of Levee)	

Computed By: QH   Checked By: LC   Date: 10/21/2019

Calculations:

Point No.	Intermediate Results						Results			
	Max. Surcharge Pressure	Distance and Angle (See Figure 1 on Sheet 7)					Stress			Strain
	p (1) (ksf)	R_0 (2) (ft)	R_1 (3) (ft)	R_2 (4) (ft)	α (5) (rad.)	β (6) (rad.)	σ_z (7) (ksf)	σ_x (8) (ksf)	σ_y (9) (ksf)	ε_z (10)
A1	2.015	1.0	1.0	142.5	0.00	1.56	1.0	1.0	0.4	
B1	2.015	24.5	24.5	144.6	0.00	1.40	1.0	0.8	0.4	
D1	2.015	100.0	100.0	174.1	0.00	0.96	0.9	0.3	0.2	
A2	1.040	142.5	142.5	202.5	0.00	0.00	0.0	0.0	0.0	
B2	1.040	144.6	144.6	204.0	0.00	0.05	0.0	0.0	0.0	
D2	1.040	174.1	174.1	225.8	0.00	0.15	0.0	0.1	0.0	
A3	2.015	46.5	1.0	1.0	1.55	0.00	1.0	0.9	0.4	
B3	2.015	52.6	24.5	24.5	1.09	0.00	0.7	0.2	0.2	
D3	2.015	110.3	100.0	100.0	0.44	0.00	0.3	0.0	0.1	
A4	0.975	105.0	142.5	142.5	0.00	0.00	0.0	0.0	0.0	
B4	0.975	107.8	144.6	144.6	0.06	0.00	0.0	0.0	0.0	
D4	0.975	145.0	174.1	174.1	0.15	0.00	0.0	0.0	0.0	
C1	2.015	63.0	63.0	155.8	0.00	1.15	1.0	0.5	0.3	
E1	2.015	128.5	128.5	191.9	0.00	0.84	0.9	0.2	0.2	
C2	1.040	155.8	155.8	212.1	0.00	0.11	0.0	0.1	0.0	
E2	1.040	191.9	191.9	239.8	0.00	0.17	0.0	0.1	0.0	
C3	2.015	78.3	63.0	63.0	0.64	0.00	0.4	0.0	0.1	
E3	2.015	136.7	128.5	128.5	0.35	0.00	0.2	0.0	0.0	
C4	0.975	122.4	155.8	155.8	0.12	0.00	0.0	0.0	0.0	
E4	0.975	165.9	191.9	191.9	0.15	0.00	0.0	0.0	0.0	

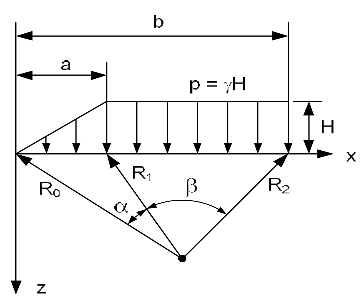


Figure 1

Notes:

1 $p = \gamma_{emb.} * H$

2 $R_0 = \sqrt{x^2 + z^2}$

3 $R_1 = \sqrt{(x-a)^2 + z^2}$

4 $R_2 = \sqrt{(b-x)^2 + z^2}$

5 $\alpha = \operatorname{atan}\left(\frac{x}{z}\right) - \operatorname{atan}\left(\frac{x-a}{z}\right)$

6 $\beta = \operatorname{atan}\left(\frac{x-a}{z}\right) + \operatorname{atan}\left(\frac{b-x}{z}\right)$

7 $\sigma_z = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} - \frac{z}{R_2^2}(x-b)\right]$

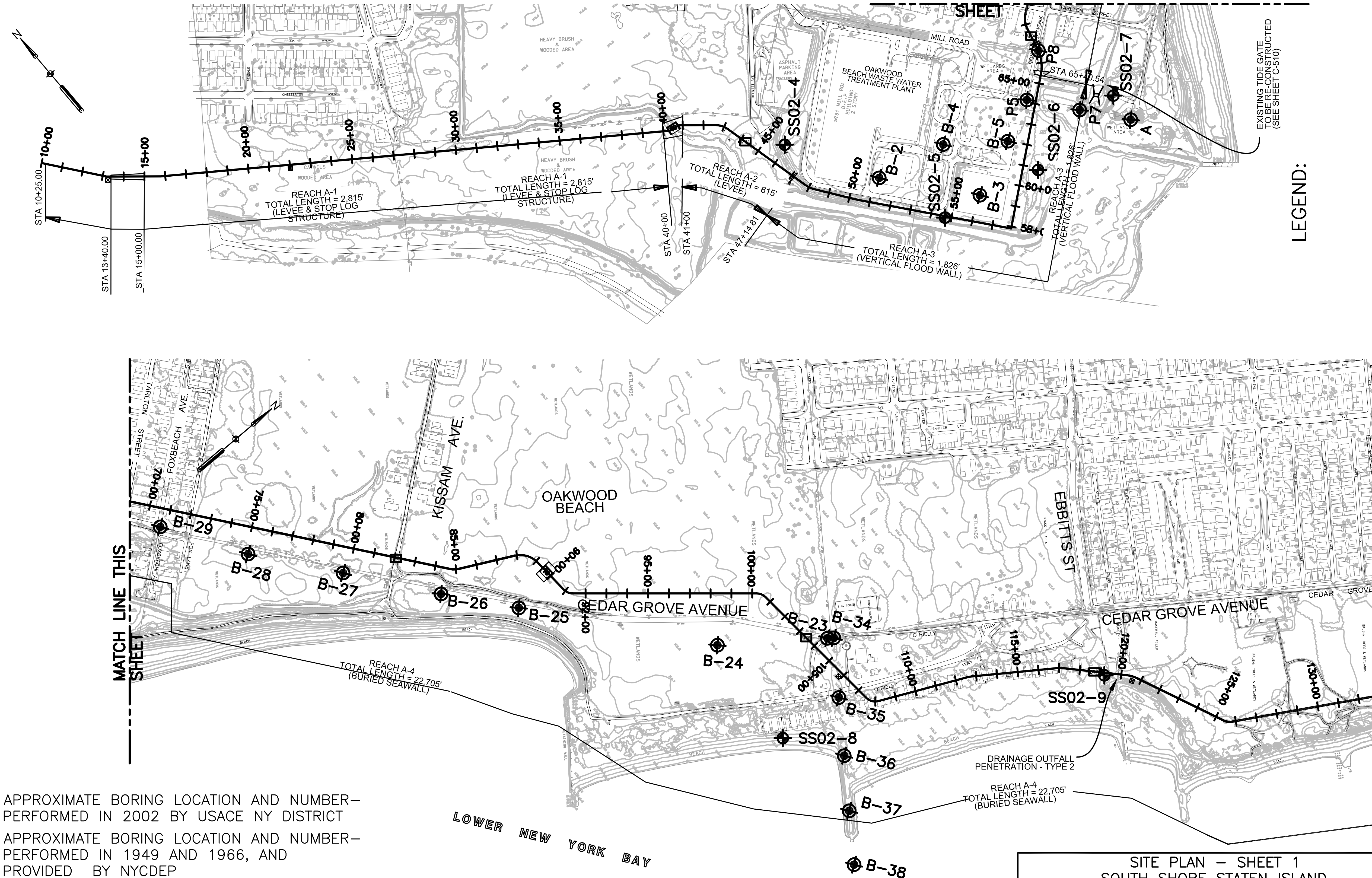
8 $\sigma_x = \frac{p}{\pi}\left[\beta + \frac{x\alpha}{a} + \frac{z}{R_2^2}(x-b) + \frac{2z}{a}\log_e\frac{R_1}{R_0}\right]$

9 $\sigma_y = \upsilon[\sigma_x + \sigma_z]$

Attachment

South Shore of Long Island

Feasibility Phase Boring Locations



LEGEND:

SS02-5	APPROXIMATE BORING LOCATION AND NUMBER– PERFORMED IN 2002 BY USACE NY DISTRICT
B-5, P5	APPROXIMATE BORING LOCATION AND NUMBER– PERFORMED IN 1949 AND 1966, AND PROVIDED BY NYCDEP
150+00	ALIGNMENT OF PROTECTION STRUCTURES (LINE OF PROTECTION)
DTP	COMBINED TRUCK & PEDESTRIAN ACCESS RAMP
MVA	MAINTENANCE VEHICLE ACCESS RAMP

SITE PLAN – SHEET 1
SOUTH SHORE STATEN ISLAND
NEW YORK

URS
CLIFTON, NEW JERSEY

DR. BY	ET	SCALE	AS SHOWN	DWG. NO.	01–PLAN	PROJ. NO.	11020302
CK'D. BY	NV	DATE	AUGUST, 2012			FIG. NO.	4

K:\CADD\11020302(SOUTH SHORE STATEN ISLAND)\01-PLAN.DWG(SHEET-1)

LEGEND:

SS02-5 ⊕	APPROXIMATE BORING LOCATION AND NUMBER–PERFORMED IN 2002 BY USACE NY DISTRICT
B-5, P5 ⊕	APPROXIMATE BORING LOCATION AND NUMBER–PERFORMED IN 1949 AND 1966, AND PROVIDED BY NYCDEP
150+00	ALIGNMENT OF PROTECTION STRUCTURES (LINE OF PROTECTION)
DTP	COMBINED TRUCK & PEDESTRIAN ACCESS RAMP
MVA	MAINTENANCE VEHICLE ACCESS RAMP

0 250 500 1000

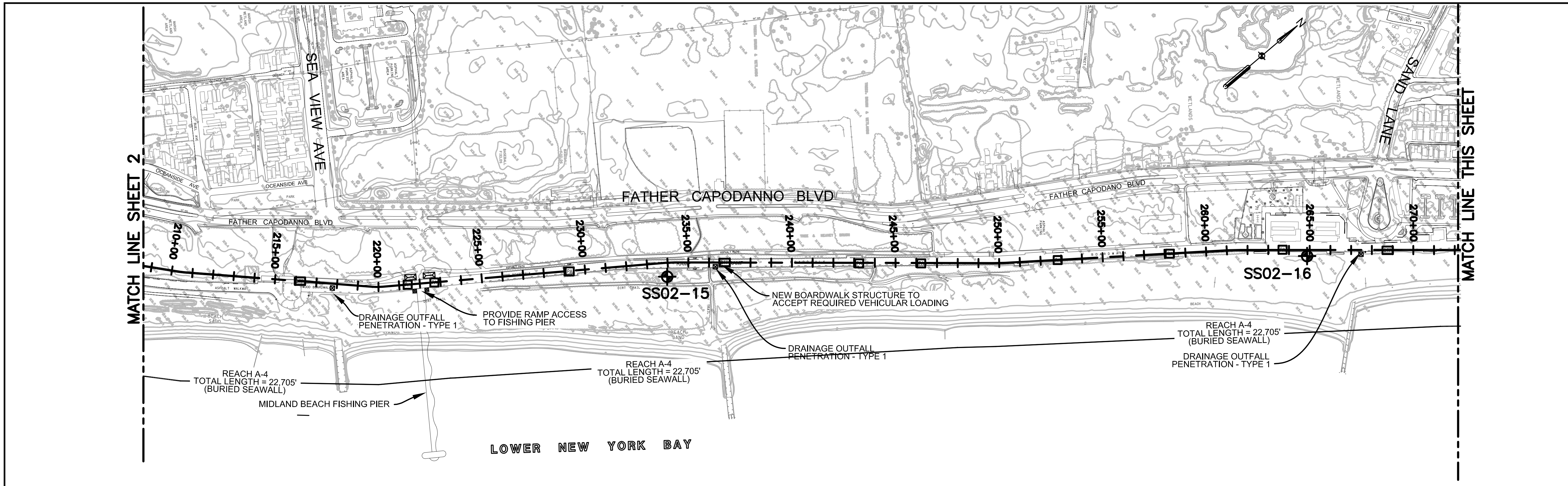
SCALE (FEET)

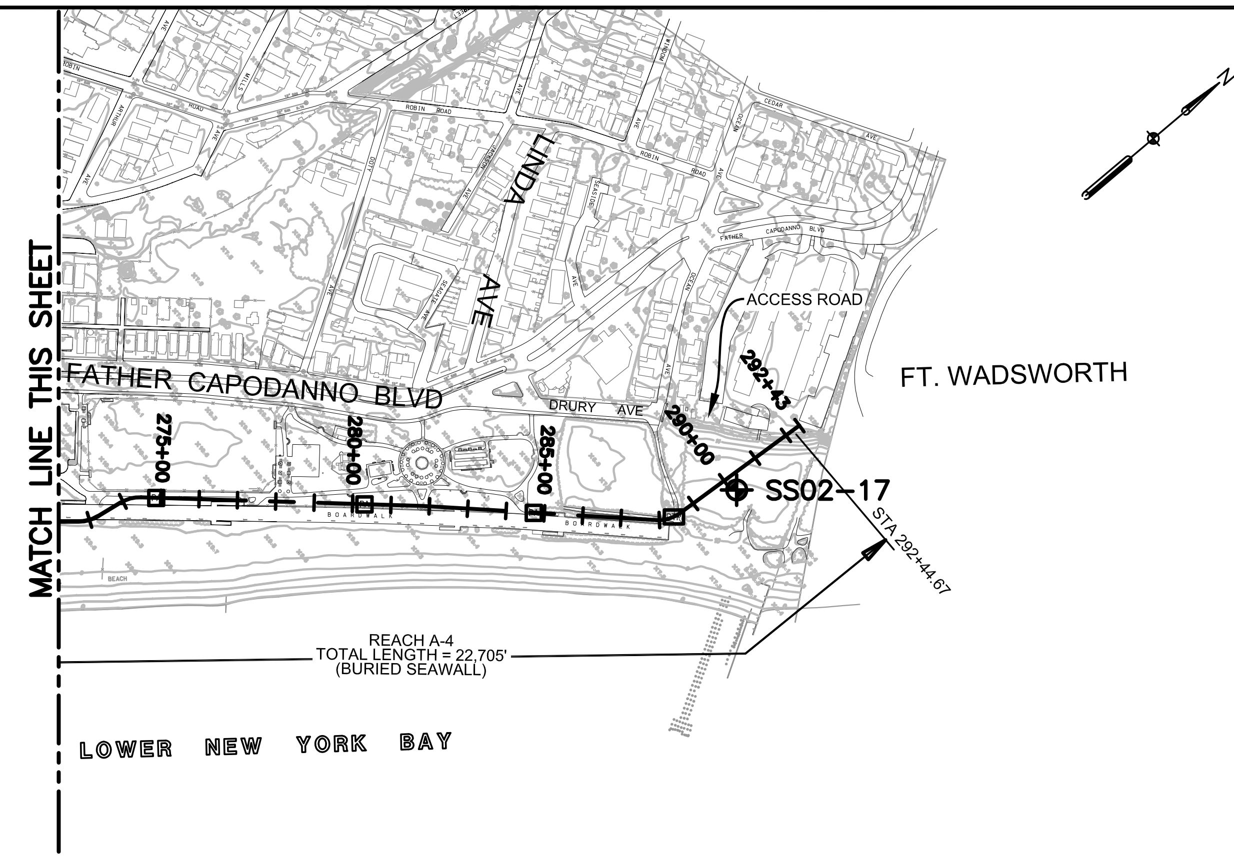
SITE PLAN – SHEET 2
SOUTH SHORE STATEN ISLAND
NEW YORK

URS
CLIFTON, NEW JERSEY

DR. BY	ET	SCALE	AS SHOWN	DWG. NO.	01–PLAN	PROJ. NO.	11020302
CK'D. BY	NV	DATE	AUGUST, 2012			FIG. NO.	5

K:\CADD\11020302(SOUTH SHORE STATEN ISLAND)\01-PLAN.DWG(SHEET-2)





LEGEND:

SS02-5 — APPROXIMATE BORING LOCATION AND NUMBER–PERFORMED IN 2002 BY USACE NY DISTRICT

B-5, P5 — APPROXIMATE BORING LOCATION AND NUMBER–PERFORMED IN 1949 AND 1966, AND PROVIDED BY NYCDEP

150+00 — ALIGNMENT OF PROTECTION STRUCTURES (LINE OF PROTECTION)

DTP — COMBINED TRUCK & PEDESTRIAN ACCESS RAMP

MVA — MAINTENANCE VEHICLE ACCESS RAMP

0 250 500 1000

SCALE (FEET)

SITE PLAN – SHEET 3
SOUTH SHORE STATEN ISLAND
NEW YORK

URS
CLIFTON, NEW JERSEY

DR. BY	ET	SCALE	AS SHOWN	DWG. NO.	01–PLAN	PROJ. NO.	11020302
CK'D. BY	NV	DATE	AUGUST, 2012			FIG. NO.	6

K:\CADD\11020302(SOUTH SHORE STATEN ISLAND)\01-PLAN.DWG(SHEET-3)