# SOUTH SHORE OF STATEN ISLAND, NY
# COASTAL RISK MANAGEMENT

# INTERIM FEASIBILITY STUDY
# FOR
# FORT WADSWORTH TO OAKWOOD BEACH

# Draft Interim Geotechnical Evaluation Appendix

US Army Corps of Engineers
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# 1.0 INTRODUCTION

1. This report presents the results of geotechnical evaluations performed by the Joint Venture of Moffatt & Nichol and URS as part of geotechnical and structural design tasks in support of the Interim Feasibility Study for Coastal Risk Management along the South Shore of Staten Island (SSSI) located in Staten Island, New York (see Figure 1).

## 1.1 Project Overview

2. The SSSI Coastal Risk Management Interim Feasibility Study project area is located between Fort Wadsworth and Oakwood Beach in Staten Island, New York (see Figure 2). The Interim Feasibility Study primarily includes three types of coastal risk management measures along the approximately 5 mile project reach. These measures consist of a buried seawall/armored levee, earth embankment levee, and floodwall (T-Wall). These primary measures make up the Line of Protection (LOP) for the Tentatively Selected Plan (TSP). The LOP is intended to act as the first line of defense against tidal and storm surges into the study area and provide coastal risk management to commercial and residential property during periods of hurricanes and other severe storms. The entire LOP is divided into 4 reaches designated as A-1 through A-4 (see Figure 3) based on the physical conditions of the shoreline, existing coastal and stormwater outfall structures, and LOP structure type. A depiction of these reaches and the corresponding LOP measures are presented in Figures 4 through 6.

3. For this study, LOP alternatives were considered for a 13.3, 14.3 and 15.6 ft. NAVD 1929 design stillwater elevation (SWEL). Typical sections of each measure type for the TSP (15.6 NGVD 1929 stillwater design level) are presented in Figures 7 through 9.

## 1.2 Interim Geotechnical Evaluations

4. The geotechnical evaluation appendix for the Interim Feasibility Study consisted of the following:

1) Reviewed the existing subsurface investigation data provided by the USACE and NYCDEP and utilized it for the geotechnical evaluations;
2) Prepared plan drawings showing the approximate locations of test borings;
3) Prepared generalized soil profiles along the alignment of the line of protection;
4) Conducted an engineering evaluation and prepared this report that includes the following:
    a) An overview of the general site and geologic conditions.
    b) A description of the previous subsurface investigation program and laboratory testing conducted.

c) A description of the generalized soil conditions throughout the project site.

d) A description of the seismic considerations including seismic site classification and liquefaction potential.

e) A description of the seepage and slope stability analyses performed and a summary of the results.

f) Appendices including boring logs, laboratory test results and calculation details.

# 2.0 SUBSURFACE INVESTIGATIONS AND SITE CONDITIONS

5. The subsurface investigation consisted of a field investigation and laboratory testing. The field investigation included a test boring program to identify soil and rock conditions for the project area. Details of the subsurface investigation and generalized subsurface conditions are presented in the subsequent sub-sections.

## 2.1 Local Geology

6. The south shore area of Staten Island is in a geological, structural, and topographic province known as the Atlantic Coastal Plain. In this area, the Coastal Plain consists of unconsolidated deposits of sands, silts, and clays that gently dip seaward. The coastal plain deposits are overlain with younger glacial deposits of till, outwash material, and moraine deposits. More recent deposits of fill, stream material, and reworked sediments overlie the glacial deposits. In general, the surficial sediments of Staten Island, from top to bottom, consist of artificial fill, outwash, terminal moraine, till, and marine and lacustrine sediments.

7. In the project area, the Coastal Plain deposits consist of the Raritan Formation, a thick sand unit, and the overlying Magothy Formation, which consists of silts, clays, with some sand layers. In the southern portion of Staten Island, the glacial deposits are over 50 ft thick and the underlying Coastal Plain sediments are over 100 ft thick (Merguerian, 2008). In the immediate site area, borings advanced to a depth of 30 ft encountered only glacial deposits.

## 2.2 Test Boring Program

8. A total of fourteen (14) test borings (designated as SS02-4 through SS02-17) were performed along the alignment of the tentatively selected LOP between Fort Wadsworth and Oakwood Beach. The subsurface exploration was conducted by Matrix Environmental Services of Florham Park, NJ in October 2002 for US Army Corps of Engineers (USACE).

9. All borings were advanced using mud rotary drilling techniques. Soil samples were obtained using techniques and equipment in general accordance with the American Society for Testing and Materials (ASTM) Standard Specification D1586-Standard Penetration Test (SPT). The SPT consists of driving a 2 inch O.D. split spoon sampler for a distance of 24 inches, with repeated blows of a 140 lb. hammer free falling a distance of 30 inches. The standard penetration, or N-value, is determined as the number of blows required to advance the sampler 12 inches after the initial 6 inches of penetration. Soils were classified using the Unified Soil Classification System (USCS) method and one to two samples per boring were chosen for laboratory analysis. Undisturbed Shelby tube samples were also obtained from relatively soft or organic fine-grained soils for laboratory testing. All test borings were advanced to final depths ranging from 24 to 30 ft below ground surface (bgs). Bedrock was not encountered in any of the test borings. The USACE (2002) test boring logs are provided in Attachment A.

10. A portion of the line of line of protection (LOP) between stations 30+00 and 85+00 (which also includes the Oakwood Beach Waste Water Treatment Plant) is located within a wetland area. Since vast majority of the test borings performed by USACE were outside the wetland area, additional soil test boring records of test borings performed within the wetland area were provided by New York City Department of Environmental Protection (NYCDEP). These test borings were performed by various New York City agencies in between 1949 and 1966. A total of 20 soil test borings were selected from the test boring records provided by NYCDEP. These selected test borings are included in Sheet 1 of the boring location plan (see Figure 4). The NYCDEP test boring records are provided in Attachment B.

## 2.3 Laboratory Testing Program

11. The laboratory testing program consisted of a variety of tests performed on selected soil samples obtained from the borings to verify the field classifications and to provide additional information for engineering evaluations. The tests included grain size, specific gravity, unit weight, Atterberg Liquid and Plastic Limits, and consolidated undrained triaxial compression. The triaxial compression strength, grain size, unit weight, and Atterberg limits tests were performed on undisturbed Shelby Tube samples. All tests were performed by SOR Testing Laboratories, Inc. of Cedar Grove, NJ. These laboratory test results are provided in Attachment C of this Appendix.

## 2.4 Generalized Subsurface Conditions

12. A generalized subsurface profile was developed along the line of protection and is presented in Figures 10 through 13. The generalized descriptions of the subsurface conditions at the site given below are primarily based on our interpretation of the results of the 2002 field investigation and laboratory test results.

**Stratum 1: SAND**

13. The primary soil type encountered within the project area was coarse to fine sand with varying amounts of silt and gravel. The laboratory tests show that the majority of the sands consist of trace to some amounts of silt and gravel. The borings also indicate the presence of some clay and silt lenses within this stratum that ranged from 1 to 9 ft in thickness, at various depths ranging from the ground surface to approximately 25 feet below ground surface. These lenses were encountered in borings SS02-5, SS02-7, SS02-13, SS02-14, and SS02-15. Boring SS02-8 indicated an approximately 2ft thick organic clay and silt lens at a depth of about 6.5 ft. Generally, the SPT N-values within this stratum generally ranged from 10 blows per foot (bpf) to 30 bpf with an average of about 18 bpf, indicative of a medium dense material. However, it should be noted that as mentioned above isolated pockets of soft silty/clayey soils and loose sandy soils were also encountered within the project limits. Since, all borings were terminated within this stratum the thickness of this stratum is not defined at present.

**Stratum 2: ORGANICS**

14. The NYCDEP soil test borings records indicated approximately 6 ft thick soft organic soils within the wetland area (Sta. 30+00 to 85+00). These organic soils were generally encountered immediately below the ground surface and overlying the sand layer (Stratum 1).

## 2.5 Groundwater Conditions

15. Considering the proximity of the site to the Lower New York Bay and the topography of the site it is anticipated that the groundwater is likely to be encountered at about +2 ft (NGVD 29).

# 3.0 ENGINEERING EVALUATIONS

16. As per the project requirements, engineering evaluations were primarily performed using the USACE design manuals, EM 1110-2-1913 “Design and Construction of Levees” (2000), EM 1110-2-1902 “Slope Stability” (2003), EM 1110-2-2502 “Retaining and Flood Walls” (1989), and EM 1110-1-1904 “Settlement Analysis” (1990).

17. The subsequent sub-sections provide descriptions and results of analysis performed to evaluate soil behavior under seismic conditions, seepage conditions, and slope stability.

## 3.1. Seismic Considerations

18. In accordance with EM 1110-2-1913, slope stability analyses should also be performed for the seismic loading case as presented in Section 3.5. At present there is no USACE Engineering Manual for seismic analysis. Therefore, seismic analyses were performed herein based on the National Earthquake Hazards Reduction Program (NEHRP, 2009) seismic provisions.

19. The seimic loading condition was evaluated using the pseudo-static method of analysis. The effects of the seismic motion were simulated by applying a pseudo-static coefficient in the horizontal direction. The pseudo-static coefficient was assumed 2/3 of the peak ground acceleration (PGA) at the foundation (ground surface) level for the 2,500-year seismic event. Considering that the depth to bedrock at the project areaappears to be greater than 100 ft, and the soils within the top 100 ft are likely to be generally medium dense to dense in compactness, the soil profile type is seismic site class ‘D’ (SD). Based on 2008 Probabilistic Hazard Curves from the U.S. Geological Survey (USGS, 2008), the PGA at the bedrock level is approximately 0.16g for a 2,500-year seismic event at the project site. Therefore, as per NEHRP (2009) provisions, the PGA at the ground surface for seismic site class D (SD) soil profile was estimated to be about 0.24g. Hence, the pseudo-static coefficient of 0.16g (i.e., $0.67 \times 0.24$) was assumed for the seismic loading case.

20. The phenomenon of soil liquefaction, or significant reduction in soil strength and stiffness as a result of shear-induced increased pore-water pressure, is a major cause of seimic damage to embankments and slopes. Since the sandy soils below the groundwater level at the project site are generally medium dense to dense in compactness, it appears that seismic induced liquefaction at the project site will not likely occur and therefore should not be a concern. Therefore, sophisticated dynamic analyses of stability and deformations under seismic loadings are not warranted. However, it should be noted that at a few isolated locations pockets of loose sandy soils were encountered and additional

investigations will be required at these locations to verify the extent of such loose sandy soils.

## 3.2. Representative Sections for Analyses

21. Seepage and stability analyses were performed for each type of structure using representative sections. The sections were selected so as to represent a maximum height of the measure above grade along the project reach. A summary of the selected representative sections are presented below in Table 3-1.

**Table 3-1: Summary of Representative Sections**

| Structure Type | Figure No. | Reach Nos. | Maximum Height (ft) | Total Length (ft) |
|---|---|---|---|---|
| Levee | 7 | A-1 and A-2 | 20 | 3,430 |
| Floodwall | 8 | A-3 | 17.5 | 1,826 |
| Buried Seawall/Armored Levee | 9 | A-4 | 19.5 | 22,705 |

## 3.3. Seepage Analyses

22. Seepage analyses for the three types of coastal risk management measures were performed in order to estimate the seepage quantity through and/or underneath the structures, exit hydraulic gradients on the land upside of the structures and the pore pressures within the embankments. The results of these analyses were used to perform the slope stability analyses described in Section 3.5.

23. Typically, a fully developed phreatic surface obtained from a steady-state seepage analysis to perform the slope stability analysis under a long-term condition is used when it is expected that the water remains at or near flood stage for a sufficient period of time to result in full embankment saturation and a condition of steady seepage. However, considering the relatively short duration (about 6 hours to 24 hours) of anticipated storms, this condition will most likely not occur during the anticipated storms. Therefore, as presented in URS memorandum dated July 22, 2011 (see Attachment D), both transient and steady seepage analyses were performed for the buried seawall/armored levee. For all other structures, only steady seepage analyses were performed because steady-state seepage analyses are conservative compared to the transient analyses.

24. The design SWEL of 15.6 ft (NGVD 29) was used in the seepage analyses. The storm hydrographs used in the transient seepage analyses are presented in Figures 14 and 15. In addition, the hypothetical storm hydrograph used to determine the duration of the storm to develop a steady-state seepage condition is presented in Figure 16.

25. The 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. As indicated in Section 3.3, these representative sections were conservatively selected at maximum height locations. One of the important parameters required to perform the seepage analyses is the hydraulic conductivity of storm damage reduction structure materials and foundation materials. The hydraulic conductivity values of various materials and the results of the seepage analyses using these values are presented in Section 3.3.1 and Section 3.3.2, respectively.

### 3.3.1. Selection of Hydraulic Conductivity Values for Analyses

26. The saturated hydraulic conductivity of porous materials varies typically by one or two orders of magnitude (e.g. silty sand, $10^{-3}$ to $10^{-5}$ cm/sec). Therefore, seepage analyses were performed for a range of hydraulic conductivity values. Based on the results of these analyses, conservative values were selected and are presented in this section.

27. The phreatic surfaces for the stability analyses were developed from the seepage analyses. In order to develop the phreatic surfaces, the materials within the embankments were modeled as saturated / unsaturated materials with hydraulic conductivity as function of the pore pressure. However, considering that the results of the seepage analyses are not sensitive to hydraulic conductivity as function of the pore pressure, only saturated hydraulic conductivity values are presented. They are:

1) The foundation soils generally consist of coarse to fine sands with varying amounts of clay, silt and gravel. Considering this, the hydraulic conductivity (k) for the foundation soils was assumed to be $1 \times 10^{-4}$ cm/sec. Typical hydraulic conductivity values were obtained from Electric Power Research Institute (EPRI) report EL-6800 (after Terzaghi and Peck, 1967), Page 7-1 and Table 7-1.
2) Compacted fill will be used for core and shell material for levee structures, and as earth cover material on the water side and impervious fill on the landside for the buried seawall/armored levee. Considering that the compacted fill should be relatively impervious, it is anticipated that silty sand (SM) and/or clay sand (SC) with a hydraulic conductivity less than $1 \times 10^{-5}$ cm/sec will be used as compacted fill. Therefore, for the compacted fill, a hydraulic conductivity (k) of $1 \times 10^{-5}$ cm/sec was assumed.
3) Armor and bedding stones will be used for the construction of the buried seawalls/armored levee. Considering that these materials will have a significant amount of voids, a hydraulic conductivity (k) of 10 cm/sec was assumed for these materials.
4) Steel sheet piles, concrete fascia and concrete caps will be used for the construction of sheet pile walls and Floodwalls. Considering that any water

seepage through the joints of the sheet piles walls and concrete cracking will be relatively small, a hydraulic conductivity (k) of $1 \times 10^{-6}$ cm/sec was assumed for these materials.

### 3.3.2. Results of Analyses

28. The seepage analyses results are presented in this section for three types of LOP measures. The analyses were performed using the representative sections presented in Table 3-1 and for a SWEL of 15.6 ft (NGVD 29) as described previously.

29. The results of both transient and steady-state seepage analyses were presented in URS memorandum dated July 22, 2011 (URS, 2011), for Buried Seawalls (see Attachment D). Based on those analyses, steady-state seepage conditions are not expected to develop during the anticipated storms. Therefore, for Buried Seawalls, the results of transient seepage analyses are presented in Table 3-2. However, for all other structures, the steady-state seepage (conservative) analyses results are presented in Table 3-2.

**Table 3-2: 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 |

30. The results of the seepage analyses are graphically presented in Figures 17 through 19 for all three types of structures. In addition, the exit corresponding hydraulic gradient graphs are presented in Figures 20 through 22. 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 next section.

## 3.4. Slope Stability Analyses

31. 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. As per EM 1110-2-1913, slope stability analyses were performed for four loading conditions as follows:

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

32. 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 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 surface associated with the minimum FOS, which is considered the critical or controlling shear surface. As mentioned in Section 3.4, 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 both 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.

33. Because of the low probability of earthquakes coinciding with severe storm events, stability analyses for the Case IV (earthquake) loading condition was performed assuming no water above the ground surface. As described in Section 3.2, pseudo-static coefficient of 0.16g was assumed for the earthquake loading case.

34. Besides knowledge of the pore pressure distribution within the embankment, the shear strength parameter values of the embankment materials and foundation soils are important for the slope stability analyses. Section 3.4.1 the assumed shear strength parameter values are presented.

### 3.4.1. Selection of Material Parameter Values for Stability Analyses

35. The material parameters required for the stability analyses are the shear strength and unit weight properties of the embankment fill and foundation soils. Considering that sandy soil and stones will be used as embankment fill materials, and since the foundations soils generally consist of sandy materials, effective stress shear strength parameter values were used in the stability analyses for all conditions as follows:

1) Foundation soils are generally medium dense to dense sandy soils. Based on the SPT N-values obtained within the foundation soils and widely used empirical correlations, a conservative effective stress friction angle of 30 degrees was used in the current analysis for the foundation soils. However, as mentioned

previously in Section 2.5, pockets of clayey/silty soils and loose sandy soils were encountered at isolated locations. But, currently it was assumed that there are no continuous layers of soft clayey/silty soils and/or loose sandy soils within the project limits. However, we have conservatively included a loose sand layer in the slope stability analyses above the medium dense sand layer.

2) Sandy fill will be compacted to a density corresponding to 95% of the maximum dry density. Therefore, a conservative effective stress friction angle of 32 degrees was used in the current analysis for the compacted fill.
3) Bedding stone and armor stone friction angle values are typically greater than 36 degrees. Therefore, conservative effective stress friction angle values of 36 degrees and 38 degrees were used in the current analyses for bedding stone and armor stone, respectively.

36. Table 3-3 below summarizes the material shear strength and unit weight parameter values used in the stability analyses.

**Table 3-3: Summary of Material Parameters for Stability Analyses**

| Materials | Unit Weight (pcf) | Friction Angle (degrees) | Cohesion (psf) |
|---|---|---|---|
| Foundation Soils (upper stratum – loose) | 120 | 26 | 0 |
| Foundation Soils (lower stratum – medium dense) | 120 | 30 | 0 |
| Compacted Fill | 125 | 32 | 0 |
| Bedding Stone | 140 | 36 | 0 |
| Armor Stone | 145 | 38 | 0 |

37. As mentioned in Section 2.5, soft compressible soils were encountered within the wetland area (Sta. 30+00 to 85+00). Therefore, the end-of-construction (Case I) slope stability analysis will need to be performed using the undrained shear strength of the organic soils (Stratum 2). At present undrained shear strength of 200 psf was assumed for the organic soils. However, during the design stage undisturbed tube samples of organic soils should be obtained and triaxial undrained shear strength tests should be performed.

### 3.4.2. Results of Analyses

38. The slope stability analyses results are presented in this section for Buried Seawall/Armored Levee and Earthen Levee. As presented in URS (2011), the slope stability analyses of buried seawalls for the Case II loading condition was 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 3-4, along with the corresponding minimum acceptable factors of safety.

**Table 3-4: Summary of Slope Stability Analyses Results**

| Slope | Design Condition | Minimum Acceptable Factor of Safety | Buried Seawall | Levee |
|---|---|---|---|---|
| Land Side | Case I: End of Construction | 1.3 | 1.4 | 1.7 |
| Lind 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 |

39. The results of the stability analyses are graphically presented in Figures 23 through 33 for the above mentioned structures. The stability analyses results of buried seawalls for the Case II loading condition, based on both transient and steady-state seepage analyses was previously summarized in URS (2011) and is provided in Attachment D.

Remarks:

40. Table 3-4 presents the factors of safety after the soft organic soils have been excavated and the compact fill added. Figures 23 and 28 depict that that 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, wherever encountered the soft organic soils should be removed and replaced with compacted sandy fill.

41. As shown in Figure 30, Case II factor of safety value is 0.8 for sacrificial cover layer of the buried seawall under steady seepage condition. However, as mentioned in Section 3.4.1 and URS (2011), steady seepage condition will most likely not occur during the anticipated storms. Further, 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.

42. Additional test borings should be performed within the wetland area and within the reminder of the alignment during the design stage to completely characterize the subsurface conditions along the LOP.

## 3.5. Settlement Analyses

43. As stated under the stability analyses remarks (Para 40), in order to have satisfactory slope stability for the levee sections, soft organic soils wherever encountered should be removed and replaced with compacted sandy fill. Thus, the existence of soft organic soils was not considered in the settlement analyses. Potential immediate settlement values were estimated as per EM 1110-1-1904. Accordingly, the estimated immediate settlement values approximately ranges from ½ inch to 1½ inches. Since most of the estimated immediate settlement will likely to occur during construction, it should not be a concern. Details of the immediate settlement analyses are provided in Attachment E. Long term consolidation settlement should not be a concern because after removing any soft organic soil layer that could be present near the ground surface the subsurface soils are generally sandy soils as indicated in Section 2.4 (Para 13).

## 3.6. Floodwall Pile Recommendations

44. It is our understanding that the floodwall is to be supported on pile foundation. Based on the subsurface conditions and DRIVEN pile capacity analyses, we recommend HP14x89 friction piles driven to sandy stratum for this purpose. Refer to Attachment F for the DRIVEN analysis results and additional details. 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 |

FIGURES

# ATTACHMENT A – TEST BORING LOGS (USACE)

# ATTACHMENT B – TEST BORING LOGS (NYCDEP)

ATTACHMENT C – LABORATORY TEST RESULTS

# ATTACHMENT D – URS MEMORANDUM

ATTACHMENT E – SETTLEMENT ANALYSES RESULTS

# ATTACHMENT F – FLOODWALL PILE ANALYSIS RESULTS