MOORHEN MARSH LOW ENERGY AQUATIC PLANT SYSTEM

SUBSURFACE SOIL EXPLORATION AND GEOTECNICAL EGINEERING EVALUATION

MOORHEN MARSH AQUATIC PLANT SYSTEM (LEAPS) PROJECT (including ADDENDUM NO. 1)

INDIAN RIVER COUNTY, FLORIDA

NOTE: AS DISCUSSED IN GENERAL CONDITIONS 4.02, THE FOLLOWING REPORT AND ADDENDUM *ARE NOT* PART OF THE BIDDING DOCUMENTS OR THE CONTRACT DOCUMENTS.

SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION MOORHEN MARSH LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS) PROJECT INDIAN RIVER COUNTY, FLORIDA

AACE FILE NO. 19-140

The Addendum referenced in this report is an addendum to the original geotechnical engineering evaluation, not an addendum to the Bidding Documents.

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Indian River County
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Vero Beach, FL 32960

Attn: Mr. Keith McCully, P.E.

**SUBSURFACE SOIL EXPLORATION AND
GEOTECHNICAL ENGINEERING EVALUATION
MOORHEN MARSH
LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS) PROJECT
INDIAN RIVER COUNTY, FLORIDA**

1.0 INTRODUCTION

In accordance with your authorization, Andersen Andre Consulting Engineers, Inc. (hereinafter referred to as AACE) has completed a subsurface exploration and geotechnical engineering analyses for the above referenced project. The purpose of performing this exploration was to explore shallow soil types and groundwater levels, and restrictions which these may place on the proposed stormwater treatment system. Our work included Standard Penetration Test (SPT) borings, auger borings, piezometer installations, field soil hydraulic conductivity and infiltration testing, laboratory testing, and engineering analyses. This report documents our explorations, presents our findings, and summarizes our conclusions and recommendations.

2.0 SITE INFORMATION AND PROJECT UNDERSTANDING

2.1 Site Location and Description

The approximately 18-acre subject site consists of two adjacent properties located on the northeast corner of 66th Avenue and 53rd Street in Vero Beach, Indian River County, Florida (within Section 17, Township 32 South, Range 39 East).

- IRC Property Appraiser Parcel No. 32391700001013000001.0 [9.12 acres]
- IRC Property Appraiser Parcel No. 32391700001013000002.1 [9.51 acres]

A Site Vicinity Map (2018 aerial photograph) which depicts the location of the subject site is included on the attached Figure No. 1. The site location is further shown superimposed on the 1983 "Vero Beach, Florida" USGS topographic Quadrangle Map also included on Figure No. 1. The Quadrangle Map depicts the subject property as being relatively level with approximate surface elevations of 19-20 feet relative to the National Geodetic Vertical Datum of 1929.

The site is an abandoned/overgrown citrus grove and appears to have been in use for citrus-growing purposes since at least 1994, based on our cursory review of readily available online aerial photographs.

The site is bordered by the following features:

- North: The IRFWCD North Relief Canal and then vacant land.
- South: 53rd Street (unpaved) and then an equestrian facility (Palema Trotting).
- West: The IRFWCD Lateral "A" Canal, 66th Avenue (paved), and then vacant land.
- East: Single-family residences and vacant land.

Based on our cursory review of the provided topographic survey prepared by CivilSurv Design Group, Inc. (dated 10/07/17), the site appears to have ground elevations ranging from about EL 17.5 feet to about 19.5 feet, corresponding mostly to the highs and lows of the north-south oriented planting beds and furrows associated with the former citrus-growing operations. <u>The datum for elevations referenced in this report is the North American Vertical Datum of 1988 (NAVD88)</u>.

2.2 Review of USDA Soil Survey

The surficial soil types identified by the USDA NRCS Web Soil Survey to be present within the subject site are as follows:

- Map ID 14: <u>Winder fine sand, 0 to 2 percent slopes</u>
  "Sandy and loamy marine deposits from flats and drainageways within historic marine terraces, with fine sand, sandy loam and loamy sands present to depths of 80 inches below grade".

- Map ID 16: <u>Pineda-Pineda, wet, fine sand, 0 to 2 percent slopes</u>
  Sandy and loamy marine deposits from flatwoods and drainageways within historic marine terraces, with fine sand and fine sandy loam present to depths of 80 inches below grade".

- Map ID 36: <u>Boca fine sand</u>
  Sandy and loamy marine deposits over limestone from within flats on historic marine terraces, with fine sands, fine sandy loam, and then limestone from 24 to 28 inches below grade.

The approximate location of the subject site is shown superimposed on a copy of the USDA Web Soil Survey aerial photograph, presented on Figure No. 1. Further, excerpts from the USDA Web Soil Survey summary report are included in Appendix I.

2.3 Project Understanding

Based on our conversations and our review of the provided project-related information, we understand that it is proposed to construct an aquatic plant water treatment system (Low Energy Aquatic Plant System - LEAPS) intended to reduce contamination/pollutants from the water in the adjacent Indian River Farms Water Control District (IRFWCD) North Relief Canal (NRC).

The LEAPS facility will have a pump station near the northwest corner of the site, pumping water from the NRC into the facility. The water will pass through two water lettuce scrubber basins after which the water is diverted through four algal reaeration units, then through two settling basins, and then finally through two wetland polishing marshes before it is released back into the NRC.

The following describes our understanding of the individual LEAPS components as it relates to our analysis:

- The intake pump station will consist of an approximately 20-ft deep concrete structure (bottom near EL 2.00 ft)

- The two water lettuce scrubber areas will have a concrete slab (EL 20.50 ft) and approximately 4-ft high HDPE-lined perimeter berms. Interior concrete curbs and divider walls will also be constructed.

- The algal reaeration units will also have concrete slabs (EL 19.30 ft) and interior walls/weirs for the water to pass over. Adjacent areas will be utilized for sludge storage and composting.

- The settling basins will have bottom elevations of 10.00 ft and a maximum water elevation of 17.50 with a compacted soil bottom that will ramp up to the adjacent wetland polishing marshes, which in turn will have compacted soil bottom elevations of 16 feet (and two interior sumps with bottom elevations of 12.50 feet).

- Wetland Polishing Marsh No. 1 will be pipe-connected directly to the NRC through Structures S8 and S9 which will have bottom elevations near EL 10.00. Marsh No. 1 will be isolated from Marsh No. 2 by a composting pad/dredge pad, a drive-aisle, and two stormwater retention areas. Similarly, Wetland Polishing Marsh No. 2 will be pipe-connected to the NRC and will only be separated from the NRC by an earthen (un-lined) low berm (top elevation of about 21.00 ft).

The LEAPS interconnectivity will be provided by various concrete structures (a few with bottom elevations near 10.00 ft) and piping. Additional project components include stormwater retention areas and swales, a single-story CMU operations building, an asphalt paved entrance roadway, and unpaved/stabilized interior drive aisles.

In brief, we understand that the LEAPS facility will be operated by pumping water from the NRC into the system for 12 hours and then letting the system rest for the next 12 hours (repeated in perpetuity). As the water migrates through the LEAPS, nitrogen and phosphorous will be removed prior to water release back into the NRC and ultimately into the Indian River Lagoon. As part of the design of the system, AACE was tasked with performing a limited study to assist in estimating the loss of water, through infiltration into the natural soils or through lateral seepage, that the proposed treatment system can be expected to experience. This is limited to the proposed interface of Water Polishing Marsh No. 2 (WPM2) and the NRC. Additionally, a limited stability analysis of the NRC canal banks was performed, including reviewing the expected seepage flow exit gradients at the NRC. Figure No. 2 presents the proposed cross-section between WPM2 and the NRC, and the following water levels were provided for the WPM2 and NRC for use in our analyses:

- WPM2 operational water level elevation: 17.50 ft
- NRC (controlled) water level elevations: 10.37 ft (low) and 15.87 ft (high)

3.0 FIELD EXPLORATION PROGRAM

To explore subsurface conditions within the site relative to the proposed LEAPS facility construction, the field exploration program summarized in Table No. 1 below was completed. The locations of the completed field work are graphically depicted on the Field Work Location Plan, presented on Figure No. 3.

Table No. 1 - Field Exploration Program

Boring Type	Standard	# of Borings/Tests	Depth Below Grade [feet]	Location
Standard Penetration Test (SPT)	ASTM D1586	11	25-60	Refer to Figure No. 3
Auger	ASTM D1452	10	5	Refer to Figure No. 3
Temporary Piezometers and Field Permeability Tests	NA	44	15-40	Refer to Figure No. 3
Double-Ring Infiltrometer Tests	ASTM D3385	3	NA	Refer to Figure No. 3

Our field work was performed in the period June-September, 2019. The field work locations shown on Figure No. 3 were determined in the field by our field crew using the provided site plan, aerial photographs, existing site features, and hand-held WAAS enabled GPS instruments. Atmospheric disturbances and local weather conditions may affect the accuracy of the GPS readings. As such, 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 15-30 feet of those shown on Figure No. 3.

Prior to commencing our field work, a limited vegetation clearing operation was performed by IRC staff so as to provide the access required for our crews and equipment.

Summaries of AACE's field procedures are included in Appendix II, and the individual boring profiles are presented on the attached Sheets No. 1-5. 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, then discarded unless you specifically request otherwise.

4.0 Observed Subsurface Conditions

4.1 General Soil Conditions

Detailed subsurface conditions are illustrated on the soil boring profiles presented on the attached Sheets No. 1-5. 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 soil boring profiles on Sheets No. 1-5, the soils on the site at the locations and the depths explored are fairly homogeneous and consist generally of a few inches of fine sands with varying amount of roots/organics [topsoil] followed by loose fine sands to depths of about 2-5 feet, followed by loose to moderately dense slightly clayey (SP-SC) to clayey fine sands (SC) to depths of about 13-18 feet, in turn followed by loose to medium dense fine sands (SP) to depths of about 18-24 feet. At this depth, very loose slightly silty fine sands (SP-SM) were encountered to depths of about 28-33 feet, in turn followed by medium dense to very dense fine sands (SP) with shell fragments reaching the termination depths of our borings.

In general, the near-surface findings of our soil borings correlate well with those described in the USDA Soil Survey.

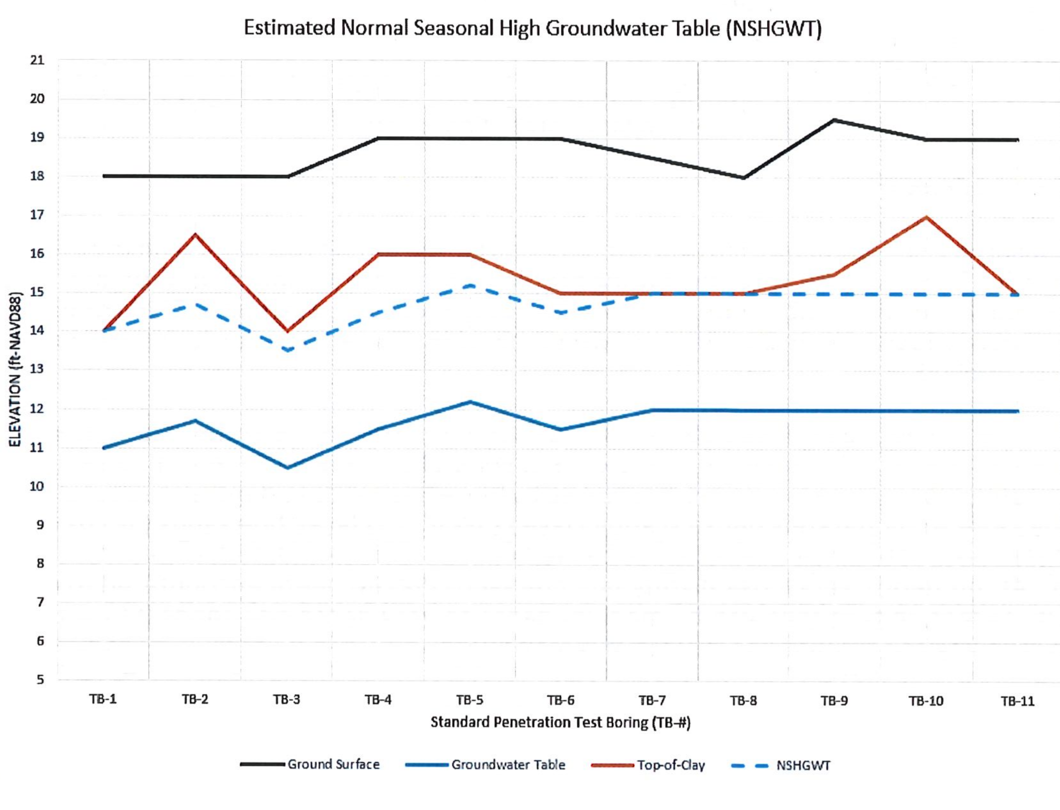
4.2 Measured Groundwater Level

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. 1-5. As can be seen, the groundwater table was generally encountered at depths ranging from 6.0 to 7.5 feet 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.

4.3 Estimated Normal Seasonal High Groundwater Table

The groundwater table will fluctuate seasonally, primarily based on rainfall. The normal seasonal high groundwater table (NHSGWT) is likely during the rainy season in Southeast Florida, typically between June and September/October 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 NHSGWT 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.

From the provided topographic survey, in Figure 1 below we have estimated the elevations of the ground surface, the encountered groundwater table, and the top of the uppermost hydraulically restrictive stratum (slightly clayey to clayey fine sands) for the completed SPT borings. Hence, combining this data with our overall field explorations and laboratory examinations, our review of the USDA soil survey, and considering the time of year when our explorations were performed (i.e. in the wet season), we estimate that the NSHGWT at the boring locations is about 2-3 feet above the levels encountered in the borings and piezometers. As such, a generalized NSHGWT elevation of 14.00 ft to 14.50 ft appears to be a reasonable recommendation for the subject site.



The estimated NHSGWT does 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. If a more accurate determination of the seasonal groundwater level variations on this site is prudent for the design of the project, we would recommend completing a period of monitoring of the groundwater level fluctuations within the installed piezometers (see below).

4.4 Piezometer Installations and Field Permeability Tests

In order to estimate the hydraulic conductivity of the soils near the interface of WPM2 and the NRC, a well cluster (WC-1, refer to Figure No. 3) consisting of four (4) piezometers was installed near the location of our Boring TB-9. The piezometers were constructed of 2-inch diameter Schedule 40 PVC with 5-foot slotted screens (0.020-inch) at different elevations relative to the findings of our boring TB-9. The piezometers were sand packed with 20/30 grade sand to a depth of 3 feet above their screened sections, after which they were cement-grouted for another 3 feet. Following the piezometer installations, field permeability tests were performed at the screened depths.

The depths of the piezometers, their screened intervals, and the results of the field permeability tests are presented in Table No. 2 below, and also presented in Appendix III.

Table No. 2 - Piezometer Depths and Field Permeability Test Results

Boring ID	Screened Depth (ft-bls)	Soil Description	Constant Head Test	Falling Head Test
			Kh (ft/day)	Kh (ft/day)
TB-9	10-15	Slightly clayey fine sand	8.1	7.8
TB-9	18-23	Fine sand	31.5	30.6
TB-9	25-30	Slightly silty fine sand	0.7	0.7
TB-9	35-40	Fine sand w. shell fragments	56.1	57.0

4.5 Double-Ring Infiltrometer Testing

Three (3) double-ring infiltrometer (DRI) tests were performed at the locations shown on Figure No. 3. These tests were completed in general accordance with the procedures recommended in ASTM D3385. Below is the general information of the DRI tests.

Inner Ring (IR):	Outer Ring (OR):	Annular Space (A):	Height of ring = 24 inches
▸ Diameter = 12 inches	▸ Diameter = 24 inches	▸ Area = 339.3 in^2	(seated 6 inches into ground
▸ Area = 113.1 in^2	▸ Area = 452.4 in^2		following removal of topsoil).

In brief, a constant head of approximately 4-6 inches of water is typically maintained in the rings throughout the duration of the tests. The volume infiltrated during timed intervals was converted to an incremental infiltration rate, and the following equations were then used to calculate the average incremental infiltration velocity, equivalent to the vertical infiltration rate.

$$V_{IR} = \Delta V_{IR} / (A_{IR} \times \Delta t) \ \& \ V_A = \Delta V_A / (A_A \times \Delta t)$$

where:
V = incremental velocity [inches/hour] of inner ring or annular space
ΔV = volume [in^3] of liquid used during time interval to maintain constant head in either the inner ring (IR) or the annular space (A)
A = internal area [in^2] of inner ring or annular space
Δt = time interval [hours]

The results of the DRI tests are presented in Table No. 3.

Table No. 3 - DRI Results

Test No.	Inner Ring Infiltration Rate, V_{IR} (ft/day)	Annular Space Infiltration Rate, V_A (ft/day)
DRI-1	6.2	7.6
DRI-2	4.5	5.3
DRI-3	4.0	3.5

The test reports, including a soil profile at each test location, are presented in Appendix IV.

5.0 LABORATORY TESTING PROGRAM

Our drillers observed the soil recovered from the borings, 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.

Additionally, limited laboratory testing of index properties (Percent Fines [ASTM D1140] and Moisture Content [ASTM D2216]) was performed on representative soil samples to aid in the classification of the soils. The soil classifications and other pertinent data obtained from our explorations and laboratory examinations are reported on Sheets No. 1-5 and summarized in Table No. 4 below.

Table No. 4 - Laboratory Test Results

Boring ID	Depth (ft-bls)	Moisture Content (%)	Fines Content (%)
TB-3	5	20.2	21.2
TB-5	5	18.2	14.6
TB-5	7	22.0	12.8
TB-6	5	20.8	15.7
TB-6	14	20.4	11.1
TB-6	24	27.2	7.1
TB-8	5	12.4	12.9
TB-8	7	22.3	15.7
TB-8	19	31.1	7.9
TB-8	24	29.1	8.9
TB-9	5	17.0	112.6
TB-9	14	19.0	11.1
TB-9	29	27.5	8.8
TB-10	25	30.8	9.5

6.0 GEOTECHNICAL ENGINEERING EVALUATION

6.1 General

Based on the findings of our site exploration, our evaluation of subsurface conditions, and judgment based on our experience with similar projects, we conclude that the majority of the soils underlying this site are generally satisfactory to support the proposed Moorhen Marsh LEAPS features. However, based on the final design of the intake pump station, this feature may need to be supported on a pile foundation due to presence of very loose sands in close proximity to the anticipated pump station bottom elevation. Further, upon filling the site and/or excavation for the various surface features, the bearing capacity of the near-surface soils should be improved in order to reduce the risk of unsatisfactory system performance. The general soil improvement we recommend includes proofrolling the various building, berm, and roadway areas with a heavy vibratory roller.

The following sections of this report present a limited seepage and slope stability analysis for the cross-section presented on Figure No. 2, followed by recommendations for site preparation procedures, foundation design, pavement systems, etc.

6.2 Limited Seepage Analyses

As noted, the cross-section presented in Figure No. 2 was analyzed further to assist in estimating the loss of water, through infiltration into the natural soils and/or through lateral seepage, that the proposed LEAPS system can be expected to experience at the interface of Water Polishing Marsh No. 2 (WPM2) and the NRC. Additionally, a limited stability analysis of the southern NRC canal bank was performed.

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 linked to the GeoStudio slope stability module SLOPE/W.

The seepage analyses was used to evaluate the following:

- Phreatic surface (for use in subsequent slope stability analyses).
- Exit gradients and factors of safety against piping.
- Flow rates through selected WPM2-NRC cross-section.

The provided topographic survey information and site plan were used to estimate the elevations, horizontal distances and NRC canal slopes (estimated to be 2H:1V) at the selected cross-section. Further, the provided WPM2 and NRC water levels were used in the analysis.

The soil layers and associated hydraulic conductivities used in the analyses are summarized in Table No. 5 below. Also included in this table are input needed for slope stability analyses (refer to Section 6.2), 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 included in Appendix III. 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.

Table No. 5: Analysis Parameters for Encountered Soil Profile (TB-9)

General Stratum Description (TB-9)	Hydraulic Conductivity, K_H (ft/day)	k Ratio (V/H)	γ_{sat} (pcf)	ϕ (degr.)	Cohesion (psf)
WPM2 Berm (well-compacted soils)	0.5	0.5	118	35	--
Slightly clayey fine sands (0 to 16' bls)	8	0.5	120	33	--
Fine sands (16' to 25' bls)	30	1	112	30	--
Slightly silty fine sands (25' to 33' bls)	0.7	1	115	32	--
Fine sands with shell fragments (33' to 60' bls)	50	1	115	31	--

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. 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.

The SEEP/W analyses were run in steady-state mode using the parameters and boundary conditions described in the previous. Individual SEEP/W finite element mesh and subsurface layering are presented in the following, and flow rates through the analyzed cross-section were evaluated using flux lines within the software.

The exit seepage gradients into the Lateral J canal 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 canal bottom, 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.

For seepage from the side slopes of the canal, 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:

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

For cohesionless soil, critical vertical gradients typically vary between 0.8 and 1.1. Using a conservative total saturated unit weight of 110 pcf for the natural soils, the <u>critical vertical gradient is approximately 0.76</u>. The resulting <u>critical horizontal exit gradient is approximately 0.44</u> for an effective friction angle of 30 degrees. These critical gradients were compared to the calculated gradients in the seepage model to determine the factor of safety against piping from WPM2 seepage.

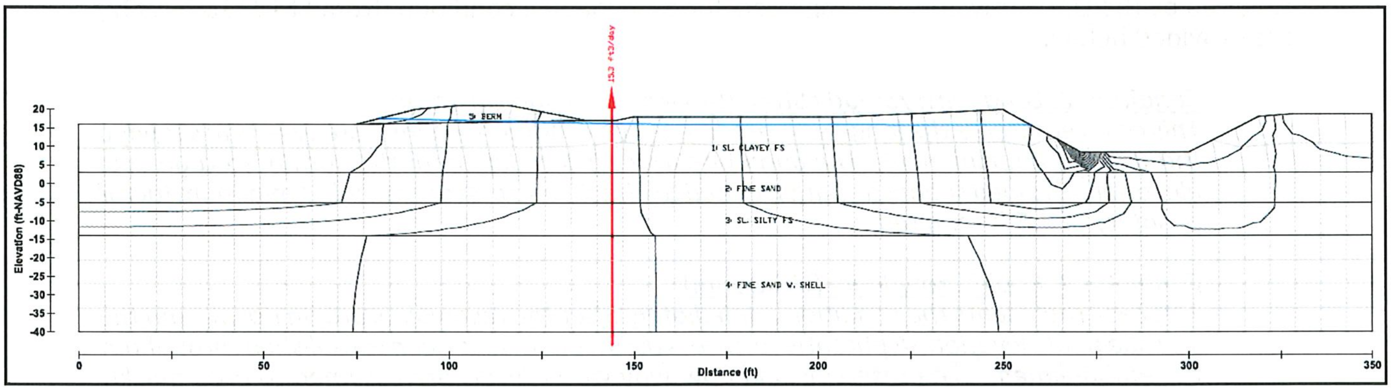
$$i_{ch} = i_{cv} \tan\phi' \left(\varphi' = \textit{effective friction angle of soil}\right)$$

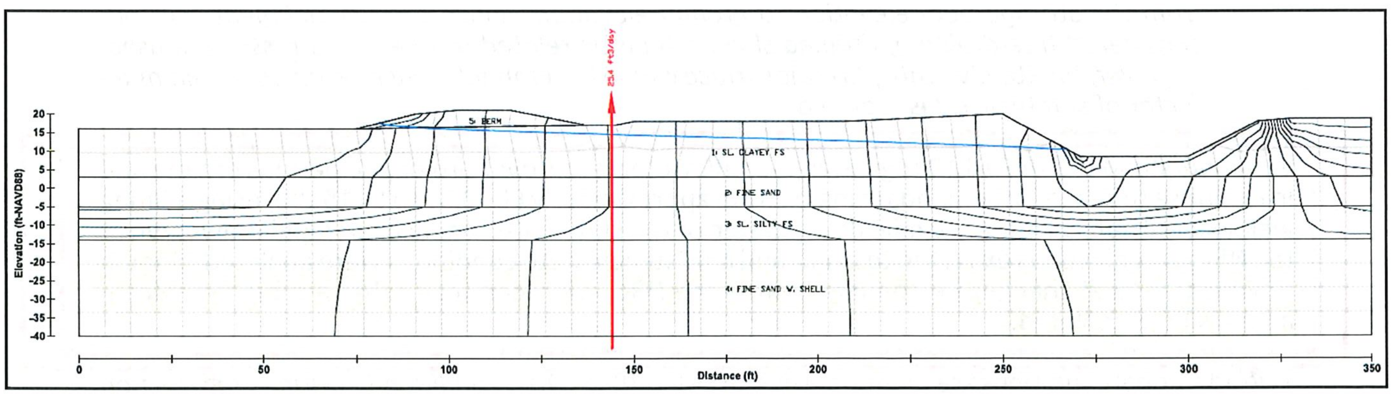
Based on our literature review, recommended factors of safety against piping range from 2 to 5. In general, higher factors of safety are recommended at the downstream toe and downstream seepage ditch for larger embankments (greater than 6 feet in height). Higher factors of safety are also recommended when limited subsurface information is available for evaluating piping potential. For the analyzed scenario, AACE recommends a minimum factor of safety of 3.

The calculated seepage rates, corresponding exit gradients and their factors of safety are summarized in Table No. 6 and presented graphically below.

Table No. 6: Seepage Analyses Results for WPM2 and NRC

WPM2 Water Level Elevation	Lateral J Canal Water Level Elevation	Maximum Exit Gradients				Seepage Rate (ft^3/day/ft)
		Horizontal	FOS	Vertical	FOS	
17.5	15.87	0.09	4.8	0.18	4.2	15.3
17.5	10.37	0.12	3.6	0.23	3.3	25.4





6.3 Limited Slope Stability Analyses

Stability analyses were performed for the selected WPM2-NRC cross-section presented on Figure No. 2. 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.

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 imported into the SLOPE/W models.

The soil parameters used for input in SLOPE/W were presented in Table No. 5. 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.

A description of applicable 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.

> *Condition 2: Long-Term/Steady State (Downstream and Upstream)*
> *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. A minimum factor of safety of 1.5 is required.*
>
> *Condition 3: Rapid Drawdown (Upstream)*
> *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. A minimum factor of safety of 1.3 is required.*

We understand that IRFWMD regulates the NRC water levels over prolonged periods of time and in response to seasonal demands (ranging from approximately EL 10.37 to EL 15.87), using a tilting weir located approximately 1-mile downstream of the LEAPS site. As such, no rapid drawdown of the NRC water levels is expected to occur and only steady-state conditions were analyzed. Should rapid water level changes be common within the NRC, we recommend that a rapid drawdown analysis be completed.

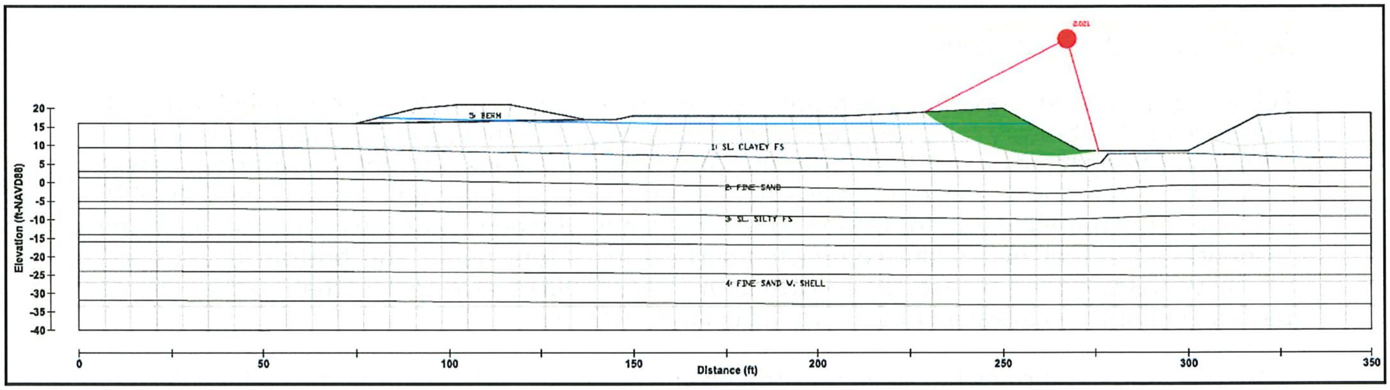
Two additional conditions are mentioned in the EM 1110-2-1902: *Condition 1 - During Construction and End-Of-Construction* and *Condition 4 - Earthquake.* Neither of these loading conditions were considered applicable for this project and were therefore not modeled.

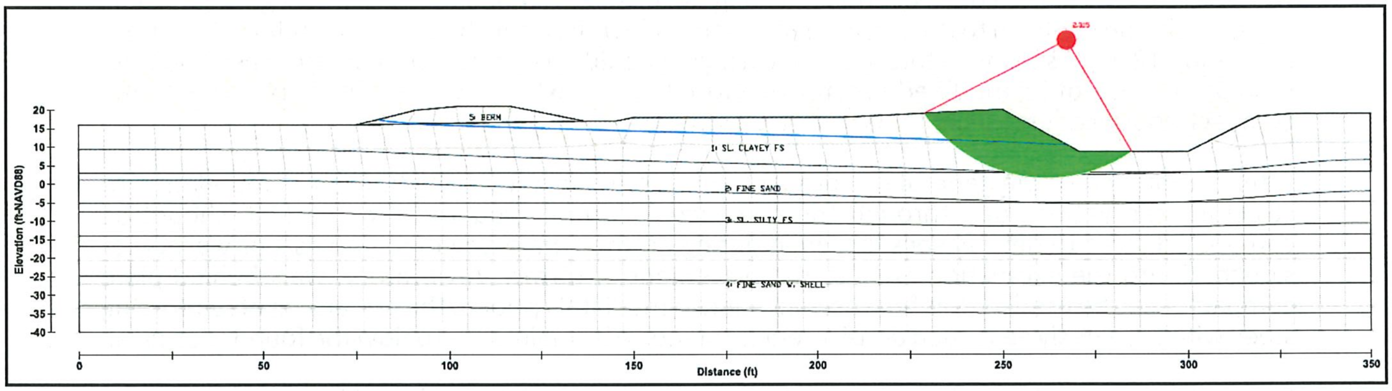
For the steady-state scenario, a minimum required factor of safety of 1.5 was used for evaluating the stability of the selected cross-section. The results of the limited slope stability analyses are summarized in Table No. 7 and presented graphically below.

CALL PETER ABOUT THIS. WHAT IS CONSIDERED A "RAPID LEVEL WATER CHANGE". IRFWCD CAN DRAIN CANAL SUDDENLY OVER A SHORT PERIOD.

Table No. 7: Slope Stability Analyses Results for WPM2 and NRC

WPM2 Water Level Elevation	Lateral J Canal Water Level Elevation	Factor of Safety
17.5	15.87	2.02
17.5	10.37	2.32





In brief, the slope stability factors of safety for the NRC canal bank were satisfactory for both the low and high NRC canal water level as compared to the WPM2 operational water level. However, we note that it is important to realize that localized areas of shallow sloughing/slumping and/or erosion may still periodically occur, especially following significant storm events. Examples of shallow failures include rutting caused by maintenance equipment working on the canal banks and the development of "gullies" from stormwater runoff. The US Army Corps of Engineers (USACE) EM 1110-2-1902 and ETL 1110-2-561 documents consider shallow failures as a maintenance issue that typically do not greatly affect the global stability of the slope if periodically repaired and maintained. However, the USACE documents also states that it is important to realize that if shallow failures are not repaired, they can progress to larger failures which, in turn, can create rotational slope failures.

It is recommended to install a riprap revetment within the NRC at the location of the LEAPS outfall pile so as to prevent scour and erosion of the canal banks due to the discharge of water.

6.4 Construction Recommendations

6.4.1 Clearing

The site surface should be cleared, grubbed and stripped of all vegetation, topsoil, trash, debris, etc. During these clearing operations, in the LEAPS areas to be filled, leveling of the existing planting beds and furrows should be completed.

6.4.2 Compaction Procedures

Following clearing and leveling, the proposed construction areas should be proofrolled with a 10 ton (minimum) vibratory roller; 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, or 2 feet below the bottom of footings, whichever is lower. In any case, the construction areas should receive not less than 10 overlapping passes, half of them in each of two perpendicular directions.

After the exposed surface has been proofrolled and tested to verify that the desired dry density has been obtained, the construction areas may be filled as needed to the desired grades. All fill material should conform to the recommendations below. It should be placed in uniform layers not exceeding 12 inches in loose thickness. Each layer should be compacted to a dry density not less than 95 percent of its modified Proctor (ASTM D1557) maximum value. These placement and compaction procedures include the construction of berms for the project.

After completion of the general site preparations discussed above, the bottom of foundation or structure excavations dug through the compacted natural ground, fill or backfill, should be compacted so as to densify soils loosened during or after the excavation process, or washed or sloughed into the excavation prior to the placement of forms. A vibratory, walk-behind plate compactor can be used for this final densification immediately prior to the placement of reinforcing steel, with previously described density requirements to be maintained below the foundation level.

Following removal of foundation forms, backfill around foundations should be placed in lifts six inches or less in thickness, with each lift individually compacted with a plate tamper. The backfill should be compacted to a dry density of at least 95 percent of the modified Proctor (ASTM D1557) maximum dry density.

We recommend that the site preparation contractor closely monitor the vibrations produced during the proofrolling operations so that they do not adversely affect any nearby structures.

6.4.3 Fill Materials and Excavated Soils

All fill material under the buildings and pavement should consist of clean sands free of organics and other deleterious materials. The fill material should have not more than 12 percent by dry weight passing the U.S. No. 200 sieve, and no particle larger than 3 inches in diameter. Backfill behind walls, if any, should be particularly pervious, with not more than 4 percent by dry weight passing the U.S. #200 sieve.

Parts of this project will include excavations and we offer the following general comments with regards to the suitability of the encountered soils for use as structural fill materials:

- Fine sands (SP) should be suitable to serve as fill soils and with proper moisture control should densify using conventional compaction equipment. Soils obtained from below the water table may require time to dry sufficiently. However, these materials should be suitable for relatively unrestricted use as fill and roadway embankment.

- Slightly clayey fine sands (SP-SC) and slightly silty fine sand (SP-SM) are suitable for structural fill, but will likely be more difficult to compact due to their inherent nature to retain excess soil moisture. If the use of slightly clayey soils is desired, it may be necessary to stockpile these soils in order for them to drain. Thinner lifts (perhaps 6 to 8 inches in loose thickness) may be required for placement and compaction of these soils. Further, it may become necessary to mix these soils with drier, cleaner granular sands prior to placement to increase the "workability" of these soils.

- Clayey fine sands (SC) with fines content in excess of 15-18 percent are generally considered unsuitable for use as structural fill because of the difficulty in conditioning and working the materials. However, after drying, clayey soils can be mixed with sands with less fines content (i.e. less than 5 percent passing the U.S. No. 200 sieve) and likely be used.

- Organic topsoil is not considered suitable for use as any type of fill, other than in landscaped areas or other non-structural areas.

If it is attempted to blend the more clayey soils with the sands containing less fines, we would recommend obtaining post-mix samples for laboratory determination of moisture contents, fines content, in addition to optimum moisture contents/maximum density relationships, so as to determine whether the soils were sufficiently mixed, and to provide guidelines for placement and compaction procedures. For the more clayey fill materials, it will be prudent to compact the soils within 1 to 2 percent of the materials' optimum moisture contents. Nevertheless, once excavated, we recommend that all soils be stockpiled as high as possible so as to increase the rate of drainage, prior to placement and compaction. If the clayey soils remain saturated they could be used in non-structural areas with no compaction requirements. It is also recommended that careful monitoring of the excavation efforts be performed in order to segregate the more clayey materials from the cleaner materials.

6.4.4 Trenches and Excavations

It is noted that the shallow subsoils consist mostly of slightly clayey to clayey sands with a thin surficial mantel of relatively clean sands. These clayey soils may be difficult to dewater. Further, due to the elevated fines content of the soils and the resulting tendency of the soils to retain excessive moisture, these site soils will be problematic if the contractor intends to excavate the soils and immediately (or shortly thereafter) return the soils to the excavation as backfill.

Excavations made through these soils may have to be deepened and backfilled partially with gravel to allow creating a firm bottom. All trench backfill should be placed in level lifts of 12 inches, with each lift compacted to a dry density of 98 percent of the modified Proctor (ASTM D1557) maximum dry density.

Any excavation should be made in accordance with applicable State and Federal requirements and guidelines. The recommendations and definitions in OSHA 29 CFR Part 1926 Subpart P "Excavations" should be reviewed and utilized for any subsurface excavation efforts. An engineered shoring system may be required to facilitate the RCP and structure installations; the design and implementation of temporary slopes and shoring systems should be the responsibility of the Contractor. The Contractor is further responsible for adherence to all relevant trench safety requirements, including any updated regulations not addressed herein.

6.5 Foundations and Concrete Slab Designs

6.5.1 Single-Story Operations Building

After the foundation soils have been prepared as recommended above, the site should be suitable for supporting the proposed single-story operations building construction on conventional shallow foundations (or a thickened-edge monolithic slab) proportioned for an allowable bearing stress of 2,500 pounds per square foot [psf], or less. To provide an adequate factor of safety against a shearing failure in the subsoils, all continuous foundations should be at least 18 inches wide, and all individual column footings should have a minimum width of 36 inches (if any). Exterior foundations (or thickened-edge slab sections) should bear at least 18 inches below adjacent outside final grades.

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

We recommend that representatives of AACE inspect all footing excavations in order to verify that footing bearing conditions are consistent with expectations. Foundation concrete should not be cast over a foundation surface containing topsoil or organic soils, trash of any kind, surface made muddy by rainfall runoff, or groundwater rise, or loose soil caused by excavation or other construction work. Reinforcing steel should also be clean at the time of concrete casting. If such conditions develop during construction, the reinforcing steel must be lifted out and the foundation surface reconditioned and approved by AACE.

6.5.2 Concrete Slabs

The Water Lettuce Scrubber basins and the Algal Reaeration units (along with other project components) will have concrete slabs. After the ground surface is proofrolled and filled, if necessary, as recommended in this report, these slabs can be placed directly on the prepared subgrade. For design purposes, we recommend using a subgrade reaction modulus of 150 pounds per cubic inch (pci) for the compacted shallow sands. In our opinion, a highly porous base material is not necessary.

The subgrade surface should be saturated immediately prior to concrete placement to provide adequate moisture for curing of the concrete. We recommend a minimum 28-day compressive strength of 5,000 psi. Construction control joints should be placed no more than 15 feet apart in either direction and should be at least one-quarter of the thickness of the concrete (or as directed by the project Structural Engineer), and should be cut as soon as the concrete will support the crew and equipment (8 to 12 hours). The concrete should be cured by moist curing or by application of a liquid curing compound.

6.5.3 Intake Pump Station

We understand that the intake pump station may consist of a 20-ft deep (±) concrete box structure with associated pump station features. Our boring TB-11, performed in close proximity to this pump station, encountered a layer of very loose slightly silty fine sands (SP-SM) and slightly clayey fine sands (SP-SC) from approximately 18 feet to 33 feet below grade. From the provided topographic survey, it is estimated that the ground surface elevation at this boring is approximately 19.00 ft, with this very loose stratum then extending from near elevation 1.00 ft to elevation -14.00 ft.

Depending on the final design of the pump station, it may be necessary to utilize a deep foundation system (i.e. piles) for its support due to these very loose soil conditions located in depth-proximity to the bottom of the pump station structure (elevation to be determined). These piles would be installed from the bottom of the pump station excavation and would derive support from the deeper dense to very dense fine sand stratum and could, if needed, also provide uplift capacities for the structure. Preliminarily, 14-inch diameter, 30-ft long augered cast-in-place (augercast) concrete piles would likely yield allowable capacities in excess of 25 tons (compression) and 15 tons (uplift). Alternatives to augercast piles include driven prestressed concrete piles or helical piles. Overall, numerous combinations of pile types, sizes, depths, etc. exist, and we remain available for specific pile foundation consultations (if needed) once the pump station design has progressed to a point where the final configuration, depth, and weight of the structure are known.

The construction of this pump station structure will likely include the installation of a steel sheet pile cofferdam and dewatering operations. With regards to the design of dewatering operations (by others), we recommend that the field permeability test results presented in Appendix III be reviewed as they include flow rates for various depths and strata.

Any steel sheet pile structure needed for the construction of the pump station, including any internal bracing, tie-back system, or other lateral restraining systems, will need to be designed by a Professional Engineer registered in the State of Florida. Soil parameters for use in the design of a sheet pile cofferdam can be obtained from Table No. 5.

7.0 Pavement Recommendations

It is our understand that portions of the proposed internal drive-aisles and the entrance off 53rd Street will be asphalt-paved, while other internal drive-aisles may simply be stabilized.

The pavement sections should be installed late in construction when most heavy construction traffic has ceased. If base material is placed during construction to provide a working surface it should be proofrolled, leveled, and thickened as required prior to paving at the end of construction.

For a flexible pavement section we recommended an asphaltic concrete wearing surface on a calcareous base course supported on stabilized subbase over well-compacted subgrade.

- After clearing and proofrolling the site surface as previously recommended, the surficial soils should be suitable to support the pavement sections. The embankment material should be compacted to a dry density of 98 percent of the modified Proctor (ASTM D1557/AASHTO T-180) maximum dry density of the compacted soil to a depth of one foot below the surface.

- The subbase material to a depth of twelve inches should have a minimum Limerock Bearing Ratio (LBR) value (FDOT FM 5-515) of 40 and it should be compacted to at least 98 percent of its modified Proctor (ASTM D1557 or AASHTO T-180) maximum dry density.

- The base course should consist of FDOT Optional Base Group 9, placed and compacted in two even layers and with each layer compacted to at least 98 percent of its modified Proctor maximum dry density.

- We recommend 2 inches of FDOT Type SP-9.5 and/or SP-12.5 asphaltic wearing surface. The two-inch wearing surface should be placed and compacted in two layers. Care must be exercised to place the asphalt over dry, well primed base material.

PUT ON DETAIL

PUT ON DETAIL

To stabilize internal drive-aisles that will not be paved, we recommend the following minimum section:

- Clear the site surface as needed and compact the sands to a dry density of 98 percent of the modified Proctor (ASTM D1557/AASHTO T-180) maximum dry density of the compacted soil to a depth of one foot below the surface.

- Place and compact 10 inches of crushed limerock, coquina or shell rock (LBR $\geq$ 100) in two even layers with each layer compacted to dry density of 98 percent of the modified Proctor (ASTM D1557/AASHTO T-180) maximum dry density of the compacted soil to a depth of one foot below the surface.

8.0 QUALITY CONTROL PROGRAM

We recommend establishing a comprehensive quality control program to verify that all site preparation and foundation and pavement construction is conducted in accordance with the appropriate plans and specifications. Materials testing and inspection services should be provided by Andersen Andre Consulting Engineers, Inc.

An experienced engineering technician should monitor all stripping and grubbing operations on a full-time basis to verify that deleterious materials have been removed. The technician should observe all compaction operations to verify that the appropriate number of passes are applied to the subgrade and that the subgrade soils exhibit an appropriate response to the compaction efforts. Further, the technician should monitor all berm construction, liner installation, pipe and structure installations, pile installations, etc.

In-situ density tests should be conducted during filling activities and below all footings, concrete slabs and pavement areas to verify that the required densities have been achieved. Similarly, density tests should be performed within all structure excavations and pipe trenches. In-situ density values should be compared to laboratory Proctor moisture-density results for each of the different natural and fill soils encountered. As such, representative samples of the various natural ground and fill soils, as well as stabilized subgrade and base materials, should be obtained and transported to our laboratory for Proctor compaction tests.

Finally, we recommend inspecting and testing the construction materials for the foundations and other structural components.

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9.0 Closure

The geotechnical evaluation submitted herein is based on the data obtained from the soil borings presented on Sheets No. 1-5 and our understanding of the proposed Moorhen Marsh LEAPS project as previously described. Limitations and conditions to this report are presented in Appendix V.

This report has been prepared in accordance with generally accepted soil and foundation engineering practices for the exclusive use of Indian River County BOCC for the 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.

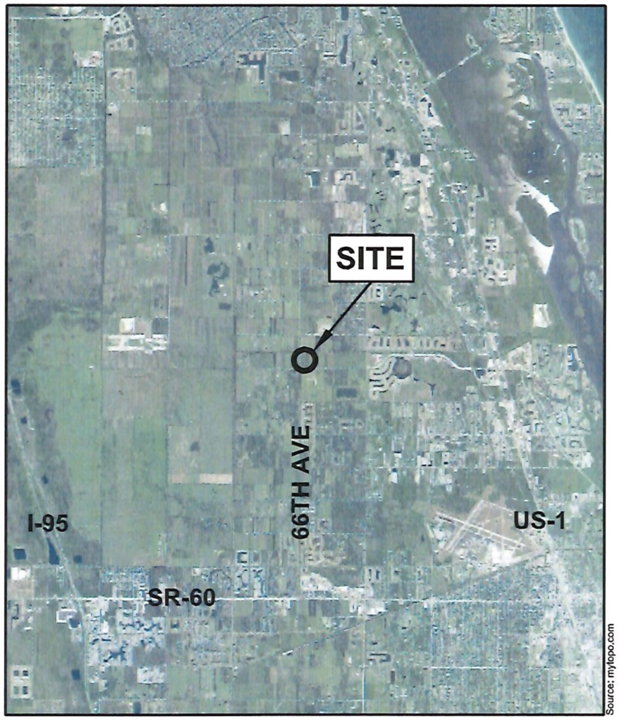
Sincerely,
Andersen Andre Consulting Engineers, Inc.
Certificate of Authorization No. 26794

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

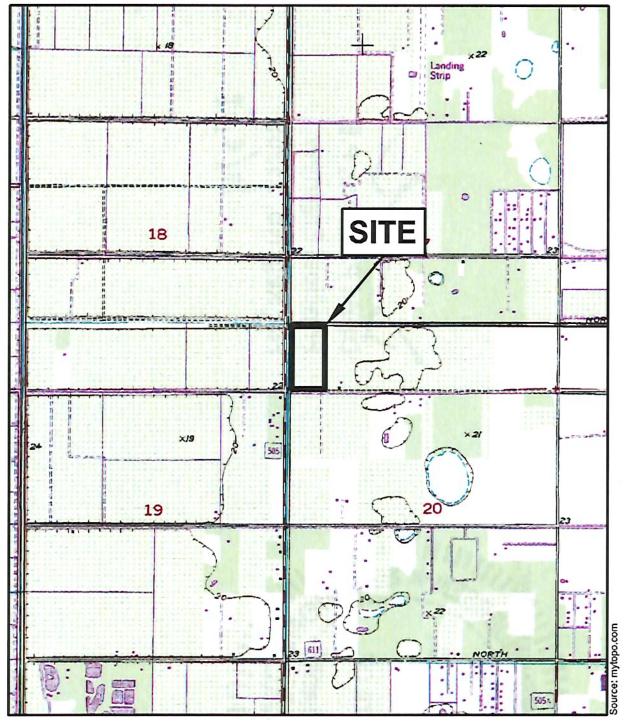
Peter G. Andersen, P.E.
Principal Engineer
Fla. Reg. No. 57956

David P. Andre, P.E.
Principal Engineer
Fla. Reg. No. 53969
10/9/19

2018 AERIAL PHOTOGRAPH



USGS TOPOGRAPHIC MAP
(1983 USGS Quadrangle Map of "Vero Beach, Florida")



USDA NRCS WEB SOIL SURVEY MAP

NORTH

NOT TO SCALE

PUBLIC LAND SURVEY SYSTEM

Section 17
Township 32 South
Range 39 East

INDIAN RIVER COUNTY PROPERTY APPRAISER

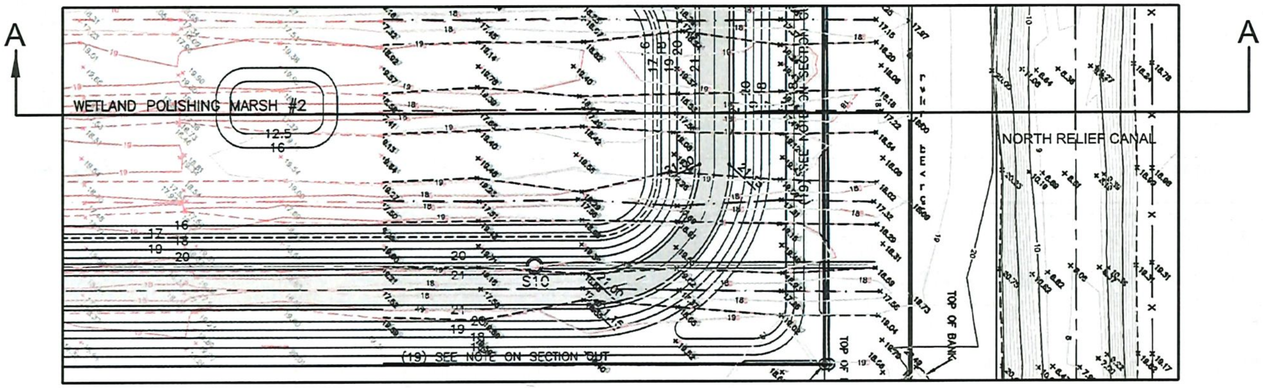
Parcel ID 32391700001013000001.0
&
Parcel ID 32391700001013000002.1

USDA NRCS SOIL SURVEY

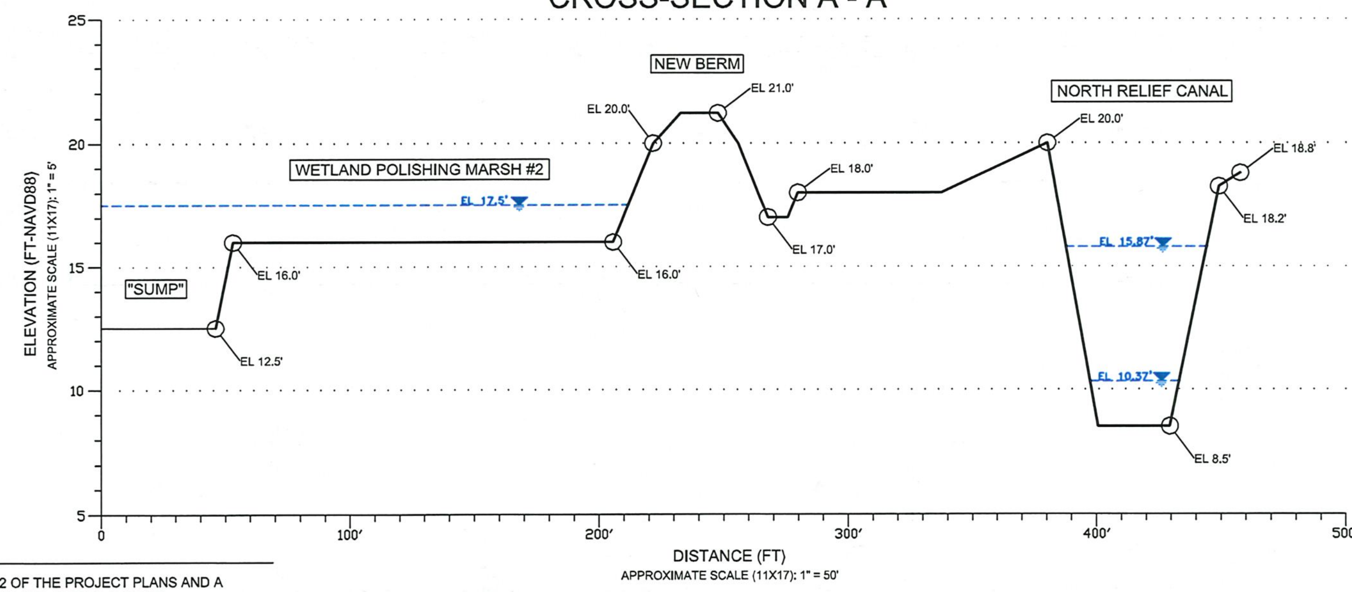
Winder fine sand, 0 to 2 percent slopes (14)
Pineda-Pineda, wet, fine sand, 0 to 2 percent slopes (16)
Boca fine sand (36)

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	SITE VICINITY MAPS	SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION MOORHEN MARSH LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS) INDIAN RIVER COUNTY, FLORIDA	Drawn by: PGA	Date: October 2019
			Checked by: DPA	Date: October 2019
			AACE File No: 19-140	**Figure No. 1**

SITE PLAN AND SURVEY OVERLAY (NOT TO SCALE)



CROSS-SECTION A - A



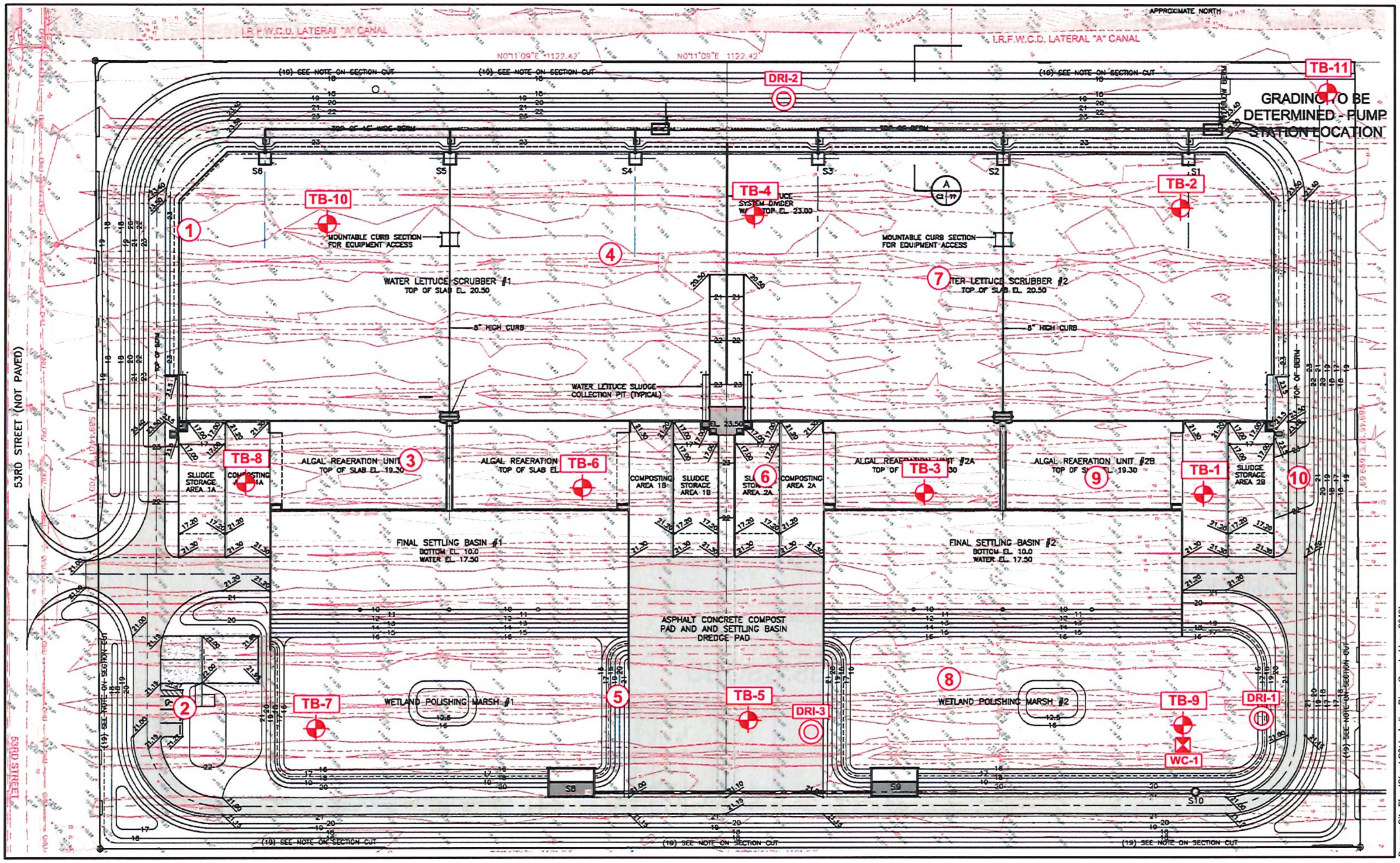
NOTES

CROSS-SECTION COMPILED FROM INFORMATION OBTAINED FROM SHEET C2 OF THE PROJECT PLANS AND A BOUNDARY/TOPOGRAPHIC SURVEY PREPARED BY CIVILSURV DESIGN GROUP, INC. (DATED 10/02/17)

WP MARSH #2 OPERATIONAL WATER LEVEL: EL 17.5'
NORTH RELIEF CANAL WATER LEVELS (PER INDIAN RIVER FARMS WATER CONTROL DISTRICT: EL 10.37' TO EL 15.87'

ELEVATION DATUM: NAVD88

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	CROSS-SECTION WP MARSH #2 AND NORTH RELIEF CANAL	SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION MOORHEN MARSH LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS) INDIAN RIVER COUNTY, FLORIDA	Drawn by: PGA Checked by: DPA AACE File No: 19-140	Date: October 2019 Date: October 2019 Figure No. 2



NORTH

NOT TO SCALE

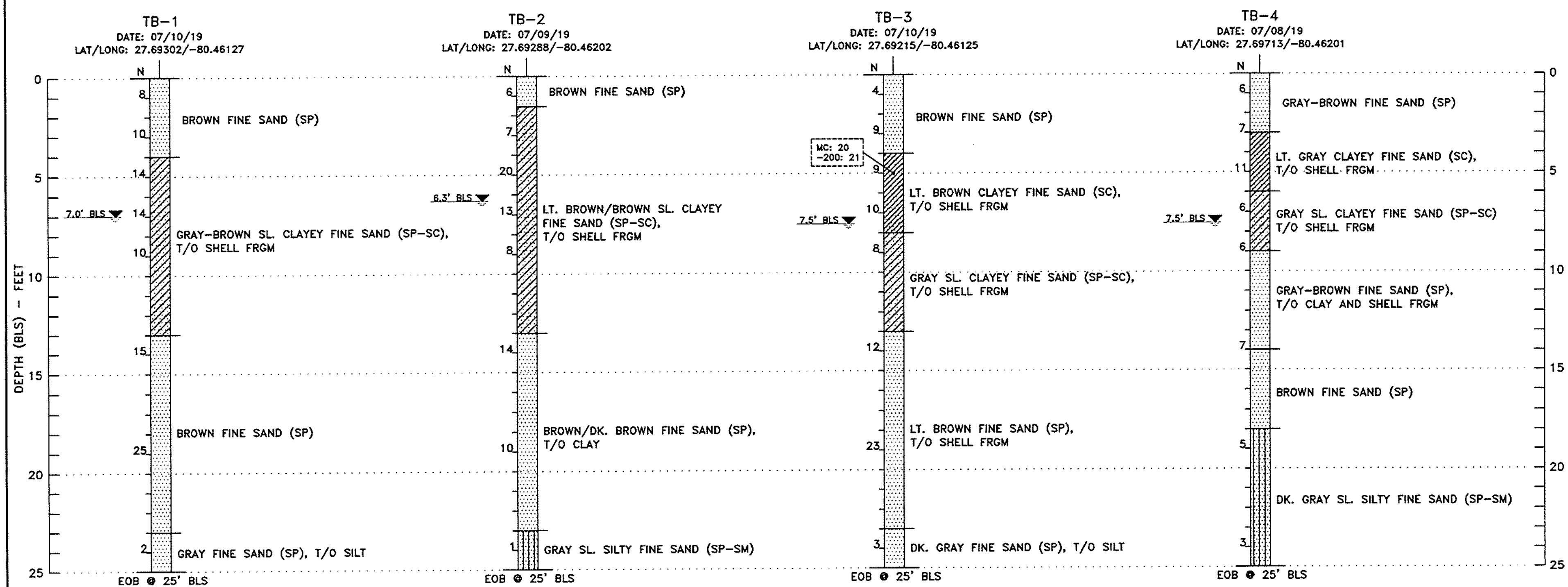
FIELD WORK LEGEND

- TB-# Standard Penetration Test Boring (TB)
- # Hand Auger Boring (HAB)
- WC-# Well Cluster (Temporary)
- DRI-# Double-Ring Infiltrometer Test

FIELD WORK NOTES

Shown and noted field work locations are approximate. All field work locations were located using the provided site plan, aerial photographs, existing site features, and hand-held WAAS enabled GPS instruments. Atmospheric disturbances and and local weather conditions may affect the accuracy of the GPS instrument readings. The shown field work locations should be considered accurate only to the degree implied by the method of measurement used.

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	**FIELD WORK LOCATION PLAN**	SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION MOORHEN MARSH LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS) INDIAN RIVER COUNTY, FLORIDA	Drawn by: PGA Checked by: DPA AACE File No: 19-140	Date: October 2019 Date: October 2019 **Figure No. 3**



SOIL GRAPHICAL LEGEND:

- FINE SAND (SP)
- SLIGHTLY SILTY FINE SAND (SP-SM)
- SL. CLAYEY FINE SAND (SP-SC)
- CLAYEY FINE SAND (SC)

3"-6" of topsoil encountered in most borings (not shown on profiles).

PROJECT NOTES:

TB-#	STANDARD PENETRATION TEST [SPT] BORING (ASTM D1586)
HAB-#	HAND AUGER BORING (ASTM D1452)
DRI-#	DOUBLE-RING INFILTROMETER TEST (ASTM D3385)
N	SPT RESISTANCE IN BLOWS PER FOOT
X.X'	GROUNDWATER TABLE (FT BELOW EXIST. GRADE) AT TIME DRILLED
EOB	END OF BORING
BLS	BELOW LAND SURFACE
SP, SP-SC, ETC:	UNIFIED SOIL CLASSIFICATION SYSTEM [USCS] USCS GROUPS DETERMINED BY VISUAL CLASSIFICATION EXCEPT FOR NOTED LABORATORY TESTS
MC	NATURAL MOISTURE CONTENT IN PERCENT (ASTM D2216)
-200	PERCENT PASSING NO. 200 SIEVE SIZE [PERCENT FINES] (ASTM D1140)

DRILL RIG: MOBILE B-57
DRILLING METHOD: ROTARY-WASH/BENTONITE SLURRY
HAMMER TYPE: SAFETY/MANUAL

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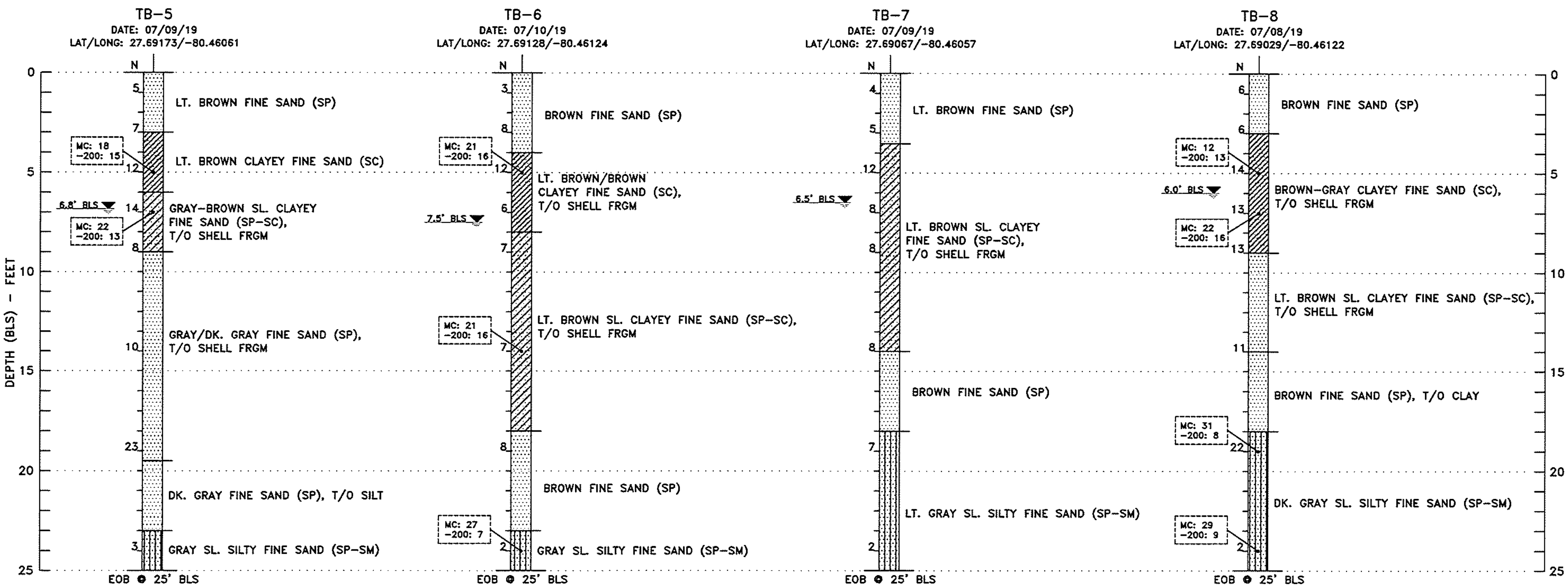
SOIL BORING PROFILES

SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION
MOORHEN MARSH
LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS)
INDIAN RIVER COUNTY, FLORIDA

Drawn by: PGA	Date: October 2019
Checked by: DPA	Date: October 2019
AACE File No: 19-140	**Sheet No. 1**



SOIL GRAPHICAL LEGEND:

- FINE SAND (SP)
- SLIGHTLY SILTY FINE SAND (SP-SM)
- SL. CLAYEY FINE SAND (SP-SC)
- CLAYEY FINE SAND (SC)

3"-6" of topsoil encountered in most borings (not shown on profiles).

PROJECT NOTES:

TB-#	STANDARD PENETRATION TEST [SPT] BORING (ASTM D1586)
HAB-#	HAND AUGER BORING (ASTM D1452)
DRI-#	DOUBLE-RING INFILTROMETER TEST (ASTM D3385)
N	SPT RESISTANCE IN BLOWS PER FOOT
X.X' ▼	GROUNDWATER TABLE (FT BELOW EXIST. GRADE) AT TIME DRILLED
EOB	END OF BORING
BLS	BELOW LAND SURFACE
SP, SP-SC, ETC:	UNIFIED SOIL CLASSIFICATION SYSTEM [USCS] USCS GROUPS DETERMINED BY VISUAL CLASSIFICATION EXCEPT FOR NOTED LABORATORY TESTS
MC	NATURAL MOISTURE CONTENT IN PERCENT (ASTM D2216)
-200	PERCENT PASSING NO. 200 SIEVE SIZE [PERCENT FINES] (ASTM D1140)

DRILL RIG: MOBILE B-57
DRILLING METHOD: ROTARY-WASH/BENTONITE SLURRY
HAMMER TYPE: SAFETY/MANUAL

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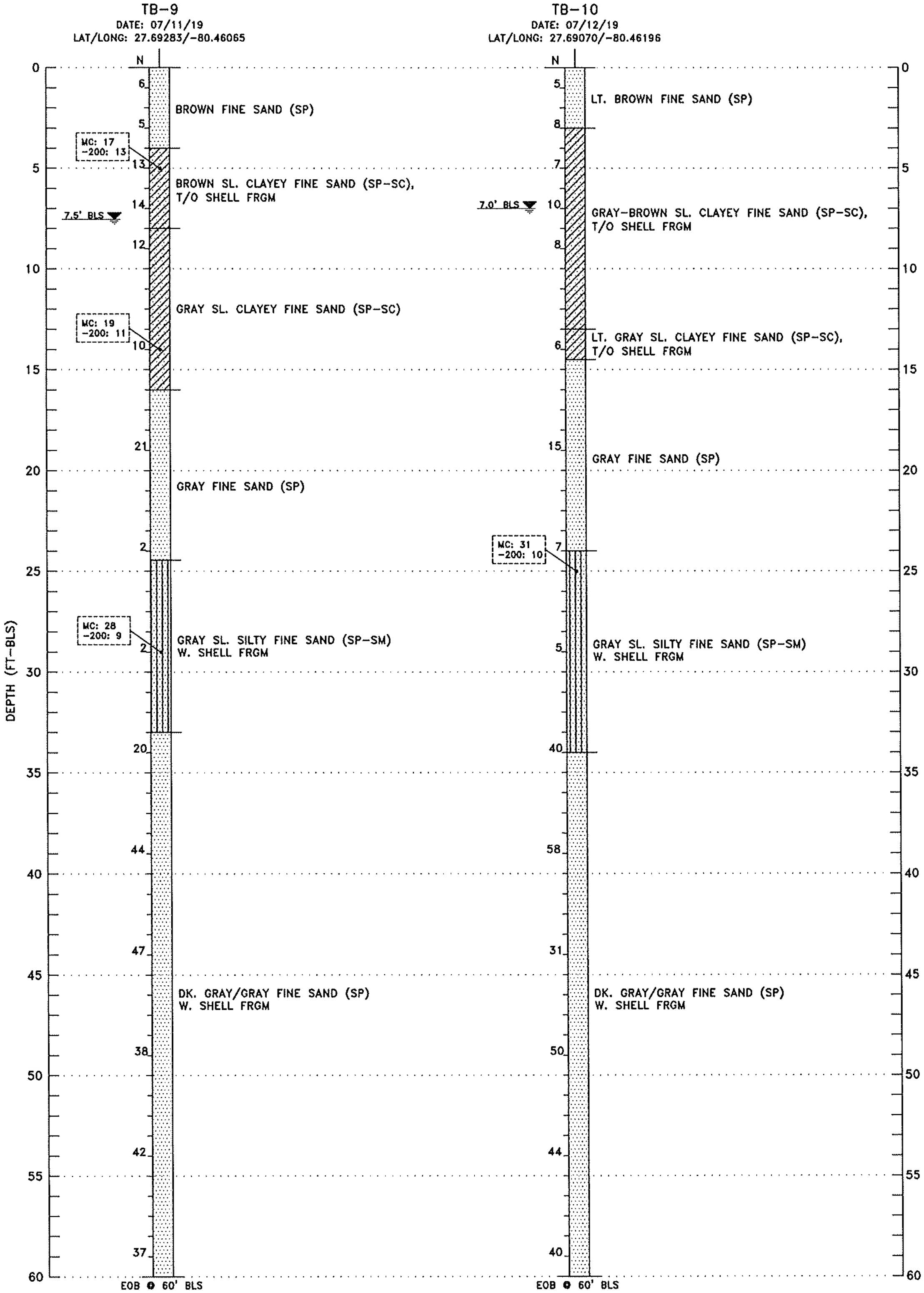
SOIL BORING PROFILES

SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION
MOORHEN MARSH
LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS)
INDIAN RIVER COUNTY, FLORIDA

Drawn by: PGA	Date: October 2019
Checked by: DPA	Date: October 2019
AACE File No: 19-140	Sheet No. 2



SOIL GRAPHICAL LEGEND:

- FINE SAND (SP)
- SLIGHTLY SILTY FINE SAND (SP-SM)
- SL. CLAYEY FINE SAND (SP-SC)
- CLAYEY FINE SAND (SC)

3"-6" of topsoil encountered in most borings (not shown on profiles).

PROJECT NOTES:

TB-# STANDARD PENETRATION TEST [SPT] BORING (ASTM D1586)
HAB-# HAND AUGER BORING (ASTM D1452)
DRI-# DOUBLE-RING INFILTROMETER TEST (ASTM D3385)
N SPT RESISTANCE IN BLOWS PER FOOT
X.X' GROUNDWATER TABLE (FT BELOW EXIST. GRADE) AT TIME DRILLED
EOB END OF BORING
BLS BELOW LAND SURFACE
SP, SP-SC, ETC: UNIFIED SOIL CLASSIFICATION SYSTEM [USCS] USCS GROUPS DETERMINED BY VISUAL CLASSIFICATION EXCEPT FOR NOTED LABORATORY TESTS
MC NATURAL MOISTURE CONTENT IN PERCENT (ASTM D2216)
-200 PERCENT PASSING NO. 200 SIEVE SIZE [PERCENT FINES] (ASTM D1140)
DRILL RIG: MOBILE B-57
DRILLING METHOD: ROTARY-WASH/BENTONITE SLURRY
HAMMER TYPE: SAFETY/MANUAL

Drawn by: PGA
Date: October 2019
Checked by: DPA
Date: October 2019
AACE File No: 19-140
Sheet No. 3

SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION
MOORHEN MARSH
LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS)
INDIAN RIVER COUNTY, FLORIDA

SOIL BORING PROFILES

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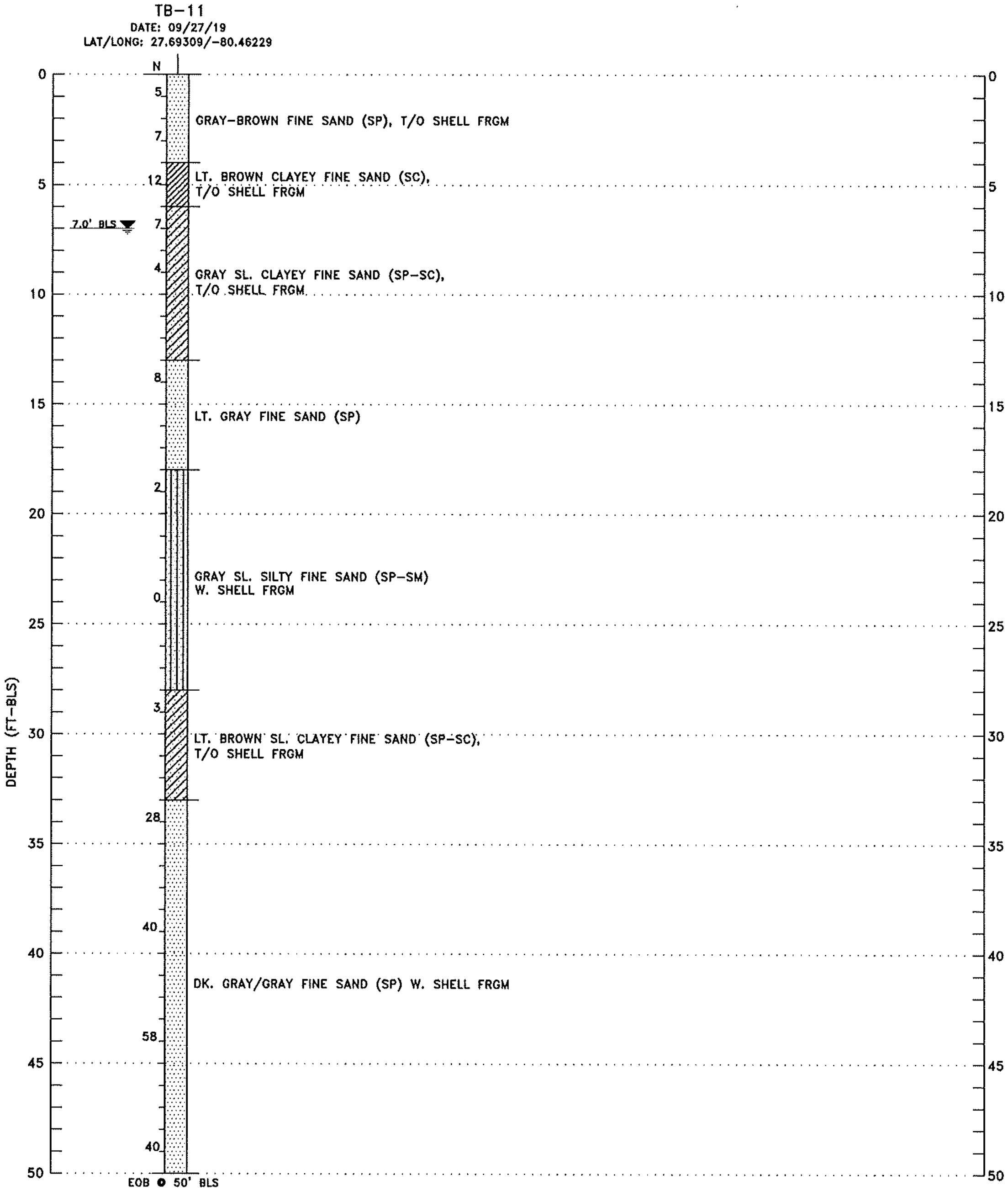

SOIL GRAPHICAL LEGEND:

- FINE SAND (SP)
- SLIGHTLY SILTY FINE SAND (SP-SM)
- SL. CLAYEY FINE SAND (SP-SC)
- CLAYEY FINE SAND (SC)

3"-6" of topsoil encountered in most borings (not shown on profiles).

PROJECT NOTES:

TB-#	STANDARD PENETRATION TEST [SPT] BORING (ASTM D1586)
HAB-#	HAND AUGER BORING (ASTM D1452)
DRI-#	DOUBLE-RING INFILTROMETER TEST (ASTM D3385)
N	SPT RESISTANCE IN BLOWS PER FOOT
X.X'	GROUNDWATER TABLE (FT BELOW EXIST. GRADE) AT TIME DRILLED
EOB	END OF BORING
BLS	BELOW LAND SURFACE
SP, SP-SC, ETC:	UNIFIED SOIL CLASSIFICATION SYSTEM [USCS] USCS GROUPS DETERMINED BY VISUAL CLASSIFICATION EXCEPT FOR NOTED LABORATORY TESTS
MC	NATURAL MOISTURE CONTENT IN PERCENT (ASTM D2216)
-200	PERCENT PASSING NO. 200 SIEVE SIZE [PERCENT FINES] (ASTM D1140)

DRILL RIG: MOBILE B-57
DRILLING METHOD: ROTARY-WASH/BENTONITE SLURRY
HAMMER TYPE: SAFETY/MANUAL

Date: October 2019
Date: October 2019
Sheet No. 4
Drawn by: PGA
Checked by: DPA
AACE File No: 19-140

SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION
MOORHEN MARSH
LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS)
INDIAN RIVER COUNTY, FLORIDA

SOIL BORING PROFILES

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.
834 SW Swan Avenue, Port St. Lucie, FL 34983 772-807-9191 www.AACEinc.com
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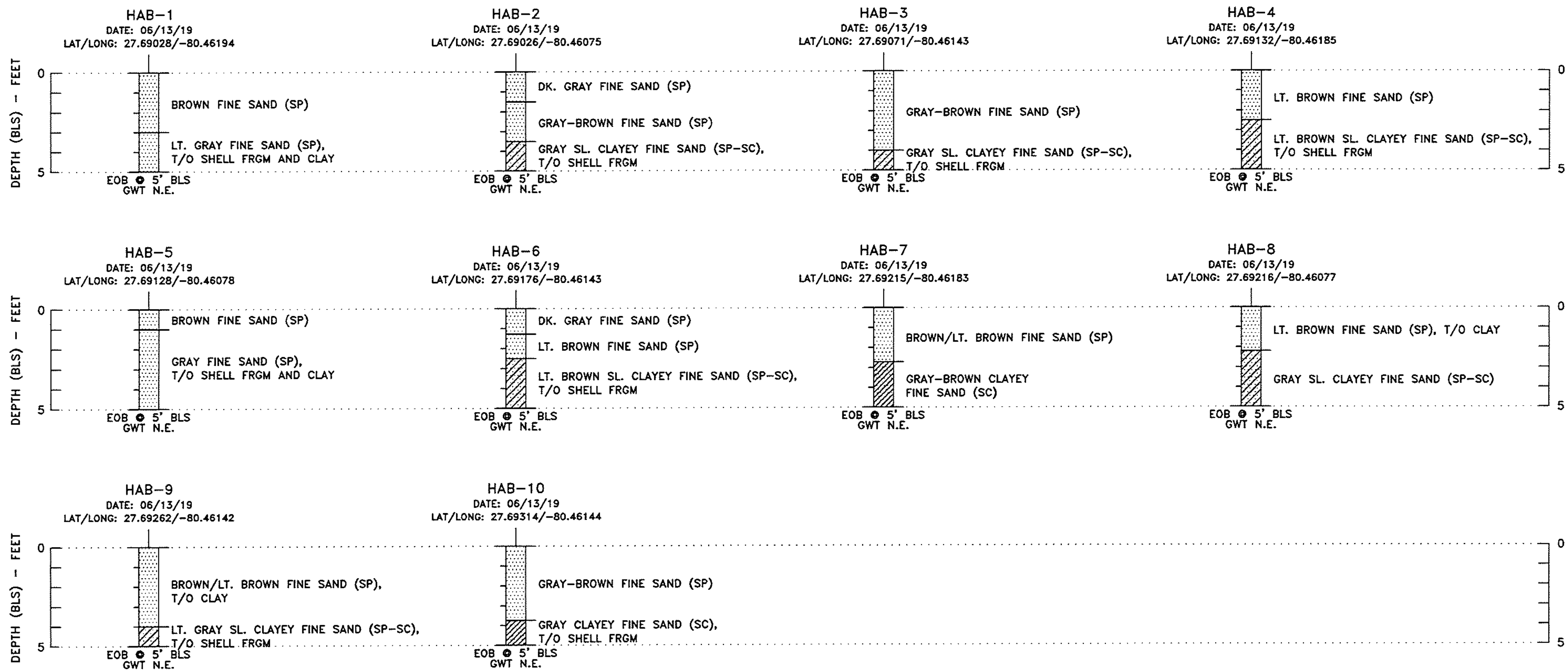
AACE



SOIL GRAPHICAL LEGEND:

- FINE SAND (SP)
- SLIGHTLY SILTY FINE SAND (SP-SM)
- SL. CLAYEY FINE SAND (SP-SC)
- CLAYEY FINE SAND (SC)

3"-6" of topsoil encountered in most borings (not shown on profiles).

PROJECT NOTES:

TB-#	STANDARD PENETRATION TEST [SPT] BORING (ASTM D1586)
HAB-#	HAND AUGER BORING (ASTM D1452)
DRI-#	DOUBLE-RING INFILTROMETER TEST (ASTM D3385)
N	SPT RESISTANCE IN BLOWS PER FOOT
X.X' ▼	GROUNDWATER TABLE (FT BELOW EXIST. GRADE) AT TIME DRILLED
EOB	END OF BORING
BLS	BELOW LAND SURFACE
SP, SP-SC, ETC:	UNIFIED SOIL CLASSIFICATION SYSTEM [USCS] USCS GROUPS DETERMINED BY VISUAL CLASSIFICATION EXCEPT FOR NOTED LABORATORY TESTS
MC	NATURAL MOISTURE CONTENT IN PERCENT (ASTM D2216)
-200	PERCENT PASSING NO. 200 SIEVE SIZE [PERCENT FINES] (ASTM D1140)

DRILL RIG: MOBILE B-57
DRILLING METHOD: ROTARY-WASH/BENTONITE SLURRY
HAMMER TYPE: SAFETY/MANUAL

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**	SUBSURFACE SOIL EXPLORATION AND GEOTECHNICAL ENGINEERING EVALUATION MOORHEN MARSH LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS) INDIAN RIVER COUNTY, FLORIDA	Drawn by: PGA	Date: October 2019
			Checked by: DPA	Date: October 2019
			AACE File No: 19-140	**Sheet No. 5**

APPENDIX I

USDA NRCS Web Soil Survey
Summary Report

United States Department of Agriculture

NRCS

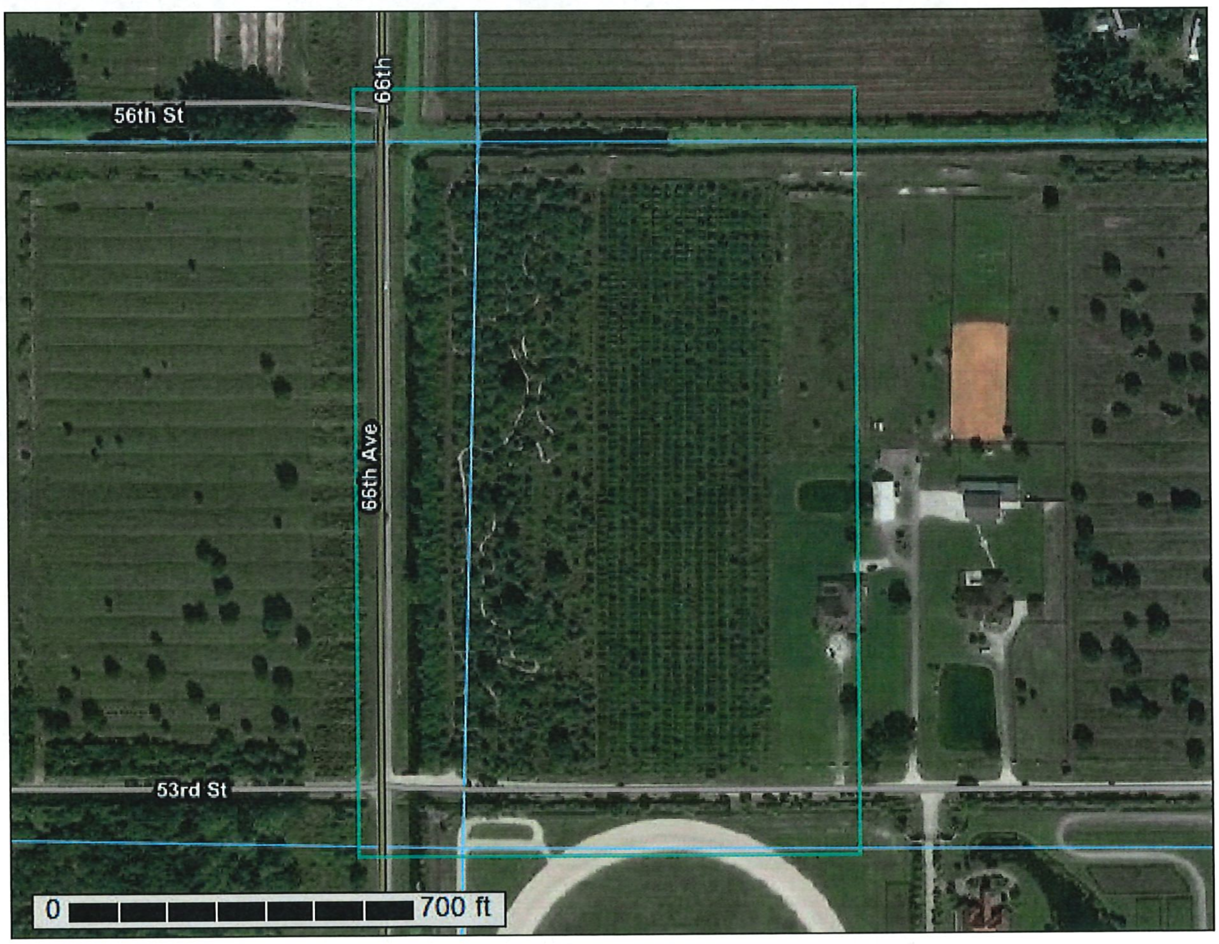
Natural Resources Conservation Service

A product of the National Cooperative Soil Survey, a joint effort of the United States Department of Agriculture and other Federal agencies, State agencies including the Agricultural Experiment Stations, and local participants

Custom Soil Resource Report for

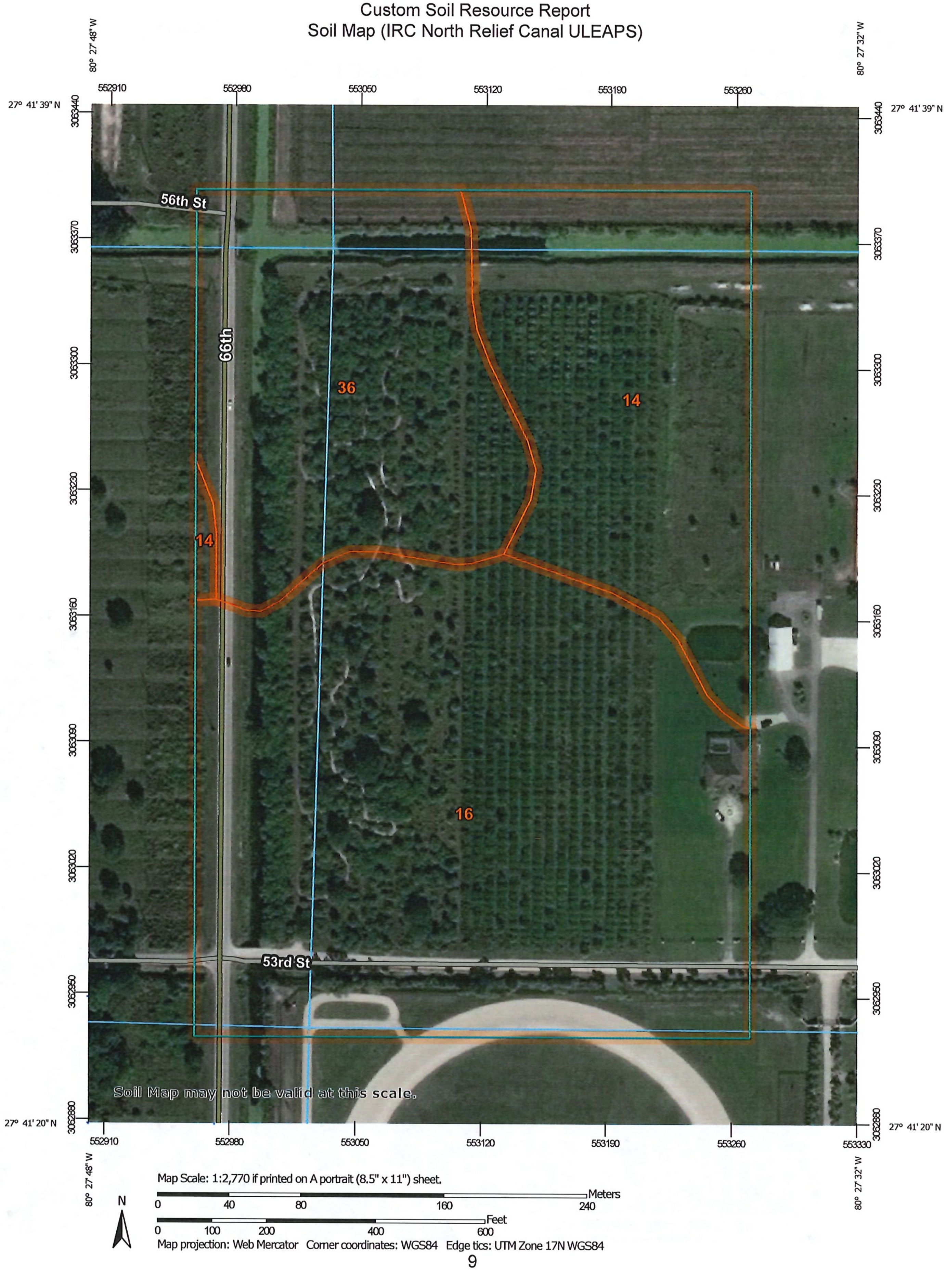
Indian River County, Florida

IRC North Relief Canal ULEAPS



July 17, 2019

Custom Soil Resource Report
Soil Map (IRC North Relief Canal ULEAPS)



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 1:20,000.

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:
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: Indian River County, Florida
Survey Area Data: Version 17, Sep 13, 2018

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

Date(s) aerial images were photographed: Nov 21, 2018—Dec 23, 2018

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 (IRC North Relief Canal ULEAPS)

Map Unit Symbol	Map Unit Name	Acres in AOI	Percent of AOI
14	Winder fine sand, 0 to 2 percent slopes	8.6	23.6%
16	Pineda-Pineda, wet, fine sand, 0 to 2 percent slopes	19.0	52.3%
36	Boca fine sand	8.7	24.1%
Totals for Area of Interest		**36.3**	**100.0%**

Map Unit Descriptions (IRC North Relief Canal ULEAPS)

The map units delineated on the detailed soil maps in a soil survey represent the soils or miscellaneous areas in the survey area. The map unit descriptions, along with the maps, can be used to determine the composition and properties of a unit.

A map unit delineation on a soil map represents an area dominated by one or more major kinds of soil or miscellaneous areas. A map unit is identified and named according to the taxonomic classification of the dominant soils. Within a taxonomic class there are precisely defined limits for the properties of the soils. On the landscape, however, the soils are natural phenomena, and they have the characteristic variability of all natural phenomena. Thus, the range of some observed properties may extend beyond the limits defined for a taxonomic class. Areas of soils of a single taxonomic class rarely, if ever, can be mapped without including areas of other taxonomic classes. Consequently, every map unit is made up of the soils or miscellaneous areas for which it is named and some minor components that belong to taxonomic classes other than those of the major soils.

Most minor soils have properties similar to those of the dominant soil or soils in the map unit, and thus they do not affect use and management. These are called noncontrasting, or similar, components. They may or may not be mentioned in a particular map unit description. Other minor components, however, have properties and behavioral characteristics divergent enough to affect use or to require different management. These are called contrasting, or dissimilar, components. They generally are in small areas and could not be mapped separately because of the scale used. Some small areas of strongly contrasting soils or miscellaneous areas are identified by a special symbol on the maps. If included in the database for a given area, the contrasting minor components are identified in the map unit descriptions along with some characteristics of each. A few areas of minor components may not have been observed, and consequently they are not mentioned in the descriptions, especially where the pattern was so complex that it was impractical to make enough observations to identify all the soils and miscellaneous areas on the landscape.

The presence of minor components in a map unit in no way diminishes the usefulness or accuracy of the data. The objective of mapping is not to delineate pure taxonomic classes but rather to separate the landscape into landforms or landform segments that have similar use and management requirements. The delineation of such segments on the map provides sufficient information for the development of resource plans. If intensive use of small areas is planned, however, onsite investigation is needed to define and locate the soils and miscellaneous areas.

An identifying symbol precedes the map unit name in the map unit descriptions. Each description includes general facts about the unit and gives important soil properties and qualities.

Soils that have profiles that are almost alike make up a *soil series*. Except for differences in texture of the surface layer, all the soils of a series have major horizons that are similar in composition, thickness, and arrangement.

Soils of one series can differ in texture of the surface layer, slope, stoniness, salinity, degree of erosion, and other characteristics that affect their use. On the basis of such differences, a soil series is divided into *soil phases*. Most of the areas shown on the detailed soil maps are phases of soil series. The name of a soil phase commonly indicates a feature that affects use or management. For example, Alpha silt loam, 0 to 2 percent slopes, is a phase of the Alpha series.

Some map units are made up of two or more major soils or miscellaneous areas. These map units are complexes, associations, or undifferentiated groups.

A *complex* consists of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the maps. The pattern and proportion of the soils or miscellaneous areas are somewhat similar in all areas. Alpha-Beta complex, 0 to 6 percent slopes, is an example.

An *association* is made up of two or more geographically associated soils or miscellaneous areas that are shown as one unit on the maps. Because of present or anticipated uses of the map units in the survey area, it was not considered practical or necessary to map the soils or miscellaneous areas separately. The pattern and relative proportion of the soils or miscellaneous areas are somewhat similar. Alpha-Beta association, 0 to 2 percent slopes, is an example.

An *undifferentiated group* is made up of two or more soils or miscellaneous areas that could be mapped individually but are mapped as one unit because similar interpretations can be made for use and management. The pattern and proportion of the soils or miscellaneous areas in a mapped area are not uniform. An area can be made up of only one of the major soils or miscellaneous areas, or it can be made up of all of them. Alpha and Beta soils, 0 to 2 percent slopes, is an example.

Some surveys include *miscellaneous areas*. Such areas have little or no soil material and support little or no vegetation. Rock outcrop is an example.

Indian River County, Florida

14—Winder fine sand, 0 to 2 percent slopes

Map Unit Setting

National map unit symbol: 2svzt
Elevation: 0 to 70 feet
Mean annual precipitation: 47 to 60 inches
Mean annual air temperature: 70 to 77 degrees F
Frost-free period: 355 to 365 days
Farmland classification: Farmland of unique importance

Map Unit Composition

Winder and similar soils: 85 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Winder

Setting

Landform: Flats on marine terraces, drainageways on marine terraces, depressions on marine terraces
Landform position (three-dimensional): Tread, talf, dip
Down-slope shape: Linear, convex, concave
Across-slope shape: Linear, concave
Parent material: Sandy and loamy marine deposits

Typical profile

A - 0 to 7 inches: fine sand
E - 7 to 17 inches: fine sand
B/E - 17 to 23 inches: sandy loam
Btg1 - 23 to 34 inches: sandy loam
Btg2 - 34 to 48 inches: sandy loam
BCg - 48 to 65 inches: sandy loam
Ckg - 65 to 80 inches: loamy sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: Very high
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 3 to 18 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 11 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 6.9 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w

Hydrologic Soil Group: C/D
Forage suitability group: Loamy and clayey soils on flats of hydric or mesic lowlands (G156BC341FL)
Other vegetative classification: Wetland Hardwood Hammock (R156BY012FL)
Hydric soil rating: Yes

Minor Components

Chobee

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
Ecological site: Freshwater Marshes and Ponds (R155XY010FL)
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL)
Hydric soil rating: Yes

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)
Hydric soil rating: Yes

Manatee

Percent of map unit: 4 percent
Landform: Drainageways on marine terraces, depressions on marine terraces
Landform position (three-dimensional): Tread, dip
Down-slope shape: Convex, linear, concave
Across-slope shape: Linear, concave
Other vegetative classification: Freshwater Marshes and Ponds (R155XY010FL)
Hydric soil rating: Yes

Riviera

Percent of map unit: 2 percent
Landform: Drainageways on marine terraces, flats 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)
Hydric soil rating: Yes

16—Pineda-Pineda, wet, fine sand, 0 to 2 percent slopes

Map Unit Setting

National map unit symbol: 2svyp
Elevation: 0 to 100 feet

Mean annual precipitation: 42 to 63 inches
Mean annual air temperature: 68 to 77 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Farmland of unique importance

Map Unit Composition

Pineda and similar soils: 45 percent
Pineda, wet, and similar soils: 40 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Pineda

Setting

Landform: Flatwoods on marine terraces, drainageways on marine terraces
Landform position (three-dimensional): Tread, talf, dip
Down-slope shape: Linear
Across-slope shape: Linear, concave
Parent material: Sandy and loamy marine deposits

Typical profile

A - 0 to 1 inches: fine sand
E - 1 to 5 inches: fine sand
Bw - 5 to 36 inches: fine sand
Btg/E - 36 to 54 inches: fine sandy loam
Cg - 54 to 80 inches: fine sand

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: Very high
Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: About 6 to 18 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 15 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: Low (about 5.7 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: A/D
Forage suitability group: Sandy over loamy soils on flats of hydric or mesic lowlands (G155XB241FL)
Other vegetative classification: South Florida Flatwoods (R155XY003FL)
Hydric soil rating: No

Description of Pineda, Wet

Setting

Landform: Drainageways on marine terraces, flats on marine terraces
Landform position (three-dimensional): Tread, dip, talf

Down-slope shape: Linear
Across-slope shape: Concave, linear
Parent material: Sandy and loamy marine deposits

Typical profile

A - 0 to 1 inches: fine sand
E - 1 to 5 inches: fine sand
Bw - 5 to 36 inches: fine sand
Btg/E - 36 to 54 inches: fine sandy loam
Cg - 54 to 80 inches: fine sand

Properties and qualities

Slope: 0 to 1 percent
Depth to restrictive feature: More than 80 inches
Natural drainage class: Poorly drained
Runoff class: Negligible
Capacity of the most limiting layer to transmit water (Ksat): High (1.98 to 5.95 in/hr)
Depth to water table: About 0 to 18 inches
Frequency of flooding: None
Frequency of ponding: Frequent
Calcium carbonate, maximum in profile: 15 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: Low (about 5.7 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: A/D
Forage suitability group: Sandy over loamy soils on flats of hydric or mesic lowlands (G155XB241FL)
Other vegetative classification: Slough (R155XY011FL)
Hydric soil rating: Yes

Minor Components

Felda

Percent of map unit: 6 percent
Landform: Drainageways on marine terraces, flats on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Linear
Across-slope shape: Concave, linear
Other vegetative classification: Slough (R155XY011FL)
Hydric soil rating: Yes

Wabasso

Percent of map unit: 3 percent
Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Tread, talf
Down-slope shape: Convex, linear
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL)
Hydric soil rating: No

Valkaria

Percent of map unit: 2 percent
Landform: Drainageways on flats on marine terraces
Landform position (three-dimensional): Tread, dip, talf
Down-slope shape: Linear
Across-slope shape: Linear, concave
Other vegetative classification: Slough (R155XY011FL)
Hydric soil rating: Yes

Boca

Percent of map unit: 2 percent
Landform: Flats on marine terraces, drainageways on marine terraces
Landform position (three-dimensional): Tread, talf, dip
Down-slope shape: Convex, linear
Across-slope shape: Linear, concave
Other vegetative classification: South Florida Flatwoods (R155XY003FL)
Hydric soil rating: Yes

Hallandale

Percent of map unit: 2 percent
Landform: Flatwoods on marine terraces
Landform position (three-dimensional): Tread, talf
Down-slope shape: Linear
Across-slope shape: Linear
Other vegetative classification: South Florida Flatwoods (R155XY003FL)
Hydric soil rating: Yes

36—Boca fine sand

Map Unit Setting

National map unit symbol: tdgl
Elevation: 0 to 200 feet
Mean annual precipitation: 52 to 60 inches
Mean annual air temperature: 68 to 75 degrees F
Frost-free period: 350 to 365 days
Farmland classification: Not prime farmland

Map Unit Composition

Boca, non-hydric, and similar soils: 60 percent
Boca, hydric, and similar soils: 25 percent
Minor components: 15 percent
Estimates are based on observations, descriptions, and transects of the mapunit.

Description of Boca, Non-hydric

Setting

Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex

Across-slope shape: Linear
Parent material: Sandy and loamy marine deposits over limestone

Typical profile

A - 0 to 7 inches: fine sand
E - 7 to 20 inches: fine sand
Bt - 20 to 24 inches: fine sandy loam
2R - 24 to 28 inches: weathered bedrock

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: 20 to 40 inches to lithic bedrock
Natural drainage class: Poorly drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.20 to 1.98 in/hr)
Depth to water table: About 6 to 18 inches
Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 10 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: Very low (about 2.1 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: C/D
Forage suitability group: Sandy over loamy soils on flats of hydric or mesic lowlands (G156BC241FL)
Other vegetative classification: South Florida Flatwoods (R155XY003FL)
Hydric soil rating: No

Description of Boca, Hydric

Setting

Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Linear
Across-slope shape: Linear
Parent material: Sandy and loamy marine deposits over limestone

Typical profile

A - 0 to 7 inches: fine sand
E - 7 to 20 inches: fine sand
Bt - 20 to 24 inches: fine sandy loam
2R - 24 to 28 inches: weathered bedrock

Properties and qualities

Slope: 0 to 2 percent
Depth to restrictive feature: 20 to 40 inches to lithic bedrock
Natural drainage class: Poorly drained
Runoff class: High
Capacity of the most limiting layer to transmit water (Ksat): Moderately high to high (0.20 to 1.98 in/hr)
Depth to water table: About 0 to 12 inches

Frequency of flooding: None
Frequency of ponding: None
Calcium carbonate, maximum in profile: 10 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: Very low (about 2.1 inches)

Interpretive groups

Land capability classification (irrigated): None specified
Land capability classification (nonirrigated): 3w
Hydrologic Soil Group: C/D
Forage suitability group: Sandy over loamy soils on flats of hydric or mesic lowlands (G156BC241FL)
Other vegetative classification: South Florida Flatwoods (R155XY003FL)
Hydric soil rating: Yes

Minor Components

Riviera

Percent of map unit: 5 percent
Landform: Drainageways on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Linear
Across-slope shape: Concave
Other vegetative classification: Cabbage Palm Flatwoods (R155XY005FL)
Hydric soil rating: Yes

Pineda

Percent of map unit: 5 percent
Landform: Drainageways on marine terraces
Landform position (three-dimensional): Dip
Down-slope shape: Linear
Across-slope shape: Concave
Other vegetative classification: Slough (R155XY011FL)
Hydric soil rating: Yes

Jupiter, non-hydric

Percent of map unit: 5 percent
Landform: Flats on marine terraces
Landform position (three-dimensional): Talf
Down-slope shape: Convex
Across-slope shape: Linear
Other vegetative classification: Upland Hardwood Hammock (R155XY008FL)
Hydric soil rating: No

APPENDIX II

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 ≤ 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 III

Piezometer Data
and Permeability Test Results

Summary of Field Well Permeability Tests							
Boring ID	Piezometer ID	Test Depth [ft]	Soil Description	Constant Head Test		Falling Head Test	
				Kh [cm/sec]	Kh [ft/day]	Kh [cm/sec]	Kh [ft/day]
TB-9	TB-9 (15)	10-15	Slightly clayey fine sand	2.87E-03	8.1	2.75E-03	7.8
TB-9	TB-9 (23)	18-23	Fine sand	1.11E-02	31.5	1.08E-02	30.6
TB-9	TB-9 (30)	25-30	Slightly silty fine sand	2.50E-04	0.7	2.60E-04	0.7
TB-9	TB-9 (40)	35-40	Fine sand	1.98E-02	56.1	2.01E-02	57.0

Test ID: TB-9 (15)
Test Depth: 10'-15'
No of Tests: 3 (average below)

Constant Head Permeability Test

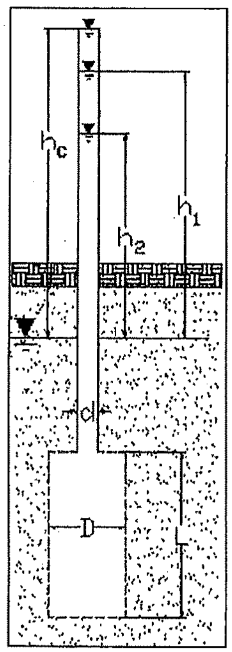
D:	8.00 in	= 20.3 cm
q:	3.30 gpm	= 208.2 cm^3/s
L:	5.00 ft	= 152.4 cm
H_c:	6.75 ft	= 205.7 cm
k_h/k_v :	1	
m:	1	

k_h : 2.87E-03 cm/s
8.13 ft/day

Variable Head Permeability Test
(48 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:	6.30 ft	= 192.0 cm
h_2:	2.30 ft	= 70.1 cm
t:	21.00 sec	
k_h/k_v :	1	
m:	1	

k_h : 2.75E-03 cm/s
7.80 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: TB-9 (23)
Test Depth: 18'-23'
No of Tests: 3 (average below)

Constant Head Permeability Test

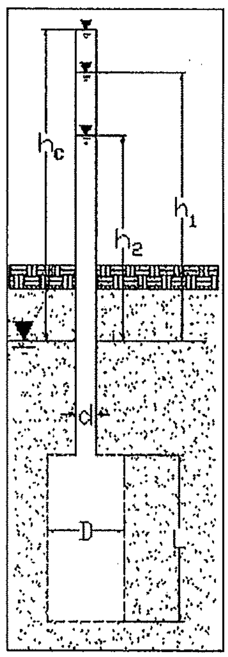
D:	8.00 in	= 20.3 cm
q:	13.70 gpm	= 864.3 cm^3/s
L:	5.00 ft	= 152.4 cm
H_c:	7.23 ft	= 220.4 cm
k_h/k_v :	1	
m:	1	

k_h : 1.11E-02 cm/s
31.49 ft/day

Variable Head Permeability Test
(48 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:	4.90 ft	= 149.4 cm
h_2:	0.90 ft	= 27.4 cm
t:	9.00 sec	
k_h/k_v :	1	
m:	1	

k_h : 1.08E-02 cm/s
30.59 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: TB-9 (30)
Test Depth: 25'-30'
No of Tests: 3 (average below)

Constant Head Permeability Test

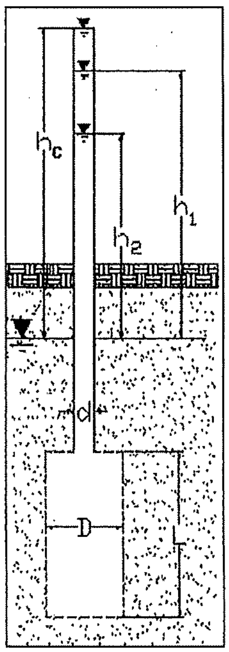
D:	8.00 in	= 20.3 cm
q:	0.30 gpm	= 18.9 cm^3/s
L:	5.00 ft	= 152.4 cm
H_c:	6.90 ft	= 210.3 cm
k_h/k_v :	1	
m:	1	

k_h : 2.55E-04 cm/s
0.72 ft/day

Variable Head Permeability Test
(48 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:	6.90 ft	= 210.3 cm
h_2:	2.90 ft	= 88.4 cm
t:	190.00 sec	
k_h/k_v :	1	
m:	1	

k_h : 2.62E-04 cm/s
0.74 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: TB-9 (40)
Test Depth: 35'-40'
No of Tests: 3 (average below)

Constant Head Permeability Test

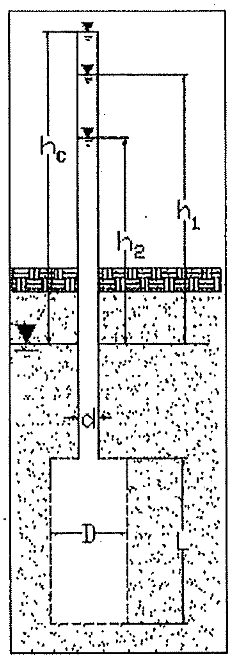
D:	8.00 in	= 20.3 cm
q:	22.60 gpm	= 1425.8 cm^3/s
L:	5.00 ft	= 152.4 cm
H_c:	6.70 ft	= 204.2 cm
k_h/k_v :	1	
m:	1	

k_h : 1.98E-02 cm/s
56.06 ft/day

Variable Head Permeability Test
(48 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:	5.30 ft	= 161.5 cm
h_2:	1.30 ft	= 39.6 cm
t:	4.00 sec	
k_h/k_v :	1	
m:	1	

k_h : 2.01E-02 cm/s
57.09 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)

APPENDIX IV

Double-Ring Infiltrometer Test Results

ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

Double-Ring Infiltrometer Test Report (ASTM D3385)

Test Number	**DRI-1**	**Technician**	RL/KC
Project Name	Moorhen Marsh LEAPS	**Engineer**	PGA
Project Number	19-140	**Weather Conditions**	Clear
Test Location	Refer to Figure No. 2	**Temperature**	95F
Date	09/16/2019	**Testing Liquid**	Water

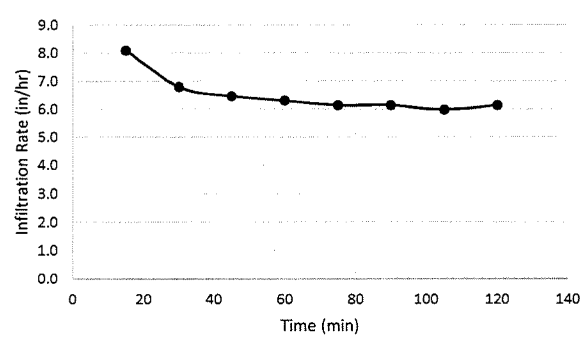
SOIL PROFILE	
Depth (ft-bls)	**Description**
0-3	Gray-brown fine sand (SP)
3-4	Lt. brown slightly clayey fine sand (SP-SC)
	EOB @ 4-ft
Groundwater Not Encountered	

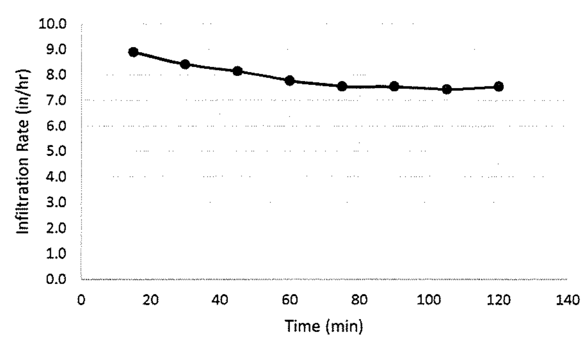
Area of Inner Ring (in^2):	113.1	**Test Depth (in):**	12	**Depth of Water - Inner Ring (in):**	4
Area of Annular Space (in^2):	339.3	**Ring Seating (in):**	6	**Depth of Water - Annular Space (in):**	4

Cycle No.	Incremental Test Time (min)	INNER RING			ANNULAR SPACE		
		Incremental Flow (ml)	Incremental Flow (in^3)	Infiltration Rate (in/hr)	Incremental Flow (ml)	Incremental Flow (in^3)	Infiltration Rate (in/hr)
1 (15 min)	1	250	15.3	8.09	825	50.3	8.90
2 (30 min)	1	210	12.8	6.80	780	47.6	8.42
3 (45 min)	1	200	12.2	6.47	755	46.1	8.15
4 (60 min)	1	195	11.9	6.31	720	43.9	7.77
5 (75 min)	1	190	11.6	6.15	700	42.7	7.55
6 (90 min)	1	190	11.6	6.15	700	42.7	7.55
7 (105 min)	1	185	11.3	5.99	690	42.1	7.45
8 (120 min)	1	190	11.6	6.15	700	42.7	7.55
		Stabilized Infiltration Rate (in /hr):		**6.2**	**Stabilized Infiltration Rate (in /hr):**		**7.6**

NOTES:
ASTM D3385 recommends using the rate of the inner ring as the vertical infiltration rate if the rates for the inner ring and the annular space differ, since such difference in rates is likely due to divergent flow.

The designer should include an appopriate factor of safety when using the presented infiltration rates in the design of drainage improvements.





ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

Double-Ring Infiltrometer Test Report (ASTM D3385)

Test Number	**DRI-2**	**Technician**	RL/KC
Project Name	Moorhen Marsh LEAPS	**Engineer**	PGA
Project Number	19-140	**Weather Conditions**	Clear
Test Location	Refer to Figure No. 2	**Temperature**	95F
Date	09/16/2019	**Testing Liquid**	Water

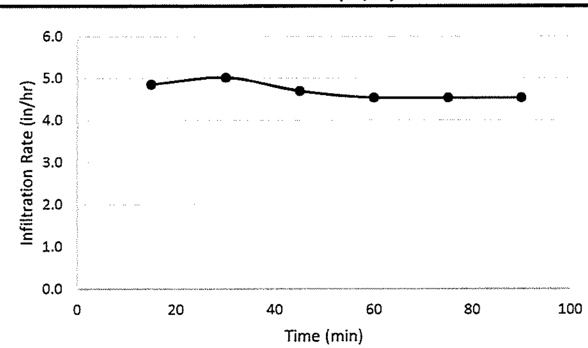
SOIL PROFILE	
Depth (ft-bls)	**Description**
0-2.3	Gray fine sand (SP)
2.3-4	Gray-brown slightly clayey fine sand (SP-SC)
	EOB @ 4-ft
Groundwater Not Encountered	

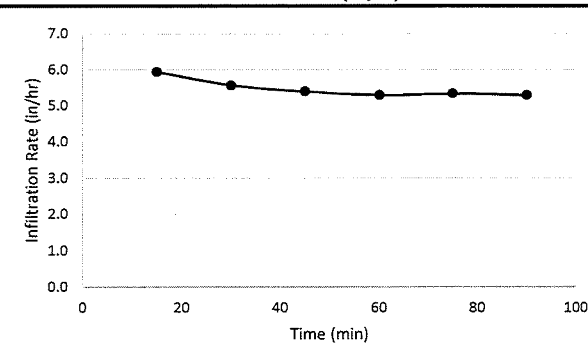
Area of Inner Ring (in^2):	113.1	**Test Depth (in):**	12	**Depth of Water - Inner Ring (in):**	4
Area of Annular Space (in^2):	339.3	**Ring Seating (in):**	6	**Depth of Water - Annular Space (in):**	4

Cycle No.	Incremental Test Time (min)	INNER RING			ANNULAR SPACE		
		Incremental Flow (ml)	Incremental Flow (in^3)	Infiltration Rate (in/hr)	Incremental Flow (ml)	Incremental Flow (in^3)	Infiltration Rate (in/hr)
1 (15 min)	1	150	9.2	4.86	550	33.6	5.94
2 (30 min)	1	155	9.5	5.02	515	31.4	5.56
3 (45 min)	1	145	8.8	4.69	500	30.5	5.40
4 (60 min)	1	140	8.5	4.53	490	29.9	5.29
5 (75 min)	1	140	8.5	4.53	495	30.2	5.34
6 (90 min)	1	140	8.5	4.53	490	29.9	5.29
		Stabilized Infiltration Rate (in /hr):		**4.5**	**Stabilized Infiltration Rate (in /hr):**		**5.3**

NOTES:
ASTM D3385 recommends using the rate of the inner ring as the vertical infiltration rate if the rates for the inner ring and the annular space differ, since such difference in rates is likely due to divergent flow.

The designer should include an appopriate factor of safety when using the presented infiltration rates in the design of drainage improvements.





ANDERSEN ANDRE CONSULTING ENGINEERS, INC.

Double-Ring Infiltrometer Test Report (ASTM D3385)

Test Number	**DRI-3**	**Technician**	RL/KC
Project Name	Moorhen Marsh LEAPS	**Engineer**	PGA
Project Number	19-140	**Weather Conditions**	Overcast
Test Location	Refer to Figure No. 2	**Temperature**	96F
Date	09/16/2019	**Testing Liquid**	Water

SOIL PROFILE	
Depth (ft-bls)	**Description**
0-2	Lt. brown fine sand (SP)
2-4	Gray slightly clayey to clayey fine sand (SP-SC/SC)
	EOB @ 4-ft
Groundwater Not Encountered	

Area of Inner Ring (in^2):	113.1	**Test Depth (in):**	12	**Depth of Water - Inner Ring (in):**	4
Area of Annular Space (in^2):	339.3	**Ring Seating (in):**	6	**Depth of Water - Annular Space (in):**	4

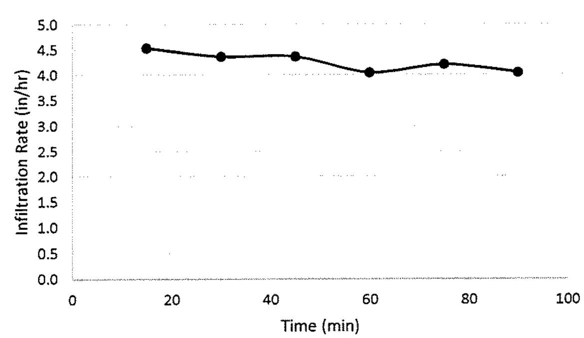
Cycle No.	Incremental Test Time (min)	INNER RING Incremental Flow (ml)	INNER RING Incremental Flow (in^3)	INNER RING Infiltration Rate (in/hr)	ANNULAR SPACE Incremental Flow (ml)	ANNULAR SPACE Incremental Flow (in^3)	ANNULAR SPACE Infiltration Rate (in/hr)
1 (15 min)	1	140	8.5	4.53	410	25.0	4.42
2 (30 min)	1	135	8.2	4.37	375	22.9	4.05
3 (45 min)	1	135	8.2	4.37	350	21.4	3.78
4 (60 min)	1	125	7.6	4.05	325	19.8	3.51
5 (75 min)	1	130	7.9	4.21	325	19.8	3.51
6 (90 min)	1	125	7.6	4.05	325	19.8	3.51

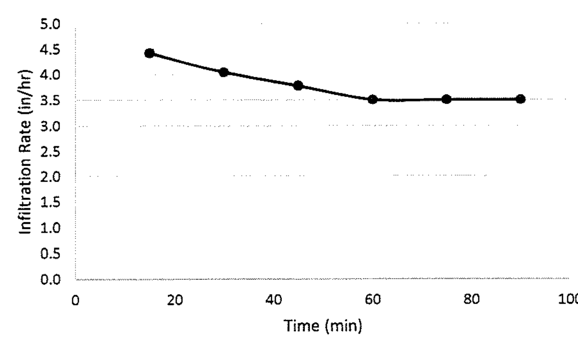
Stabilized Infiltration Rate (in /hr): **4.0**

Stabilized Infiltration Rate (in /hr): **3.5**

NOTES:
ASTM D3385 recommends using the rate of the inner ring as the vertical infiltration rate if the rates for the inner ring and the annular space differ, since such difference in rates is likely due to divergent flow.

The designer should include an appopriate factor of safety when using the presented infiltration rates in the design of drainage improvements.





APPENDIX V

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.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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ADDENDUM 1 TO

SUBSURFACE SOIL EXPLORATION AND GEOTECNICAL EGINEERING EVALUATION

MOORHEN MARSH AQUATIC PLANT SYSTEM (LEAPS) PROJECT

INDIAN RIVER COUNTY, FLORIDA

AACE File No. 19-140
March 10, 2020

Indian River County
Board of County Commissioners
Public Works - Stormwater Division
1801 237th Street
Vero Beach, FL 32960

Attn: Mr. Keith McCully, P.E.

ADDENDUM NO. 1
SUBSURFACE SOIL EXPLORATION AND
GEOTECHNICAL ENGINEERING EVALUATION
MOORHEN MARSH
LOW ENERGY AQUATIC PLANT SYSTEM (LEAPS) PROJECT
INDIAN RIVER COUNTY, FLORIDA

Andersen Andre Consulting Engineers, Inc. (AACE) has completed additional geotechnical engineering analyses for above referenced project relative to portions of the structural engineering design efforts by Kimley-Horn. This addendum to our 10/09/19 report briefly summarizes our recommendations relative to the discussed features.

Headworks/Pump Station Structure

We understand that the proposed headworks and pump station structure will have some foundation elements with bottom elevations of approximately 1.5 feet (NAVD88 datum) and, as such, in close proximity to a layer of loose slightly silty fine sands (USCS 'SP-SM') found at depths of about 20 feet below the existing grades. Based on the depth of this structure and the corresponding amount of soils that will be excavated as part of its installation, we recommend using a maximum allowable bearing pressure beneath the foundations of 1,800 psf.

We note that the construction of this structure will likely include the installation of a temporary steel sheet pile retaining wall and will require significant dewatering operations. With regards to the design of dewatering operations (by others), we recommend that the field permeability test results presented in 10/09/19 be reviewed as they include flow rates for various depths and strata.

Any steel sheet pile structure needed for the construction of the pump station, including any internal bracing, tie-back system, and/or other lateral restraining systems, will need to be designed by a Professional Structural Engineer registered in the State of Florida. Further, pre-construction video surveying and construction vibration monitoring at any existing nearby structural features are recommended.

It be may necessary to install a layer of compacted 57-stone as base for the foundation(s) to provide a firm bearing surface.

Proposed Settling Basin Wall and Misc. Minor interior Walls

We understand that the central settling basin wall will be 8-9 feet high with a foundation bottom elevation near 14 feet, and additional minor walls within the system will be 4-6 feet tall with foundation bottom elevations near 16 to 18 feet. The foundations for these wall elements can be proportioned for an allowable bearing pressure of 2,500 psf. If a LRFD approach is being used in the foundation design, a resistance factor of 0.5 can be used.

We understand that the central settling basin wall will have an approximately 4-ft wide horizontal area in front of it, after which the bottom of the settling basin will slope down from elevation 15.5 feet to elevation 10 feet with a 4H:1V slope with partial riprap protection/stabilization on top of the soil layer covering the proposed Geosynthetic Clay Liner (GCL). This scenario should provide for a safe global slope stability for the wall.

Per our 10/09/19 report, the groundwater was encountered near EL 11-12 feet and the normal seasonal high groundwater table elevation is estimated to be near EL 14-14.5 feet. As such, excavation bracing and dewatering may potentially be required during the construction of these wall foundations.

Bearing Capacity and Settlements

Based upon the boring information and the assumed loading conditions, we estimate that the recommended allowable foundation bearing pressures will provide a minimum factor of safety in excess of two against bearing capacity failure. With the site prepared and the foundations designed and constructed as recommended, we anticipate total settlements of one inch or less, and differential settlement between adjacent similarly loaded footings of less than one-quarter of an inch. Because of the granular nature of the subsurface soils, the majority of the settlements should occur during construction; post-construction settlement should be minimal.

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.

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

Peter G. Andersen, P.E.
Principal Engineer
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