JEA

Greenland Energy Center
Jacksonville, Florida

Geotechnical Report

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1.0 Introduction

The Greenland Energy Center (GEC) will be a 2 on 1 combined cycle generating facility that is scheduled for simple cycle operation beginning in June 2010 and combined cycle operation in June 2012. The facility footprint also includes a 2 on 1 combined cycle power block on the south end of the site that is to be constructed after 2012. This report presents a summary of the site subsurface conditions, geotechnical data, and geotechnical design recommendations for the proposed center, which is located at the southeast edge of Jacksonville, Florida.

This report describes the results of the subsurface investigation program that was conducted between January 15 and February 14, 2008. Black & Veatch performed the subsurface investigation to determine the site stratigraphy, pertinent geotechnical engineering properties, and the design parameters for the soils at the project site.

The investigation consisted of 21 soil borings, 16 soil resistivity tests, 14 Cone Penetration Test soundings (CPTs), 13 Seismic Cone Penetration Test soundings (SCPTs), 7 dilatometer tests (DMTs), 5 test pits, 1 double ring infiltrometer (DRI) test, appurtenant laboratory tests, and the construction of 4 piezometers. This report includes the following information:

- Site location and description.
- Project description.
- Details of the subsurface investigation program.
- Site characterization.
- Engineering recommendations.
- Logs of borings, piezometers, test pits, CPTs, SCPTs, and DMTs.
- Laboratory and field testing results.

1.1 Limitations

The data contained in this report are based on tests done at locations identified on Figures 4-1 and 4-2 and provide an indication of the site conditions existing at the time of the subsurface investigations. Black & Veatch has assumed that the information obtained from the investigations is representative of the subsurface conditions throughout the site. The information provided is indicative of the conditions of the site and may be relied upon for detailed foundation design.

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2.0 Summary

The information based on the data obtained during the current investigation is summarized as follows:

- The proposed location is a greenfield site that was logged. Smaller trees remain on the site and the dispersed grub piles generated from the minimal clearing required for the logging operations.
- Estimated plant grade is at Elevation 32 feet. The highest point is approximately 41 feet in elevation and is located to the west of the southern cooling tower. The lowest point is 21 feet in elevation and lies in the southeast corner of the proposed detention pond footprint.
- The historical groundwater table elevation generally fluctuates between Elevations 18 and 24 feet.
- The geotechnical investigation consisted of 21 soil borings, 16 soil resistivity tests, 14 CPTs, 13 SCPTs, 7 DMTs, 5 test pits, 1 DRI test, appurtenant laboratory tests, and the construction of 4 piezometers.
- The site is predominantly covered by sand overburden with varying density, to an elevation of approximately -30 feet, where in some areas across the site a layer of weathered limestone exists, with characteristics similar to very dense sand.
- No geologic hazards are identified for the site.
- With an estimated plant grade of Elevation 32 feet, site grading will primarily entail filling, generally ranging from 1 foot of fill at the western fuel oil tanks to 3 feet of fill at the southern end of the substation.
- The soils encountered across the site are adequate to support major foundations without causing excessive settlement. It is anticipated that all major and minor equipment can be founded on shallow foundations.

3.0 Site Conditions

3.1 Site Location

The GEC site is located at the southeast edge of Jacksonville, Florida, and can be accessed from of Phillips Highway by turning east onto Phillips Industrial Boulevard and southeast onto Davis Creek Road. The site location is approximately 2.5 miles east of the I-95/I-295 interchange as shown on Figure 3-1.

3.2 Site Description

The average elevation of the site is approximately 32 feet above mean sea level (National Geodetic Vertical Datum [NGVD] 29). The site is formerly densely wooded, and the existing topography of the site is slightly rolling. A number of drainage features are present and trend northeast to southwest. The site area has been stripped of valuable wood, and what is remaining includes smaller trees and dispersed grub piles generated from the minimal clearing required for the logging operations. The site was left with a hummocky landscape as the result of the tree removal. A 25 foot elevation contour wraps around the site from the southeast corner to the southwest to the northwest with approximately four mounded high points in the center of the site. The highest point is approximately 41 feet in elevation and is located to the west of the southern cooling tower; the lowest point at 21 feet in elevation and lies in the southeast corner of the proposed detention pond footprint.

3.3 Proposed Facility

The GEC will be a 2 on 1 F Class combined cycle power plant with a nominal net output rating of 504 megawatts (MW) at average ambient temperature conditions. JEA will consider installing future units at the site through space allocation and installing facilities required to support future units at the site when appropriate. The GEC will be dual fueled with pipeline gas as the primary fuel and ultra-low sulfur diesel (ULSD) fuel oil as a backup fuel. The combined cycle power plant will include heat recovery steam generators (HRSGs) provided with pipeline gas fired supplemental duct burners to increase power generation and a steam turbine bypass to the condenser to allow for simple cycle operation.

The proposed power plant will include two power blocks, each having two combustion turbines (CTs) that will operate in combined cycle with an HRSG. The major components of each unit will include two HRSGs, two CTs, a steam turbine (ST), four transformers, a cooling tower, fuel oil tanks, demineralized water tanks, three 5 million gallon prestressed concrete reuse tanks, and appurtenant structures.

Figure 4-1 depicts the relative location of the power blocks and the equipment arrangement.

Surface drainage will be directed to detention ponds that will be excavated south of the power blocks. Soil from the detention ponds will be used to raise the site adjacent to the pond. The plant grade is currently estimated to be approximately 32 feet, requiring cuts up to 9 feet on the eastern half of the site and fills up to 6 feet on the western half. The earthwork operations will be balanced in cut and fill.

4.0 Subsurface Investigation

4.1 Field Testing Program

B&V performed the subsurface investigation, conducted between December 13 and February 14, 2008, to determine the site stratigraphy and pertinent geotechnical engineering properties of the soil that underlies the power plant site. The subsurface investigation program included 21 soil borings, installation of 4 piezometers, 16 soil resistivity tests, and 27 cone penetration tests (CPTs and SCPTs), 5 test pits, 7 DMTs, and 1 DRI test. Figure 4-1 shows the location of these investigation points, excluding soil resistivity test locations, which are shown on Figure 4-2. Table 4-1 includes a list of this investigation work, along with basic information about each investigation point.

Ellis & Associates, Inc. (E&A) of Jacksonville, Florida, was subcontracted to perform the investigation under the direction of B&V. Field supervision was performed by engineers from B&V. Surveying for field investigation points was performed by DeGrove Surveyors, Inc. B&V relocated some of the investigation points during subsurface investigation following changes that were made in the layout of the facilities of the plant after the investigation was underway. A number of new investigation points were also added during this process. The location of the relocated and new investigation points, and the ground surface elevation of all of the investigation points were surveyed after the completion of investigation by the surveying subcontractor (DeGrove Surveyors, Inc.).

4.1.1 Soil Test Borings

Twenty-one soil borings, ranging in depth from 24.0 to 125.0 feet. E&A advanced the borings with truck-mounted Central Mining Equipment (specifically CME 45), Diedrich D50, and BK-81 drill rigs. A B&V geotechnical engineer logged the majority of the borings. A few of the borings were logged by E&A's engineer. The boring logs are presented in Appendix B.

E&A performed the Standard Penetration Test (SPT) using a standard safety hammer (140 pounds in weight), controlled with a rope and cathead to free fall 30 inches in accordance with American Society for Testing and Materials (ASTM) 1586. During the SPT, the number of blows for the first 6 inch increment of driving was disregarded as a "seating" value, and the total number of impacts to drive the sampler for the second and third 6 inch advance was recorded. This value is referred to as the N-value or standard penetration resistance. Therefore, the standard penetration resistance represents an average resistance over 12 inches of advancement of a 2 inch outside diameter (OD) split-barrel sampler.

Table 4-1
Exploration Points for Subsurface Investigation

Investigation Point Label	Coordinates		Ground Surface Elevation (feet)	Depth (feet)	Remarks
	North (feet)	East (feet)			
B-1	2,119,306.2	492,883.5	29.5	99.5	Boring
B-2	2,118,866.9	492,903.7	27.6	100.0	Boring
B-3	2,117,066.7	492,154.3	24.1	75.0	Boring
PZ-3	2,117,066.7	492,158.3	27.1	25.0	Piezometer
B-4	2,118,407.9	492,195.6	25.2	50.0	Boring
PZ-4	2,118,391.9	492,189.4	27.8	24.0	Piezometer
B-5	2,118,477.0	492,883.0	32.6	125.0	Boring
B-5A	2,118,481.0	492,887.0	33.7	108.0	Boring
B-6	2,118,543.0	493,597.0	32.7	75.0	Boring
B-7	2,118,249.0	492,846.0	38.1	125.0	Boring
B-7A	2,118,256.0	492,846.0	37.8	80.0	Boring
B-8	2,118,247.0	492,356.0	29.1	50.0	Boring
B-9	2,117,846.0	492,856.0	32.3	87.0	Boring
B-9A	2,117,846.0	492,864.0	33.1	125.0	Boring
B-9B	2,117,838.0	492,864.0	32.9	76.0	Boring
B-10	2,118,040.0	493,373.0	32.1	75.0	Boring
PZ-10	2,118,039.5	493,382.9	34.9	25.0	Piezometer
B-12	2,117,403.0	492,787.0	33.8	125.0	Boring
B-12A	2,117,393.0	492,787.0	34.3	125.0	Boring
B-13	2,118,978.3	493,488.8	36.0	119.3	Boring
PZ-13	2,118,988.3	493,493.8	39.0	25.0	Piezometer
B-14	2,117,066.3	493,053.5	25.2	50.0	Boring
B-16	2,118,988.3	493,396.5	35.5	50.0	Boring
B-17	2,118,933.2	493,397.0	35.4	50.0	Boring
B-18	2,119,033.5	493,494.3	36.1	50.0	Boring
CPT-1	2,117,066.0	493,053.0	25.2	9.7	Cone Penetration Test
CPT-1A	2,117,064.0	493,053.0	25.2	9.2	Cone Penetration Test
CPT-2	2,119,256.0	493,010.6	30.6	66.1	Cone Penetration Test
CPT-3	2,118,891.0	492,915.0	27.6	61.2	Cone Penetration Test
CPT-4	2,119,009.1	492,878.9	28.6	86.1	Cone Penetration Test
CPT-5	2,118,549.8	492,313.3	26.4	85.9	Cone Penetration Test
SCPT-6	2,118,561.0	492,786.0	29.9	120.3	Seismic Cone Penetration Test

Table 4-1 (Continued)
Exploration Points for Subsurface Investigation

Investigation Point Label	Coordinates		Ground Surface Elevation (feet)	Depth (feet)	Remarks
	North (feet)	East (feet)			
SCPT-7	2,118,561.0	492,960.0	33.2	120.9	Seismic Cone Penetration Test
CPT-8	2,118,569.0	493,635.0	33.0	17.2	Cone Penetration Test
CPT-8A	2,118,567.0	493,635.0	33.0	78.4	Cone Penetration Test
SCPT-9	2,118,387.0	492,786.0	33.3	101.0	Seismic Cone Penetration Test
SCPT-10	2,118,387.0	492,960.0	33.6	116.0	Seismic Cone Penetration Test
CPT-11	2,118,374.7	492,178.0	24.7	75.1	Cone Penetration Test
SCPT-12	2,118,137.0	492,781.0	33.7	70.3	Seismic Cone Penetration Test
SCPT-12A	2,118,135.0	492,781.0	33.7	105.4	Seismic Cone Penetration Test
CPT-13	2,118,069.0	493,428.0	31.6	120.2	Cone Penetration Test
SCPT-14	2,117,972.0	492,786.0	31.8	64.6	Seismic Cone Penetration Test
CPT-15	2,118,130.4	492,186.5	27.4	120.0	Cone Penetration Test
SCPT-16	2,117,765.0	492,786.0	30.4	77.2	Seismic Cone Penetration Test
SCPT-17	2,117,766.0	492,960.0	32.6	125.8	Seismic Cone Penetration Test
SCPT-18	2,117,590.0	492,786.0	33.2	65.9	Seismic Cone Penetration Test
SCPT-18A	2,117,588.0	492,786.0	33.2	126.1	Seismic Cone Penetration Test
SCPT-19	2,117,591.0	492,960.0	32.9	120.4	Seismic Cone Penetration Test
CPT-20	2,117,921.5	492,152.8	26.6	61.7	Cone Penetration Test
SCPT-21	2,117,398.0	492,740.0	32.8	110.6	Seismic Cone Penetration Test
CPT-22	2,119,005.3	493,258.9	32.6	116.9	Cone Penetration Test
CPT-23	2,118,950.0	493,401.0	35.8	120.2	Cone Penetration Test
DMT-1	2,119,253.0	492,889.5	29.8	50.0	Dilatometer Test
DMT-2	2,118,466.0	492,973.0	34.6	45.0	Dilatometer Test
DMT-3	2,118,491.0	493,544.0	32.4	9.0	Dilatometer Test
DMT-4	2,118,251.8	492,319.9	28.5	35.0	Dilatometer Test
DMT-5	2117888.0	492,923.0	33.4	25.0	Dilatometer Test
DMT-6	2,117,409.0	492,644.0	34.6	10.0	Dilatometer Test
DMT-7	2,119,005.1	493,471.7	36.0	45.0	Dilatometer Test
TP-1	2,116,942.0	492,370.0	23.7	8.0	Test Pit
TP-2	2,118,807.0	493,174.0	33.0	9.0	Test Pit
TP-3	2,118,440.0	493,472.0	34.1	8.0	Test Pit
TP-4	2,118,172.0	493,113.0	37.4	8.0	Test Pit
TP-5	2,118,774.4	493,479.0	35.4	8.0	Test Pit
DRI-1	2,117,051.6	492,463.5	25.7	10.0	Double Ring Infiltrometer Test

E&A continuously sampled the first (top) 10 feet of all soil borings by five 2 foot SPT runs. In these cases, although the number of hammer blows during the last (fourth) 6 inch penetration was recorded, the SPT resistance was still obtained by adding the blows during the second and the third 6 inch penetrations, in a similar fashion as described above.

At the end of the first 10 feet of continuous sampling, B&V increased the sampling interval to 5.0 feet (i.e., sampling starting at depth of 13.5 feet, 18.5 feet, and so on). Furthermore, E&A employed conventional mud rotary techniques, with 2-7/8 inch tricone roller bit or drag bit and bentonite mud (more specifically, Super Gel-X mud) as the drilling fluid, in anticipation of the water table. Drilling below the water table creates an upward pressure gradient at the bottom of a borehole that can cause a reduction in soil strength, artificially reducing the N-value. E&A performed SPTs so that there was a head of drilling fluid equal to or greater than the hydrostatic pressure at the sample depth inside the borehole when the N-value was measured. The boring logs describe the details of advancement used at each location.

E&A collected relatively undisturbed cohesive samples with thin-walled tube samplers. E&A pushed these samplers were pushed approximately 2 feet by (the thrust applied through) drill rods. In the borings where tube sampling was planned, E&A performed rotary drilling using a 4 inch drill bit. In one of the borings (Boring B-9A), using this larger diameter drill bit caused caving, and E&A installed temporary steel casing to allow continuation of work and tube sampling in this boring.

E&A sealed the two ends of the tube sampler by a plastic cap and duct tape, promptly after its retrieval from the boring and its inspection. Samples were transported to E&A's soils laboratory. Additionally, pocket penetrometer readings were performed on disturbed and undisturbed samples of cohesive material.

4.1.2 Piezometers

Four piezometers (PZ-3, PZ-4, PZ-10, and PZ-13), were installed near Borings B-3, B-4, B-10, and B-13, respectively. The piezometer installation logs are included in Appendix F, and Table 4-2 summarizes details about piezometer installation and development. E&A drilled the piezometer holes with a hollow stem auger, 8 inches in external diameter, with end plug. The screen and riser were installed through the auger stem. The filter material and primary seal were put in place as the auger was slowly withdrawn. The annulus between the riser and the surrounding medium was filled with grout from the top of the seal to ground surface.

Table 4-2 Piezometer Installation and Development						
Piezometer	Ground Surface Elevation (feet)	Depth (feet)		Bottom of Screen/Filter Elevation (feet)	Development	
		Screen Length	Bottom of Screen/ Filter		Development Time (minutes)	Water Clarity at Completion of Development
PZ-3	27.1	10.0	25.0	+2.1	20	Clear
PZ-4	27.8	10.0	24.0	+3.8	21	Clear
PZ-10	34.9	10.0	25.0	+9.9	23	Clear
PZ-13	39.0	10.0	25.0	+4.0	30	Clear

The piezometers were constructed using 2 inch diameter polyvinyl chloride (PVC) riser pipe, with 10 feet of PVC screen with 0.01 inch slots. Silica sand was used as the filter material around the screen. Bentonite pellets was used as the primary seal material. The piezometers were developed by a small submersible battery-powered pump until the water was clear. A lockable, aboveground protective cover was installed over the PVC riser pipe.

4.1.3 Test Pits

As indicated in Table 4-1, E&A excavated five test pits. The purpose of these test pits was to observe and log the soil profile directly, and to collect bulk samples from these pits for compaction and thermal tests. Test pits were excavated to depths between 8 to 9 feet, using a Takeuchi TB-145 trackhoe. A B&V engineer logged the each test pit. The test pits were backfilled with soil that had been removed during the excavation, promptly after the completion of the logging. Test pit logs are included in Appendix C.

4.1.4 Cone Penetrometer Test Soundings

Cone penetrometer testing was performed by Southern Earth Sciences (SES) of Mobile, Alabama. Each probe contained transducers to monitor tip resistance, side friction, and pore pressures during testing. SES probed twenty-seven holes to depths of up to 126 feet. SES performed seismic downhole testing (SCPT) at 13 locations by inducing a shear waves by a sledge hammer striking a steel plate on the ground surface. The plate was secured to the ground surface with pad pressure from the rig. The cone penetrometer below the ground surface recorded the time that it took the shear waves to travel from the ground surface to the cone penetrometer, thereby allowing determination of the shear wave velocity. Plots of CPT soundings and shear wave velocity profiles can be found in Appendix D.

4.1.5 Dilatometer Tests

E&A performed seven dilatometer tests (DMTs) at the locations indicated in Table 4-1. In these tests, which are also known as flat bed DMT estimates, the pressure needed to penetrate the center of a standard size (60 millimeter [mm] in diameter) membrane by 1.1 mm into the surrounding ground is measured. This pressure is applied to the back of the disk by gas (e.g., nitrogen) through pneumatic cable, and it is measured by calibrated pressure gauges at ground surface. Relationships are used to convert the measured pressures to different parameters that represent soil type and properties, such as internal friction angle and stiffness. Appendix H contains the original dilatometer data collected at the field during these tests.

4.1.6 Soil Resistivity Testing

Geoview, Inc., performed soil resistivity tests using a Supersting R8 Earth Resistivity Meter manufactured by Advanced Geosciences, Inc. The purpose of the investigation was to determine the electrical resistance across the project site.

Geoview used a Wenner four-point electrical resistivity arrayto determine the in situ values of ground electrical resistivity. Tests were conducted at a total of 16 locations within the project site. Two orthogonally orientated arrays were performed at each location for a total of 32 arrays for the entire project. There were 14 “top” locations (TSR) using electrode spacings (a-spacings) of 1, 2, 3, 6, 10, 20, and 30 feet and two “deep” locations (DSR) using a-spacings of 1, 2, 3, 6, 10, 20, 30, 60, 100, 200, and 300 feet. Complete results of the survey are provided in Appendix I.

4.2 Laboratory Testing Program

E&A performed laboratory testing to classify and characterize the soils encountered during the investigation and to estimate relevant engineering properties of the soils. B&V assigned the laboratory tests based on observations made in the field, and E&A performed the testing. The tests performed on fine-grained (cohesive) soil specimens included moisture content, Atterberg limits and consolidation tests, and unconsolidated undrained triaxial compression (UU) tests. Sieve analyses were performed on both cohesive and granular soil specimens. One set of chemical tests was performed, as well as five compaction tests and two thermal resistivity tests. No solid rock was encountered during the investigation; thus, no rock testing was performed. Thermal resistivity results are included in Appendix A. The complete results of laboratory testing are presented in Appendix E. Results of the laboratory testing are further discussed in Section 5.0.

The laboratory tests and their main purposes are described in more detail as follows:

- Moisture Content: To determine the in-place properties of the soils.
- Atterberg Limits: To determine the relative plasticity of the soil samples and to assist in classifying the fine-grained portion of the samples. The liquid limit can be an indication of soil compressibility.
- Hydrometer Analysis: To determine the relative proportions of silt and clay.
- Grain Size Analyses: To determine the relative proportions of fines and sand found in the soil samples.
- Consolidation: To determine the compressibility of cohesive deposits.
- Unconsolidated Unconfined Triaxial Compression (UU): To determine shear strength of cohesive soil specimens.
- Organic Content: To determine the proportion of the organic material.
- Chemical Analysis: To determine the corrosive potential of foundation soils by measuring the pH, chloride, and sulfate content of foundation soils.
- Modified Proctor: To assess compaction characteristics of soil.
- Thermal Resistivity: To determine the thermal dry-out characteristics of the soils.

All laboratory testing was performed in general accordance with established ASTM or US Army Corps of Engineers (USACE) standard procedures. Results from the laboratory testing program are included in Appendix E.

5.0 Subsurface Conditions

5.1 Regional Geology

The project area is located in northern Florida, within the coastal lowlands. Regionally, Florida can be divided into north and central highlands and coastal lowlands. The landscape of northern Florida is dominated by the northern highlands, a series of gently sloping plateaus, bordered to the south by a scarp, which separates the highlands from the Gulf Coast lowlands. The Gulf Coast lowlands extend south to the Caloosahatchee River.

5.2 Site-Specific Geology

The project site is located in southern Duval County. The site is located near the intersection of I-95 and US 9A, approximately 1 mile east of Phillips Industrial Boulevard and 2 miles east of Greenland Park.

Duval County is located within the Northern or Proximal Zone. The principal physiographic features within Duval County are the Duval Upland, the Eastern Valley, the St. Mary's Meander Plain, the Center Park Ridge, and the Atlantic Coastal Ridge. The uppermost deposit at the site is Undifferentiated Pleistocene to Holocene recent deposits. Low-1ying portions of the modern Florida peninsula were submerged during the Pleistocene series of sea level transgressions, which are believed to have been as much as 60 feet above the present sea level. The Pleistocene and Holocene deposits of northeastern Florida generally consist of tan to yellow, fine- to medium-grained, unconsolidated quartz sands. The thickness of the sediments ranges from 10 feet near the St: Jøhns Rivér to 100 feet in western Duval County.

The Hawthorn Formation-Miocene sediments underly the undifferentiated deposits. It represents the upper confining layer for the Florida Aquifer in peninsular Florida. In general, the Hawthorn Formation consists of silica plastic and carbonaceous sandy clays or marls, and clayey sands interbedded with thin discontinuous layers of phosphatic sands, limestone, and dolomites. The limestone and dolomite layers are thicker and more prevalent near the base of the Hawthorn Formation (Leve, 1966). The Hawthorn Formation is commonly identified by the presence or absence of phosphate, which is differentially distributed throughout the sediments. The Hawthorn Formation, in Duval County, ranges in thickness from approximately 350 feet in the southwestern portion of the county to approximately 500 feet in the eastern portion of the county (Scott, et al., 1988) with an estimated thickness of about 450 feet in the site vicinity. The Hawthorn Formation is considered a regional confining unit, providing a barrier to direct recharge to the Florida Aquifer in recharge areas and retarding the upward movement of water to the intermediate artesian aquifer, the surficial aquifer, and/or springs in discharge areas.

Beneath the Hawthorn Formation is the Ocala Group Limestone - a late Eocene limestone that is generally characterized as a tan to buff, soft, granular to massive, porous crystalline marine limestone (lower portion) to a white to cream weathered chalky massive marine limestone in borings (upper portion) (Randazzo and Jones 1997). The Ocala Limestone, in combination with the underlying carbonate units comprises the Floridan Aquifer system. The top of the Ocala Limestone in the vicinity of the site is at an elevation of approximately 550 feet below mean sea level.

5.3 Geological Hazards

5.3.1 Soil Liquefaction

Because of the low seismic hazard of the site, soil liquefaction is not considered to be a hazard.

5.3.2 Swelling Soils/Bedrock

No swelling soils or bedrock were identified near the project elevation during the investigation. All identified clays were well below the groundwater surface and have little potential for swell. Any clay removed during construction should not be reused as structural fill.

5.3.3 Land Subsidence

No land subsidence is reported at the site.

5.3.4 Flooding

The plant elevation is approximately 32 feet above mean sea level. The Federal Emergency Management Agency (FEMA) 100 year flood elevation for the site is 19 feet above sea level. Flooding is not considered a hazard at the site.

5.3.5 Frost Heave

Frost heave at the site is not considered to be a potential hazard. Frost is not a concern in the 2004 Florida Building Code.

5.3.6 Collapsible Soils

No soils from the investigation beneath proposed facilities are prone to collapsible behavior.

5.3.7 Ground Rupture and Ground Shaking

No ground rupture and ground shaking were observed at the time of the investigation.

5.3.8 Karst and Sinkhole

Karst terrains develop in areas underlain by carbonate rocks such as limestone. They often have drainage systems that are reflected on the surface as sinkholes, springs, disappearing streams, or even caves.

According to the "Sinkhole Type, Development and Distribution in Florida Map," (http://www.dep.state.fl.us/geology/publications/sinkholetype3.pdf), the site is located in the cover over limestone, which is reported to be more than 200 feet thick. In the general area of the site, sinkholes are reported to be very few. A check of the sinkhole database established by the former Sinkhole Research Institute and now maintained by the State of Florida indicates on1y seven sinkholes have been reported in Duval County, one of which was 1ocated in the Ds Point area (latitude 30°25'00" and longitude 81°33'05''). However, given the extremely thick overburden in the area, it is doubtful that this was a true limestone collapse feature. Furthermore, there were no indicators of Karst geology, such as voids or sinkholes noted in the area. During the course of the investigation, no soft or muck-filled areas were identified within the borings.

5.4 Seismicity

The project site is located in an area that can be considered a low hazard area. According to the 2004 Florida Building Code, the site is categorized as Site Class D. Table 5-1 presents the ground motion parameters assigned to the location in accordance with the seismic hazard map in the Code.

Table 5-1
Seismic Ground Motion Parameters

Ground Motion Parameter	Value
S_s - Short period spectral response acceleration	0.15g
S_1 - 1 second period spectral response acceleration	0.08g

5.5 Site-Specific Ground Conditions

5.5.1 Site Stratigraphy

The review of subsurface information indicates that despite some common features, soil profile varies throughout the site to the extent that the site had to be divided into eight different areas in terms of soil profile and properties. Figure 5-1 shows these areas, which are labeled as Area 1 to Area 8. Table 5-2 includes details about the plant's facilities covered by each area, a list of geotechnical exploratory points related to each area, and the figure number where the related soil profile is available.

Table 5-2
Description of Different Areas Across the Site

Area	Facility Included	Investigation Points	Figure
Area 1	Power Block - South: Combustion Turbines 7, 6, and 5; Steam Turbines 4 and 3; Admin/Control/Maintenance Building	B-12, 12A, 9, 9A, 9B, 7, 7A; SCPT-21, 19, 18, 17, 16, 14, 12; DMT-6 and 5	5-2
Area 2	Power Block - North: Combustion Turbines 2 and 1	B-5, 5A; SCPT-10, 9, 7, 6; DMT-2	5-2
Area 3	Fuel Tanks Containment Area	B-2, 1; CPT-4, 3, 2; DMT-1	5-2
Area 4	Reuse Surge Tanks (5,000,000 gallons)	B-17, 13; CPT-23, 22; DMT-7	5-3
Area 5	North Cooling Tower	B-6; CPT-8A; DMT-3	5-4
Area 6	South Cooling Tower	B-10; CPT-13	5-4
Area 7	Substation	B-8, 3; CPT-15, 11; DMT-4	5-5
Area 8	Detention Pond	B-14, 3; CPT-1A, 1	5-6

Tables 5-3 through 5-10 include descriptions of soil units in each area along with average SPT resistance (blow count) in each unit and estimated unit weights. In the boring logs, for SPT resistance N ">50" (i.e., refusal), N = 75 has been used in calculating average blow count, $(N)_{av}$. Furthermore, whenever N>75, it has been substituted by N = 75 for averaging (i.e., conservative, and consistent with the case of refusal previously described).

Table 5-3
Description of Soil Profile in Area 1 (Power Block - South)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
GS1	Very Loose and Loose Sand	34.5	20	14.5	90.0	115.0	7	---
GS2	Upper Medium Dense Sand	20	5	15.0	100.0	120.0	25	---
GS3	Dense Sand	5	-20	25.0	110.0	125.0	50	---
GS4	Lower Medium Dense Sand	-20	-25	5.0	100.0	120.0	28	---
GS5	Very Dense Sand	-25	-40	15.0	110.0	125.0	64	---
GS6c(1)	Stiff Cohesive	-40	-47	7.0	70.0	110.0	19	4.7
GS7	Medium Dense Sand w/fines	-47	---	---	100.0	120.0	15	---

Note 1: Refer to Table 5-11 for detailed clay properties.

Table 5-4
Description of Soil Profile in Area 2 (Power Block; North)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
GN1	Very Loose and Loose Sand	32.5	20	12.5	90	115	7	---
GN2	Upper Medium Dense Sand	20	10	10	100	120	25	---
GN3	Dense Sand	10	-23	33	110	125	40	---
GN4	Very Dense Sand	-23	-50	27	110	125	56	---
GN5	Medium Dense and Dense Sand	-50	-80	30	100	120	30	---
GN6	Medium Dense Sand w/fines	-80	---	---	100	120	15	---

Table 5-5
Description of Soil Profile in Area 3 (Fuel Tanks Containment Area)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
FT1	Very Loose and Loose Sand	28.5	20	8.5	90	115	9	---
FT2	Upper Medium Dense Sand	20	0	20	100	120	18	---
FT3	Dense Sand	0	-23	23	110	125	45	---
FT4	Upper Very Dense Sand	-23	-30	7	110	125	70	---
FT5	Lower Very Dense sand	-30	-70	40	110	125	57	---
FT6	Medium Dense Sand w/fines	-70	---	---	100	120	15	---

Table 5-6
Description of Soil Profile in Area 4 (Reuse Surge Tanks)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
RS1	Very Loose and Loose Sand	37	24	13	90	115	5	---
RS2	Upper Medium Dense Sand	24	-15	39	100	120	24	---
RS3	Dense Sand	-15	-30	15	110	125	40	---
RS4	Lower Medium Dense Sand	-30	-45	15	100	120	15	---
RS5	Very Dense sand	-45	-70	25	110	125	55	---
RS6	Medium Dense Sand w/fines	-70	---	---	100	120	15	---

Table 5-7
Description of Soil Profile in Area 5 (North Cooling Tower)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
CT1	Very Loose and Loose Sand	32	24	8.0	90.0	115.0	6	---
CT2	Upper Medium Dense Sand	24	20	4.0	100.0	120.0	20	---
CT3	Cemented Sand (very dense)	20	17	3.0	110.0	125.0	75	---
CT4	Upper Loose Sand	17	10	7.0	90.0	115.0	8	---
CT5	Lower Medium Dense Sand	10	-18	28.0	100.0	120.0	19	---
CT8	Lower Loose sand	-18	-36	18.0	90.0	115.0	8	---
CT9	Very Dense Sand (weathered limestone)	-36	-44	8.0	110.0	125.0	51	---
CT10c(1)	Stiff Clay	-44	-51	7.0	70.0	110.0	15	4.7
CT11	Medium Dense Sand w/fines	-51	---	---	100.0	120.0	15	---

Note 1: Refer to Table 5-11 for complete clay properties.

Table 5-8 Description of Soil Profile in Area 6 (South Cooling Tower)								
Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
CT1	Very Loose and Loose Sand	32	24	8.0	90.0	115.0	6	---
CT2	Upper Medium Dense Sand	24	20	4.0	100.0	120.0	20	---
CT3	Cemented Sand (very dense)	20	17	3.0	110.0	125.0	75	---
CT4	Upper Loose Sand	17	10	7.0	90.0	115.0	8	---
CT5	Lower Medium Dense Sand	10	5	5.0	100.0	120.0	19	---
CT6	Very Loose Sand	5	-5.5	10.5	90.0	115.0	1	---
CT7	Dense Sand	-5.5	-32	26.5	110.0	125.0	39	---
CT9	Very Dense Sand (weathered limestone)	-32	-40	8.0	110.0	125.0	51	---
CT10c(1)	Stiff Clay	-40	-47	7.0	70.0	110.0	15	4.7
CT11	Medium Dense Sand w/fines	-47	---	---	100.0	120.0	15	---
Note 1: Refer to Table 5-11 for complete clay properties.								

Table 5-9
Definition of Soil Profile in Area 7 (Substation)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
SB1	Very Loose Sand	28	24	4	90	115	4	---
SB2	Loose Sand	24	10	14	90	115	9	---
SB3	Medium Dense / Dense Sand	10	-28	38	110	125	34	---
SB4	Loose Sand w/fines	-28	-38	10	90	115	9	---
SB5	Very Dense Sand (weathered limestone)	-38	-43	5	110	125	75	---
SB6c(1)	Stiff Cohesive	-43	-50	7	70	110	12	4.7
SB7	Medium Dense Sand w/fines	-50	---	---	100	120	15	---

Note 1: Refer to Table 5-11 for complete clay properties.

Table 5-10 Definition of Soil Profile in Area 8 (Detention Pond)								
Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	Lab Su (ksf)
DP1	Very Loose to Medium Dense Sand	24.5	16	8.5	90	115	8	---
DP2	Upper Very Dense Sand (cemented)	16	13	3	110	125	63	---
DP3	Medium Dense Sand	13	0	13	100	120	18	---
DP4	Dense Sand	0	-26	26	110	125	35	---
DP5	Loose Sand	-26	-38	12	90	115	9	---
DP6	Very Dense Sand (weathered limestone)	-38	-42	4	110	125	75	---
DP7c(1)	Stiff Cohesive	-42	-52	10	70	110	12	4.7
DP8	Medium Dense Sand w/fines	-52	---	---	100	120	15	---
Note 1: Refer to Table 5-11 for complete clay properties.								

The most significant feature common to all of these areas is that the soil profile is mostly composed of fine sand, which at most depths can be described as clean sand. The site is covered with a very loose and loose sand layer with a thickness that varies between 4.0 feet (in Area 7) to about 15.0 feet (in Area 1). The density of sand increases with depth, and sometimes reaches to very dense and even causes refusal of SPT runs. However, this increase in density is irregular, and sometimes reverses to lower density with increasing depth at some locations and depths.

A layer of cemented sand, dark brown in color, was detected in some of the probings (such as Borings B-12, B-14, and B-3) with its top at Elevation +16.0 to +20.0 feet. This layer is estimated to be about 3.0 to 4.0 feet thick, and is believed to have been responsible for refusal in some of the CPT and DMT attempts (e.g., CPT-1, CPT-1A, and DMT-6). In some of the areas where this layer was detected in all of the probings (such as in Areas 5, 6, and 8), this layer has been included among the soil units. In some areas where this layer does was not detected in all of the probings (e.g., Area 1, where this layer was detected in Boring B-12, but not in B-9), this layer has been eliminated from the soil profile, in order to maintain a conservative estimate of soil properties for this area. Despite its large density, because of its relatively small thickness, eliminating this layer from these profiles is not expected to result in overly conservative estimates of settlement or bearing capacity.

A layer of weathered limestone was encountered in a number of borings. In Area 1, this layer appears at an elevation of -25.0 to -30.0 feet, in the form of very dense sand with calcareous (shell) fragments. At approximately Elevation -40 feet, this layer transforms to a non-plastic hard silt (e.g., Boring B-12). or a plastic clay layer (both of which are probably still the continuation of the weathered limestone layer). This clay layer is about 7.0 feet thick, and is referred to as a “Stiff Clay” layer, because of its consistency (i.e., SPT resistance in the range of 11 to 31 whenever it is detected across the site). The properties of this layer, as estimated through laboratory tests, are summarized in Table 5-11. The engineering design parameters for this cohesive unit are reported in Section 6.0, and are slightly different from those in Table 5-11, because other information, such as SPT resistance is also taken into account to assess engineering parameters.

A sand layer with fines (some clay and silt, but not quite enough to call it clayey sand or a silty sand layer) underlies the “Stiff Clay” layer. The average SPT blow count in this sand layer is about 15. Even in the areas where the “Stiff Clay” layer has not been detected, this layer is believed to exist. This layer is predominantly olive gray in color, and because of its fine content and density, it is referred to as “Medium Dense Sand

Table 5-11
Clay Properties from Laboratory Tests[1]

Index and Strength Properties	
Natural Moisture Content, Wo (%)	53.7
Liquid Limit, WL (%)	95.3
Plastic Limit, Wp (%)	45.8
Plasticity Index, Ip (%)	49.5
Dry Unit Weight, γ_d (pcf)	69.2
Saturated Unit Weight, γsat (pcf)	106.7
Average Laboratory Undrained Shear Strength, Su (ksf)[2]	4.72
Consolidation Parameters	
Initial Void Ratio, e_o	1.416
Preconsolidation Pressure, σ'_p (ksf)	8.00
Compression Index, Cc	0.308
Rebound Index, Cr	0.050
Overconsolidation Ratio, OCR[3]	1.75

Notes:

[1]The properties reported in this table are only from laboratory tests. For final engineering design parameters for the clay units, refer to Section 6.0, where other information, such as SPT resistance is also taken into account.

[2]Su determined from UU tests.

[3]The OCR is based on an estimated in situ vertical effective stress of $\sigma'_{vo} = 4.59$ ksf.

w/fines." It appears as the lowermost layer in all of the soil profiles shown in Figures 5-2 through 5-7 (e.g., soil Units GS7, GN6, and FT6 in Figure 5-2), and in the corresponding tables. Despite some differences in density of this layer across the site, an average (and conservative) estimate of properties of this layer is used throughout the site, regardless of these small differences.

5.5.2 Groundwater Conditions

B&V monitored groundwater condition using four standpipe piezometers that were installed during subsurface investigation at the site. Table 5-12 shows groundwater elevation from the four piezometers at two dates shortly after these piezometers were developed. Data reported in this table (in conjunction with Figure 4-1, which shows the location of these piezometers) indicate that in general, the groundwater elevation drops from east to west, and from north to south. The deepest groundwater level is at PZ-3, which is located at the southwest of the site.

Table 5-12
Groundwater Elevation from Four Piezometers
Across the Site

Date	PZ-3	PZ-4	PZ-10	PZ-13
February 12, 2008	15.32	18.93	22.10	23.66
February 14, 2008	15.43	19.03	22.08	23.65

In addition to the data shown in Table 5-12, historical data about groundwater elevation are available from a piezometer at the premises of the subsurface investigation subcontractor (E&A), which is located close to the site. Figure 5-7 presents this set of data. This figure indicates that, at this location, groundwater fluctuates between Elevations +18 and +24 feet, and in general, it is lower during the winter months.

Using the grain size distribution of the fine sand, which dominants the overburden across the site, the hydraulic conductivity is estimated to be between 0.030 foot and 0.050 foot per minute (0.014 and 0.025 centimeter per second [cm/sec]).

6.0 Engineering Design Properties

Engineering parameters and design recommendations were developed using the results of the field investigation and the laboratory testing results.

6.1 Geotechnical Design Properties

Engineering design parameters were estimated for each soil unit comprising soil profiles in the eight areas defined in Section 5.0. The procedures followed to estimate these parameters, along with the values of the parameters are discussed in this section. Parameters were estimated to be a combination of the information collected during field investigation and from laboratory tests. The field data included those from SPT, CPT, and DMT investigations. The laboratory tests on cohesive units (i.e., clay or silt) included moisture content, Atterberg limits, unit weight, hydrometer, consolidation, and UU testing. For granular soils (mostly sand and rarely gravel in this site), sieve analysis was the main laboratory test conducted. The following is a summary of the procedures followed to estimate design parameters.

6.1.1 Description of the Procedures to Estimate Parameters

Relative density of granular material, Dr, is initially estimated using the established methods based on SPT resistance. The Dr is also estimated from CPT data. The following rule is used to decide the final value of Dr:

If $(Dr)_{spt} > (Dr)_{cpt}$ ➔ use $(Dr)_{spt}$
If $(Dr)_{spt} < (Dr)_{cp}$t ➔ use the average of $(Dr)_{spt}$ and $(Dr)_{cpt}$

The above procedure is primarily due to the fact that in general, $(Dr)_{cpt}$ happens to be smaller than $(Dr)_{spt}$, at least at this site. This is perhaps an artifact of the relationships used to estimate $(Dr)_{cpt}$. Relative density estimates from SPT resistance (N) have been used for a very long time and believed to be on the conservative side. B&V used thie procedure to avoid overly conservative estimates of Dr.

Estimation of some engineering parameters requires SPT resistance assuming a 60 percent hammer efficiency (i.e., 60 percent of the energy of the hammer is transferred to the SPT sampler at the end of the drilling rod). When the SPT resistance is corrected for 60 percent hammer efficiency, it is shown as N_{60}. The value of N_{60} is calculated assuming a hammer efficiency of 50 percent during field investigation (generally, a conservative assumption). With this assumption, $N_{60} = N/1.2$. One additional step in normalizing SPT resistance is to eliminate the effect of vertical effective stress, as if vertical effective stress at all depths is 1.0 atmospheric pressure. The result is shown by $(N_1)_{60}$.

Estimates of internal friction angle, Φ, using CPT and DMT data, happen to be quite often larger (and some times, significantly larger) than those from SPT. To remain on the safe side, $(\Phi)_{spt}$ is used primarily, unless the average from SPT, CPT, and DMT is smaller than $(\Phi)_{spt}$, in which case, this average is used.

For the elastic modulus of soil, Es, a weighted average of Es values from SPT, CPT, and DMT, is used. Weights (factors) of 3.0, 1.0, and 1.0 have been used, respectively, for these estimates, giving more importance to $(Es)_{spt}$. Estimates of $(Es)_{spt}$ are known to be quite conservative. By enforcing the above factors, the level of conservatism is reduced, while it still remains safe. $(Es)_{spt}$ is estimated by linear interpolation (or extrapolation, if needed) between the following points:

SPT N = 1 ➔ Es = 50 ksf
SPT N = 10 ➔ Es = 400 ksf
SPT N = 30 ➔ Es = 1,000 ksf
SPT N = 50 ➔ Es = 2,000 ksf

The results of laboratory tests on specimens from the clay unit, which was detected in an elevation of approximately -40 to -50 feet in some of the probings, were presented in Section 5.0. The undrained shear strength, Su, from laboratory tests is reduced to take into account the range of SPT resistance (N) encountered in this layer. The use of the same set of parameters for this clay layer across the site (i.e., in any of the areas where this layer is deemed present), is recommended in spite of the slight differences in N. For clay, undrained elastic modulus (Eu) is computed as Eu = 150 x Su, which is a conservative estimate.

Dynamic shear modulus, Gmax, is calculated directly from the shear wave velocity, Vs, of the strata as measured in the SCPT. These values are used for dynamic design of foundations for vibrating and rotating equipment, such as turbines, and are only available within Areas 1 and 2 where seismic testing was conducted. Vs values for each strata area were roughly averaged within each strata from plots of Vs versus elevation, and the values are presented in Subsection 6.1.2.

6.1.2 Values of the Engineering Design Parameters

By following the procedures described in Subsection 6.1.1, engineering parameters are estimated for all soil units across the site. These parameters are given in Tables 6-1 through 6-8 for Areas 1 through 8, which were delineated in Section 4.0.

Table 6-1
Summary of Soil Profile and Engineering Soil Properties in Area 1 (Power Block - South)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)	Vs (fps)	Gmax (ksf)
GS1	Very Loose and Loose Sand	34.5	20	14.5	90.0	115.0	7	4	9	50	30	380	0.0	680	1651
GS2	Upper Medium Dense Sand	20	5	15.0	100.0	120.0	25	20	21	82	35	900	0.0	850	2693
GS3	Dense Sand	5	-20	25.0	110.0	125.0	50	42	34	97	39	1,850	0.0	850	2805
GS4	Lower Medium Dense Sand	-20	-25	5.0	100.0	120.0	28	23	17	71	34	850	0.0	850	2693
GS5	Very Dense Sand	-25	-40	15.0	110.0	125.0	64	53	35	95	37	2,450	0.0	850	2805
GS6c(1)	Stiff Cohesive	-40	-47	7.0	70.0	110.0	19	16	10		0	450	3.0	1280	5597
GS7	Medium Dense Sand w/fines	-47	---	---	100.0	120.0	15	13	7	48	30	475	0.0	1280	6106

Note 1: Refer to Table 6-9 for a complete set of properties for the clay layer.

Table 6-2
Summary of Soil Profile and Engineering Soil Properties in Area 2 (Power Generating - North)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)	Vs (fps)	Gmax (ksf)
GN1	Very Loose and Loose Sand	32.5	20	12.5	90	115	7	4	9	50	30	400	0.0	450	723
GN2	Upper Medium Dense Sand	20	10	10	100	120	25	18	21	86	35	930	0.0	880	2886
GN3	Dense Sand	10	-23	33	110	125	40	33	28	90	37	1,400	0.0	880	3006
GN4	Very Dense Sand	-23	-50	27	110	125	56	47	31	90	36	1,850	0.0	880	3006
GN5	Medium Dense and Dense sand	-50	-80	30	100	120	30	25	14	65	33	1,150	0.0	1250	5823
GN6	Medium Dense Sand w/fines	-80	---	---	100	120	15	13	6	48	30	475	0.0	1250	5823

Table 6-3
Summary of Soil Profile and Engineering Soil Properties in Area 3 (Fuel Tanks Containment Area)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)
FT1	Very Loose and Loose Sand	28.5	20	8.5	90	115	9	6	15	65	31	500	0.0
FT2	Upper Medium Dense Sand	20	0	20	100	120	18	13	15	74	34	725	0.0
FT3	Dense Sand	0	-23	23	110	125	45	38	32	95	38	1,550	0.0
FT4	Upper Very Dense Sand	-23	-30	7	110	125	70	58	43	100	38	2,450	0.0
FT5	Lower Very Dense sand	-30	-70	40	110	125	57	48	30	88	38	2,250	0.0
FT6	Medium Dense Sand w/fines	-70	---	---	100	120	15	13	7	48	30	475	0.0

Table 6-4
Summary of Soil Profile and Engineering Soil Properties in Area 4 (Reuse Surge Tanks)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation. (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)
RS1	Very Loose and Loose Sand	37	24	13	90	115	5	3	6	43	30	380	0.0
RS2	Upper Medium Dense Sand	24	-15	39	100	120	24	20	18	75	34	990	0.0
RS3	Dense Sand	-15	-30	15	110	125	40	33	24	81	36	1,525	0.0
RS4	Lower Medium Dense Sand	-30	-45	15	100	120	15	15	8	50	31	650	0.0
RS5	Very Dense sand	-45	-70	25	110	125	55	46	26	80	36	2,000	0.0
RS6	Medium Dense Sand w/fines	-70	---	---	100	120	15	19	10	48	30	475	0.0

Table 6-5
Summary of Soil Profile and Engineering Soil Properties in Area 5 (North Cooling Tower)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)
CT1	Very Loose and Loose Sand	32	24	8.0	90.0	115.0	6	4	10	50	30	400	0.0
CT2	Upper Medium Dense Sand	24	20	4.0	100.0	120.0	20	13	17	86	34	1,000	0.0
CT3	Cemented Sand (very dense)	20	17	3.0	110.0	125.0	75	53	67	100	44	2,900	0.0
CT4	Upper Loose Sand	17	10	7.0	90.0	115.0	8	6	6	49	30	360	0.0
CT5	Lower Medium Dense Sand	10	-18	28.0	100.0	120.0	19	16	14	66	33	700	0.0
CT8	Lower Loose sand	-18	-36	18.0	90.0	115.0	8	7	5	42.	30	540	0.0
CT9	Very Dense Sand (Weathered Limestone)	-36	-44	8.0	110.0	125.0	51	43	28	87	37	2,000	0.0
CT10c(1)	Stiff Clay	-44	-51	7.0	70.0	110.0	15	13	8		0	450	3.0
CT11	Medium Dense Sand w/fines	-51	---	---	100.0	120.0	15	13	8	48	30	475	0.0

Note 1: Refer to Table 6-9 for a complete set of properties for the clay layer.

Table 6-6
Summary of Soil Profile and Engineering Soil Properties in Area 6 (South Cooling Tower)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)
CT1	Very Loose and Loose Sand	32	24	8.0	90.0	115.0	6	4	10	50	30	250	0.0
CT2	Upper Medium Dense Sand	24	20	4.0	100.0	120.0	20	13	17	86	34	1,000	0.0
CT3	Cemented Sand (very dense)	20	17	3.0	110.0	125.0	75	53	67	100	44	2,900	0.0
CT4	Upper Loose Sand	17	10	7.0	90.0	115.0	8	6	6	54	30	360	0.0
CT5	Lower Medium Dense Sand	10	5	5.0	100.0	120.0	19	15	16	71	34	700	0.0
CT6	Very Loose Sand	5	-5.5	10.5	90.0	115.0	1	1	1	30	28	290	0.0
CT7	Dense Sand	-5.5	-32	26.5	110.0	125.0	39	33	25	85	37	1,400	0.0
CT9	Very Dense Sand (weathered limestone)	-32	-40	8.0	110.0	125.0	51	43	28	88	37	2,000	0.0
CT10c[1]	Stiff Clay	-40	-47	7.0	70.0	110.0	15	13	8		0	450	3.0
CT11	Medium Dense Sand w/fines	-47	---	---	100.0	120.0	15	13	8	48	30	475	0.0

Note 1: Refer to Table 6-9 for a complete set of properties for the clay layer.

Table 6-7
Soil Profile and Engineering Soil Properties in Area 7 (Substation)[1]

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)
SB1	Very Loose Sand	28	24	4	90	115	4	3	8	46	31	310	0.0
SB2	Loose Sand	24	10	14	90	115	9	6	8	55	31	600	0.0
SB3	Medium Dense/Dense Sand	10	-28	38	110	125	34	28	25	85	36	1,200	0.0
SB4	Loose Sand w/fines	-28	-38	10	90	115	9	8	5	43	30	440	0.0
SB5	Very Dense Sand (weathered limestone)	-38	-43	5	110	125	75	63	42	100	40	2,850	0.0
SB6c[2]	Stiff Cohesive	-43	-50	7	70	110	12	10	6		0	450	3.0
SB7	Medium Dense Sand w/fines	-50	---	---	100	120	15	13	8	48	30	475	0.0

Note 1: Refer to Table 6-10 for L-Pile parameters at the substation area (Area 7).

Note 2: Refer to Table 6-9 for a complete set of properties for the clay layer.

Table 6-8
Soil Profile and Engineering Soil Properties in Area 8 (Detention Pond)

Soil Unit	Description	Top Elevation (ft)	Bottom Elevation (ft)	Thickness (ft)	γ_d (pcf)	γ_{sat} (pcf)	SPT $(N)_{av}$	N_{60}	$(N_1)_{60}$	Dr (%)	Φ' (deg)	Es (ksf)	Su (ksf)
DP1	Very Loose to Medium Dense Sand	24.5	16	8.5	90	115	8	5	9	60	31	360	0.0
DP2	Upper Very Dense Sand (Cemented)	16	13	3	110	125	63	39	55	100	43	2,650	0.0
DP3	Medium Dense Sand	13	0	13	100	120	18	13	15	73	33	640	0.0
DP4	Dense Sand	0	-26	26	110	125	35	29	25	86	37	1,250	0.0
DP5	Loose Sand	-26	-38	12	90	115	9	8	5	40	30	360	0.0
DP6	Very Dense Sand (Weathered Limestone	-38	-42	4	110	125	75	63	43	100	41	3,250	0.0
DP7c(1)	Stiff Cohesive	-42	-52	10	70	110	12	10	7		0	450	3.0
DP8	Medium Dense Sand w/fines	-52	---	---	100	120	15	13	8	48	30	475	0.0

Note 1: Refer to Table 6-9 for a complete set of properties for the clay layer.

Engineering design parameters for the only clay layer that was consistently encountered in some of the areas across the site are given in Table 6-9. The substation area (Area 7) is the only area where deep foundations (piles) are expected to be used. For this area, L-Pile parameters that are needed for analysis of the lateral capacity of piles are estimates and are presented in Table 6-10.

Table 6-9
Clay Engineering Parameters

Unit Weight and Strength Parameters	
Dry Unit Weight, γ_d (pcf) [2]	70.0
Saturated Unit Weight, γsat (pcf) [3]	110.0
Undrained Shear Strength, Su (ksf)	3.0
Consolidation Parameters	
Initial Void Ratio, e_o	1.416
Preconsolidation Pressure, σ'_p (ksf)	8.00
Compression Index, Cc	0.308
Rebound Index, Cr	0.050
Overconsolidation ratio, OCR [1]	1.75
Note 1: The OCR is based on an estimated in situ vertical effective stress of σ'_{vo} = 4.59 ksf.	

Table 6-10
L-Pile Parameters at Area 7 (Substation)

Soil Unit	Top Elevation (ft)	Bottom Elevation (ft)	γ' (pcf)	Φ' (deg)	K_h (pci)	C (psi)	e_{50}
SB1	28	24	0.052	31	81	---	---
SB2	24	10	0.030	31	80	---	---
SB3	10	-28	0.036	36	208	---	---
SB4	-28	-38	0.030	30	51	---	---
SB5	-38	-43	0.036	40	290	---	---
SB6c	-43	-50	0.028	---	1000	20.8	0.0050
SB7	-50	---	0.033	30	57	---	---

6.2 Corrosion Exposure

Testing for chemical analyses was performed by Environmental Conservation Laboratories Co. (ENCO) on five samples from the different layers in the soil profile. The tests included determination of sulfate content, chloride content, and pH. Chemical testing was reported in ENCO's Email Report of March 10, 2008. The following general guidelines were used in the initial screening for soil corrosiveness. Soils are generally considered a corrosive/aggressive environment for steel and/or concrete if the criteria and/or concentrations listed below are exceeded.

General Guidelines for Corrosive Soils (modified from FHWA, 1998)
pH below 4.5 (AASHTO T-289, ASTM G 51)
Water-Soluble Sulfate Content above 200 parts per million (ppm) (AASHTO T-290, ASTM D516M, ASTM D4327)
Water-Soluble Chloride Content above 100 ppm (AASHTO T-291, ASTM D512, ASTM D4327)

The results of chemical tests on soil specimens are included in Appendix E and are summarized in Table 6-11. The results indicate sulfate content ranging from 9.4 to 28.5 parts per million (ppm), and chloride content ranging from 5.3 ppm to 6.3 ppm. The pH of the site soils ranged from 4.6 to 5.9. The sulfate and chloride contents, as well as pH values, indicate a non-potentially corrosive environment.

Table 6-11
Chemical Test Results

Boring ID	Sample Mid-Depth (ft)	Chloride Content (mg/kg)	Sulfate Content (mg/kg)	pH
TP5	3.0	6.1	11.3	5.4
B-1	13.5	6.2	28.5	5.9
B-8	13.5	6.1	15.8	5.4
B-10	6.0	6.3	17.0	5.6
B-12	8.0	5.3	9.4	4.6

Concrete pile design is governed by American Concrete Institute (ACI) 543R-00, "Design, Manufacture, and Installation of Concrete Piles." ACI 543R-00 states that the cement type should be selected based on the exposure conditions and the durability requirements of ACI 318. A summary of the concrete sulfate exposure requirements from ACI 318 is provided below.

Concrete Sulfate Exposure Requirements (Modified from Table 4.3.1 ACI 318 (ACI, 2005)			
Sulfate Exposure	Water-Soluble Sulfate (SO_4) in Soil (percent by weight)	Sulfate (SO_4) in Water (ppm)	ACI Recommended Cement Type
Negligible	<0.10	<150	-
Moderate	0.10 to 0.20	150 to 1,500	II
Severe	0.20 to 2.00	1,500 to 10,000	V
Very Severe	>2.00	>10,000	V plus pozzolan

The maximum sulfate content among five samples was 0.00285 percent by weight, indicating a negligible sulfate exposure. Based on the results of the chemical analysis, Type I portland cement is recommended for foundation construction.

The chemical analysis results apply only to soils originating on the GEC site. If imported fill native material is used as fill beneath or adjacent to the foundations, additional chemical testing should be performed to verify that the corrosion protection recommendations are appropriate.

6.3 Electrical Resistivity

Complete results of the electrical soil resistivity survey are provided in Appendix I.

6.4 Soil Thermal Resistivity

Geotherm, Inc. performed Thermal resistivity testing. Tests were performed on samples at in situ moisture content and in a totally dry condition. The tests included the measurement of moisture content and thermal resistivity and were conducted in accordance with the Institute of Electrical and Electronics Engineers (IEEE) Standard 442. Table 6-12 provides a summary of the results. The thermal dryout curves are presented in Appendix A.

Table 6-12 Thermal Resistivity Test Results					
Location	Depth (ft)	Soil Description	In Situ Moisture Content (%)	Thermal Resistivity at In Situ Moisture Content (°C-cm/W)	Thermal Resistivity in Totally Dry Condition (°C-cm/W)
Pit 2	0.5 ~ 8.0	Yellow to tan sand	5.0	66	310
Pit 4	0.5 ~ 8.0	Light brownish yellow sand	3.0	128	347

7.0 Engineering Recommendations

The soils encountered across the site are adequate to support major foundations without causing excessive settlement. It is anticipated that all major and minor equipment can be founded on shallow foundations. B&V has estimated the footprints and equipment loadings provided in Table 7-1 for all equipment. A structural engineer should review the estimated footprints and equipment loadings for consistency and actual project information.

7.1 Objectives and Requirements

The objective of the foundation analysis is to provide foundation design recommendations with sufficient bearing capacity and acceptable differential and total settlements.

7.2 Shallow Foundations

Shallow foundations are recommended to have a minimum factor of safety of 3.0 against bearing capacity failure, 1.5 against sliding, and 1.5 against overturning, unless otherwise noted. For seismic, wind, equipment transient loading, or other unusual load combinations, a factor of safety of 1.1 for sliding and overturning may be used. All foundations, except tank foundations, will be limited to a total settlement of 1.5 inches and a differential settlement of 0.1 percent slope between adjacent concentrated load points or loaded areas, unless lower allowable settlements are specifically required for a particular structure. Industry practice is to allow settlements for tanks of up to about 6 inches.

As with major equipment, it is anticipated that the lightly loaded structures, such as equipment houses, small transformer pads, and auxiliary electrical equipment will be placed on shallow foundations. Any foundations for belowground pits should be designed for hydrostatic uplift water pressures.

7.2.1 Mat Foundations

Major plant equipment includes CTs, STs, and HRSGs. Allowable bearing capacity for each of these foundations far exceeds applied bearing pressures; therefore, equipment-induced settlements control design. Table 7-1 shows an estimate of maximum settlements anticipated for these three types of equipment. The estimates are based on a foundation embedment depth of 1 foot below ground surface. Additional embedment will further reduce total settlement. These settlements are immediate settlements that will occur during construction. No long-term settlement is anticipated.

Table 7-1 Estimated Total Settlement for Major Equipment			
	Footprint (ft)	Applied Pressure (ksf)	Estimated Settlement (in.)
CT	36 x 150	1.0	< 0.5
ST	80 x 105	1.5	1.2
HRSG	55 x 120	1.5	0.8
Df = 1.0 ft			

7.2.2 Tanks

There are several tank types for the proposed construction: reuse surge tanks, fuel oil tanks, and demineralized water tanks. Each of these tanks types has been analyzed for assumed loading conditions, and total and differential settlement results are reported in Table 7-2.

Table 7-2 Estimated Settlement of Tanks							
	Capacity (million gal)	Diameter (ft)	Fluid Height (ft)	Applied Pressure (ksf)	Estimated Edge Settlement (in.)	Estimated Center Settlement (in.)	Estimated Differential Settlement (in.)
Reuse Tank*	5.1	190	24	1.5	3.9	5.3	1.4
Demineralized Water Tank	1.6	90	34	2.1	2.4	1.8	0.6
Fuel Oil Tank	3.8	125	41	2.6	3.1	4.0	0.9
*Additional reuse tanks were added and locations were modified after completion of the investigation. ACI 372 recommends more investigation locations for prestressed concrete tanks of this diameter than were performed. Additional investigation may be required depending on manufacturer recommendations.							

The allowable bearing capacity is more than 8.0 ksf, much greater than the design bearing pressure of 2.0 ksf. Since the tanks are in groups, which is the effect of overlapping soil stresses was analyzed. Settlements at the perimeters of the tanks were analyzed when only one tank is filled versus when more than one tank is filled. Filling one tank induces a maximum settlement of 5.3 inches at the center of a full tank, 3.9 inches at the outside edge of the lone full tank, and 1.25 inches at the edge of the neighboring tank. The settlement at the edge of the neighboring tank and the edges of the diagonally neighboring tanks are unaffected because of their distance.

The estimated settlements are conservatively based on the loading occurring at the ground surface rather than at an embedment depth. An increased embedment depth will decrease total and differential settlement. Although these results are likely to be conservative upper-bound settlement estimates, they should be confirmed to appropriately represent the final configuration and loading conditions once the final configuration and manufacturer foundation requirements are finalized.

Due to the relative uniformity of the subsurface material and the depth of the more compressible stratum, tank settlements are anticipated to be reasonably uniform. Steel tanks typically require concrete ringwall foundations with a well compacted sand or gravel base underneath the tank. Ringwalls are 6 feet or less in depth (governed mostly by frost depth). The ringwalls may require a footing if the tank is subjected to high environmental lateral loads (i.e., seismic) or heavy peripheral loads from piping or equipment.

A concrete tank will use a shallow mat to support the walls. The tank wall will be poured monolithically after the foundation, anchored by radial dowels, and sealed with a waterstop installed at the wall prior to the foundation mat pour. Floor thickness will be governed by fluid bending forces at the bottom of the wall in the full condition, and external wind and supporting soil load in the empty load condition.

7.2.3 Square and Strip Footings

Square and strip footings should be sized in accordance with Figures 7-1 and 7-2. Bearing pressures were calculated for an embedment depth of 1 foot. For settlements exceeding 0.5 inch, the maximum allowable bearing pressures are limited to 3 ksf. For equipment requiring settlements less than 0.5 inch, the limiting pressure is shown on Figures 7-1 and 7-2. Bearing capacity will increase with embedment depth.

The following limitations apply to shallow foundations:

- The minimum footing width is 2.0 feet.
- The minimum footing embedment is based upon the Florida Building Code and is set at 12 inches.
- A strip footing is defined as having a ratio of length (L) to width (B) greater than 10. A square footing is defined as having L/B = 1. To estimate the footing settlement with 1>L/B>10, the following interpolation procedure can be used:
  - Obtain settlements from the design charts for both a square and a strip footing using the actual footing width B.
  - Interpolate the settlements for the actual L/B ratio, assuming L/B = 1 for a square footing and L/B = 10 for a strip footing.

- No shallow footing should be founded on topsoil, debris, or loose fill.
- Footing pressure shall be controlled by either allowable settlements or allowable bearing capacity of the subgrade soils. B&V recommends that individual column footings adjacent to strip footings be proportioned so that settlements of the two foundations minimize potential differential settlements.
- The settlement values presented are for rigid foundation systems.
- Calculations are based on a groundwater Elevation of 22 feet.

For foundations with bearing pressures greater than the allowable pressures shown, with widths less than 2.0 feet and/or with alternative embedment depths, the recommended foundation configuration should be developed on a case-by-case basis. The effect of adjacent footings on foundation capacity due to stress overlap is not included in the information provided on Figures 7-1 and 7-2. To avoid overstressing the soil, it is recommended that all shallow footings be spaced at a distance of four times the base width. Closer spacings could increase both settlement and rotation of individual footings and would require a case-by-case analysis.

7.3 Deep Foundations

Drilled piers will probably be required for transmission structures in the substation. It is not anticipated that deep foundations will be required for other structures on the site; however, if settlements reported in Subsections 7.2.1 and 7.2.2 are found to be in excess of manufacturer requirements, deep foundations and/or ground improvement may be considered. Augered cast-in-place (ACIP) piles are probably the most cost-effective alternative based on ground conditions.

7.3.1 Augered Cast-in-Place Piles

ACIP piles are installed by boring a continuous flight auger to a specific depth and introducing grout while concurrently withdrawing the auger. If necessary for uplift, moment, or shear loads, a reinforcement cage is lowered into the fluid grout column immediately after the auger is withdrawn. The length of the reinforcement cage for shear and moment loading is not necessarily the full length of the pile and is governed by the magnitude of the loads. For uplift piles, a single center bar is typically installed to a minimum depth necessary to ensure full tensile capacity. Air entrainment or Type II concrete should be utilized to minimize the corrosive effects of the soil. Piles should generally be installed with a minimum pile-to-pile spacing of three pile diameters.

7.3.2 Lateral Pile Capacity

The allowable lateral capacity is typically defined as the load required to generate 0.25 inch and 0.5 inch of pile head deflection for fixed and free head conditions, respectively. Fixed head conditions allow no rotation of the pile head within the pile cap and transfer bending moments to the cap. Free head conditions do not restrain pile head rotation and do not transfer bending moments to the pile.

For lateral pile analysis using Ensoft's LPILE Plus Version 4.0, parameters are provided in Table 7-3. To avoid capacity inefficiencies, a minimum center-to-center pile spacing of three diameters should be used for pile groups. Pile capacities should be adjusted for group effects using the following equation (Mokwa 1999) and as shown on Figures 7-3 and 7-4.

$$G_e = \frac{\sum f_m}{N}$$

Where:

G_e = Group efficiency,

Σf_m = Sum of row p-multipliers, and

N = Number of rows in the direction of load.

Table 7-3
L-Pile Parameters for Lateral Pile Analysis

Soil Type	Elevation Interval (ft)	γ' (pci)	C (psi)	e_{50}	Phi (deg)	k_h (pci)
Sand	28 to 24	0.052	--	--	31	81
Sand	24 to 10	0.030	--	--	31	80
Sand	10 to -28	0.036	--	--	36	208
Sand	-28 to -38	0.030	--	--	30	51
Sand	-38 to -43	0.039	--	--	40	290
Stiff Clay w/o Free Water	-43 to -50	0.028	20.8	0.0050	--	1,000
Sand	Below -50	0.033	--	--	30	57

7.4 Ground Improvement

It is anticipated that unmodified existing soil conditions are sufficient to support the proposed structures. However, if settlements reported in Subsections 7.2-1 and 7.2-2 are found to be in excess of manufacturer requirements, deep foundations and/or ground improvement may be considered. Ground improvement could densify and improve the unsaturated very loose to loose sands in the upper 5 to 15 feet. Based on cost and suitability to the application, deep dynamic compaction (DDC) is probably the best option at this site. Another option for ground improvement is vibro-densification. Both options are discussed in the following subsections. For both options, a design verification of the treatment program would be required and would involve field testing following completion of the treatment program.

7.4.1 Deep Dynamic Compaction

DDC involves the application of energy to the soil by dropping a weight (6 to 35 tons) from a predetermined height (40 to 120 feet) to rearrange soil particles into a denser configuration. The denser configuration provides better bearing and settlement characteristics for foundation performance under both static and seismic (reduced liquefaction potential) loading. DDC has been effectively used in Florida. The effectiveness of DDC is generally limited to freely draining soils and soils above the water table. Some special cases, however, have been performed in saturated conditions. Compaction is accomplished by repeatedly raising and dropping a large weight from a crane, thus imparting large amounts of energy into the soil. Soil densification and strengthening occur as pore water pressures drain from the effected soil. The resulting radial fissures serve to assist the drainage of low permeability soils. DDC is not effective for consolidating clays.

Basic design considerations involve the selection of the weight and drop height, the number of applications (passes), and to what extent the weight or drop height is varied between passes. Dynamic compaction is not suitable at sites where the groundwater table is encountered within the crater left after dropping the weight (usually 5 to 12 feet).

Specially modified cranes should be used for dynamic compaction to withstand the reactions that occur after the weight is released. Earth moving equipment should be on hand to fill the impact craters with soil (preferably with clean sand) following each pass of dynamic compaction.

7.4.2 Vibro-Densification

In situ densification of granular soils can be accomplished by vibrating a steel pipe into the soil and allowing the surrounding soils to compact and densify. This method does not introduce new material into the soil, but compacts soils in situ as a result of the applied energy, commonly resulting in ground subsidence. Soils with a relative density below 60 percent are best suited for this method. Vibro-densification is not effective for treating clays.

7.5 Lateral Earth Pressures

The recommended procedures for calculating pressures against yielding and non-yielding walls are presented on Figure 7-5, which is based on the evaluation and testing of existing site soils. If backfill is obtained from sources other than the areas excavated for the foundations, the lateral earth pressure coefficients must be reevaluated.

8.0 Geotechnical Construction Considerations

8.1 Site Preparation

With an estimated plant grade of Elevation 32 feet, site grading will entail mostly filling, generally ranging from 1 foot of fill at the western fuel oil tanks to 3 feet of fill at the southern end of the substation. Contours indicate that some isolated high spots will be encountered. The site was left with a hummocky landscape as the result of the tree removal with some sizeable brush piles from previous clearing and grubbing. Clearing and grubbing of the remaining trees, shrubs, debris, and vegetation flush with the ground surface will be required prior to the start of cutting and filling. Deleterious materials such as organics and topsoil must be completely removed prior to subgrade preparation. Overexcavation and replacement of such unsuitable subgrade materials is anticipated in low-lying regions.

8.2 Earthwork

Earthwork at the site will consist of structural excavation and fill, and general site fill. When overexcavation and fill are required beneath the structures (i.e., when organics are encountered), the areas to receive fill should be moisture conditioned and proof rolled. All identified loose or soft zones should be removed and backfilled with material, and compacted in place. Structural fill should be placed in uniform, horizontal loose lifts, moisture conditioned as necessary, and compacted by mechanical means to at least 98 percent of maximum dry density as determined by ASTM D1557 (Modified Proctor). Moisture contents should fall within the range of ±2 percent of optimum moisture content. Backfill lifts should be limited to 8 inches or less in uncompacted lift thickness. Particles larger than 3 inches should be excluded from the top 1 foot of fill.

General fill consists of all areas that are required to raise the grade or fill in the depressions during the grading operations at the site. General site fill should be compacted to 95 percent of maximum dry density as determined by ASTM D1557. Moisture content should fall within the range of ±2 percent of optimum moisture content. General fill lifts should be limited to 8 inches or less in compacted thickness.

Maximum Modified Proctor dry densities for tested clean sands are approximately 102 pounds per cubic foot (pcf), and optimum moisture content is approximately 16 percent. Fines contents ranged from 0.8 percent to 3.6 percent. Modified Proctor test results are included in Appendix E.

8.3 Temporary/Permanent Excavations

Excavations within the top 12 feet of soil will likely be in a loose granular soil. Excavations that are greater than 4 feet in depth will likely require some sort of bracing or sloping of the sidewalls. The soil at the site is classified by the Occupational Safety and Health Administration (OSHA) as Type C soil; therefore, all excavations and subsequent shoring should be designed to such standards. All trench side slopes should be constructed to a maximum of 1.5 horizontal : 1 vertical.

8.4 Dewatering

For any excavation that is greater than 10 feet in depth, it is likely that groundwater will be encountered; these excavations will probably require dewatering. The maximum groundwater level reported in the area was Elevation 23.8 in August 2007; historically, the water level generally stays below Elevation 22 feet in the wetter summer months and below 21 feet in the drier winter months. Fluctuations in groundwater have shown increasing variability in recent years that should be anticipated to continue. The fine sand acts as an aquifer for transport of free water to excavations. Deeper excavations, or those that encounter free water at shallow depths, should be dewatered with an engineered well point system. All excavations should be monitored for deep erosion because of the infiltrating groundwater. The dewatering should lower the groundwater level a minimum of 2 feet below the base of the excavation.

8.5 Temporary/Permanent Slopes

Any constructed slopes will require case-by-case design and analysis.

9.0 References

1. BCI Engineers and Scientists, Inc., Final Report, "Density of Hydraulic Fill and Settlement Estimates," Corbett Area B, Compliance with Backfill Agreement, August 2005.

2. Federal Emergency Management Agency Web, site, http://www.fema.gov.

3. International Building Code, 2003.

4. Natural Resources Conservation Service, Web Soil Survey, http://websoilsurvey.nrcs.usda.gov.

5. Bowles, Joseph E., *Foundation Analysis and Design*, McGraw-Hill Companies, Inc., 1996.

6. 2004 Florida Building Code

7. FHWA Cone Penetrometer Test, Publication Number SA-91-043, 1991, p. 71.

8. Mokwa, R. L. "Investigation of the Resistance of Pile Caps to Lateral Loading," 1999.

9. Leve, G. W., Ground Water in Duval and Nassau Counties, Florida: Florida Board of Conservation, Division of Geology Report of Investigations No. 43, 1966.

10. Scott, T. M., The lithostratigraphy of the Hawthorn Group (Miocene) of Florida: Florida Geological Survey Bulletin 59, 1988.

11. Randazzo and Jones, The Geology of Florida, Tallahassee, University of Florida Press, 1997.

12. Sinclair, William C., and Stewart, J.W.; Sinkhole Type, Development and Distribution in Florida Map, Florida Dept. of Natural Resources Bureau of Geology, Tallahassee, Florida, 1985

Figures

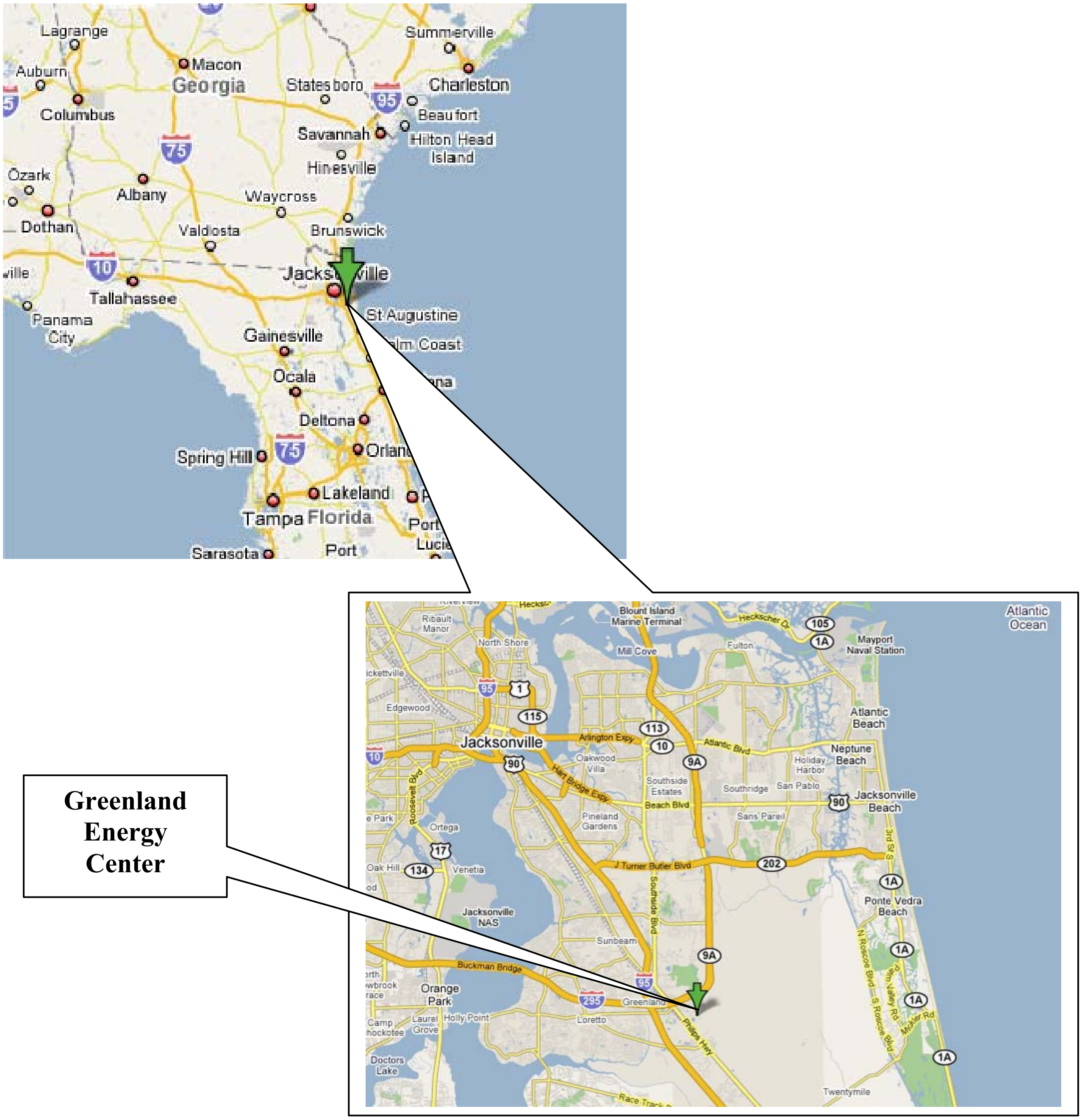
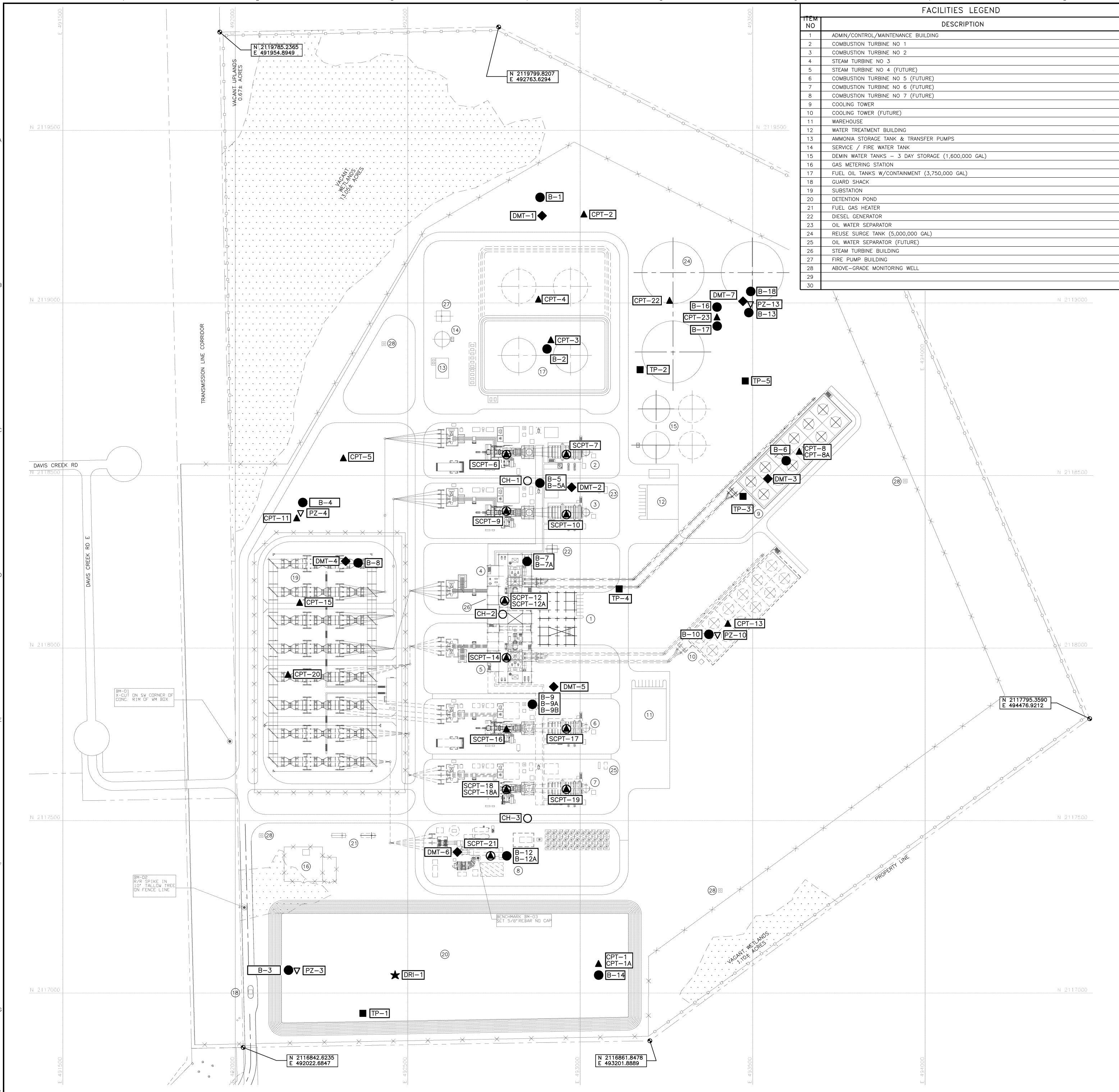


Figure 3-1
Project Location



FACILITIES LEGEND

ITEM NO	DESCRIPTION
1	ADMIN/CONTROL/MAINTENANCE BUILDING
2	COMBUSTION TURBINE NO 1
3	COMBUSTION TURBINE NO 2
4	STEAM TURBINE NO 3
5	STEAM TURBINE NO 4 (FUTURE)
6	COMBUSTION TURBINE NO 5 (FUTURE)
7	COMBUSTION TURBINE NO 6 (FUTURE)
8	COMBUSTION TURBINE NO 7 (FUTURE)
9	COOLING TOWER
10	COOLING TOWER (FUTURE)
11	WAREHOUSE
12	WATER TREATMENT BUILDING
13	AMMONIA STORAGE TANK & TRANSFER PUMPS
14	SERVICE / FIRE WATER TANK
15	DEMIN WATER TANKS – 3 DAY STORAGE (1,600,000 GAL)
16	GAS METERING STATION
17	FUEL OIL TANKS W/CONTAINMENT (3,750,000 GAL)
18	GUARD SHACK
19	SUBSTATION
20	DETENTION POND
21	FUEL GAS HEATER
22	DIESEL GENERATOR
23	OIL WATER SEPARATOR
24	REUSE SURGE TANK (5,000,000 GAL)
25	OIL WATER SEPARATOR (FUTURE)
26	STEAM TURBINE BUILDING
27	FIRE PUMP BUILDING
28	ABOVE-GRADE MONITORING WELL
29	
30	

PROJECT SURVEY CONTROL

CONTROL MONUMENT LOCATIONS

MONUMENT NO.	PLANT COORDINATES NORTHING	PLANT COORDINATES EASTING	MONUMENT NO.	STATE PLANE COORDINATES NORTHING	STATE PLANE COORDINATES EASTING	ELEVATION
BM-01	–	–	BM-01	2117750.54	491972.45	20.98
BM-02	–	–	BM-02	2117269.19	492013.21	20.90
BM-03	–	–	BM-03	2117410.43	492691.36	35.08

NOTES:
1. HORIZONTAL CONTROL IS BASED ON THE FLORIDA STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAD83. VERTICAL CONTROL IS BASED ON THE NAVD88 DATUM.
2. VERTICAL CONTROL IS BASED ON NATIONAL GEODETIC VERTICAL DATUM (NGVD) OF 1929 WITH THE ORIGINATING BENCHMARK BEING A BRASS JEA DISK STAMPED "GPS CONTROL POINT – STATION 179" SET IN CONCRETE WINGED END WALL, WESTERLY SIDE OF JEA POWER LINE EASEMENT – ELEVATION BEING 26.83 FEET.
3. BENCHMARK COORDINATES ARE BASED ON ASSUMED VALUES. CONTRACTOR TO VERIFY BENCHMARKS PRIOR TO WORK.

INVESTIGATION POINTS

INVESTIGATION	COORDINATES NORTH (FEET)	COORDINATES EAST (FEET)	GROUND SURFACE ELEVATION (FEET)	FINAL DEPTH (FEET)	REMARKS
B-1	2119306.2	492883.5	29.5	99.5	
B-2	2118866.9	492903.7	27.6	100	
B-3	2117066.7	492154.3	24.1	75	
PZ-3	2117066.7	492158.3	27.1	25	PIEZOMETER
B-4	2118407.9	492195.6	25.2	50	
PZ-4	2118391.4	492189.4	27.8	24	PIEZOMETER, SLUG TEST
B-5	2118477	492883	32.6	125	
B-5A	2118481	492887	33.7	108	
B-6	2118543	493597	32.7	75	
B-7	2118249	492846	38.1	125	
B-7A	2118256	492846	37.8	80	
B-8	2118247	492356	29.1	50	
B-9	2117846	492856	32.3	87	
B-9A	2117846	492864	33.1	125	
B-9B	2117838	492864	32.9	76	
B-10	2118040	493373	32.1	75	
PZ-10	2118039.5	493382.9	34.9	25	PIEZOMETER, SLUG TEST
B-12	2117403	492787	33.8	125	
B-12A	2117393	492787	34.3	125	
B-13	2118978.3	493488.8	36	119.3	
PZ-13	2118988.3	493493.8	39	25	PIEZOMETER
B-14	2117066.3	493053.5	25.2	50	
B-16	2118988.3	493396.5	35.5	50	
B-17	2118933.2	493397	35.4	50	
B-18	2119033.5	493494.3	36.1	50	
CH-1	2118478	492849			
CH-2	2118093	492777			
CH-3	2117504	492849			
CPT-1	2117066	493053		9.7	
CPT-1A	2117064	493053		9.2	
CPT-2	2119256	493010.6	30.6	66.1	
CPT-3	2118891	492915		61.2	
CPT-4	2119009.1	492878.9	28.6	86.1	
CPT-5	2118549.8	492313.3	26.4	85.9	
SCPT-6	2118561	492786	29.9	120.3	SHEAR WAVE EVERY OTHER ROD
SCPT-7	2118561	492960	33.2	120.9	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
CPT-8	2118569	493635	33.0	17.2	
CPT-8A	2118567	493635	33.0	78.4	
SCPT-9	2118387	492786	33.3	101	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-10	2118387	492960	33.6	116	SHEAR WAVE EVERY OTHER ROD
CPT-11	2118374.7	492178	24.7	75.1	
SCPT-12	2118137	492781	33.7	70.3	SHEAR WAVE EVERY OTHER ROD
SCPT-12A	2118135	492781	33.7	105.4	SHEAR WAVE EVERY OTHER ROD
CPT-13	2118069	493428	31.6	120.2	
SCPT-14	2117972	492786	31.8	64.6	SHEAR WAVE EVERY OTHER ROD
CPT-15	2118130.4	492186.5	27.4	120	
SCPT-16	2117765	492786	30.4	77.2	
SCPT-17	2117766	492960	32.6	125.8	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-18	2117590	492786	33.2	65.9	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-18A	2117588	492786	33.2	126.1	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-19	2117591	492960	32.9	120.4	SHEAR WAVE EVERY OTHER ROD
CPT-20	2117921.5	492152.8	26.6	61.7	SHEAR WAVE EVERY OTHER ROD
SCPT-21	2117398	492740	32.8	110.6	SHEAR WAVE EVERY OTHER ROD
CPT-22	2119005.3	493258.9	32.6	116.9	SHEAR WAVE EVERY OTHER ROD
CPT-23	2118950	493401	35.8	120.2	
DMT-1	2119253	492889.5	29.8	50	MAY REQUIRE AUGERING TO MARL
DMT-2	2118466	492973	34.6	45	MAY REQUIRE AUGERING TO MARL
DMT-3	2118491	493544	32.4	9	
DMT-4	2118251.8	492319.9	28.5	35	
DMT-5	2117888	492923	33.4	25	MAY REQUIRE AUGERING TO MARL
DMT-6	2117409	492644	34.6	10	MAY REQUIRE AUGERING TO MARL
DMT-7	2119005.1	493471.7	36	45	MAY REQUIRE AUGERING TO MARL
DRI-1	2117051.6	492463.5	25.7		TEST ELEVATION 3 FT ABOVE GROUNDWATER TABLE
TP-1	2116942	492370	23.7	8	
TP-2	2118807	493174	33	9	
TP-3	2118440	493472	34.1	8	
TP-4	2118172	493113	37.4	8	
TP-5	2118774.4	493479	35.4	8	
DSR-1	2118621	493010		N/A	SEE DRAWING 160167-DS-0002
DSR-2	2117771	492630		N/A	SEE DRAWING 160167-DS-0002
TSR-1	2119036	492572		N/A	SEE DRAWING 160167-DS-0002
TSR-2	2119060	493198		N/A	SEE DRAWING 160167-DS-0002
TSR-3	2118672	492288		N/A	SEE DRAWING 160167-DS-0002
TSR-4	2118766	493620		N/A	SEE DRAWING 160167-DS-0002
TSR-5	2118540	492661		N/A	SEE DRAWING 160167-DS-0002
TSR-6	2118521	493291		N/A	SEE DRAWING 160167-DS-0002
TSR-7	2118220	492455		N/A	SEE DRAWING 160167-DS-0002
TSR-8	2118244	493053		N/A	SEE DRAWING 160167-DS-0002
TSR-9	2118243	493674		N/A	SEE DRAWING 160167-DS-0002
TSR-10	2117849	492279		N/A	SEE DRAWING 160167-DS-0002
TSR-11	2117968	492881		N/A	SEE DRAWING 160167-DS-0002
TSR-12	2117744	493251		N/A	SEE DRAWING 160167-DS-0002
TSR-13	2117378	492280		N/A	SEE DRAWING 160167-DS-0002
TSR-14	2117369	493014		N/A	SEE DRAWING 160167-DS-0002

LEGEND

- ▲ CPT (CONE PENETRATION TEST)
- ◆ DILATOMETER TEST
- ○ CH (SEISMIC CROSS-HOLE TEST)
- SEISMIC CPT
- ★ DOUBLE RING INFILTROMETER
- ▽ PIEZOMETERS
- ● SPT BORING
- ■ TEST PIT

NOTES:
1. ADDITIONAL TEST PITS MAY BE ADDED.
2. FOR DMT-1, 2, 5 & 7 PUSH WITH CONE RIG UNTIL REFUSAL, THEN AUGER THROUGH DENSE CEMENTED SANDS AND CONTINUE TO A DEPTH OF 120 FT, INCLUDED 90' DRILLING FOR EACH.
3. START CPT'S CONCURRENT WITH SPT. FIRST SPT SHOULD BE B-5.
4. SPT SAMPLING – 4 SPOONS IN TOP 10 FEET, EVERY 5' THEREAFTER UNLESS DIRECTED OTHERWISE.
5. AT SCPT-7, 9, 17 & 18 AN OFFSET LOCATION IS TO BE PREDRILLED TO DEPTH SPECIFIED BY B&V FIELD REPRESENTATIVE AND BACKFILLED WITH APPROVED GRANULAR MATERIAL TO OBTAIN SHEAR WAVE VELOCITY MEASUREMENTS BELOW REFUSAL ZONE.
6. CROSS-HOLE TESTING WILL BE PERFORMED IF SCPT RESULTS REFERENCED IN NOTE 5 ARE UNSATISFACTORY.
7. DMT INTERVALS – 5, 10, 15, 20, 30, ETC. TO REF: FRO DEEP THEN 10' INTERVALS INTO MARL AFTER AUGERING THROUGH DENSE ZONE.

NOT TO BE USED FOR CONSTRUCTION

NO	DATE	REVISIONS AND RECORD OF ISSUE	DRN	DES	CHK	PDE	APP
1	01/MAY/08	ISSUED FOR ADDENDUM 1	DS	DS		TD	DG
0	26/MAR/08	ISSUED FOR INFORMATION ONLY	DS	DS		TD	DG

100' 50' 0 100' 200'
1" = 100'

I HEREBY CERTIFY THAT THIS DOCUMENT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF FLORIDA.
SIGNED ______
DATE ______ REG NO. ______

BLACK & VEATCH
10751 Deerwood Park Boulevard, Suite 130, Jacksonville, Florida 32256

ENGINEER | DRG | DRAWN DAS
CHECKED | DATE

JEA
GREENLAND ENERGY CENTER

PLANT ARRANGEMENT
SUBSURFACE INVESTIGATION PLAN

PROJECT 160167-DS-0001 (DRAWING NUMBER) REV 1
CODE | AREA
CERTIFICATE OF AUTHORIZATION NO. 8132

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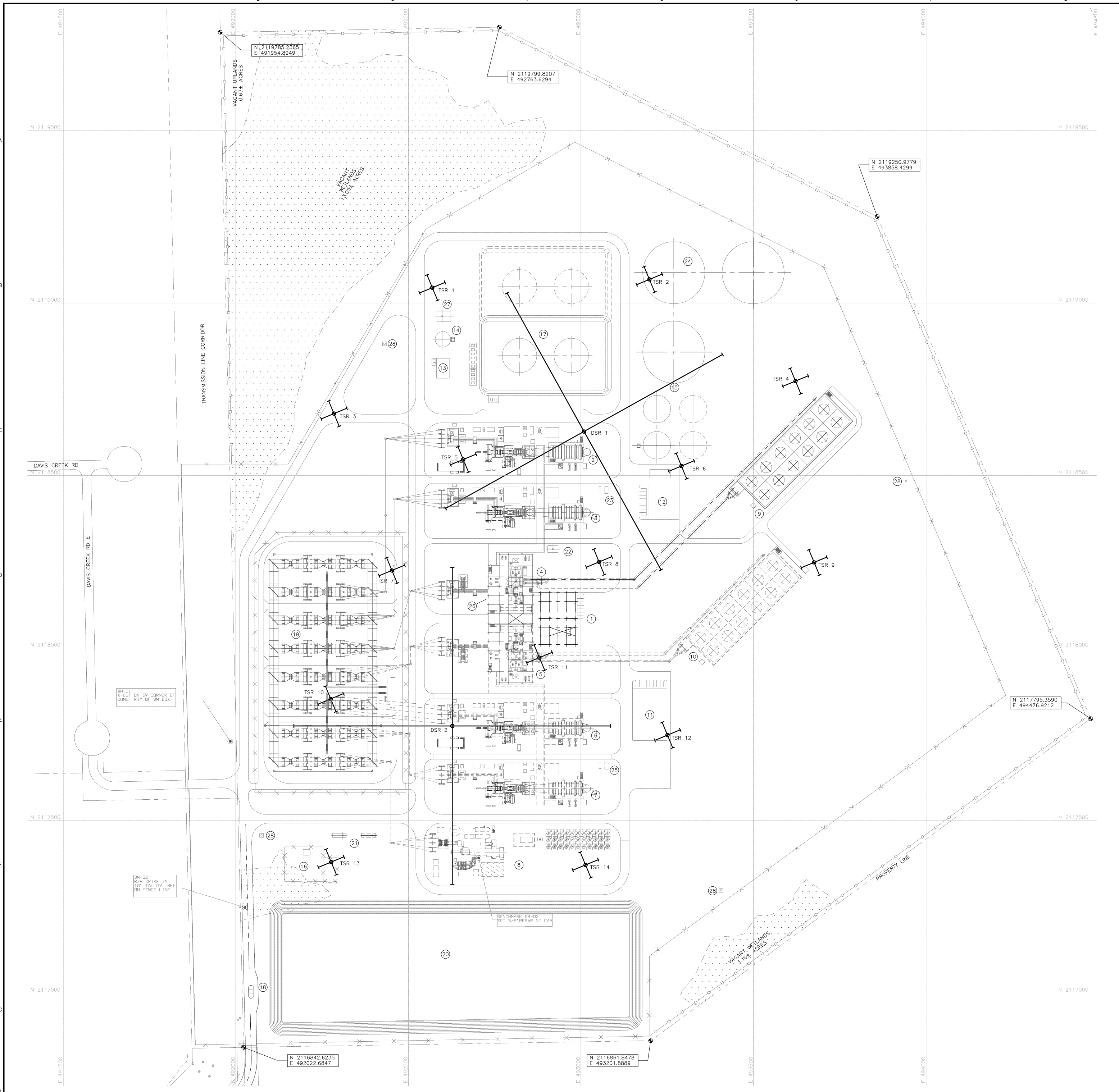

FACILITIES LEGEND

ITEM NO	DESCRIPTION
1	ADMIN/CONTROL/MAINTENANCE BUILDING
2	COMBUSTION TURBINE NO 1
3	COMBUSTION TURBINE NO 2
4	STEAM TURBINE NO 3
5	STEAM TURBINE NO 4 (FUTURE)
6	COMBUSTION TURBINE NO 5 (FUTURE)
7	COMBUSTION TURBINE NO 6 (FUTURE)
8	COMBUSTION TURBINE NO 7 (FUTURE)
9	COOLING TOWER
10	COOLING TOWER (FUTURE)
11	WAREHOUSE
12	WATER TREATMENT BUILDING
13	AMMONIA STORAGE TANK & TRANSFER PUMPS
14	SERVICE / FIRE WATER TANK
15	DEMIN WATER TANKS – 3 DAY STORAGE (1,600,000 GAL)
16	GAS METERING STATION
17	FUEL OIL TANKS W/CONTAINMENT (3,750,000 GAL)
18	GUARD SHACK
19	SUBSTATION
20	DETENTION POND
21	FUEL GAS HEATER
22	DIESEL GENERATOR
23	OIL WATER SEPARATOR
24	REUSE SURGE TANK (5,000,000 GAL)
25	OIL WATER SEPARATOR (FUTURE)
26	STEAM TURBINE BUILDING
27	FIRE PUMP BUILDING
28	ABOVE-GRADE MONITORING WELL (SEE DWG 160167-DS-0055)
29	
30	

PROJECT SURVEY CONTROL

CONTROL MONUMENT LOCATIONS

MONUMENT NO.	PLANT COORDINATES		MONUMENT NO.	STATE PLANE COORDINATES		ELEVATION
	NORTHING	EASTING		NORTHING	EASTING	
BM-01	–	–	BM-01	2117750.54	491972.45	20.98
BM-02	–	–	BM-02	2117269.19	492013.21	20.90
BM-03	–	–	BM-03	2117410.43	492691.36	35.08

NOTES:

1. HORIZONTAL CONTROL IS BASED ON THE FLORIDA STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAD83. VERTICAL CONTROL IS BASED ON THE NAVD88 DATUM.
2. VERTICAL CONTROL IS BASED ON NATIONAL GEODETIC VERTICAL DATUM (NGVD) OF 1929 WITH THE ORIGINATING BENCHMARK BEING A BRASS JEA DISK STAMPED "GPS CONTROL POINT – STATION 179" SET IN CONCRETE WINGED END WALL, WESTERLY SIDE OF JEA POWER LINE EASEMENT – ELEVATION BEING 26.83 FEET.
3. BENCHMARK COORDINATES ARE BASED ON ASSUMED VALUES. CONTRACTOR TO VERIFY BENCHMARKS PRIOR TO WORK.
4. FOR TSR AND DSR COORDINATE LOCATIONS, SEE PROPOSED INVESTIGATIONS TABLE ON DWG. 160167-DS-0001.

SOIL RESISTIVITY ELECTRODE "A" SPACINGS

TSR (FT)	DSR (FT)
1	1
2	2
3	3
6	6
10	10
20	20
30	30
	60
	100
	200
	300

NOT TO BE USED
FOR CONSTRUCTION

NO	DATE	REVISIONS AND RECORD OF ISSUE	DRN	DES	CHK	PDE	APP
1	01/MAY/08	ISSUED FOR ADDENDUM 1	DS	DS		TD	DG
0	26/MAR/08	ISSUED FOR INFORMATION ONLY	DS	DS		TD	DG

100' 50' 0 100' 200'
1"=100'

I HEREBY CERTIFY THAT THIS DOCUMENT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF FLORIDA.
SIGNED ____________
DATE ________ REG NO. ________

BLACK & VEATCH 10751 Deerwood Park Boulevard, Suite 103 Jacksonville, Florida 32256
ENGINEER DRG DRAWN DAS
CHECKED DATE

JEA
GREENLAND ENERGY CENTER
PLANT ARRANGEMENT
RESISTIVITY TEST LOCATIONS PLAN

PROJECT DRAWING NUMBER 160167-DS-0002 REV 1
CODE
AREA
CERTIFICATE OF AUTHORIZATION NO. 8132

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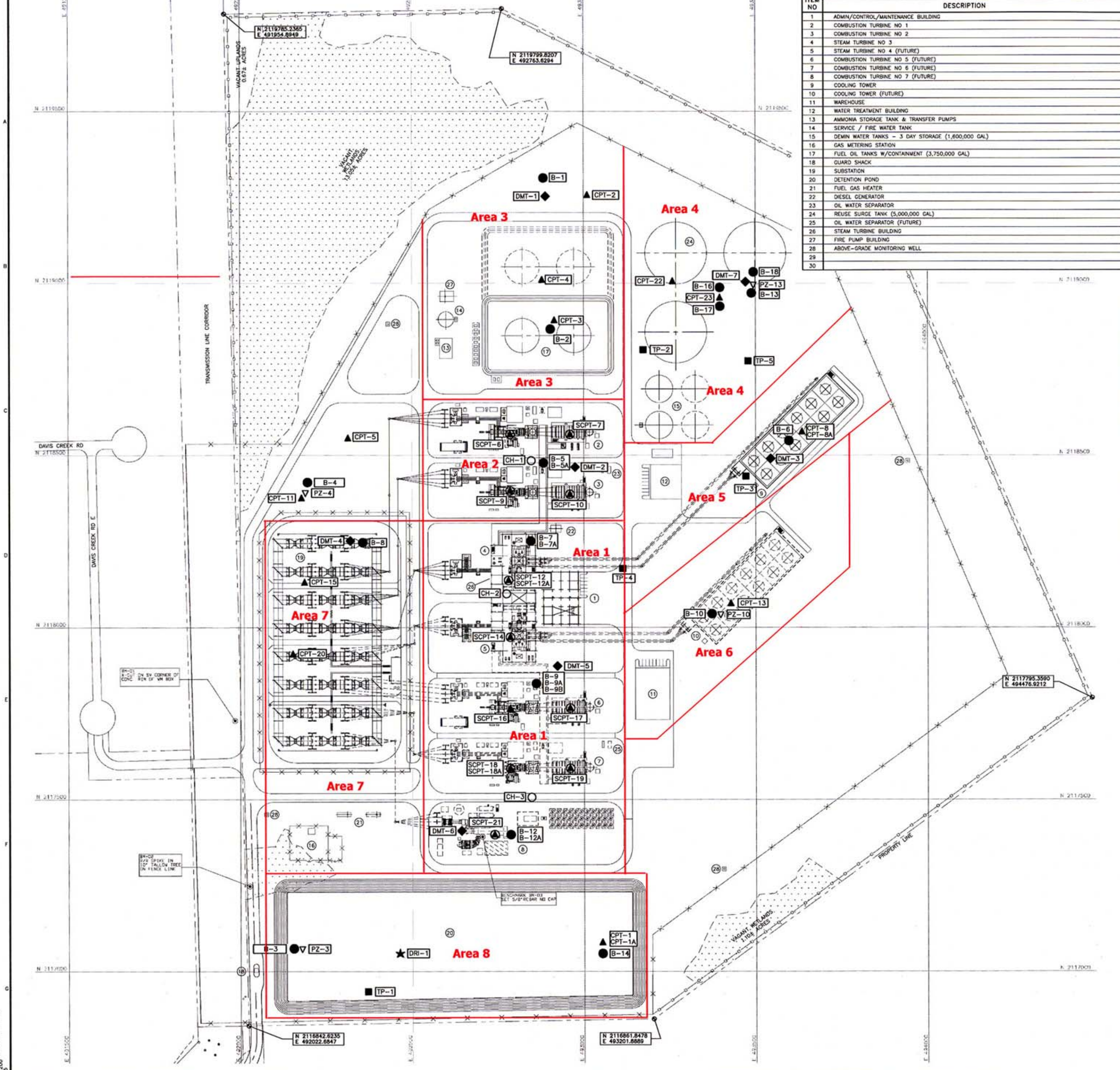


FACILITIES LEGEND

ITEM NO	DESCRIPTION
1	ADMIN/CONTROL/MAINTENANCE BUILDING
2	COMBUSTION TURBINE NO 1
3	COMBUSTION TURBINE NO 2
4	STEAM TURBINE NO 3
5	STEAM TURBINE NO 4 (FUTURE)
6	COMBUSTION TURBINE NO 5 (FUTURE)
7	COMBUSTION TURBINE NO 6 (FUTURE)
8	COMBUSTION TURBINE NO 7 (FUTURE)
9	COOLING TOWER
10	COOLING TOWER (FUTURE)
11	WAREHOUSE
12	WATER TREATMENT BUILDING
13	AMMONIA STORAGE TANK & TRANSFER PUMPS
14	SERVICE / FIRE WATER TANK
15	DEMIN WATER TANKS – 3 DAY STORAGE (1,600,000 GAL)
16	GAS METERING STATION
17	FUEL OIL TANKS W/CONTAINMENT (3,750,000 GAL)
18	GUARD SHACK
19	SUBSTATION
20	DETENTION POND
21	FUEL GAS HEATER
22	DIESEL GENERATOR
23	OIL WATER SEPARATOR
24	REUSE SURGE TANK (5,000,000 GAL)
25	OIL WATER SEPARATOR (FUTURE)
26	STEAM TURBINE BUILDING
27	FIRE PUMP BUILDING
28	ABOVE-GRADE MONITORING WELL
29	
30	

PROJECT SURVEY CONTROL

CONTROL MONUMENT LOCATIONS

MONUMENT NO.	PLANT COORDINATES		MONUMENT NO.	STATE PLANE COORDINATES		ELEVATION
	NORTHING	EASTING		NORTHING	EASTING	
BM-01	–	–	BM-01	2117750.54	491972.45	20.98
BM-02	–	–	BM-02	2117269.19	492013.21	20.90
BM-03	–	–	BM-03	2117410.43	492691.36	35.08

NOTES:

1. HORIZONTAL CONTROL IS BASED ON THE FLORIDA STATE PLANE COORDINATE SYSTEM, EAST ZONE, NAD83. VERTICAL CONTROL IS BASED ON THE NAVD88 DATUM.
2. VERTICAL CONTROL IS BASED ON NATIONAL GEODETIC VERTICAL DATUM (NGVD) OF 1929 WITH THE ORIGINATING BENCHMARK BEING A BRASS JEA DISK STAMPED "GPS CONTROL POINT – STATION 179" SET IN CONCRETE WINGED END WALL, WESTERLY SIDE OF JEA POWER LINE EASEMENT – ELEVATION BEING 26.83 FEET.
3. BENCHMARK COORDINATES ARE BASED ON ASSUMED VALUES. CONTRACTOR TO VERIFY BENCHMARKS PRIOR TO WORK.

INVESTIGATION POINTS

INVESTIGATION	COORDINATES NORTH (FEET)	COORDINATES EAST (FEET)	GROUND SURFACE ELEVATION (FEET)	FINAL DEPTH (FEET)	REMARKS
B-1	2119306.2	492883.5	29.5	99.5	
B-2	2118866.9	492903.7	27.6	100	
B-3	2117066.7	492154.3	24.1	75	
PZ-3	2117066.7	492158.3	27.1	25	PIEZOMETER
B-4	2118407.9	492195.6	25.2	50	
PZ-4	2118391.4	492189.4	27.8	24	PIEZOMETER, SLUG TEST
B-5	2118477	492883	32.6	125	
B-5A	2118481	492887	33.7	108	
B-6	2118543	493597	32.7	75	
B-7	2118249	492846	38.1	125	
B-7A	2118256	492846	37.8	80	
B-8	2118247	492356	29.1	50	
B-9	2117846	492856	32.3	87	
B-9A	2117846	492864	33.1	125	
B-9B	2117838	492864	32.9	76	
B-10	2118040	493373	32.1	75	
PZ-10	2118039.5	493382.9	34.9	25	PIEZOMETER, SLUG TEST
B-12	2117403	492787	33.8	125	
B-12A	2117393	492787	34.3	125	
B-13	2118978.3	493488.8	36	119.3	
PZ-13	2118988.3	493493.8	39	25	PIEZOMETER
B-14	2117066.3	493053.5	25.2	50	
B-16	2118988.3	493396.5	35.5	50	
B-17	2118933.2	493397	35.4	50	
B-18	2119033.5	493494.3	36.1	50	
CH-1	2118478	492849			
CH-2	2118093	492777			
CH-3	2117504	492849			
CPT-1	2117066	493053		9.7	
CPT-1A	2117064	493053		9.2	
CPT-2	2119256	493010.6	30.6	66.1	
CPT-3	2118891	492915		61.2	
CPT-4	2119009.1	492878.9	28.6	86.1	
CPT-5	2118549.8	492313.3	26.4	85.9	
SCPT-6	2118561	492786	29.9	120.3	SHEAR WAVE EVERY OTHER ROD
SCPT-7	2118561	492960	33.2	120.9	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
CPT-8	2118569	493635	33.0	17.2	
CPT-8A	2118567	493635	33.0	78.4	
SCPT-9	2118387	492786	33.3	101	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-10	2118387	492960	33.6	116	SHEAR WAVE EVERY OTHER ROD
CPT-11	2118374.7	492178	24.7	75.1	
SCPT-12	2118137	492781	33.7	70.3	SHEAR WAVE EVERY OTHER ROD
SCPT-12A	2118135	492781	33.7	105.4	SHEAR WAVE EVERY OTHER ROD
CPT-13	2118069	493428	31.6	120.2	
SCPT-14	2117972	492786	31.8	64.6	SHEAR WAVE EVERY OTHER ROD
CPT-15	2118130.4	492186.5	27.4	120	
SCPT-16	2117765	492786	30.4	77.2	
SCPT-17	2117766	492960	32.6	125.8	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-18	2117590	492786	33.2	65.9	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-18A	2117588	492786	33.2	126.1	SHEAR WAVE EVERY OTHER ROD, SEE NOTES
SCPT-19	2117591	492960	32.9	120.4	SHEAR WAVE EVERY OTHER ROD
CPT-20	2117921.5	492152.8	26.6	61.7	SHEAR WAVE EVERY OTHER ROD
SCPT-21	2117398	492740	32.8	110.6	SHEAR WAVE EVERY OTHER ROD
CPT-22	2119005.3	493258.9	32.6	116.9	SHEAR WAVE EVERY OTHER ROD
CPT-23	2118950	493401	35.8	120.2	
DMT-1	2119253	492889.5	29.8	50	MAY REQUIRE AUGERING TO MARL
DMT-2	2118466	492973	34.6	45	MAY REQUIRE AUGERING TO MARL
DMT-3	2118491	493544	32.4	9	
DMT-4	2118251.8	492319.9	28.5	35	
DMT-5	2117888	492923	33.4	25	MAY REQUIRE AUGERING TO MARL
DMT-6	2117409	492644	34.6	10	MAY REQUIRE AUGERING TO MARL
DMT-7	2119005.1	493471.7	36	45	MAY REQUIRE AUGERING TO MARL
DRI-1	2117051.6	492463.5	25.7		TEST ELEVATION 3 FT ABOVE GROUNDWATER TABLE
TP-1	2116942	492370	23.7	8	
TP-2	2118807	493174	33	9	
TP-3	2118440	493472	34.1	8	
TP-4	2118172	493113	37.4	8	
TP-5	2118774.4	493479	35.4	8	
DSR-1	2118621	493010		N/A	SEE DRAWING 160167-DS-0002
DSR-2	2117771	492630		N/A	SEE DRAWING 160167-DS-0002
TSR-1	2119036	492572		N/A	SEE DRAWING 160167-DS-0002
TSR-2	2119060	493198		N/A	SEE DRAWING 160167-DS-0002
TSR-3	2118672	492288		N/A	SEE DRAWING 160167-DS-0002
TSR-4	2118766	493620		N/A	SEE DRAWING 160167-DS-0002
TSR-5	2118540	492661		N/A	SEE DRAWING 160167-DS-0002
TSR-6	2118521	493291		N/A	SEE DRAWING 160167-DS-0002
TSR-7	2118220	492455		N/A	SEE DRAWING 160167-DS-0002
TSR-8	2118244	493053		N/A	SEE DRAWING 160167-DS-0002
TSR-9	2118243	493674		N/A	SEE DRAWING 160167-DS-0002
TSR-10	2117849	492279		N/A	SEE DRAWING 160167-DS-0002
TSR-11	2117968	492881		N/A	SEE DRAWING 160167-DS-0002
TSR-12	2117744	493251		N/A	SEE DRAWING 160167-DS-0002
TSR-13	2117378	492280		N/A	SEE DRAWING 160167-DS-0002
TSR-14	2117369	493014		N/A	SEE DRAWING 160167-DS-0002

LEGEND

▲ CPT (CONE PENETRATION TEST) ◆ DILATOMETER TEST ○ CH (SEISMIC CROSS-HOLE TEST)

SEISMIC CPT ★ DOUBLE RING INFILTROMETER ▽ PIEZOMETERS

● SPT BORING ■ TEST PIT

NOTES:

1. ADDITIONAL TEST PITS MAY BE ADDED.
2. FOR DMT-1, 2, 5 & 7 PUSH WITH CONE RIG UNTIL REFUSAL, THEN AUGER THROUGH DENSE CEMENTED SANDS AND CONTINUE TO A DEPTH OF 120 FT, INCLUDED 90' DRILLING FOR EACH.
3. START CPT'S CONCURRENT WITH SPT. FIRST SPT SHOULD BE B-5.
4. SPT SAMPLING – 4 SPOONS IN TOP 10 FEET, EVERY 5' THEREAFTER UNLESS DIRECTED OTHERWISE.
5. AT SCPT-7, 9, 17 & 18 AN OFFSET LOCATION IS TO BE PREDRILLED TO DEPTH SPECIFIED BY B&V FIELD REPRESENTATIVE AND BACKFILLED WITH APPROVED GRANULAR MATERIAL TO OBTAIN SHEAR WAVE VELOCITY MEASUREMENTS BELOW REFUSAL ZONE.
6. CROSS-HOLE TESTING WILL BE PERFORMED IF SCPT RESULTS REFERENCED IN NOTE 5 ARE UNSATISFACTORY.
7. DMT INTERVALS – 5, 10, 15, 20, 30, ETC. TO REF: FRO DEEP THEN 10' INTERVALS INTO MARL AFTER AUGERING THROUGH DENSE ZONE.

NOT TO BE USED FOR CONSTRUCTION

NO	DATE	REVISIONS AND RECORD OF ISSUE	DRN	DES	CHK	PDE	APP
1	01/MAY/08	ISSUED FOR ADDENDUM 1	DS	DS		TD	DG
0	26/MAR/08	ISSUED FOR INFORMATION ONLY	DS	DS		TD	DG

1"=100'

I HEREBY CERTIFY THAT THIS DOCUMENT WAS PREPARED BY ME OR UNDER MY DIRECT SUPERVISION AND THAT I AM A DULY REGISTERED PROFESSIONAL ENGINEER UNDER THE LAWS OF THE STATE OF FLORIDA.

SIGNED
DATE REG NO.

BLACK & VEATCH

DRAWN DAS

JEA
GREENLAND ENERGY CENTER

PLANT ARRANGEMENT
SUBSURFACE INVESTIGATION PLAN

PROJECT | DRAWING NUMBER | REV 1

CERTIFICATE OF AUTHORIZATION NO. 8132

FIGURE 5-1

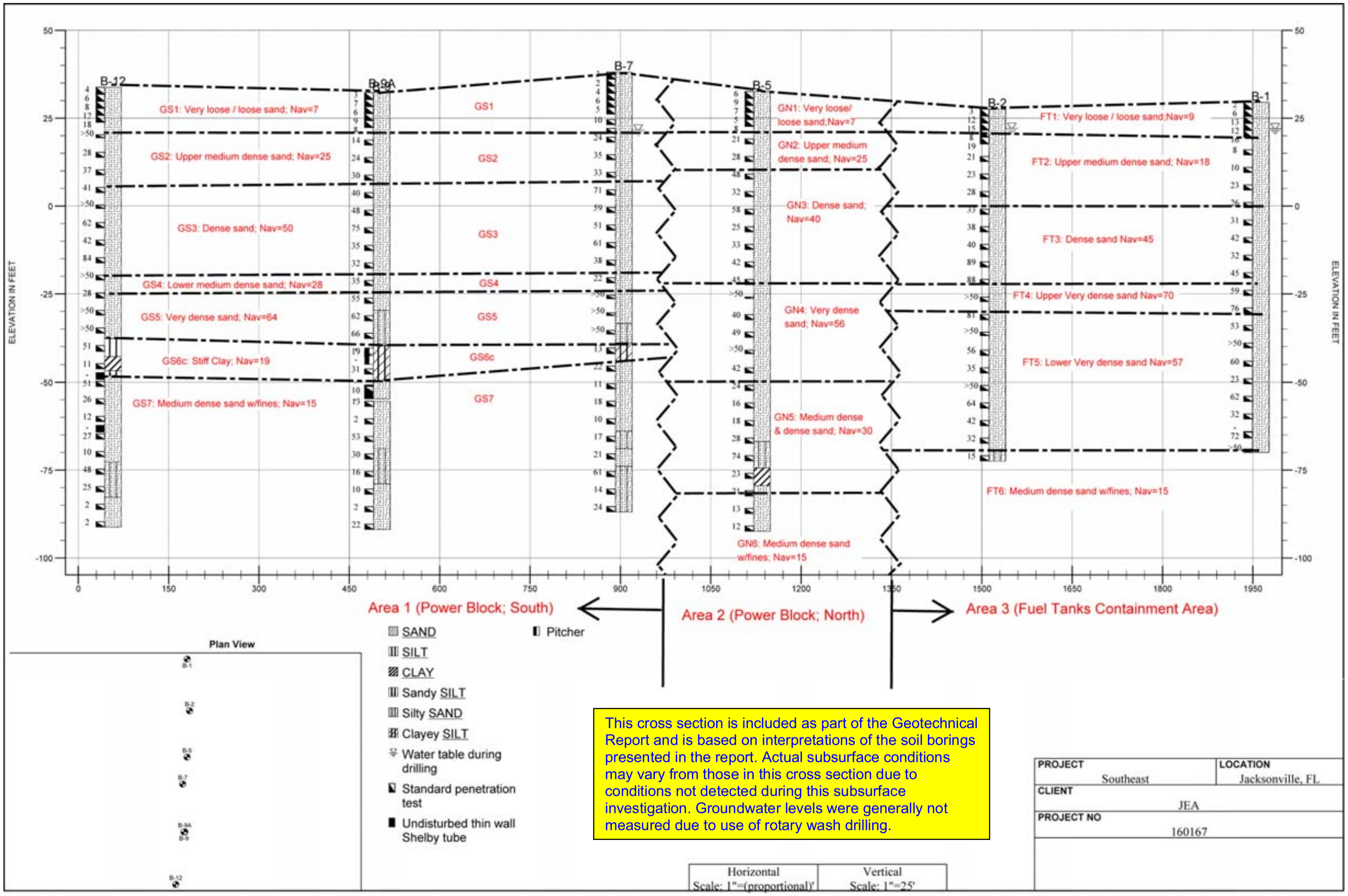


Figure 5-2
Plant Design Cross Section South

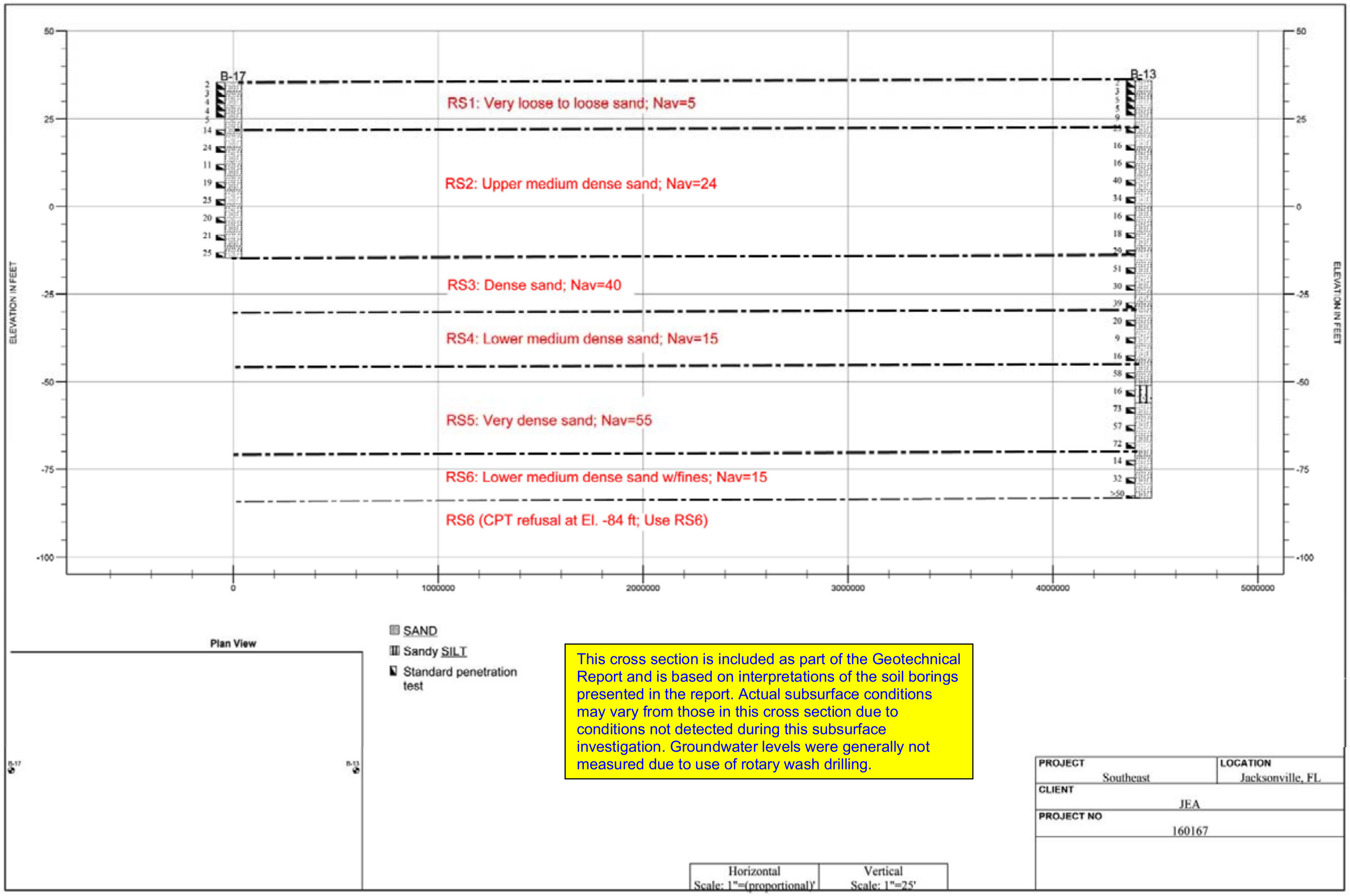


Figure 5-3
Area 4 Design Cross Section

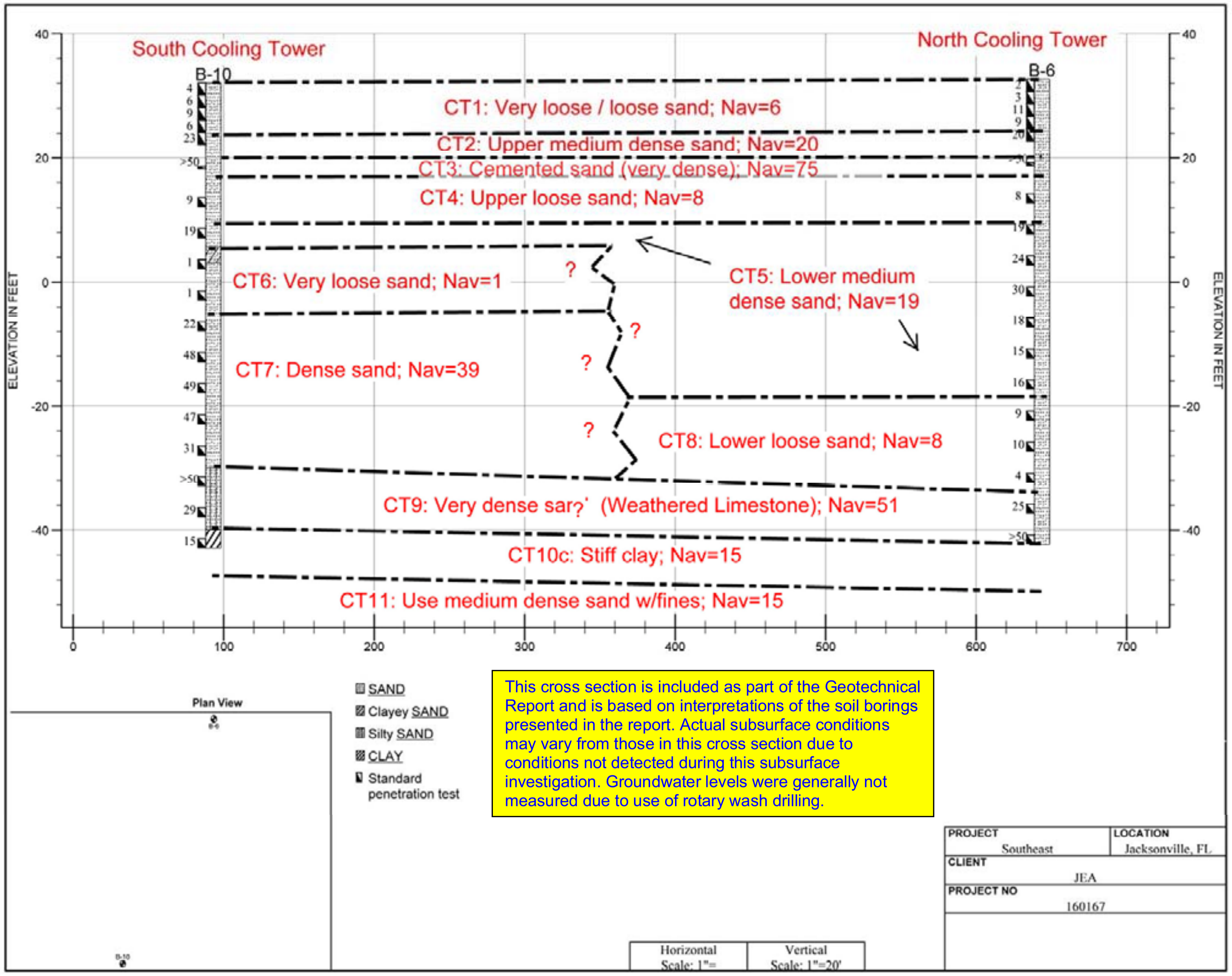


Figure 5-4
Areas 5 and 6 Area 4 Design Cross Section

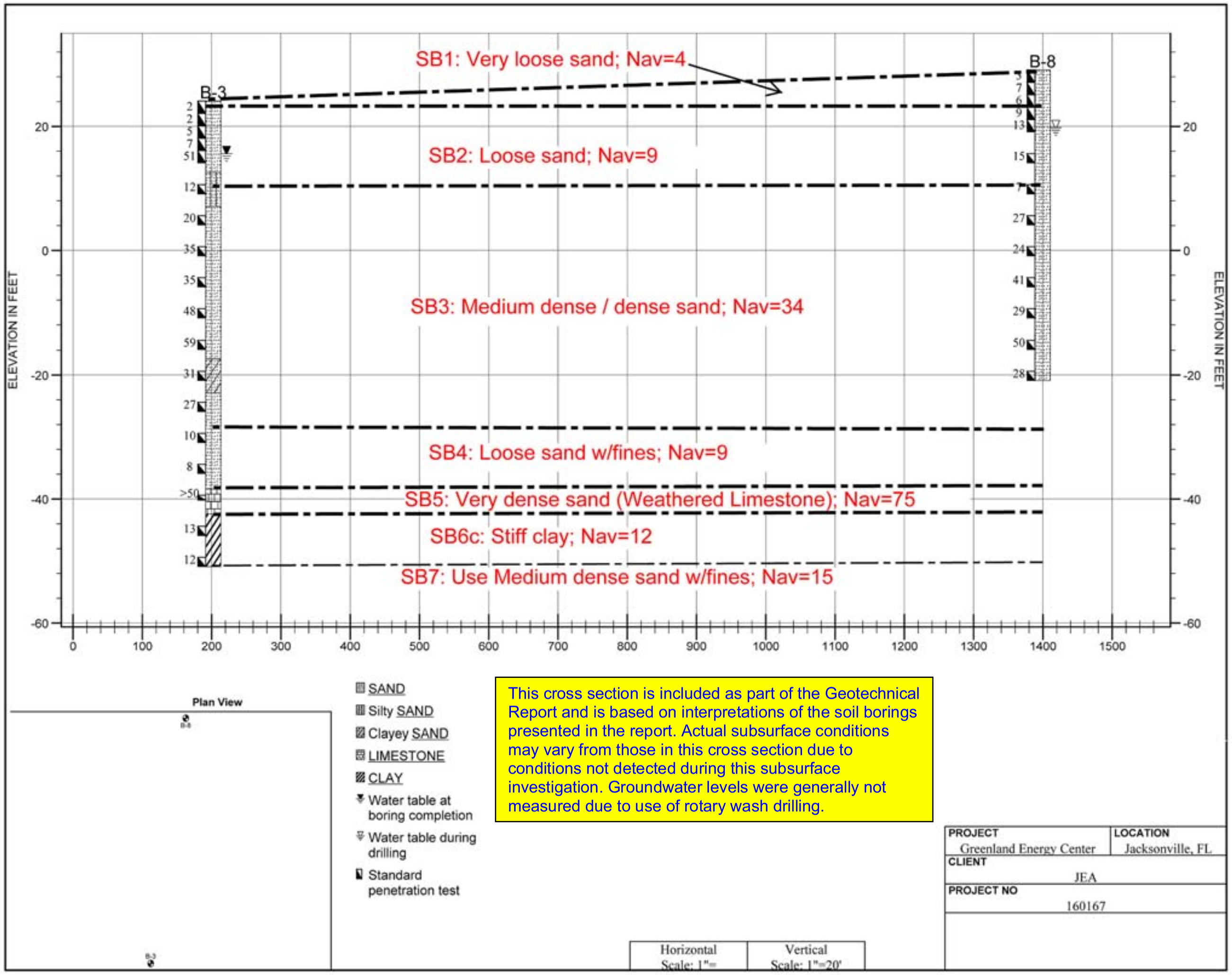


Figure 5-5
Area 7 Design Cross Section

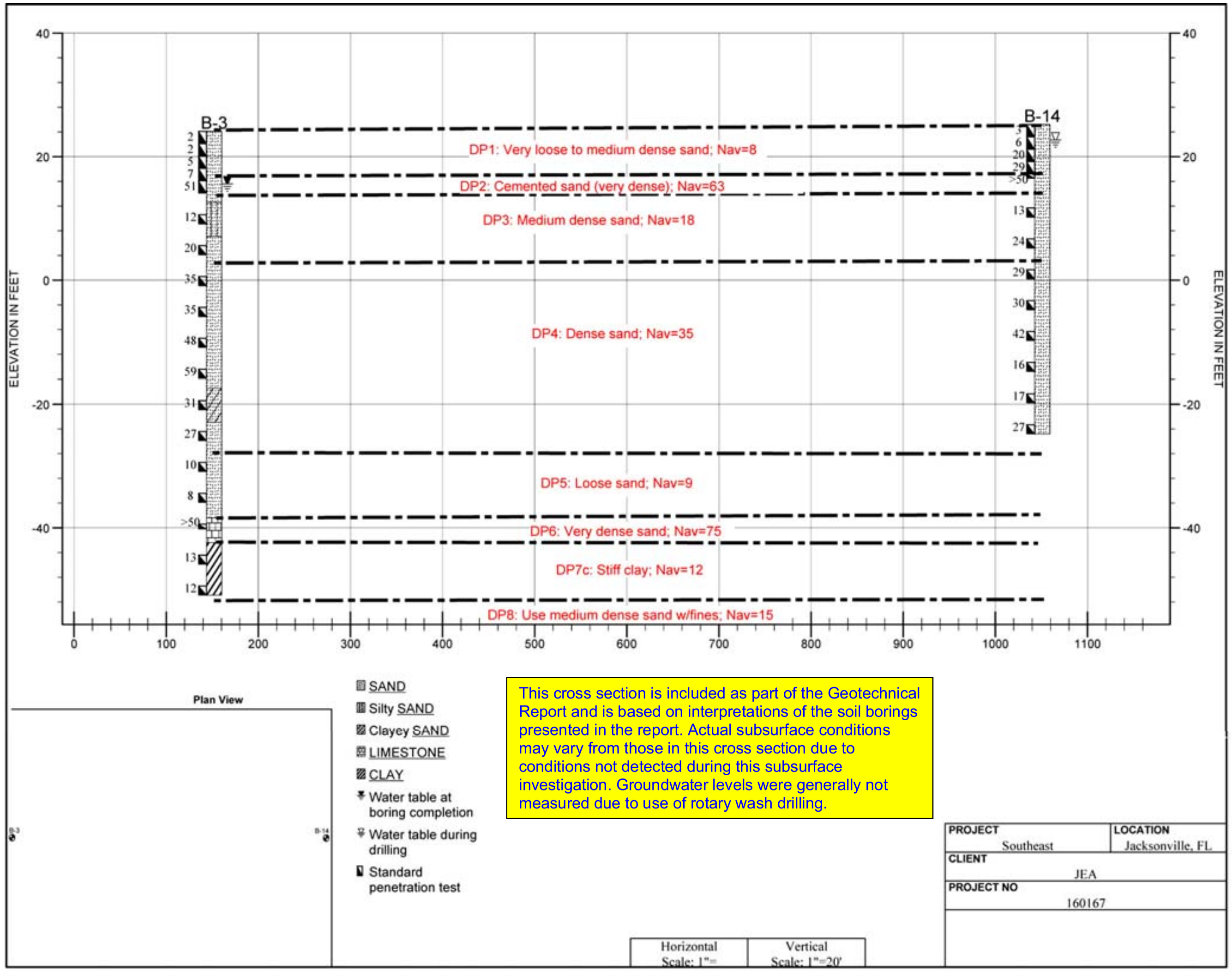
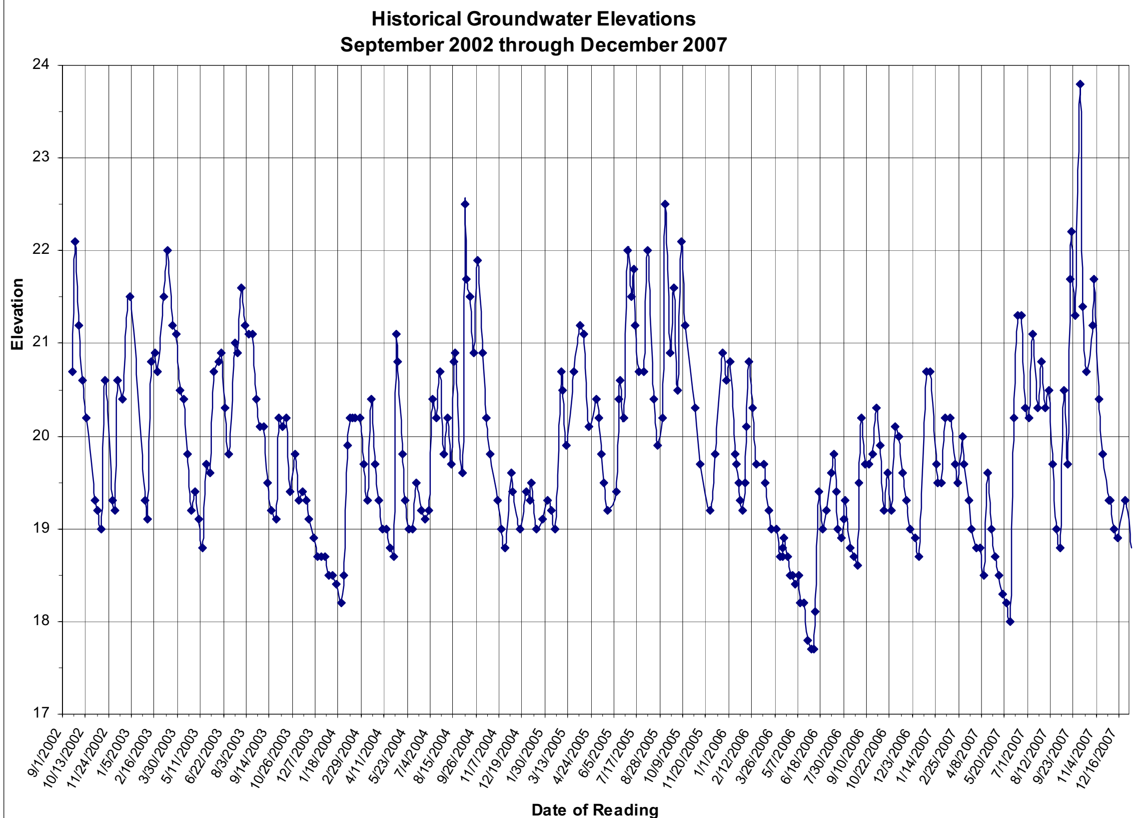


Figure 5-6
Area 8 Design Cross Section



Readings from Ellis & associates office, 7064 Davis Creek Rd, Jacksonville, FL 32256, less than 1 mile from project site. Provided via email from Jason McKinnon, 9Jan08.

Figure 5-7
Historical Groundwater Elevations

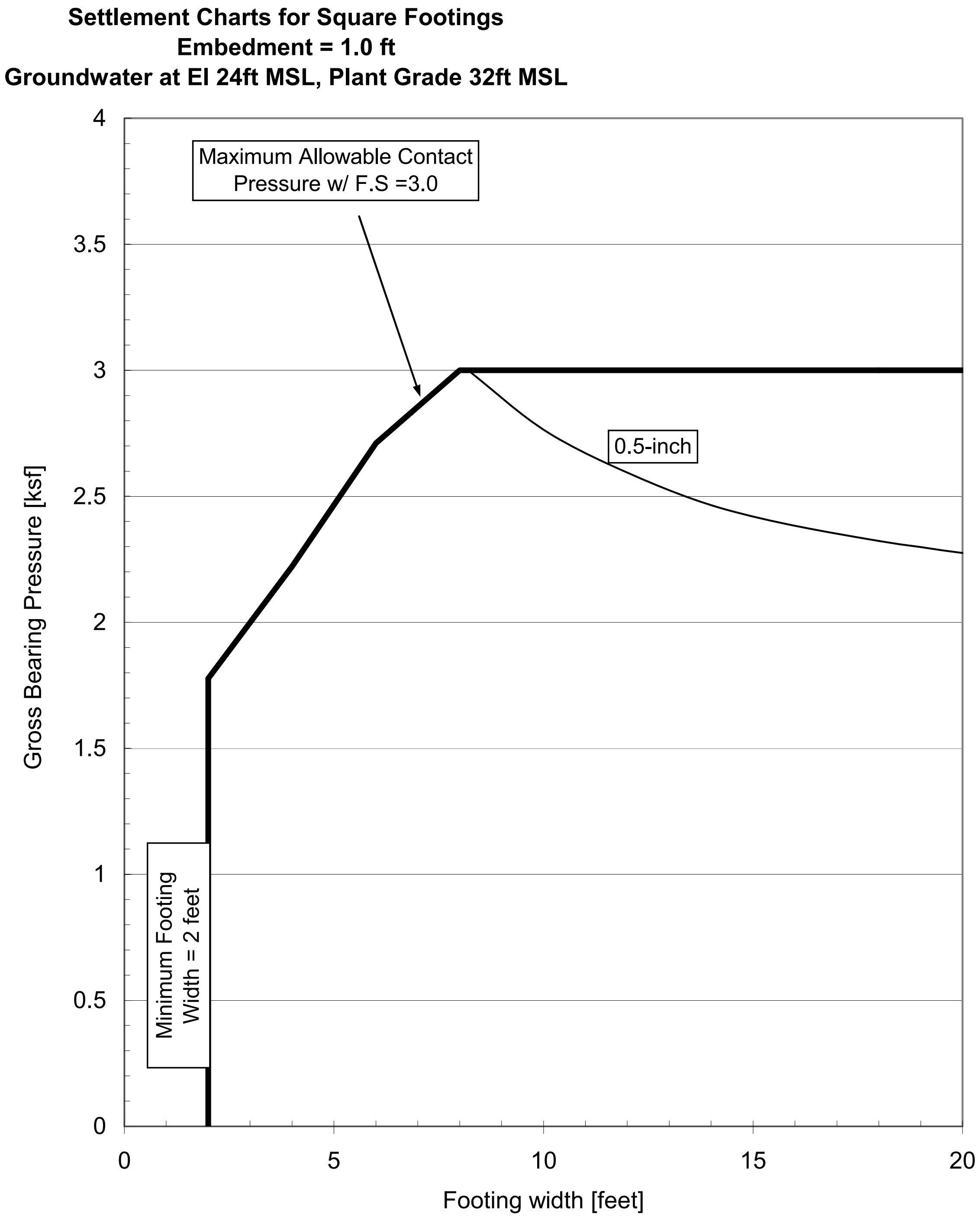


Figure 7-1
Settlement Chart for Square Footings

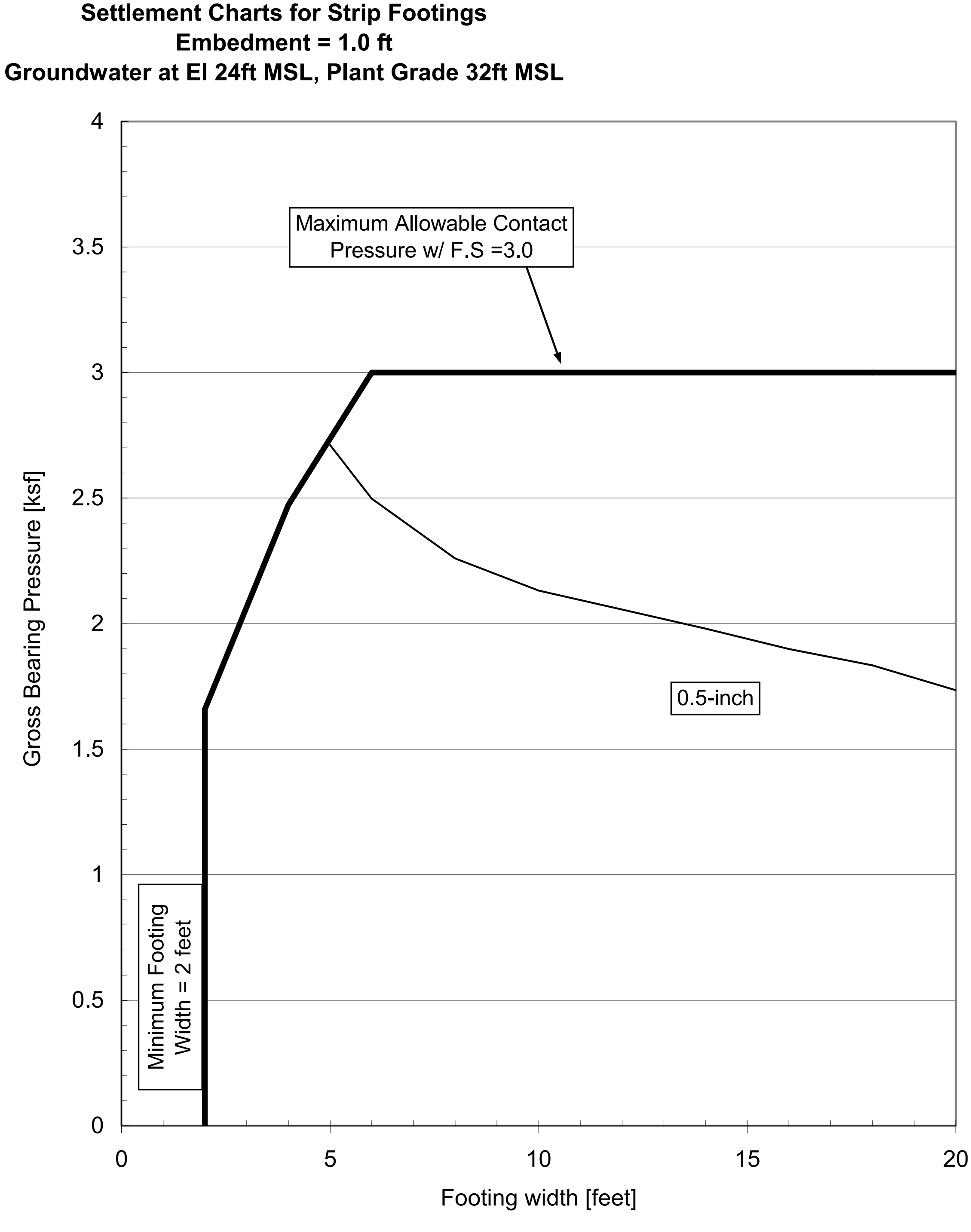


Figure 7-2
Settlement Chart for Strip Footings, 1.0 Foot Embedment

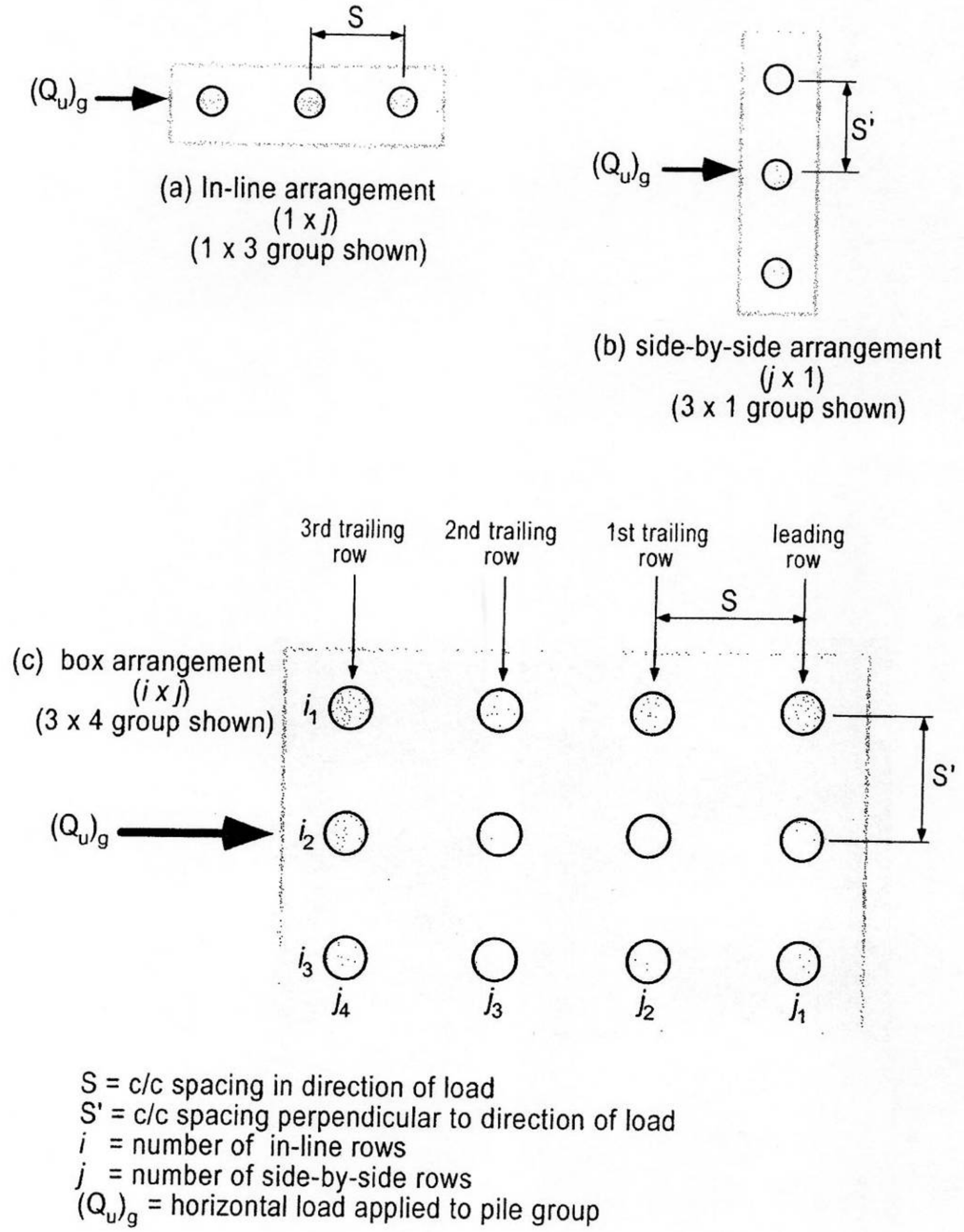
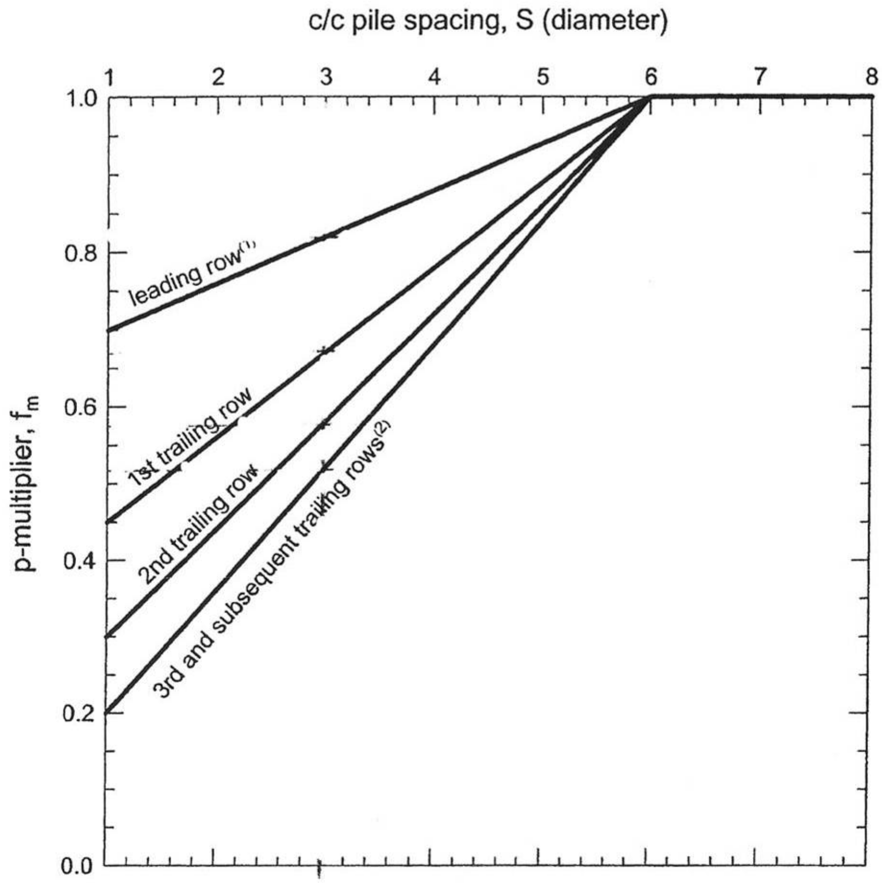


Figure 7-3
Group Modification Factors for Lateral Pile Analysis



Notes:

(1) The term row used in this chart refers to a line of piles oriented perpendicular to the direction of applied load.

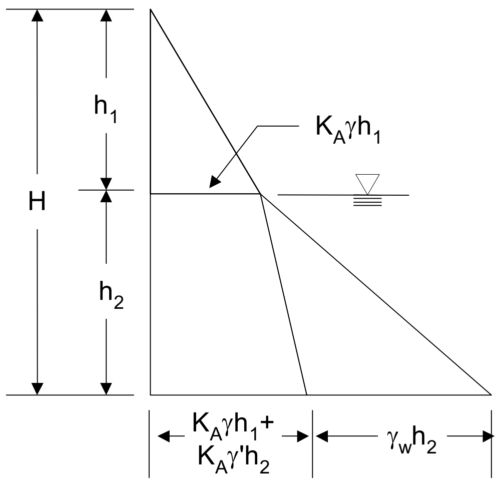
(2) Use the f_m values recommended for the 3rd trailng row for all rows beyond the third trailing row.

(3) Bending moments and shear forces computed for the corner piles should be adjusted as follows:

side by side spacing	corner pile factor
3D	1.0
2D	1.2
1D	1.6

Figure 7-4
Group Modification Factors for Lateral Pile Analysis

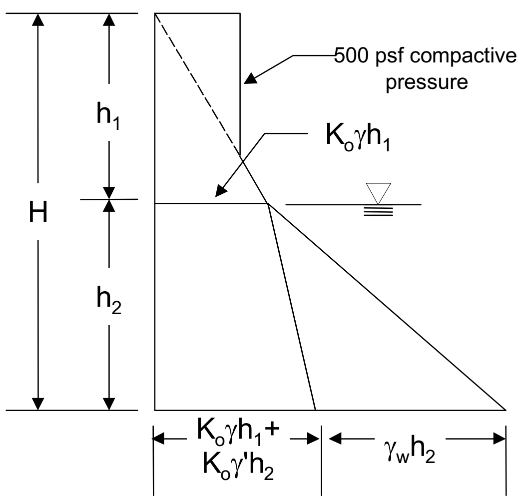
YIELDING WALLS



NOTES:

1. Surcharge loads (psf) add to static effect as rectangular pressure diagram of K_oS for non-yielding walls of K_AS for yielding walls.
2. Water table varies across site. For design, assume ground water table at existing ground surface.
3. The subsurface profile varies across site.

NON-YIELDING WALLS



LEGEND:

H	total depth of wall
h	depth below grade
$\mathbf{h_1}$	depth to water table
$\mathbf{h_2}$	depth below water table to depth of wall
γ	moist unit weight
γ`	buoyant unit weight
γ_S	saturated unit weight
γ_W	unit weight of water (62.4 pcf)
$\mathbf{K_A}$	coefficient of active earth pressure
$\mathbf{K_P}$	coefficient of passive earth pressure
$\mathbf{K_o}$	coefficient of at-rest earth pressure
S	surcharge pressure

Material	γ (pcf)	γ` (pcf)	K_A	K_P	Ko
Sand	115	52.6	0.33	3.0	0.5

$K_A = \tan^2(45-\phi/2)$

$K_P = \tan^2(45+\phi/2)$

Figure 7-5
Lateral Earth Pressure Diagram