GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

AACE FILE NO. 16-112

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TABLE OF CONTENTS

PAGE #

1.0 INTRODUCTION 1

2.0 SITE INFORMATION AND PROJECT UNDERSTANDING 1
- ***2.1 Site Location and Description*** 1
- ***2.2 USDA Soil Survey*** 2
- ***2.3 Evaluation of Sinkhole Potential*** 2
  - 2.3.1 Mechanics of Sinkhole Formation 2
  - 2.3.2 Sinkhole Potential 3
- ***2.4 Project Understanding*** 4

3.0 FIELD EXPLORATION PROGRAM 5

4.0 LABORATORY TESTING PROGRAM 5

5.0 OBSERVED SUBSURFACE CONDITIONS 6
- ***5.1 Soil Conditions*** 6
  - 5.1.1 SPT Borings 6
  - 5.1.2 Hand Auger Borings/Penetrometer Probings 6
- ***5.2 Measured Groundwater Level*** 8
- ***5.3 Estimated Normal Seasonal High Groundwater Table*** 8
- ***5.4 Piezometer Installations and Field Permeability Tests*** 8

6.0 GEOTECHNICAL ENGINEERING EVALUATION 9
- ***6.1 General*** 9
- ***6.2 Seepage Analyses*** 9
  - 6.2.1 Selection of Engineering Properties 10
  - 6.2.2 Boundary Conditions 11
  - 6.2.3 Analyses Approach and Results 11
- ***6.3 Slope Stability Analyses*** 13
  - 6.3.1 Geometry and Soil Parameters 13
  - 6.3.2 Loading Conditions and Required Factors of Safety 13
  - 6.3.3 Analyses Results 14
- ***6.4 Discussion of Analyzed Scenarios/Limited Parametric Study*** 15
  - 6.4.1 Seepage Collection Ditch Levels 15
  - 6.4.2 Berm Material Shear Strength 16
  - 6.4.3 Berm Material Permeability 17
  - 6.4.4 General 17
- ***6.5 Berm Construction Considerations*** 18
- ***6.6 Pump Station and Emergency Overflow Structure*** 18
- ***6.7 Piping Effects - Structures and Pipes*** 19
- ***6.8 Additional Considerations*** 20
- ***6.9 Quality Control*** 20

7.0 CLOSURE 21

- Figure No. 1 Regional Map and USGS Topographic Map
- Figure No. 2 USDA Soil Survey Map
- Figure No. 3 Typical Design Cross-Section
- Figure No. 4 Field Work Location Plan
- Sheet No. 1 General Notes
- Sheets No. 2-4 Soil Boring Profiles
- Sheet No. 5 In-Situ Well Permeability Test Results
- Sheets No. 6-9 Seepage and Slope Stability Analyses Results

- Appendix I: Site Photographs
- Appendix II: USDA Web Soil Survey Summary Report
- Appendix III: General Notes (AACE Soil Boring, Sampling and Testing Methods)
- Appendix IV: Laboratory Test Results
- Appendix V: AACE Project Limitations and Conditions

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
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AACE File No. 16-112
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1.0 INTRODUCTION

In accordance with the request and authorization of Royal Consulting Services, Inc. (RCS), Andersen Andre Consulting Engineers, Inc. (AACE) has completed a subsurface soil exploration program and geotechnical engineering evaluation for the above referenced project. The purpose of this study is to evaluate the geotechnical conditions within this site as it relates to the design and construction of a proposed minor above-ground impoundment. Our work included multiple site visits and review of available relevant literature, followed by the completion of field exploration and laboratory testing programs from which the results were incorporated into our engineering analysis and evaluation.

2.0 SITE INFORMATION AND PROJECT UNDERSTANDING

2.1 Site Location and Description

The Istokpoga Marsh Watershed Improvement District (IMWID) is located in the southwest portion of Highlands County, Florida (directly south of Lake Istokpoga) and consists of approximately 22,000 acres of agricultural land with about 28 miles of internal canals that serve as both water supply and drainage. The existing canal system does not currently provide adequate water quality treatment and/or storage for the needs of the district. As such, the overall long-term goal for IMWID is to construct above-ground impoundments which will capture, store, treat and recycle stormwater for irrigation purposes.

The proposed IMWID Cell No. 2 is part of the planned above-ground impoundments and is located about 3 miles southeast of Lake Istokpoga (within Sections 3, 4, 9 & 10, Township 37 South and Range 31 East). The location of the subject site is graphically depicted on a 2014 aerial photograph included as Figure No. 1, and on a reproduction of the 1972 USGS Quadrangle Map of "Basinger SW, Florida" also included on Figure No. 1. The USGS Quadrangle Map depicts the subject property as being relatively level with a ground surface elevation of about 34 feet relative to the National Geodetic Vertical Datum of 1929.

The roughly rectangular, east-west oriented, approximately 400-acre subject site is currently undeveloped and in use for cattle grazing , and was reportedly previously used for sod farming. It is bordered to the north, south, and west by similar agricultural land with associated drainage ditches and canals. Further, the site is bordered to the east by the South Florida Water Management (SFWMD) C-41A Canal. The property is accessed from County Road 621 and the unpaved Driggers Road, using agricultural roads which lead to the west edge of the site. Aside from a few dilapidated cattle shade structures, the site is undeveloped, relatively level, and absent of vegetation apart from low-creeping grasses and weeds. Internal shallow drainage ditches are present throughout the site. The average east-west cross-distance is about 1.2 miles and the average north-south cross-distance is about 0.45 miles.

Representative site photographs taken during the initial site reconnaissance and also during the completion of our field work are presented in Appendix I.

2.2 USDA Soil Survey

Based on the U.S. Department of Agriculture (USDA) Web Soil Survey, the subject parcel is located in an area characterized by the following two soil types:

- Kaliga muck, frequently flooded, 0-1 percent slopes (Map Unit Symbol 18) [present within the eastern ⅔ of the site[
- Tequesta muck, frequently flooded, 0-1 percent slopes (Map Unit Symbol 26) [present within the eastern ⅓ of the site[

These two soil types are noted to originate from depressions on marine terraces and consisting of herbaceous organic material over sandy and loamy marine deposits, with a surficial layer of organics/muck (12-25 inches thick) followed by fine sands, fine sandy loam, sandy clay loam and sands extending to depths in excess of 80 inches below grade.

The location of the subject site is shown superimposed on the USDA NRCS Web Soil Survey presented as our Figure No. 2. Further, the summary report from the USDA NRCS Web Soil Survey is included in Appendix II.

2.3 Evaluation of Sinkhole Potential

Unless specifically requested to do so, as a standard of practice in this area we do not typically include field explorations sufficient to assess the potential for sinkhole/karst activity, mainly because of the rare occurrences of sinkholes in this part of Highlands County. Rather, we believe the following discussion will be sufficient to allay concerns about sinkholes at the subject site.

2.3.1 Mechanics of Sinkhole Formation

There are three distinct types of sinkholes that have developed in Florida. The first type is the classical collapse sinkhole, which is generally steep-sided and rocky. It occurs when a cavity in the limestone can no longer support the weight of the overlying soil and rock. These types of sinkhole generally occur when the limestone is at or near the surface and solution weathering is still very active.

The second type of sinkhole, which is more common though not as dramatic as the collapse sinkhole, is called a doline or solution sinkhole. There is no physical disturbance of the soluble rock beneath a doline. Subsidence of the overlying soil occurs due to gradual lowering of the rock surface and/or the gradual dissolution or leaching of calcium carbonate from the calcareous soil and rock which exists between the ground surface and the underlying aquifers. The Florida Geological Survey estimates that this type of subsidence occurs at the rate of one foot every five to six thousand years. Because the water flows radially to the intersection of vertical joints where the water enters the rock mass, the surface expression of the rock lowering or the leaching of the soluble soil constituents is a shallow depression located over the intersection of the joints. In some cases, the surface depression has the same shape as the surface of the underlying calcareous deposits, as in the case of a shell bed that has dissolved or partially dissolved since deposition.

The third type of sinkhole and probably the most common type occurring in Florida is the erosion sinkhole. Erosion sinkholes occur most frequently in an environment with the following characteristics:

- Limestones overlain by relatively pervious unconsolidated sediments, e.g., sandy soils.
- Cavity systems present in the limestone.
- A water table higher than the potentiometric surface of the underlying limestone.
- A breach of the limestone into the cavernous zone creating an area of high recharge to the artesian aquifer.

Under these circumstances, water moving down into the limestone may take large amounts of sediments into the cavernous system creating a void in the overlying sediment. When the void in the overlying sediment reaches the size at which the roof is no longer stable, the overburden will suddenly collapse. In many cases, the overburden is visible after the collapse, but some sinkholes of this type have occurred in which the collapsed overburden disappeared into the cavity system. In other cases, the sudden subsidence of the ground surface is only six inches to one foot deep.

2.3.2 Sinkhole Potential

While the mechanics of cavity and sinkhole formations are generally understood, the evaluation of a particular site for potential sinkhole development is not yet amenable for precise scientific prediction. Present tools utilized for such evaluation include local experience, review of geological history, assessment of regional surficial and bedrock geology, review of hydrogeological information, and review of aerial photographs and topographic maps.

Factors that must be considered in assessing the potential for sinkhole activity include the presence of linear surface features, thickness of clay beds above the limestone layers, hydraulic head difference between the water table and potentiometric surface in artesian aquifers, groundwater pumping, etc.

Based upon the geology and the hydrogeology of the site vicinity, the elevation of the potentiometric surface of the uppermost artesian aquifer (Floridan aquifer system) is probably about 40 to 75 feet NGVD[1]. This is above the ground surface at the subject site. The potential flow direction between the aquifers is, therefore, upward from the Floridan aquifer system into the surficial aquifer. Even if local pumping created a drawdown within the Floridan aquifer system such that the potential flow direction was induced to be downward from the surficial aquifer into the artesian aquifer, the beds of clay separating the aquifers would greatly restrict this flow.

Our review of available USGS maps and aerial photographs, as well as our review of recorded sinkhole activity of Highlands County[2], indicates the presence of only one potential sinkhole within a 100 square mile area around the subject site.

In summary, the past and present geologic, hydrologic and geotechnical evidence available to date indicates that the type of conditions favorable for the development of sinkholes probably do not exist in the vicinity of this site. Furthermore, no evidence of recent sinkhole development has been observed and recorded in the area, nor do aerial photographs indicate recent sinkhole activity. It is our opinion that the potential for sinkhole activity at the subject site is extremely low and a field exploration program to help assess the potential does not appear warranted.

References:
(1) USGS Scientific Investigations Report 2010-5097
"Hydrogeology and Groundwater Quality of Highlands County, Florida"
(2) University of South Florida, College of Engineering
"Map of Highlands County Sinkholes"

2.4 Project Understanding

The IMWID Cell No. 2 above-ground impoundment is proposed as a "fit-for-purpose" agricultural impoundment which will provide storage and water treatment for agricultural water/runoff prior to release into the SFWMD C-41A canal.

Based on our design team conversations, and following our review of the provided 90% Design Plans prepared by RCS (dated December 2016), we understand that the proposed impoundment will receive water from the adjacent IMWID Channel 'A', via a proposed pump station, at its northwest corner and will have an emergency overflow feature (top Elevation 33.7 ft-NAVD) near its southwest corner releasing back into Channel 'A'. The impoundment will be constructed with earthen perimeter berms and an outside perimeter seepage collection ditch on the north, east and south sides. An interior borrow ditch will be excavated to accommodate the construction of the earthen impoundment berms.

The average ground elevation within the proposed impoundment area is approximately 29.7 ft-NAVD and the height of the impoundment berm will be 6.5 feet, providing for a top berm elevation of 36.2 ft-NAVD. The impoundment berm is currently proposed to have a crest-to-crest width of 15 feet, 3H:1V side slopes, and will be cross-graded to drain towards the impoundment. The maximum normal storage pool within the impoundment is proposed to be 4 feet (EL 33.7 ft-NAVD), and increasing to 4.5 feet (EL 34.2 ft-NAVD) during the design storm event. We note that AACE has not been requested to complete a Freeboard Analysis (i.e. wave runup analysis based on existing fetch distances and proposed water storage depth); based on our conversations with the design team and SFWMD, we understand that such analysis will not be required for this project.

The proposed perimeter seepage collection ditch to be constructed on the north, east and south sides of the impoundment will be approximately 4.5-5 feet deep, have a bottom width of 4 feet, and is expected to have side slopes of 2H:1V or flatter. The area between the downstream/outside toe of the impoundment berm will be raised and cross-graded to drain away from the berm and towards the seepage ditch. The interior borrow ditch is expected to have a design configuration of 36-ft top width, 12-ft bottom width and 2H:1V side slopes, however, it may be deepened, widened, etc. depending on the quality of the soils (relative to earthwork/berm construction) at a given location. Overall, the borrow ditch will maintain a minimum distance of 5 times its depth to the interior/downstream toe of the impoundment berm.

A schematic showing the proposed design cross-section (as understood by AACE) is presented on Figure No. 3.

The proposed pump station feature will consist of an approximately 10-ft deep riprap-lined sump with concrete intake structures and associated pumps, piping, headwalls, etc. Further, the proposed emergency overflow feature is currently designed with four approximately 10-ft deep concrete box structures with associated piping allowing for emergency discharge into the adjacent Channel 'A'. Both, the pump station piping and emergency overflow piping will cross through and/or below the proposed impoundment berms.

3.0 FIELD EXPLORATION PROGRAM

The following subsurface exploration program was completed relative to the proposed impoundment project:

Table 1 - Field Exploration Summary

Boring/Field Work Type	Number	ASTM	Depth below grade [feet]	Location
Standard Penetration Test	12	D1586	30 - 35	Refer to Figure No. 4
Hand Auger/Muck Probe	30	D1452	0.5 - 4	Refer to Figure No. 4
Piezometers	6	NA	10 - 30	Refer to Figure No. 4

The subsurface exploration program summarized in Table 1 was performed in the period March 7-11, 2016. The boring locations shown on Figure No. 4 were determined in the field by our field crew using WAAS enabled hand-held GPS instruments, in addition to obtained aerial photographs, provided surveys and site plans, and existing site features as references. The locations should be considered accurate only to the degree implied by the method of measurement used. We preliminarily anticipate that the actual locations are within 30 feet of those shown on Figure No. 4.

Descriptions of our drilling and testing procedures are presented in Appendix III and summarized on Sheet No. 1, and the individual SPT boring logs are presented on Sheets No. 2-4. Samples obtained during performance of the borings were visually classified in the field, and representative portions of the samples were transported to our laboratory in sealed sample jars for further classification. The soil samples recovered from our explorations will be kept in our laboratory for 60 days following the date of this report, then discarded unless you specifically request otherwise.

4.0 LABORATORY TESTING PROGRAM

Our drillers and field engineers observed the soil recovered from the SPT samplers, placed the recovered soil samples in moisture proof containers, and maintained a log for each boring. The recovered soil samples, along with the field boring logs, were transported to our Port St. Lucie soils laboratory where they were visually examined by AACE's project engineer to determine their engineering classification. The visual classification of the samples was performed in general accordance with the Unified Soil Classification System, USCS. Further, representative samples of the encountered soils were selected for limited index laboratory testing, consisting of"percent fines" tests (ASTM D1140), moisture content tests (ASTM D2216), and organic content tests (ASTM D2974). These tests were performed to aid in classifying the soils and to help evaluate the general engineering characteristics of the site soils. The results of our laboratory testing are included in Appendix IV and are shown on the soil boring profiles presented on Sheets No. 2-4.

5.0 OBSERVED SUBSURFACE CONDITIONS

5.1 Soil Conditions

5.1.1 SPT Borings

Detailed subsurface conditions are illustrated on the SPT soil boring profiles presented on the attached Sheets No. 2-4. The stratification of the boring profiles represents our interpretation of the field boring logs and the results of laboratory examinations of the recovered samples. The stratification lines represent the approximate boundary between soil types. The actual transitions may be more gradual than implied.

As shown by the SPT soil boring profiles on Sheets No. 2-4, the soils on the site at the locations and the depths explored consist generally of a thin surficial mantle of organic soils (peat) ranging in thickness from a few inches to 2.5-3 feet, followed by intermixed layers of loose to medium dense fine sands (SP), slightly clayey fine sands (SP-SC), clayey fine sands (SC), and slightly silty (SP-SM) and silty (SM) fine sands reaching the termination depths of our borings.

The above soil profile is outlined in general terms only. Please refer to Sheets No. 2-4 for individual soil profiles. We specifically emphasize that SPT borings TB-10, TB-11 and T-12 encountered 5-7 feet of surficial silty fine sands (SM) with fines content in excess of 20-30 percent; these soils may not be suitable for berm construction and, as such, the proposed borrow ditch excavations may not be preferred in this portion of the site.

5.1.2 Hand Auger Borings/Penetrometer Probings

In addition to the twelve (12) completed SPT borings, thirty (30) shallow hand auger borings and penetrometer probings were completed in between the SPT boring locations so as to explore the thickness of the surficial mantle of organic soils. The findings of these “muck probes” are summarized in Table 2 below, along with the organic thickness reported to us by RCS from their in-house test pit explorations as well as the findings from our SPT borings.

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Table 2 - Thickness of Encountered Surficial Organic Soil Stratum

RCS Test Pit No.	Thickness (in)	AACE Probe No.	Thickness (in)	AACE SPT boring	Thickness (in)
1	16	1	6	1	4
2	16	2	12	2	6
3	16	3	0	3	12
4	16	4	9	4	6
5	16	5	6	5	6
6	24	6	16	6	12
7	16	7	4	7	12
8	16	8	6	8	36
9	16	9	3	9	28
10	16	10	6	10	24
11	21	11	6	11	18
12	24	12	12	12	10
13	30	13	6	Average	14.5
14	27	14	4		
15	10	15	12		
16	12	16	4		
17	12	17	24		
18	20	18	19		
19	20	19	36		
20	9	20	24		
21	20	21	24		
22	6	22	24		
23	8	23	16		
24	6	24	48		
25	6	25	21		
26	8	26	12		
27	8	27	8		
28	12	28	6		
29	6	29	18		
30	9	30	3		
31	6	Average	13.2		
32	26				
33	6				
Average	14.5				

In general, the near-surface findings of our soil borings correlate well with those described in the USDA Soil Survey, with the surficial muck types (Kaliga and Tequesta) reported to have thicknesses of 25 inches and 12 inches, respectively. Further, the explorations by RCS and AACE indicate that the surficial organic soils are thickest (2-3 feet) near the southeast/south portion of the site.

5.2 Measured Groundwater Level

The groundwater table depth as encountered in the borings during the field investigations is shown adjacent to the soil profiles on the attached Sheets No. 2-4. As can be seen, the groundwater table was generally encountered at depths ranging from about 1.5 feet to about 2.5 below the existing ground surface. Fluctuations in groundwater levels should be anticipated throughout the year primarily due to seasonal variations in rainfall, and other factors that may vary from the time the borings were conducted.

5.3 Estimated Normal Seasonal High Groundwater Table

Our field work was completed in March of 2016 which is typically considered the “dry season” in South Florida. The groundwater table will fluctuate seasonally, primarily based on rainfall. The normal seasonal high groundwater level is likely during the rainy season in Southeast Florida, typically between June and September of each year. The water table elevations associated with a 100-year flood level (or during an extreme storm event) would be much higher than the normal seasonal high water table elevation. The normal seasonal high groundwater table can also be influenced by the presence of relief points such as canals, lakes, ponds, swamps, etc., as well as by the drainage characteristics of the in-situ soils.

Based upon our field exploration, our observation of recovered soil samples and on review of the soil survey, we estimate that the normal seasonal high groundwater level at the boring locations is about 1-2 feet above the levels encountered in the borings, providing for potential flooded conditions. Further, temporary ponding of rainwater is likely to occur atop the surficial mantle of organics soils, particularly in the areas of the site where the organics are underlain by clayey soils.

The estimated normal seasonal high groundwater levels do not provide any assurance that the groundwater levels will not exceed these estimated levels during any given year in the future. Drainage impediments, storm events or other such occurrences may result in groundwater levels exceeding our estimates.

5.4 Piezometer Installations and Field Permeability Tests

Following the completion of the SPT borings, and after an initial engineering field review of the boring logs and recovered soil samples, a total of six (6) piezometers were installed, with 2 groups of 3 piezometers installed at the locations of our SPT borings TB-5 and TB-9 (i.e. Well Nest East [WN-E] and Well Nest South [WN-S]). The piezometers were installed using the hollow-stem auger drilling method. The piezometers were constructed of 2-inch diameter schedule 40 PVC with 5-foot 0.020 slot screens at various depths (selected based on the encountered soil conditions). The piezometers were sand packed with 20/30 grade sand to a depth of 1 foot above their screened sections, after which they were cement-grouted for another 3 feet and then backfilled with accumulated soil cuttings. The depths of the piezometers and their screened intervals are summarized in Table 3 below.

Table 3 - Piezometer Information

WN-E (TB-5)				WN-S (TB-9)			
Piezometer ID	**Depth (ft-bls)**	**Screened Depth (ft-bls)**	**GWT Depth (ft-bls)**	**Piezometer ID**	**Depth (ft)**	**Screened Depth (ft-bls)**	**GWT Depth (ft-bls)**
PZ-E1	10	5-10	1.6	PZ-S1	9	4-9	1.1
PZ-E2	20	15-20	1.5	PZ-S2	19	14-19	1.1
PZ-E3	30	25-30	1.6	PZ-S3	28	23-28	1.2

Following the piezometer installations and 2-day stabilization period, field permeability tests were performed at the screened depths in the installed piezometers. The results of these tests are presented on Sheet No. 5 and summarized in Tables 4 and 5 below.

Table 4 - Field Permeability Test Results (WN-E / TB-5)

Screened Depth (ft-bls)	Constant Head Test		Variable Head Test	
	k_H (cm/s)	k_H (ft/day)	k_H (cm/s)	k_H (ft/day)
5-10	1.64×10^{-3}	4.6	1.50×10^{-3}	4.3
15-20	1.00×10^{-3}	2.8	8.68×10^{-4}	2.5
25-30	3.46×10^{-4}	1.0	2.85×10^{-4}	0.8

Table 5 - Field Permeability Test Results (WN-S / TB-9)

Screened Depth (ft-bls)	Constant Head Test		Variable Head Test	
	k_H (cm/s)	k_H (ft/day)	k_H (cm/s)	k_H (ft/day)
4-9	7.08×10^{-4}	2.0	7.26×10^{-4}	2.1
14-19	6.29×10^{-3}	17.8	5.56×10^{-3}	15.8
25-30	8.45×10^{-3}	23.9	7.78×10^{-3}	22.0

6.0 GEOTECHNICAL ENGINEERING EVALUATION

6.1 General

Based on the findings of our site exploration and laboratory testing program, our evaluation of subsurface soil conditions, and judgment based on our experience with similar projects, we conclude that the soils underlying this site are generally satisfactory to support the proposed impoundment berms and associated pumping and overflow features, provided that the encountered surficial mantle of organic soils are removed from within the construction areas. Further, for the most part, the surficial soils (aside from the organics) on the site preliminarily appear to be suitable as borrow materials for the construction of the earthen impoundment berms, with only conventional clearing operations prior to the start of the filling and compaction efforts.

We do not that SPT borings TB-10, TB-11 and T-12 encountered 5-7 feet of surficial silty fine sands (SM) (below the organic mantle soils) with fines content in excess of 20-30 percent; these soils may not be suitable for berm construction and, as such, the proposed borrow ditch excavations may not be preferred in this portion of the site.

Specific earthwork recommendations are provided in subsequent sections of this report.

6.2 Seepage Analyses

A seepage analysis was completed for two locations (each addressing two impoundment water levels) using the provided design cross-section of the proposed impoundment berm (see Figure No. 3). The two selected locations were near our SPT boring TB-5/Well Nest WN-E and near our SPT boring TB-9/Well Nest WN-S (see Figure No. 4).

The seepage analyses presented in this report were performed using the SEEP/W module of the GeoStudio 2012 software. SEEP/W is a two-dimensional finite element seepage modeling program used to model a wide range of geotechnical engineering scenarios, including slope stability and groundwater flow analyses for regional flow systems, infiltration, etc. Further, the SEEP/W module is directly integrated with the GeoStudio slope stability module SLOPE/W.

The seepage analyses was used to evaluate the following:

- Phreatic surface location (for use in subsequent slope stability analyses).
- Critical exit gradients and factors of safety against boiling/heaving (seepage ditch).
- Flow rates out of the selected cross-sections of the impoundment.

6.2.1 Selection of Engineering Properties

The engineering properties of the various soil layers in the seepage analyses are summarized in Tables 6 and 7 below. Also included in this table are input needed for slope stability analyses (refer to Section 6.3), which were selected based on laboratory testing and SPT N-values, published correlations, and our experience with similar soil types. The horizontal hydraulic conductivity values that were used for the seepage analyses were obtained from the field permeability tests. The soil anisotropy (i.e. ratio of vertical hydraulic conductivity to horizontal hydraulic conductivity) was selected based on our experience with similar projects and soil conditions. A limited sensitivity study was performed relative to the effect on the analysis results of varying the soil anisotropy as well as other parameters (see discussion in Section 6.4).

The two selected cross-sections consist of 3 and 4 soil strata representing the encountered strata within the analyzed depths. As noted, only minor variations in the input soil parameters were utilized, due to the fairly similar boring and field permeability test results. Nevertheless, the selected cross-sections were independently analyzed so as to provide a measure of conservatism with regards to the overall analyses of the impoundment. Although one boring is specifically identified with the selected cross-sections, comparisons were made between adjacent borings, and an overall interpretation of the encountered subsurface conditions was implemented in the selection of soil parameters.

Table 6: Engineering Properties
Cross-section by TB-5 (WN-E)

General Stratum	k_H		k Ratio (V/H)	γ_{sat} (pcf)	ϕ (degr.)	Cohesion (psf)
	ft/day	cm/sec				
Impoundment Berm (Composite/Compacted)	0.2	7.0×10^{-5}	0.33	118	35	0
Fine sands (SP) to slightly silty fine sands (SP-SM)	10	3.5×10^{-3}	0.50	112	30	0
Slightly clayey fine sands (SP-SC)	4.5	1.5×10^{-3}	0.50	120	33	0
Slightly silty fine sands (SP-SM)	2.5	8.8×10^{-4}	0.5	115	31	0
Slightly clayey fine sands (SP-SC)	1	3.5×10^{-4}	0.50	125	35	0

Table 7: Engineering Properties
Cross-section by TB-9 (WN-S)

General Stratum	k_H		k Ratio (V/H)	γ_{sat} (pcf)	ϕ (degr.)	Cohesion (psf)
	ft/day	cm/sec				
Impoundment Berm (Composite/Compacted)	0.2	7.0×10^{-5}	0.33	118	35	0
Slightly clayey fine sands (SP-SC)	2	7.0×10^{-4}	0.50	120	32	0
Fine sands (SP)	18	6.3×10^{-3}	0.50	112	31	0
Fine/medium sands (SP)	23	8.1×10^{-4}	0.50	115	33	0

6.2.2 Boundary Conditions

No regional groundwater modeling was performed by AACE for this project. Instead, the lower reaches of our borings typically form the base of the model. As discussed in the following, only limited seepage flow was observed in the lower layers and consequently, this is considered a reasonable approach in lieu of physically extending borings to lower geologic units which could possibly be considered confining units.

The SEEP/W model does not include precipitation and evapotranspiration effects; such effects are considered minimal as it relates to the overall purpose of this study.

6.2.3 Analyses Approach and Results

The SEEP/W analyses were run in steady-state mode using the parameters and boundary conditions described in the previous. The analyses were completed for both, the normal maximum storage pool (i.e. 33.7 ft-NAVD) and the design storm pool (34.2 ft-NAVD). Further, conservatively, the adjacent seepage collection ditches were assumed to be empty (see discussion in Section 6.4).

Individual SEEP/W finite element mesh and subsurface layering are presented on Sheets No. 6-9. Flow rates out of the analyzed cross-sections were evaluated using flux sections (i.e. computation of flow quantities across multiple mesh segments).

The exit seepage gradients into the seepage collection ditch were evaluated as part of the analyses. From the U.S. Army Corps of Engineers EM 110-2-1913, the critical seepage gradient (i_c) is defined "as the gradient required to cause boils or heaving (flotation) of the landside top stratum and is taken as the ratio of the submerged weight of soil comprising the top stratum and the unit weight of water". For seepage into a flat ditch bottom (i.e. the seepage ditch), only the vertical component of the critical seepage gradient is considered, and is equal to the buoyant unit weight of the soil divided by the unit weight of water.

$$i_{cv} = \frac{\gamma_b}{\gamma_w}$$

For seepage from the side slopes of the seepage ditch, both the vertical and horizontal component of the critical seepage gradient are considered. The critical horizontal seepage gradient can be expressed in terms of the critical vertical gradient as follows:

$$i_{ch} = i_{cv} \tan\phi' \left(\varphi' = effective\ friction\ angle\ of\ soil\right)$$

For cohesionless soil, critical vertical gradients typically vary between 0.8 and 1.1. Using a conservative total saturated unit weight of 112 pcf (TB-5) and 120 pcf (TB-9) for the natural soils at the two analyzed cross-sections the critical vertical gradients are approximately 0.79 and 0.92, respectively. The resulting critical horizontal exit gradients are approximately 0.45 and 0.57, respectively. These critical gradients were compared to the calculated gradients in the seepage model to determine the factor of safety against piping from the impoundment into the seepage ditches.

Based on our literature review, recommended factors of safety against piping range from 1.5 to 5. In general, higher factors of safety are recommended at the downstream toe and downstream seepage ditch for larger embankments with water storage heights greater than 8-10 feet. Higher factors of safety are also recommended when limited subsurface information is available for evaluating piping potential. For the analyzed scenarios and considering the amount of subsurface subsurface soil information there is available, it is our opinion that a minimum factor of safety of 2 is acceptable.

The results of our seepage analysis are presented on Sheets No. 6-9 and summarized below in Tables 8 and 9.

Table 8: Seepage Analyses Results
TB-5 Cross-Section

Reservoir Level (ft-NAVD)	Seepage Ditch Level (ft-NAVD)	Maximum Exit Gradients				Seepage Rate (ft^3/day/ft)
		Horizontal	FOS	Vertical	FOS	
33.7	24.7	0.10	4.6	0.15	5.3	15.8
34.2	24.7	0.10	4.6	0.20	4.0	16.6

Table 9: Seepage Analyses Results
TB-9 Cross-Section

Reservoir Level (ft-NAVD)	Seepage Ditch Level (ft-NAVD)	Maximum Exit Gradients				Seepage Rate (ft^3/day/ft)
		Horizontal	FOS	Vertical	FOS	
33.7	24.7	0.1	5.7	0.4	2.3	23.5
34.2	24.7	0.15	3.8	0.45	2.1	24.8

As can be seen, the factors of safety against piping from the impoundment and into the adjacent seepage collection ditch are in excess of 2 and do not appear to be a critical consideration for the current design.

Further, our analyses shows that between 15.8 and 24.8 ft^3/day per foot of berm of water is expected to be lost via seepage through/under the impoundment berm. For approximately 3.5 miles of berm alignment (18,500 linear feet) this corresponds to about 290,000 and 460,000 ft^3/day. Using the normal maximum storage height of 4 feet (i.e. elevation 32.7 ft-NAVD) on the approximately 400-acre reservoir, the total amount of stored water is approximately 69,700,000 ft^3. Hence, between 0.4 and 0.7 percent of the stored volume will conservatively be lost per day, with less loss to be expected should the stored water height be less than 4 feet.

6.3 Slope Stability Analyses

Stability analyses were performed for the design cross-section at the two previously selected locations. The analyses were performed using the SLOPE/W module of the GeoStudio 2012 software. While several stability methods are available for the SLOPE/W software, the Spencer method was selected for the analyses presented in this report.

6.3.1 Geometry and Soil Parameters

The sections and geometries analyzed are the same as those used for the seepage analyses described in Section 6.1. Pore pressures/phreatic lines from the SEEP/W analyses were automatically integrated into the SLOPE/W models.

The soil parameters used for input in SLOPE/W were presented in Tables 6 and 7. They are based on field and laboratory testing, and published correlations with SPT N-values. A moist unit weight of approximately 105 pcf was utilized for the soils above the phreatic surface, and the listed saturated unit weights were utilized for soils below the phreatic line.

6.3.2 Loading Conditions and Required Factors of Safety

The loading conditions and the minimum slope stability factors of safety required by the U.S. Army Corps of Engineers for each loading condition (from EM 1110-2-1902) are provided below.

Table 10: Loading Conditions and Minimum Required Factors of Safety

Condition No.	Condition Description	Min. Required Factor of Safety	Slope
2	Long-Term/Steady State This case represents the long-term condition. The condition assumes steady-state seepage through the embankment. The phreatic surface is developed for the normal storage pool elevation. Drained shear strengths related to effective stresses are used.	1.5	Downstream
3	Rapid-Drawdown This case represents the condition immediately after the reservoir is drawn down from the storage pool elevation. A phreatic surface is assumed to have been established throughout the embankment. The reservoir and flow-way water levels are assumed to drop quickly from the storage pool elevation to ground elevation. Since the embankment soils are considered free-draining, drained shear strengths related to effective stresses are used; however, the steady-state phreatic surface within the embankment is retained.	1.1-1.3	Upstream

Two additional conditions are mentioned in the EM 1110-2-1902: *Condition 1 - During Construction and End-Of-Construction* and *Condition 4 - Earthquake.* Neither of these loading conditions were considered applicable for this project and were therefore not modeled.

6.3.3 Analyses Results

Stability of the impoundment berm and the seepage collection ditches were analyzed for the steady state seepage and the rapid drawdown scenarios previously described. For the steady state scenario, water levels were the same as those utilized in the seepage analyses (i.e. 33.7 ft-NAVD and 34.2 ft-NAVD). In the case of rapid drawdown, the lowest elevation that the water level could drop to in the impoundment was assumed to be elevation 29.7 ft-NAVD, as opposed to the bottom of the deeper interior borrow ditch. This scenario is considered to be unlikely and thus, highly conservative. Rapid drawdown in the seepage collection ditches was not considered applicable because the water levels in the ditches are controlled, and are generally lower, than the surrounding ambient groundwater levels.

The results of the slope stability analyses are presented on Sheets No. 6-9 and are summarized in Tables 11-14 below.

Table 11: Slope Stability Analyses Results
TB-5 Cross-Section
Storage Pool @ 33.7 ft-NAVD

Steady-State FOS				**Rapid Drawdown FOS**
Impoundment Berm Downstream	Impoundment Berm Upstream	Seepage Ditch Downstream	Global Downstream	Impoundment Berm Upstream
2.2	2.8	1.6	2.3	1.5
Minimum Required FOS				
1.5	1.5	1.5	1.5	1.3

Table 12: Slope Stability Analyses Results
TB-5 Cross-Section
Storage Pool @ 34.2 ft-NAVD

Steady-State FOS				**Rapid Drawdown FOS**
Impoundment Berm Downstream	Impoundment Berm Upstream	Seepage Ditch Downstream	Global Downstream	Impoundment Berm Upstream
2.1	2.7	1.3	2.1	1.3
Minimum Required FOS				
1.5	1.5	1.5	1.5	1.3

Table 13: Slope Stability Analyses Results
TB-9 Cross-Section
Storage Pool @ 33.7 ft-NAVD

Steady-State FOS				**Rapid Drawdown FOS**
Impoundment Berm Downstream	Impoundment Berm Upstream	Seepage Ditch Downstream	Global Downstream	Impoundment Berm Upstream
2.5	3.0	1.8	2.1	1.7
Minimum Required FOS				
1.5	1.5	1.5	1.5	1.3

Table 14: Slope Stability Analyses Results
TB-9 Cross-Section
Storage Pool @ 34.2 ft-NAVD

Steady-State FOS				**Rapid Drawdown FOS**
Impoundment Berm Downstream	Impoundment Berm Upstream	Seepage Ditch Downstream	Global Downstream	Impoundment Berm Upstream
2.4	3.0	1.6	2.0	1.5
Minimum Required FOS				
1.5	1.5	1.5	1.5	1.3

In brief, the computed slope stability factors of safety for the impoundment berm and seepage ditch were satisfactory for all the scenarios listed in Tables 10-13. The only exception noted was for cross-section TB-5 (storage pool @ 34.2 ft-NAVD) where the steady-state FOS for the seepage ditch was computed as 1.3 and, hence, slightly less than the minimum required FOS of 1.5. It is our opinion that the minimum required factors of safety provided herein are likely more applicable to the actual impoundment berms than to the adjacent seepage collection ditch slopes. As such, we believe the computed FOS is acceptable, especially since any slope failure for this relatively shallow ditch likely will occur as “sloughing” of the banks, rather than a conventional slope failure. Finally, throughout our analysis, we have considered the encountered soils (and the proposed berm construction soils) as being cohesionless. In actuality, it is likely that these soils will exhibit some minor measure of shear strength (say, 50-100 psf) which significantly improves the overall slope stability (see Section 6.4).

6.4 Discussion of Analyzed Scenarios/Limited Parametric Study

6.4.1 Seepage Collection Ditch Levels

It is our opinion that the completed steady-state seepage analyses presented in the previous are somewhat conservative, with respect to the analyzed scenario where the impoundment is at its maximum storage pool (i.e. elevation 33.7 ft-NAVD) while the approximately 4.5-5 feet deep seepage collection ditch is empty. The groundwater (in March) was encountered in our borings at relatively shallow depths, say 1-2 feet below grade (i.e. near elevation 28-29 ft-NAVD), so it is likely that some water will be present in the drainage ditches. As such, a limited parametric study was completed relative to the seepage collection ditch water levels versus both, the seepage rates and the factor of safety against slope failure for the downstream impoundment berm slope (see Tables 15 and 16 below).

Table 15: Sensitivity Study - Seepage Ditch Water Levels
TB-5 Cross-Section
Storage Pool @ 33.7 ft-NAVD

Seepage Ditch Water Level (ft-NAVD)	**Seepage Flow Rate (ft^3/day/ft)**	**FOS Slope Failure (downstream berm)**
24.7 (empty)	15.8	2.2
27.2 (ambient water table)	11.8	2.0
29.7 (full)	7.9	1.8

Table 16: Sensitivity Study - Seepage Ditch Water Levels
TB-9 Cross-Section
Storage Pool @ 33.7 ft-NAVD

Seepage Ditch Water Level (ft-NAVD)	Seepage Flow Rate (ft^3/day/ft)	FOS Slope Failure (downstream berm)
24.7 (empty)	23.5	2.5
27.2 (ambient water table)	16.9	2.3
29.7 (full)	11.7	2.0

As can be seen, a significant reduction in the seepage flow rate is associated with an increased level of water in the seepage collection ditch. Conversely, the factor of safety against slope failure is reduced about 20 percent while still exceeding the minimum required factor of safety as previously described.

However, these analyzed scenarios indicate that the phreatic surface may potentially “daylight” near the toe of the downstream/outside berm slope, based on the provided design cross-section. Following a review of the cross-sections presented in the RCS 90% Design Plans for the project, the majority of the berm alignment will have 1-2 feet of fill placed along the outside berm slope so as to provide “positive” drainage way from the berm and towards the seepage collection ditch. This added fill will provide additional cover relative to any daylighting scenarios. Alternatively, an outside toe drain could be utilized to possibly capture the phreatic line, if needed. We remain available for discussions in this regard.

6.4.2 Berm Material Shear Strength

As discussed in the previous, throughout our analysis we have considered the encountered soils (and the proposed berm construction soils) as being cohesionless. Practically, it is likely that these soils will exhibit some measure of shear strength (say, 50-100 psf) which significantly improves the overall slope stability. To verify this, the cohesion value for the berm materials was varied as follows (see Tables 17 and 18 below).

Table 17: Sensitivity Study - Berm Shear Strength
TB-5 Cross-Section
Storage Pool @ 33.7 ft-NAVD / Seepage Ditch Level @ 29.7 (full)

Cohesion (psf)	FOS Slope Failure (downstream berm)
0	1.8
50	2.0
100	2.1

Table 18: Sensitivity Study - Berm Shear Strength
TB-9 Cross-Section
Storage Pool @ 33.7 ft-NAVD / Seepage Ditch Level @ 29.7 (full)

Cohesion (psf)	FOS Slope Failure (downstream berm)
0	2.0
50	2.1
100	2.2

Hence, when disregarding any shear strength of the compacted berm materials as the analyses presented herein, the factor of safety against slope failure is likely somewhat conservative.

6.4.3 Berm Material Permeability

Aside from the assumed permeability of the berm materials, all other utilized permeability rates were actual field values (see Section 5.4). Hence, the assumed permeability value of 0.2 ft/day for the berm materials was varied an order of magnitude up and down so as to determine the sensitivity of the model relative to the computed seepage flow rates. The results of this limited parametric study are summarized in Tables 19 and 20 below.

Table 19: Sensitivity Study - Berm Material Permeability
TB-5 Cross-Section
Storage Pool @ 33.7 ft-NAVD / Seepage Ditch Level @ 24.7 (empty)

Berm Material Permeability (ft/day)	Seepage Flow Rate (ft^3/day/ft)
0.02	18.1
0.2	15.8
2	15.4

Table 20: Sensitivity Study - Berm Material Permeability
TB-9 Cross-Section
Storage Pool @ 33.7 ft-NAVD / Seepage Ditch Level @ 24.7 (empty)

Berm Material Permeability (ft/day)	Seepage Flow Rate (ft^3/day/ft)
0.02	25.2
0.2	23.5
2	23.1

It is our opinion that the utilized value of 0.2 ft/day is conservative. The permeability of the compacted, somewhat clayey berm materials is likely to be less than 0.2 ft/day, however, as evident from these results only a minor variation in seepage flow rates is to be expected.

6.4.4 General

The SEEP/W model was subjected to a cursory sensitivity analysis with respect to the boundary conditions and the soil anisotropy. As such, the widths and depths of the analyzed models were increased incrementally, and the ratio of vertical to horizontal hydraulic conductivity (k) was varied from ⅓ to 1 for the soil layering. In brief, the results of these efforts showed a combined effect on the results of less then 10 percent which, in our opinion, is an acceptable variance for this type of analysis.

6.5 Berm Construction Considerations

Subsequent to clearing, grubbing and stripping of all surface vegetation, topsoil, and organic mantle soils along the impoundment berm alignment, the cleared areas should be proofrolled with a 6-8 ton vibratory roller that exerts a centrifugal linear load not less than 340 pounds per linear inch. Any soft, yielding soils detected should be excavated and replaced with clean, compacted backfill that conforms with the recommendations below. Sufficient passes should be made during the proofrolling operations to produce dry densities not less than 95 percent of the modified Proctor (ASTM D1557) maximum dry density of the compacted material to depths of 2 feet below the compacted surface.

As mentioned, the existing surficial soils to depths in excess of 30 feet consist generally of loose to moderately dense fine sands, clayey fine sands and more or less silty fine sands which are generally considered suitable for use as berm construction materials provided that they are placed and compacted as recommended. During the excavation of the interior borrow ditch, the materials which contain little or no fines should be mixed as much as possible with the more clayey and silty materials and then placed in individually compacted lifts of 12 inches with no special consideration to wait time between lifts.

To facilitate the densification process, we recommend that the fill material be relatively dry (moisture content near 10-15 percent) at the time of placement. Furthermore, the berm lifts should be graded so that rainfall would tend to run off their surface. Each lift should be allowed to dry as needed to approach the optimum moisture content of the material prior to compaction. For compaction of the berm materials we recommend using a sheepsfoot or similar type of non-smooth roller or a heavy smooth-wheeled vibratory roller. The fill berm materials should be placed in level lifts of 12 inches and individually compacted to a dry density not less than 95 percent of its modified Proctor (ASTM D1557) maximum value. The berm fill materials should be free of organics and other deleterious materials. As a general rule, the (mixed) fill materials should have between eight and fifteen percent by dry weight passing the U.S. No. 200 sieve, and no particle larger than 3 inches in diameter. Materials with less percentage of fines should be placed in the downstream side of the berms.

It is recommended to have dozers and/or disking equipment available to assist with drying, mixing, and mechanical manipulation of the excavated borrow ditch soils.

Based upon the soil boring information and the weight of the proposed impoundment berm, expect total settlements of one-half inch or less. Because of the nature of the subsurface soils, the majority of the settlements should occur during the berm construction; post-construction settlement should be minimal.

6.6 Pump Station and Emergency Overflow Structure

Based on our cursory review of the pump station and emergency overflow details, we understand that the excavation for the proposed structures will reach depths of about 10-15 feet with respect to the existing ground surface. Consequently, dewatering will be required to facilitate the excavation and the proper compaction of the bottom of the excavations. The dewatering system is likely to consist of one or more wellpoint arrays at the perimeter of the excavation, maintaining the groundwater level at least 2 feet below the bottom of the excavation.

We preliminary expect that an open excavation will be utilized to allow for the construction of the pump station and overflow structure. However, depending on the proximity of these proposed features to existing features, bracing or sheeting of the excavation slopes may be necessary. The pump station and overflow structures should be constructed so as to withstand groundwater buoyancy forces acting on their bottom and sides after the wellpoints are turned off. For design purposes we recommend that the groundwater table be considered to rise to at least 2 feet above the current ground surface.

No need for special digging equipment is anticipated to excavate the encountered loose to moderately dense granular materials. However, it may be necessary to overexcavate the proposed pump station and overflow structure excavations by 6-12 inches and then backfill with well-compacted washed gravel in order to facilitate creating a firm uniform bearing layer. The bearing layer should be compacted with a vibratory roller or heavy hand-led compaction equipment until a firm surface is produced. Materials compacted as recommended should withstand contact pressures of up to 2,500 pounds per square foot [psf]. The coefficient of subgrade reaction can conservatively be assumed to be 150 pounds per cubic inch for well-compacted gravel at the bottom of the excavations.

Based upon the boring information and assumed loading conditions, we estimate that the recommended allowable bearing stress will provide a minimum factor of safety in excess of four against bearing capacity failure. With the site prepared and the foundations designed and constructed as recommended, we anticipate total settlements of one-half inch or less. Because of the nature of the subsurface soils, the majority of the settlements should occur during construction; post-construction settlement should be minimal.

6.7 Piping Effects - Structures and Pipes

Piping of soils could be produced by the flow of water under the structures and pipes Hence, the structures and the pipes could form an artificial “roof” over the seepage path, so that an open channel would be maintained. Vertical barriers (i.e. anti-seep collars) would reduce the possibility of the development of this phenomenon. The effective seepage path length can be computed using the empirical coefficient called the weighted creep ratio, R_c which can be calculated using the following formula:

$$R_c = [\tfrac{1}{3}H + V] / h$$

Where: H is the length of the horizontal contacts ($\leq 45°$)

V is the length of the vertical contacts ($> 45°$)

h is the differential head across the structure.

A minimum weighted creep ratio of 4-5 is recommended for the assumed mixture of sands, silty and clayey materials. Should piping be of concern, poured-in-place concrete keyways, sheet pile walls, or flexible membranes are typically used to increase seepage path lengths, depending on the need and type of structure/pipe. We remain available for consultations in these matters, as needed.

6.8 Additional Considerations

During clearing of the proposed berm alignment, care should be taken to minimize the amount of disturbance of the soils. Consideration should be given to performing the clearing operations during the dry season.

Variations in soil profile along sections of the proposed berm alignment which remain unexplored may necessitate widening the berms for stability purposes, and/or constructing a limerock fill keyway in the berms so as to control seepage. Alternatively, a sand drain layer may have to be added over the outside toe of segments of the berms that exhibit excessive seepage once the impoundment site is filled with water. Such a sand layer would be about one foot thick, placed at a slope of 4 feet horizontal to 1 foot vertical.

To minimize erosion from rainwater runoff we recommend that the slopes of the finished berms be either sodded or otherwise covered by low-creeping vegetation.

6.9 Quality Control

We recommend establishing a comprehensive quality assurance program to verify that all site preparation, berm construction, excavation, bedding (pump station/overflow) and backfilling is conducted in accordance with the appropriate plans and specifications. Materials testing and inspection services should be provided AACE.

As a minimum, an on-site engineering technician should monitor all stripping and grubbing to verify that all deleterious materials have been removed and should observe the compaction rolling operation to ensure that the appropriate number of passes are applied to the berm foundation soils in addition to individual lifts. In-situ density tests should be conducted during backfilling activities and below all footings to verify that the required densities have been achieved. In-situ density values should be compared to laboratory Proctor moisture-density results for each of the different natural and fill soils encountered.

Careful observation of the borrow ditch and seepage collection ditch excavations should be performed continuously so as to evaluate the excavated materials and document any encountered buried debris (wood, stumps, etc). If any such debris or anomalies are encountered, the Geotechnical Engineer of Record (AACE) and/or the Design Engineer of Record (RCS) should be notified immediately for review, evaluation and commenting. It is recommended that the ditch excavations are periodically photo or video documented.

In South Florida, earthwork testing is typically performed on an on-call basis when the contractor has completed a portion of the work. The test result from a specific location is only representative of a larger area if the contractor has used consistent means and methods and the soils are practically uniform throughout. The frequency of testing can be increased and full-time construction inspection can be provided to account for variations. We recommend that the following minimum testing frequencies be utilized.

Natural ground under berm segments should be tested at 300-foot intervals. Berm fill material should be tested at a minimum frequency of one in-place density test for each 12-inch lift for each 200 lineal feet berm alignment. Additional tests should be performed in backfill around the pump station and the emergency overflow feature.

Representative samples of the various natural ground/fill soils should be obtained and transported to our laboratory for Proctor compaction tests. These tests will determine the maximum dry density and optimum moisture content for the materials tested and will be used in conjunction with the results of the In-place density tests to determine the degree of compaction achieved.

We recommend that AACE inspect the berm conditions and performance during construction, immediately after the filling of the impoundment, and then periodically thereafter.

7.0 CLOSURE

The geotechnical evaluation submitted herein is based on the data obtained from the soil borings presented on Sheets No. 2-4 and our understanding of the proposed minor above-ground impoundment project as previously described. Limitations and conditions to this report are presented in Appendix V.

This report does not reflect any variations which may occur adjacent to or between the borings. The nature and extent of the variations between the borings may not become evident until during construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations presented in this report after performing on-site observations during the construction period and noting the characteristics of the variations.

In the event any changes occur in the design, nature, or location of the proposed improvements, we should review the applicability of conclusions and recommendations contained in this report. We also recommend a general review of final design and specifications by our office to make sure that earthwork and foundation recommendations are properly interpreted and implemented in the design specifications. AACE should attend pre-bid and preconstruction meetings to ensure that the bidders/contractor understand the recommendations contained in this report.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices for the exclusive use of Royal Consulting Services, Inc. and the design team for the proposed subject project. No other warranty, expressed or implied, is made.

We are pleased to be of assistance to you on this phase of your project. When we may be of further service to you or should you have any questions, please contact us.

Sincerely,
ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
Certificate of Authorization No. 26794

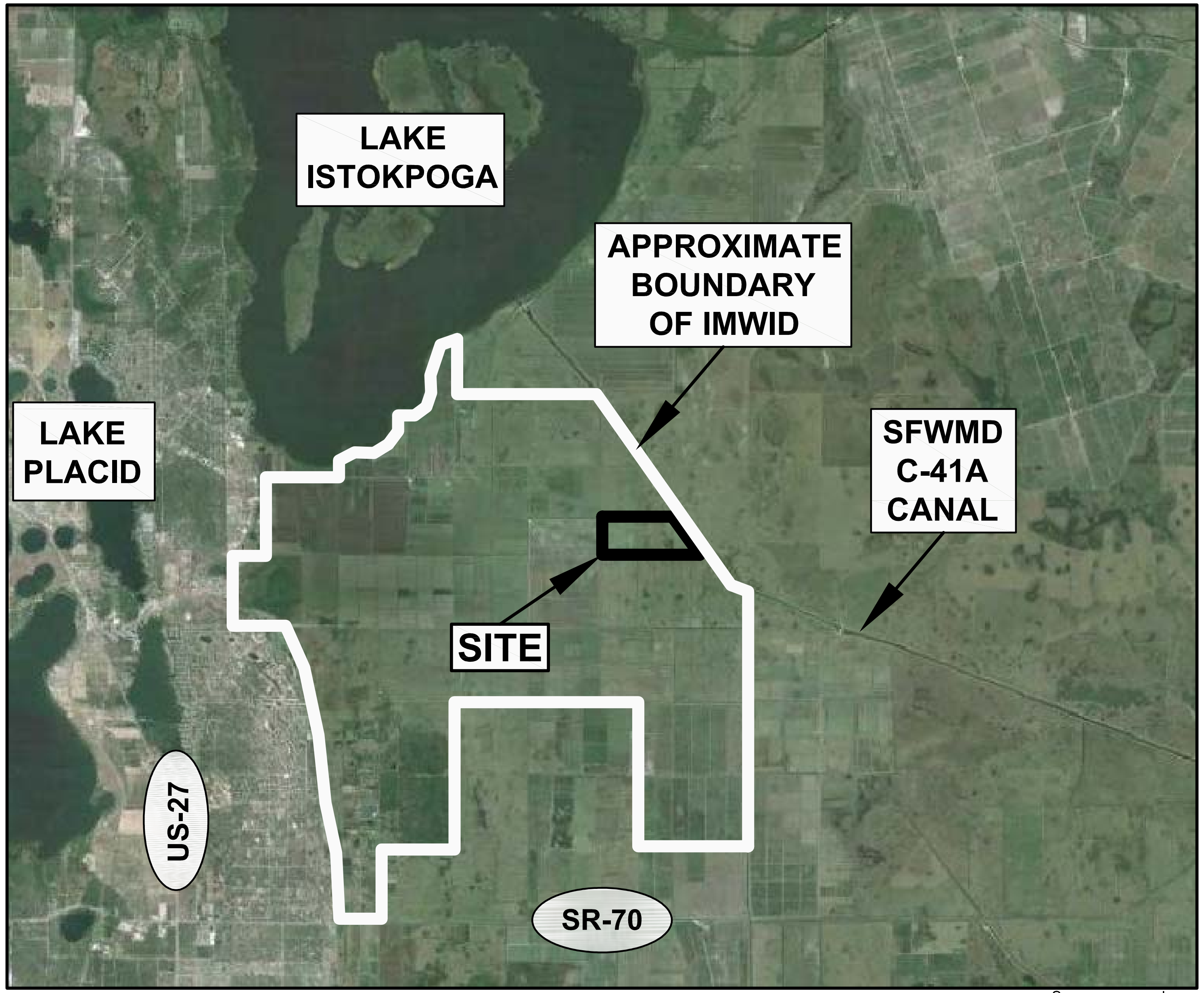
PETER G. ANDERSEN LICENSE No. 57956 STATE OF FLORIDA PROFESSIONAL ENGINEER

Peter G. Andersen, P.E.
Principal Engineer
Fla. Reg. No. 57956
1-12-2017

David P. Andre, P.E.
Principal Engineer
Fla. Reg. No. 53969
1/12/17

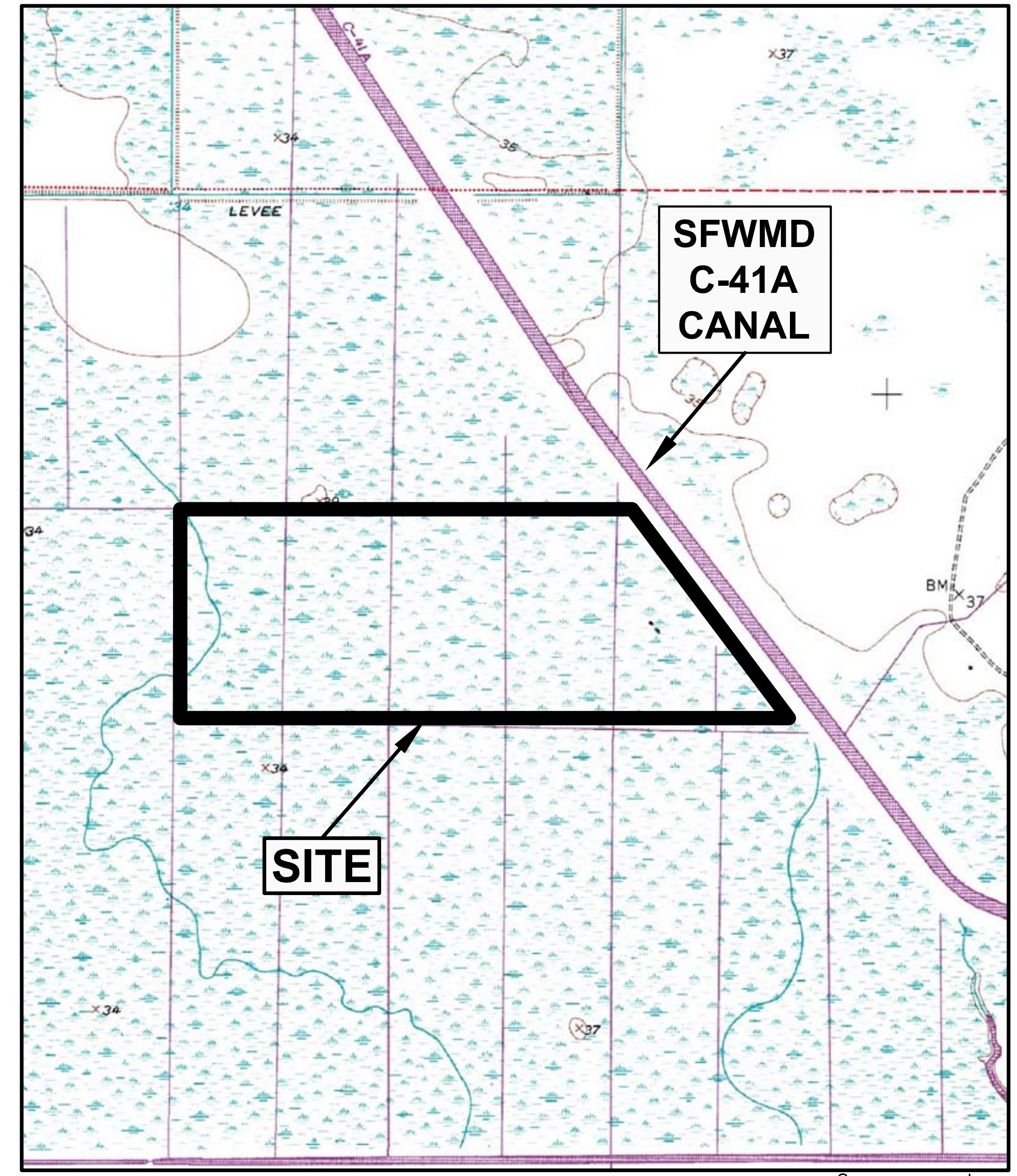
PGA/DPA:pa

2014 AERIAL PHOTOGRAPH



Source: mapcard.com

USGS TOPOGRAPHIC MAP
(1972 USGS Quadrangle Map of "Basinger SW, Florida")



Source: mapcard.com

NORTH
NOT TO SCALE

...within Sections 3, 4, 9 & 10
Township 37 South
Range 31 East

AACE **ANDERSEN ANDRE CONSULTING ENGINEERS, INC.** 834 SW Swan Avenue, Port St. Lucie, FL 34983 772-807-9191 www.AACEinc.com Certificate of Authorization No. 26794	**REGIONAL MAP AND USGS TOPOGRAPHIC MAP**	GEOTECHNICAL ENGINEERING EVALUATION ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID) PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT HIGHLANDS COUNTY, FLORIDA	Drawn by: PGA	Date: January 2016
			Checked by: DPA	Date: January 2016
			AACE File No: 16-112	**Figure No. 1**

Source: USDA NRCS Web Soil Survey

Summary of USDA Web Soil Survey

Map Unit Symbol	Map Unit	Landform	Parent Materials	Typical Profile (depths in inches)	Natural Drainage Class	K_{sat} (in/hr)[1]	Natural Depth to Water Table (in)	Depth (in) vs Permeability (in/hr)[2]
18	Kaliga muck, frequently flooded, 0-1 percent slopes	Depressions on flatwoods on marine terraces	Herbaceous organic material over loamy marine deposits	0-25: Muck 25-35: Fine sandy loam 35-60: Sandy clay loam 60-80+: Sandy clay loam	Very poorly drained	0.06 to 0.20	Approx. 0	0-39: 6.0-20 39-45: 0.6-6.0 45-68: <0.2 68-80: 2.0-20
26	Tequesta muck, frequently flooded, 0-1 percent slopes	Depressions on marine terraces	Herbaceous organic material over loamy marine deposits	0-12: Muck 12-44: Fine sand 44-72: Fine sandy loam 72-80+: Sand	Very poorly drained	0.6-6.0	Approx. 0	0-12: 6.0-20 12-32: 6.0-20 32-77: 0.2-0.6 77-80: 6.0-20

Notes:
(1) K_{sat} defined as "Capacity of the most limiting layer to transmit water"
(2) From USDA Soil Survey Manuscipt of Highlands County (1989)

NORTH

APPROXIMATE SCALE [11x17]: 1"=600'
0 300 600 1200
FEET

AACE

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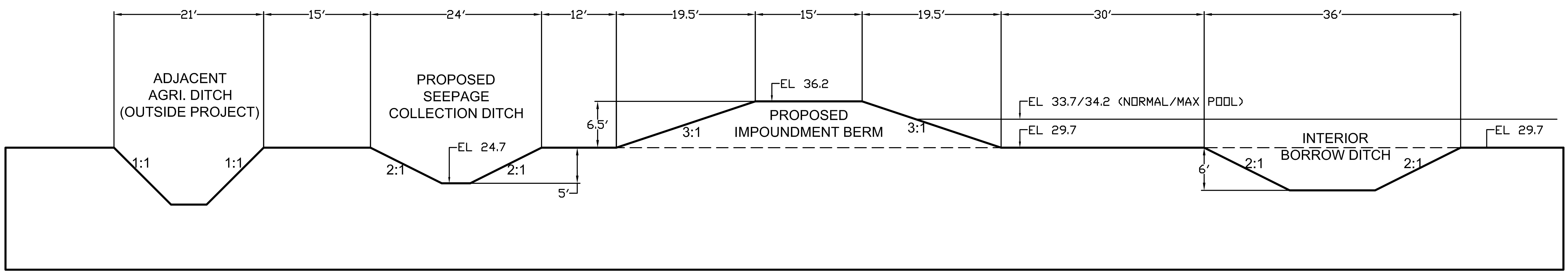
USDA SOIL SURVEY MAP

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

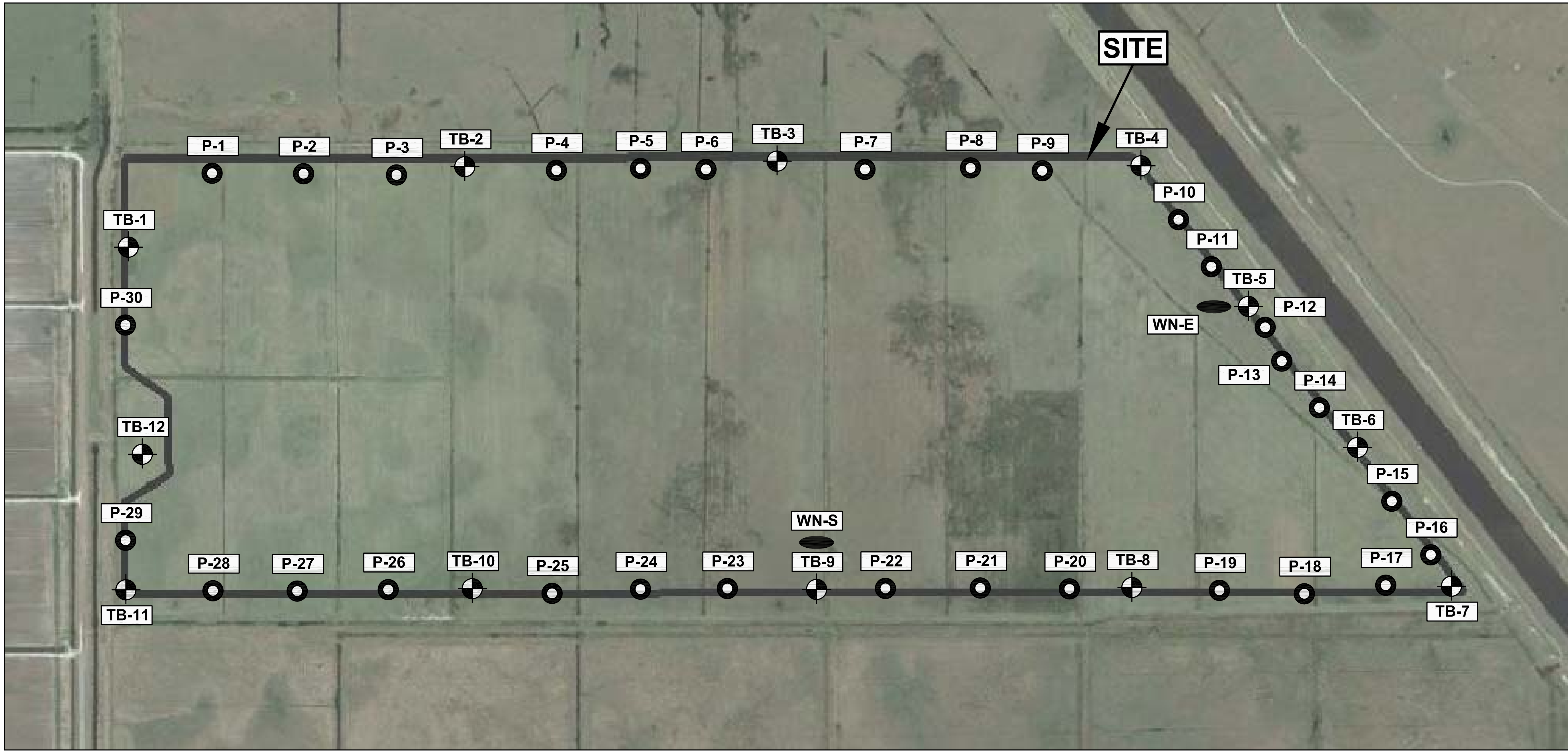
Drawn by: PGA | Date: January 2016
Checked by: DPA | Date: January 2016
AACE File No: 16-112

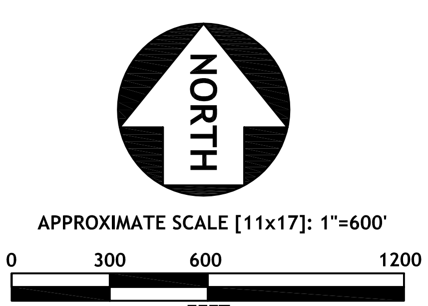
Figure No. 2



NOT TO SCALE
(FOR ILLUSTRATIVE PURPOSE ONLY)

ANDERSEN ANDRE CONSULTING ENGINEERS, INC. 834 SW Swan Avenue, Port St. Lucie, FL 34983 772-807-9191 www.AACEinc.com Certificate of Authorization No. 26794	**TYPICAL DESIGN CROSS-SECTION**	GEOTECHNICAL ENGINEERING EVALUATION ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID) PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT HIGHLANDS COUNTY, FLORIDA	Drawn by: PGA	Date: January 2016
			Checked by: DPA	Date: January 2016
			AACE File No: 16-112	**Figure No. 3**





LEGEND & NOTES

TB-#	Approximate Standard Penetration Test boring
P-#	Approximate Muck Probe Location
WN-#	Approximate Well/Piezometer Nest Location

Shown and noted field work locations are approximate. All field work locations were located using the provided site plan, obtained aerial photographs, existing site features, and a combination of a WAAS-enabled handheld GPS instrument and tape/wheel measurements. The shown field work locations should be considered accurate only to the degree implied by the method of measurement used.

Figure No. 4 Source: GoogleEarthPro and provided impoundment layout (CAD)

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FIELD WORK LOCATION PLAN

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	Figure No. 4

SOIL BORING, SAMPLING AND TESTING METHODS

(abbreviated version for project specific methods and soil conditions)

GENERAL

Andersen Andre Consulting Engineers, Inc. (AACE) borings describe subsurface conditions only at the locations drilled and at the time drilled. They provide no information about subsurface conditions below the bottom of the boreholes. At locations not explored, surface conditions that differ from those observed in the borings may exist and should be anticipated.

The information reported on our boring logs is based on our drillers' logs and on visual examination in our laboratory of disturbed soil samples recovered from the borings. The distinction shown on the logs between soil types is approximate only. The actual transition from one soil to another may be gradual and indistinct.

The groundwater depth shown on our boring logs is the water level the driller observed in the borehole when it was drilled. These water levels may have been influenced by the drilling procedures, especially in borings made by rotary drilling with bentonitic drilling mud. An accurate determination of groundwater level requires long-term observation of suitable monitoring wells. Fluctuations in groundwater levels throughout the year should be anticipated.

The absence of a groundwater level on certain logs indicates that no groundwater data is available. It does not mean that groundwater will not be encountered at that boring location at some other point in time.

HAND AUGER BORINGS

Hand auger borings are used, if soil conditions are favorable, when the soil strata are to be determined within a shallow (approximately 5-foot [1.5m]) depth or when access is not available to power drilling equipment. A 3-inch (75mm) diameter hand bucket auger with a cutting head is simultaneously turned and pressed into the ground. The bucket auger is retrieved at approximately 6-inch (0.15m) interval and its contents emptied for inspection. On occasion post-hole diggers are used, especially in the upper 3 feet (1m) or so. Penetrometer probings can be used in the upper 5 feet (1.5m) to determine the relative density of the soils. The soil sample obtained is described and representative samples put in bags or jars and transported to the AACE soils laboratory for classification and testing, if necessary.

STANDARD PENETRATION TEST

The Standard Penetration Test (SPT) is a widely accepted method of in situ testing of foundation soils (ASTM D-1586). A 2-foot (0.6m) long, 2-inch (50mm) O.D. split-barrell sampler attached to the end of a string of drilling rods is driven 24 inches (0.60m) into the ground by successive blows of a 140-pound (63.5 Kg) hammer freely dropping 30 inches (0.76m). The number of blows needed for each 6 inches (0.15m) increments penetration is recorded. The sum of the blows required for penetration of the middle two 6-inch (0.15m) increments of penetration constitutes the test result of N-value. After the test, the sampler is extracted from the ground and opened to allow visual description of the retained soil sample. The N-value has been empirically correlated with various soil properties allowing a conservative estimate of the behavior of soils under load. The following tables relate N-values to a qualitative description of soil density for cohesionless soils:

Cohesionless Soils:	N-Value	Description
	0 to 4	Very loose
	4 to 10	Loose
	10 to 30	Medium dense
	30 to 50	Dense
	Above 50	Very dense

Cohesive Soils:	N-Value	Description	Qu
	0 to 2	Very soft	Below 0.25 tsf (25 kPa)
	2 to 4	Soft	0.25 to 0.50 tsf (25 to 50 kPa)
	4 to 8	Medium stiff	0.50 to 1.0 tsf (50 to 100 kPa)
	8 to 15	Stiff	1.0 to 2.0 tsf (100 to 200 kPa)
	15 to 30	Very stiff	2.0 to 4.0 tsf (200 to 400 kPa)
	Above 30	Hard	Above 4.0 tsf (400 kPa)

The tests are usually performed at 5 foot (1.5m) intervals. However, more frequent or continuous testing is done by AACE through depths where a more accurate definition of the soils is required. The test holes are advanced to the test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine grains in suspension. The circulating fluid, which is bentonitic drilling mud, is also used to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly highly pervious ones, flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or prevent the loss of circulating fluid. After completion of a test borings, the hole is kept open until a steady state groundwater level is recorded. The hole is then sealed by backfilling, either with accumulated cuttings or lean cement.

Representative split-spoon samples from each sampling interval and from different strata are brought to our laboratory in air-tight jars for classification and testing, if necessary. Afterwards, the samples are discarded unless prior arrangement have been made.

SFWMD EXFILTRATION TESTS (USUAL CONDITION TEST)

In order to estimate the hydraulic conductivity of the upper soils, constant head or falling head exfiltration tests can be performed. These tests are performed in accordance with methods described in the South Florida Water Management District (SFWMD) Permit Information Manual, Volume IV. In brief, for the "Usual Condition Test", a 6 to 9 inch diameter hole is augered to depths of about 5 to 7 feet; the bottom one foot is filled with 57-stone; and a 6-foot long slotted PVC pipe is lowered into the hole. The distance from the groundwater table and to the ground surface is recordedand the hole is then saturated for 10 minutes with the water level maintained at the ground surface.

If a constant head test is performed, the rate of pumping will be recorded at fixed intervals of 1 minute for a total of 10 minutes, following the saturation period.

LABORATORY TEST METHODS

Soil samples returned to the AACE soils laboratory are visually observed by a geotechnical engineer or a trained technician to obtain more accurate description of the soil strata. Laboratory testing is performed on selected samples as deemed necessary to aid in soil classification and to help define engineering properties of the soils. The test results are presented on the soil boring logs at the depths at which the respective sample was recovered, except that grain size distributions or selected other test results may be presented on separate tables, figures or plates as discussed in this report. The soil descriptions shown on the logs are based upon visual-manual procedures in accordance with local practice. Soil classification is performed in general accordance with the United Soil Classification System (ASTM D-2487) and is also based on visual-manual procedures.

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHEAST FLORIDA

For use with the ASTM D-2487 Unified Soil Classification System

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

BOULDERS (>12" [300 MM]) and COBBLES (3" [75 MM] TO 12" [300 MM]):

GRAVEL: Coarse Gravel: 3/4" (19 mm) to 3" (75 mm)
Fine Gravel: No. 4 (4.75 mm) Sieve to 3/4" (19 mm)

Descriptive adjectives:

0 - 5%	- no mention of gravel in description
5 - 15%	- trace
15 - 29%	- some
30 - 49%	- gravelly (shell, limerock, cemented sands)

SANDS:

COARSE SAND: No. 10 (2 mm) Sieve to No. 4 (4.75 mm) Sieve
MEDIUM SAND: No. 40 (425 μm) Sieve to No. 10 (2 mm) Sieve
FINE SAND: No. 200 (75 μm) Sieve to No. 40 (425 μm) Sieve

Descriptive adjectives:

0 - 5%	- no mention of sand in description
5 - 15%	- trace
15 - 29%	- some
30 - 49%	- sandy

SILT/CLAY: < #200 (75μm) Sieve

SILTY OR SILT: PI < 4
SILTY CLAYEY OR SILTY CLAY: $4 \leq PI \leq 7$
CLAYEY OR CLAY: PI > 7

Descriptive adjectives:

< - 5%	- clean (no mention of silt or clay in description)
5 - 15%	- slightly
16 - 35%	- clayey, silty, or silty clayey
36 - 49%	- very

ORGANIC SOILS:

Organic Content	Descriptive Adjectives	Classification
0 - 2.5%	Usually no mention of org.	See Above
2.6 - 5%	slightly organic	add "with organic fines" to group name
5 - 30%	organic	SM with organic fines
		Organic Silt (OL)
		Organic Clay (OL)
		Organic Silt (OH)
		Organic Clay (OH)

NOTES:

TB-#	STANDARD PENETRATION TEST (SPT) BORING [ASTM D1586]
N	SPT RESISTANCE IN BLOWS PER FOOT
X.X' ▼	GROUNDWATER TABLE (GWT) MEASURED ON THE DATE DRILLED
N.E.	NOT ENCOUNTERED
EOB	END OF BORING
BLS	BELOW LAND SURFACE
SP, SP-SM, ETC:	UNIFIED SOIL CLASSIFICATION SYSTEM (USCS) USCS GROUPS DETERMINED BY VISUAL CLASSIFICATION, EXCEPT FOR NOTED LABORATORY TEST RESULTS.
MC	NATURAL MOISTURE CONTENT IN PERCENT [ASTM D2216]
OC	ORGANIC CONTENT IN PERCENT [ASTM D2974]
-200	PERCENT PASSING NO. 200 SIEVE SIZE (PERCENT FINES) [ASTM D1140]

SPT BORING DATA
DRILL CREW: CL/DTH/AACE
DRILL RIGS: CME-45/MOBILE B-57
DRILL METHOD: ROTARY-WASH W. BENTONITE DRILLING SLURRY
SPT DATA:
SPOON I.D. = 1.375"
SPOON O.D. = 2.0"
HAMMER DROP = 30"
HAMMER WEIGHT = 140 lbs.
HAMMER TYPE = MANUAL

SOIL LEGEND:

- ORGANICS/PEAT
- FINE SAND W. T/O SILT/CLAY (SP)
- SLIGHTLY SILTY FINE SAND (SP-SM)
- SILTY FINE SAND (SM)
- SLIGHTLY CLAYEY FINE SAND (SP-SC)
- CLAYEY FINE SAND (SC)

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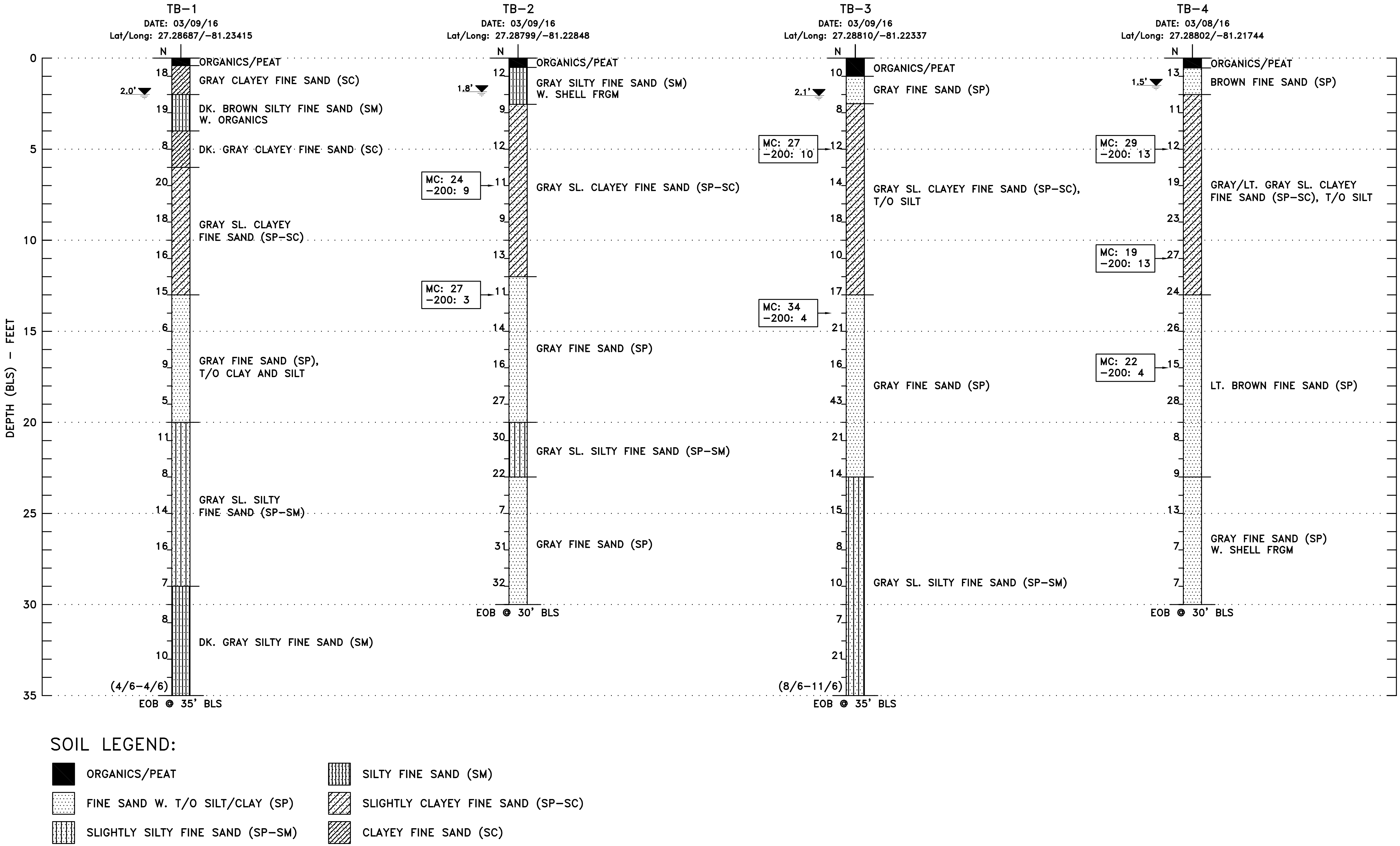
GENERAL NOTES

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 15-185	Sheet No. 1



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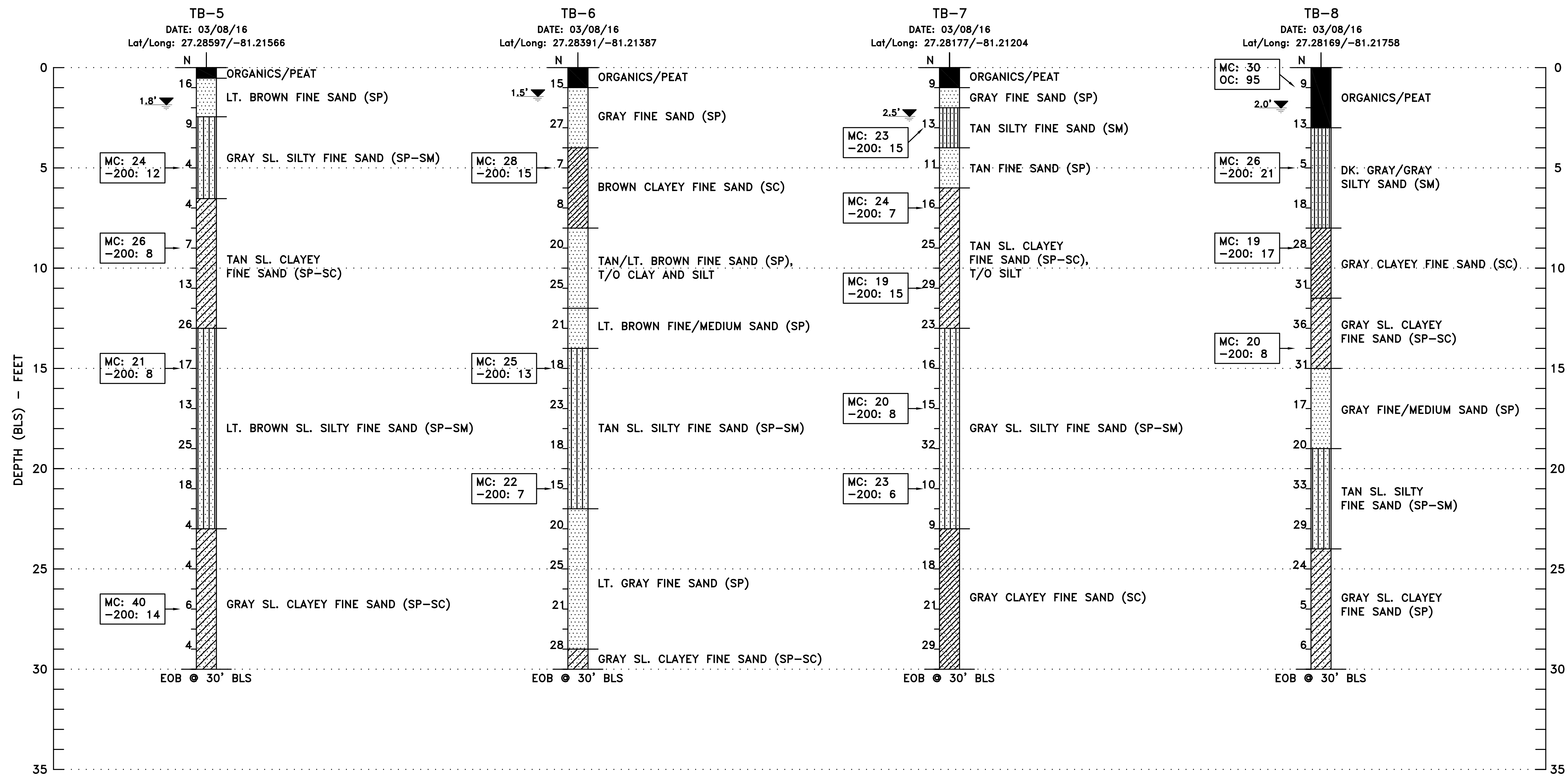
SOIL BORING PROFILES

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

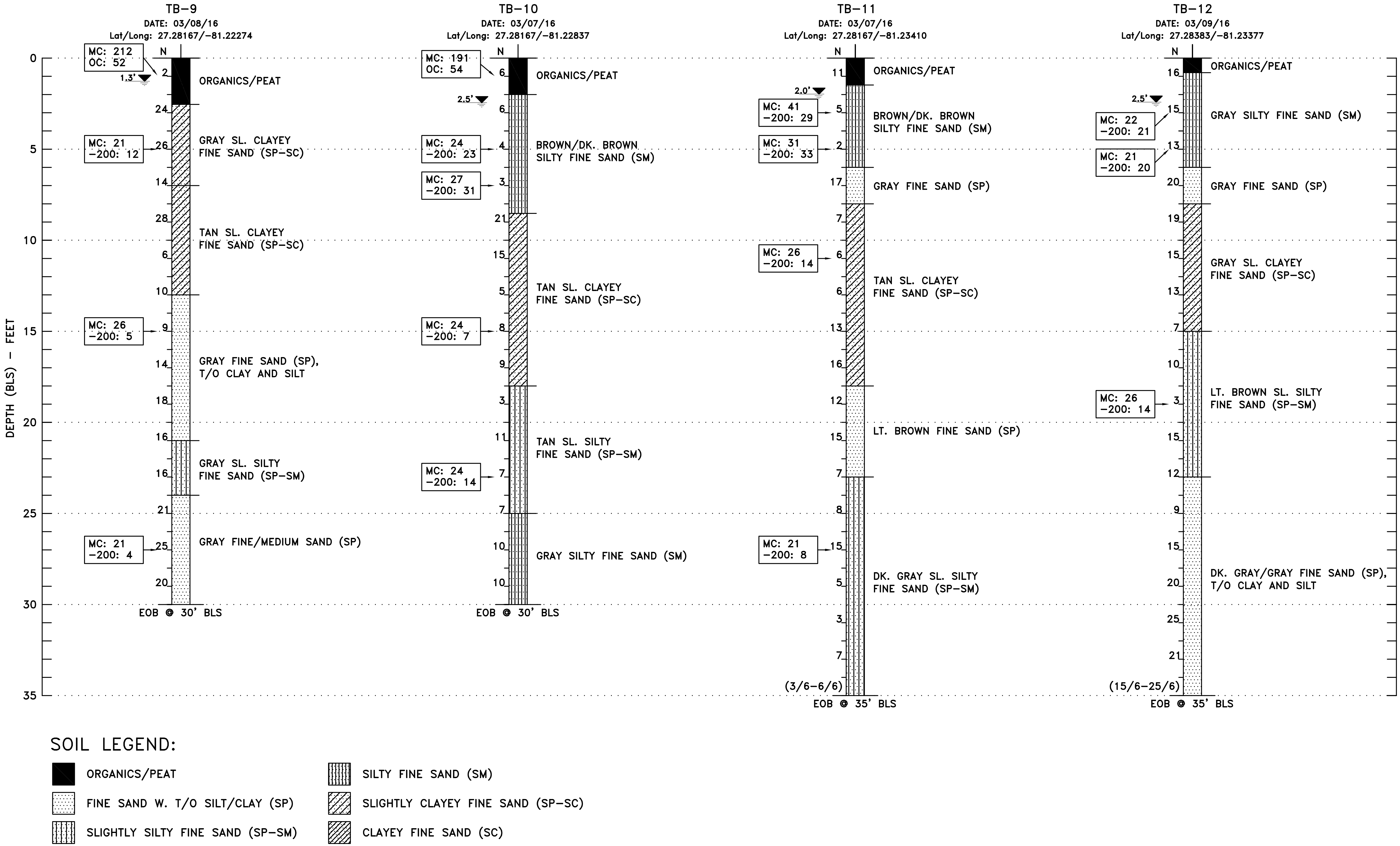
Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	**Sheet No. 2**



SOIL LEGEND:

- ORGANICS/PEAT
- FINE SAND W. T/O SILT/CLAY (SP)
- SLIGHTLY SILTY FINE SAND (SP-SM)
- SILTY FINE SAND (SM)
- SLIGHTLY CLAYEY FINE SAND (SP-SC)
- CLAYEY FINE SAND (SC)

AACE **ANDERSEN ANDRE CONSULTING ENGINEERS, INC.** 834 SW Swan Avenue, Port St. Lucie, FL 34983 772-807-9191 www.AACEinc.com Certificate of Authorization No. 26794	**SOIL BORING PROFILES**	GEOTECHNICAL ENGINEERING EVALUATION ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID) PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT HIGHLANDS COUNTY, FLORIDA	Drawn by: PGA Checked by: DPA AACE File No: 16-112	Date: January 2016 Date: January 2016 **Sheet No. 3**



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SOIL BORING PROFILES

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	**Sheet No. 4**

Test ID: PZ-1S
Test Depth: 4-9
No of Tests: 3 (average below)

Constant Head Permeability Test

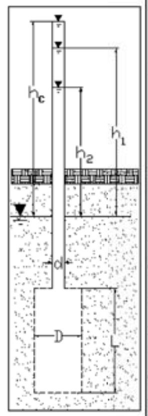
D: 8.00 in = 20.3 cm
q: 0.44 gpm = 27.4 cm^3/s
L: 5.00 ft = 152.4 cm
H_c: 3.60 ft = 109.7 cm
k_h/k_v : 1
m: 1

k_h : 7.08E-04 cm/s
2.01 ft/day

Variable Head Permeability Test
(24 inch drop)

d: 2.00 in = 5.1 cm
D: 8.00 in = 20.3 cm
L: 5.00 ft = 152.4 cm
h_1: 3.60 ft = 109.7 cm
h_2: 1.60 ft = 48.8 cm
t: 64.00 sec
k_h/k_v : 1
m: 1

k_h : 7.26E-04 cm/s
2.06 ft/day



Constant Head Permeability Test

$$k_h = \frac{q \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$$

Variable Head Permeability Test

$$k_h = \frac{d^2 \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} \le 4$$

$$k_h = \frac{d^2 \cdot \ln\left[\frac{2 \cdot m \cdot L}{D}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} > 4$$

Where:

D : intake diameter (cm)
d : standpipe diameter (cm)
m : transformation ratio $= \sqrt{\frac{k_h}{k_v}}$
L : intake length (cm)
t : elapsed time (sec)
q : water flow (cm^3 / sec)
h_c : constant piezometric head (cm)
h_1 : initial piezometric head (cm)
h_2 : final piezometric head (cm)
k_h : horizontal permeability (cm / sec)
k_v = vertical permeability (cm / sec)

Reference: Seepage, Drainage, and Flow Nets
Harry R. Cedergren, (1989)

Test ID: PZ-2S
Test Depth: 14-19
No of Tests: 3 (average below)

Constant Head Permeability Test

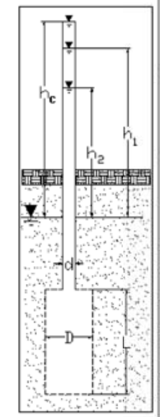
D: 8.00 in = 20.3 cm
q: 3.27 gpm = 206.3 cm^3/s
L: 5.00 ft = 152.4 cm
H_c: 3.05 ft = 93.0 cm
k_h/k_v : 1
m: 1

k_h : 6.29E-03 cm/s
17.82 ft/day

Variable Head Permeability Test
(24 inch drop)

d: 2.00 in = 5.1 cm
D: 8.00 in = 20.3 cm
L: 5.00 ft = 152.4 cm
h_1: 3.05 ft = 93.0 cm
h_2: 1.05 ft = 32.0 cm
t: 11.00 sec
k_h/k_v : 1
m: 1

k_h : 5.56E-03 cm/s
15.75 ft/day



Constant Head Permeability Test

$$k_h = \frac{q \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$$

Variable Head Permeability Test

$$k_h = \frac{d^2 \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} \le 4$$

$$k_h = \frac{d^2 \cdot \ln\left[\frac{2 \cdot m \cdot L}{D}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} > 4$$

Where:

D : intake diameter (cm)
d : standpipe diameter (cm)
m : transformation ratio $= \sqrt{\frac{k_h}{k_v}}$
L : intake length (cm)
t : elapsed time (sec)
q : water flow (cm^3 / sec)
h_c : constant piezometric head (cm)
h_1 : initial piezometric head (cm)
h_2 : final piezometric head (cm)
k_h : horizontal permeability (cm / sec)
k_v = vertical permeability (cm / sec)

Reference: Seepage, Drainage, and Flow Nets
Harry R. Cedergren, (1989)

Test ID: PZ-3S
Test Depth: 23-28
No of Tests: 3 (average below)

Constant Head Permeability Test

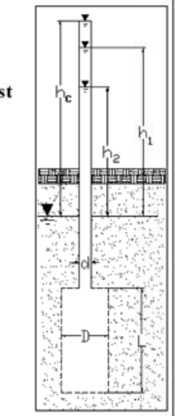
D: 8.00 in = 20.3 cm
q: 4.35 gpm = 274.4 cm^3/s
L: 5.00 ft = 152.4 cm
H_c: 3.02 ft = 92.0 cm
k_h/k_v : 1
m: 1

k_h : 8.45E-03 cm/s
23.94 ft/day

Variable Head Permeability Test
(24 inch drop)

d: 2.00 in = 5.1 cm
D: 8.00 in = 20.3 cm
L: 5.00 ft = 152.4 cm
h_1: 3.02 ft = 92.0 cm
h_2: 1.02 ft = 31.1 cm
t: 8.00 sec
k_h/k_v : 1
m: 1

k_h : 7.78E-03 cm/s
22.05 ft/day



Constant Head Permeability Test

$$k_h = \frac{q \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$$

Variable Head Permeability Test

$$k_h = \frac{d^2 \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} \le 4$$

$$k_h = \frac{d^2 \cdot \ln\left[\frac{2 \cdot m \cdot L}{D}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} > 4$$

Where:

D : intake diameter (cm)
d : standpipe diameter (cm)
m : transformation ratio $= \sqrt{\frac{k_h}{k_v}}$
L : intake length (cm)
t : elapsed time (sec)
q : water flow (cm^3 / sec)
h_c : constant piezometric head (cm)
h_1 : initial piezometric head (cm)
h_2 : final piezometric head (cm)
k_h : horizontal permeability (cm / sec)
k_v = vertical permeability (cm / sec)

Reference: Seepage, Drainage, and Flow Nets
Harry R. Cedergren, (1989)

Test ID: PZ-1E
Test Depth: 5-10
No of Tests: 3 (average below)

Constant Head Permeability Test

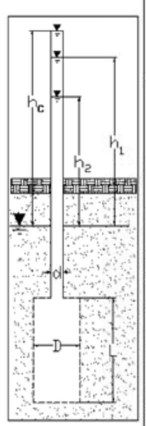
D: 8.00 in = 20.3 cm
q: 1.13 gpm = 71.3 cm^3/s
L: 5.00 ft = 152.4 cm
H_c: 4.05 ft = 123.4 cm
k_h/k_v : 1
m: 1

k_h : 1.64E-03 cm/s
4.64 ft/day

Variable Head Permeability Test
(24 inch drop)

d: 2.00 in = 5.1 cm
D: 8.00 in = 20.3 cm
L: 5.00 ft = 152.4 cm
h_1: 4.05 ft = 123.4 cm
h_2: 2.05 ft = 62.5 cm
t: 26.00 sec
k_h/k_v : 1
m: 1

k_h : 1.50E-03 cm/s
4.26 ft/day



Constant Head Permeability Test

$$k_h = \frac{q \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$$

Variable Head Permeability Test

$$k_h = \frac{d^2 \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} \le 4$$

$$k_h = \frac{d^2 \cdot \ln\left[\frac{2 \cdot m \cdot L}{D}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} > 4$$

Where:

D : intake diameter (cm)
d : standpipe diameter (cm)
m : transformation ratio $= \sqrt{\frac{k_h}{k_v}}$
L : intake length (cm)
t : elapsed time (sec)
q : water flow (cm^3 / sec)
h_c : constant piezometric head (cm)
h_1 : initial piezometric head (cm)
h_2 : final piezometric head (cm)
k_h : horizontal permeability (cm / sec)
k_v = vertical permeability (cm / sec)

Reference: Seepage, Drainage, and Flow Nets
Harry R. Cedergren, (1989)

Test ID: PZ-2E
Test Depth: 15-20
No of Tests: 3 (average below)

Constant Head Permeability Test

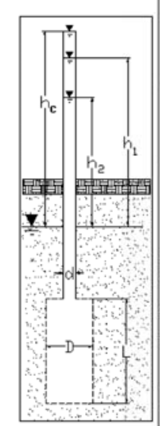
D: 8.00 in = 20.3 cm
q: 0.73 gpm = 45.8 cm^3/s
L: 5.00 ft = 152.4 cm
H_c: 4.25 ft = 129.5 cm
k_h/k_v : 1
m: 1

k_h : 1.00E-03 cm/s
2.84 ft/day

Variable Head Permeability Test
(24 inch drop)

d: 2.00 in = 5.1 cm
D: 8.00 in = 20.3 cm
L: 5.00 ft = 152.4 cm
h_1: 4.25 ft = 129.5 cm
h_2: 2.25 ft = 68.6 cm
t: 42.00 sec
k_h/k_v : 1
m: 1

k_h : 8.68E-04 cm/s
2.46 ft/day



Constant Head Permeability Test

$$k_h = \frac{q \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$$

Variable Head Permeability Test

$$k_h = \frac{d^2 \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} \le 4$$

$$k_h = \frac{d^2 \cdot \ln\left[\frac{2 \cdot m \cdot L}{D}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} > 4$$

Where:

D : intake diameter (cm)
d : standpipe diameter (cm)
m : transformation ratio $= \sqrt{\frac{k_h}{k_v}}$
L : intake length (cm)
t : elapsed time (sec)
q : water flow (cm^3 / sec)
h_c : constant piezometric head (cm)
h_1 : initial piezometric head (cm)
h_2 : final piezometric head (cm)
k_h : horizontal permeability (cm / sec)
k_v = vertical permeability (cm / sec)

Reference: Seepage, Drainage, and Flow Nets
Harry R. Cedergren, (1989)

Test ID: PZ-3E
Test Depth: 25-30
No of Tests: 3 (average below)

Constant Head Permeability Test

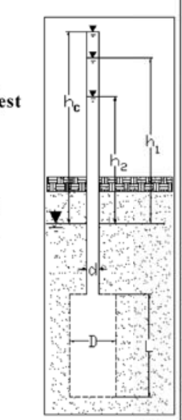
D: 8.00 in = 20.3 cm
q: 0.25 gpm = 16.0 cm^3/s
L: 5.00 ft = 152.4 cm
H_c: 4.30 ft = 131.1 cm
k_h/k_v : 1
m: 1

k_h : 3.46E-04 cm/s
0.98 ft/day

Variable Head Permeability Test
(24 inch drop)

d: 2.00 in = 5.1 cm
D: 8.00 in = 20.3 cm
L: 5.00 ft = 152.4 cm
h_1: 4.30 ft = 131.1 cm
h_2: 2.30 ft = 70.1 cm
t: 126.00 sec
k_h/k_v : 1
m: 1

k_h : 2.85E-04 cm/s
0.81 ft/day



Constant Head Permeability Test

$$k_h = \frac{q \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{2 \cdot \pi \cdot L \cdot h_c}$$

Variable Head Permeability Test

$$k_h = \frac{d^2 \cdot \ln\left[\frac{m \cdot L}{D} + \sqrt{1+\left(\frac{m \cdot L}{D}\right)^2}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} \le 4$$

$$k_h = \frac{d^2 \cdot \ln\left[\frac{2 \cdot m \cdot L}{D}\right]}{8 \cdot L \cdot t} \ln\left(\frac{h_1}{h_2}\right) \quad for \quad \frac{m \cdot L}{D} > 4$$

Where:

D : intake diameter (cm)
d : standpipe diameter (cm)
m : transformation ratio $= \sqrt{\frac{k_h}{k_v}}$
L : intake length (cm)
t : elapsed time (sec)
q : water flow (cm^3 / sec)
h_c : constant piezometric head (cm)
h_1 : initial piezometric head (cm)
h_2 : final piezometric head (cm)
k_h : horizontal permeability (cm / sec)
k_v = vertical permeability (cm / sec)

Reference: Seepage, Drainage, and Flow Nets
Harry R. Cedergren, (1989)

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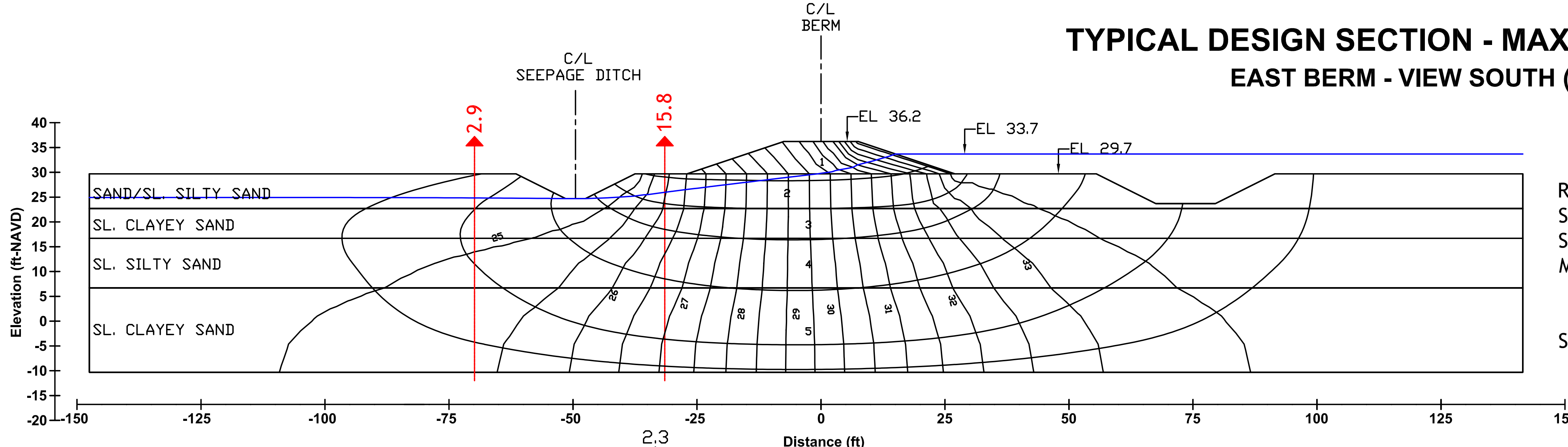
IN-SITU WELL PERMEABILITY TEST RESULTS

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

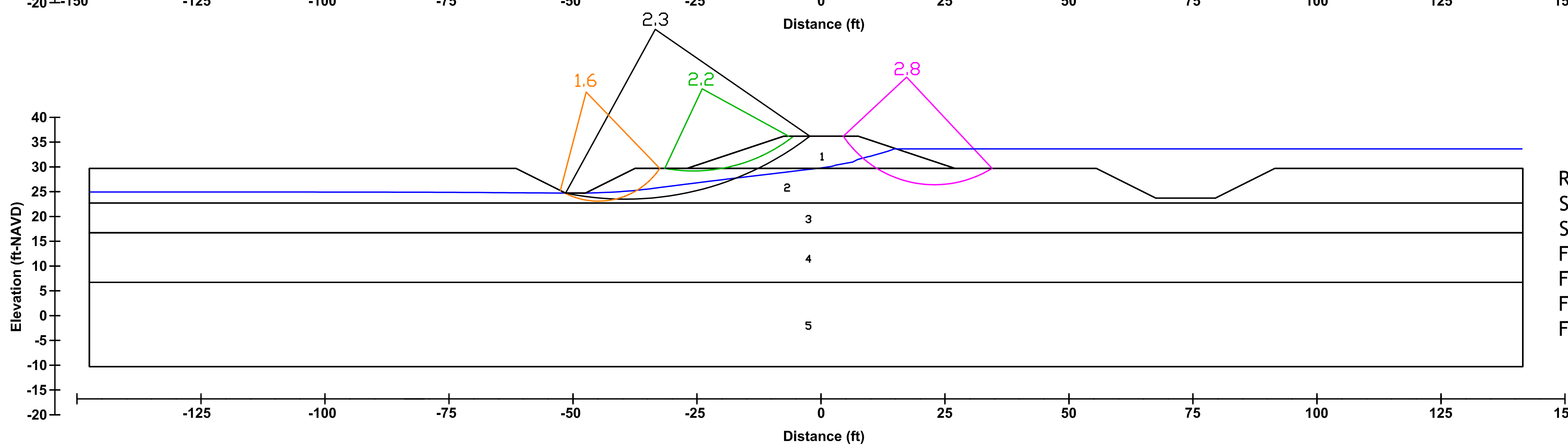
Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	Sheet No. 5

TYPICAL DESIGN SECTION - MAX. NORMAL STORAGE POOL

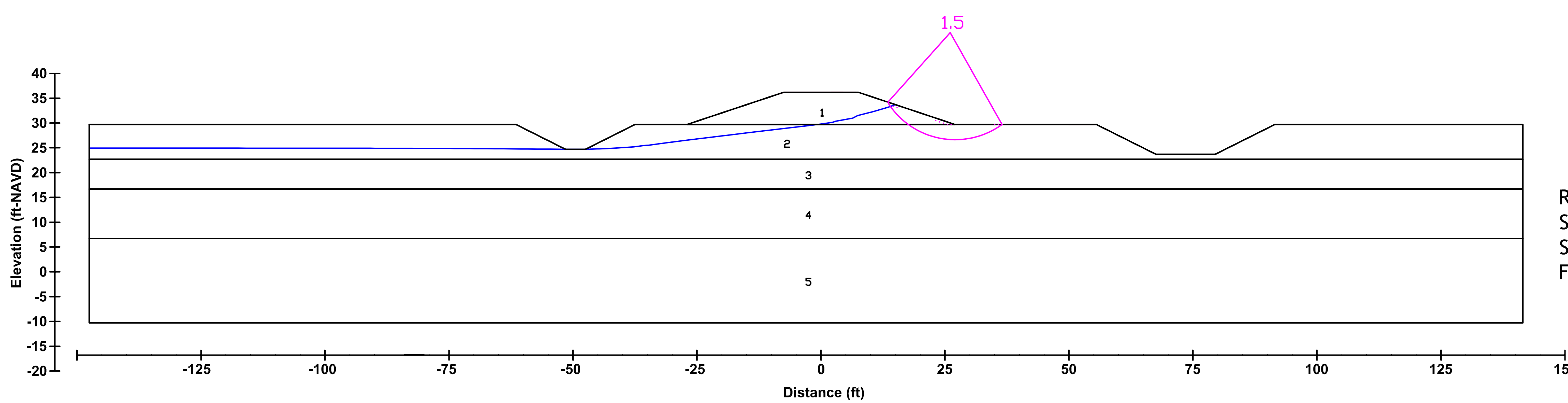
EAST BERM - VIEW SOUTH (NEAR BORING TB-5)



Reservoir Level: 33.7' NAVD (normal max. pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
Maximum Exit Gradients:
Horizontal: 0.10; FS = 4.6
Vertical: 0.15; FS = 5.3
Seepage Rate: 15.8 ft3/day/ft



Reservoir Level: 33.7' NAVD (normal max. pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
FS = 2.2 (Berm - Outside)
FS = 2.8 (Berm - Inside)
FS = 1.6 (Seepage Ditch)
FS = 2.3 (Global)



Reservoir Level: 29.7' NAVD (zero pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Rapid Drawdown (to Zero Pool)
FS = 1.5

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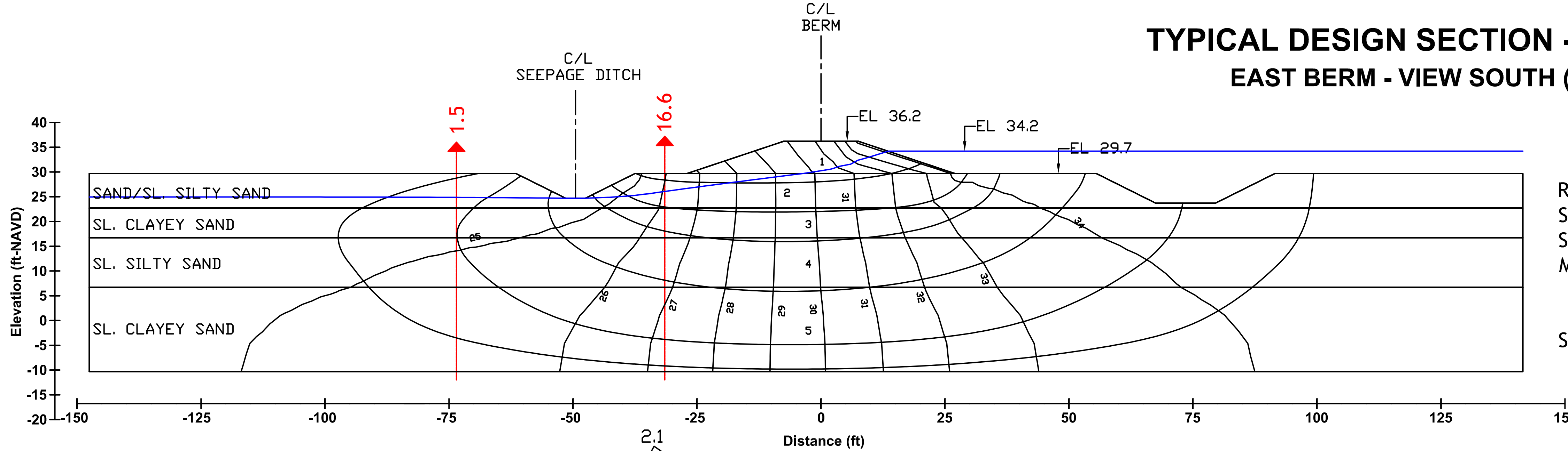
SEEPAGE AND SLOPE STABILITY ANALYSES RESULTS

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

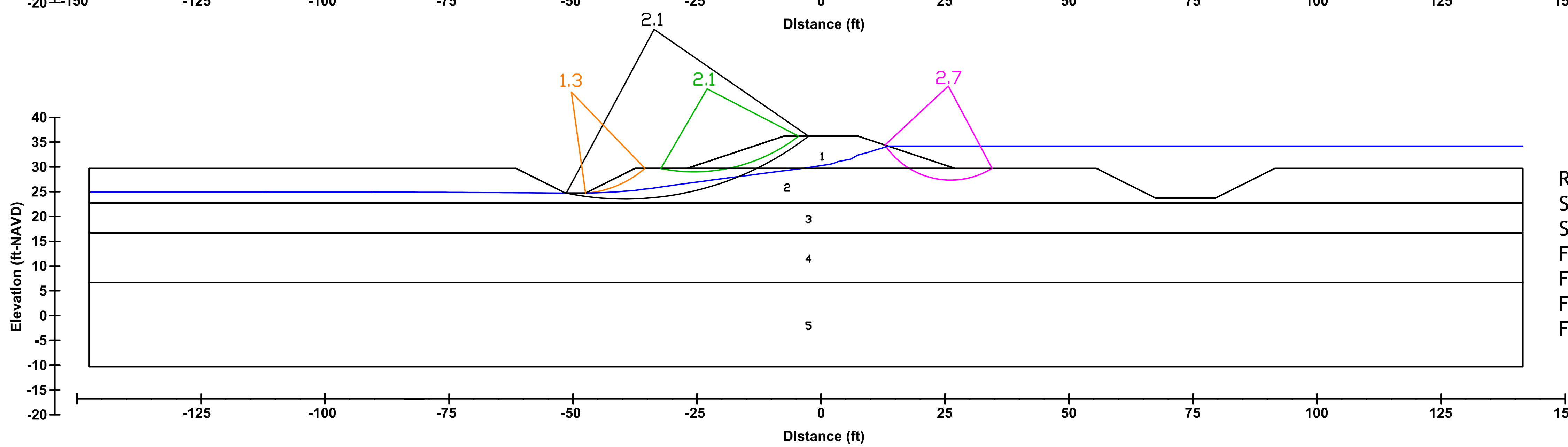
Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	Sheet No. 6

TYPICAL DESIGN SECTION - STORM EVENT POOL

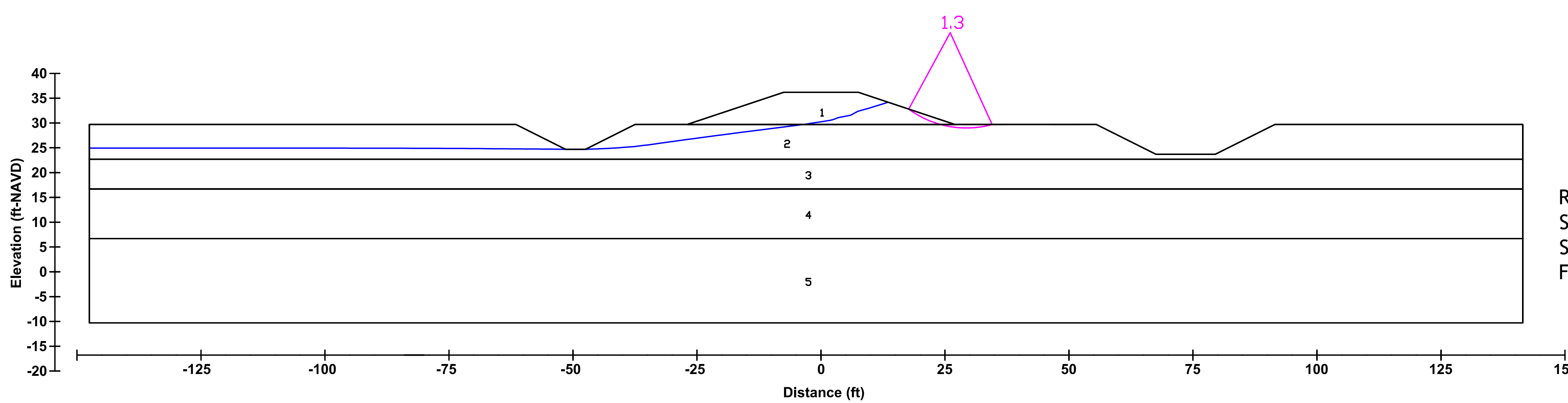
EAST BERM - VIEW SOUTH (NEAR BORING TB-5)



Reservoir Level: 34.2' NAVD (storm event pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
Maximum Exit Gradients:
Horizontal: 0.10; FS = 4.6
Vertical: 0.20; FS = 4.0
Seepage Rate: 16.6 ft3/day/ft



Reservoir Level: 34.2' NAVD (storm event pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
FS = 2.1 (Berm - Outside)
FS = 2.7 (Berm - Inside)
FS = 1.3 (Seepage Ditch)
FS = 2.1 (Global)



Reservoir Level: 29.7' NAVD (zero pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Rapid Drawdown (to Zero Pool)
FS = 1.3

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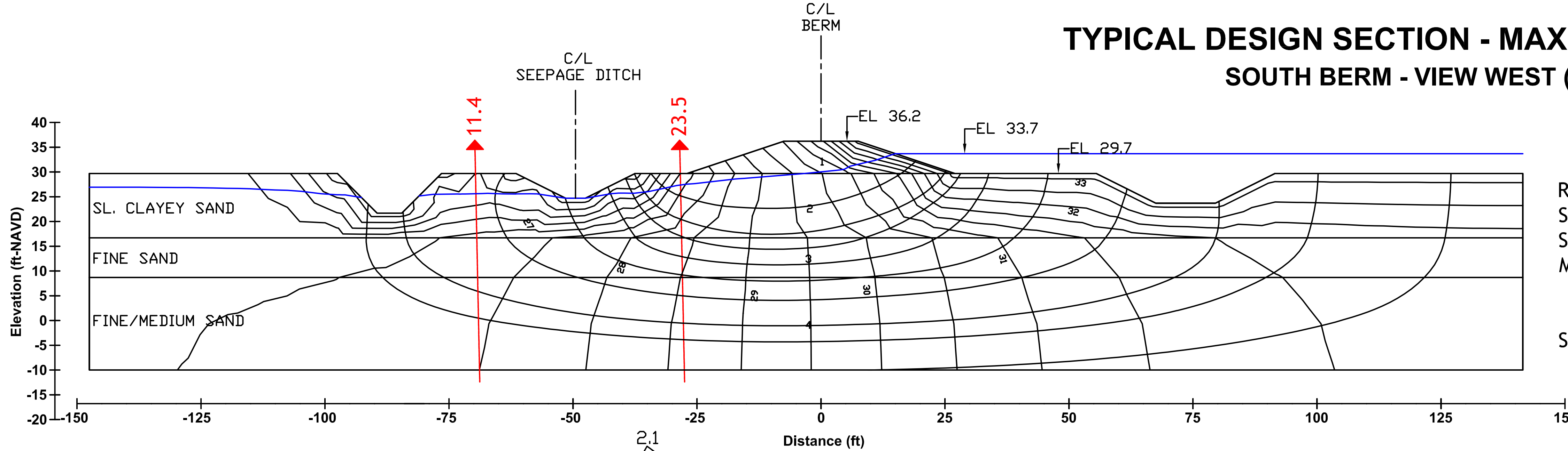
SEEPAGE AND SLOPE STABILITY ANALYSES RESULTS

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

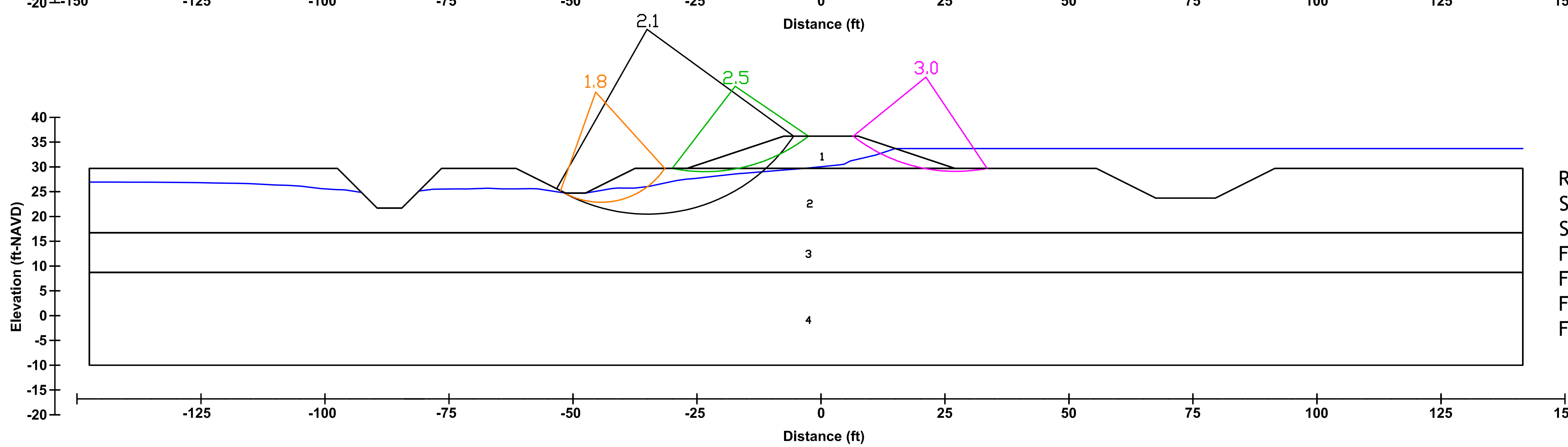
Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	Sheet No. 7

TYPICAL DESIGN SECTION - MAX. NORMAL STORAGE POOL

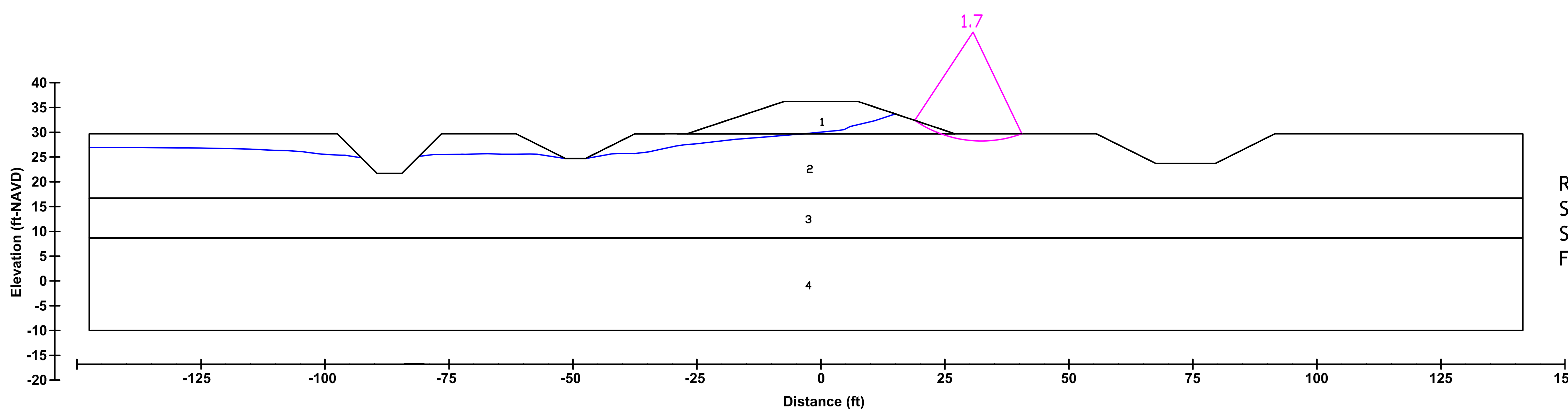
SOUTH BERM - VIEW WEST (NEAR BORING TB-9)



Reservoir Level: 33.7' NAVD (normal max. pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
Maximum Exit Gradients:
Horizontal: 0.10; FS = 5.7
Vertical: 0.40; FS = 2.3
Seepage Rate: 23.5 ft3/day/ft



Reservoir Level: 33.7' NAVD (normal max. pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
FS = 2.5 (Berm - Outside)
FS = 3.0 (Berm - Inside
FS = 1.8 (Seepage Ditch)
FS = 2.1 (Global)



Reservoir Level: 29.7' NAVD (zero pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Rapid Drawdown (to Zero Pool)
FS = 1.7

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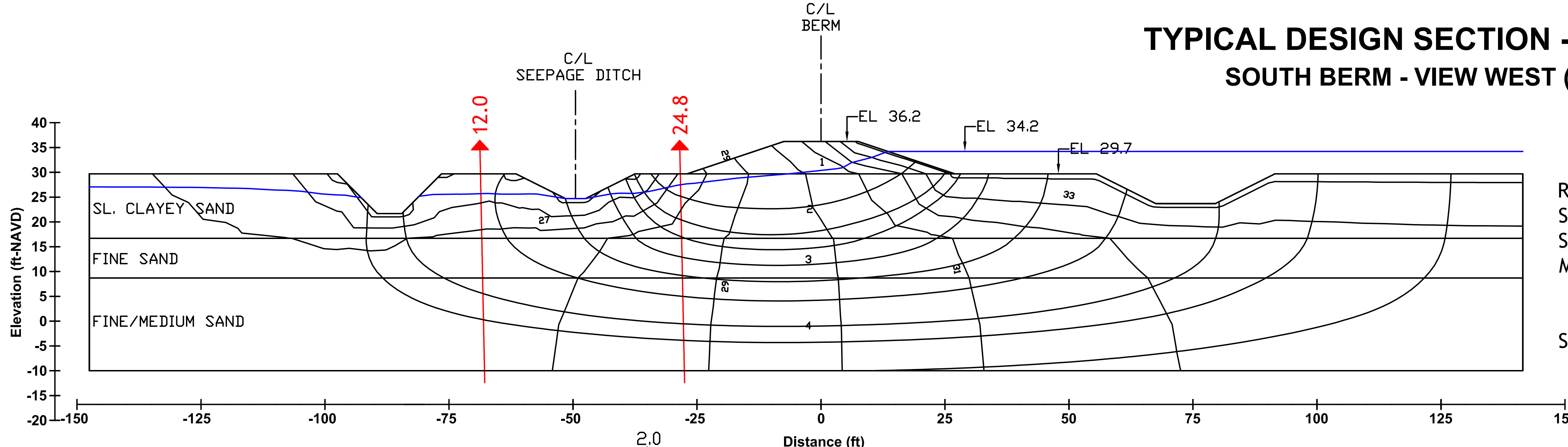
SEEPAGE AND SLOPE STABILITY ANALYSES RESULTS

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

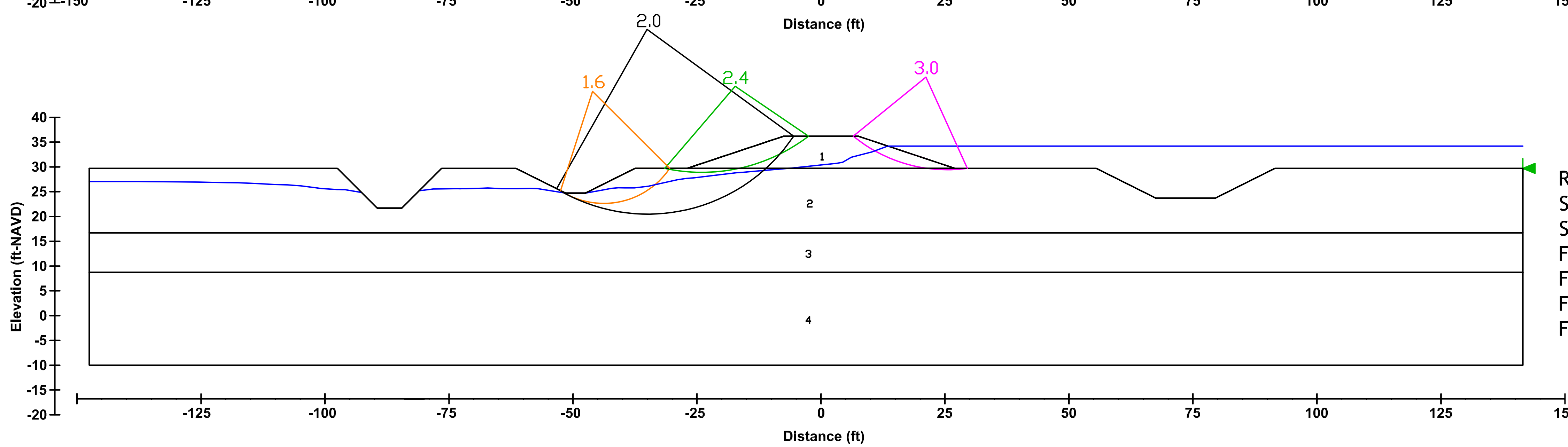
Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	Sheet No. 8

TYPICAL DESIGN SECTION - STORM EVENT POOL

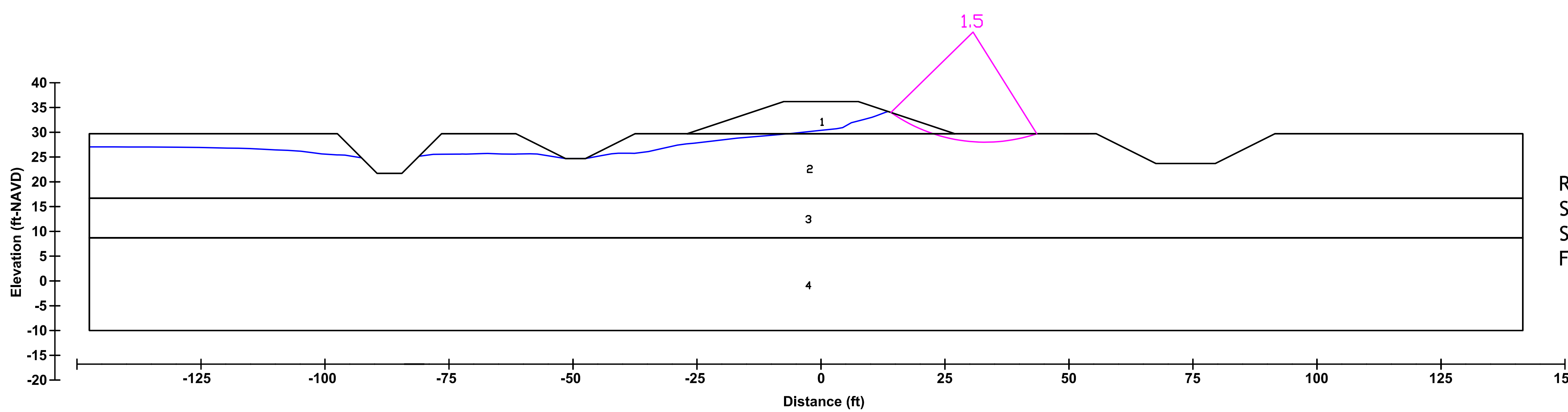
SOUTH BERM - VIEW WEST (NEAR BORING TB-9)



Reservoir Level: 34.2' NAVD (storm event pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
Maximum Exit Gradients:
Horizontal: 0.15; FS = 3.8
Vertical: 0.45; FS = 2.1
Seepage Rate: 24.8 ft3/day/ft



Reservoir Level: 34.2' NAVD (storm event pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Steady State
FS = 2.4 (Berm - Outside)
FS = 3.0 (Berm - Inside
FS = 1.6 (Seepage Ditch)
FS = 2.0 (Global)



Reservoir Level: 29.7' NAVD (zero pool)
Seepage Ditch Level: 24.7' NAVD (empty)
Scenario: Rapid Drawdown (to Zero Pool)
FS = 1.5

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SEEPAGE AND SLOPE STABILITY ANALYSES RESULTS

GEOTECHNICAL ENGINEERING EVALUATION
ISTOKPOGA MARSH WATER IMPROVEMENT DISTRICT (IMWID)
PROPOSED CELL NO. 2 MINOR ABOVE-GROUND IMPOUNDMENT
HIGHLANDS COUNTY, FLORIDA

Drawn by: PGA	Date: January 2016
Checked by: DPA	Date: January 2016
AACE File No: 16-112	Sheet No. 9

APPENDIX I

Site Photographs

Aerial Photograph of Site
(source: GoogleEarthPro)

Typical view of interior of site

Typical view of interior of site

Typical view of interior of site.
Roadway along south side.

Typical view of interior of site.
Drilling SPT borings.

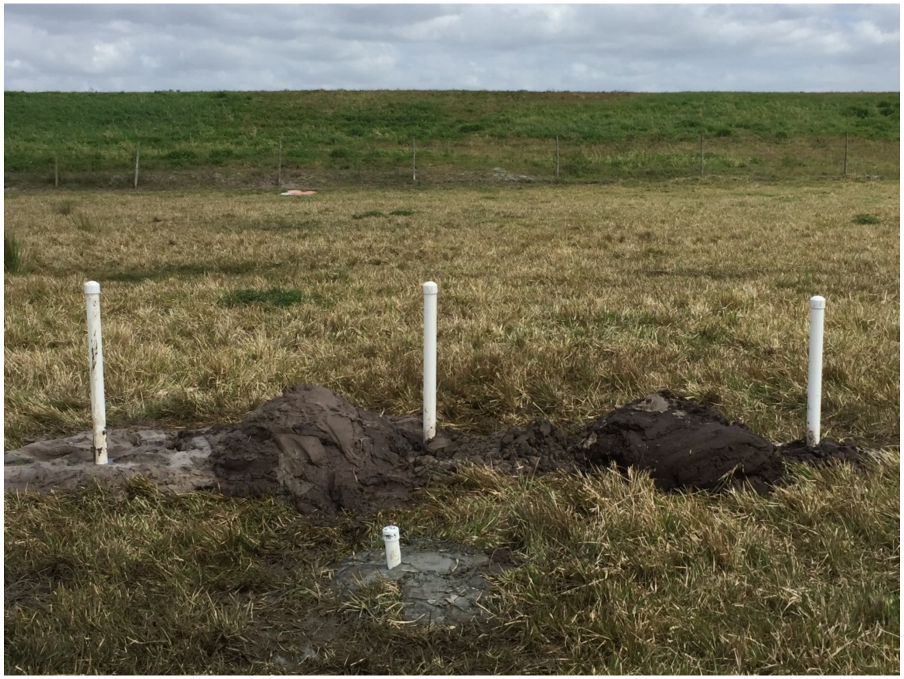
Typical view of interior of site.

Typical view of interior of site.
Standing water.

Geotechnical Engineering Evaluation IMWID Cell No. 2 Highlands County, Florida	ANDERSEN ANDRE CONSULTING ENGINEERS, INC. 772.807.9191 | WWW.AACEINC.COM	SITE PHOTOGRAPHS

Completed SPT boring (typ.), marked for later surveying.

Completed SPT boring (typ.), marked for later surveying.



Installed piezometers near
SPT boring TB-5

Installed piezometers near
SPT boring TB-9

Field permeability testing near SPT boring TB-5

Field permeability testing near SPT boring TB-9

Geotechnical Engineering Evaluation **IMWID Cell No. 2** **Highlands County, Florida**	**ANDERSEN ANDRE CONSULTING ENGINEERS, INC.** **772.807.9191 | WWW.AACEINC.COM**	**SITE PHOTOGRAPHS**

APPENDIX II

USDA Web Soil Survey Summary Report

Soil Map—Highlands County, Florida
(IMWID Cell No. 2)

Map Scale: 1:12,600 if printed on A landscape (11" x 8.5") sheet.

N

Meters: 0, 150, 300, 600, 900

Feet: 0, 500, 1000, 2000, 3000

Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 17N WGS84

MAP LEGEND

Area of Interest (AOI)

- Area of Interest (AOI)

Soils

- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

Special Point Features

- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

Water Features

- Streams and Canals

Transportation

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

Background

- Aerial Photography

MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:24,000.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Highlands County, Florida
Survey Area Data: Version 14, Nov 19, 2015

Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Dec 8, 2010—Mar 8, 2011

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

Map Unit Legend

Highlands County, Florida (FL055)			
Map Unit Symbol	**Map Unit Name**	**Acres in AOI**	**Percent of AOI**
18	Kaliga muck, frequently ponded, 0 to 1 percent slopes	271.4	71.0%
26	Tequesta muck, frequently ponded, 0 to 1 percent slopes	111.0	29.0%
Totals for Area of Interest		**382.5**	**100.0%**

Highlands County, Florida

18—Kaliga muck, frequently ponded, 0 to 1 percent slopes

Map Unit Setting

National map unit symbol: 2tzw6
Elevation: 0 to 130 feet
Mean annual precipitation: 44 to 55 inches
Mean annual air temperature: 70 to 77 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Farmland of unique importance

Map Unit Composition

Kaliga and similar soils: 80 percent
Minor components: 20 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Kaliga

Setting

Landform: Depressions on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Concave, linear
Across-slope shape: Concave, linear
Parent material: Herbaceous organic material over loamy marine deposits

Typical profile

Oa - 0 to 25 inches: muck
C1 - 25 to 35 inches: fine sandy loam
C2 - 35 to 60 inches: sandy clay loam
C3 - 60 to 80 inches: sandy clay loam

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Very poorly drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Moderately low to moderately high (0.06 to 0.20 in/hr)
Depth to water table: About 0 inches
Frequency of flooding: None
Frequency of ponding: Frequent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Very high (about 12.6 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7w

Hydrologic Soil Group: C/D
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL), Organic soils in depressions and on flood plains (G155XB645FL)

Minor Components

Samsula

Percent of map unit: 5 percent
Landform: Depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Concave
Across-slope shape: Concave
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL), Organic soils in depressions and on flood plains (G155XB645FL)

Tequesta

Percent of map unit: 4 percent
Landform: Depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Concave
Across-slope shape: Concave
Other vegetative classification: Freshwater Marshes and Ponds (R156BY010FL), Organic soils in depressions and on flood plains (G156AC645FL)

Felda

Percent of map unit: 4 percent
Landform: Depressions on marine terraces, flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Linear
Across-slope shape: Concave, linear
Ecological site: Slough (R155XY011FL)
Other vegetative classification: Slough (R155XY011FL), Sandy over loamy soils on flats of hydric or mesic lowlands (G155XB241FL)

Chobee

Percent of map unit: 3 percent
Landform: Depressions on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Concave, linear
Across-slope shape: Concave, linear
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL), Organic soils in depressions and on flood plains (G155XB645FL)

Placid

Percent of map unit: 3 percent
Landform: Depressions on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Concave, linear
Across-slope shape: Concave, linear

Other vegetative classification: Sandy soils on stream terraces, flood plains, or in depressions (G155XB145FL)

Nittaw

Percent of map unit: 1 percent
Landform: Depressions on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Concave, linear
Across-slope shape: Concave, linear
Other vegetative classification: Organic soils in depressions and on flood plains (G155XB645FL)

Data Source Information

Soil Survey Area: Highlands County, Florida
Survey Area Data: Version 14, Nov 19, 2015

Highlands County, Florida

26—Tequesta muck, frequently ponded, 0 to 1 percent slopes

Map Unit Setting

National map unit symbol: 2tzwx
Elevation: 0 to 100 feet
Mean annual precipitation: 47 to 61 inches
Mean annual air temperature: 68 to 77 degrees F
Frost-free period: 355 to 365 days
Farmland classification: Farmland of unique importance

Map Unit Composition

Tequesta and similar soils: 87 percent
Minor components: 13 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Tequesta

Setting

Landform: Depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Concave
Across-slope shape: Concave
Parent material: Herbaceous organic material over sandy and loamy marine deposits

Typical profile

Oa - 0 to 12 inches: muck
A - 12 to 25 inches: fine sand
Eg - 25 to 44 inches: fine sand
Btg/E - 44 to 56 inches: fine sandy loam
Btg - 56 to 72 inches: fine sandy loam
2C - 72 to 80 inches: sand

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Very poorly drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.60 to 5.95 in/hr)
Depth to water table: About 0 inches
Frequency of flooding: None
Frequency of ponding: Frequent
Calcium carbonate, maximum in profile: 4 percent
Salinity, maximum in profile: Nonsaline to very slightly saline (0.0 to 2.0 mmhos/cm)
Sodium adsorption ratio, maximum in profile: 4.0
Available water storage in profile: Moderate (about 8.0 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 7w
Hydrologic Soil Group: A/D
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL), Organic soils in depressions and on flood plains (G155XB645FL)

Minor Components

Basinger

Percent of map unit: 4 percent
Landform: Depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Concave, linear
Across-slope shape: Concave, linear
Other vegetative classification: Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)

Holopaw

Percent of map unit: 3 percent
Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Tread, talf
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Slough (R155XY011FL), Sandy soils on flats of mesic or hydric lowlands (G155XB141FL)

Sanibel

Percent of map unit: 3 percent
Landform: Depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Concave
Across-slope shape: Concave
Other vegetative classification: Organic soils in depressions and on flood plains (G156AC645FL)

Kaliga

Percent of map unit: 3 percent
Landform: Depressions on flatwoods on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Concave, linear
Across-slope shape: Concave, linear
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL), Organic soils in depressions and on flood plains (G155XB645FL)

Data Source Information

Soil Survey Area: Highlands County, Florida
Survey Area Data: Version 14, Nov 19, 2015

APPENDIX III

General Notes
(Soil Boring, Sampling and Testing Methods)

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
SOIL BORING, SAMPLING AND TESTING METHODS

GENERAL

Andersen Andre Consulting Engineers, Inc. (AACE) borings describe subsurface conditions only at the locations drilled and at the time drilled. They provide no information about subsurface conditions below the bottom of the boreholes. At locations not explored, surface conditions that differ from those observed in the borings may exist and should be anticipated.

The information reported on our boring logs is based on our drillers' logs and on visual examination in our laboratory of disturbed soil samples recovered from the borings. The distinction shown on the logs between soil types is approximate only. The actual transition from one soil to another may be gradual and indistinct.

The groundwater depth shown on our boring logs is the water level the driller observed in the borehole when it was drilled. These water levels may have been influenced by the drilling procedures, especially in borings made by rotary drilling with bentonitic drilling mud. An accurate determination of groundwater level requires long-term observation of suitable monitoring wells. Fluctuations in groundwater levels throughout the year should be anticipated.

The absence of a groundwater level on certain logs indicates that no groundwater data is available. It does not mean that groundwater will not be encountered at that boring location at some other point in time.

STANDARD PENETRATION TEST

The Standard Penetration Test (SPT) is a widely accepted method of in situ testing of foundation soils (ASTM D-1586). A 2-foot (0.6m) long, 2-inch (50mm) O.D. split-barrell sampler attached to the end of a string of drilling rods is driven 24 inches (0.60m) into the ground by successive blows of a 140-pound (63.5 Kg) hammer freely dropping 30 inches (0.76m). The number of blows needed for each 6 inches (0.15m) increments penetration is recorded. The sum of the blows required for penetration of the middle two 6-inch (0.15m) increments of penetration constitutes the test result of N-value. After the test, the sampler is extracted from the ground and opened to allow visual description of the retained soil sample. The N-value has been empirically correlated with various soil properties allowing a conservative estimate of the behavior of soils under load. The following tables relate N-values to a qualitative description of soil density and, for cohesive soils, an approximate unconfined compressive strength (Qu):

Cohesionless Soils:	N-Value	Description
	0 to 4	Very loose
	4 to 10	Loose
	10 to 30	Medium dense
	30 to 50	Dense
	Above 50	Very dense

Cohesive Soils:	N-Value	Description	Qu
	0 to 2	Very soft	Below 0.25 tsf (25 kPa)
	2 to 4	Soft	0.25 to 0.50 tsf (25 to 50 kPa)
	4 to 8	Medium stiff	0.50 to 1.0 tsf (50 to 100 kPa)
	8 to 15	Stiff	1.0 to 2.0 tsf (100 to 200 kPa)
	15 to 30	Very stiff	2.0 to 4.0 tsf (200 to 400 kPa)
	Above 30	Hard	Above 4.0 tsf (400 kPa)

The tests are usually performed at 5 foot (1.5m) intervals. However, more frequent or continuous testing is done by AACE through depths where a more accurate definition of the soils is required. The test holes are advanced to the test elevations by rotary drilling with a cutting bit, using circulating fluid to remove the cuttings and hold the fine grains in suspension. The circulating fluid, which is bentonitic drilling mud, is also used to keep the hole open below the water table by maintaining an excess hydrostatic pressure inside the hole. In some soil deposits, particularly highly pervious ones, flush-coupled casing must be driven to just above the testing depth to keep the hole open and/or prevent the loss of circulating fluid. After completion of a test borings, the hole is kept open until a steady state groundwater level is recorded. The hole is then sealed by backfilling, either with accumulated cuttings or lean cement.

Representative split-spoon samples from each sampling interval and from different strata are brought to our laboratory in air-tight jars for classification and testing, if necessary. Afterwards, the samples are discarded unless prior arrangement have been made.

POWER AUGER BORINGS

Auger borings (ASTM D-1452) are used when a relatively large, continuous sampling of soil strata close to the ground surface is desired. A 4-inch (100 mm) diameter, continuous flight, helical auger with a cutting head at its end is screwed into the ground in 5-foot (1.5m) sections. It is powered by the rotary drill rig. The sample is recovered by withdrawing the auger our of the ground without rotating it. The soil sample so obtained, is classified in the field and representative samples placed in bags or jars and returned to the AACE soils laboratory for classification and testing, if necessary.

HAND AUGER BORINGS

Hand auger borings are used, if soil conditions are favorable, when the soil strata are to be determined within a shallow (approximately 5-foot [1.5m]) depth or when access is not available to power drilling equipment. A 3-inch (75mm) diameter hand bucket auger with a cutting head is simultaneously turned and pressed into the ground. The bucket auger is retrieved at approximately 6-inch (0.15m) interval and its contents emptied for inspection. On occasion post-hole diggers are used, especially in the upper 3 feet (1m) or so. Penetrometer probings can be used in the upper 5 feet (1.5m) to determine the relative density of the soils. The soil sample obtained is described and representative samples put in bags or jars and transported to the AACE soils laboratory for classification and testing, if necessary.

UNDISTURBED SAMPLING

Undisturbed sampling (ASTM D-1587) implies the recovery of soil samples in a state as close to their natural condition as possible. Complete preservation of in situ conditions cannot be realized; however, with careful handling and proper sampling techniques, disturbance during sampling can be minimized for most geotechnical engineering purposes. Testing of undisturbed samples gives a more accurate estimate of in situ behavior than is possible with disturbed samples.

Normally, we obtain undisturbed samples by pushing a 2.875-inch (73 mm) I.D., thin wall seamless steel tube 24 inches (0.6 m) into the soil with a single stoke of a hydraulic ram. The sampler, which is a Shelby tube, is 30 (0.8 m) inches long. After the sampler is retrieved, the ends are sealed in the field and it is transported to our laboratory for visual description and testing, as needed.

ROCK CORING

In case rock strata is encountered and rock strength/continuity/composition information is needed for foundation or mining purposes, the rock can be cored (ASTM D-2113) and 2-inch to 4-inch diameter rock core samples be obtained for further laboratory analyses. The rock coring is performed through flush-joint steel casing temporarily installed through the overburden soils above the rock formation and also installed into the rock. The double- or triple-tube core barrels are advanced into the rock typically in 5-foot intervals and then retrieved to the surface. The barrel is then opened so that the core sample can be extruded. Preliminary field measurements of the recovered rock cores include percent recovery and Rock Quality Designation (RQD) values. The rock cores are placed in secure core boxes and then transported to our laboratory for further inspection and testing, as needed.

SFWMD EXFILTRATION TESTS

In order to estimate the hydraulic conductivity of the upper soils, constant head or falling head exfiltration tests can be performed. These tests are performed in accordance with methods described in the South Florida Water Management District (SFWMD) Permit Information Manual, Volume IV. In brief, a 6 to 9 inch diameter hole is augered to depths of about 5 to 7 feet; the bottom one foot is filled with 57-stone; and a 6-foot long slotted PVC pipe is lowered into the hole. The distance from the groundwater table and to the ground surface is recordedand the hole is then saturated for 10 minutes with the water level maintained at the ground surface.

If a constant head test is performed, the rate of pumping will be recorded at fixed intervals of 1 minute for a total of 10 minutes, following the saturation period.

LABORATORY TEST METHODS

Soil samples returned to the AACE soils laboratory are visually observed by a geotechnical engineer or a trained technician to obtain more accurate description of the soil strata. Laboratory testing is performed on selected samples as deemed necessary to aid in soil classification and to help define engineering properties of the soils. The test results are presented on the soil boring logs at the depths at which the respective sample was recovered, except that grain size distributions or selected other test results may be presented on separate tables, figures or plates as discussed in this report.

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHEAST FLORIDA

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

The soil descriptions shown on the logs are based upon visual-manual procedures in accordance with local practice. Soil classification is performed in general accordance with the United Soil Classification System and is also based on visual-manual procedures.

BOULDERS (>12" [300 MM]) and COBBLES (3" [75 MM] TO 12" [300 MM]):

GRAVEL:

Coarse Gravel: 3/4" (19 mm) to 3" (75 mm)

Fine Gravel: No. 4 (4.75 mm) Sieve to 3/4" (19 mm)

Descriptive adjectives:

0 - 5%	– no mention of gravel in description
5 - 15%	– trace
15 - 29%	– some
30 - 49%	– gravelly (shell, limerock, cemented sands)

SANDS:

COARSE SAND: No. 10 (2 mm) Sieve to No. 4 (4.75 mm) Sieve

MEDIUM SAND: No. 40 (425 µm) Sieve to No. 10 (2 mm) Sieve

FINE SAND: No. 200 (75 µm) Sieve to No. 40 (425 µm) Sieve

Descriptive adjectives:

0 - 5%	– no mention of sand in description
5 - 15%	– trace
15 - 29%	– some
30 - 49%	– sandy

SILT/CLAY: < #200 (75µM) Sieve

SILTY OR SILT: PI $<$ 4

SILTY CLAYEY OR SILTY CLAY: 4 $\leq$ PI $\leq$ 7

CLAYEY OR CLAY: PI $>$ 7

Descriptive adjectives:

< - 5%	– clean (no mention of silt or clay in description)
5 - 15%	– slightly
16 - 35%	– clayey, silty, or silty clayey
36 - 49%	– very

ORGANIC SOILS:

Organic Content	Descriptive Adjectives	Classification
0 - 2.5%	Usually no mention of organics in description	See Above
2.6 - 5%	slightly organic	add “with organic fines” to group name
5 - 30%	organic	SM with organic fines
		Organic Silt (OL)
		Organic Clay (OL)
		Organic Silt (OH)

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHEAST FLORIDA
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

Organic Clay (OH)

HIGHLY ORGANIC SOILS AND MATTER:

Organic Content	Descriptive Adjectives	Classification
30 - 75%	sandy peat	Peat (PT)
	silty peat	Peat (PT)
> 75%	amorphous peat	Peat (PT)
	fibrous peat	Peat (PT)

STRATIFICATION AND STRUCTURE:

Descriptive Term		Thickness
with interbedded		
seam	--	less than ½ inch (13 mm) thick
layer	--	½ to 12-inches (300 mm) thick
stratum	--	more than 12-inches (300 mm) thick
pocket	--	small, erratic deposit, usually less than 1-foot
lens	--	lenticular deposits
occasional	--	one or less per foot of thickness
frequent	--	more than one per foot of thickness
calcareous	--	containing calcium carbonate (reaction to diluted HCL)
hardpan	--	spodic horizon usually medium dense
marl	--	mixture of carbonate clays, silts, shells and sands

ROCK CLASSIFICATION (FLORIDA) CHART:

Symbol	Typical Description
LS	Hard Bedded Limestone or Caprock
WLS	Fractured or Weathered Limestone
LR	Limerock (gravel, sand, silt and clay mixture)
SLS	Stratified Limestone and Soils

THE PROJECT SOIL DESCRIPTION PROCEDURE FOR SOUTHEAST FLORIDA

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

LEGEND FOR BORING LOGS

N:	Number of blows to drive a 2-inch OD split spoon sampler 12 inches using a 140-pound hammer dropped 30 inches
R:	Refusal (less than six inches advance of the split spoon after 50 hammer blows)
MC:	Moisture content (percent of dry weight)
OC:	Organic content (percent of dry weight)
PL:	Moisture content at the plastic limit
LL:	Moisture content at the liquid limit
PI:	Plasticity index (LL-PL)
qu:	Unconfined compressive strength (tons per square foot, unless otherwise noted)
-200:	Percent passing a No. 200 sieve (200 wash)
+40:	Percent retained above a No. 40 sieve
US:	Undisturbed sample obtained with a thin-wall Shelby tube
k:	Permeability (feet per minute, unless otherwise noted)
DD:	Dry density (pounds per cubic foot)
TW:	Total unit weight (pounds per cubic foot)

APPENDIX IV

Laboratory Test Results

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

Moisture Content (ASTM D2216), Percent Fines Passing US No. 200 Sieve (ASTM D1140)

Job No: **16-112** Location: **Highlands County, FL** Date: **03/23/16**

Project: **IMWID** Station: **NA** Technician **Mult.**

Sample ID	*Pan #*	*Tare weight [grams]*	*Wet Weight Before Wash*	*Dry Weight Before wash*		*Water Weight [grams]*	*Dry Weight After wash*		***Moisture (%)***	***Fines (%)***
			Soil + tare weight [grams]	*Soil + tare weight [grams]*	*Soil weight [grams]*		*Soil + tare weight [grams]*	*Soil weight [grams]*		
B2/4	P40	86.8	286.0	248.0	161.2	38.0	233.1	146.3	**23.6**	**9.2**
B2/7	T11	87.2	219.6	191.5	104.3	28.1	188.1	100.9	**26.9**	**3.3**
B3/3	T18	86.7	287.9	245.0	158.3	42.9	229.8	143.1	**27.1**	**9.6**
B3/6	P17	87.2	324.6	264.0	176.8	60.6	257.7	170.5	**34.3**	**3.6**
B4/3	P13	87.0	282.4	239.1	152.1	43.3	219.0	132.0	**28.5**	**13.2**
B4/6	P15	87.3	263.5	235.9	148.6	27.6	217.2	129.9	**18.6**	**12.6**
B4/9	T15	85.5	286.6	250.9	165.4	35.7	244.0	158.5	**21.6**	**4.2**
B5/3	P20	86.5	244.2	214.0	127.5	30.2	199.0	112.5	**23.7**	**11.8**
B5/5	P5	87.4	273.2	234.7	147.3	38.5	227.3	139.9	**26.1**	**5.0**
B5/8	P10	87.6	241.6	215.2	127.6	26.4	204.9	117.3	**20.7**	**8.1**
B5/14	T14	88.0	288.2	231.1	143.1	57.1	210.8	122.8	**39.9**	**14.2**
B6/3	P44	85.5	282.6	239.7	154.2	42.9	217.0	131.5	**27.8**	**14.7**

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

Moisture Content (ASTM D2216), Percent Fines Passing US No. 200 Sieve (ASTM D1140)

Job No: **16-112** Location: **Highlands County, FL** Date: **03/23/16**

Project: **IMWID** Station: **NA** Technician **Mult.**

Sample ID	*Pan #*	*Tare weight [grams]*	*Wet Weight Before Wash*	*Dry Weight Before wash*		*Water Weight [grams]*	*Dry Weight After wash*		***Moisture (%)***	***Fines (%)***
			Soil + tare weight [grams]	*Soil + tare weight [grams]*	*Soil weight [grams]*		*Soil + tare weight [grams]*	*Soil weight [grams]*		
B6/8	P43	85.2	252.5	218.9	133.7	33.6	202.0	116.8	**25.1**	**12.6**
B6/11	P11	85.8	322.7	279.9	194.1	42.8	267.1	181.3	**22.1**	**6.6**
B7/2	P21	87.1	260.1	227.9	140.8	32.2	206.7	119.6	**22.9**	**15.1**
B7/4	P9	88.3	258.1	224.9	136.6	33.2	215.2	126.9	**24.3**	**7.1**
B7/6	P2	87.2	264.8	236.5	149.3	28.3	214.6	127.4	**19.0**	**14.7**
B7/9	P23	87.4	269.9	239.7	152.3	30.2	227.8	140.4	**19.8**	**7.8**
B7/11	P33	87.6	331.8	286.8	199.2	45.0	274.5	186.9	**22.6**	**6.2**
B8/3	P18	87.5	267.6	230.7	143.2	36.9	200.7	113.2	**25.8**	**20.9**
B8/5	P1	86.2	281.9	250.7	164.5	31.2	223.4	137.2	**19.0**	**16.6**
B8/7	T17	85.8	311.7	274.4	188.6	37.3	259.4	173.6	**19.8**	**8.0**
B9/3	P43	87.3	253.9	225.3	138.0	28.6	209.0	121.7	**20.7**	**11.8**
B9/8	P18	87.6	247.1	214.6	127.0	32.5	203.2	115.6	**25.6**	**9.0**

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ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

Moisture Content (ASTM D2216), Percent Fines Passing US No. 200 Sieve (ASTM D1140)

Job No: **16-112** Location: **Highlands County, FL** Date: **03/23/16**

Project: **IMWID** Station: **NA** Technician **Mult.**

Sample ID	*Pan #*	*Tare weight [grams]*	*Wet Weight Before Wash*	*Dry Weight Before wash*		*Water Weight [grams]*	*Dry Weight After wash*		***Moisture (%)***	***Fines (%)***
			Soil + tare weight [grams]	*Soil + tare weight [grams]*	*Soil weight [grams]*		*Soil + tare weight [grams]*	*Soil weight [grams]*		
B9/14	P15	88.4	277.7	245.2	156.8	32.5	239.1	150.7	**20.7**	**3.9**
B10/3	P44	86.6	276.7	240.3	153.7	36.4	204.7	118.1	**23.7**	**23.2**
B10/4	P21	85.9	305.3	259.1	173.2	46.2	205.9	120.0	**26.7**	**30.7**
B10/8	P11	86.9	285.8	247.0	160.1	38.8	235.6	148.7	**24.2**	**7.1**
B10/12	P9	86.6	298.7	258.0	171.4	40.7	234.0	147.4	**23.7**	**14.0**
B11/2	T11	86.2	243.4	198.1	111.9	45.3	165.7	79.5	**40.5**	**29.0**
B11/3	P40	87.4	283.6	236.7	149.3	46.9	187.5	100.1	**31.4**	**33.0**
B11/6	P23	87.6	282.7	242.8	155.2	39.9	220.5	132.9	**25.7**	**14.4**
B11/14	T15	85.5	317.3	277.6	192.1	39.7	262.8	177.3	**20.7**	**7.7**
B12/2	P33	87.4	226.7	201.4	114.0	25.3	177.5	90.1	**22.2**	**21.0**
B12/3	T12	87.3	237.4	211.4	124.1	26.0	186.6	99.3	**21.0**	**20.0**
B12/7	P33	87.9	267.0	230.1	142.2	36.9	210.9	123.0	**25.9**	**13.5**

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

AACE

Organic Content Work Sheet (AASHTO T-267 / ASTM D2974)

Project Name:	**IMWID**	USCS/AASHTO:	**NA**
File Number:	**16/112**	Date Sampled:	**Varies**
Sample Location:	**Varies**	Date Tested:	**3/26/2016**
Sample Description:	**Refer to Log**	Tested By:	**SM**

Loss On Ignition (LO) Test

	Sample ID	B10/1
	Sample Location	As noted on log
	Depth	As noted on log
	Tare Number	P42
	Wt. Of Tare (g) - A	24.1
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	39.0
a.i.	Wt. Tare+Soil (g) - C	30.9
	% Organics: 100x(B-C)/(B-A)	54
		(Moisture: 191%)

Loss On Ignition (LO) Test

	Sample ID	B9/1
	Sample Location	As noted on log
	Depth	As noted on log
	Tare Number	P22
	Wt. Of Tare (g) - A	22.3
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	38.3
a.i.	Wt. Tare+Soil (g) - C	30.0
	% Organics: 100x(B-C)/(B-A)	52
		(Moisture: 212%)

Loss On Ignition (LO) Test

	Sample ID	B8/1
	Sample Location	As noted on log
	Depth	As noted on log
	Tare Number	T12
	Wt. Of Tare (g) - A	22.5
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	39.1
a.i.	Wt. Tare+Soil (g) - C	34.2
	% Organics: 100x(B-C)/(B-A)	30
		(Moisture: 95%)

Loss On Ignition (LO) Test

	Sample ID	
	Sample Location	
	Depth	
	Tare Number	
	Wt. Of Tare (g) - A	
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	
a.i.	Wt. Tare+Soil (g) - C	
	% Organics: 100x(B-C)/(B-A)	

Loss On Ignition (LO) Test

	Sample ID	
	Sample Location	
	Depth	
	Tare Number	
	Wt. Of Tare (g) - A	
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	
a.i.	Wt. Tare+Soil (g) - C	
	% Organics: 100x(B-C)/(B-A)	

Loss On Ignition (LO) Test

	Sample ID	
	Sample Location	
	Depth	
	Tare Number	
	Wt. Of Tare (g) - A	
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	
a.i.	Wt. Tare+Soil (g) - C	
	% Organics: 100x(B-C)/(B-A)	

Loss On Ignition (LO) Test

	Sample ID	
	Sample Location	
	Depth	
	Tare Number	
	Wt. Of Tare (g) - A	
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	
a.i.	Wt. Tare+Soil (g) - C	
	% Organics: 100x(B-C)/(B-A)	

Loss On Ignition (LO) Test

	Sample ID	
	Sample Location	
	Depth	
	Tare Number	
	Wt. Of Tare (g) - A	
b.i.	Wt. Of Tare+Soil+Orgn (g) - B	
a.i.	Wt. Tare+Soil (g) - C	
	% Organics: 100x(B-C)/(B-A)	

Notes: b.i - before ignition, a.i - after ignition
report organics to 0.1%

APPENDIX V

AACE Project Limitations and Conditions

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

Project Limitations and Conditions

Andersen Andre Consulting Engineers, Inc. has prepared this report for our client for his exclusive use, in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made herein. Further, the report, in all cases, is subject to the following limitations and conditions:

VARIABLE/UNANTICIPATED SUBSURFACE CONDITIONS

The engineering analysis, evaluation and subsequent recommendations presented herein are based on the data obtained from our field explorations, at the specific locations explored on the dates indicated in the report. This report does not reflect any subsurface variations (e.g. soil types, groundwater levels, etc.) which may occur adjacent or between borings.

The nature and extent of any such variations may not become evident until construction/excavation commences. In the event such variations are encountered, Andersen Andre Consulting Engineers, Inc. may find it necessary to (1) perform additional subsurface explorations, (2) conduct in-the-field observations of encountered variations, and/or re-evaluate the conclusions and recommendations presented herein.

We at Andersen Andre Consulting Engineers, Inc. recommend that the project specifications necessitate the contractor immediately notifying Andersen Andre Consulting Engineers, Inc., the owner and the design engineer (if applicable) if subsurface conditions are encountered that are different from those presented in this report.

No claim by the contractor for any conditions differing from those expected in the plans and specifications, or presented in this report, should be allowed unless the contractor notifies the owner and Andersen Andre Consulting Engineers, Inc. of such differing site conditions. Additionally, we recommend that all foundation work and site improvements be observed by an Andersen Andre Consulting Engineers, Inc. representative.

SOIL STRATA CHANGES

Soil strata changes are indicated by a horizontal line on the soil boring profiles (boring logs) presented within this report. However, the actual strata’s changes may be more gradual and indistinct. Where changes occur between soil samples, the locations of the changes must be estimated using the available information and may not be at the exact depth indicated.

SINKHOLE POTENTIAL

Unless specifically requested in writing, a subsurface exploration performed by Andersen Andre Consulting Engineers, Inc. is not intended to be an evaluation for sinkhole potential.

MISINTERPRETATION OF SUBSURFACE SOIL EXPLORATION REPORT

Andersen Andre Consulting Engineers, Inc. is responsible for the conclusions and recommendations presented herein, based upon the subsurface data obtained during this project. If others render conclusions or opinions, or make recommendations based upon the data presented in this report, those conclusions, opinions and/or recommendations are not the responsibility of Andersen Andre Consulting Engineers, Inc.

CHANGED STRUCTURE OR LOCATION

This report was prepared to assist the owner, architect and/or civil engineer in the design of the subject project. If any changes in the construction, design and/or location of the structures as discussed in this report are planned, or if any structures are included or added that are not discussed in this report, the conclusions and recommendations contained in this report may not be valid. All such changes in the project plans should be made known to Andersen Andre Consulting Engineers, Inc. for our subsequent re-evaluation.

USE OF REPORT BY BIDDERS

Bidders who are reviewing this report prior to submission of a bid are cautioned that this report was prepared to assist the owners and project designers. Bidders should coordinate their own subsurface explorations (e.g.; soil borings, test pits, etc.) for the purpose of determining any conditions that may affect construction operations. Andersen Andre Consulting Engineers, Inc. cannot be held responsible for any interpretations made using this report or the attached boring logs with regard to their adequacy in reflecting subsurface conditions which may affect construction operations.

IN-THE-FIELD OBSERVATIONS

Andersen Andre Consulting Engineers, Inc. attempts to identify subsurface conditions, including soil stratigraphy, water levels, zones of lost circulation, “hard” or “soft” drilling, subsurface obstructions, etc. However, lack of mention in the report does not preclude the presence of such conditions.

LOCATION OF BURIED OBJECTS

Users of this report are cautioned that there was no requirement for Andersen Andre Consulting Engineers, Inc. to attempt to locate any man-made, underground objects during the course of this exploration, and that no attempts to locate any such objects were performed. Andersen Andre Consulting Engineers, Inc. cannot be responsible for any buried man-made objects which are subsequently encountered during construction.

PASSAGE OF TIME

This report reflects subsurface conditions that were encountered at the time/date indicated in the report. Significant changes can occur at the site during the passage of time. The user of the report recognizes the inherent risk in using the information presented herein after a reasonable amount of time has passed. We recommend the user of the report contact Andersen Andre Consulting Engineers, Inc. with any questions or concerns regarding this issue.

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

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