



SOLID WASTE AUTHORITY OF
PALM BEACH COUNTY, FLORIDA

PALM BEACH RENEWABLE ENERGY FACILITY No. 2
PALM BEACH COUNTY, FL
GEOTECHNICAL ENGINEERING REPORT
MAY 2010



Prepared for:

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May 14, 2010
Project No. WPB-09-7594

Attention: Ms. Leah Richter, P.E.

Subject: ***Geotechnical Engineering Report***
Palm Beach Renewable Energy Facility No. 2
Palm Beach County, Florida

Dear Ms. Richter:

Dunkelberger Engineering & Testing, Inc. (Dunkelberger) has completed a geotechnical engineering study for the proposed Renewable Energy Facility No. 2 in Palm Beach County, Florida. The following report presents factual data obtained from field explorations and laboratory testing, describes the methods of study, a summary of the assumptions and methodology utilized for the analysis, and provides design criteria for the geotechnical engineering aspects of the project. These design criteria are presented with the assumption that earthwork mass grading of the site, inclusive of reclamation of portions of the existing lake, has been accomplished by the Solid Waste Authority prior to the design-build-operate contractor's mobilization to the site.

We appreciate the opportunity to provide geotechnical engineering services for this very challenging project. Should you have any questions regarding the contents of this report, please call.

Very truly yours,

DUNKELBERGER ENGINEERING & TESTING, INC.

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cc: Addressee (5)...*via hand delivery*

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INTRODUCTION

Pursuant to your written authorization (Task Order No. 2, dated March 5, 2010), Dunkelberger Engineering & Testing, Inc. (Dunkelberger) has completed a geotechnical engineering study in connection with the proposed Palm Beach Renewable Energy Facility No. 2 that is to be constructed at the North County Resource Recovery Facility in Palm Beach County, Florida. This report includes the methods of study, key findings from the work and includes design criteria for foundations and related matters in connection with the construction of the proposed facility. Design criteria provided herein are based upon subsurface exploration completed as part of this assignment. The subsurface exploration plan described herein has been presented to and approved by the vendors who are bidding this project as a design-build-operate contract.

PROJECT CONSIDERATIONS

The project is to include the design, construction and operation of a new renewable energy facility, located immediately north of the existing waste-to-energy facility and west of Jog Road. The project setting is provided on Sheet 1. Structural loading information has not been made available to us at the time of this writing. We have assumed that structure loadings will be relatively heavy, at least for most of the structures associated with this project. Access roadways, truck ramps and other paved areas will be provided for vehicular traffic control. Retaining walls will be used to provide grade separation between various structures and truck ramps.

The existing landform will be raised several feet above its present condition in order to reach proposed subgrade levels for the construction. The site currently slopes down from south to north, with elevations ranging from approximately +40 feet with respect to the National Geodetic Vertical Datum of 1929 (NGVD) adjacent to the existing facility to approximately +18 feet NGVD in some of the low areas of the site. Finished floor elevations are to range between +19 and +54 feet NGVD. Some of the building areas overlie an existing lake that will be reclaimed by filling. The project site will be filled to subgrade levels (including lake backfilling) by Palm Beach County Solid Waste Authority (Authority) forces under separate contract. Subsurface conditions within the areas of the proposed facilities that fall within the footprint of the existing lake will be explored subsequent to lake backfilling.

The table on the following page provides a brief description of the major project structures that are planned.

Structure Name	Plan Dimensions (feet)	Finished Floor Elevation (feet – NGVD)	Roof Elevation (feet – NGVD)
Stack	30 x 40	Assumed 24	Flue 334
APC Building – FFH Area	240 x 130	24	100
APC Building – SDA Area	280 x 50	24	146
Turbine Generator	140 x 60	24	93
Boiler Building	350 x 120	24	178
Refuse Storage Pit and Building	420 x 125	19	132
Tipping Floor and Building	420 x 150	54	105
Air Cooled Condenser	400 x 90	Assumed 24	84
Ash Management Building	300 x 150	24	76
Water Treatment Building with Storage Tanks	110 x 80 Tank Diameters = 10 to 20	Assumed 24	--
Dirty Wastewater Tank	Diameter = 50 ft	Assumed 24	--
Switch Gear Room	140 x 30	Assumed 24	--
Warehouse Storage/Facility Maintenance Offices*	380 x 80	24	46
0.5 MG Fire Water Storage Tank*	Diameter = 50 ft	Assumed 24	--

* Structures that will be located within the area of the existing lake. These structures will be constructed subsequent to lake backfilling.

Of the structures planned, the southernmost will be the Tipping Floor and Building which will also have the highest finished floor elevation of the planned structures (i.e. +54 ft NGVD). Between about 24 and 32 feet of fill will be required to bring the Tipping Floor area to the floor slab bearing level. This structure will be located about 70 feet north of the existing Waste to Energy Structure. New truck ramps will be constructed to provide access to the southeast and southwest corners of the Tipping Floor and Building. Retaining walls will be constructed to provide grade separation along the truck ramps and around the perimeter of the Tipping Floor and Building. Sloped embankments are also planned along the southern side of the proposed truck ramps.

SITE CONDITIONS

The site of the proposed construction is currently vacant, with the exception of an existing asphalt paved driveway that allows truck traffic to enter the existing facility to the south and to circulate through the site. The existing ground surface slopes down and away from the existing building in a somewhat terraced manner to a gravel covered roadway currently used for trailer storage. North of the storage area is an approximately 600 foot long by 100 foot wide stormwater retention basin. This area is open but was wet at the time of the field explorations for this project. An east-west trending berm separates the stormwater retention basin from an approximately 5 acre wooded area occupying the north-central portion of the site. This wooded area was also wet at the time of our field study and appears to be utilized for stormwater retention. Roadway areas are built up approximately 4 to 5 feet above the stormwater retention areas. A relatively large lake encroaches on the northern limits of the project site.

Photographs of the Palm Beach Renewable Energy Facility follow. Additional photographs of the site are provided in Appendix A.



Eastern roadway viewing southwest at staked location of Boring TB-24.



Northern Roadway, looking west at location of Boring TB-1

PHYSIOGRAPHIC SETTING

Based upon review of the U. S. Geological Survey Delta, FL quadrangle map, dated 1945 and photo-revised 1983, the project site is within Section 34, Township 42S, Range 42E. The map indicates the site is located on the eastern fringe of the Loxahatchee Slough, and the natural ground surface is approximately +18 feet NGVD. Research of the Soil Survey of Palm Beach County Area, FL, published by the U. S. Department of Agriculture, Soil Conservation Service (SCS) indicates the site in its natural condition was mantled by several soil associations including Riviera sand, Riviera sand-depressional, Pinellas find sand, Oldsmar sand, Anclote fine sand and others. These are described by the SCS as nearly level, poorly drained sands that are found in low areas and poorly defined drainage ways. The Riviera sands have a loamy (i.e. clayey) subsoil at relatively shallow depths (within approximately 20 to 40 inches of the natural ground surface. Prior to the implementation of local drainage improvements (i.e. canal excavations and/or pumping), the water table ranged between slightly below the natural ground surface (dry season) to above it (wet season).

According to Dr. H. K. Brooks's publication "Guide to the Physiographic Divisions of Florida" 1981, the site lies near the southern limits of the Eastern Flatwoods District (Green Ridge-Loxahatchee Karst). The Green Ridge-Loxahatchee Karst is described as poorly drained open flatwoods with cypress strands and heads. In his work entitled "Geologic Map of Florida," Dr. Brooks maps the site within the Caloosahatchee Formation (Qc), which is described as a calcareous shelly sand with diverse extinct tropical marine fauna that may contain multiple very hard sandy cap rocks.

The U. S. Geological Survey "Altitude of Water Table Aquifer for Palm Beach County, Florida" maps (dated 1984) indicate the water table in the vicinity of the Palm Beach Renewable Energy Facility No. 2 site to be near +16 feet NGVD. The groundwater gradient is generally to the east. The deep artesian aquifer has a piezometric head that is above that of the water table aquifer, resulting in an upward hydraulic gradient between the two aquifers.

The project site is located in northeast Palm Beach County, Florida. This area is not considered to be seismically active. Therefore, the liquefaction potential of the foundation soils at the project site is relatively low. Further, considering the hydrogeology of the site, there is low potential for the development of sinkholes.

SUBSURFACE CONDITIONS

Exploration Methodology

Subsurface conditions were explored using thirty-five (35) Standard Penetration Test (SPT) borings, drilled in accordance with ASTM D 1586 protocol. The SPT borings were accomplished utilizing Central Mine Equipment Models 45 and 55 (CME 45 and 55) drilling rigs employing mud rotary methodology. Four borings were drilled to 150 feet deep, twelve to 100 feet deep, sixteen to 50 feet deep and three to 25 feet deep. The borings were situated at strategic locations based upon the proposed facility. Since some of the areas were not accessible to conventional truck mounted drilling equipment, the Authority provided earth filling (i.e. fill pads) in order to improve access. Photographs of the drilling operations on the fill pads follow.



Boring locations were staked in the field and horizontal coordinates determined by aid of a hand held Magellan Model Explorist 100 GPS unit. The project surveyor determined the as-drilled horizontal and vertical locations of the borings (survey data is provided in Appendix B).

Samples of the subsurface materials were obtained continuously in the upper ten (10) to twenty (20) feet of the borings, and at 5-foot vertical centers thereafter. Groundwater levels were recorded for each SPT boring. The boreholes were sealed with cement-bentonite grout in a bottom-to-top manner when complete.

In addition to the drilling of the SPT borings, eleven auger profile borings were advanced to a depth of ten (10) feet below the ground surface. The auger profile borings were accomplished using continuous flight augers powered by the drilling rigs. Upon completion, the auger profile borings were backfilled with soil cuttings.

Lastly, forty-three soft sediment probes were conducted within the existing wooded and stormwater retention areas. These areas were under about a foot or less of water at the time the probes were performed (April 27, 2010). The probes were accomplished by pushing a hand held 3/8-inch diameter solid steel rod into the surficial soils (natural ground surface) until firm resistance was met. Locations of the soft sediment probes were recorded using the GPS device.

The locations of the SPT borings and auger profile borings are shown graphically on Sheets 1A and 1B. Sheet 1A presents the boring locations on the project topographic survey, while Sheet 1B indicates the boring locations with respect to the project building components. These sheets also show the proposed locations of Borings TB-36 through TB-40, which will be drilled at a later date once the existing lake has been reclaimed. The locations and results of the soft sediment probes are depicted on Sheet 1C.

Stratigraphy

At seven of the boring locations, a flexible pavement section consisting of asphaltic concrete over shell rock base course was present. The table below shows the boring number and asphalt and base course thicknesses measured at each location.

Boring No.	Asphaltic Concrete Thickness (inches)	Base Course Thickness (inches)
TB-1	2.8	12
TB-32	2.8	12
TB-34	2.8	12
TB-35	3.5	12
AB-2	3.6	5.4
AB-6	3.6	6.6
AB-7	1.8	4.8

Subsurface conditions disclosed by the borings generally indicate the project site to be mantled by a moderate thickness granular fill material over natural soils. The fill consists of sand with trace to some sand-sized shell fragments, gravel and silt and extends to an average depth of 7 feet (or elevation +14 feet NGVD) below surface grades at the boring locations. Underlying the fill are naturally occurring clean sands, slightly silty sands, clayey sands and interlayered shelly sands with gravel and clean to slightly silty sands that extend to an average depth of roughly 65 feet (or elevation –45 feet NGVD). A discontinuous formation of coquina limestone enters the subsurface profile at this depth and persists to the maximum depth of exploration at 150 feet (elevation –130 feet NGVD). Slightly silty to silty sands containing varying amounts of sand to gravel sized shell and gravel are present where the limestone is discontinuous. Some of the borings also disclosed relatively thin (i.e. 1 to 11 feet thick) layers of silty sands and silt within the depth interval of 30 to 75 feet below grade.

The following table provides basic subsurface profile components in terms of average elevations, material descriptions and average relative density/consistency/hardness.

Stratum No.	Elevation Interval (feet)	Material Description	Relative Density/Consistency/Hardness
1	+21 to +14	Light brown to brown fine SAND with trace to some sand-sized shell fragments and gravel, some lenses with silt (SP, SP-SM) (FILL)	Loose to Dense
2	+14 to +11	Brown to light brown fine SAND, some lenses with silt (SP, SP-SM)	Loose to Medium Dense
3	+14 to +10	Brown to light brown clayey fine SAND (SC)	Loose to Medium Dense
4	+10 to 0	Light gray slightly silty to silty fine SAND with some sand to gravel-sized shell fragments (SP-SM, SM)	Loose to Medium Dense
5	0 to -16	Brown to light brown fine SAND, trace to some silt (SP, SP-SM)	Very Loose to Medium Dense
6	-16 to -45	Gray to light gray slightly silty to silty fine SAND with some sand to gravel-sized shell fragments and gravel, some cemented sand and shell lenses (SP-SM, SM)	Loose to Medium Dense
7	-8 to -12, -25 to -30, -40 to -45	Brown to greenish-gray silty fine SAND with trace sand-sized shell fragments (SM) and sandy SILT (ML)	Very Loose to Loose
8	-45 to -130	Light brown to light gray cemented sand and shell, sandy zones (COQUINA LIMESTONE)	Weakly cemented to Well Cemented

Note: Elevations reflect average subsurface profile conditions. Variations do exist. Refer to Sheets 2A through 2L for strata boundaries for each boring.

The Standard Penetration Test (SPT) is completed by dropping a 140-pound hammer for a vertical distance of 30 inches and summing the number of hammer blows required to drive a 2-inch diameter split barrel sampler a distance of 12 inches (the SPT after ASTM D 1586). The results of the SPT data were used to estimate the relative density of granular soils and the relative hardness of the limestone formation. The table on the following page provides the range and average SPT N-Values for each of the subsurface components that were measured in the upper 150 feet of the profiles.

Material Type	Stratum No.	SPT N-Value (blows per foot)		
		High	Low	Average
FILL	1	54	1	22
SAND	2	74	4	22
CLAYEY SAND	3	21	7	12
SAND/SHELL	4	75	2	11
SAND	5	20	1	8
SAND/SHELL	6	49	1	14
SILTY SAND	7	36	1	7
LIMESTONE	8	100	4	34

The results of the soft sediment probes indicate that the surficial soils within the stormwater retention and wooded areas consist of relatively clean fine sand with trace to some organic matter that is very loose in its uppermost 7 to 48 inches (typically 7 to 12 inches). The height of water above grade in these areas ranged from 0 to 16 inches.

Detailed subsurface profiles for each of the borings are provided on Sheets 2A through 2L of this report. In addition, a south to north geologic cross section through borings TB-1, TB-4, TB-9, TB-11 and TB-14 is provided on Sheet 3.

Groundwater

Groundwater was measured in the borings at depths that ranged between 1.5 and 13.2 feet below surface grades on the dates that the borings were drilled (March 22 through April 21, 2010). Based upon ground surface elevation data provided by the project surveyor, these depths correspond to elevations ranging from about +16 to +21 feet NGVD. Some of the values recorded are not considered representative of the groundwater level and are attributed to lack of hydraulic equilibration within the borehole at the times the readings were recorded. We estimate that the groundwater elevation across the site during the dates the borings were drilled ranged from about +17 to +19 feet NGVD. The higher values exist within the southern portion of the site and are attributed to natural groundwater mounding owing to higher grades within this area. Also, considering that the site vicinity has experienced more than normal rainfall during March and April, we anticipate that the groundwater levels at the time of drilling were near that of the average seasonal high water level.

LABORATORY TEST RESULTS

Index Properties

Representative samples of the materials recovered from the borings were tested for index properties including moisture content (ASTM D 2216) and grain size distribution (ASTM D 422). The results of these laboratory tests are presented on Sheets C1 through C23 in Appendix C. Average values of the test results are summarized in the following table.

Material Type	Stratum No.	Moisture Content (%)	Amount Passing Sieve Size (%)						
			¾"	#4	#10	#40	#60	#100	#200
FILL	1	9.9	100.0	97.1	95.2	83.5	53.7	28.8	6.1
SAND	2	18.5	100.0	100.0	100.0	94.3	71.3	44.6	5.1
CLAYEY SAND	3	39.9	100.0	100.0	100.0	94.8	76.2	52.6	14.8
SAND/SHELL	4	17.8	100.0	86.5	74.7	59.3	42.0	26.9	9.0
SAND	5	22.2	100.0	100.0	99.9	91.9	69.3	40.6	4.2
SANDSHELL	6	21.5	100.0	89.8	82.2	75.1	66.0	41.6	7.8
SILTY SAND/SILT	7	31.5	100.0	100.0	99.9	96.7	82.1	58.4	33.8

Limerock Bearing Ratio

Four (4) samples of the surficial fill materials were obtained and subjected to Limerock Bearing Ratio (LBR) testing and grain size distribution. The samples were taken from a depth of 0 to 6 inches below the existing grades at the locations of Borings TB-2, TB-4, 20 feet south of TB-1 and 50 feet east of TB-23. Some of the samples were taken from drill pads that were placed for access purposes. These materials were considered to be representative of those that will be used to fill the site to proposed subgrade levels. The results of the testing are provided on Sheets C24 through C31 in Appendix C. A summary of the test results are provided in the table on the following page.

Sample Description	Sample Location	Optimum Moisture Content (%)	Dry Density (pcf)	Maximum LBR Value
Light brown fine to medium SAND with some fragmental limestone (SP)	TB-4	11.5	109.9	37
Light brown fine to medium SAND with some fragmental limestone (SP)	20 ft South of TB-1	11.7	108.5	35
Dark brown fine to medium SAND with some fragmental limestone (SP)	TB-2	12.2	109.8	39
Light brown fine to medium SAND with some shell fragments (SP)	50 ft East of TB-23	14.4	107.1	34

Reference to the test results indicate that the LBR values of the fill materials sampled range from about 34 to 39 and their maximum dry densities range from 107 to 110 pounds per cubic foot (pcf).

Corrosion Testing

Three (3) soil samples collected from Borings TB-1, TB-18 and TB-23 in the depth interval of 2 to 6 feet below the ground surface and an additional sample collected from a “fill pad” in the depth interval of 0 to 2 feet at the location of Boring TB-4 were tested for the corrosion parameters of pH, resistivity, sulfate content and chloride content in order to provide an environmental classification of the subsurface materials. Results of the tests are shown in the following table.

Parameter / FDOT Classification	TB-1	TB-4	TB-18	TB-23
pH	7.9	7.7	7.7	8.1
Resistivity (ohm-centimeter)	6,000	5,900	5,100	6,900
Chloride Content (ppm)	57.6	43.2	50.4	50.4
Sulfate Content (ppm)	14.7	19.8	< 1.6	24.9
FDOT Classification	Slightly Aggressive	Slightly Aggressive	Slightly Aggressive	Slightly Aggressive

Comparison of the corrosion series test results with criteria established in Section 1.3 Environmental Classifications of the Florida Department of Transportation (FDOT) Structures Design Guidelines indicates that all of the samples tested are classified as slightly aggressive.

Moisture and Organic Content Testing

The upper 0 to 36 inches of the surficial sands from the locations of soft sediment probes MP-21, MP-26 and MP-37 were sampled and tested for moisture (ASTM D 2216) and organic (ASTM D 2974) contents. The test results indicate that the moisture contents for these soils ranged between approximately 27 and 85 percent while their organic contents ranged between 1 and 8 percent. Complete test results are provided on Sheet C32 of Appendix C.

Consolidation Testing

One dimensional consolidation testing was performed on two undisturbed samples of the silt (Stratum 7). One sample was obtained from Boring TB-22 from a depth of 35 to 36 feet below grade. The second sample was from Boring TB-26 from a depth interval of 32 to 33 feet below grade. A third sample was obtained from Boring TB-26 from the depth interval of 33 to 35 feet below grade but was too granular to be tested for consolidation characteristics. The results of the consolidation testing are provided on Sheets C33 and C34 in Appendix C. Sheet C35 in the Appendix shows the results of moisture content, Atterberg limits, fines content and specific gravity testing for the deeper Boring TB-26 sample.

The consolidation test results indicate that both silt samples are over-consolidated and have experienced a maximum past pressure of about 7 ksf. The over-consolidation ratio for these silts is between about 3.5 and 4 and the compression ratio of the consolidation curve up to the maximum past pressure is less than 0.01 (strain basis).

GEOTECHNICAL OVERVIEW

The subsurface conditions at the site are considered suitable for construction from a geotechnical perspective. A variety of foundation options are available for support of the planned structures. These include conventional spread footings, high bearing pressure spread footings after treating the underlying soils via vibro-replacement and deep foundations including augercast and driven piling. The foundation selection will be largely dependent on the structural loadings (which have not been determined at this time) which will impact foundation settlement performance. Lightly loaded columns (of about 200 kip loads or less) can be supported on shallow foundations after normal site preparation (i.e. soil compaction using a heavy vibratory roller) while foundation areas for heavier column loads will require pre-treatment by vibro-replacement (for columns loads of about 200 to 800 kips) or piling (for column loads in excess of about 800 kips). For each of the structures slab-on-grade construction may be employed for the ground level floors. Specific foundation design criteria for each of the foundation options are provided in this report.

Retaining walls for the Tipping Floor and Truck Ramps will retain up to 30 feet or more of soil and will need to rest on soils that are treated with vibro-replacement in order to limit their settlements to tolerable values.

Excavations for foundation construction and utility installation that bottom below the ambient groundwater level will need to be dewatered for these features to be constructed in the dry. It must be considered that as part of obtaining a dewatering permit with South Florida Water Management District that documentation will likely be required that demonstrates that the proposed dewatering will not create hydraulic gradients that could increase the potential for movement of highly mineralized groundwater from the areas of the landfills.

More detailed recommendations for design and construction of foundations and retaining walls, temporary excavations, pavement construction, utility installation and embankment construction for the truck ramps are provided hereafter.

EXCAVATIONS

Unbraced Excavations

Excavations needed to reach structure bottom levels will likely be made through existing and/or new granular fill, relatively clean sands, clayey sands and sand with shell. Such excavations may be completed using track-mounted hydraulic backhoes. Excavation sideslopes should be stable when cut at an inclination of 2:1 (horizontal: vertical) and when not subjected to seepage forces that result from dewatering via open pumping. However, should the excavation slopes be subjected to seepage forces that result from dewatering, we anticipate that the slopes will ravel to a more gentle inclination. The dewatering and excavation will therefore require very careful planning and permitting. If space limitations preclude the use of such open cut excavation slopes, then excavation support methods such as sheet piling will be required.

Sheet Piling

Steel sheet piling used to support the excavation sidewalls should be designed using the geotechnical parameters shown in the following table. These parameters were selected based upon the subsurface conditions disclosed in the borings and standard correlations available in the geotechnical literature.

Material	Unit Weight (pcf)		ϕ Degrees	δ Degrees	C (psf)	Coefficient of Lateral Earth Pressure			Elevation (feet- NGVD)
	Moist	Buoyant				Ka	Ko	Kp	
Sand Fill, Sand, Clayey Sand	105	48	30	17	0	0.33	0.50	3.00	+30 to +8
Sand, Sand/Shell	100	43	29	17	0	0.35	0.52	2.88	+8 to -32

Notes:

- ϕ indicates angle of internal friction
- δ indicates angle of wall friction
- C indicates cohesion
- Ka indicates coefficient of active lateral earth pressure
- Ko indicates coefficient of at-rest lateral earth pressure
- Kp indicates coefficient of passive lateral earth pressure

Dewatering

Should excavations bottom below the groundwater level that exists at the time of construction, dewatering may be required to enable the work to be completed in the dry. Since the project site is located very close to the existing Authority landfills, the South Florida Water Management District will likely require documentation that demonstrates the proposed dewatering will not create hydraulic gradients that could increase the potential for movement of highly mineralized groundwater from the areas of the landfills, all as part of a dewatering permit application.

CONVENTIONAL SPREAD FOOTINGS

Foundation Support

Lightly to moderately loaded columns and walls may be safely supported on conventional spread footings that are based in the granular engineered fill and/or in the native sands after normal site preparation that consists of soil compaction using heavy vibratory rollers. The footings may be designed for a net allowable bearing pressure of 4,000 pounds per square foot (psf). To provide an adequate factor of safety against a bearing capacity failure, the footing dimensions should be not less than those shown below. Please note that a slightly greater embedment depth and footing size is required for foundations within the refuse storage pit since they will likely bear below the groundwater table.

Finished Floor Elevation (feet-NGVD)	Minimum Footing Embedment Depth* (ft)	Minimum Footing Width (ft)
+19 (refuse storage pit)	3.5	3 (isolated footing), 2.5 (continuous footing)
+24 & +54	2.5	2 (isolated footing), 1.5 (continuous footing)

*Embedment depth refers to depth of footing bottom with respect to the lowest adjacent grade.

In the event that the footings are located adjacent to sloping ground (i.e. top of bank), the depth of embedment should be at least 4 feet below adjacent exterior grades, provided the fill slope is not steeper than 3:1 (horizontal:vertical). A minimum horizontal distance of 10 feet should also be maintained between the outside edge of the footing and the slope's top of bank.

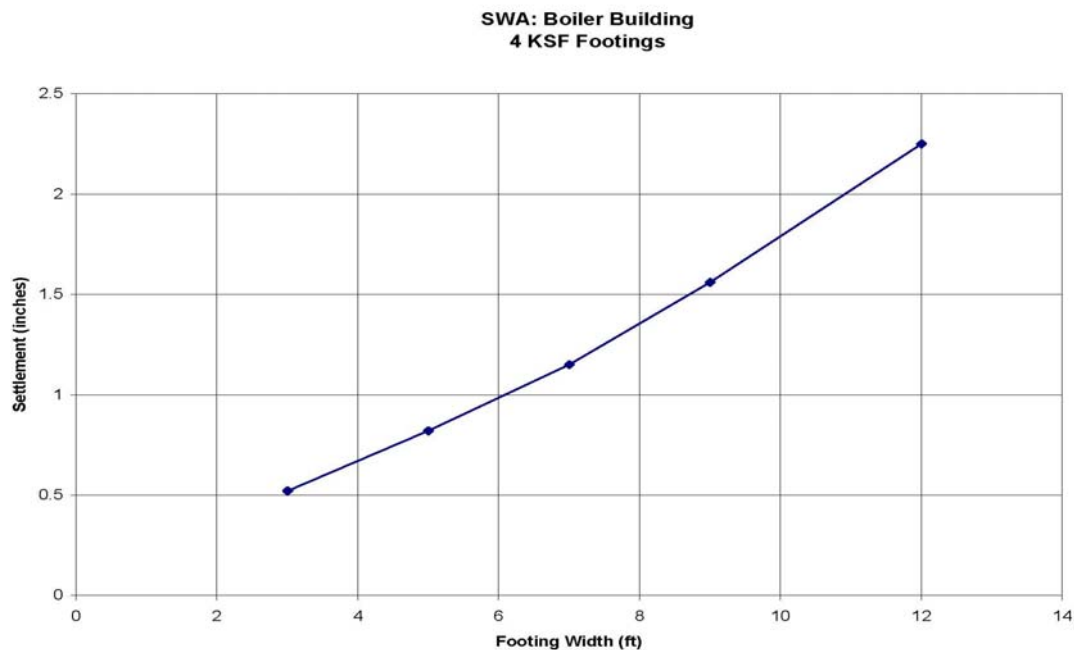
The allowable bearing pressure refers to dead and live loads and reflects a net increase in pressure over and above that exerted by the overburden soils. Consequently, the weight of the foundation concrete and soil backfill may be neglected in the sizing computations.

Individual footings should be excavated, formed and cast in the dry. The excavation bottoms should be densified (tamped) with a walk-behind type compactor prior to placement of reinforcing steel. The compactive efforts should achieve a minimum density of 95 percent of the ASTM D 1557 maximum dry density to a depth of 24 inches below the bottom of the footing.

We anticipate that some foundations within the refuse storage pit and building may bear below the groundwater level. If that is the case, groundwater lowering will be required to allow footing construction and compaction of the bearing soils. The amount of groundwater lowering can be minimized by over-excavating the foundation bottoms by 6 inches and replacing the over-excavated volumes with FDOT No. 57 coarse aggregate (that is completely enveloped within a filter fabric) in lieu of subgrade compaction.

Settlement

The settlement of the conventional spread footings that impose a net bearing pressure of 4,000 psf was estimated using the results of the borings and a method for predicting the elastic settlement of granular soils developed by Dr. John Schmmertman. The method uses the footing size and bearing pressure together with an estimate of the soil compressibility to predict the settlement of the substructure. Graphs of estimated settlement versus footing width (square) footings were developed for several of the project components using the borings drilled specifically for those structures. The graphs are provided in Appendix D. The plot of estimated settlement versus footing width for the Boiler Building foundations is shown below.



As may be expected the settlement increases with increasing footing size. Because of the granular nature of the subsoils, the settlements should occur rapidly and be virtually complete by the time the loads are fully imposed.

The settlement of spread footings is normally limited to 1 inch. In the case of the Boiler Building this occurs at a footing size of 6 feet, which corresponds to a column load of 144 kips. The load corresponding to 1 inch of settlement ranges from about 150 to 250 kips for most of the proposed structures. More heavily loaded columns and walls will require that soils beneath the spread footings be improved by vibro-replacement or that piles be used for support of the columns/walls. Design criteria for those options are described later in this report.

Lateral and Uplift Resistance

Foundations subject to lateral forces will resist the thrusts by earth pressure acting on the vertical faces of the footings located at right angles to the load direction and shearing resistance mobilized on the footing bottoms. Earth pressure resistance available from well-compacted moist granular backfill should be determined using an equivalent fluid pressure of 180 pounds per square foot per foot depth (psf/ft). The value for submerged backfill should be 90 psf/ft. Shearing resistance at the base of the footings should be calculated using a friction factor of 0.40. Use of the above values assumes that the footings can tolerate a horizontal translation on the order of three eighths to ½-inch. Resistance to sliding determined in accordance with the foregoing should be considered an ultimate value and factored for safety. A safety factor of at least 1.5 should be used.

Resistance to uplift forces that act on individual footings can be achieved with a combination of the dead weight of the foundation concrete and the weight of soil backfill overlying the foundations. We recommend that a moist soil unit weight of 115 pounds per cubic foot be used to compute the weight of soil backfill.

HIGH BEARING PRESSURE SPREAD FOOTINGS

Foundation Support

Columns and walls whose loads induce settlements of more than one inch as described in the Conventional Spread Footing section of this report will require alternative foundation support to limit such settlement. One foundation alternative is to use high bearing pressure spread footings after carrying out a program of ground improvement using vibro-replacement. Vibro-replacement consists of inserting a large vibratory probe into the ground to reinforce the soils by forming columns of gravel (i.e. stone columns) and to densify the sandy soils between stone columns. The treatment should extend to depths of twice the footing widths for column footings and four footing widths for continuous foundations, all measured from the base of the footings. Crushed limestone should be employed as add fill in the vibro-replacement points over the intervals to be treated. Foundations resting on vibro-replacement treated profile components may be designed for a net allowable bearing pressure of 8,000 pounds per square foot for dead plus normal live loads.

Vibro-Replacement

The vibro-replacement operations should be accomplished with a 150-horsepower (minimum) vibroflot having a centrifugal force of not less than 22 tons. Treatment of the profile components should extend from the footing bottoms to depths of twice the footing widths for column footings and four footing widths for continuous foundations. Add fill used in the compaction points should consist of crushed limestone meeting the gradational requirements established for Number 57 coarse aggregate within the Florida Department of Transportation *Standard Specifications for Roadways and Bridges*.

Evaluation of the adequacy of the vibro-replacement work should be based on review of vibro-replacement records that are maintained for each vibro-point. The records should include such information as add fill quantity, vibroflot withdrawal rate and amperage buildup during add fill compaction. In addition, the field quality assurance measures should include post vibro-replacement testing of the profile components using a static cone penetrometer equipped with a friction mantle (ASTM D 3441). Specifications prepared for the project should require that the granular soils over the treated interval afford an average cone penetrometer tip resistance of not less than 80 tons per square foot when tested anywhere in the compaction pattern.

Vibro-replacement operations should be performed by an experienced ground-improvement contractor using a plan that he prepares. The plan should utilize center-to-center spacings for the vibro-replacement points that are normally used in granular soils to achieve 80 percent relative density. Before adoption, the ground improvement plan should be reviewed by the geotechnical engineer and structural engineer for conformance with the intent of the foundation design.

Foundation Design and Settlement

To provide an adequate factor of safety against a bearing capacity failure the dimensions of the high bearing pressure footing should be not less than those shown below.

Finished Floor Elevation (feet-NGVD)	Minimum Footing Embedment Depth* (ft)	Minimum Footing Width (ft)
+19 (refuse storage pit)	5.0	4 (isolated footing), 3.5 (continuous footing)
+24 & +54	3.5	3 (isolated footing), 2.5 (continuous footing)

*Embedment depth refers to depth of footing bottom with respect to the lowest adjacent grade.

Settlement calculations for high bearing pressure spread footings followed the methods described earlier in this report for conventional spread footings. The estimated settlement for high bearing pressure spread footings proportioned for an allowable bearing pressure of 8 ksf are provided on Sheet D11 in Appendix D. As presented on this sheet, the anticipated settlement for an 800 kip column load should be about 1 inch. Owing to the granular nature of the subsurface materials, the settlement should occur coincident with application of load.

Lateral and Uplift Resistance

Resistance to lateral and uplift loads for high bearing pressure spread footings should be calculated using the design criteria presented earlier in this report for conventional spread footings.

TANK FOUNDATIONS

The Dirty Wastewater Tank, the Chemical Storage Tanks and the Water Treatment Storage Tanks may be supported on steel reinforced membrane slabs based in the granular engineered fill and/or in the improved native sands. The soils will first need to be improved either by compaction using a heavy vibratory roller (normal ground improvement) or vibro-replacement as described earlier. The improvement method selected should be chosen considering the anticipated tank settlements.

Settlement estimates were made for the tanks using the methods described earlier in this report for conventional spread footings. The tank diameters were anticipated to be 50 ft and 20 ft for the Dirty Wastewater Tank and the Chemical Storage Tanks, respectively. The diameters of the Water Treatment Storage Tanks were anticipated to range from 10 to 20 ft. The depth of the enclosed fluids were estimated to range up to approximately 30 feet, which is expected to result in a maximum bearing pressure of about 2 kips per square foot. The following table provides a summary of the predicted foundation settlements for the tank structures.

ESTIMATED TANK FOUNDATION SETTLEMENT (INCHES)							
Structure	Diameter (feet) and Contact Pressure						Subgrade Preparation
	10		20		50		
	1 ksf	2 ksf	1 ksf	2 ksf	1 ksf	2 ksf	
Chemical Storage	NA	NA	0.7	1.5	NA	NA	Normal Ground Improvement
Wastewater Tank	NA	NA	NA	NA	1.7	3.6	Normal Ground Improvement
Wastewater Tank	NA	NA	NA	NA	0.5	1.1	Vibro-Replacement
Water Treatment Building Storage	0.4	0.8	1.0	2.1	NA	NA	Normal Ground Improvement

Note: Normal ground improvement consists of compaction at the tank bearing levels using a heavy vibratory roller.

The results of the analyses indicate that the Chemical Storage and Water Treatment Storage Tanks should have low risk of more than normal foundation settlement (i.e. about 2 inches or less). Similarly, the 50-foot diameter tank loaded to 1 ksf should also result in foundation settlement of less than approximately 2 inches. However, if the 50-foot diameter tank is loaded to about 2 ksf then approximately 3.5 inches of total settlement is anticipated. This settlement magnitude is considered to be excessive. If the wastewater tank foundation pressure will reach 2 ksf then it will require ground improvement by preloading the tank footprint or by vibro-replacement as described earlier in this report (see High Bearing Pressure Spread Footings). The preloading would require placement of fill (with a minimum unit weight of 100 pounds per cubic foot) across the tank footprint to a height of not less than 20 feet above the proposed tank bearing level. The depth of vibro-treatment for the 50-foot diameter tank should extend to 50 feet below current grade (elevation of about -27 ft NGVD). The tank may also be supported on deep foundations as described later in this report.

The profile components in their existing condition, or after ground improvement, will have sufficient strength to safely accept the imposed loading of the tanks. We expect that the tank foundations will have a factor of safety in excess of three with respect to a shearing failure in the subsoils (provided their bearing pressures do not exceed 2 ksf), which is satisfactory.

Owing to the granular nature of the profile components, the tank foundation settlements will occur rapidly following full imposition of the weight of the fluids enclosed within the tanks. If possible, connections to the tanks should be delayed until the settlement is complete. The duration for hydrostatic or pressure testing of the tank for tightness could be utilized to pre-compress the subsurface materials, provided the test period is carried out over a several day period. Such pre-compression would likely limit the total settlement of the tanks to about one half inch upon reloading.

AUGERCAST PILES

General

The most heavily loaded foundations of the project and those structures that may need substantial resistance to overturning (e.g. the smoke stack) may be supported on deep foundations that consist of either precast prestressed concrete (PPC) piles or augercast piles (ACP). Design considerations for ACPs are presented hereafter (those for PPC Piles are provided later in the report).

Axial Capacities

Pressure grouted auger piling (ACPs) will derive their capacities from side shear in the sands and limestone formation (Stratum 8) that underlie the site. Since the top of the Strata 6 and 8 materials exist at variable elevations across the site, the limestone formation is discontinuous and the relative density and/or hardness of the materials also varies, it is clear that pile capacities will vary with respect to location. As a result we estimated the ACP allowable compressive pile capacities on a structure by structure basis. The analysis was performed for the Stack, APC Building, the Boiler Building, the Refuse Storage Pit/Building and the Tipping Floor/Building.

For each building, the borings drilled within that building were used to predict the allowable compressive capacities of 16, 18 and 24-inch diameter augercast piles of variable lengths. The allowable compressive capacities are presented in the table provided on Sheet E1 in Appendix E. Reference to the table shows that 16 inch diameter ACPs penetrating to a tip elevation of about -60 ft NGVD will have allowable compressive capacities that range from about 120 to 185 tons depending on location. For the 18 inch and 24 inch diameter ACPs the pile capacity ranges are 135 to 205 tons and 185 to 280 tons, respectively.

Piles fully reinforced for tension will be capable of safely supporting an uplift load of one half of the design vertical compressive capacities shown on Sheet E1.

The settlement of pile supported structures will be dependent on the magnitude of column/wall load, and the number, length and diameter of the piles supporting each column/wall. Provided the piling are socketed into the limestone formation, individual column foundation settlements are likely to not exceed 1 inch although this is dependent on loading. The most heavily loaded foundation is anticipated to be that of the 50 foot diameter dirty wastewater tank. If supported on piling that penetrates at least 10 feet into the limestone formation and assuming a tank load of about 2,000 tons (equivalent to 2 ksf bearing pressure at the tank base), we anticipate that the tank will settle about ½ inch when fully loaded.

Owing to the granular nature of the profile components that are to provide support to the pile foundations, the settlements should occur virtually as rapidly as the structural loads are applied and be nearly complete by full imposition of the dead load.

Lateral Capacities

The piles will resist lateral loads through a combination of pile stiffness (EI) and earth pressure. To model the pile-soil-lateral load interaction, we utilized the computer program COM624P developed by Shih Tower Wang and Dr. Lymon C. Reese. Variables in the analysis included soil properties (soil unit weight, friction angle and lateral soil modulus), pile parameters (i.e. pile length, butt width, elastic modulus and moment of inertia) and the lateral load acting at the pile head. The soil parameters used in the analysis are provided in the table on the following page.

Elevation (feet-NGVD)	Unit Weight (pcf)		ϕ Degrees	C (psf)	K (pci)
	Moist	Buoyant			
+16 to +8	105	48	30	0	27
+8 to -32	100	43	29	0	14
-20 to -32	--	43	29	0	20
-32 to -48	--	48	36	0	36
-48 to -60	--	48	38	0	50

Notes: ϕ indicates angle of internal friction
C indicates cohesion
K indicates lateral soil modulus

In our analysis the pile heads were taken to be fixed within rigid pile caps (i.e. pile heads are able to translate but not rotate). We assumed the pile heads will be located at an elevation of +16 feet NGVD and will be completely supported laterally by soil. The estimated pile lateral load capacities and maximum moments are presented in the following table.

Pile Diameter	Lateral Load Capacity Per Pile (kips)	Maximum Moment in Pile (inch-kips)
16 Inch Diameter ACP	8	675
18 Inch Diameter ACP	10	880
24 Inch Diameter ACP	15	1610

Notes: Lateral loads are assumed to act at pile head. The lateral capacity assumes that the pile heads can safely sustain a lateral translation of up to ½ inch. The capacities apply to group piles and have been reduced to account for interference effects between piles.

The results indicate that lateral load capacities of 8 to 15 kips can be anticipated for the piling. In the event that greater capacities are required, the designer should consider battering the piles. Battered ACPs can normally be installed at an angle of 6 vertical to 1 horizontal.

It should be recognized that the lateral load capacities presented in the table are based solely on stresses developed in the subsurface materials. Allowable stresses for the selected pile section should be verified by the designer.

Pile Load Testing

Pile load testing will be needed to confirm that the design capacity for the piles can be achieved at the site. Testing required for this purpose should be performed in accordance with ASTM D 1143 and Section 1806.4 of the Florida Building Code. For each pile diameter to be used on the project we recommend that two piles be load tested. We recommend that the compression test piles be instrumented with vibrating wire strain gauges to allow the load distribution in the rock socket to be determined. This will enable the mobilized side shear to be calculated. The mobilized side shear and evaluation of the pile settlement performance may then be used to finalize the embedment depths for all piles.

The piles intended for load testing should be installed with the same equipment that is to be used for the production piling. The piles that may be needed for anchorage of the load test frame should be constructed at production piling locations if feasible. Additionally, the piles should be constructed in close proximity to one of the engineering borings to allow correlation of the drilling characteristics and load-settlement relations with known subsurface conditions.

Augercast Pile Installation

The piles should be composed of grout having a 28-day unconfined compressive strength of not less than 3 times the gross bearing stress of the cross section of the pile at its design load. The piles should be installed with a minimum center-to-center spacing equivalent to three pile diameters. At cutoff, the centroids of the heads of group piles should be within 3 inches of design center. The centroids of the heads of single piles should be within 1 inch of design center.

Specifications for the installation of the augercast piles should require a grout factor (i.e. actual grout volume of pile divided by theoretical grout volume) of not less than 1.20. Lesser grout factors recorded for the production piles should be evaluated by the structural engineer and geotechnical engineer on a case-by-case basis for possible acceptance. All piles should be installed as true to vertical as practical with an out-of-plumbness that does not exceed two percent.

DRIVEN PILING

Axial Capacities

Driven precast prestressed concrete (PPC) pile axial compressive capacities were estimated using the subsurface data from borings drilled within the footprints of the Stack, the APC Building (FFH and SDA areas), the Boiler Building, the Refuse Storage Pit/Building and the Tipping Floor/Building. The capacities were estimated for 18-inch and 24-inch square PPC piles using the computer program FB-Deep. This program determines the capacity of driven piles using a methodology based on empirical correlations between cone penetrometer tests and Standard Penetration Tests for typical Florida soils and limestone. The calculated allowable pile compressive capacity is equal to half of the sum of the pile's ultimate side friction resistance plus its mobilized end-bearing capacity (which is 1/3 of the ultimate end bearing capacity). The allowable axial pile compressive capacities for various pile tip elevations extending to about elevation -80 feet NGVD are shown on Sheets E2 to E13 in Appendix E. The top of each pile was assumed to be at the existing ground surface.

Review of the analysis results shows that 18 inch square PPC piles that are driven to about elevation -70 ft NGVD will attain an allowable compressive capacity ranging from about 70 to more than 160 tons, depending on location. The 24 inch square PPC piles driven to -70 ft NGVD will attain allowable axial compressive capacities ranging from about 100 tons to more than 180 tons. It should be noted that the maximum load permissible per FDOT is 100 tons for 18 inch square PPC piles and 150 tons for 24 inch square PPC piles.

A table of the allowable pile tensile capacities is provided on Sheet E14 in Appendix E. The pile tensile capacities were taken to be one-third of the ultimate frictional resistance (determined by FB-Deep) at the pile-soil interface plus the dead weight of the pile. Eighteen inch square PPC piles with tip elevations of -70 ft NGVD are anticipated to attain allowable tensile capacities ranging from about 30 to 55 tons. Twenty four inch square PPC piles driven to this pile tip elevation are estimated to have allowable tensile capacities ranging from about 50 to 75 tons.

The allowable axial compressive load capacities presented in the Appendix are based solely on stresses mobilized in the subsurface materials. Structural stresses induced in the piles by driving may place greater restrictions on the allowable capacity and should be verified by the designer.

Lateral Capacities

The allowable lateral load capacities of the 18 inch and 24 inch Square PPC Piles were estimated as performed for the PPC piles. The estimated allowable lateral loads for the 18 and 24 inch square PPC piles are 12 kips and 20 kips, respectively. The maximum pile bending moment at these capacities is estimated to be 1,200 inch kips for the 18 inch square PPC pile and 2,500 inch kips for the 24 inch square PPC piles. These lateral load capacities and moments assume that the pile heads can safely sustain a lateral translation of up to ½ inch.

Battered piles with angles not exceeding 12 vertical to 1 horizontal may be used if additional lateral load resistance is needed.

Pile Load Testing

We recommend that the PPC piles be installed in accordance with Section 455 (Structures Foundations) of the FDOT's *Standard Specifications for Road and Bridge Construction*. The piles should be driven to a minimum blow count criteria that is determined during the implementation of a test pile program. The test pile program should include using the computer program Wave Equation Analysis of Piles (WEAP) after the performance of a dynamic load test (i.e. using the Pile Driving Analyzer). The test pile installations should be monitored by recording blows per foot over the full length of the pile's penetration. Data from the test pile monitoring should then be input into WEAP for selection of the finalized driving criteria and for ordering production pile lengths. Two piles should be load tested for each pile size (width) to be used for the project.

Vibrations from Pile Driving

Vibrations associated with pile driving operations should be monitored using a seismometer to verify that they are below the threshold levels that could damage the adjacent Waste to Energy structure. It has been our experience that ground vibrations at the location of structures supported on shallow foundations that are constructed upon loose sandy soils will need to be maintained at a peak particle velocity of 0.15 inch per second or less if seismic densification is to be avoided. In addition to vibration monitoring we recommend that surveying equipment be used to monitor vertical movement of the structure during pile driving.

A pre-construction survey should be performed for the nearby structure to identify any pre-existing cracks prior to pile driving. The survey should consist of photographic documentation of the existing facility and should be made in a concerted effort with the Authority. It should be noted that ground vibrations and noise due to pile driving may be a nuisance to occupants of the existing structure even if they do not cause structural damage to it. Further, the use of diesel hammers has been known to spray diesel fuel on adjacent facilities, particularly on windy days.

RETAINING WALLS

Ground Improvement

The retaining walls that will be needed for grade separation around the perimeter of the Tipping Floor area and along the truck ramps will retain fill heights that vary from a few feet at the base of the truck ramps to about 30 to 35 feet at the top of the ramps and around the perimeter of the Tipping Floor area. With respect to the higher walls, the magnitude of fill height to be retained will result in a relatively high vertical load being imposed on the wall foundations which could result in excessive foundation settlement.

Using the method described earlier in Conventional Spread Footings we estimated the settlement for the high walls assuming that they will consist of concrete cantilever retaining walls resting on continuous footings that are 30 feet wide. This width was estimated to be the approximate width required for the wall to be stable with respect to bearing, sliding and overturning (although the actual width must be determined by the wall designer). For an anticipated wall foundation bearing pressure of about 4,000 psf we estimated the high walls will settle about 4 inches in the event that normal site preparation is performed (i.e. vibratory roller compaction of subgrade soils). This amount of settlement is considered excessive. For this reason, the ground beneath the retaining walls should be improved using the vibro-replacement method described earlier in this report (see High Bearing Pressure Spread Footings) or should be supported on piling. We estimate that after vibro-replacement is performed the high walls will undergo a maximum total settlement of about 1-1/2 inch.

The vibro-replacement operations should be accomplished with equipment similar to that described in High Bearing Pressure Spread Footings and should extend from the footing bottoms to an elevation of -30 ft NGVD. The vibro-treatment should be accomplished as described earlier in this report (see High Bearing Pressure Spread Footings).

Foundation Design

If supported on shallow foundations, the retaining wall base for the greatest retained fill height (anticipated bearing pressure of about 4,000 psf) should be embedded at least 2 feet below the lowest adjacent grade. To avoid a global type bearing capacity failure the foundations should be not less than 16 feet wide. However, the design of the wall foundation against sliding and overturning will likely require a wider footing (as described earlier).

The retaining wall foundation should be excavated, formed and cast in the dry. The excavation bottoms should be densified (tamped) with a walk-behind type compactor prior to placement of reinforcing steel. The compactive efforts should achieve a minimum of 95 percent of the ASTM D 1557 maximum dry density to a depth of 24 inches below the bottom of the footing.

Wall Design

The retaining walls should be designed to resist lateral forces induced by granular backfill and surcharge loads from Tipping Floor traffic. Unbalanced hydrostatic pressures should be added to the lateral pressure as appropriate. Earth pressure induced by granular backfill should be computed in accordance with the following criteria:

Moist Soil Unit Weight	110 pounds per cubic foot
Buoyant Soil Unit Weight	55 pounds per cubic foot
Angle of Internal Friction	30 degrees
Earth Pressure Coefficients	
Active	0.33
Passive	3.00
At-Rest	0.50

TIPPING FLOOR

The placement of 30± feet of fill within the Tipping Floor footprint is anticipated to result in about 5 inches of settlement. This must be considered if any underground features (aside from the retaining wall foundations) will be based within or beneath the fill. Owing to the granular nature of the subsurface materials, the settlement should occur coincident with application of load. As a result, we anticipate that the settlement will be complete by the time the floor slab is to be constructed.

The settlement along the Tipping Floor perimeter is anticipated to be about 1-½ inches provided the walls are supported on soils treated with vibro-replacement. Settlement will attenuate away from the Tipping Floor with increasing distance owing to aerial subsidence. We estimate that ground subsidence will attenuate outward from the building edges at a rate of about 1/2 inch vertically per 30 feet laterally. This effect should be accounted for when selecting grades that depend upon specific gradients for proper function. To minimize differential settlement consideration should be given to delaying construction of the Refuse Storage Pit and Building until the fill within the Tipping Floor has been placed. The aerial subsidence is anticipated to result in about ½ inch of settlement at the location of the existing Waste to Energy structure.

The fill for the Tipping Floor building pad should be carefully placed so that they do not impose lateral pressures that exceed those used to design the retaining walls. The backfill should consist of clean, granular materials that are free of debris, cinders, combustibles, roots, sod, wood, cellulose and organic material. The fill should have not more than 10 percent passing the U.S. Standard No. 200 sieve (dry weight basis), have no particle size larger than 3 inches, and have not more than 2 percent organic matter (by weight). The backfill should be placed in lift thicknesses of 12 inches loose measure. Within 10 feet of the retaining walls, the backfill should be compacted with a hand-operated or walk-behind compactor. Each lift should be compacted until it meets a minimum density of 95 percent of the maximum dry density (per ASTM D 1557).

TRUCK RAMP EMBANKMENTS

Fill required to construct earthen embankments that may be needed for the truck ramps should consist of clean, granular materials as described previously under Tipping Floor. Fill slopes should be constructed at 3:1 (horizontal: vertical) or flatter. If steeper slopes are required to meet project grades then geogrid reinforcement may be needed. Establishment of grass root development on finished non-structural slopes should be promoted as early as possible in order to lessen the potential for erosion that results from stormwater runoff.

CONCRETE SLAB-ON-GRADE

Slab-on-grade construction may be used for the ground floors of the buildings. The slabs should be designed for the anticipated range of loadings that they will sustain. If the design employs elastic methods, a modulus of subgrade reaction of 200 pounds per cubic inch (pci) should be used for the supporting soils when well compacted.

Provisions should be made for the installation of an impervious membrane at the slab-on-grade and subgrade interface. Ordinarily, a 6-mil thick film of polyethylene is sufficient for most construction. However, floor coverings which are particularly sensitive to moisture flux may require a thicker membrane and/or one of different composition.

ASPHALTIC CONCRETE PAVEMENTS

The project site is underlain by sands of variable density. Earthwork construction to raise the site to proposed grades will result in densification of the subgrade in the areas to be paved. After densification, the soils will provide suitable support to a conventional flexible (asphalt surfaced) pavement section. The pavements along major haul routes should be designed specifically for the vehicle load intensities and frequencies anticipated. Pavement design for such haul routes should include layer coefficients of 0.44 for structural asphalt, 0.18 for limerock base course, and 0.08 for stabilized subbase. Properly compacted roadbed soils similar to the LBR tested fill materials should achieve a resilient modulus of at least 10,000 psi. The section presented on the following page is typically used in this geographic area for light to moderate duty pavements.

Component	Thickness and Composition
Wearing Course	1.5 inch structural asphaltic concrete
Base Course	8 inches of aggregate base course (LBR = 100 minimum)
Subbase	12 inches of stabilized subgrade material (LBR = 40 minimum)

Depending on the type of aggregate base course materials that are used, the base materials should meet the requirements of either Sections 911 or 913A of the Florida Department of Transportation “Standard Specifications for Road and Bridge Construction.” They should have a Limerock Bearing Ratio (LBR) of at least 100 and be compacted to not less than 98 percent of the Modified Proctor (AASHTO T-180) maximum dry density. Subbase materials should be densified to an equivalent relative compaction.

BURIED UTILITIES

Excavations

Depending upon the actual groundwater levels that exist at the time of construction, and proposed pipeline invert elevations, portions of the proposed pipelines may bottom near the prevailing water table. The excavations for pipeline construction should be made in a dry condition. Dewatering may be accomplished most readily by pumping from sumps, wellpoints, or wells, or combinations thereof. The prevailing water table should be lowered at least 2 feet below the proposed pipeline invert elevations. Considering the proximity of the existing Authority landfills, obtaining a dewatering permit will be very difficult. For this reason, buried utility inverts should be kept as high as practical.

The results of the borings indicate that the excavations may be performed using conventional backhoe equipment. Excavated materials consisting of organic-free sands and gravels with particle sizes less than 1 inch in diameter and no more than 10 percent fines (particles passing through the U.S. Standard Number 200 Sieve) may be stockpiled and re-used for backfill. Organic soils (i.e. muck) are unsuitable as backfill and should be wasted if encountered.

Pipe Support

The granular soils found in the borings at the anticipated pipeline invert elevations should be stable and provide adequate support for the proposed pipelines.

Bedding and Backfill

Should the excavations bottom within granular soils (sands), these soils will provide for uniform bedding of the piping. Pipelines that bottom in the clayey sands should be over-excavated to a depth of 6 inches for the subsequent placement of pipe bedding materials. Such bedding materials should consist of sands having a maximum size of 1 inch and not more than 10 percent materials passing the U.S. Standard No. 200 Sieve.

Groundwater levels should be maintained at least 24 inches below the excavation bottom during tamping of the bedding soil and placing of the pipe. Over-excavations, if necessary to remove unsuitable soils, will need to be dewatered so that the bottoms can be inspected and tamped. During backfilling, groundwater should be maintained a minimum of 24 inches below the working height of the fill.

Backfill should be placed in the dry and compacted in 12-inch thick lifts to at least 100 percent of the maximum dry density as determined by ASTM D 698 (AASHTO T-99). We recommend the use of only relatively light, hand-held compaction equipment to limit the potential for damage to the pipelines.

Thrust Blocks

Thrust blocks used to provide resistance to change in momentum at pipe bends should be supported laterally by granular materials compacted to high density. Excavations for thrust blocks should be backfilled with sand or sand and gravel (ASTM D 2487), placed in 12-inch thick lifts, and each lift should be compacted to at least 100 percent of maximum density (ASTM D 698). The zone of highly compacted backfill is defined by a truncated prism that is contained within planes which extend upward and outward from the base of the thrust block at a 3:1 (horizontal:vertical) slope.

LAKE AREA STRUCTURES

As indicated earlier in this report, subsurface exploration for the new pavements and structures to be located within the area of the existing lake (i.e. the 0.5 Million Gallon Fire Water Storage Tank and the Warehouse/Facility Maintenance Offices) will be performed after the lake is backfilled. Foundation design recommendations will be provided for those structures at that time. This notwithstanding, we anticipate that the borings will show relatively loose soil conditions since the lake backfilling will likely need to be performed in the wet without benefit of soil compaction owing to the previously mentioned issues with respect to dewatering. As a result, treatment (vibro-treatment or dynamic deep compaction) will likely be needed for the planned structures to allow the structures to be supported on shallow foundations. In addition, the loose soils within the pavement areas will likely need to be improved (via surcharging) prior to pavement construction.

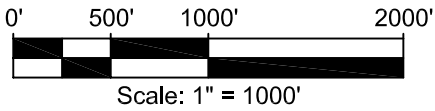
LIMITATIONS

Dunkelberger Engineering & Testing, Inc. has completed a geotechnical study for Malcolm Pirnie and the Solid Waste Authority of Palm Beach County, Florida in connection with the proposed Renewable Energy Facility No. 2 project in Palm Beach County, Florida. The purpose of the study was to explore the general subsurface conditions of the site and provide geotechnical related design criteria for the project. Dunkelberger warrants that the geotechnical design criteria presented in this report were promulgated based on recognized practice in the disciplines of soil mechanics, foundation engineering, and engineering geology. No other warranties are expressed or implied.

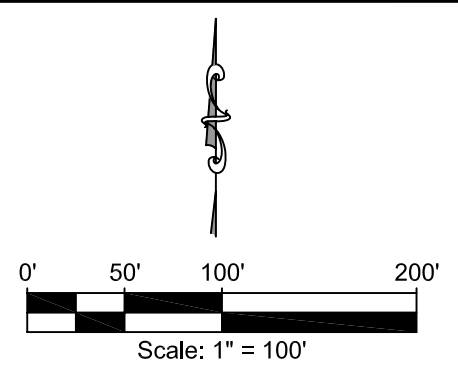
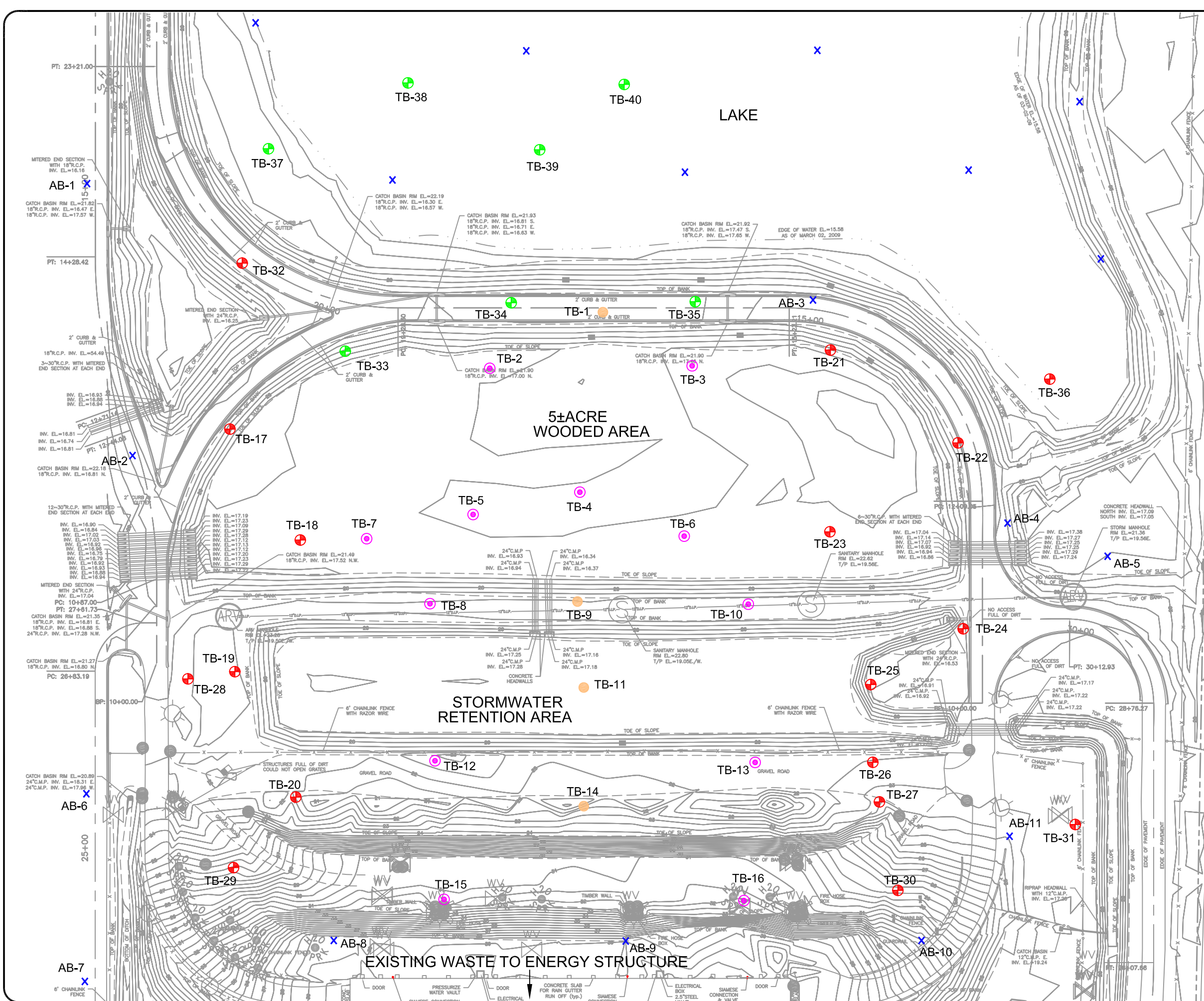
Z:\Projects\2009\7594 - AUTHORITY Waste to Energy Facility\21\Task Order 2\Reports\Geo Engineering Report\AUTHORITY WTE Geo Rpt (05-14-10):kea



SOURCE: GOOGLE EARTH.COM



DRAWN	GD	SITE LOCATION PLAN		
CHECKED	JD	PALM BEACH RENEWABLE ENERGY FACILITY NO. 2		
APPROVED	KA	PALM BEACH COUNTY, FLORIDA		
SCALE	1"= 1000'	DUNKELBERGER engineering & testing, inc.		
REVISED		DATE	5-14-10	PROJ. NO. WPB-09-7594
				FIGURE 1



LEGEND

- X - AUGER BORING LOCATION (10' DEEP)
- - STANDARD PENETRATION TEST (SPT) BORING LOCATION (25' DEEP)
- ⊕ - SPT BORING LOCATION (50' DEEP)
- - SPT BORING LOCATION (100' DEEP)
- - SPT BORING LOCATION (150' DEEP)
- TB-1 - TEST BORING AND NUMBER

SOURCE: GREENHORNE & OMARA, INC.

DRAWN	GD
CHECKED	JD
APPROVED	KA
SCALE	1"= 100'
REVISED	

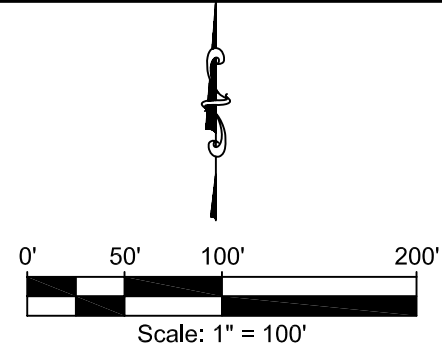
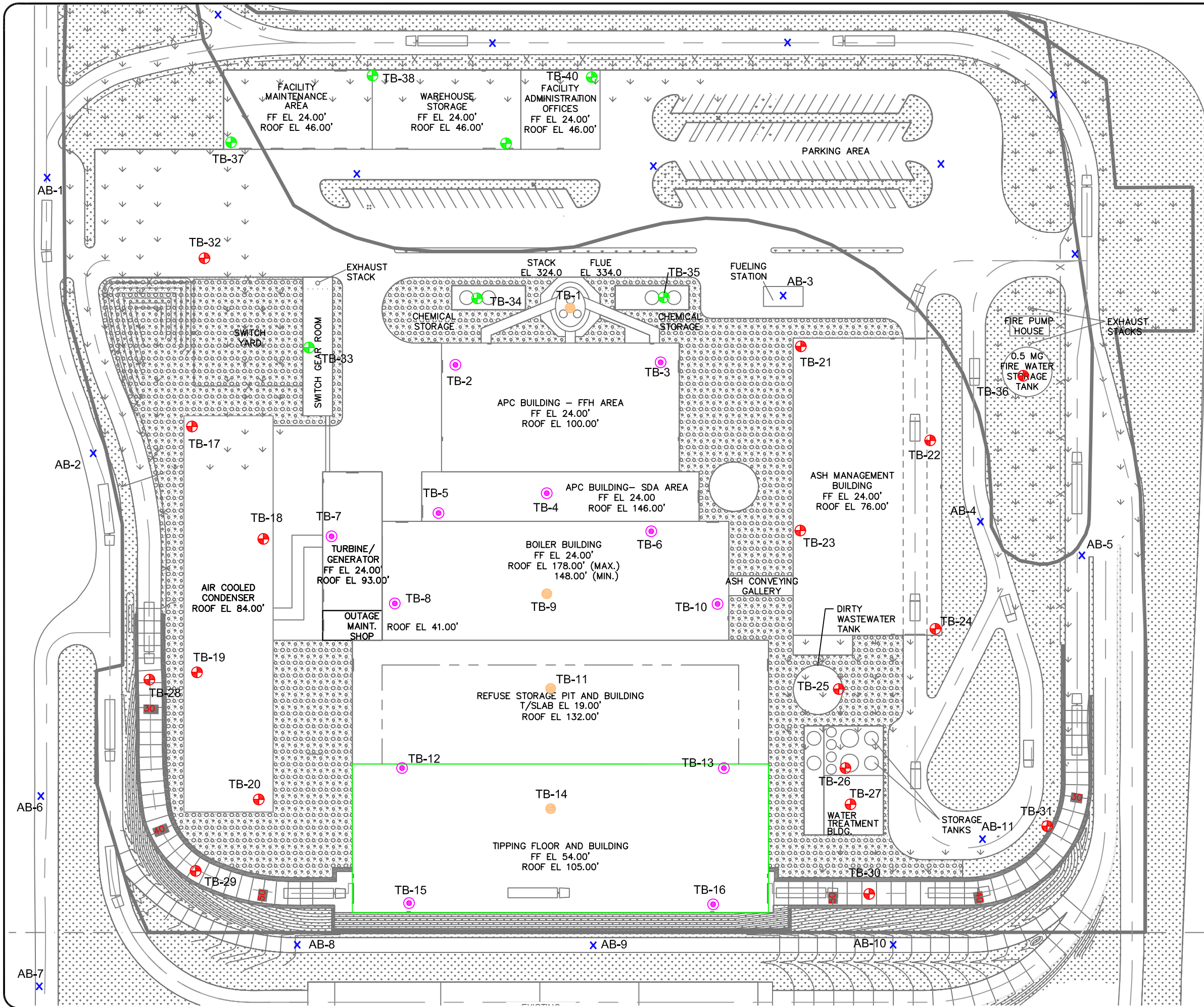
SUBSURFACE EXPLORATION PLAN

PALM BEACH RENEWABLE ENERGY FACILITY NO. 2

PALM BEACH COUNTY, FLORIDA

DUNKELBERGER
engineering & testing, inc.

DATE	5-14-10	PROJ. NO.	WPB-09-7594	SHEET	1A
------	---------	-----------	-------------	-------	----



LEGEND

✕

-

AUGER BORING LOCATION
(10' DEEP)

●

-

STANDARD PENETRATION TEST (SPT)
BORING LOCATION (25' DEEP)

⊕

-

SPT BORING LOCATION (50' DEEP)

●

-

SPT BORING LOCATION (100' DEEP)

●

-

SPT BORING LOCATION (150' DEEP)

TB-1

-

TEST BORING AND NUMBER

DRAWN

GD

CHECKED

JD

APPROVED

KA

SCALE

1"= 100'

REVISED

SUBSURFACE EXPLORATION PLAN

PALM BEACH RENEWABLE ENERGY FACILITY NO. 2

PALM BEACH COUNTY, FLORIDA

DUNKELBERGER
engineering & testing, inc.

DATE

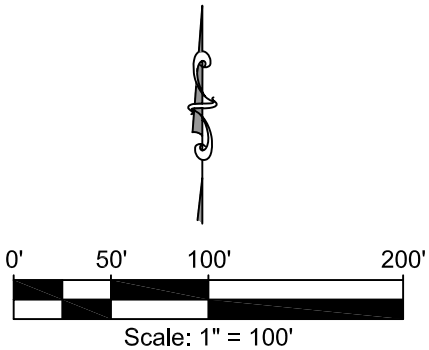
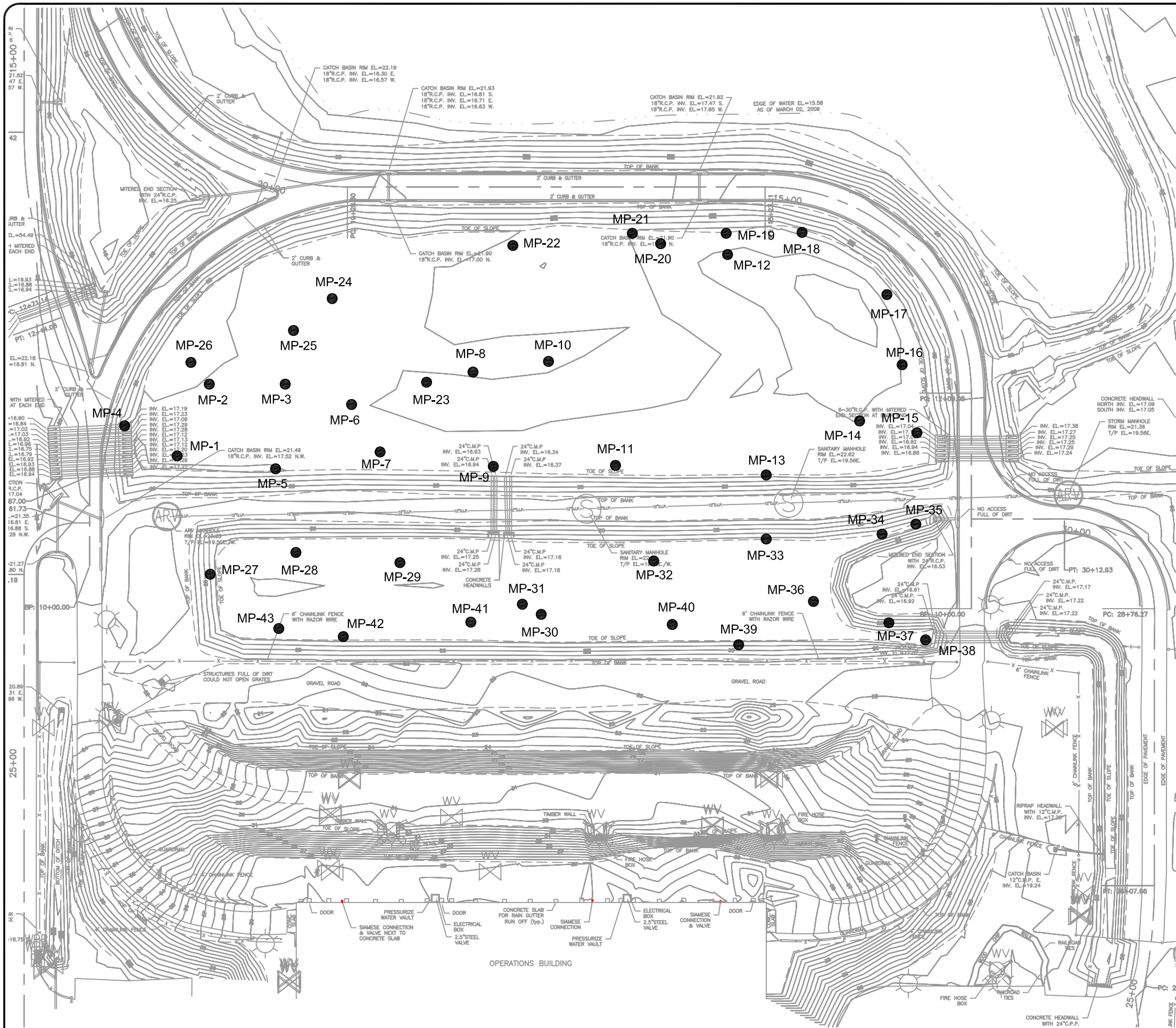
5-14-10

PROJ. NO.

WPB-09-7594

SHEET

1B



LEGEND

● - SOFT SEDIMENT PROBE LOCATION AND NUMBER

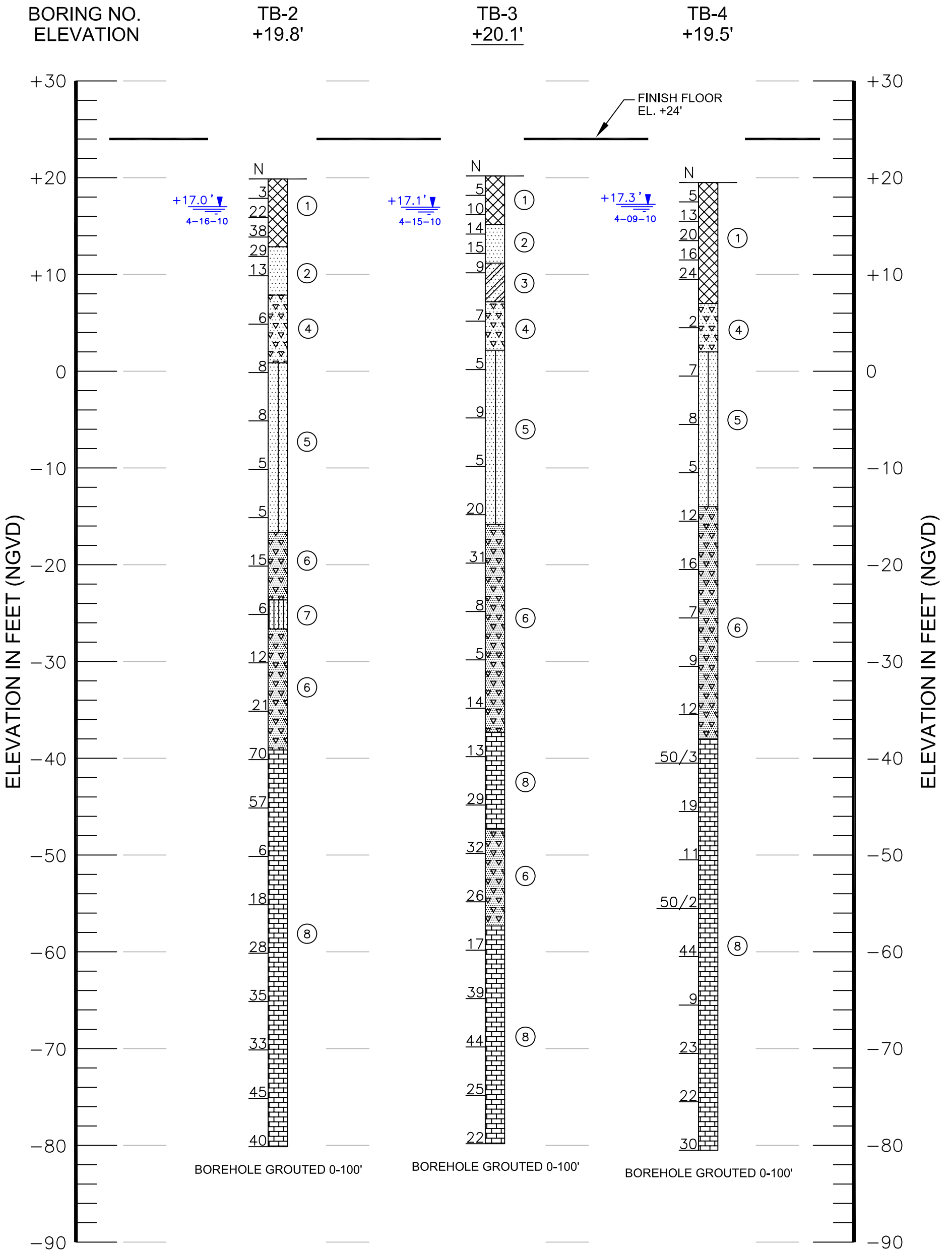
SOFT SEDIMENT PROBE NO.	WATER DEPTH (INCHES)	SOFT SEDIMENT THICKNESS (INCHES)
1	10"	7"
2	0	9"
3	3"	48"
4	5"	17"
5	12"	9"
6	5"	16"
7	11"	12"
8	4.5"	28"
9	12"	12"
10	5"	12"
11	10"	11"
12	9"	40"
13	9"	9"
14	13"	12"
15	9"	12"
16	8"	16"
17	8.5"	12"
18	9.5"	18"
19	12"	13"
20	8"	23"
21	0	10"
22	8"	17"
23	3"	44"
24	12"	12"
25	5"	48"
26	0"	36"
27	14"	10"
28	16"	11"
29	16"	11"
30	16"	11"
31	16"	11"
32	14"	8"
33	13"	13"
34	13.5"	17"
35	4.5"	12"
36	15.5"	14"
37	10"	7.5"
38	11"	14"
39	12.5"	10.5"
40	12"	11"
41	16"	8"
42	14"	7"
43	13"	6.5"

SOURCE: GREENHORNE & OMARA, INC.

DRAWN		GD	SUBSURFACE EXPLORATION PLAN			
CHECKED		JD	PALM BEACH RENEWABLE ENERGY FACILITY NO. 2			
APPROVED		KA	PALM BEACH COUNTY, FLORIDA			
SCALE		1"= 100'	DUNKELBERGER engineering & testing, inc.			
REVISED						
DATE		5-14-10	PROJ. NO.	WPB-09-7594	SHEET	1C

DUNKELBERGER
engineering & testing, inc.

(APC BUILDING - FFH AREA)



NOTE: FOR DETAILED LEGEND AND NOTES, SEE SHEET 2A

APC BUILDING - FFH AREA
SUBSURFACE PROFILES

PALM BEACH RENEWABLE ENERGY FACILITY NO. 2

PALM BEACH COUNTY, FLORIDA

DUNKELBERGER
engineering & testing, inc.

DRAWN	GD
CHECKED	JD
APPROVED	KA
SCALE	1" = 10'
REVISED	

DATE	5-14-10	PROJ. NO.	WPB-09-7594	SHEET	2B
------	---------	-----------	-------------	-------	----

TURBINE / GENERATOR

TB-7
+19.1'



- | | | | | | |
|------|---------|--------------|-------------|-------|----|
| DATE | 5-14-10 | PROJ.
NO. | WPB-09-7594 | SHEET | 2C |
|------|---------|--------------|-------------|-------|----|

(BOILER BUILDING)

BORING NO.
ELEVATION

TB-8
+21.1'

TB-9
+22.0'

TB-10
+22.3'

ELEVATION IN FEET (NGVD)

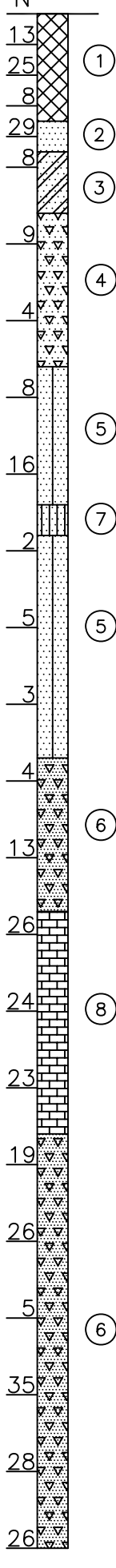
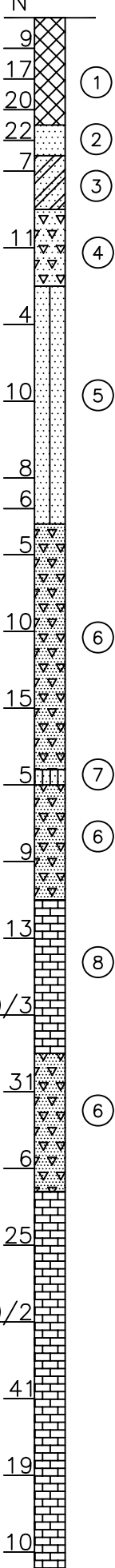
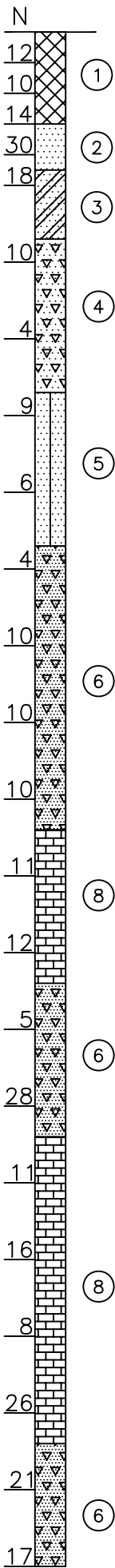
ELEVATION IN FEET (NGVD)

FINISH FLOOR
EL. +24'

+17.1'▼
3-22-10

+17.2'▼
4-01-10

+17.2'▼
3-31-10

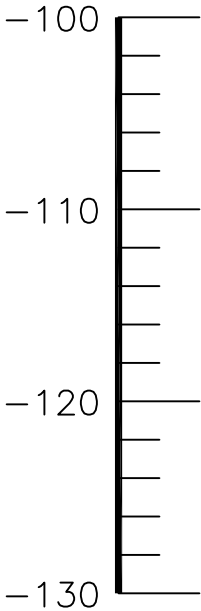


BOREHOLE GROUTED 0-100'

BOREHOLE GROUTED 0-100'

BORING NO.
ELEVATION

TB-9
CONTINUED



BOREHOLE GROUTED 0-150'

- ① SAND FILL (SP)
- ② SAND (SP)
- ③ CLAYEY SAND (SC)
- ④ SAND/SHELL (SP-SM, SM)
- ⑤ SAND (SP)
- ⑥ SAND/SHELL (SP-SM, SM)
- ⑦ SAND-SILT (SM, ML)
- ⑧ LIMESTONE

NOTE: FOR DETAILED LEGEND AND NOTES, SEE SHEET 2A

DRAWN	GD
CHECKED	JD
APPROVED	KA
REVISION	
SCALE	1" = 10'
DATE 5-14-10	
PROJ. NO. WPB-09-7594	
SHEET 2D	
SUBSURFACE PROFILES	
PALM BEACH RENEWABLE ENERGY FACILITY NO. 2	
PALM BEACH COUNTY, FLORIDA	
DUNKELBERGER engineering & testing, inc.	

(REFUSE STORAGE PIT AND BUILDING)

BORING NO.
ELEVATION

TB-11
+20.7'

TB-12
+22.1'

TB-13
+22.3'

ELEVATION IN FEET (NGVD)

ELEVATION IN FEET (NGVD)

FINISH FLOOR
EL. +19'

+17.5'▼
4-08-10

+17.9'▼
4-02-10

+17.3'▼
4-05-10

-Black organic
fine SAND (PT)

BOREHOLE GROUTED 0-100"

BOREHOLE GROUTED 0-100"

NOTE: FOR DETAILED LEGEND AND NOTES, SEE SHEET 2A

TB-11
CONTINUED

BOREHOLE GROUTED 0-150"

- ① SAND FILL (SP)
- ② SAND (SP)
- ③ CLAYEY SAND (SC)
- ④ SAND/SHELL (SP-SM, SM)
- ⑤ SAND (SP)
- ⑥ SAND/SHELL (SP-SM, SM)
- ⑦ SAND-SILT (SM, ML)
- ⑧ LIMESTONE

REFUSE STORAGE PIT AND BUILDING

SUBSURFACE PROFILES

PALM BEACH RENEWABLE ENERGY FACILITY NO. 2

PALM BEACH COUNTY, FLORIDA

DUNKELBERGER
engineering & testing, inc.

DRAWN	GD
CHECKED	JD
APPROVED	KA
SCALE	1" = 10'
REVISED	

DATE	5-14-10	PROJ. NO.	WPB-09-7594	SHEET	2E

(TIPPING FLOOR AND BUILDING)

BORING NO.
ELEVATION

TB-15
+31.0'

FINISH FLOOR
EL. +54'

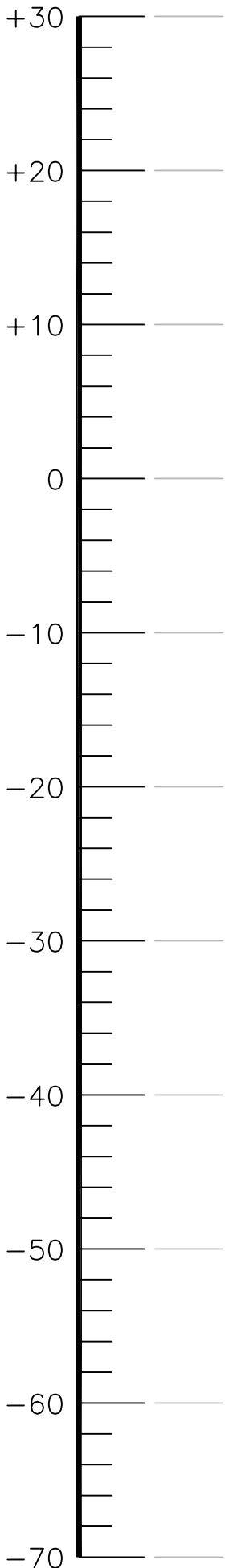
TB-14
+21.1'

FINISH FLOOR
EL. +54'

TB-16
+31.8'

FINISH FLOOR
EL. +54'

ELEVATION IN FEET (NGVD)



+20.8'
3-24-10

+19.0'
3-30-10

+18.6'
3-25-10

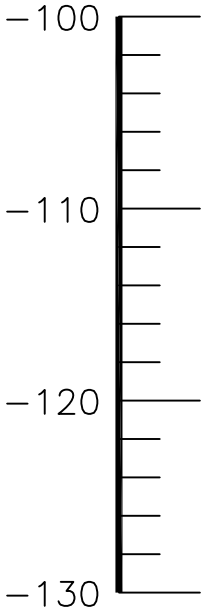
ELEVATION IN FEET (NGVD)

BOREHOLE GROUTED 0-100'

BOREHOLE GROUTED 0-100'

BORING NO.
ELEVATION

TB-14
CONTINUED



BOREHOLE GROUTED 0-150'

- ① SAND FILL (SP)
- ② SAND (SP)
- ③ CLAYEY SAND (SC)
- ④ SAND/SHELL (SP-SM, SM)
- ⑤ SAND (SP)
- ⑥ SAND/SHELL (SP-SM, SM)
- ⑦ SAND-SILT (SM, ML)
- ⑧ LIMESTONE

NOTE: FOR DETAILED LEGEND AND NOTES, SEE SHEET 2A

DRAWN
GD

CHECKED
JD

APPROVED
KA

SCALE
1" = 10'

REVISED

DATE
5-14-10

PROJ. NO.
WPB-09-7594

SHEET
2F

TIPPING FLOOR AND BUILDING

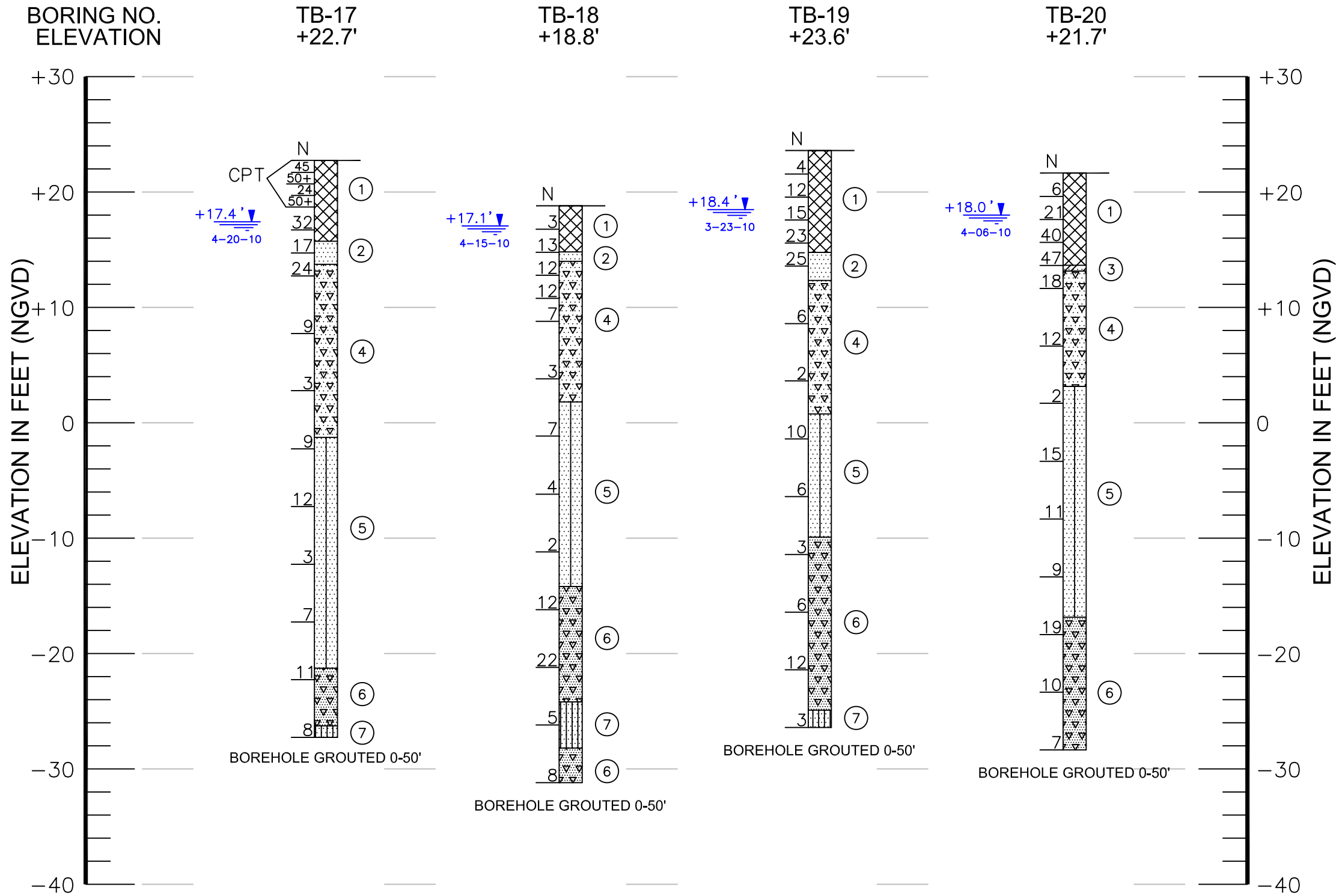
SUBSURFACE PROFILES

PALM BEACH RENEWABLE ENERGY FACILITY NO. 2

PALM BEACH COUNTY, FLORIDA

DUNKLEBARGER
engineering & testing, inc.

(AIR COOLED CONDENSER)

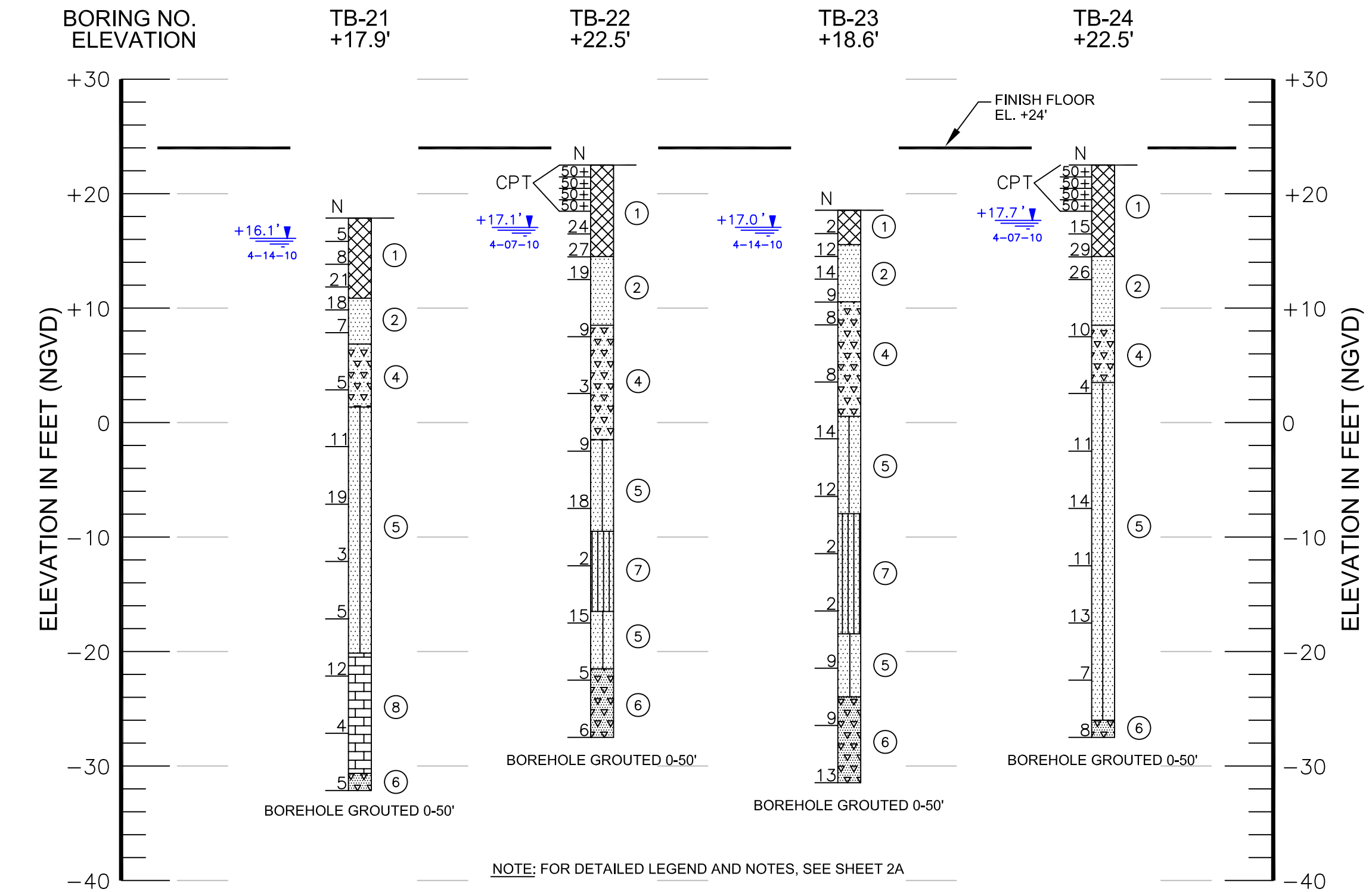


- ① SAND FILL (SP)
- ② SAND (SP)
- ③ CLAYEY SAND (SC)
- ④ SAND/SHELL (SP-SM, SM)
- ⑤ SAND (SP)
- ⑥ SAND/SHELL (SP-SM, SM)
- ⑦ SAND-SILT (SM, ML)
- ⑧ LIMESTONE

DRAWN	GD
CHECKED	JD
APPROVED	KA
SCALE	1" = 10'
REVISED	

AIR COOLED CONDENSER		
SUBSURFACE PROFILES		
PALM BEACH RENEWABLE ENERGY FACILITY NO. 2		
PALM BEACH COUNTY, FLORIDA		
DUNKELBERGER engineering & testing, inc.		
DATE	5-14-10	PROJ. NO. WPB-09-7594
		SHEET 2G

(ASH MANAGEMENT BUILDING)



- ① SAND FILL (SP)
- ② SAND (SP)
- ③ CLAYEY SAND (SC)
- ④ SAND/SHELL (SP-SM, SM)
- ⑤ SAND (SP)
- ⑥ SAND/SHELL (SP-SM, SM)
- ⑦ SAND-SILT (SM, ML)
- ⑧ LIMESTONE

DRAWN	GD
CHECKED	JD
APPROVED	KA
SCALE	1" = 10'
REVISED	

ASH MANAGEMENT BUILDING

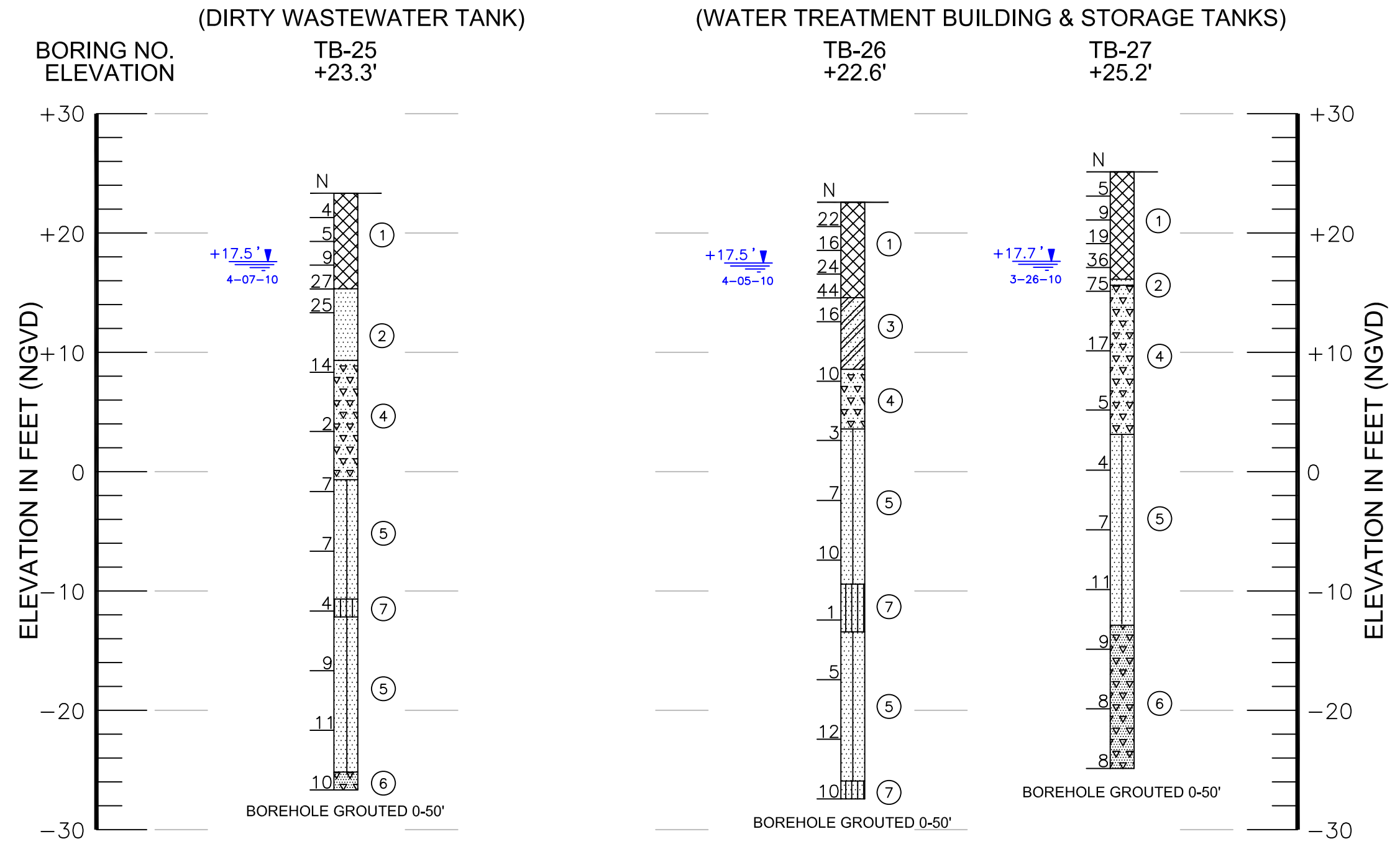
SUBSURFACE PROFILES

PALM BEACH RENEWABLE ENERGY FACILITY NO. 2

PALM BEACH COUNTY, FLORIDA

DUNKELBERGER
engineering & testing, inc.

DATE	5-14-10	PROJ. NO.	WPB-09-7594	SHEET	2H
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NOTE: FOR DETAILED LEGEND AND NOTES, SEE SHEET 2A

- ① SAND FILL (SP)
- ② SAND (SP)
- ③ CLAYEY SAND (SC)
- ④ SAND/SHELL (SP-SM, SM)
- ⑤ SAND (SP)
- ⑥ SAND/SHELL (SP-SM, SM)
- ⑦ SAND-SILT (SM, ML)
- ⑧ LIMESTONE

DRAWN GD

CHECKED JD

APPROVED KA

SCALE 1" = 10'

REVISED

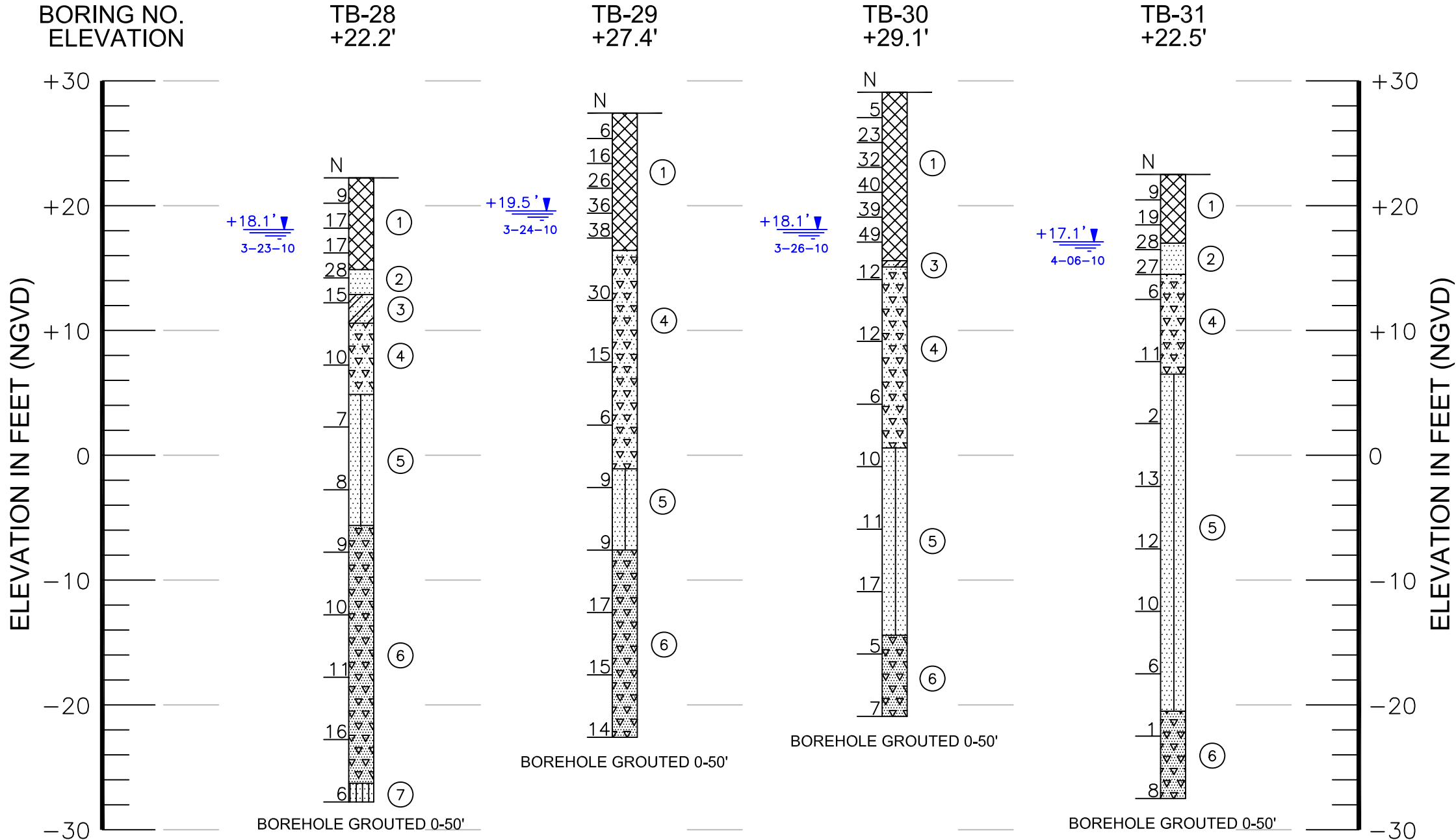
DIRTY WASTEWATER TANK
WATER TREATMENT BUILDING & STORAGE TANKS
SUBSURFACE PROFILES
PALM BEACH RENEWABLE ENERGY FACILITY NO. 2
PALM BEACH COUNTY, FLORIDA
DUNKELBERGER
engineering & testing, inc.

DATE 5-14-10

PROJ. NO. WPB-09-7594

SHEET 21

(TRUCK RAMPS & RETAINING WALLS)



- | | |
|---|--------------------------|
|  | ① SAND FILL (SP) |
|  | ② SAND (SP) |
|  | ③ CLAYEY SAND (SC) |
|  | ④ SAND/SHELL (SP-SM, SM) |
|  | ⑤ SAND (SP) |
|  | ⑥ SAND/SHELL (SP-SM, SM) |
|  | ⑦ SAND-SILT (SM, ML) |
|  | ⑧ LIMESTONE |

NOTE: FOR DETAILED LEGEND AND NOTES, SEE SHEET 2A

DRAWN	GD
CHECKED	JD
APPROVED	KA
SCALE	1" = 10'
REVISED	

TRUCK RAMPS & RETAINING WALLS

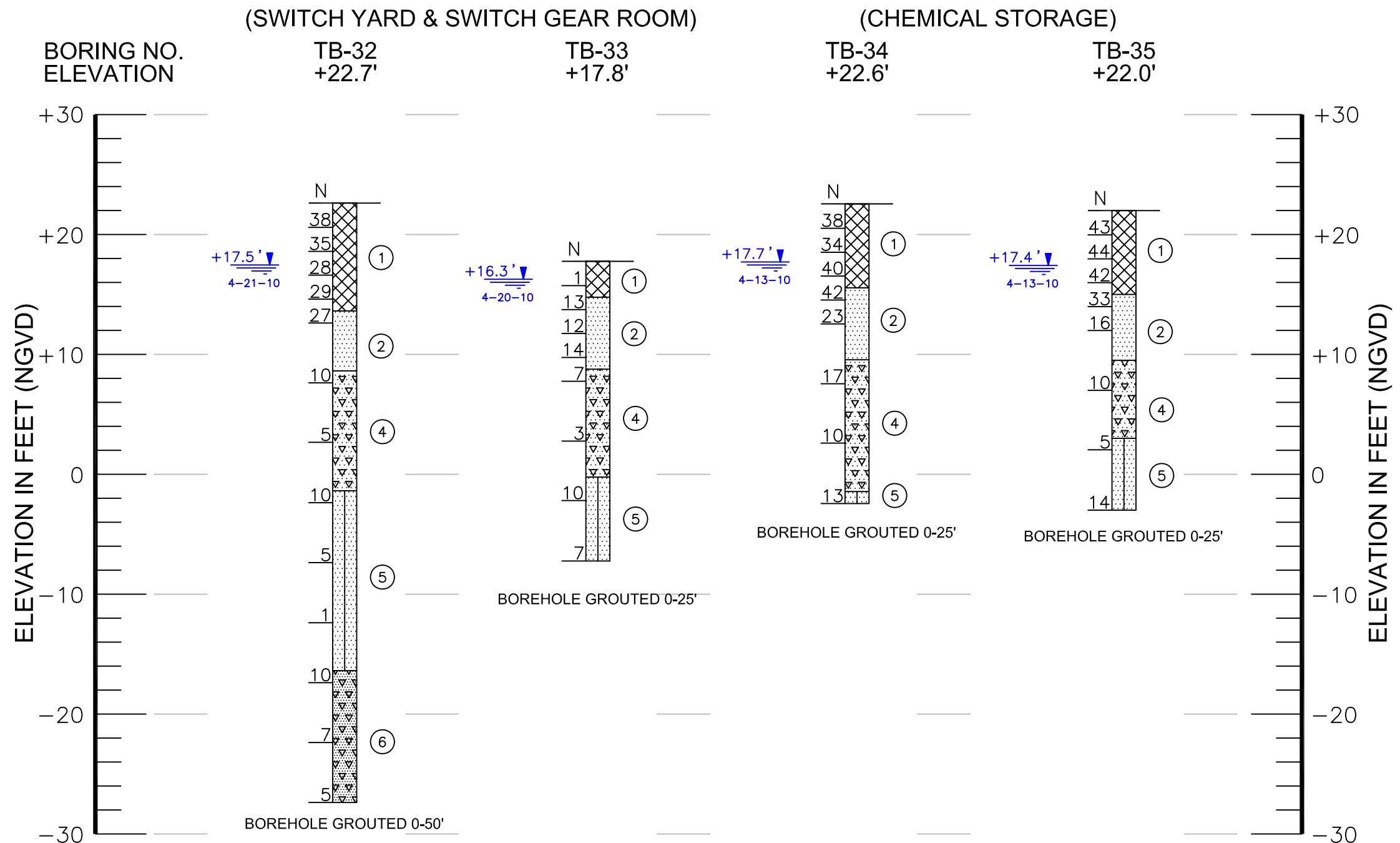
SUBSURFACE PROFILES

PALM BEACH RENEWABLE ENERGY FACILITY NO. 2

PALM BEACH COUNTY, FLORIDA

DUNKELBERGER
engineering & testing, inc.

DATE	5-14-10	PROJ. NO.	WPB-09-7594	SHEET	2J
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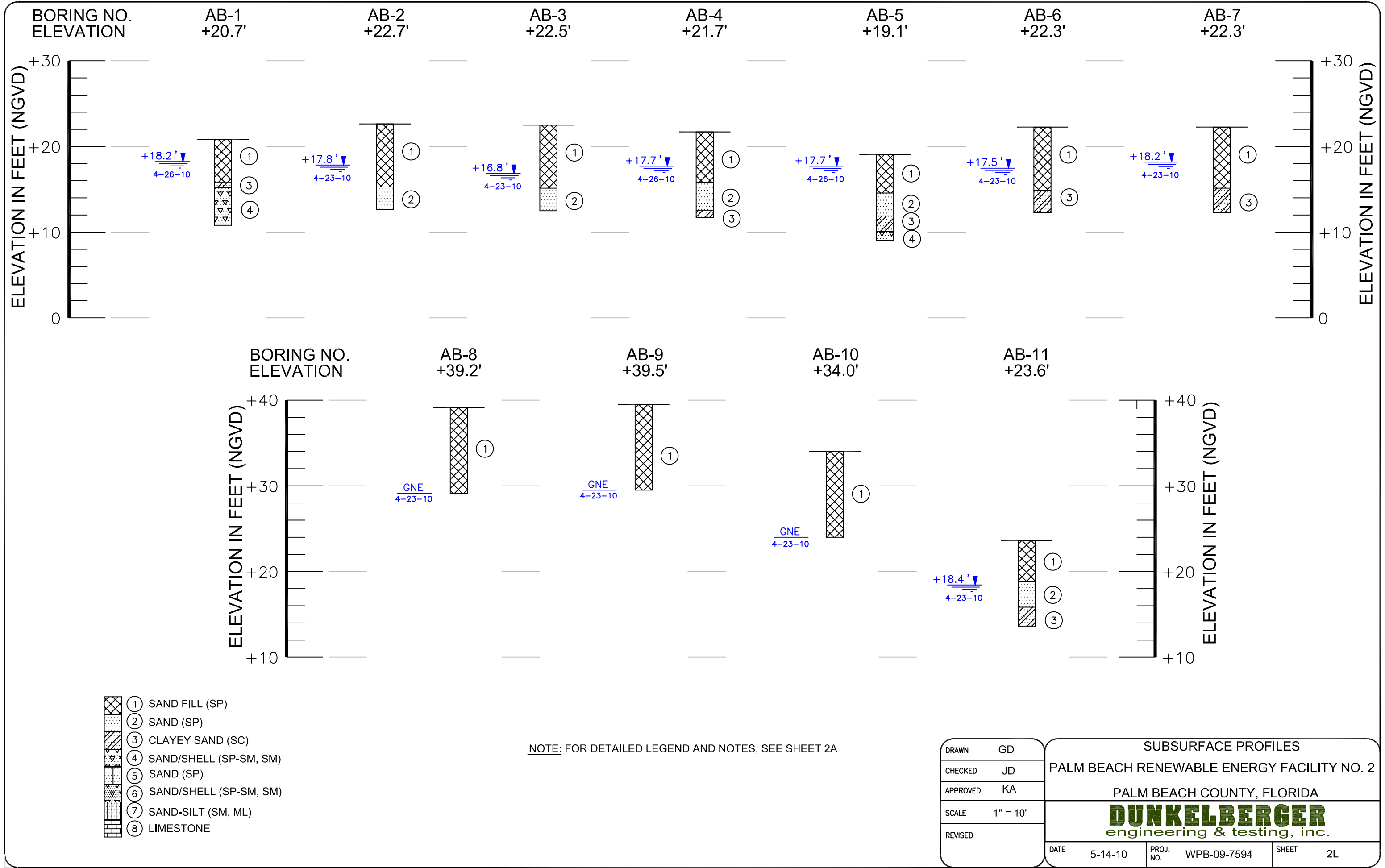


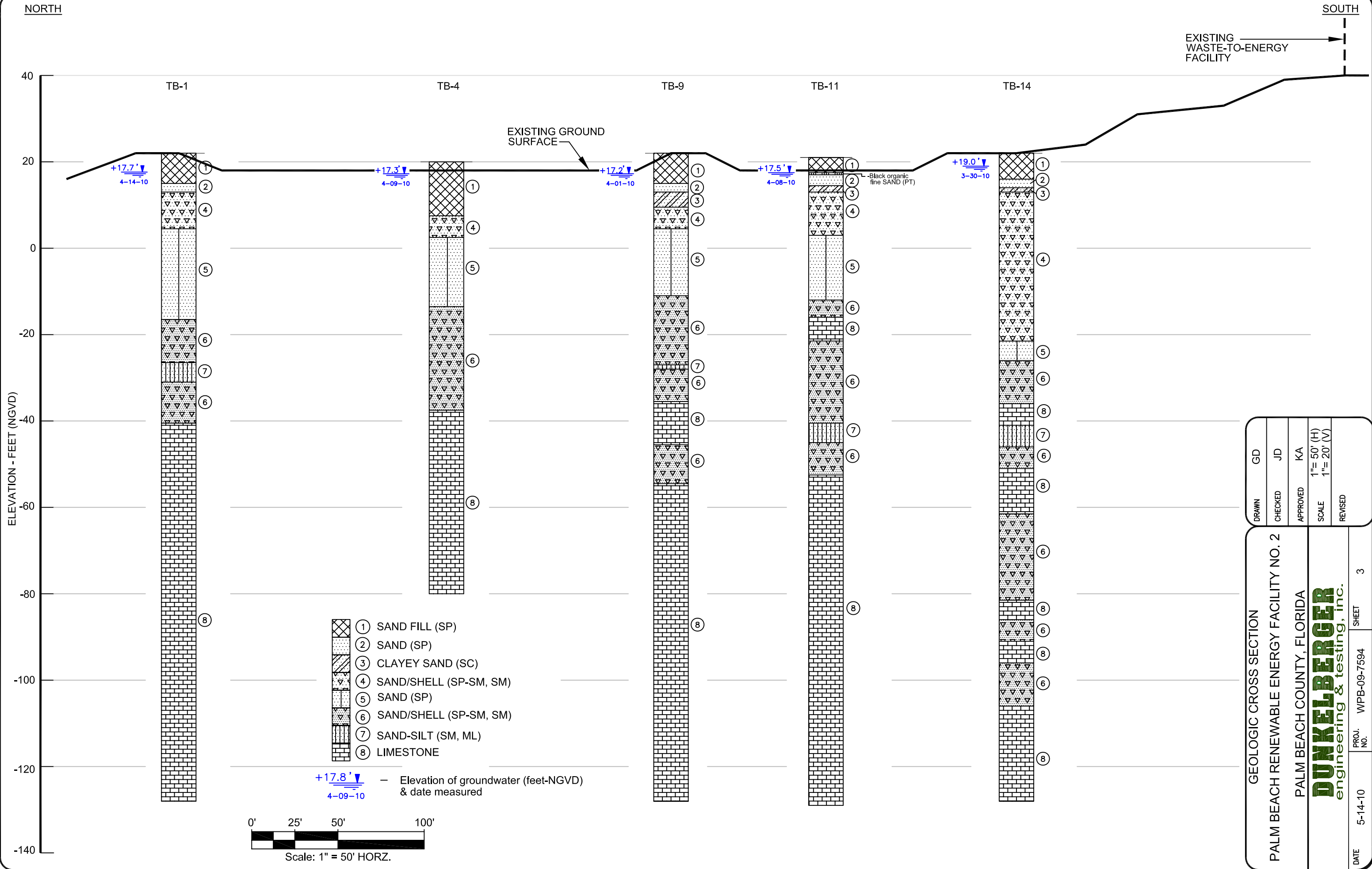
NOTE: FOR DETAILED LEGEND AND NOTES, SEE SHEET 2A

- ① SAND FILL (SP)
- ② SAND (SP)
- ③ CLAYEY SAND (SC)
- ④ SAND/SHELL (SP-SM, SM)
- ⑤ SAND (SP)
- ⑥ SAND/SHELL (SP-SM, SM)
- ⑦ SAND-SILT (SM, ML)
- ⑧ LIMESTONE

SWITCH YARD & SWITCH GEAR ROOM
CHEMICAL STORAGE

DRAWN	GD	SUBSURFACE PROFILES	
CHECKED	JD	PALM BEACH RENEWABLE ENERGY FACILITY NO. 2	
APPROVED	KA	PALM BEACH COUNTY, FLORIDA	
SCALE	1" = 10'	DUNKELBERGER engineering & testing, inc.	
REVISED		DATE	5-14-10
		PROJ. NO.	WPB-09-7594
		SHEET	2K





APPENDIX A

SITE PHOTOGRAPHS



Photograph 1: Site viewed west from near NE Corner of existing facility.



Photograph 2: Site viewed NW from near NE Corner of existing facility.



Photograph 3: Site viewed north from near NE Corner of existing facility.



Photograph 4: Site viewed NE from near NE Corner of existing facility.



Photograph 5: Drilling rig located on Boring TB-10, viewing west.



Photograph 6: Lake located at northern portion of site.



Photograph 7: Wooded portion of site.



Photograph 8: Eastern portion of site viewing SE from north roadway.

APPENDIX B

SURVEY DATA (BORING COORDINATES/ELEVATIONS)

BORING LOCATIONS AT THE WASTE ENERGY PLANT

<u>POINT NUMBER</u>	<u>NORTHING</u>	<u>EASTING</u>	<u>ELEVATION</u>	<u>DESCRIPTION</u>
2000	888241.896	779490.790	22.17	TB#28
2001	888038.414	779556.497	27.43	TB#29
2002	888238.777	779547.779	23.61	TB#19
2003	888123.384	779585.261	21.68	TB#20
2004	888027.151	779777.042	31.04	TB#15
2005	888116.508	779897.341	21.10	TB#14
2006	888023.778	780065.514	31.82	TB#16
2007	888024.932	780253.821	29.07	TB#30
2008	888093.372	780228.190	25.17	TB#27
2009	888163.864	780222.457	22.64	TB#26
2010	888074.145	780422.139	22.51	TB#31
2011	888168.697	780089.630	22.29	TB#13
2012	888168.731	779723.821	22.13	TB#12
2013	888290.413	780303.001	22.53	TB#24
2014	888230.863	780218.316	23.34	TB#25
2015	888289.912	780316.509	19.53	TB#9

THESE WERE SHOT ON 3-22-10

2050	888485.138	780300.630	22.52	TB#22
2051	888639.739	780012.691	22.04	TB#35
2052	888620.646	779928.637	22.39	TB#1
2053	888638.844	779821.985	22.56	TB#34
2054	888697.357	779535.821	22.68	TB#32

THESE WERE SHOT ON 3-26-10

2055	888572.664	780196.386	17.86	TB#21
2056	888595.135	780050.608	18.05	TB#3
2057	888602.101	779774.813	19.83	TB#2
2058	888570.943	779646.242	17.81	TB#33
2059	888518.380	779547.073	22.66	TB#17

THESE WERE SHOT ON 4-02-10

2067	888381.291	779615.306	18.80	TB#18
2068	888382.942	779680.701	19.13	TB#7
2069	888400.095	779803.145	19.97	TB#5
2070	888436.855	779896.783	19.54	TB#4
2071	888244.622	779910.73	20.66	TB#11
2072	888396.575	780041.784	20.26	TB#6
2073	888401.613	780199.526	18.60	TB#23
2074	888335.665	779736.535	21.12	TB#8

THESE WERE SHOT ON 4-14-10

BORING LOCATIONS AT THE WASTE ENERGY PLANT

<u>POINT NUMBER</u>	<u>NORTHING</u>	<u>EASTING</u>	<u>ELEVATION</u>	<u>DESCRIPTION</u>
2075	888324.720	779905.058	21.96	TB#9
2076	888312.678	780112.980	22.32	TB#10

THESE WERE SHOT ON 4-19-10

2078	888598.749	780039.122	20.78	TB#3
------	------------	------------	-------	------

THIS WAS SHOT ON 4-20-10

	888802.337	779377.974	20.66	AB#1
	888473.068	779442.485	22.66	AB#2
	888645.119	780131.504	22.52	AB#3
	888398.181	780344.112	21.71	AB#4
	888340.500	780452.681	19.09	AB#5
	888143.334	779397.448	22.26	AB#6
	887931.067	779396.610	22.29	AB#7
	887959.544	779665.014	39.23	AB#8
	887956.818	779917.561	39.49	AB#9
	887968.131	780234.780	34.04	AB#10
	888094.254	780348.038	23.63	AB#11

THESE WERE SHOT ON 4-22-10

APPENDIX C

LABORATORY TEST RESULTS

SHEET C1

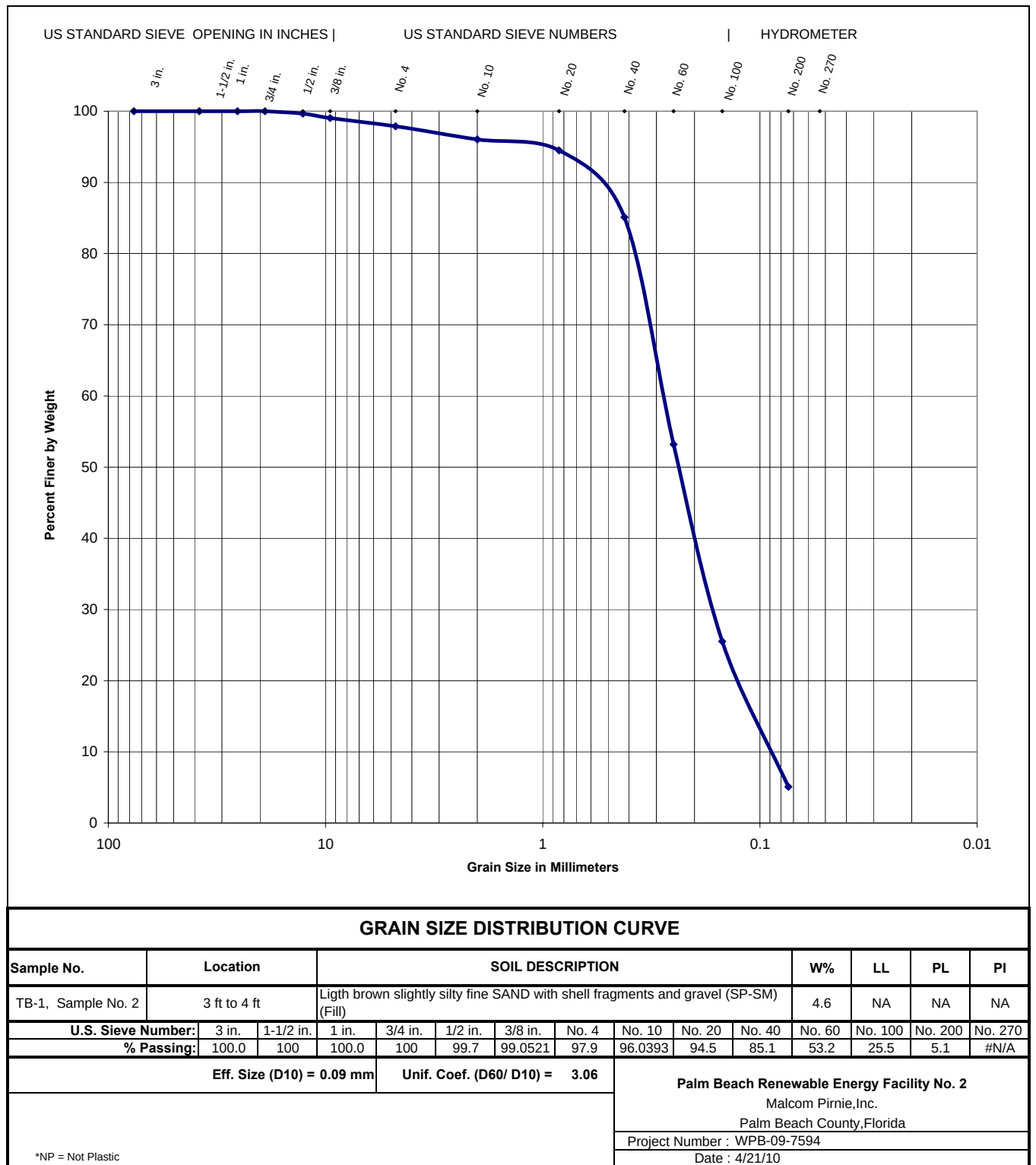
SUMMARY OF INDEX PROPERTY TEST RESULTS
PALM BEACH RENEWABLE ENERGY FACILITY No. 2
PALM BEACH COUNTY, FLORIDA

BORING NUMBER	SAMPLE DEPTH	MOISTURE CONTENT (%)	SIEVE ANALYSES											USCS GROUP
			1"	3/4"	1/2"	3/8"	#4	#10	#20	#40	#60	#100	#200	
FILL														
TB - 1	3.0 - 4.0	4.6	100.0	100.0	99.7	99.1	97.9	96.0	94.5	85.1	53.2	25.5	5.1	SP-SM
TB - 8	3.0 - 4.0	13.1	100.0	100.0	100.0	98.9	95.5	92.5	90.1	77.7	46.3	23.4	4.6	SP
TB - 15	5.0 - 6.0	15.1	100.0	100.0	100.0	99.3	97.6	97.3	96.6	90.6	68.8	43.9	9.6	SP-SM
TB - 16	9.0 - 10.0	6.7	100.0	100.0	100.0	99.3	97.3	94.8	92.8	80.7	46.3	22.5	4.9	SP
Average		9.9	100.0	100.0	99.9	99.2	97.1	95.2	93.5	83.5	53.7	28.8	6.1	
SAND														
TB - 1	7.0 - 8.0	18.1	100.0	100.0	100.0	100.0	100.0	100.0	99.9	93.3	69.2	42.2	6.1	SP-SM
TB - 10	7.0 - 8.0	18.3	100.0	100.0	100.0	100.0	100.0	100.0	99.9	94.5	71.9	44.2	3.5	SP
TB - 28	7.0 - 8.0	19.0	100.0	100.0	100.0	100.0	100.0	100.0	99.9	95.0	72.9	47.3	5.8	SP-SM
Average		18.5	100.0	100.0	100.0	100.0	100.0	100.0	99.9	94.3	71.3	44.6	5.1	
CLAYEY SAND														
TB - 8	9.0 - 10.0	18.8	100.0	100.0	100.0	100.0	100.0	100.0	99.7	94.6	76.1	53.9	15.5	SC
TB - 28	9.0 - 10.0	60.9	100.0	100.0	100.0	100.0	100.0	100.0	99.9	94.9	76.2	51.3	14.0	SC
Average		39.9	100.0	100.0	100.0	100.0	100.0	100.0	99.8	94.8	76.2	52.6	14.8	
SAND WITH SHELL														
TB - 1	9.0 - 10.0	14.0	100.0	96.7	83.1	77.3	69.7	62.6	58.1	52.9	41.6	29.0	10.6	SP-SM
TB - 8	14.0 - 15.0	16.7	100.0	100.0	98.0	96.6	85.0	73.5	65.5	55.2	30.9	14.7	5.2	SP-SM
TB - 10	14.0 - 15.0	18.0	100.0	100.0	96.5	93.5	78.5	59.0	48.2	41.4	29.6	20.4	12.0	SM
TB - 15	19.0 - 20.0	22.6	100.0	100.0	100.0	100.0	96.1	91.6	87.2	81.2	65.5	45.7	9.8	SP-SM
Average		17.8	100.0	100.0	98.2	96.7	86.5	74.7	67.0	59.3	42.0	26.9	9.0	
SAND														
TB - 1	24.0 - 25.0	21.7	100.0	100.0	100.0	100.0	100.0	100.0	99.9	98.6	89.0	50.2	3.0	SP
TB - 8	29.0 - 30.0	23.5	100.0	100.0	100.0	100.0	100.0	99.8	99.7	93.1	70.6	52.0	6.0	SP-SM
TB - 10	24.0 - 25.0	21.7	100.0	100.0	100.0	100.0	100.0	100.0	99.9	92.6	72.1	46.2	3.2	SP
TB - 15	34.0 - 35.0	22.0	100.0	100.0	100.0	100.0	100.0	99.9	99.4	90.1	65.2	23.7	3.4	SP
Average		22.2	100.0	100.0	100.0	100.0	100.0	99.9	99.7	91.9	69.3	40.6	4.2	
SAND WITH SHELL														
TB - 1	44.0 - 45.0	26.0	100.0	100.0	100.0	99.3	97.1	94.4	92.1	88.1	81.6	67.9	9.1	SP-SM
TB - 15	49.0 - 50.0	17.1	100.0	100.0	95.8	91.8	79.8	62.5	56.6	53.1	41.6	17.4	7.5	SP-SM
TB - 16	49.0 - 50.0	21.4	100.0	100.0	97.0	97.0	92.5	89.6	87.8	84.2	74.7	39.5	6.8	SP-SM
Average		21.5	100.0	100.0	97.6	96.0	89.8	82.2	78.8	75.1	66.0	41.6	7.8	
SILTY SAND														
TB - 1	49.0 - 50.0	25.6	100.0	100.0	91.6	88.5	83.2	78.9	75.4	72.1	67.4	54.0	28.4	SM
TB - 10	33.0 - 34.0	37.3	100.0	100.0	100.0	100.0	100.0	99.9	99.6	96.7	82.1	58.4	33.8	SM
Average		31.5	100.0	100.0	100.0	100.0	100.0	99.9	99.6	96.7	82.1	58.4	33.8	

GRAIN SIZE DISTRIBUTION CURVES

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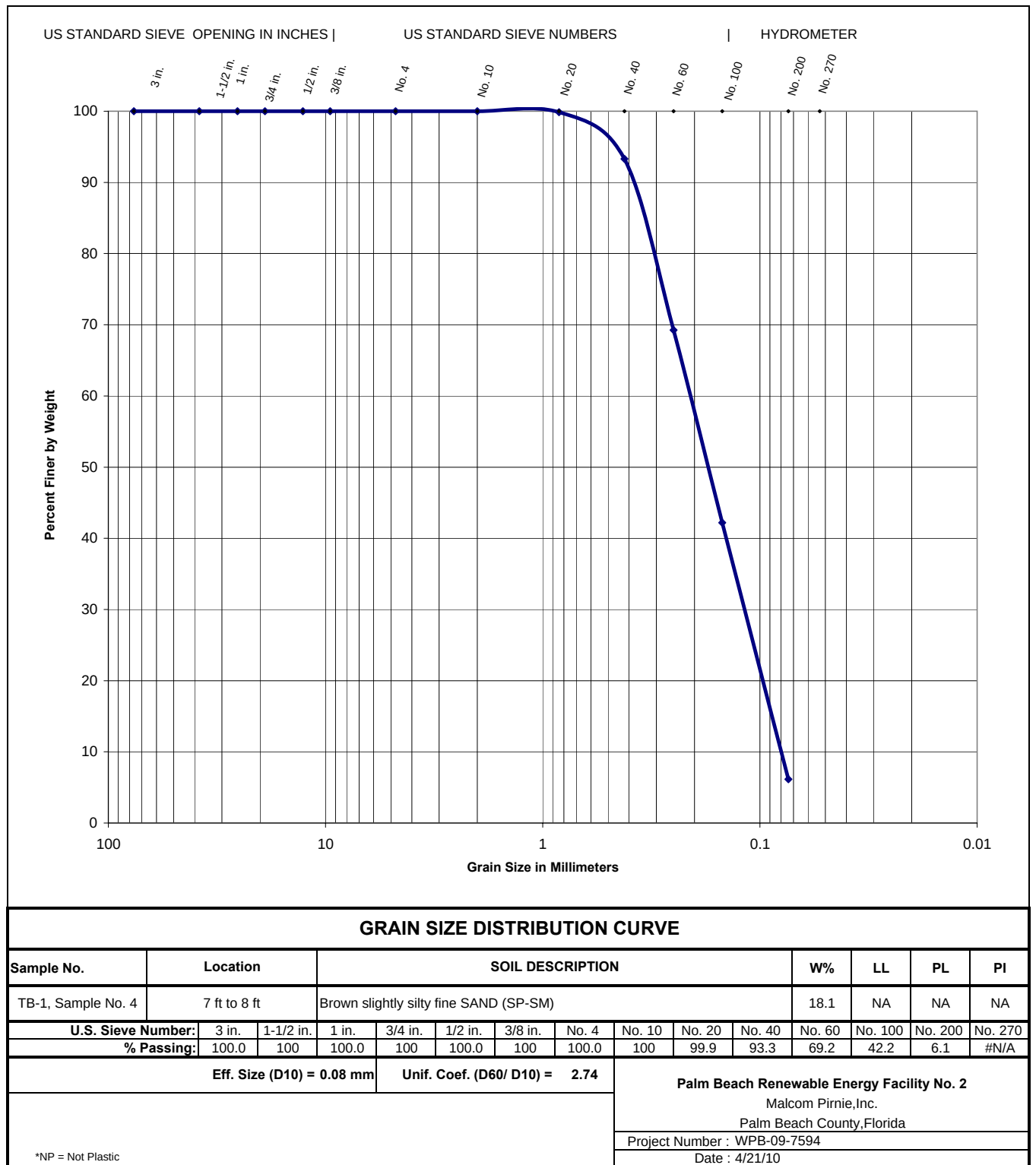
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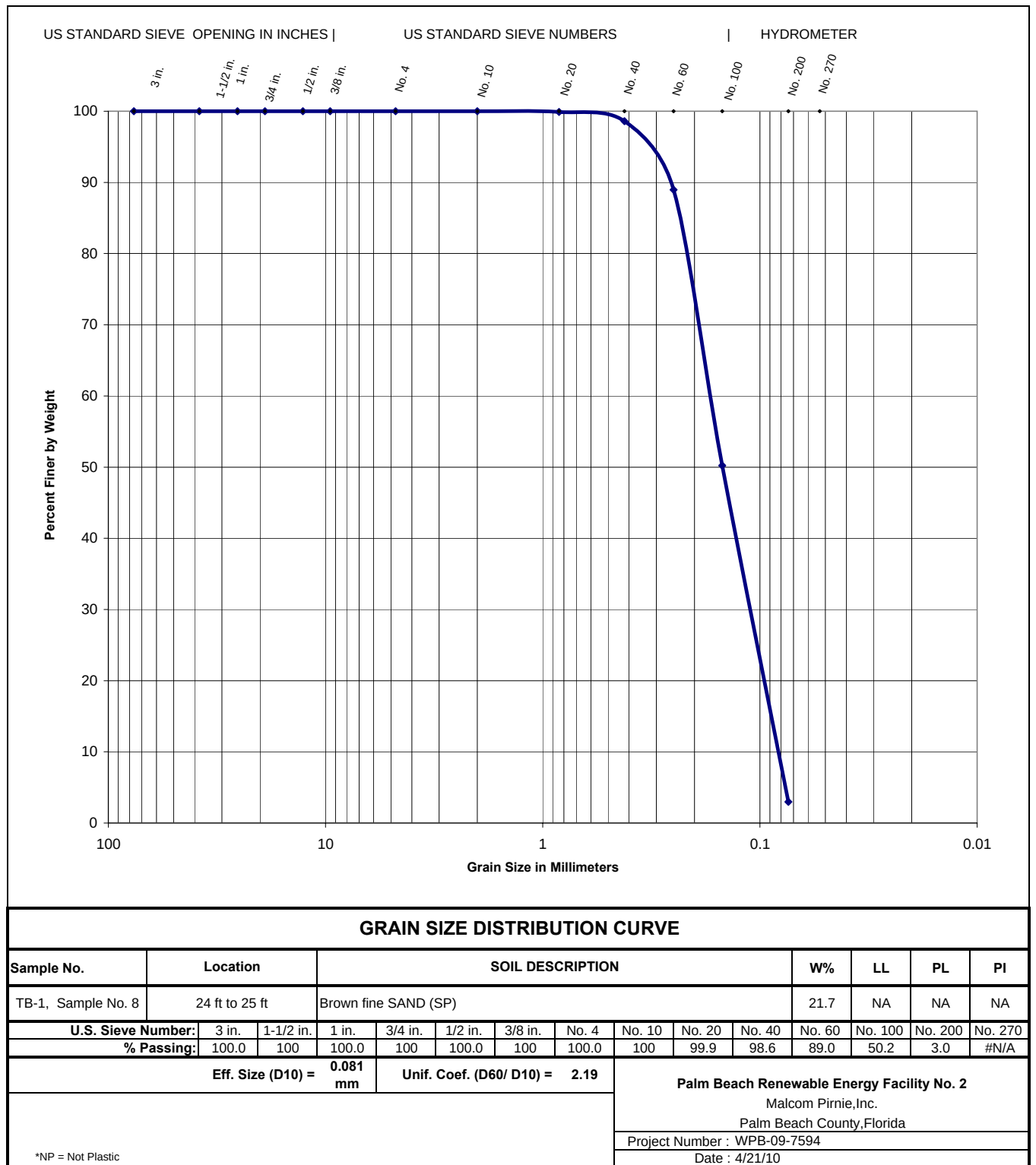
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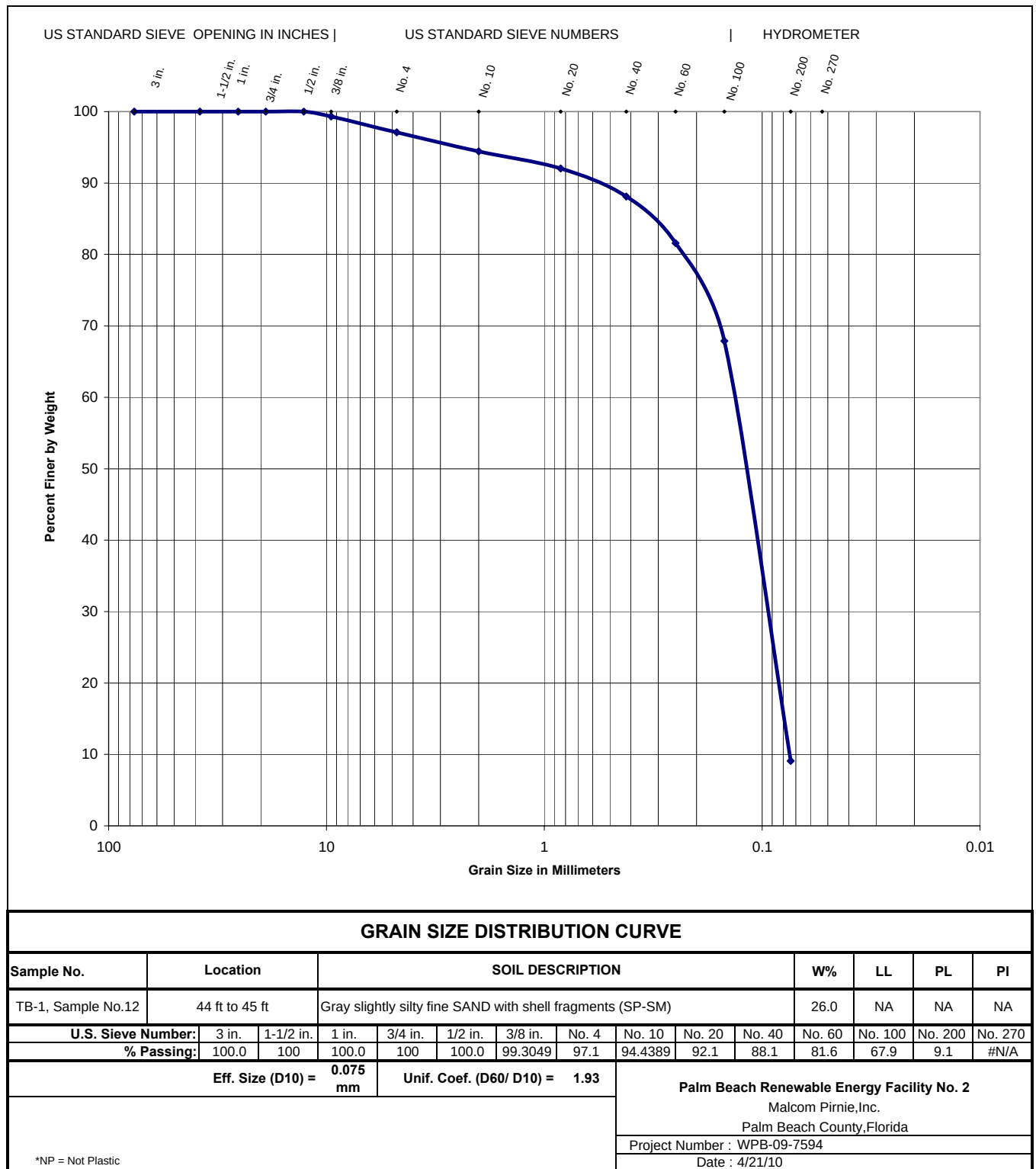
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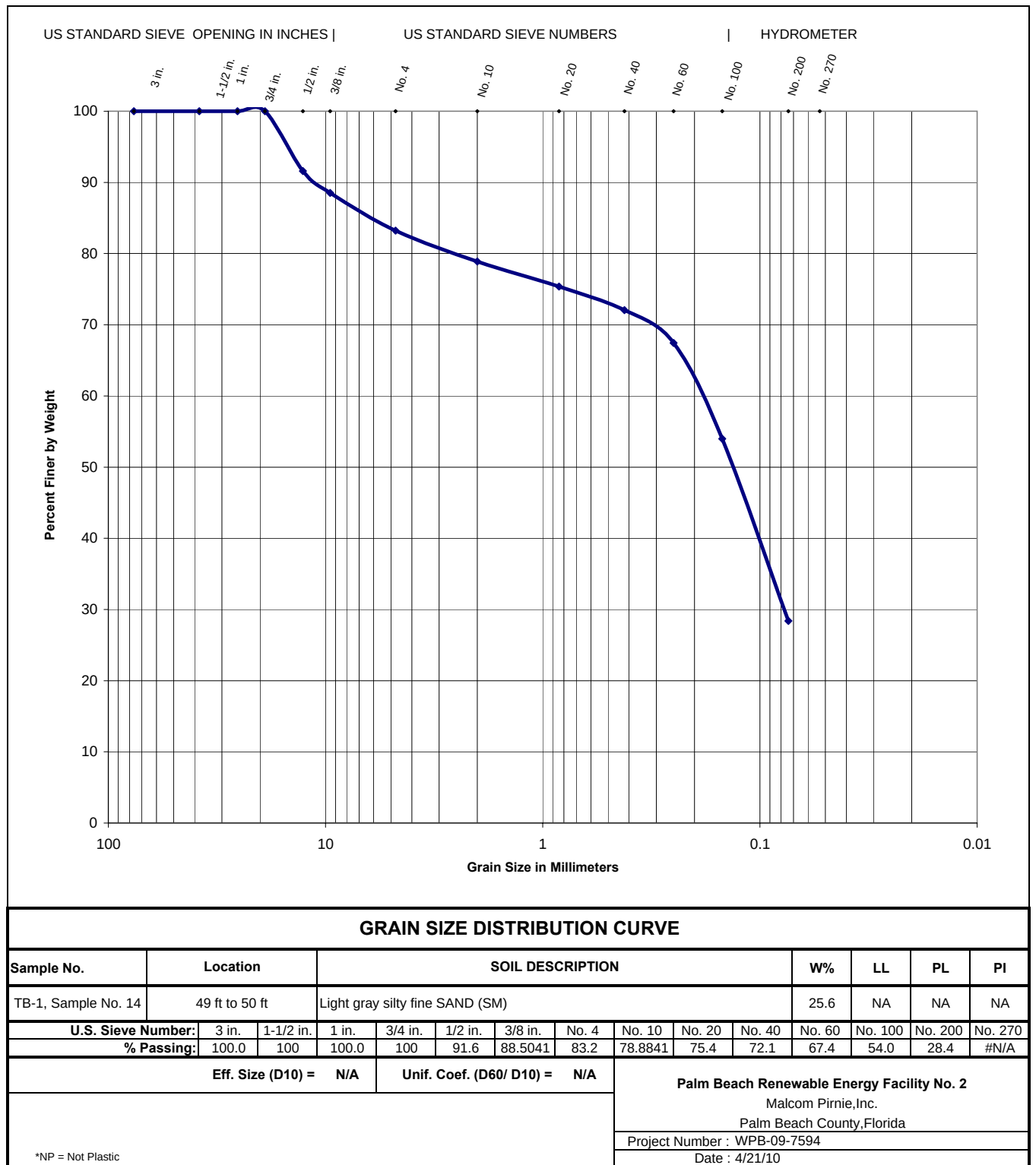
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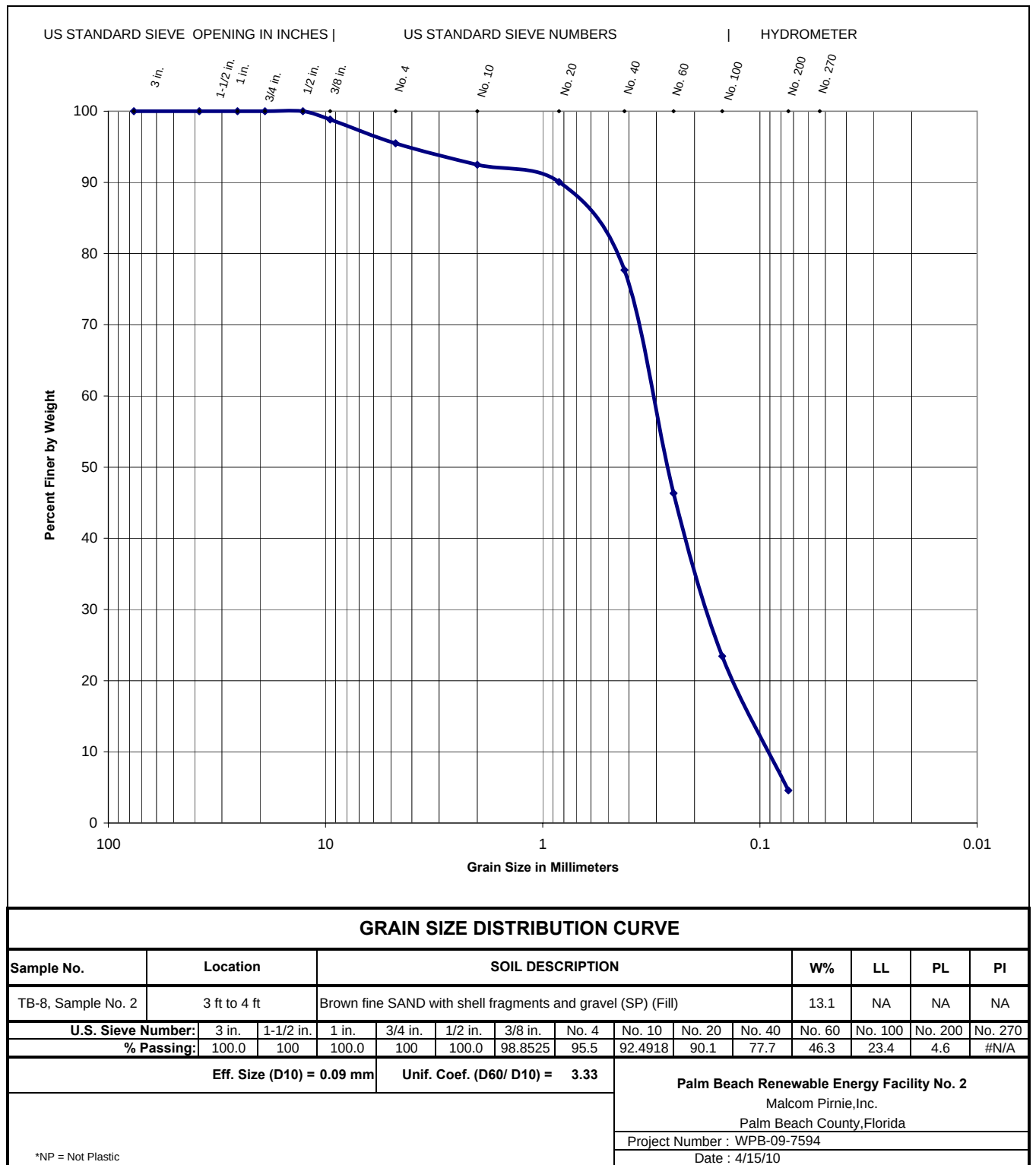
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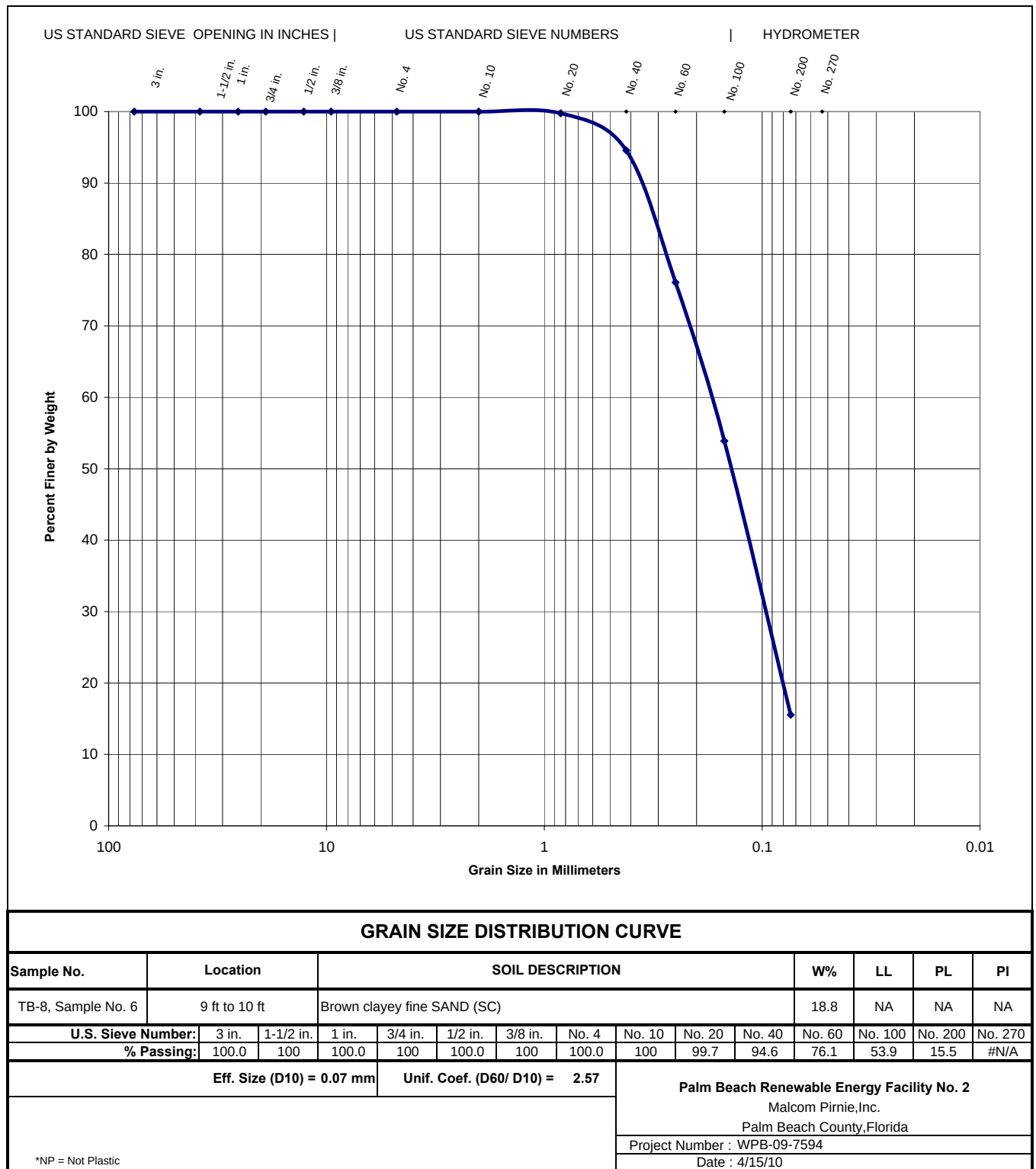
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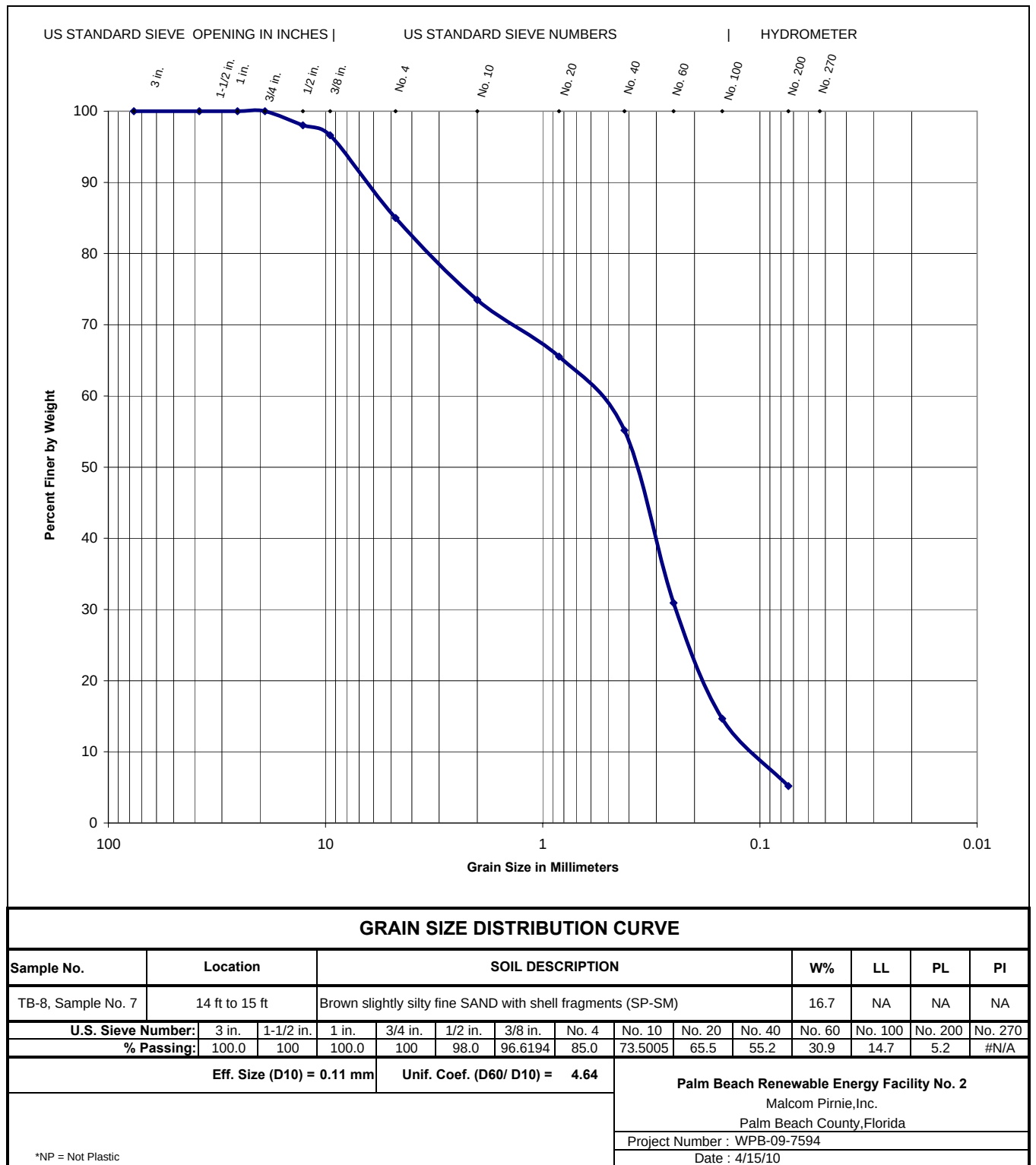
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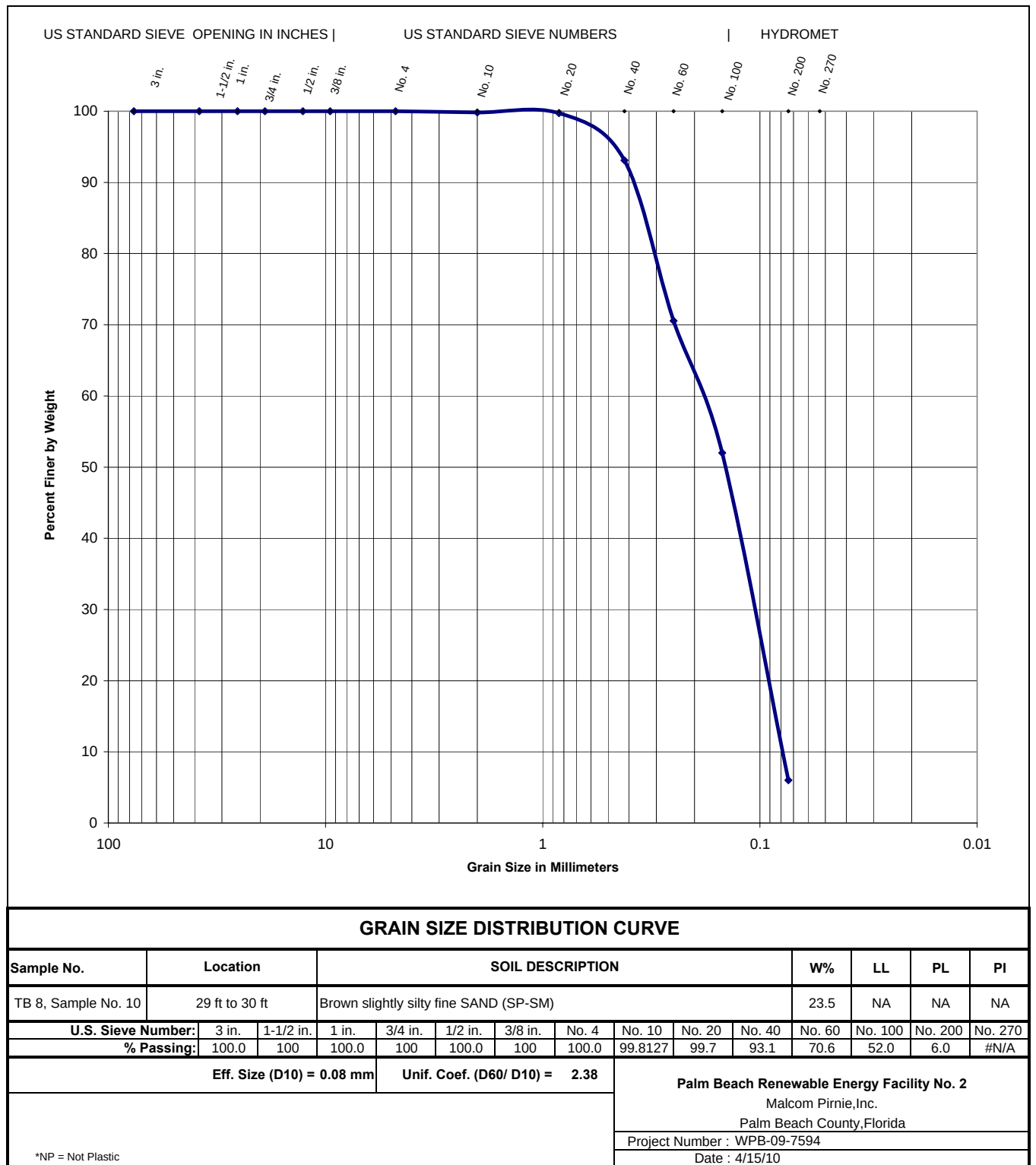
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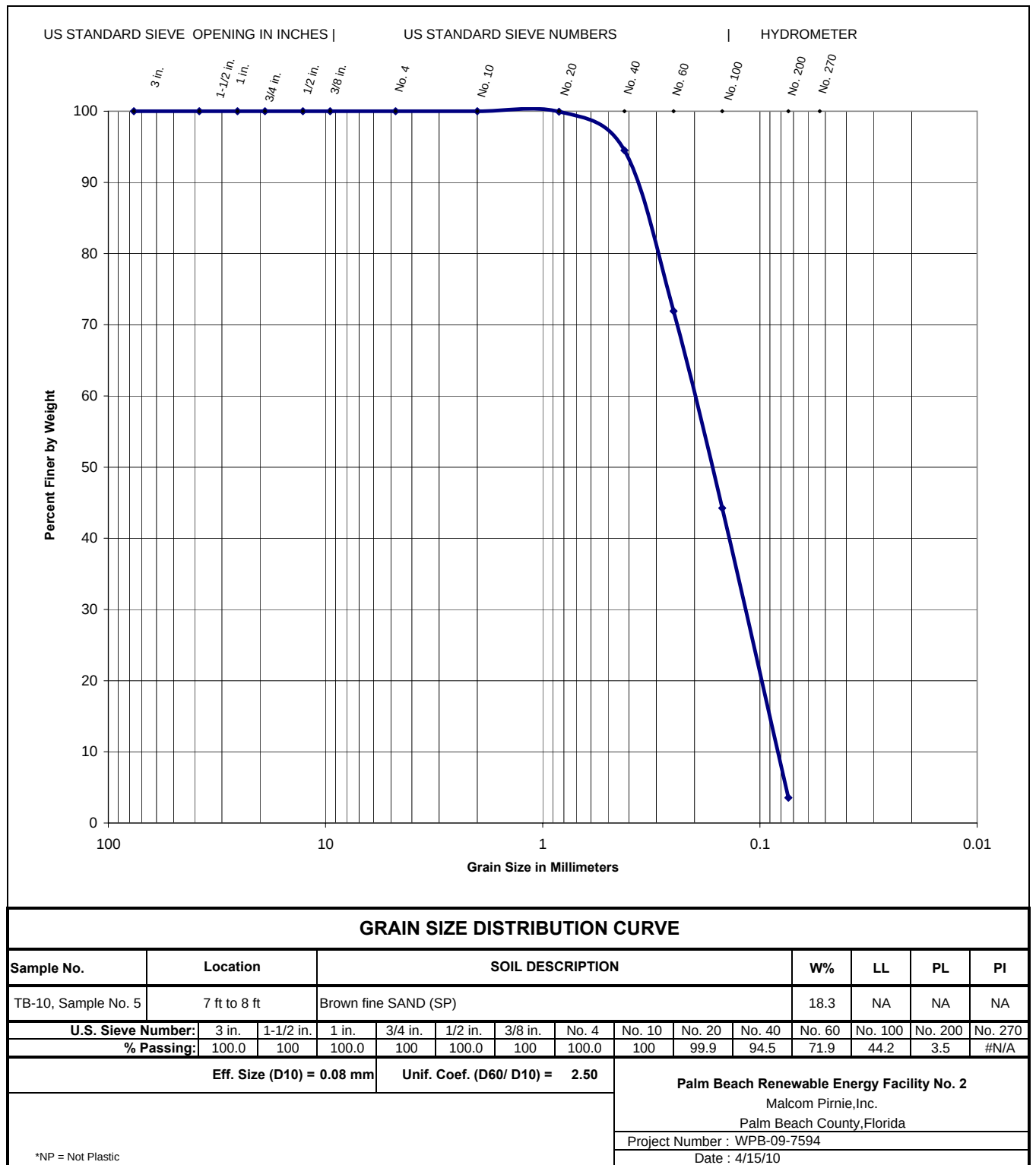
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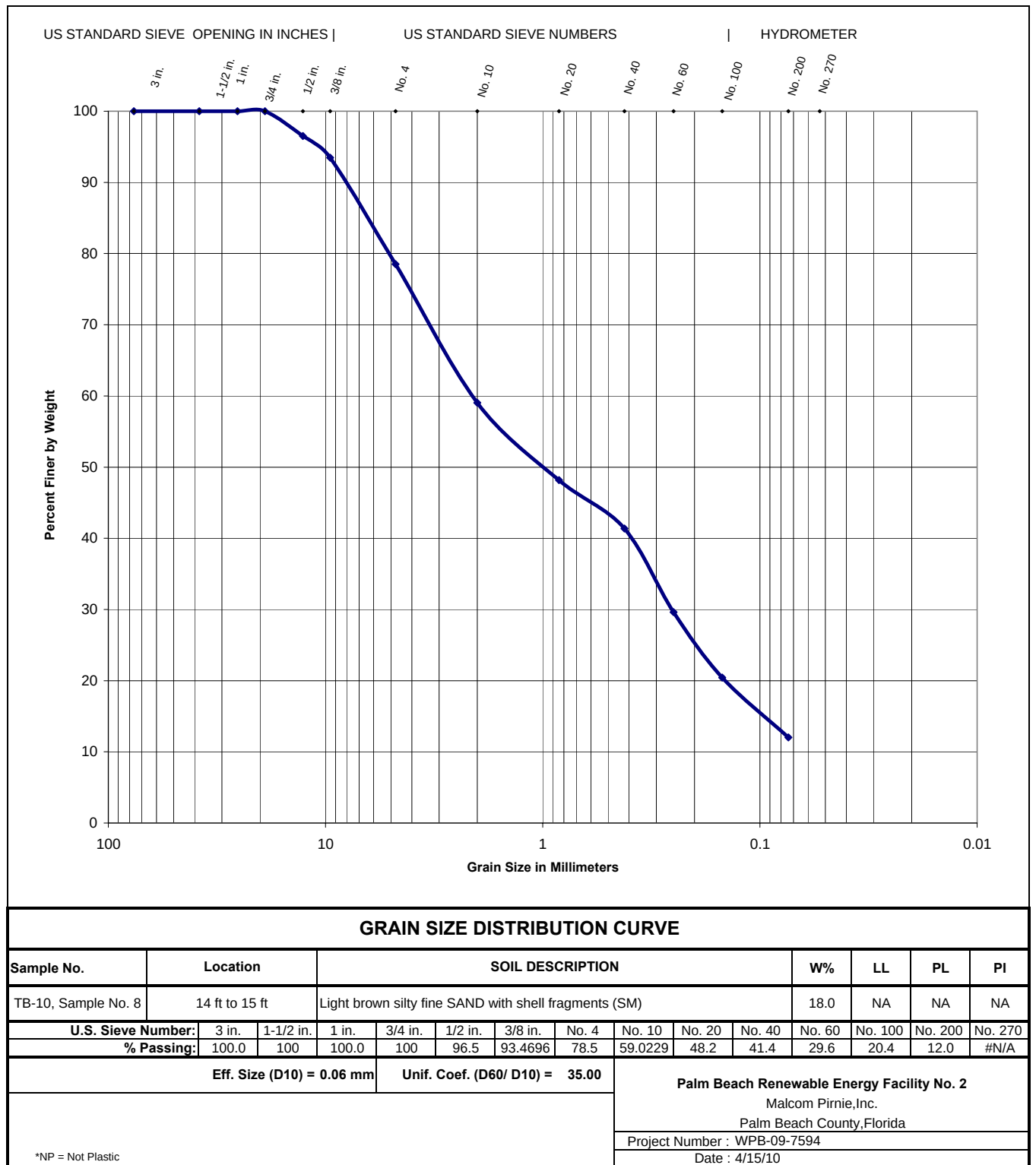
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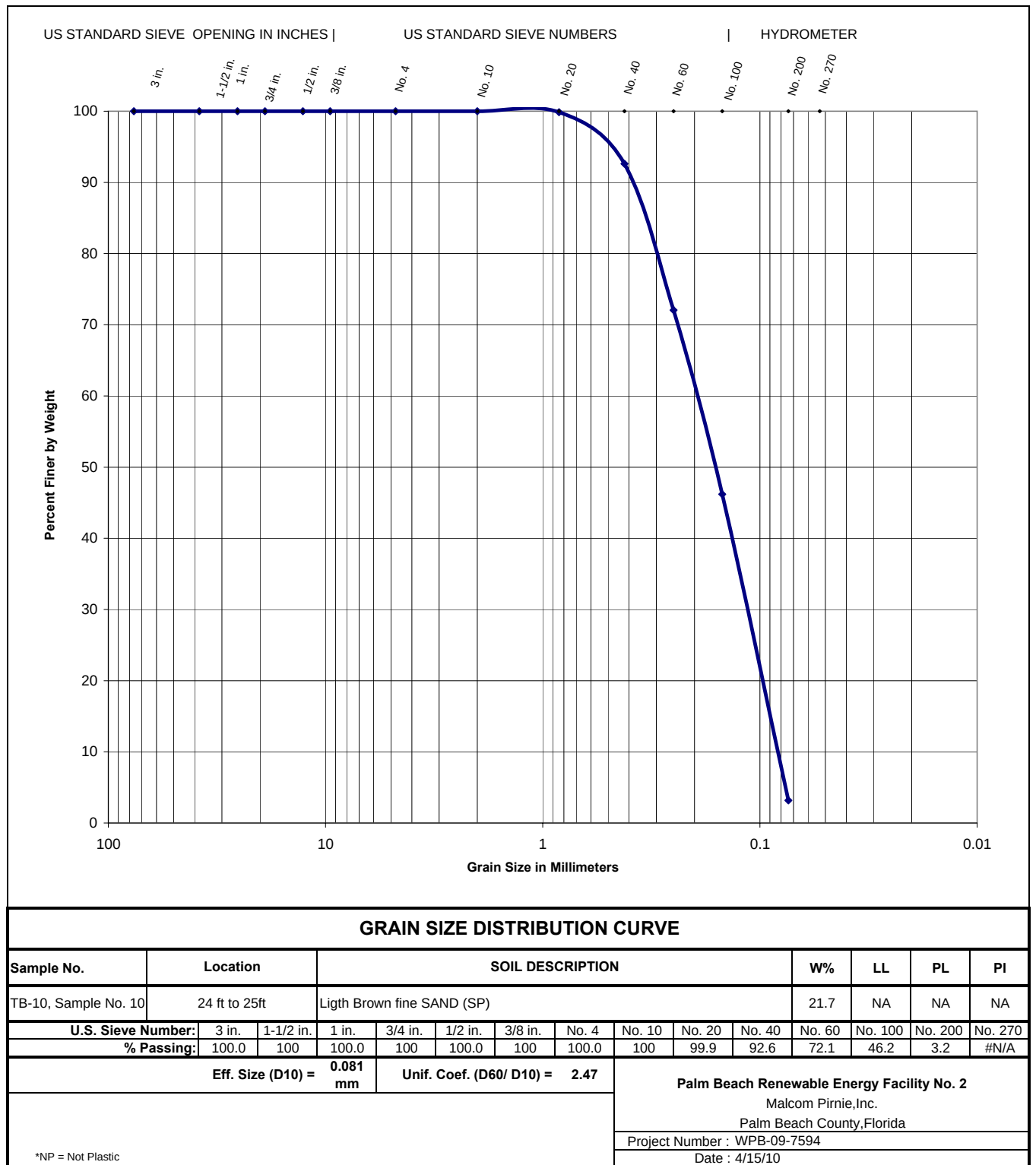
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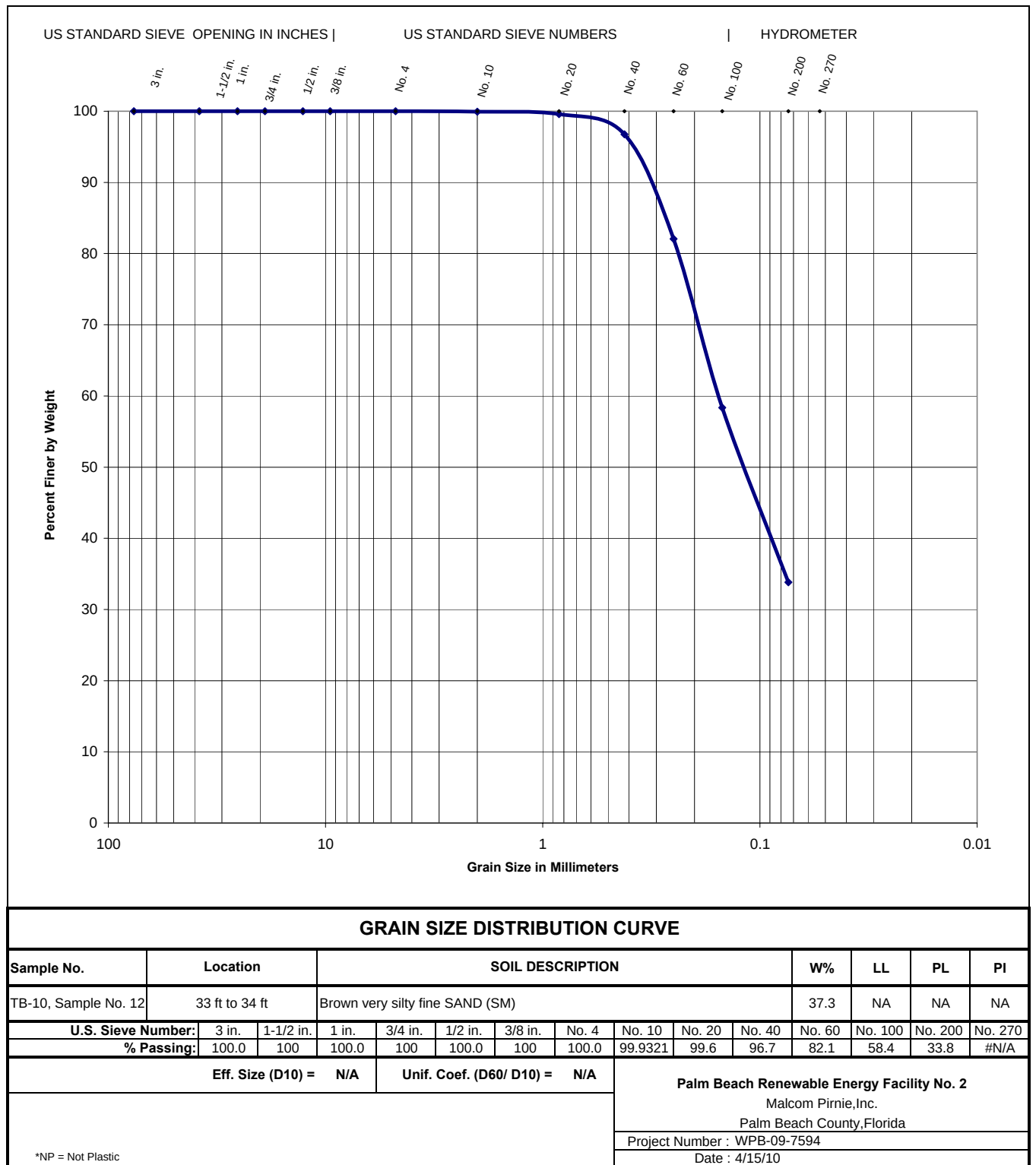
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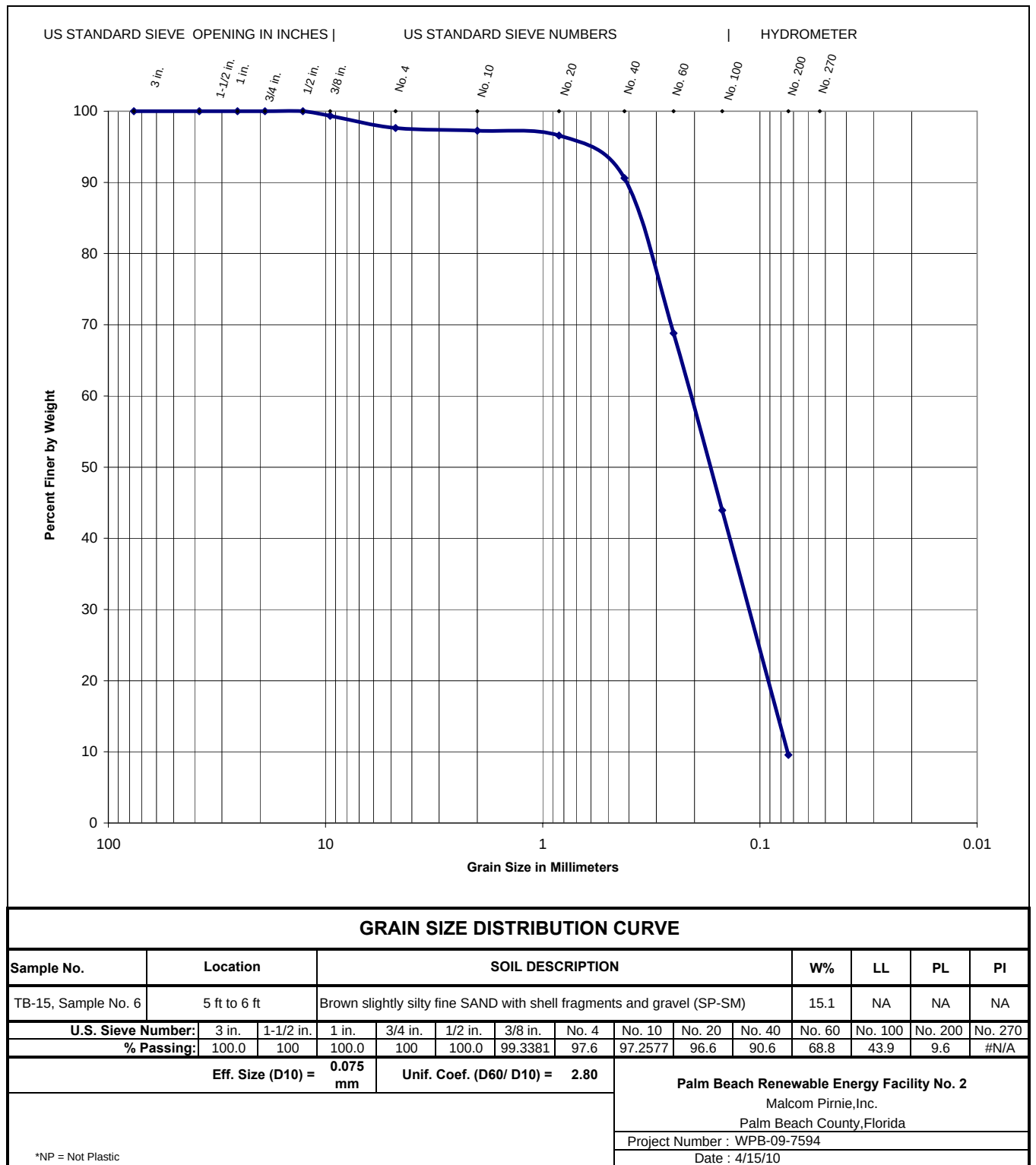
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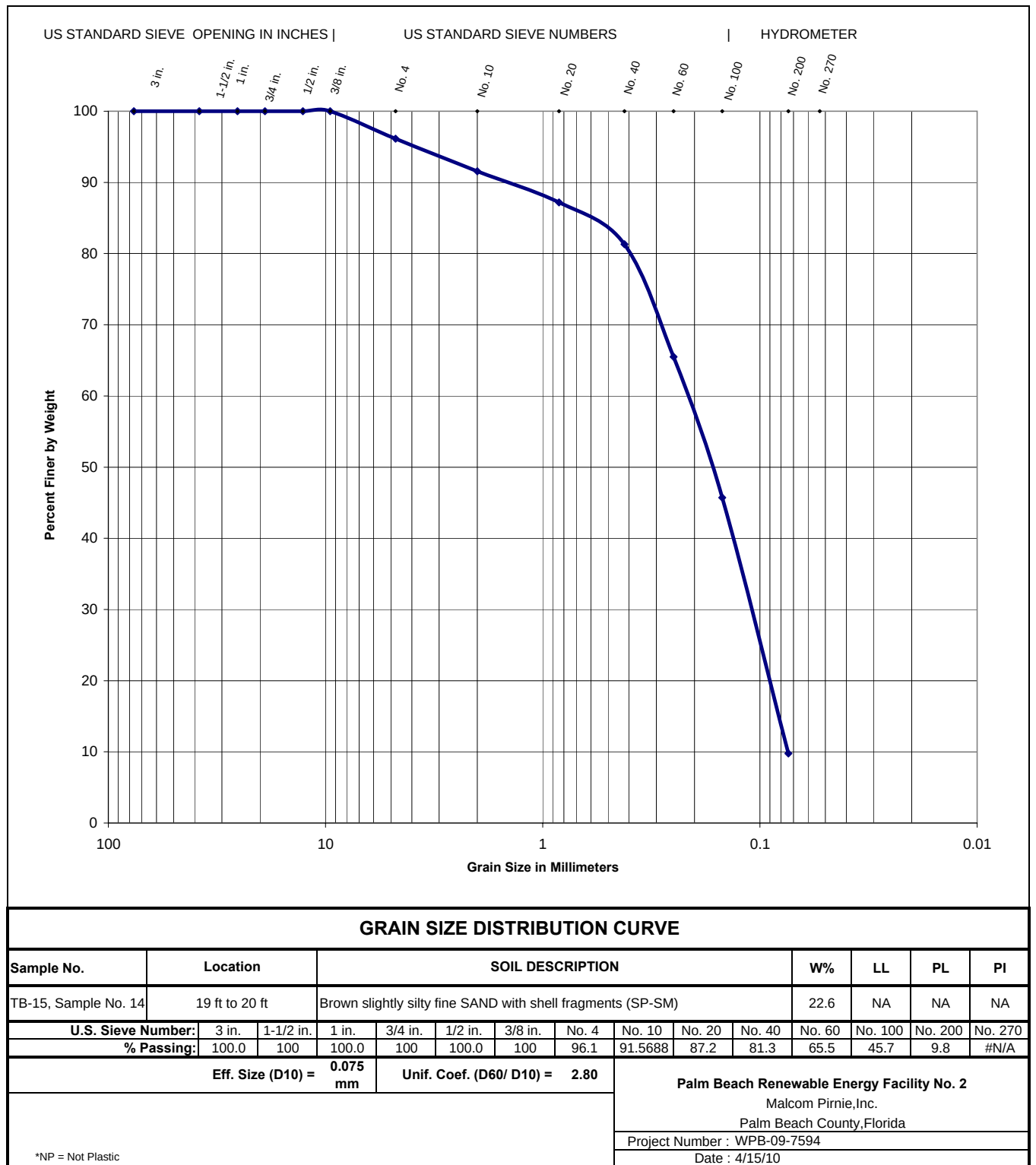
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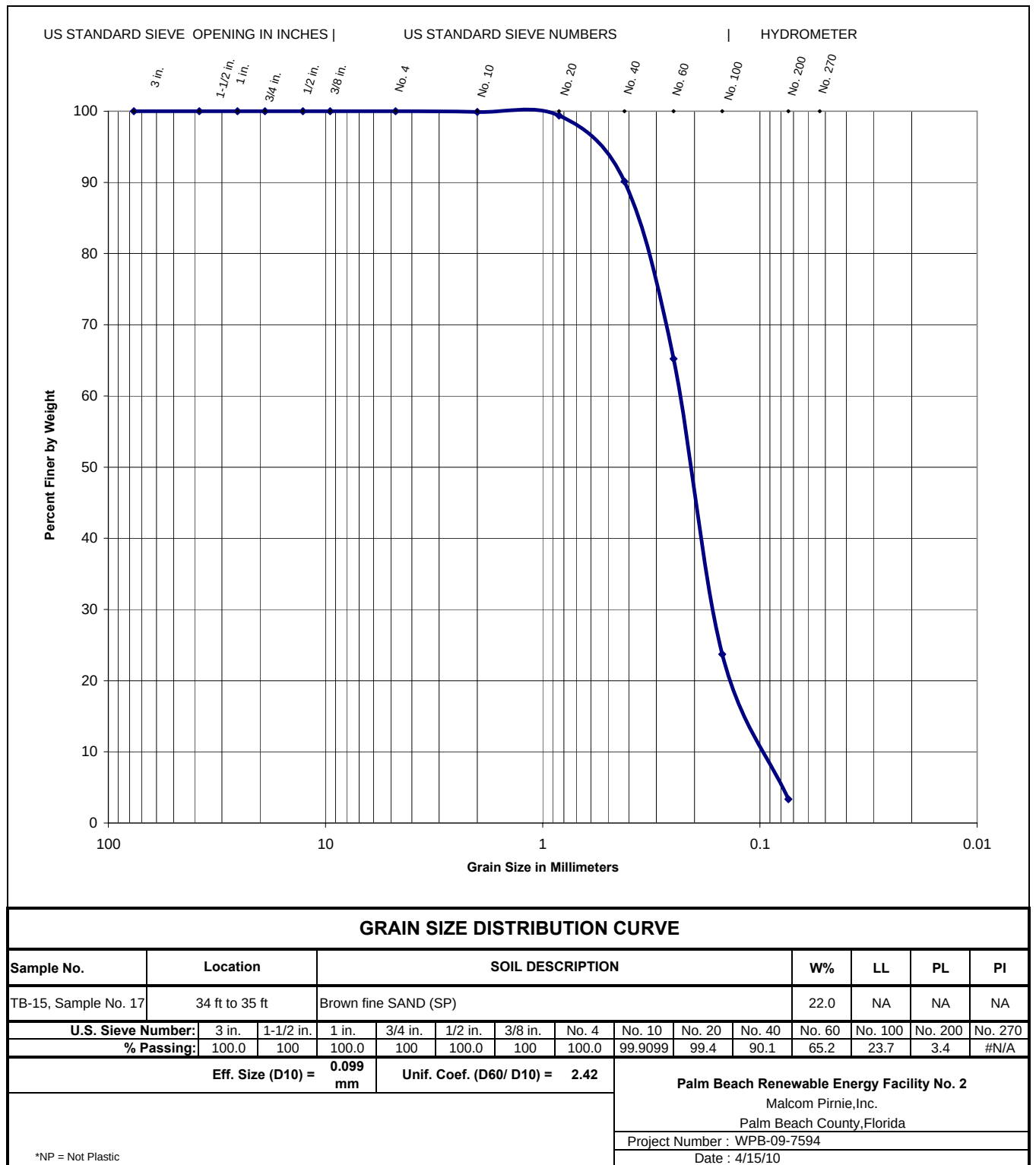
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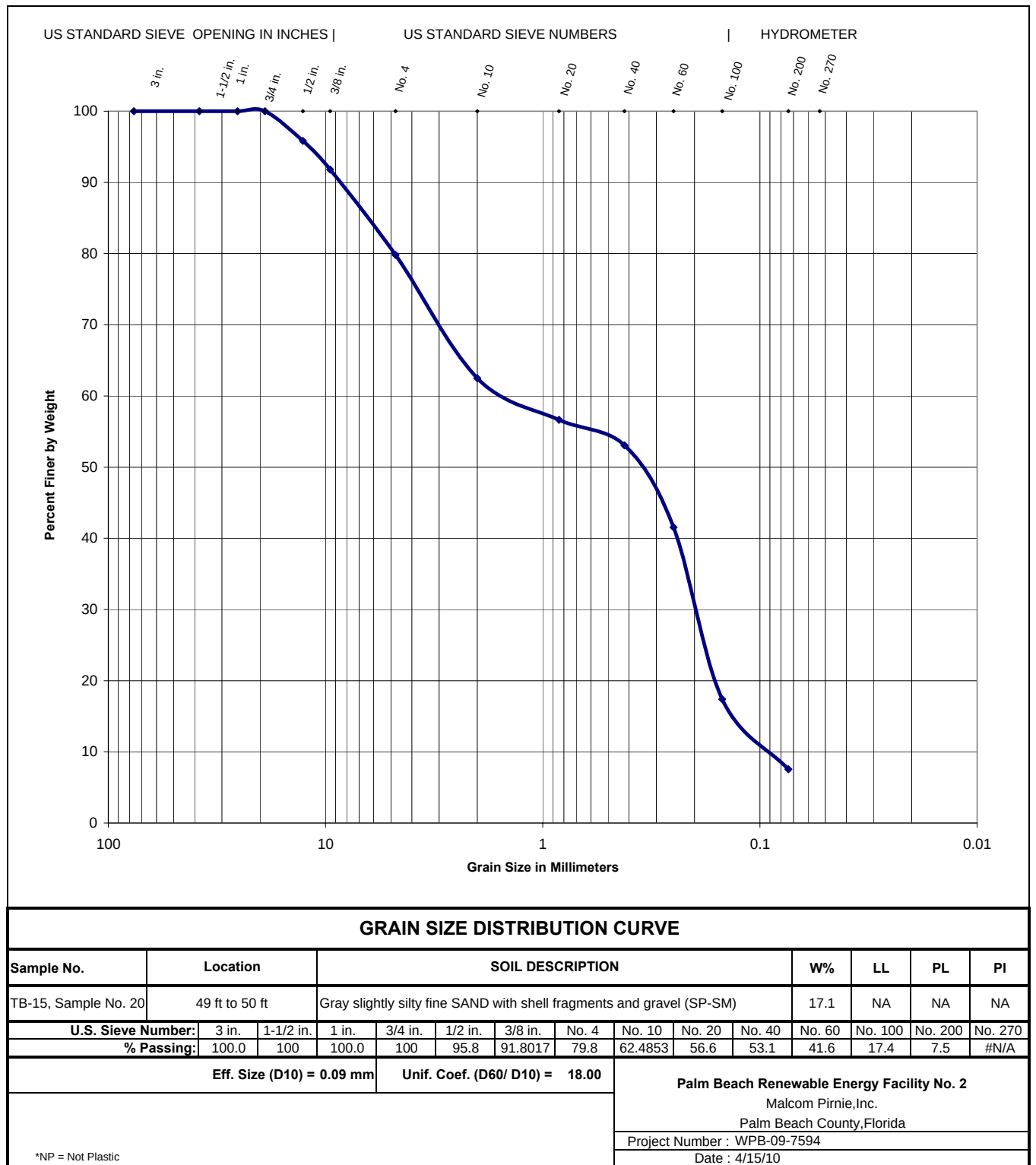
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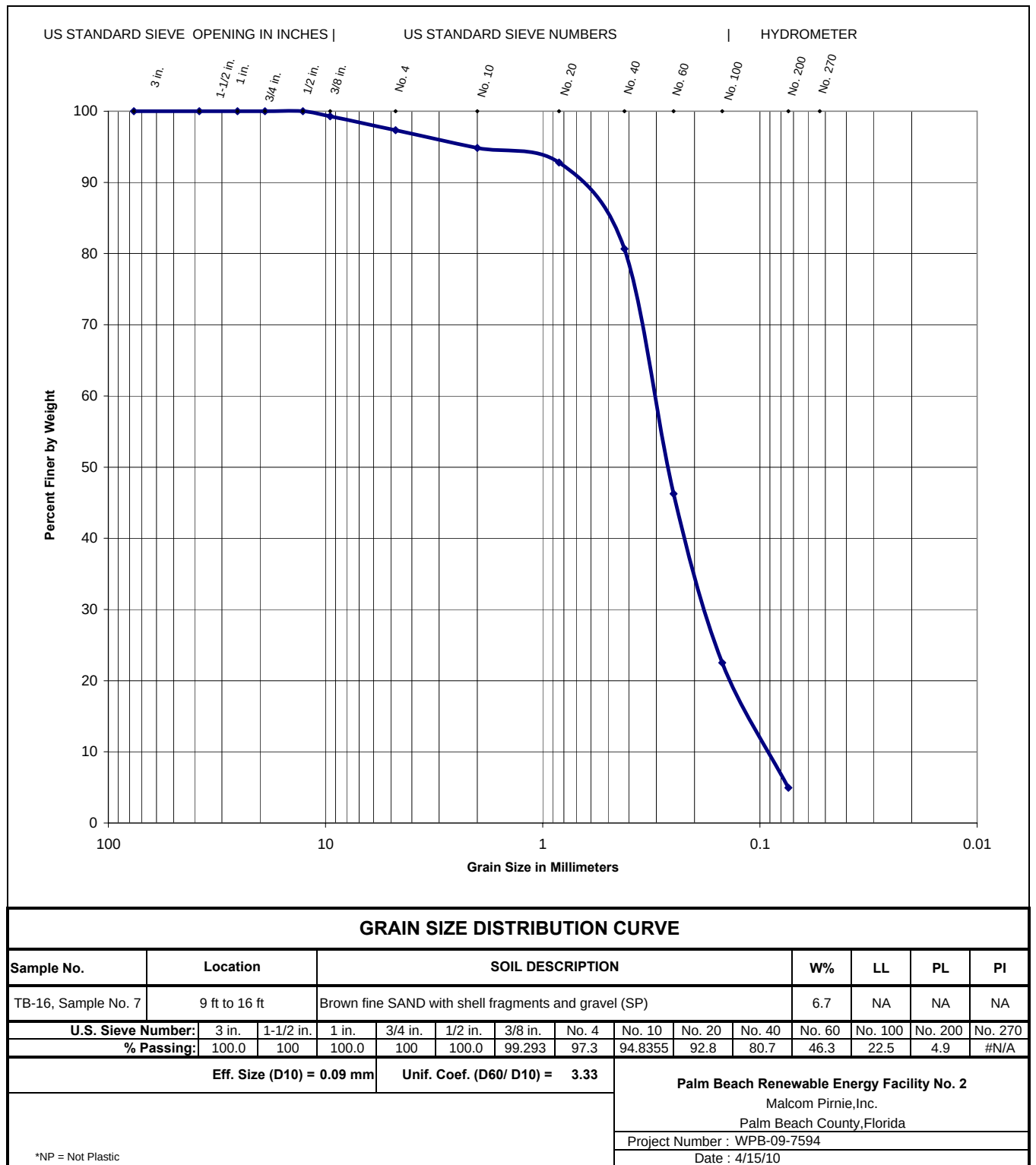
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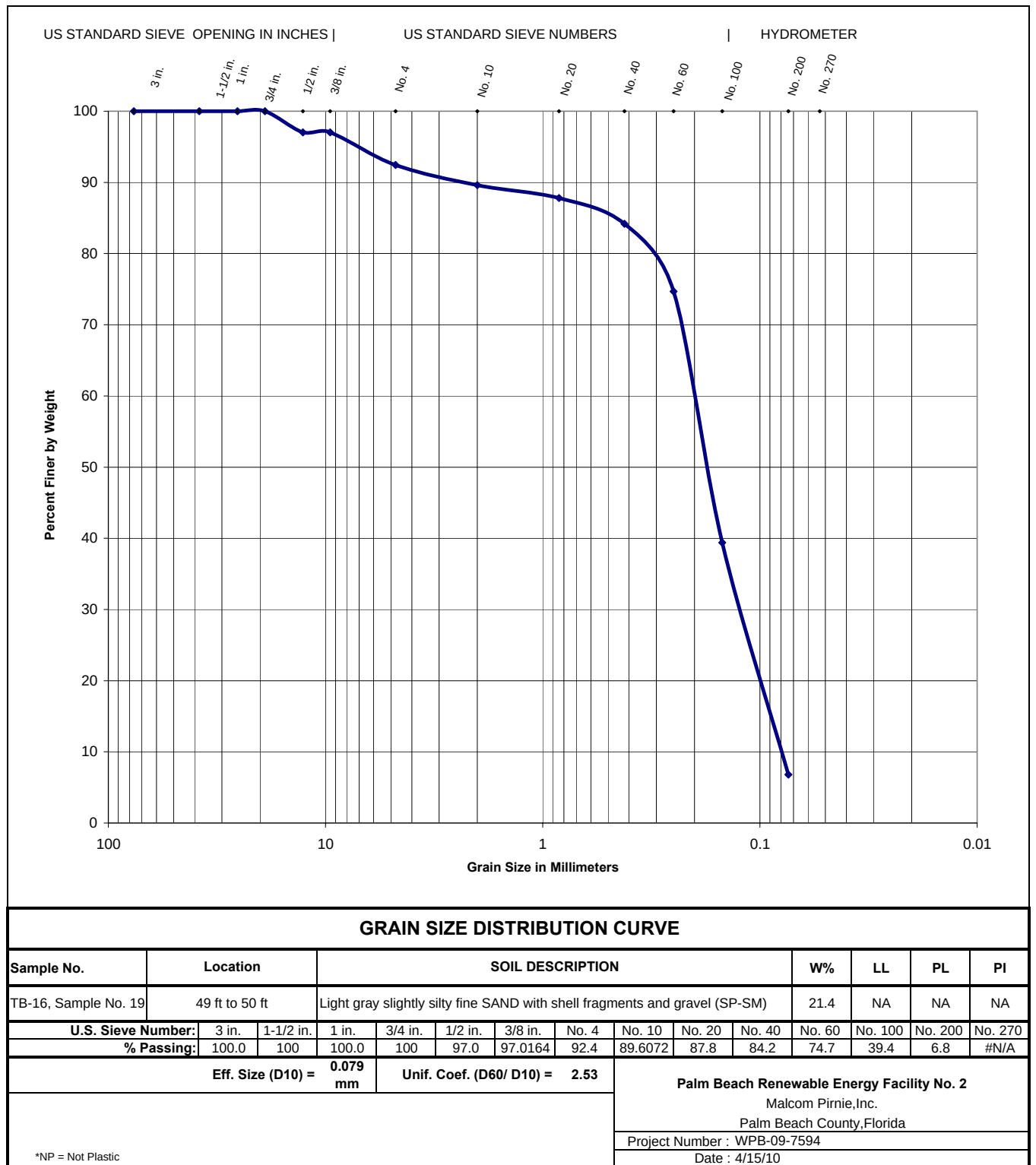
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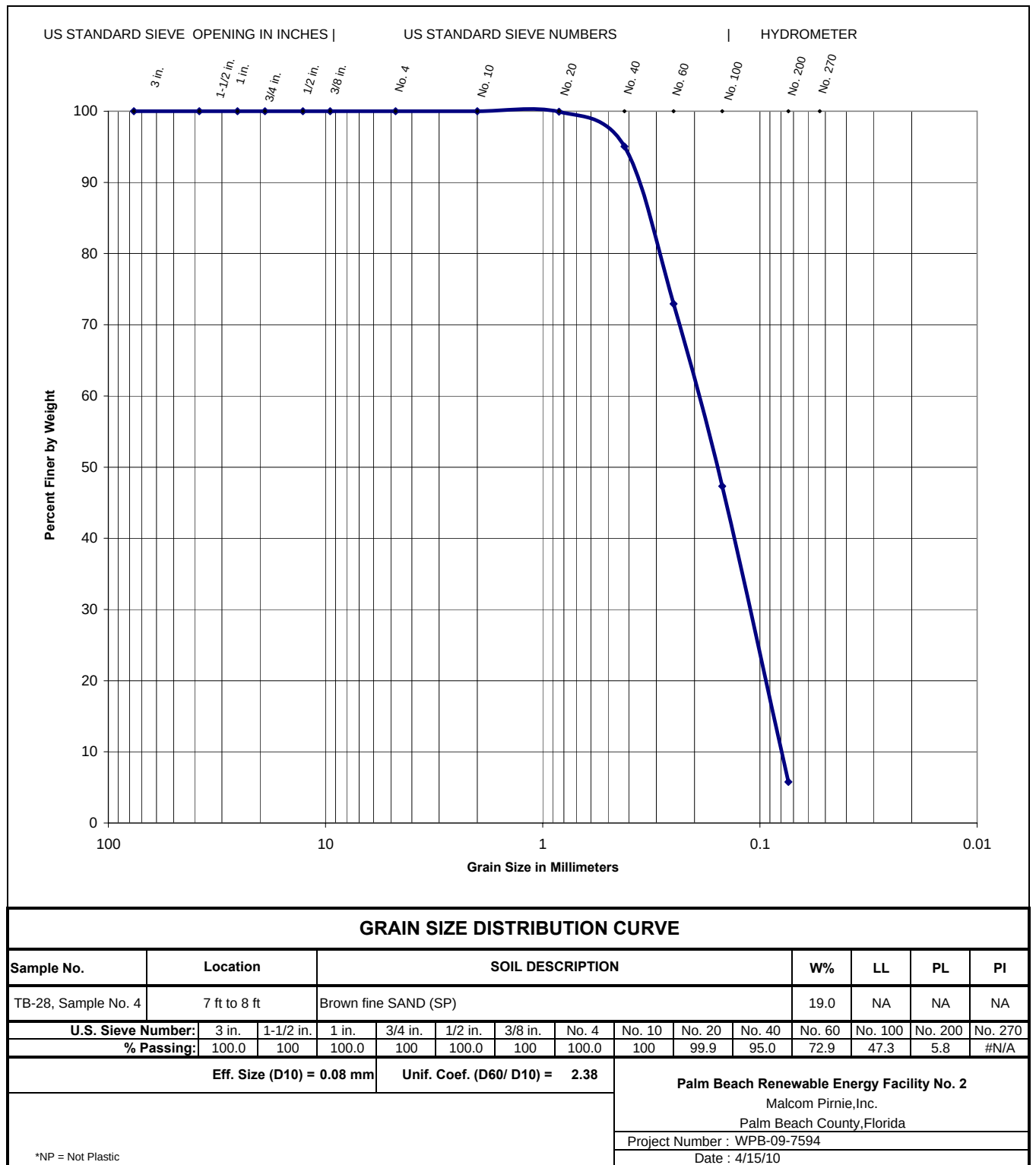
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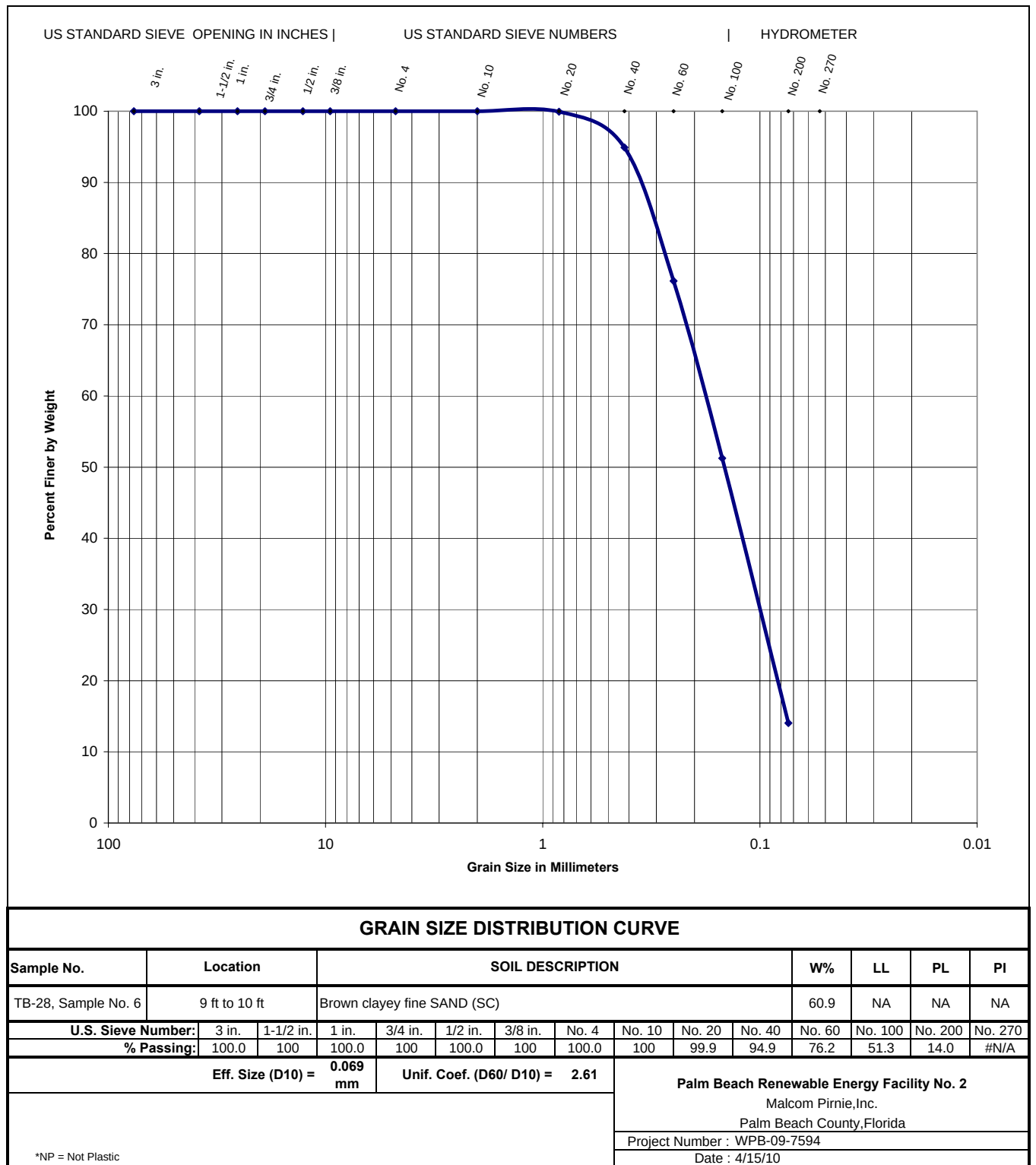
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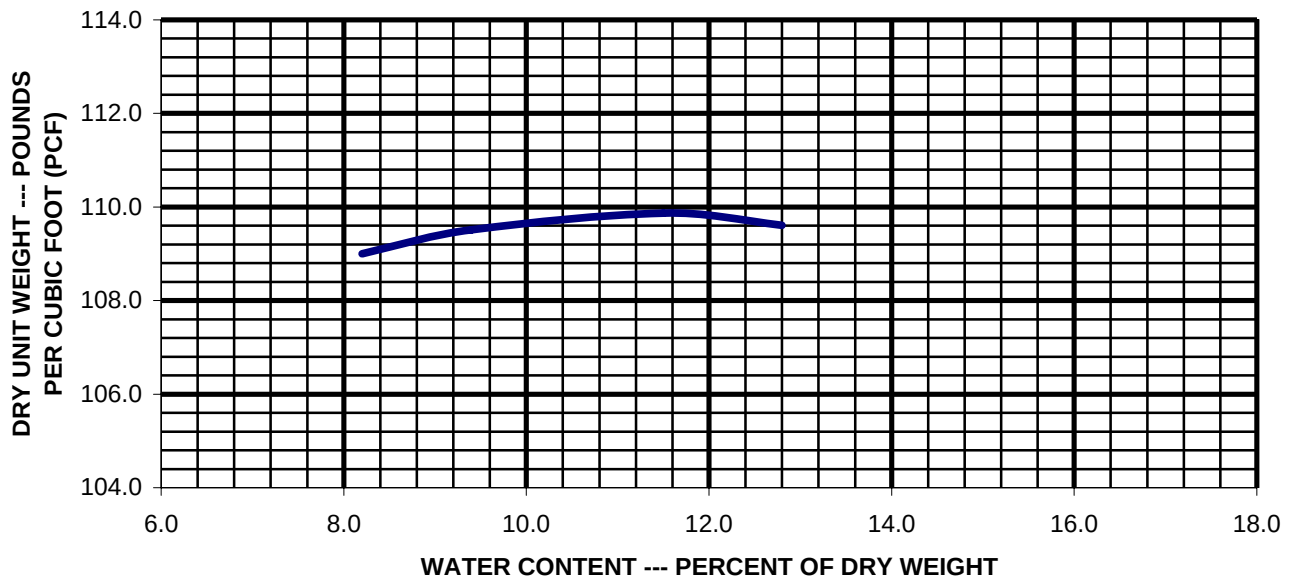
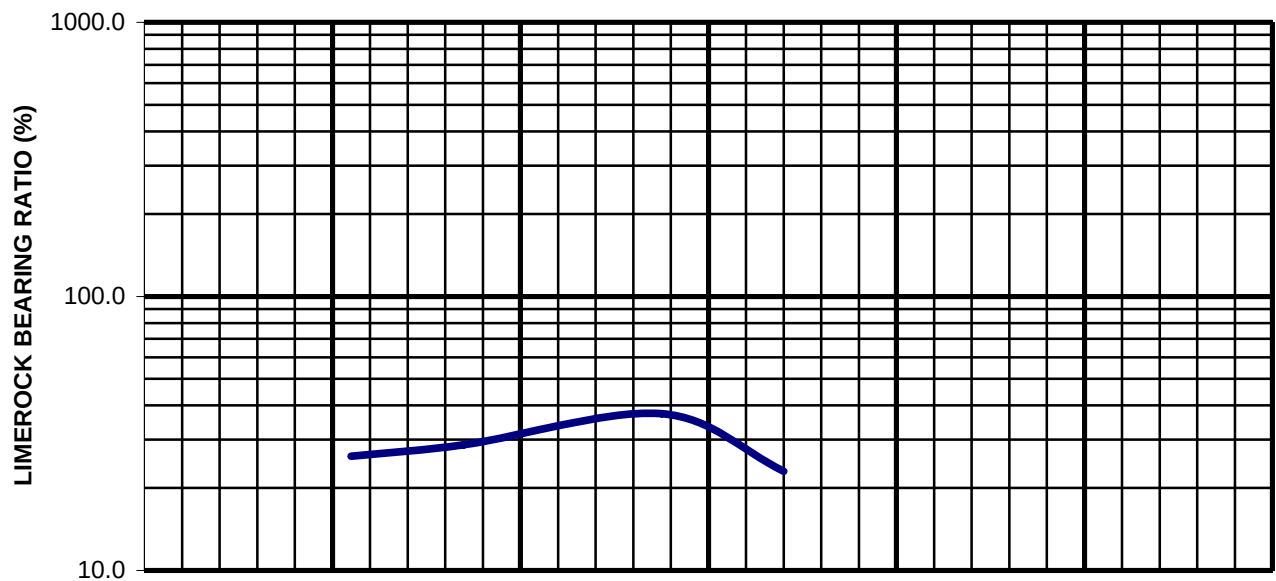
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LIMEROCK BEARING RATIO TEST RESULTS



SUMMARY OF TEST RESULTS		POINT VALUES			
		LBR VALUES		PROCTOR VALUES	
MAXIMUM LBR VALUE (%):	37.3	8.2	26.1	8.2	109.0
MAXIMUM DRY DENSITY (pcf):	109.9	9.4	28.8	9.4	109.5
OPTIMUM WATER CONTENT (%):	11.5	11.5	37.3	11.5	109.9
		12.8	23.0	12.8	109.6
Sample No.: 10362 Location: Drill Pad @ Boring TB-4 Depth: 0 - 0.5 feet Material Descript.: Light brown fine to medium SAND with some fragmental limestone (SP) Project Requirement: N/A		LIMEROCK BEARING RATIO			
		Geotechnical Study			
		Palm Beach Renewable Energy Facility No. 2			
		Palm Beach County, Florida			
		DUNKELBERGER ENGINEERING & TESTING, INC.			
		Geotechnical-Materials-Testing/Inspection-Environmental			
Drawn By: AR		Date Tested: 4/12/2010	Project No.: WPB-09-7594		
Checked By: JD		Date: 4/20/2010	Sheet: C24		

PERCENT FINER BY WEIGHT

PERCENT COARSER BY WEIGHT

HYDROMETER

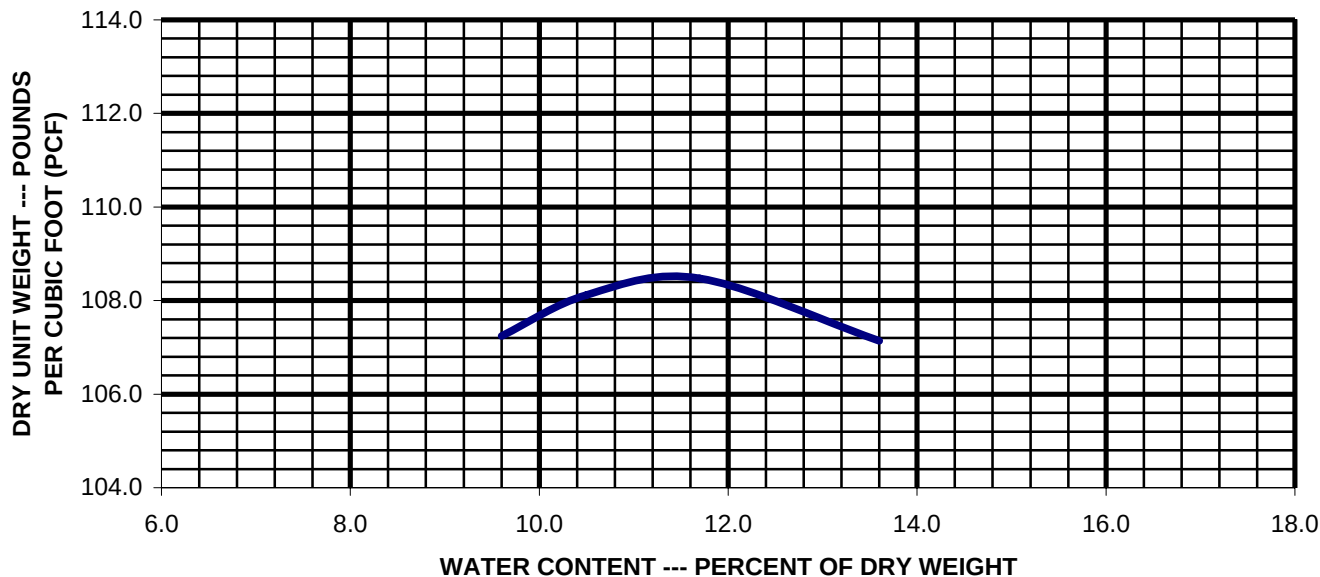
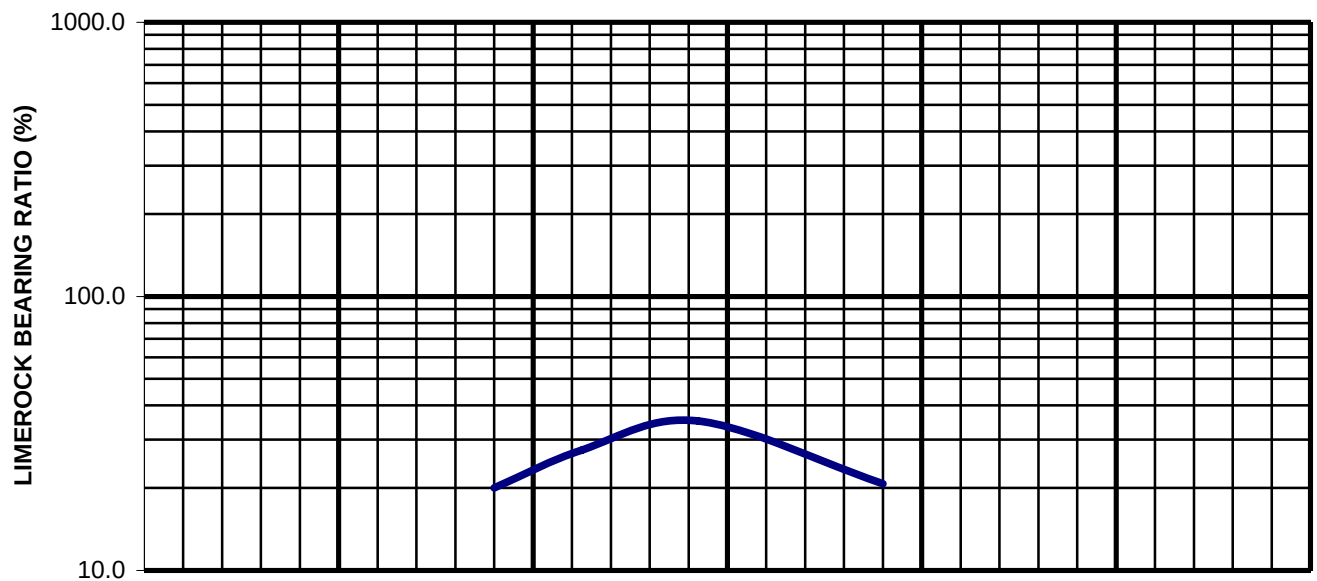
U.S. STANDARD SIEVE NUMBERS

U.S. STANDARD SIEVE OPENING IN INCHES

GRAIN SIZE in millimeters

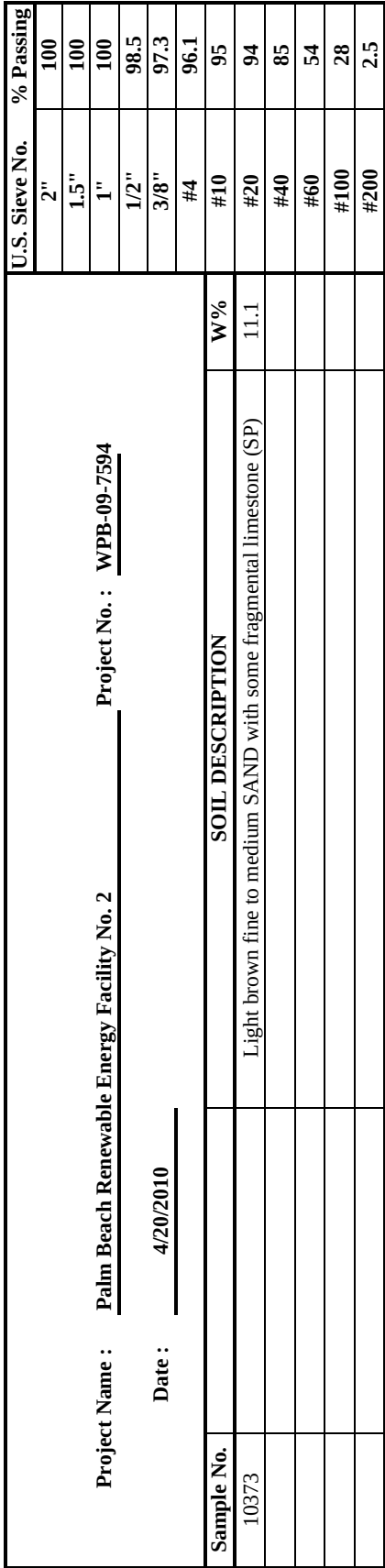
Grain Size (mm)	Percent Finer (%)	Sieve / Note
0.001	100	
0.002	100	
0.004	100	
0.0075	100	No. 20
0.015	100	No. 10
0.03	100	No. 5
0.06	100	No. 2.5
0.075	100	No. 200
0.15	100	No. 100
0.3	100	No. 50
0.6	100	No. 25
1.18	100	No. 12
2.0	100	No. 7.5
3.55	100	No. 4.75
6.0	100	No. 2.5
10.0	100	No. 1.5
15.0	100	No. 1.0
20.0	100	No. 0.75
30.0	100	No. 0.5
45.0	100	No. 0.35
75.0	100	No. 0.2
100.0	100	No. 0.15
200.0	100	No. 0.075
425.0	100	No. 0.0425
750.0	100	No. 0.02
1060.0	100	No. 0.015
2000.0	100	No. 0.0075
3000.0	100	No. 0.006
4250.0	100	No. 0.00425
6000.0	100	No. 0.003
10000.0	100	No. 0.001

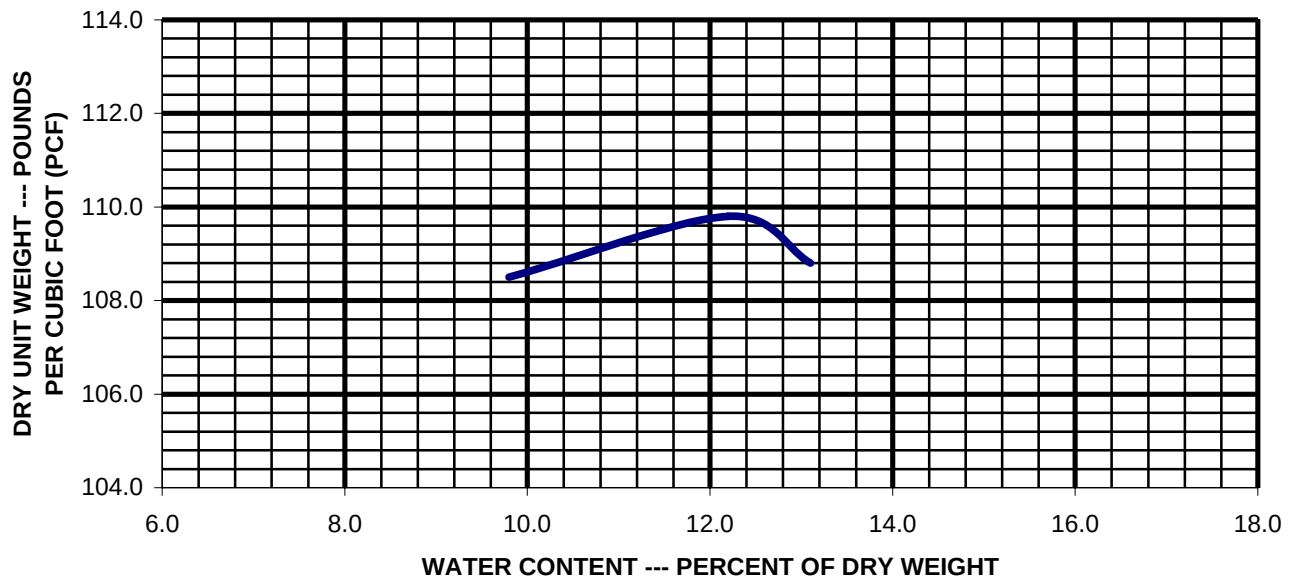
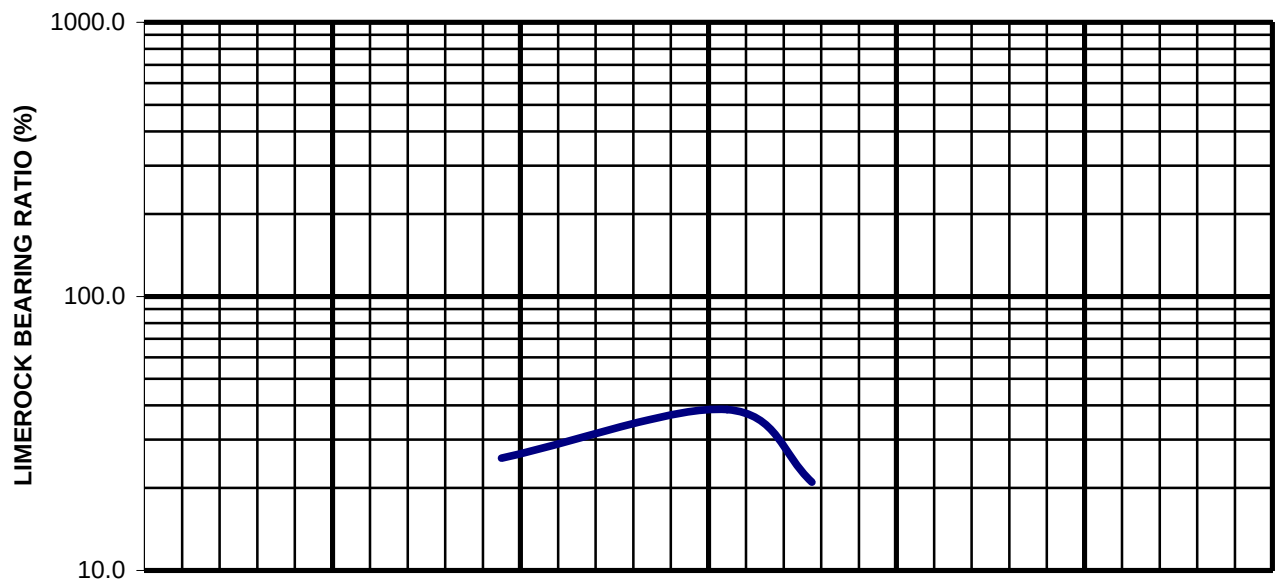
Project Name : Palm Beach Renewable Energy Facility No. 2				Project No. : WPB-09-7594	
Date : 4/20/2010					
Sample No.	SOIL DESCRIPTION			W%	
10362	Light brown fine to medium SAND with some fragmental limestone (SP)			1.1	
U.S. Sieve No.				% Passing	
2"				100	
1.5"				99.7	
1"				98.8	
1/2"				94.6	
3/8"				93.7	
#4				91.9	
#10				91	
#20				90	
#40				83	
#60				63	
#100				40	
#200				0.45	



SUMMARY OF TEST RESULTS		POINT VALUES			
		LBR VALUES		PROCTOR VALUES	
MAXIMUM LBR VALUE (%):	35.1	9.6	20.0	9.6	107.2
MAXIMUM DRY DENSITY (pcf):	108.5	10.5	27.5	10.5	108.1
OPTIMUM WATER CONTENT (%):	11.7	11.7	35.1	11.7	108.5
		13.6	20.7	13.6	107.1
Sample No.: 10373 Location: Toe Of Slope, 20' South Of TB-1 Depth: 0 - 0.5 feet Material Descript.: Light brown fine to medium SAND with some fragmental limestone (SP) Project Requirement: N/A		LIMEROCK BEARING RATIO			
		Geotechnical Study			
		Palm Beach Renewable Energy Facility No. 2			
		Palm Beach County, Florida			
		DUNKELBERGER ENGINEERING & TESTING, INC.			
		Geotechnical-Materials-Testing/Inspection-Environmental			
Drawn By: AR		Date Tested: 4/18/2010		Project No.: WPB-09-7594	
Checked By: JD		Date: 4/20/2010		Sheet: C26	

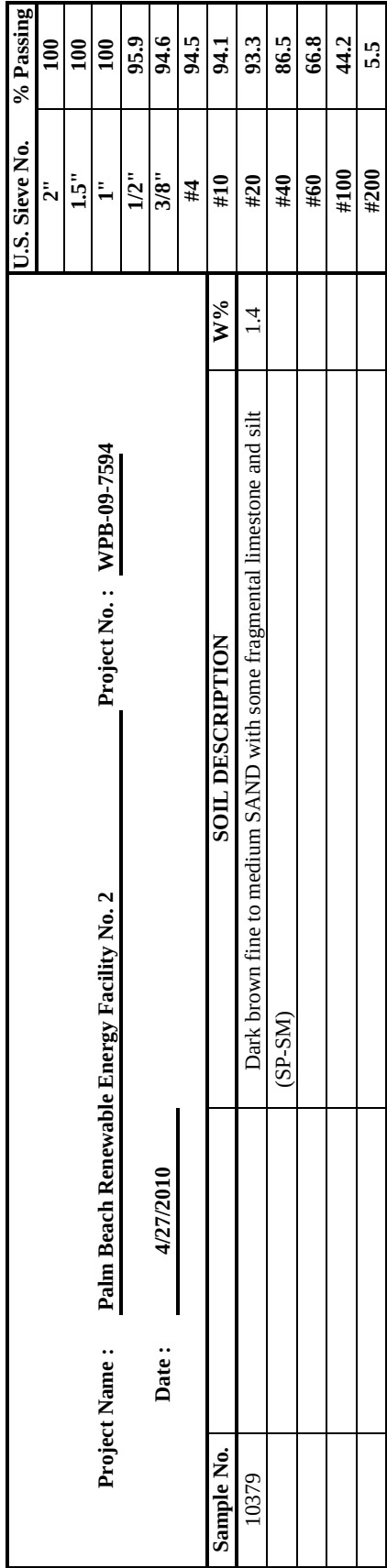
Geotechnical-Materials Testing-Inspection-Environmental

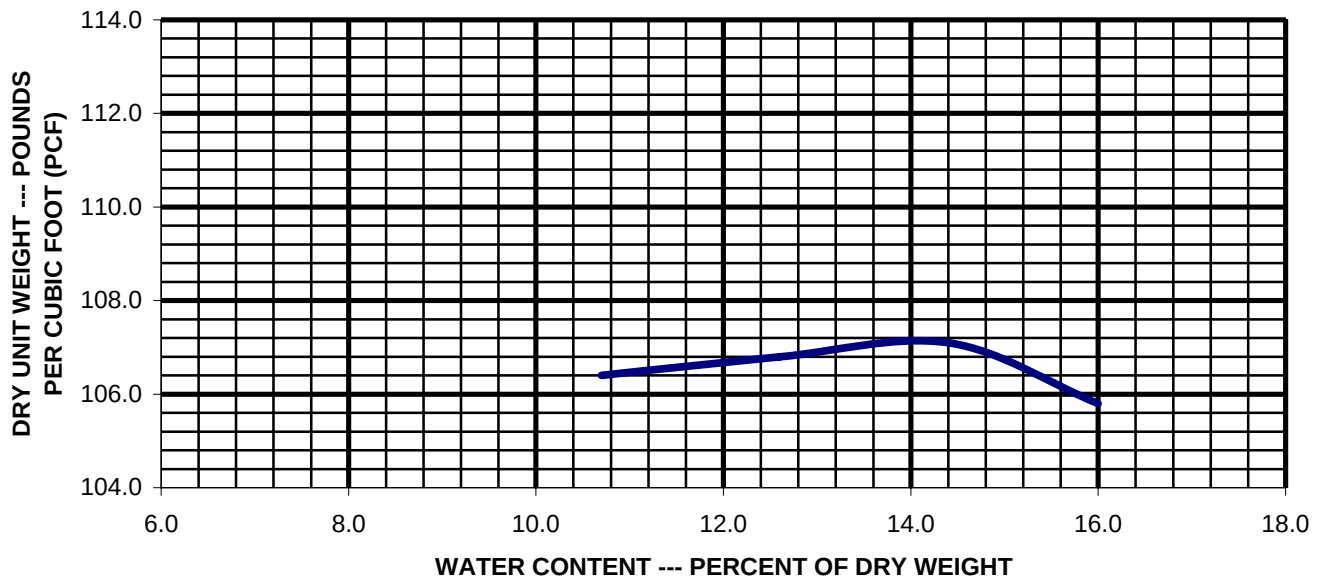
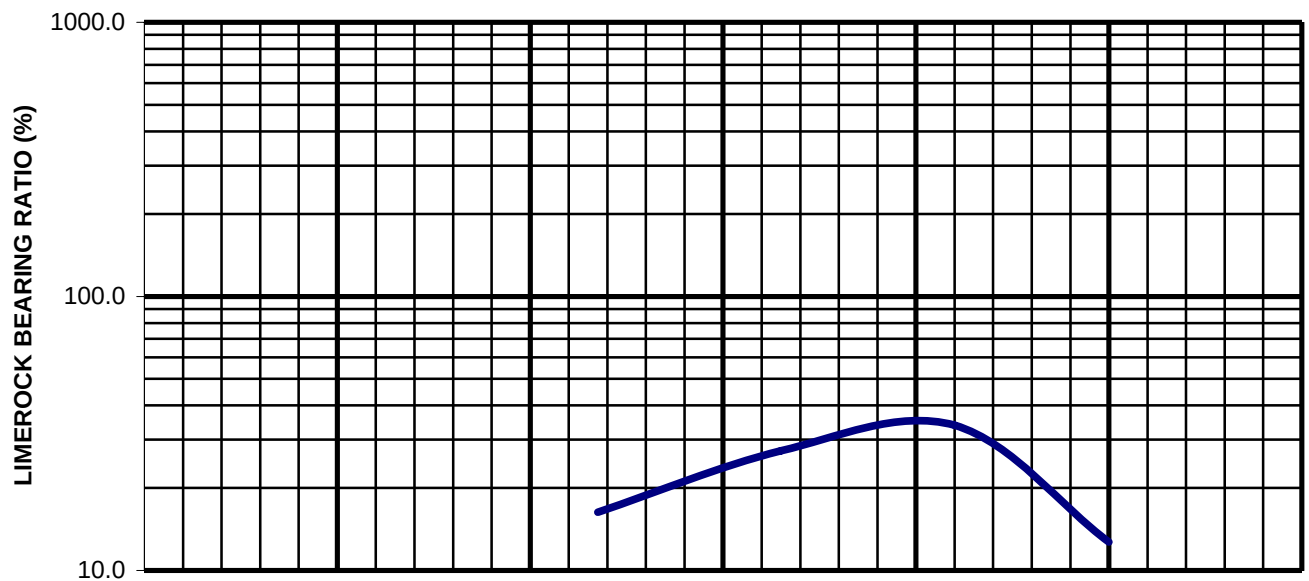




SUMMARY OF TEST RESULTS		POINT VALUES			
MAXIMUM LBR VALUE (%):	38.7	LBR VALUES		PROCTOR VALUES	
MAXIMUM DRY DENSITY (pcf):	109.8	9.8	25.7	9.8	108.5
OPTIMUM WATER CONTENT (%):	12.2	12.2	38.7	12.2	109.8
		13.1	21.0	13.1	108.8
Sample No.: 10379 Location: Drill Pad @ Boring TB-2 Depth: 0 - 0.5 feet Material Descript.: Dark brown fine to medium SAND with some fragmental limestone and silt (SP-SM) Project Requirement: N/A	LIMEROCK BEARING RATIO				
	Geotechnical Study Palm Beach Renewable Energy Facility No. 2 Palm Beach County, Florida				
	DUNKELBERGER ENGINEERING & TESTING, INC.				
	DUNKELBERGER Geotechnical-Materials-Testing/Inspection-Environmental				
	Drawn By: AR	Date Tested: 4/21/2010	Project No.: WPB-09-7594		
	Checked By: JD	Date: 4/27/2010	Sheet: C28		

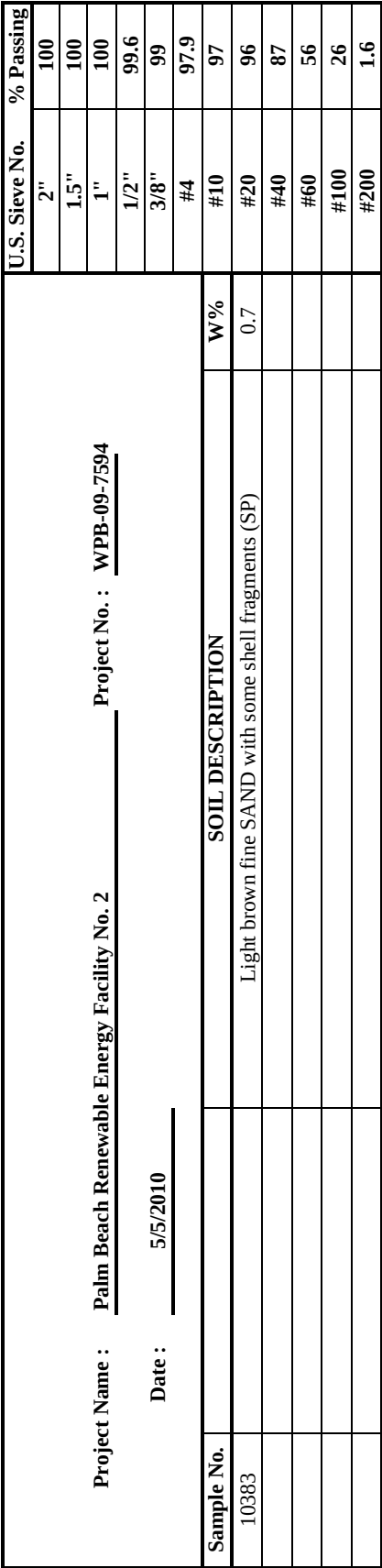
Geotechnical-Materials Testing-Inspection-Environmental





SUMMARY OF TEST RESULTS		POINT VALUES			
		LBR VALUES		PROCTOR VALUES	
MAXIMUM LBR VALUE (%):	33.9	10.7	16.3	10.7	106.4
MAXIMUM DRY DENSITY (pcf):	107.1	12.6	27.3	12.6	106.8
OPTIMUM WATER CONTENT (%):	14.4	14.4	33.9	14.4	107.1
		16.0	12.7	16.0	105.8
Sample No.: 10383 Location: Toe Of Slope, 50' East Of TB-23 Depth: 0 - 0.5 feet Material Descript.: Light brown fine to medium SAND with some shell fragments (SP) Project Requirement: N/A		LIMEROCK BEARING RATIO			
		Geotechnical Study			
		Palm Beach Renewable Energy Facility No. 2			
		Palm Beach County, Florida			
		DUNKELBERGER ENGINEERING & TESTING, INC.			
		DUNKELBERGER Geotechnical-Materials-Testing/Inspection-Environmental			
Drawn By: AR		Date Tested: 4/21/2010	Project No.: WPB-09-7594		
Checked By: JD		Date: 5/5/2010	Sheet: C30		

Geotechnical-Materials Testing-Inspection-Environmental



MOISTURE-ORGANIC CONTENT TEST RESULTS

<i>Soft Sediment Probe No.</i>	<i>Depth Interval (inches)</i>	<i>Moisture Content (%)</i>	<i>Organic Content (%)</i>
MP-21	0 - 10	27.4	0.97
MP-26	0 – 36	84.9	8.2
MP-37	0 – 7.5	39.6	1.9

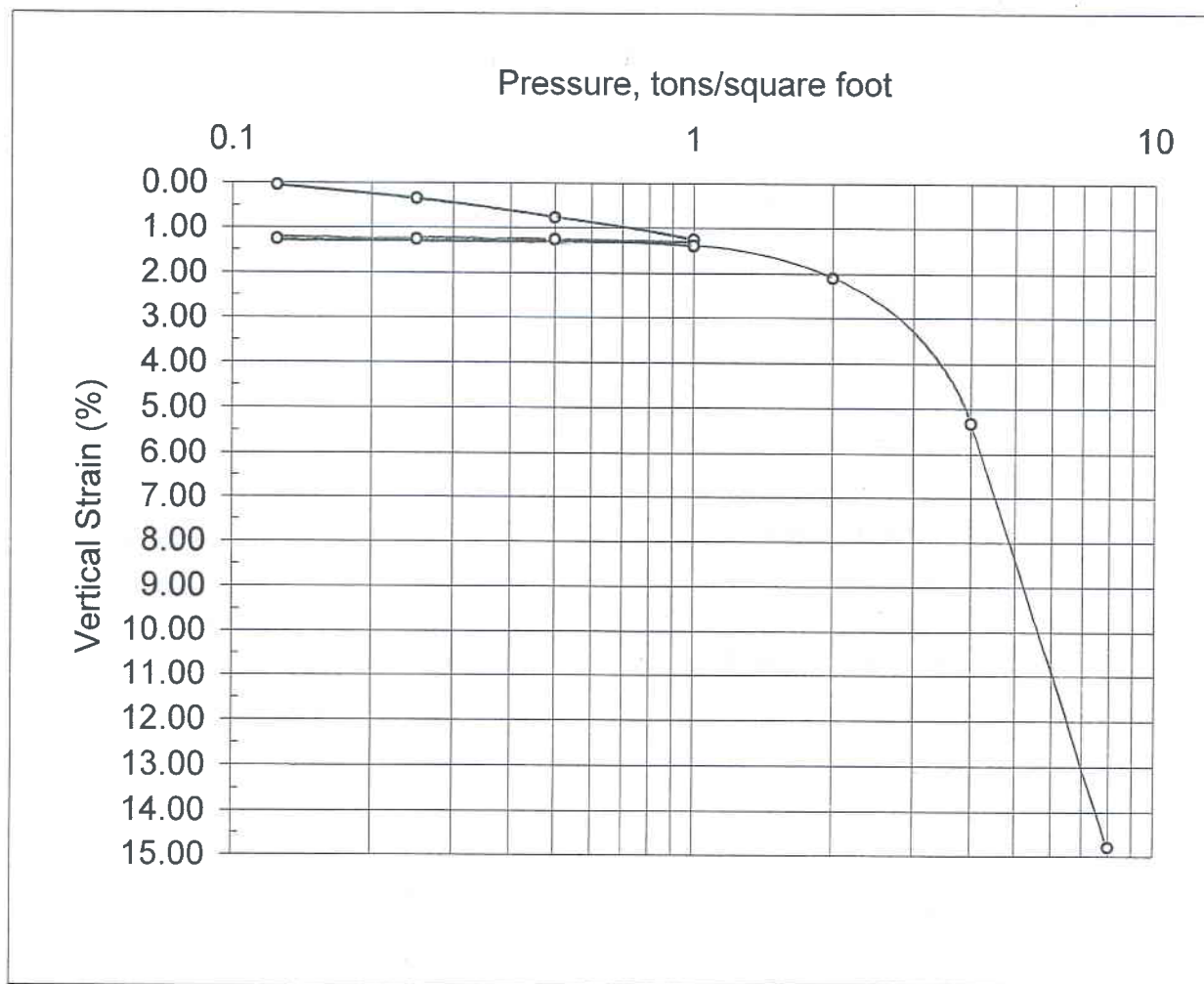
CONSOLIDATION TEST RESULTS

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CONSOLIDATION TEST REPORT

Boring No.: <u>TB-22</u>	Sample No.: <u>---</u>	Sample Depth (feet): <u>35 to 36</u>
Sample Description: <u>Gray Elastic Silt (MH)</u>		
Init. Water Content (%): <u>78.8</u>	Init. Dry Unit Wt. (pcf): <u>53.1</u>	Init. Saturation (%): <u>98.4</u>
Final Water Content (%): <u>64.4</u>	Initial Void Ratio (e): <u>2.1437</u>	Specific Gravity: <u>2.68</u>
Liquid Limit: <u>53</u>	Plastic Limit: <u>37</u>	Percent -200 (%): <u>88.6</u>



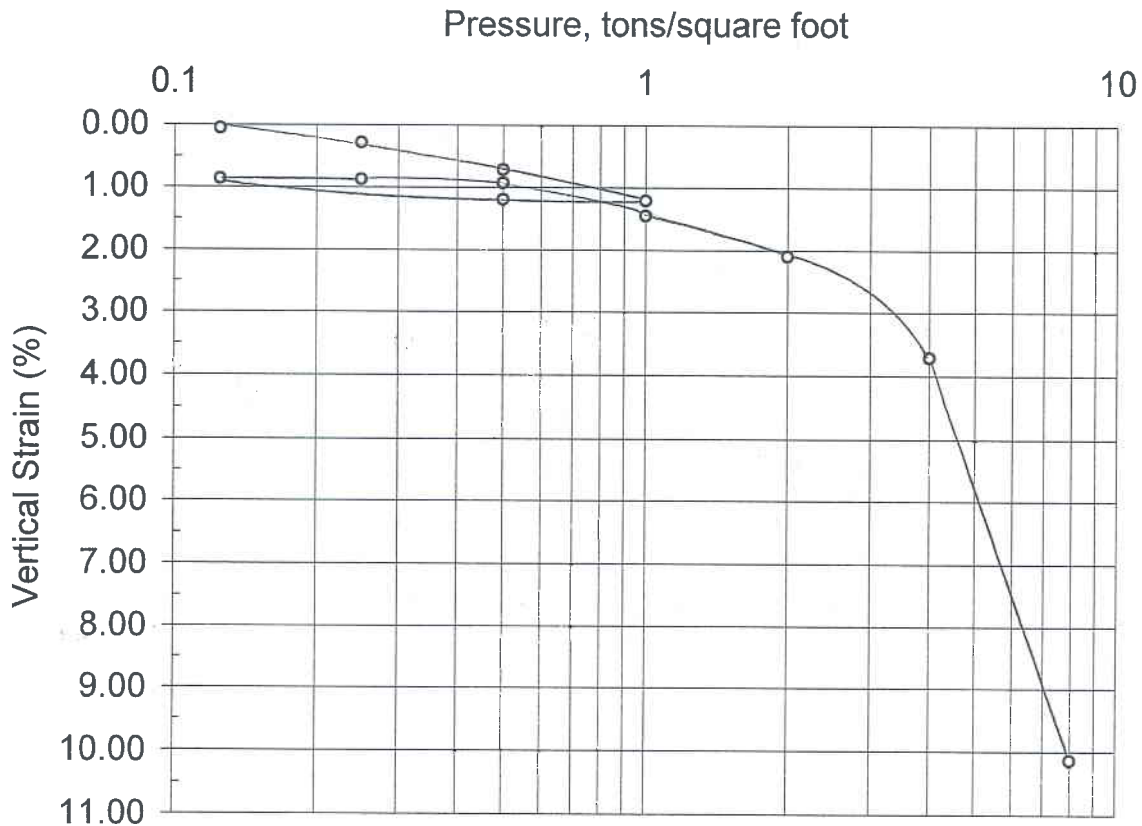
Project Name:	<u>The Palm Beach Renewable Energy Facility No. 2</u>		
Project Location:	<u>Palm Beach County, Florida</u>		
Project No.:	<u>WPB-09-7594 (BG 21.2)</u>	Date:	<u>20-Apr-10</u>

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CONSOLIDATION TEST REPORT

Boring No.: <u>TB-26</u>	Sample No.: <u>—</u>	Sample Depth (feet): <u>32 to 33</u>
Sample Description: <u>Gray sandy silt (ML)</u>		
Init. Water Content (%): <u>49.1</u>	Init. Dry Unit Wt. (pcf): <u>70.6</u>	Init. Saturation (%): <u>96.2</u>
Final Water Content (%): <u>43.3</u>	Initial Void Ratio (e): <u>1.4221</u>	Specific Gravity: <u>2.74</u>
Liquid Limit: <u>41</u>	Plastic Limit: <u>34</u>	Percent -200 (%): <u>67.4</u>



Project Name:	<u>The Palm Beach Renewable Energy Facility No. 2</u>		
Project Location:	<u>Palm Beach County, Florida</u>		
Project No.:	<u>WPB-09-7594 (BG 21.2)</u>	Date:	<u>20-Apr-10</u>

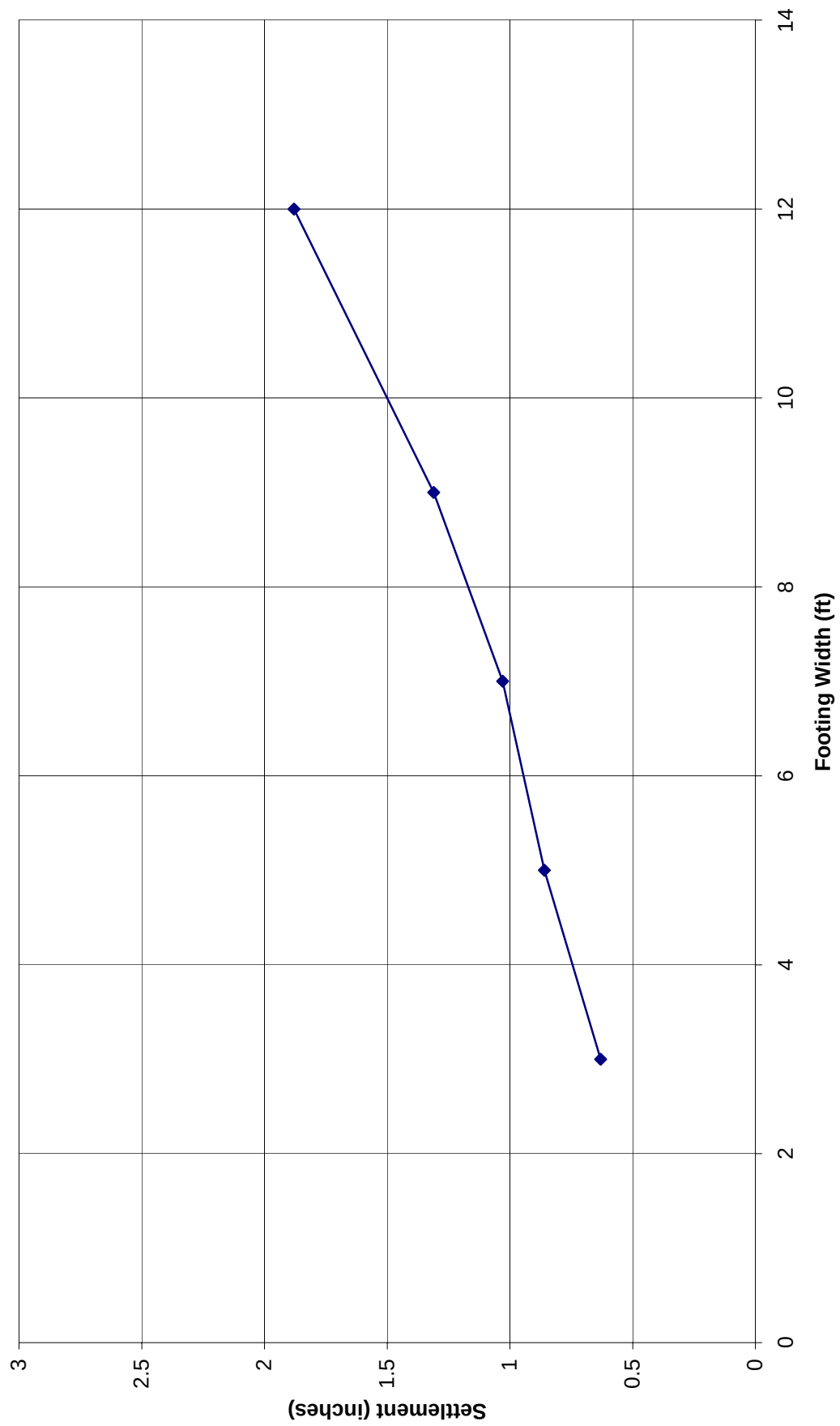
DUNKELBERGER
engineering & testing, inc.

Boring No.: TB-26				Sample No.: --		Sample Depth (feet): 33 to 35	
Sample Description:				Gray slightly silty sand with shell fragments (SP-SM)			
Init. Water Content (%):		29.1		Init. Dry Unit Wt. (pcf):		Init. Saturation (%):	
Final Water Content (%):				Initial Void Ratio (e):		Specific Gravity: 2.66	
Liquid Limit: 25				Plastic Limit: 20		Percent -200 (%): 7.3	
CONSOLIDATION TEST NOT PERFORMED FOR THIS SAMPLE							
Project Name:				The Palm Beach Renewable Energy Facility No. 2			
Project Location:				Palm Beach County, Florida			
Project No.:		WPB-09-7594 (BG 21.2)		Date:		4-May-10	

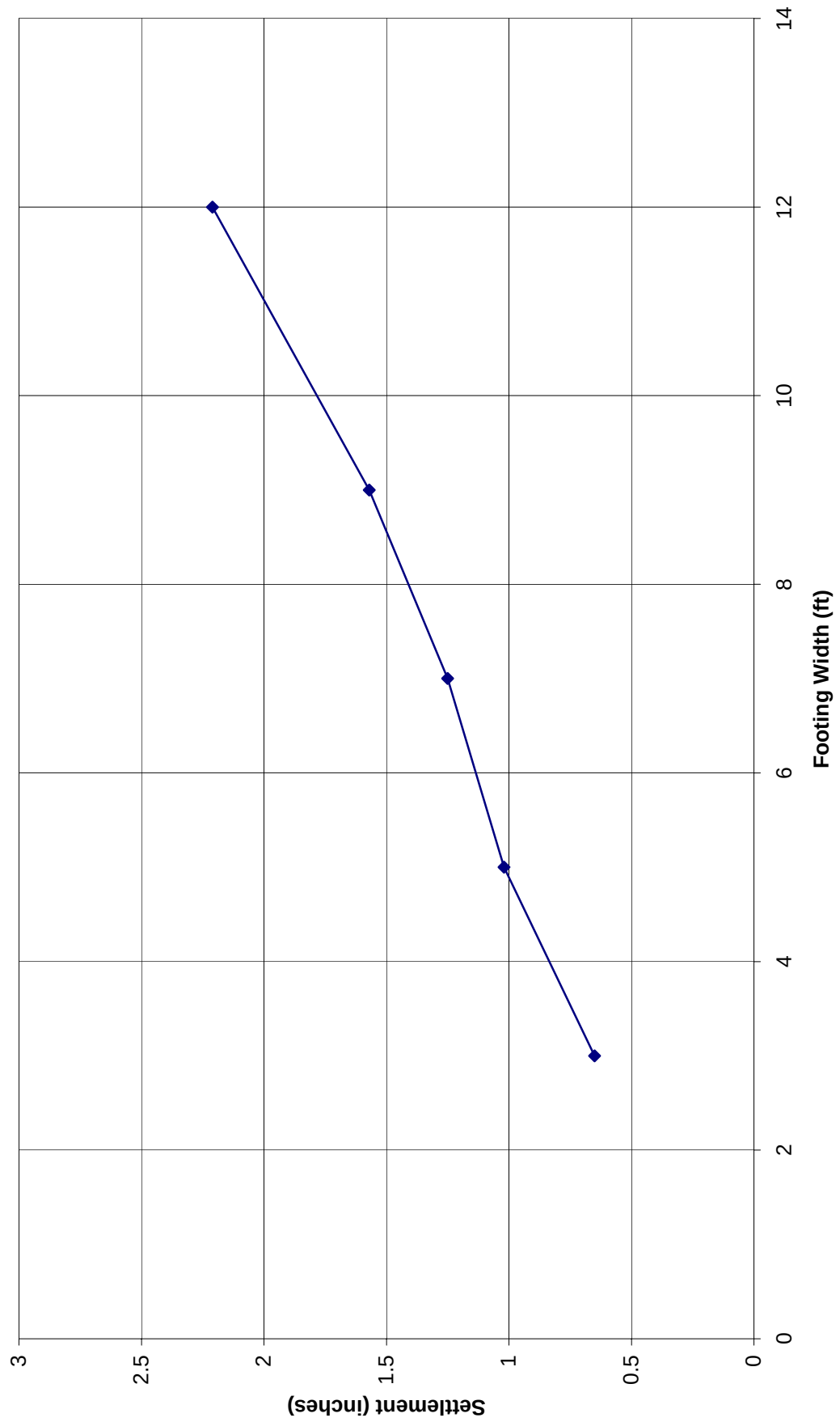
APPENDIX D

SHALLOW FOUNDATION SETTLEMENT ESTIMATES

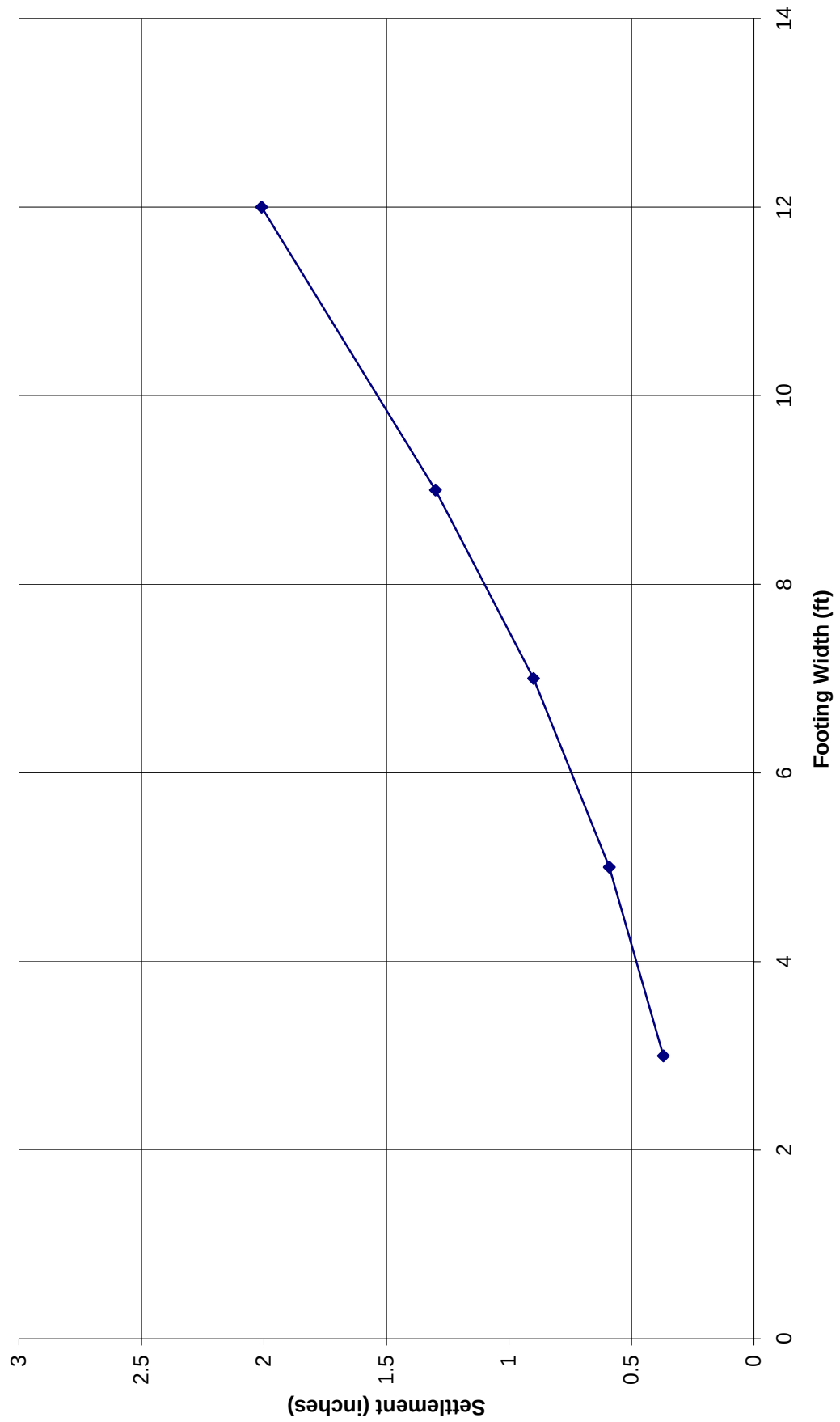
**APC Building - FFH Area Footing Settlement
4 KSF Footings**



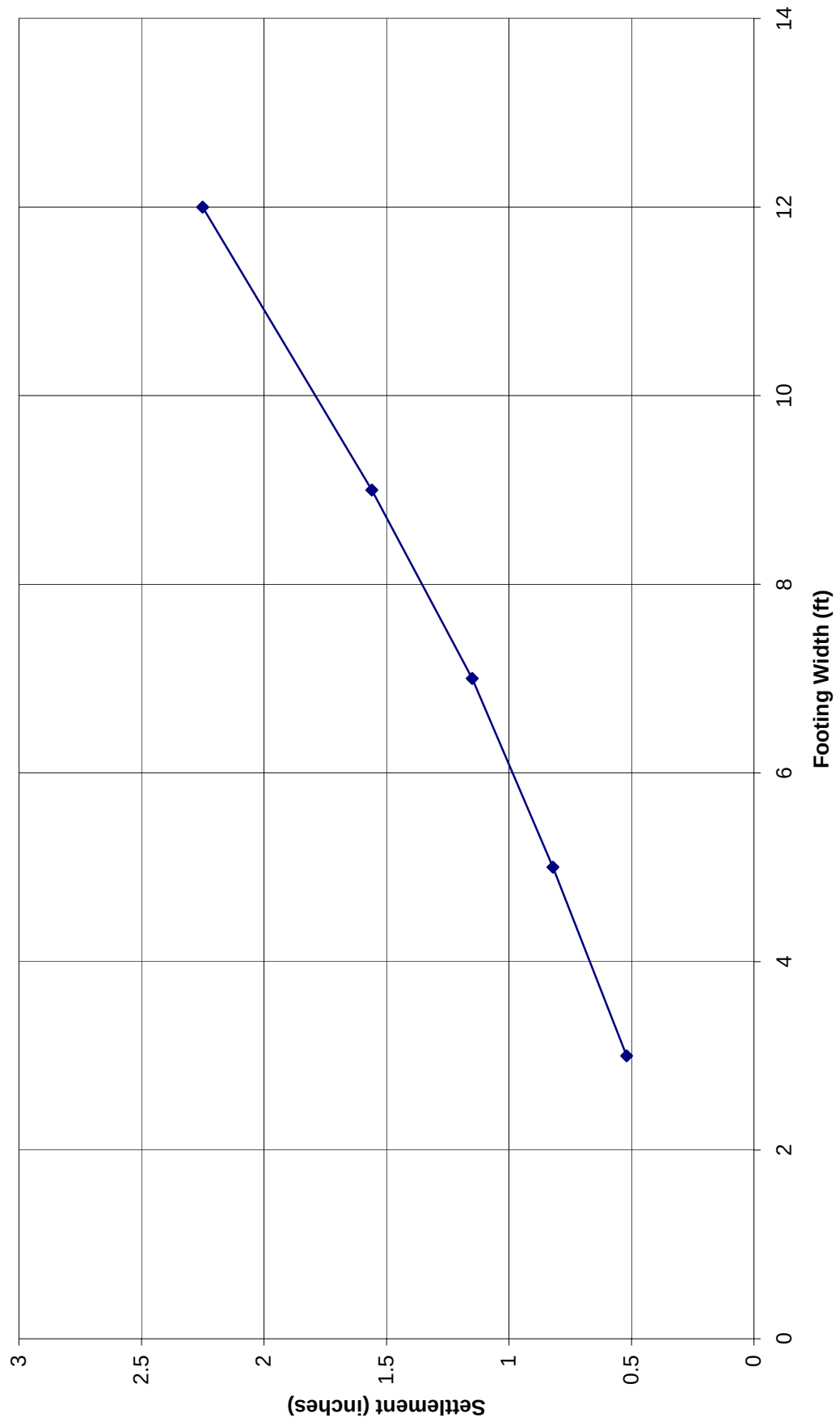
**APC Building - SDA Area Footing Settlement
4 KSF Footings**



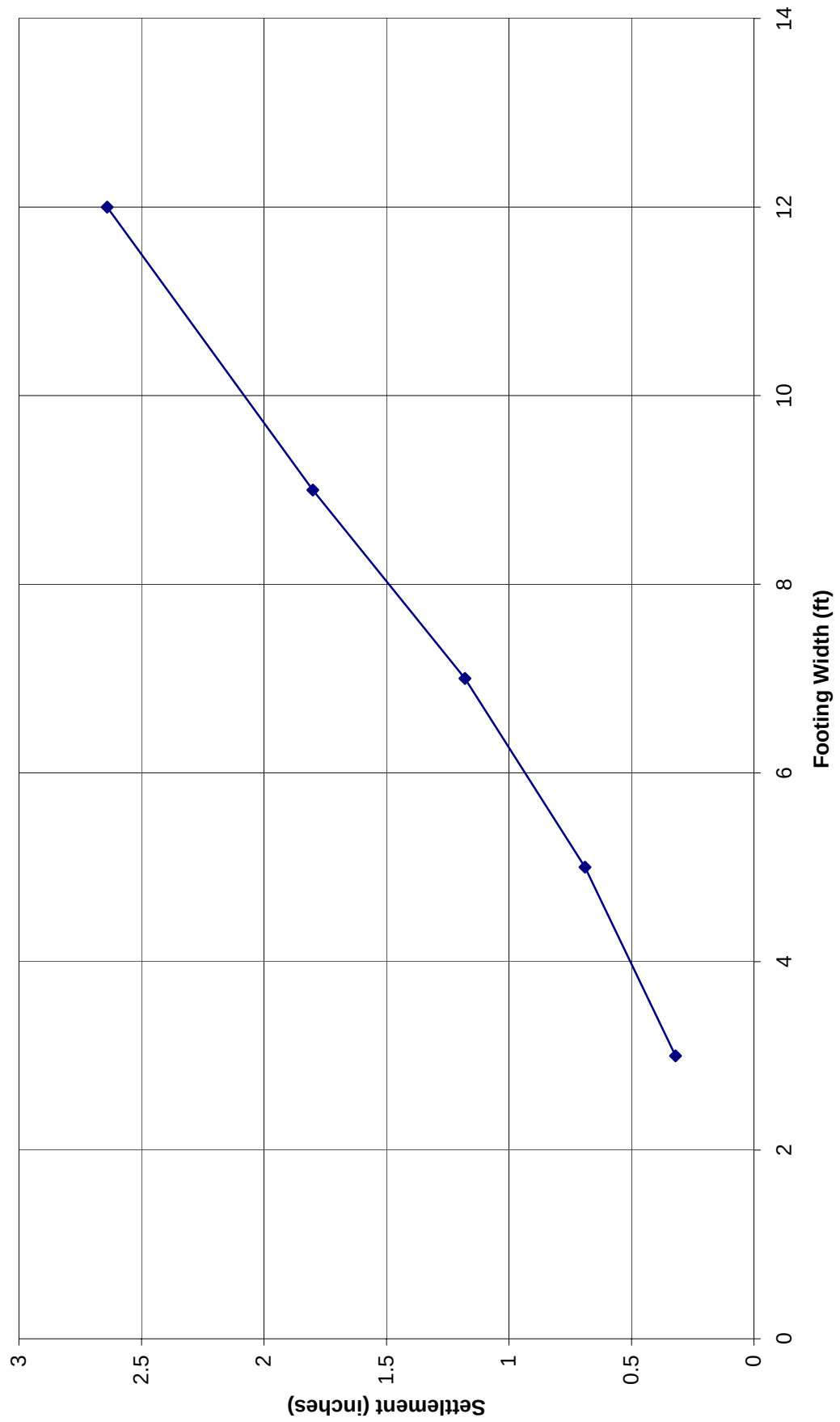
**Turbine/Generator Building Footing Settlement
4 KSF Footings**



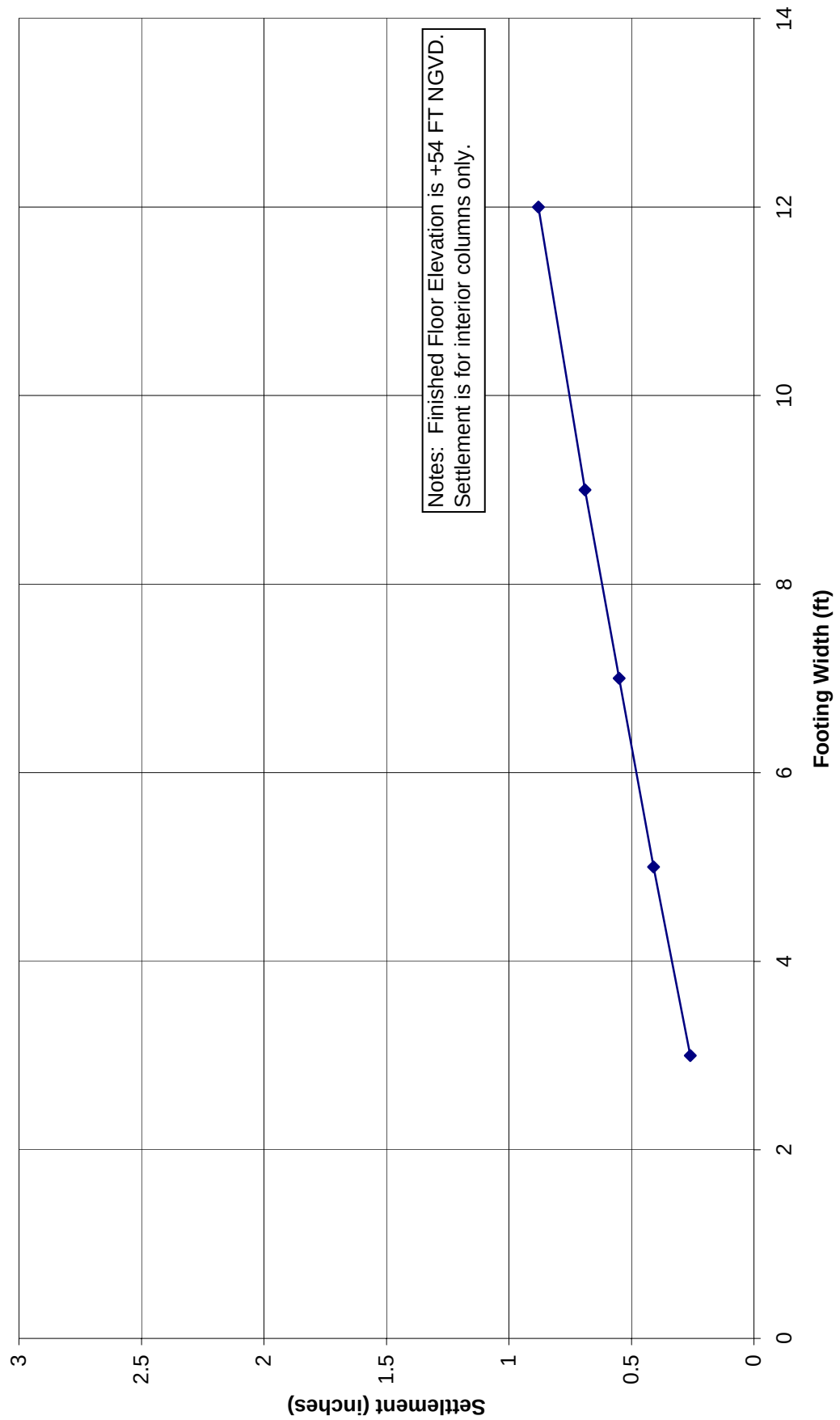
**Boiler Building Foundation Settlement
4 KSF Footings**



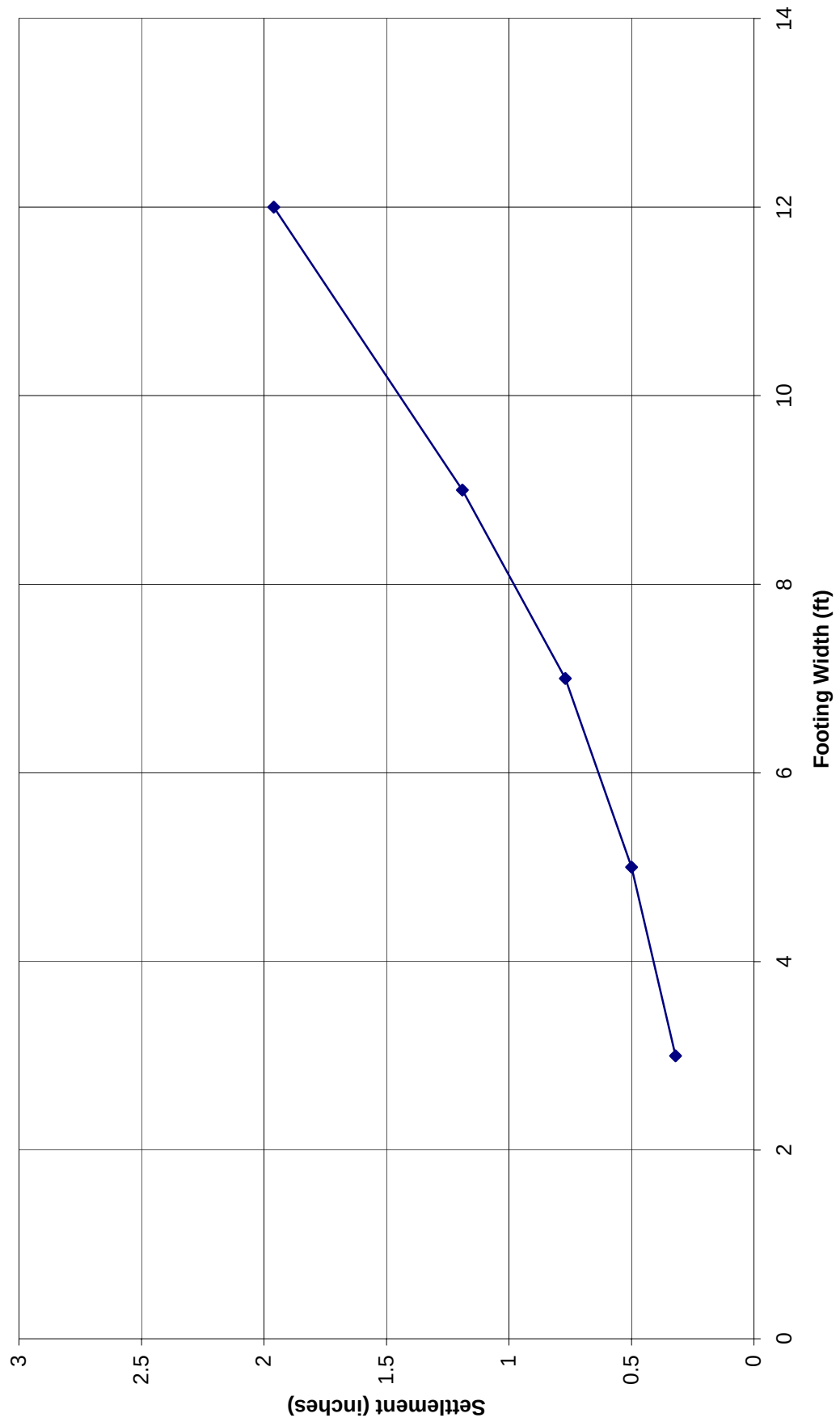
**Refuse Storage Pit and Building Footing Settlement
4 KSF Footings**



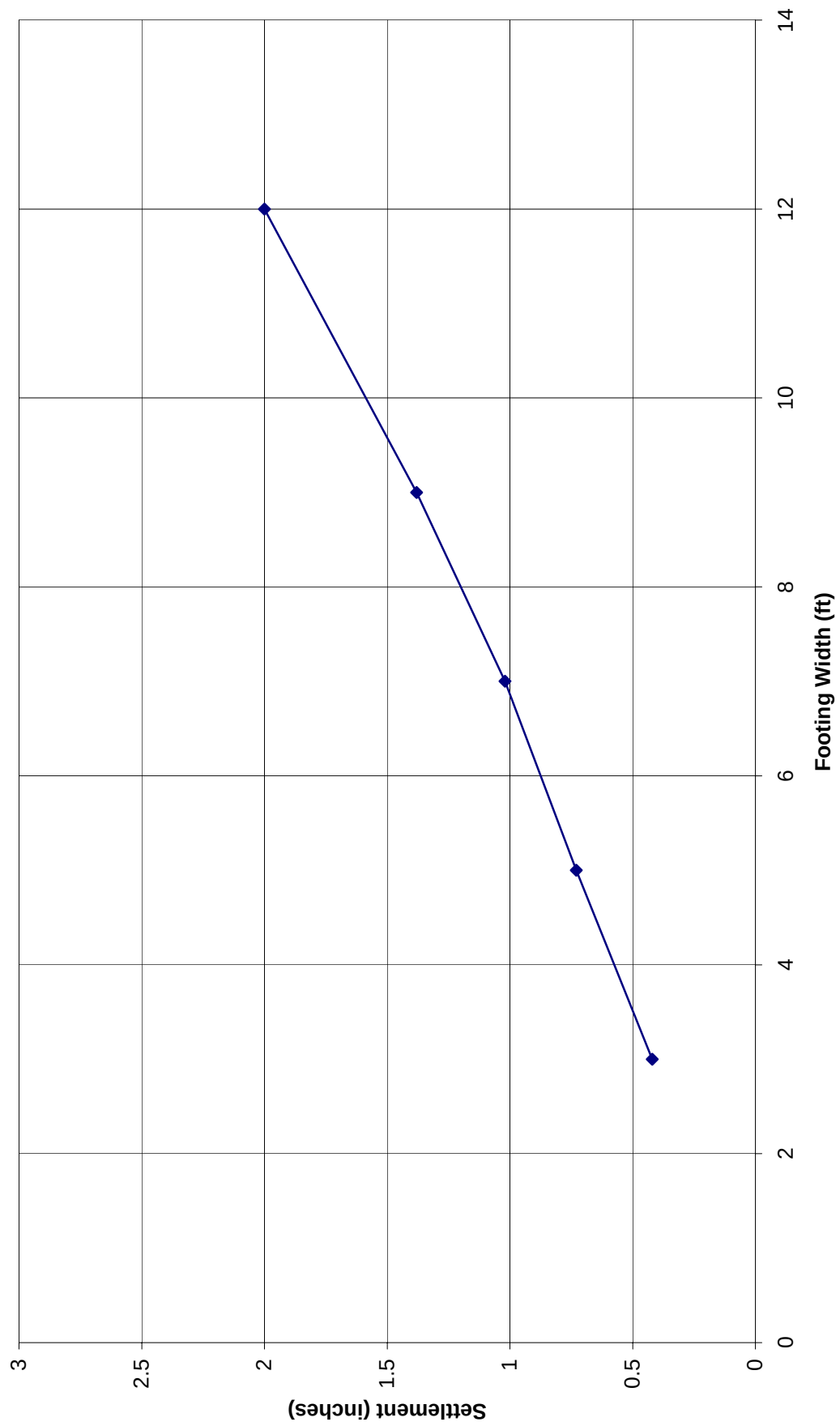
Tipping Floor and Building Footing Settlement 4 KSF Footings



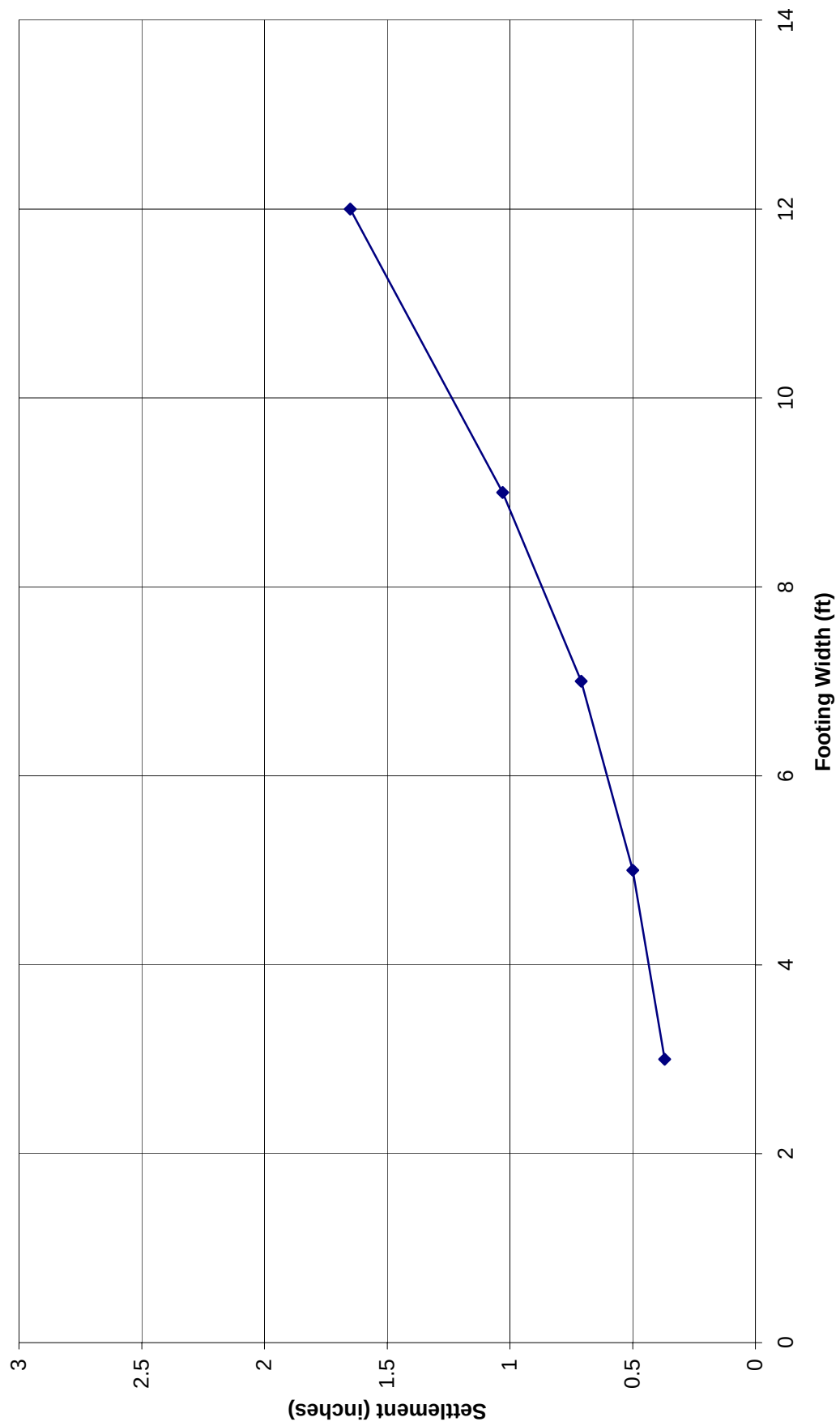
**Air Cooled Condenser Building Footing Settlement
4 KSF Footings**



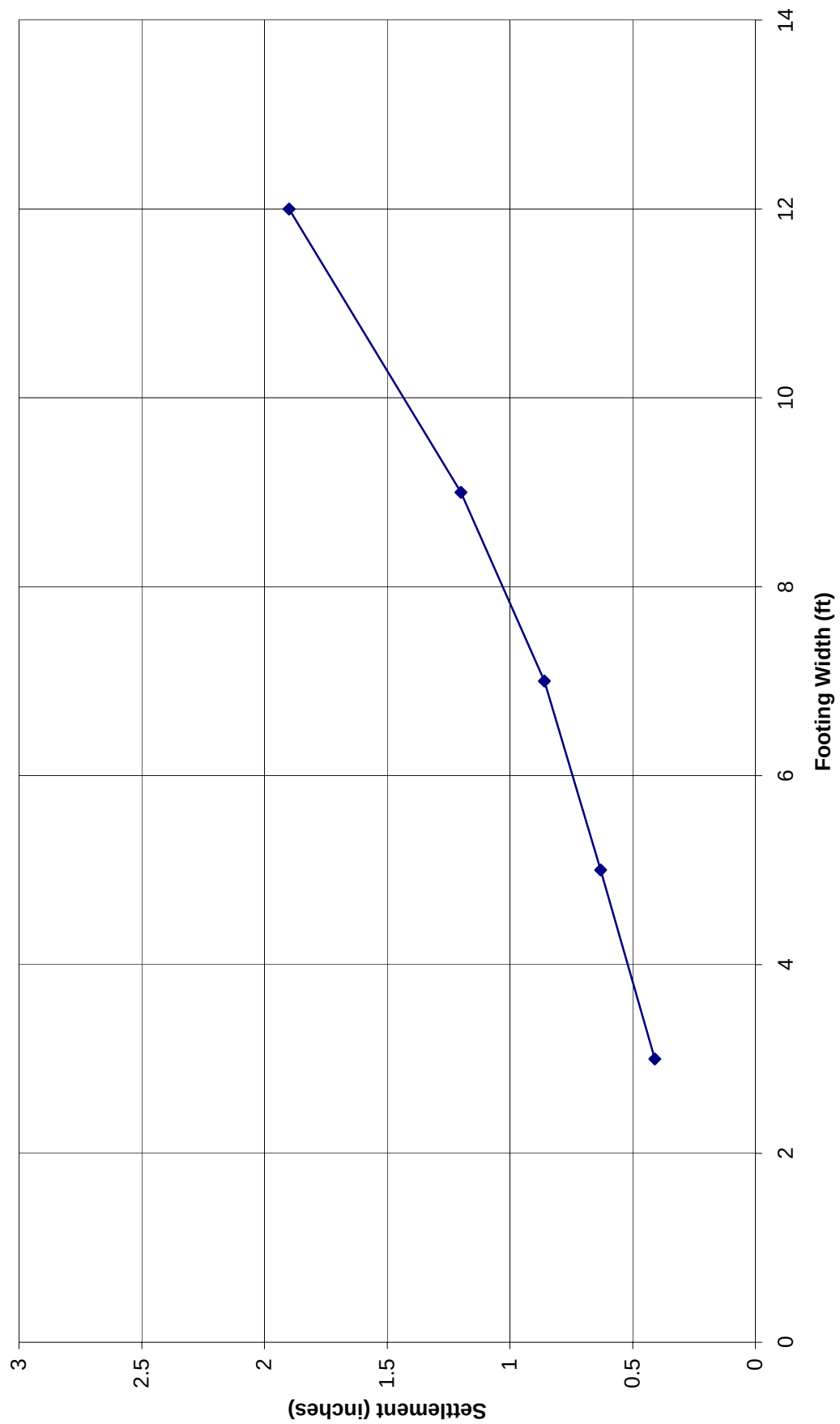
ASH Management Building Footing Settlement
4 KSF Footings



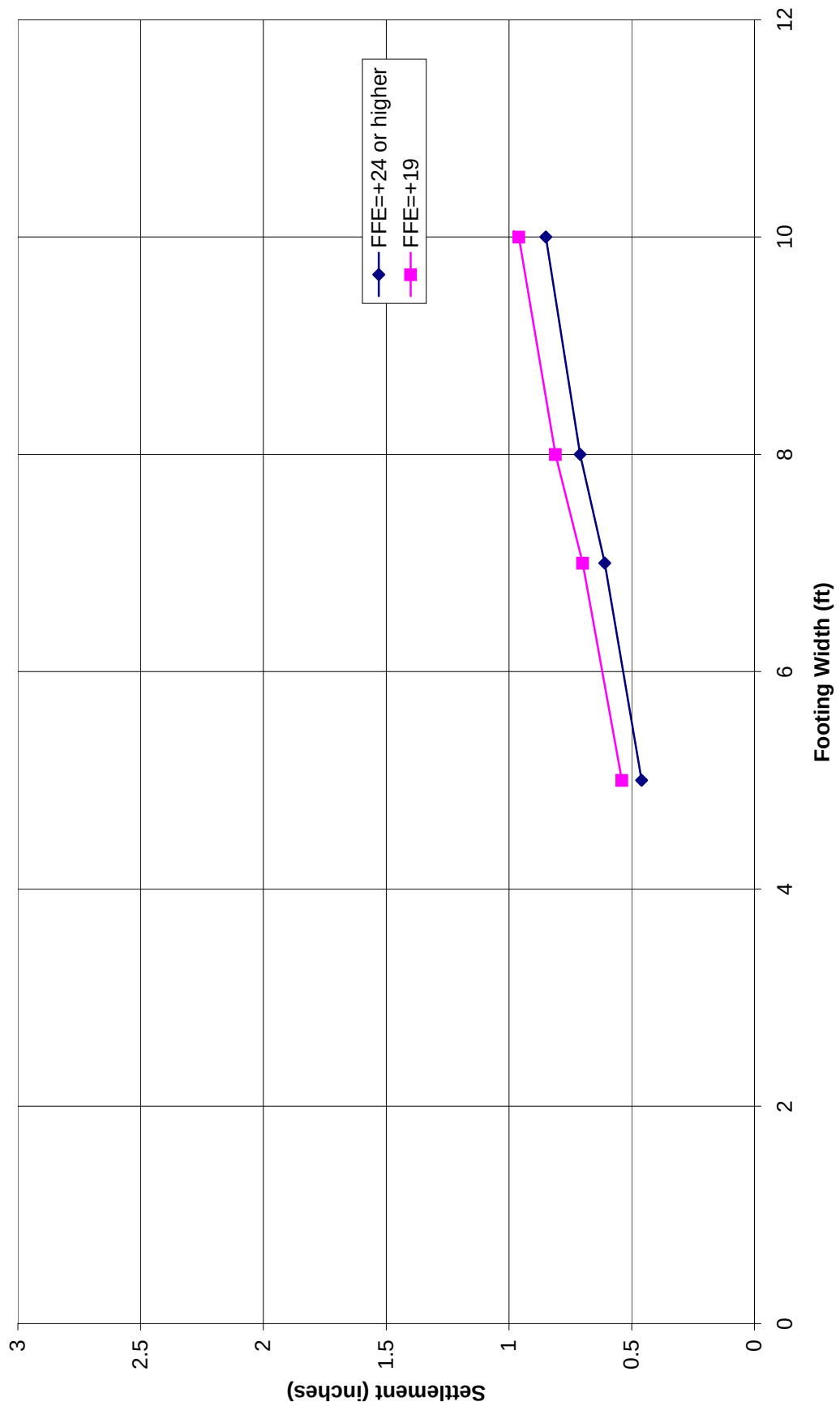
**Water Treatment Building Footing Settlement
4 KSF Footings**



Switch Gear Room Footing Settlement
4 KSF Footings



Settlement of Footings on Soils Treated by Vibro-replacement (i.e. Stone Columns)
8 KSF Footings



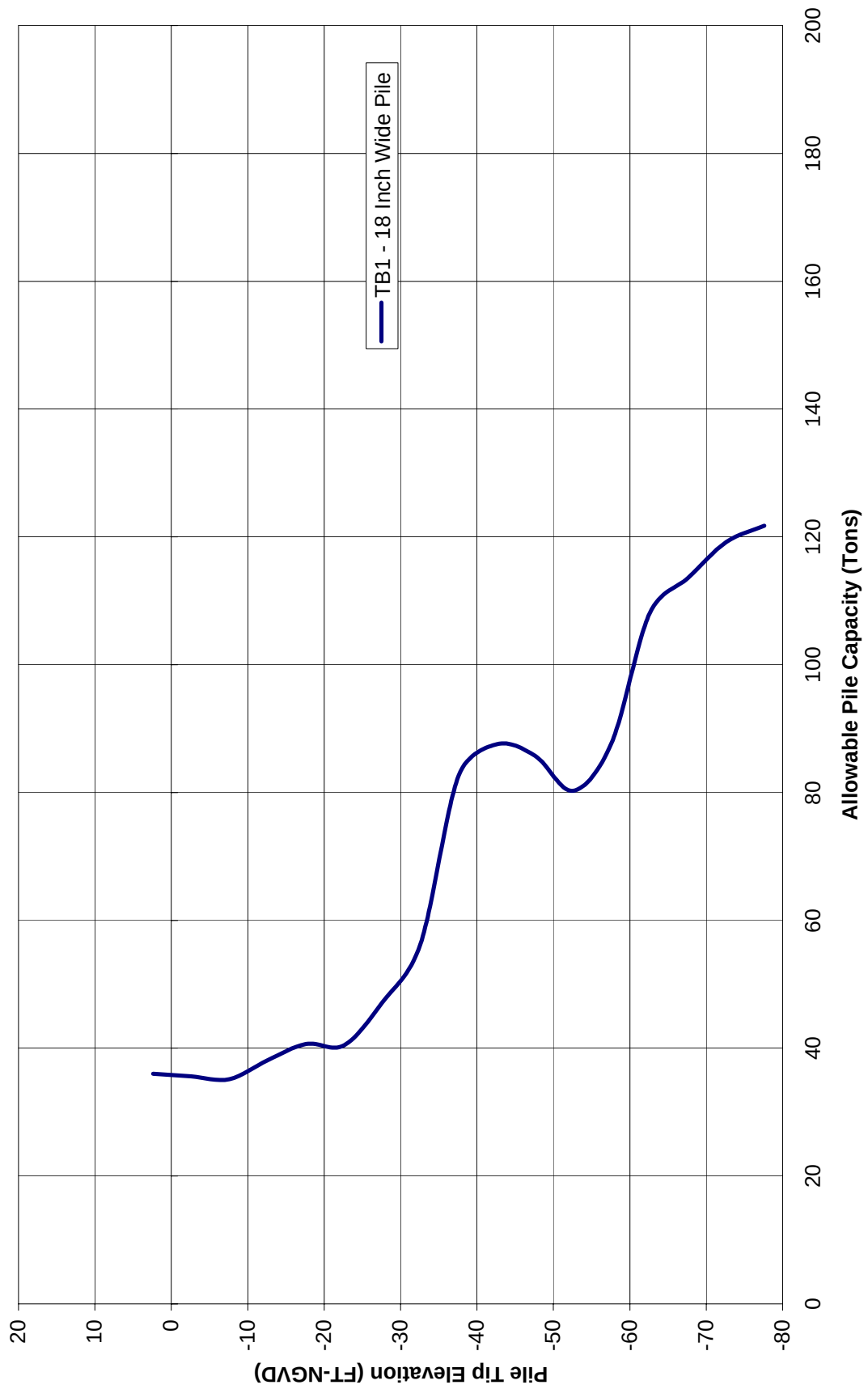
APPENDIX E

ALLOWABLE PILE AXIAL CAPACITIES

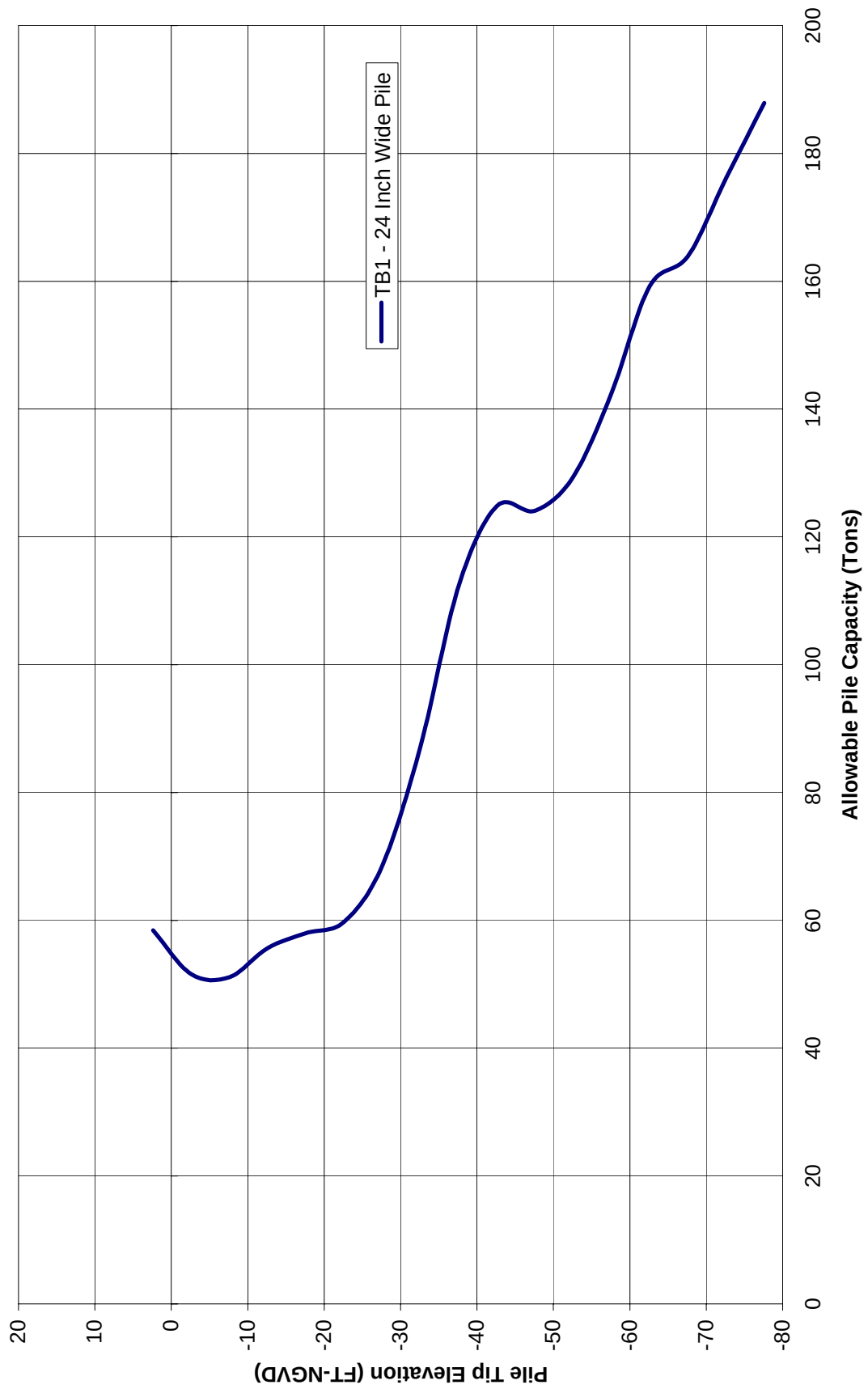
ALLOWABLE COMPRESSIVE CAPACITIES OF PRESSURE GROUTED AUGERCAST PILES (ACPS)					
Stack					
Rock Socket Length	Pile Tip	Allow. Capacity (tons) for ACP with Diam (in.) of:			Average
(feet)	Elev.	16	18	24	Tallow (tsf)
0	-32	30	35	45	0.2
10	-42	65	75	100	0.4
20	-52	135	150	200	0.8
30	-62	190	215	285	1.1
APC Bldg - FFH Area					
Rock Socket Length	Pile Tip	Allow. Capacity (tons) for ACP with Diam (in.) of:			Average
(feet)	Elev.	16	18	24	Tallow (tsf)
0	-27	25	30	40	0.2
10	-37	45	55	70	0.3
20	-47	115	130	170	0.7
30	-57	155	180	235	1.0
APC Bldg - SDA Area and Turbine/Generator					
Rock Socket Length	Pile Tip	Allow. Capacity (tons) for ACP with Diam (in.) of:			Average
(feet)	Elev.	16	18	24	Tallow (tsf)
0	-32	25	25	35	0.1
10	-42	50	55	80	0.3
20	-52	90	105	140	0.5
30	-62	155	175	235	0.9
Boiler Building					
Rock Socket Length	Pile Tip	Allow. Capacity (tons) for ACP with Diam (in.) of:			Average
(feet)	Elev.	16	18	24	Tallow (tsf)
0	-28	25	30	40	0.2
10	-38	55	60	80	0.3
20	-48	110	125	165	0.7
30	-58	140	160	210	0.9
Refuse Storage Pit and Building and Dirty Wastewater Tank					
Rock Socket Length	Pile Tip	Allow. Capacity (tons) for ACP with Diam (in.) of:			Average
(feet)	Elev.	16	18	24	Tallow (tsf)
0	-45	60	70	90	0.3
10	-55	90	105	140	0.4
20	-65	150	170	230	0.7
30	-75	210	235	315	0.9
Tipping Floor and Building					
Rock Socket Length	Pile Tip	Allow. Capacity (tons) for ACP with Diam (in.) of:			Average
(feet)	Elev.	16	18	24	Tallow (tsf)
0	-47	55	65	85	0.2
10	-57	155	175	235	0.7
20	-67	255	285	385	1.1
30	-77	265	295	395	1.1

Note: Average Tallow is the average allowable pile skin friction resistance from elevation +10 ft NGVD to pile bottom. For analysis purposes the top of the rock socket was taken to be the top of the limestone that enters the profile below the bottom of the lowest layer of silt or silty sand (Stratum 7).

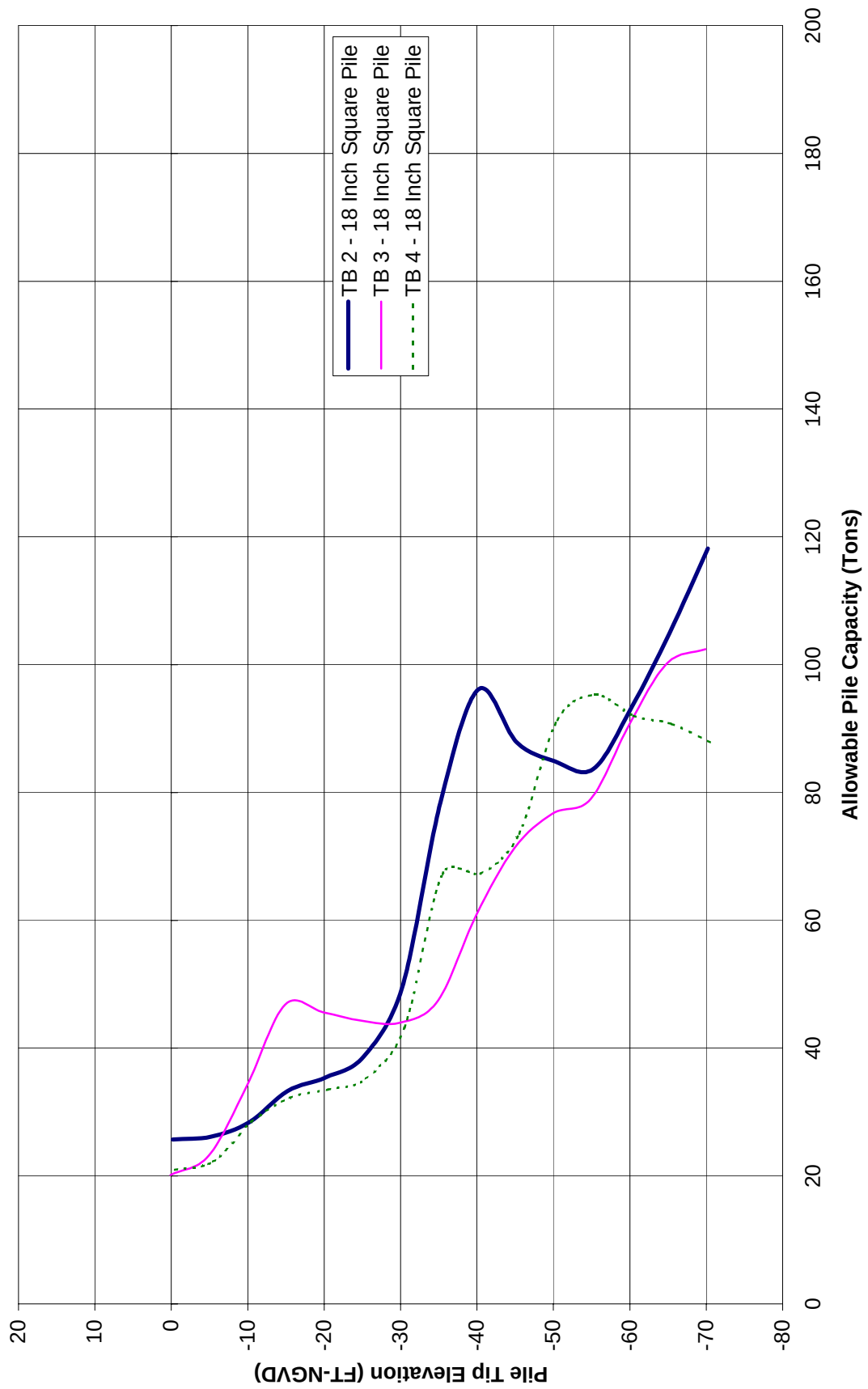
STACK
TB1 - 18 Inch Square Pile



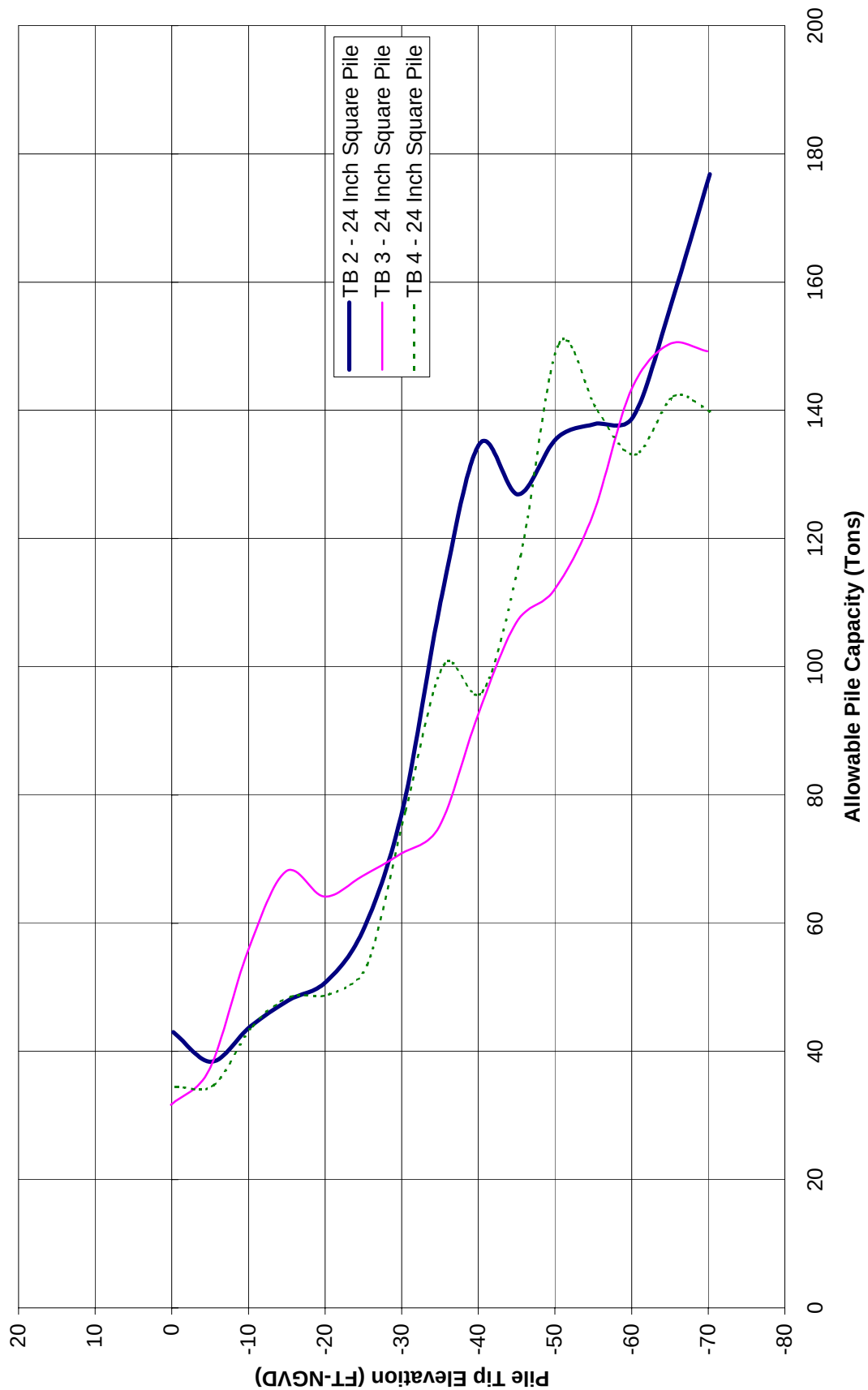
STACK
TB1 - 24 Inch Square Pile



APC BUILDING - FFH AREA
TB 2, 3 and 4 - 18 Inch Square Piles

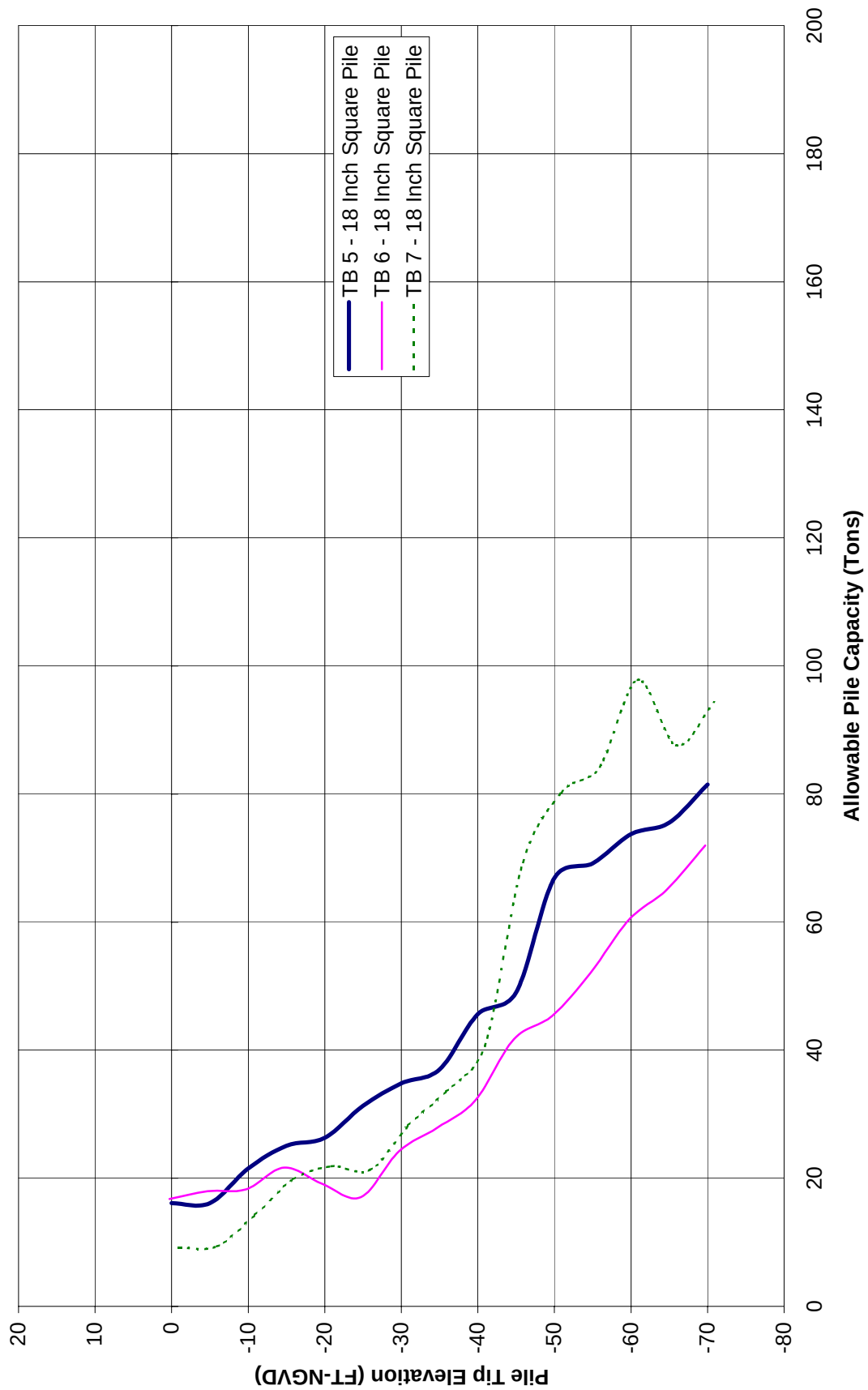


APC BUILDING - FFH AREA
TB 2, 3 and 4 - 24 Inch Square Piles



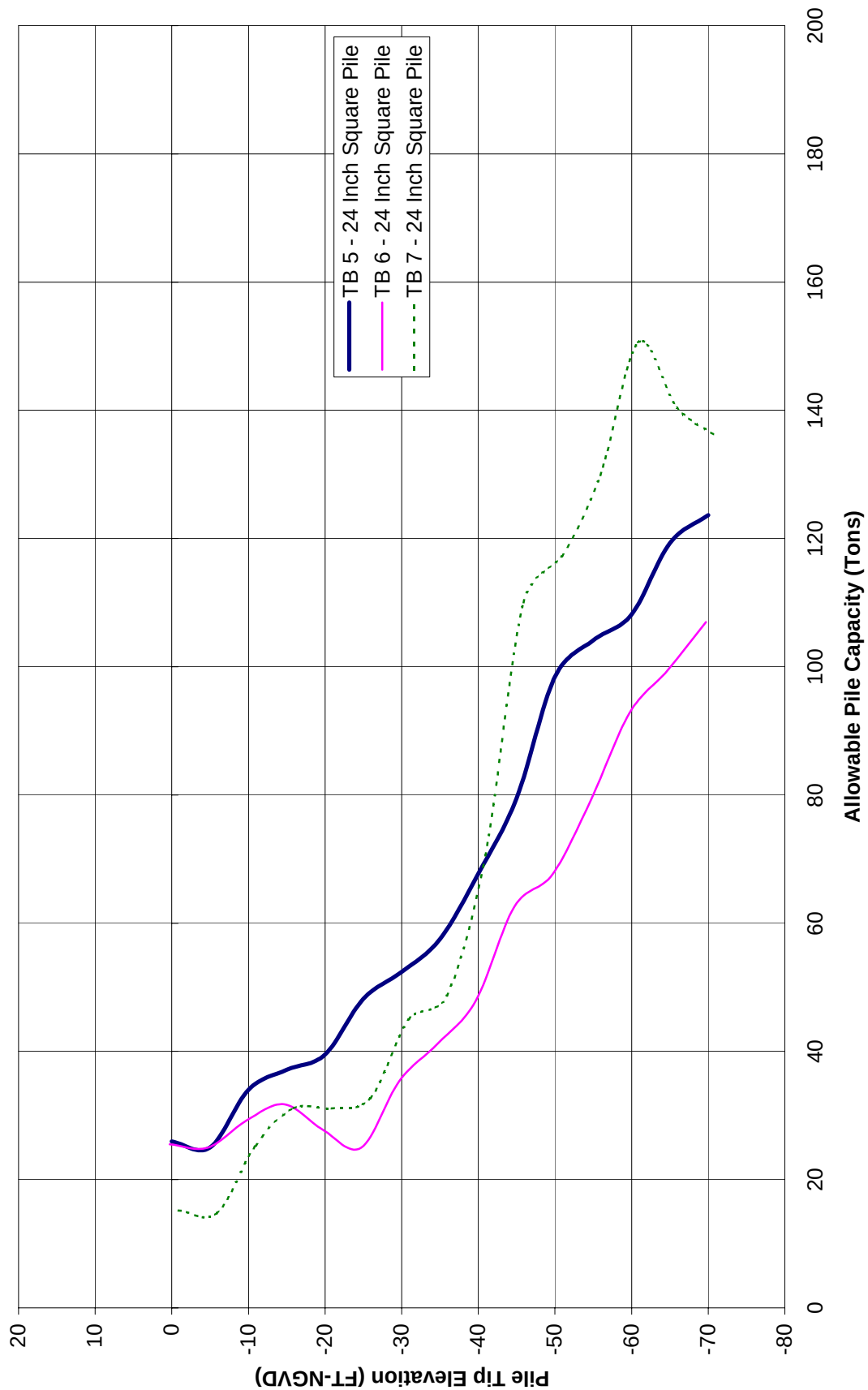
APC BUILDING - SDA AREA AND TURBINE/GENERATOR

TB 5, 6 and 7 - 18 Inch Square Piles

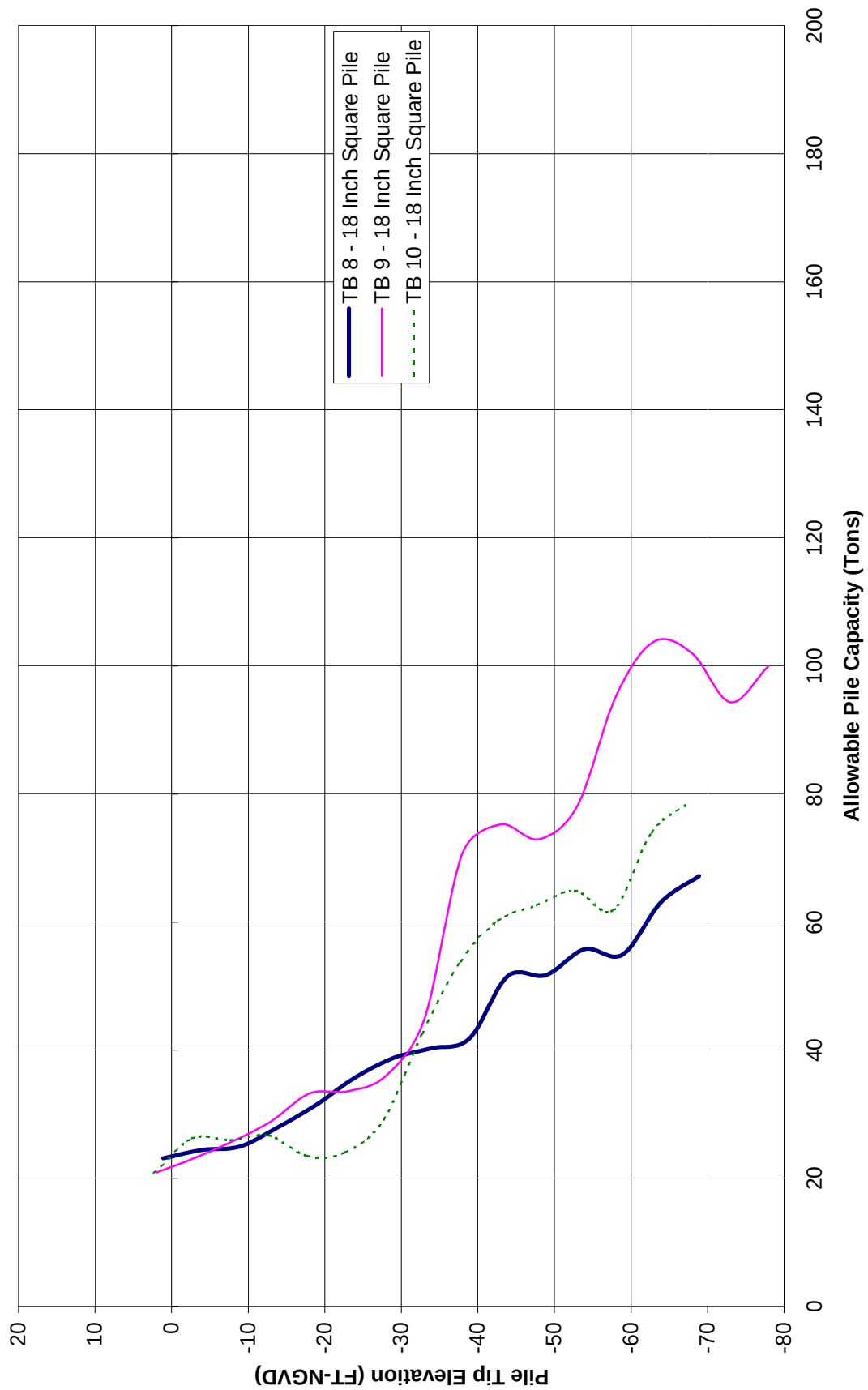


APC BUILDING - SDA AREA AND TURBINE/GENERATOR

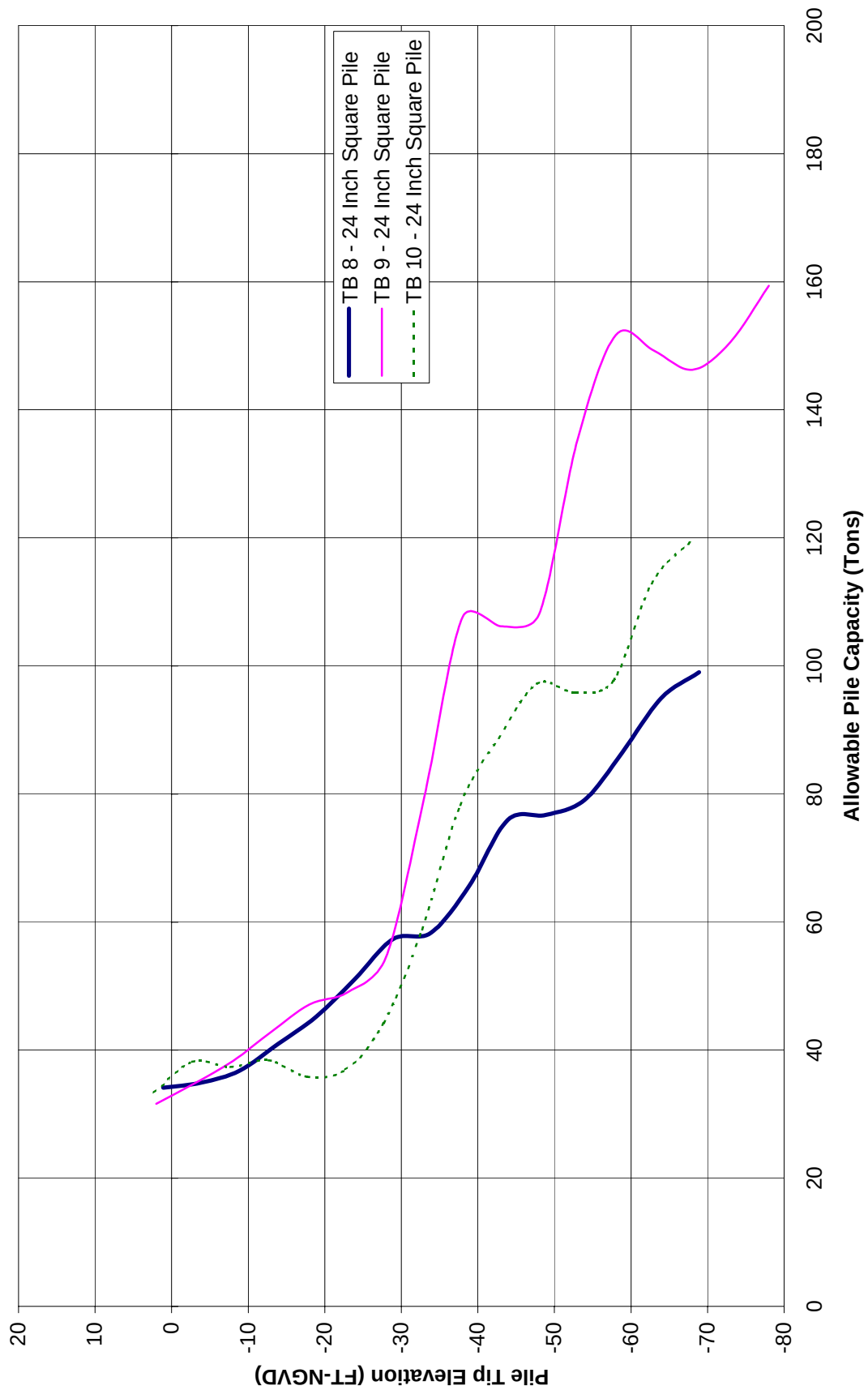
TB 5, 6 and 7 - 24 Inch Square Piles



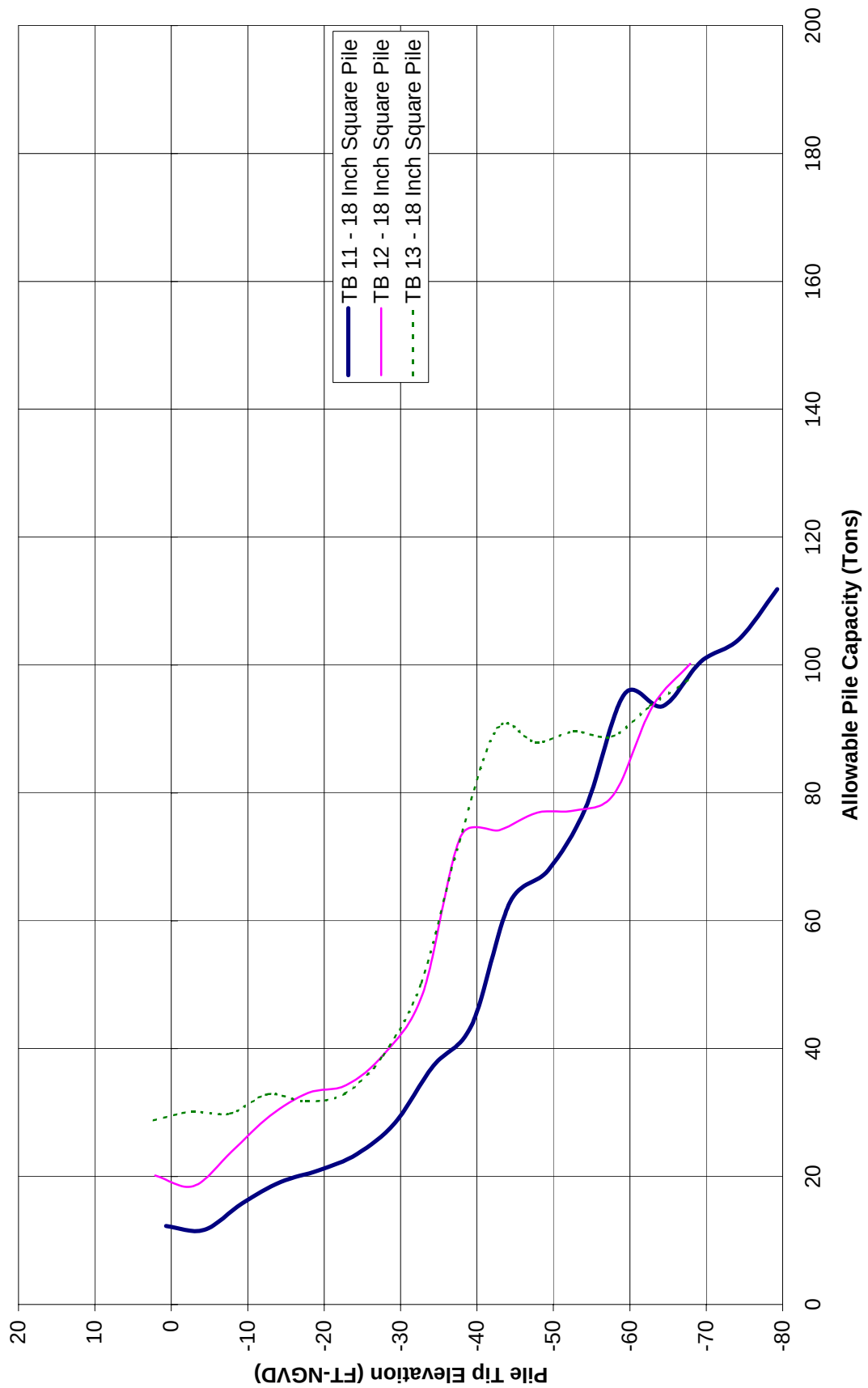
BOILER BUILDING
TB 8, 9 and 10 - 18 Inch Square Piles



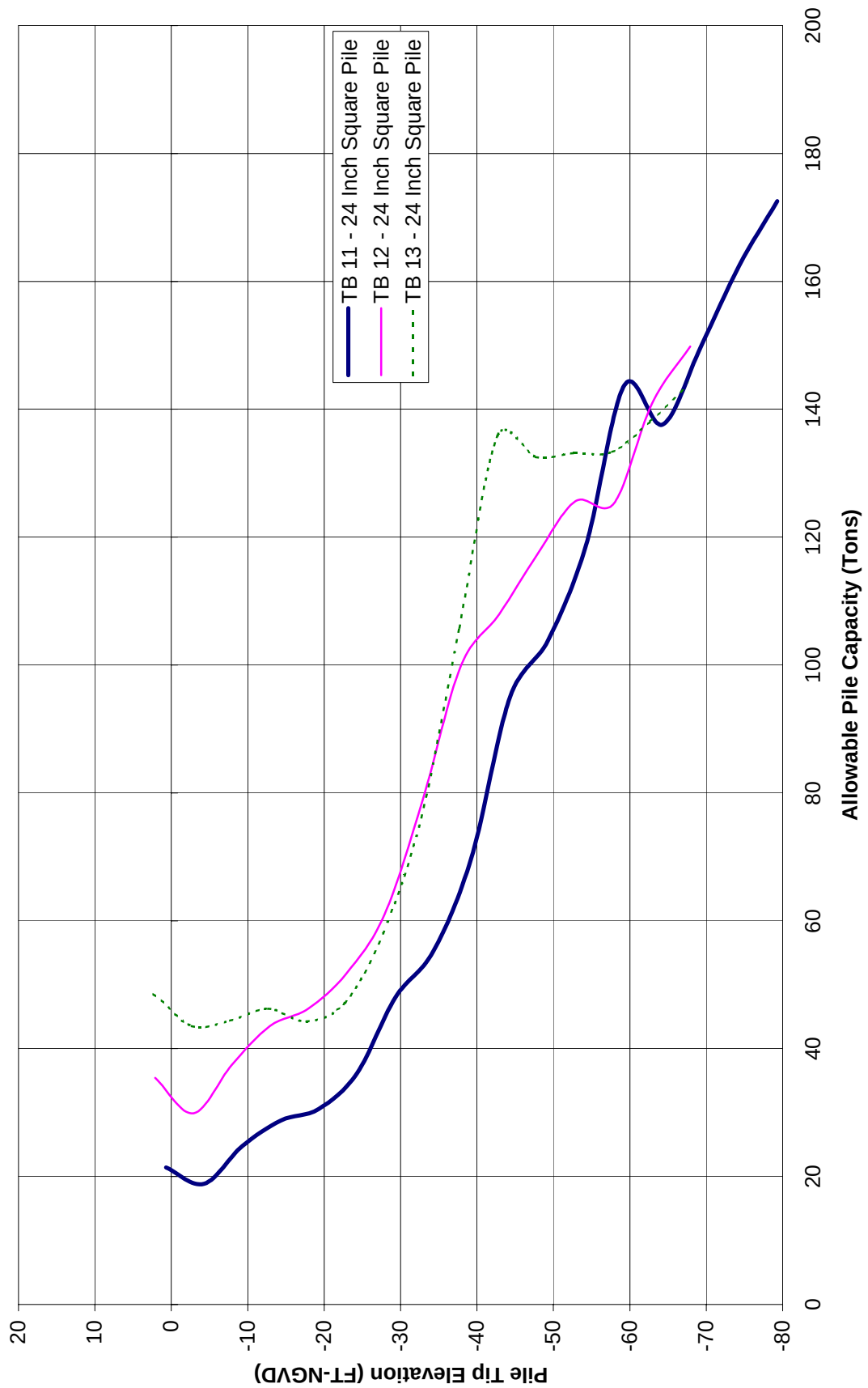
BOILER BUILDING
TB 8, 9 and 10 - 24 Inch Square Piles



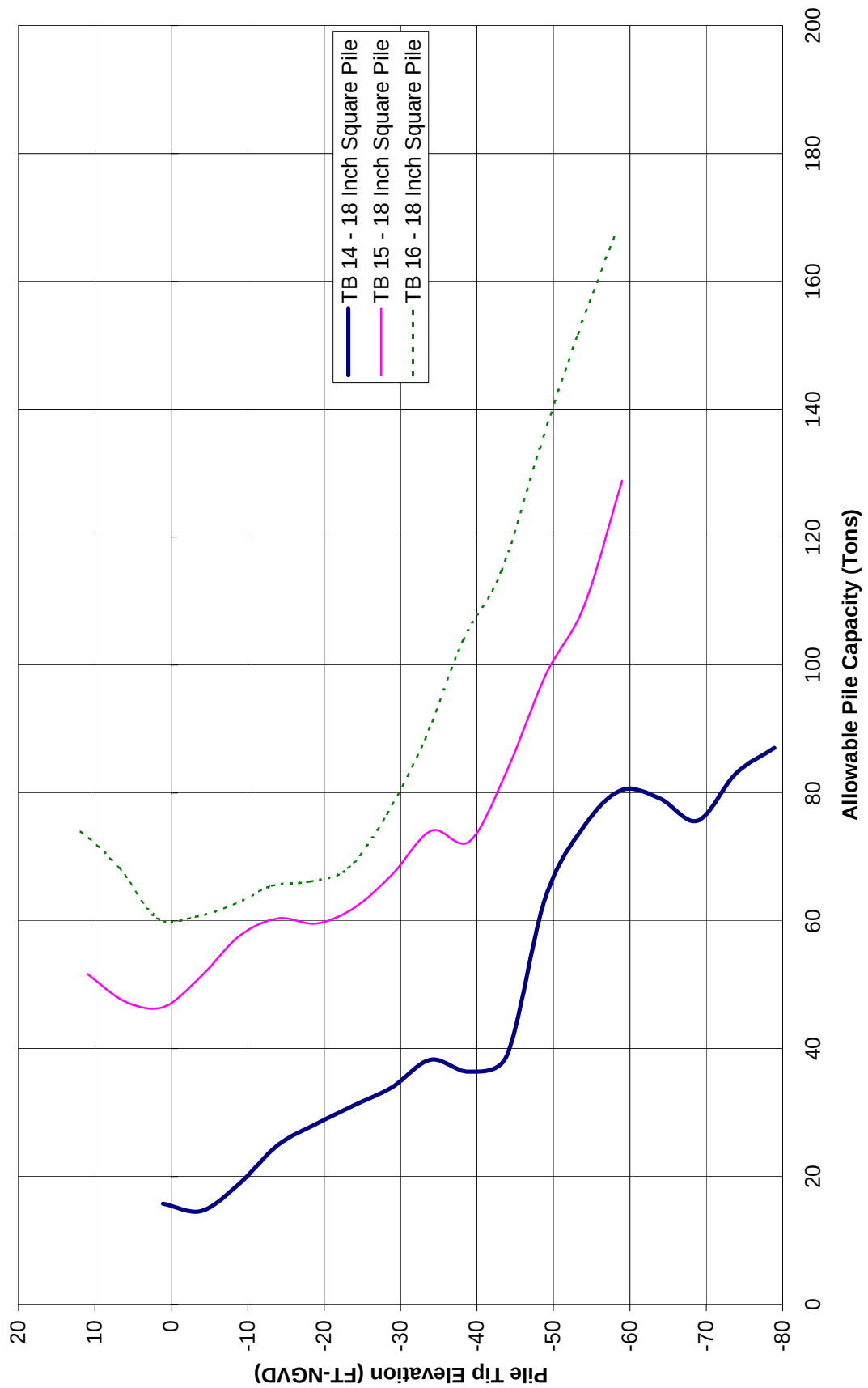
REFUSE STORAGE PIT AND BUILDING TB 11, 12 and 13 - 18 Inch Square Piles



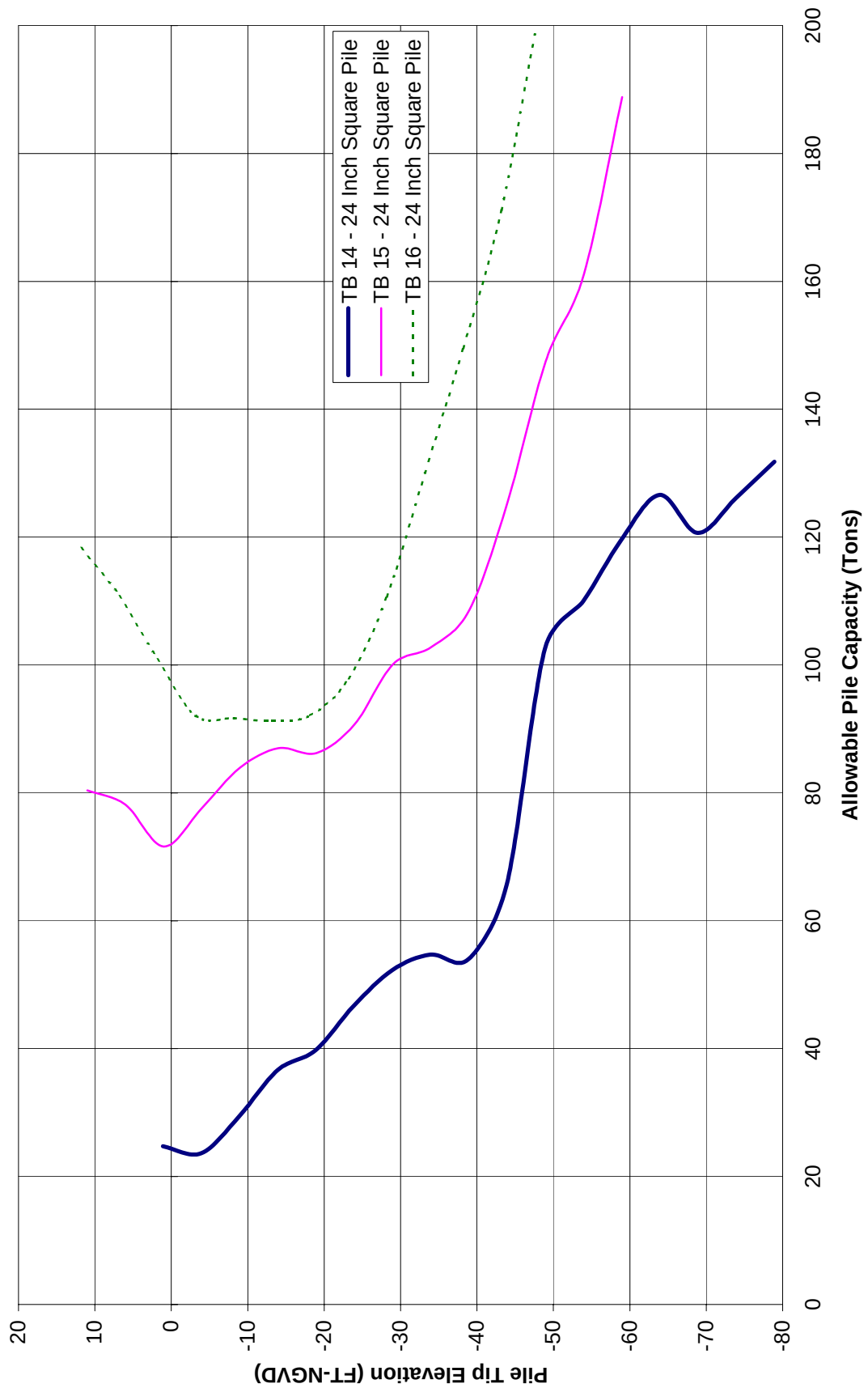
REFUSE STORAGE PIT AND BUILDING
TB 11, 12 and 13 - 24 Inch Square Piles



TIPPING FLOOR AND BUILDING
TB 14, 15 and 16 - 18 Inch Square Piles



TIPPING FLOOR AND BUILDING
TB 14, 15 and 16 - 24 Inch Square Piles



ALLOWABLE TENSILE CAPACITIES OF DRIVEN PPC PILES

18 Inch Diameter Driven Concrete Pile Allowable Uplift Capacities in Tons																
Pile Tip Elevation (FT-NGVD)	TB1	TB2	TB3	TB4	TB5	TB6	TB7	TB8	TB9	TB10	TB11	TB12	TB13	TB14	TB15	TB16
-30	29	24	23	20	19	16	15	21	20	20	14	23	24	19	39	46
-50	41	39	34	32	26	23	25	29	33	30	32	35	44	27	49	61
-70	55	52	48	47	39	33	42	36	48	39	46	47	54	40	--	--

24 Inch Diameter Driven Concrete Pile Allowable Uplift Capacities in Tons																
Pile Tip Elevation (FT-NGVD)	TB1	TB2	TB3	TB4	TB5	TB6	TB7	TB8	TB9	TB10	TB11	TB12	TB13	TB14	TB15	TB16
-30	41	34	33	28	27	23	22	30	30	28	21	33	33	27	54	64
-50	58	54	48	46	38	33	36	41	47	42	46	50	62	39	68	85
-70	77	73	68	66	55	48	58	52	69	55	64	66	75	58	--	--



Florida Department of Environmental Protection

Marjory Stoneman Douglas Building
3900 Commonwealth Boulevard
Tallahassee, Florida 32399-3000

Charlie Crist
Governor

Jeff Kottkamp
Lt. Governor

Michael W. Sole
Secretary

May 26, 2010

CERTIFIED MAIL - RETURN RECEIPT REQUESTED

Marc Bruner
Chief Administrative Officer
Solid Waste Authority of Palm Beach County
7501 North Jog Road
West Palm Beach, Florida 33412

RE: North County Resource Recovery Facility
DEP Case Number PA 84-20
Post-certification Amendment dated October 6, 2009

Dear Mr. Bruner:

On October 6, 2009, the Florida Department of Environmental Protection's Siting Coordination Office (Department) received a request from Solid Waste Authority of Palm Beach County (SWA) for a post certification amendment to their North County Resource Recovery Facility's Site Certification Application (SCA) to fill the South Borrow Lake located on the property in order to accommodate future expansion.

Pursuant to Section 62-17.205(1), Florida Administrative Code ("F.A.C"), the Department has determined that this proposal is an amendment to the application that will not require modification to the Conditions of Certification (COC) for the Facility.

Based on a review of the information submitted on October 6, 2009 and supplemental information received on January 4, 2010 and April 26, 2010, the Department has no objections to the proposed project and hereby approves the amendment request, provided that SWA continues to comply with all terms and conditions of the existing COC for the Facility.

The Department's Order shall become final unless a timely petition for an administrative hearing is filed under sections 120.569 and 120.57, Florida Statutes ("F.S."), within **21** days of receipt of this Order. The procedures for petitioning for a hearing are set forth below.

Persons affected by this Order have the following options:

If you choose to accept the above decision by the Department concerning the postcertification amendment you do not have to do anything. This Order is final and effective as of the date on the top of the first page of this Order.

If you disagree with the decision, you may do one of the following:

1. File a petition for administrative hearing with the Department's Office of General Counsel within **21** days of receipt of this Order; or
2. File a request for an extension of time to file a petition for hearing with the Department's Office of General Counsel within **21** days of receipt of this Order. Such a request should be made if you wish to meet with the Department in an attempt to informally resolve any disputes without first filing a petition for hearing.

Please be advised that mediation of this decision pursuant to section 120.573, F.S., is not available.

How to Request an Extension of Time to File a Petition for Hearing

For good cause shown, pursuant to rule 62-110.106(4), F.A.C., the Department may grant a request for an extension of time to file a petition for hearing. Such a request must be filed (received) in the Office of General Counsel of the Department at 3900 Commonwealth Boulevard, Mail Station 35, Tallahassee, Florida, 32399-3000, within **21** days of receipt of this Order. Petitioner, if different from the addressee, shall mail a copy of the request to the addressee at the time of filing. Timely filing a request for an extension of time tolls the time period within which a petition for administrative hearing must be made.

How to File a Petition for Administrative Hearing

A person whose substantial interests are affected by this Order may petition for an administrative proceeding (hearing) under sections 120.569 and 120.57, F.S. The petition must contain the information set forth below and must be filed (received) in the Office of General Counsel of the Department at 3900 Commonwealth Boulevard, MS 35, Tallahassee, Florida, 32399-3000, within **21** days of receipt of this Order. Petitioner, if different from the addressee, shall mail a copy of the petition to the addressee at the time of filing. Failure to file a petition within this time period shall waive the right of anyone who may request an administrative hearing under sections 120.569 and 120.57, F.S.

Pursuant to subsections 120.54(5)(b)4 and 120.569(2), F.S. and rule 28-106.201, F.A.C., a petition for administrative hearing shall contain the following information:

- a) The name, address, and telephone number of each petitioner, the name, address, and telephone number of the petitioner's representative, if any, the site owner's name and address, if different from the petitioner, the DEP facility number, and the name and address of the facility;
- b) A statement of how and when each petitioner received notice of the Department's action or proposed action;
- c) An explanation of how each petitioner's substantial interests are or will be affected by the Department's action or proposed action;
- d) A statement of the material facts disputed by the petitioner, or a statement that there are no disputed facts;
- e) A statement of the ultimate facts alleged, including a statement of the specific facts the petitioner contends warrant reversal or modification of the Department's action or proposed action;
- f) A statement of the specific rules or statutes the petitioner contends requires reversal or modification of the Department's action or proposed action; and
- g) A statement of the relief sought by the petitioner, stating precisely the action petitioner wishes the Department to take with respect to the Department's action or proposed action.

This Order is final and effective as of the date on the top of the first page of this Order. Timely filing a petition for administrative hearing postpones the date this Order takes effect until the Department issues either a final order pursuant to an administrative hearing or an Order Responding to Supplemental Information provided to the Department pursuant to meetings with the Department.

Judicial Review

Any party to this Order has the right to seek judicial review of it under section 120.68, F.S., by filing a notice of appeal under rule 9.110 of the Florida Rules of Appellate Procedure with the clerk of the Department in the Office of General Counsel, Mail Station 35, 3900 Commonwealth Boulevard, Tallahassee, Florida 32399-3000, and by filing a copy of the notice of appeal accompanied by the applicable filing fees with the

May 26, 2010

appropriate district court of appeal. The notice of appeal must be filed within thirty days after this order is filed with the clerk of the Department (see below).

Questions

Any questions regarding the Department's review of your postcertification amendment should be directed to Jessica Dalton at 850/245-2004. Questions regarding legal issues should be referred to the Department's Office of General Counsel at 850/245-2242. Contact with any of the above does not constitute a petition for administrative hearing or request for an extension of time to file a petition for administrative hearing.

Sincerely,

Michael P. Halpin, P.E.

Michael P. Halpin, P.E.
Administrator,
Siting Coordination Office

FILING AND ACKNOWLEDGMENT

FILED, on this date, pursuant to §120.52
Florida Statutes, with the designated
Department Clerk, receipt of which is
hereby acknowledged.

Essie Turner
Clerk

5-26-10
Date

North County Resource Recovery Facility

Page 5 of 5

May 26, 2010

Service List: CC by certified mail or email (return receipt requested)

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toni.sturtevant@dep.state.fl.us

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capercat@bellsouth.net

Jennifer Brubaker, Attorney Supervisor
Florida Public Service Commission
Office of General Counsel
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Kimberly Menchion, Esq.
Assistant General Counsel
Department of Transportation
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Tallahassee, FL 32399-0458
kimberly.menchion@dot.state.fl.us

LAKE BACKFILLING

Clearing, Grubbing and Stripping

Prior to backfilling the existing lake, the existing ground surface along the lake banks shall be thoroughly cleared and grubbed to remove existing vegetation and root zones, and stripped to remove topsoil or other soils having organic contents in excess of 2 percent by dry weight when determined in accordance with ASTM D 2974. The resulting stripped grades shall be inspected by the Owner-Engineer prior to placement of new fills.

Lake Backfill Materials

The existing lake shall be backfilled to a level that is three (3) feet above the prevailing surface water level that exists in the lake at the time of backfilling. Lake backfill materials shall consist of sands or sand and gravel (ASTM D 2487) having a maximum size of 1 inch and not more than 10 percent passing the U.S. Standard No. 200 Sieve. The lake backfill shall have an organic content of no more than 2 percent (ASTM D 2974).

Dewatering

Dewatering the lake prior to backfilling is not practical owing to its size and the potential for contaminant migration due to the proximity of the nearby landfill. Placement of the backfill in a wet condition will likely result in submerged fill slopes that are relatively flat, of the order of 4:1 to 5:1 (horizontal:vertical). The backfill shall be placed in a carefully controlled manner such that the surface water level in the lake does not rise more than 0.5 feet above its ambient level that exists at the onset of the backfilling operations.

Densification

Once the lake is backfilled to a level that is three feet above the prevailing water line, densification of the fill at this point will take the form of surface roller compaction using a heavy, self-propelled vibratory roller which imparts a dynamic drum force of not less than 55,000 pounds. The backfill shall be compacted using repeated, overlapping passes with the roller as it operates at its maximum vibrational frequency and a speed of 2 miles per hour. The compaction should continue until the subgrade is densified to not less than 95 percent of the maximum dry density determined in accordance with Modified Proctor (ASTM D 1557) to a depth corresponding to the prevailing groundwater surface. Subsequent lifts necessary to raise the site to proposed subgrade levels shall consist of embankment fill, similar to that described above for lake backfill, which is placed in lifts not exceeding 12 inches in loose thickness and compacted to at least 95 percent of the maximum dry density (ASTM D 1557).

Surcharge Fill

Utilization of the lake backfill method described previously will necessitate placement of surcharge fills over the lake backfill areas that are to support pavements. The surcharge should consist of granular soils similar to that described earlier for Lake Backfill, having a dry density of not less than 100 pounds per cubic foot. It should be placed to a level that is 5 feet above the required subgrade elevation in areas to be paved. The surcharge fill should be left in place for a minimum duration of 30 days. Economy may be gained with this method by utilizing the surcharge fill in a “rolling” manner (i.e. use of a constant volume of fill that is moved across the site).

The top of the surcharge fill should extend laterally to five horizontal feet beyond the outside lines of the areas to be paved. Surcharge fill should be constructed on a 2:1 (horizontal:vertical) slope. If space limitations do not allow for such slopes, then a temporary wall will be required (e.g. the Hilfiker welded wire wall, or equivalent). The design-build team will be responsible for design of the temporary wall system.

Settlement

Based upon assumed lake backfill thicknesses in the range of 20 to 25 feet associated with this project, we estimate that the submerged portion of the lake backfill will compress approximately 12 inches under the weight of the overlying fill. An additional 2 inches of settlement should be expected to occur as a result of the placement of 5 feet of surcharge fill. As most of this settlement will occur during fill placement, the long term effects of lake backfill compression should not be problematic. However, earthwork quantities should account for this magnitude of settlement.

Settlement Plates

Subsidence induced by the surcharge should be monitored using settlement plates that are surveyed by a professional surveyor/mapper. The settlement plates should conform to the requirements shown on the Florida Department of Transportation Design Standard Index No. 540. The frequency of reading the plates should be weekly for the duration of the surcharge. Accuracy of the readings should be 0.01 feet. Four settlement plates should be constructed at a level that is 3 feet above the ambient surface water elevation that exists in the lake at the onset of backfilling. The plates should be located within the areas of the site that are to be paved. The contractor should use hand labor and plate tamping equipment to place and compact the fill within 3 horizontal feet of the riser pipe.

Surcharge Removal

The removal of the surcharge will be determined by the Owner-Engineer with the following parameters involved in making this decision:

- a. The time rate of settlement during the fill placement and during the 30 day surcharge period shall depend on the shape of the time – settlement curve as evaluated by the Owner-Engineer.

- b. The settlement is not expected to continue beyond the 30 day surcharge period, however there is the potential for this to occur.
- c. Contractor should approach the settlement time issue by allowing a surcharge period of 60 days duration with an allowance for early removal of the surcharge after a minimum of 30 days, provided the time – settlement curve data supports this decision.

Vibro-Replacement

The structures under consideration here consist of the Facility Maintenance Area, Warehouse Storage, Facility Administration, Fire Water Storage Tank and Fire Pump House. Vibro-replacement consists of inserting a large vibratory probe into the ground to reinforce the soils by forming columns of gravel (i.e. stone columns) and to densify the sandy soils between stone columns. Crushed limestone should be employed as add fill in the vibro-replacement points over the intervals to be treated. Foundations resting on vibro-replacement treated profile components may be designed for a net allowable bearing pressure of 6,000 pounds per square foot for dead plus normal live loads.

The vibro-replacement operations should be accomplished with a 150-horsepower (minimum) vibroflot having a centrifugal force of not less than 22 tons. Treatment of the profile components should extend to depths equivalent to the full depth of the lake backfill, or twice the footing widths for column footings and four footing widths for continuous foundations, measured from the base of the footings, whichever is deeper. The treatment depth for the Fire Water Storage tank structure should be not less than 50 feet. Add fill used in the compaction points should consist of crushed limestone meeting the gradational requirements established for Number 57 coarse aggregate within the Florida Department of Transportation *Standard Specifications for Roadways and Bridges*.

Specifications prepared for the project should require that the granular soils over the treated interval afford an average cone penetrometer tip resistance of not less than 80 tons per square foot when tested anywhere in the compaction pattern.

Vibro-replacement operations should be performed by an experienced ground-improvement contractor using a plan that he prepares. The plan should utilize center-to-center spacings for the vibro-replacement points that are normally used in granular soils to achieve 80 percent relative density. Before adoption, the ground improvement plan should be reviewed by the geotechnical engineer and structural engineer for conformance with the intent of the foundation design.

Alternatively, it may be more cost effective to utilize sand backfill as the add fill for this work, a method known as vibro-compaction. If this method is utilized, the add fill should consist of relatively clean sands having not more than 5 percent passing the US Standard No. 200 Sieve, and a Suitability Number less than 20. The Suitability Number is quantified based upon the grain size distribution characteristics of the sand add fill. Treatment depths, required densification and design bearing pressures should be considered as those stated above for vibro-replacement.

EARTHWORK PLAN NOTES

Following are notes that apply to the lake backfill, surcharge fill and the vibro-replacement activities that are to be implemented for this project.

1. Lake backfill materials shall consist of sands or sand and gravel (ASTM D 2487) having a maximum size of 1 inch and not more than 10 percent passing the U.S. Standard No. 200 Sieve. The lake backfill shall have an organic content of no more than 2 percent (ASTM D 2974).
2. The lake backfill soils shall be placed to a level that is three (3) feet above the prevailing surface water level that exists in the lake at the time of backfilling. The backfill shall be placed in a carefully controlled manner such that the surface water level in the lake does not rise more than 0.5 feet above its ambient level that exists at the onset of the backfilling operations.
3. Densification of the lake backfill soils shall begin once the lake is backfilled to a level that is three feet above the prevailing water level. Densification of the fill shall consist of surface roller compaction using a heavy, self-propelled vibratory roller which imparts a dynamic drum force of not less than 55,000 pounds. The backfill shall be compacted using repeated, overlapping passes with the roller as it operates at its maximum vibrational frequency and a speed of 2 miles per hour. The compaction should continue until the subgrade is densified to not less than 95 percent of the maximum dry density determined in accordance with Modified Proctor (ASTM D 1557) to a depth corresponding to the prevailing groundwater surface.
4. Subsequent lifts (site grading fill) necessary to raise the site to proposed subgrade levels shall consist of embankment fill, similar to that described above for lake backfill, which is placed in lifts not exceeding 12 inches in loose thickness and compacted to at least 95 percent of the maximum dry density (ASTM D 1557).
5. The surcharge shall consist of granular soils having a moist density of not less than 100 pounds per cubic foot.
6. The surcharge fill shall be placed to a level that is 5 feet above the required pavement subgrade elevation, and shall be constructed with a top of bank that is 5 feet horizontally beyond the proposed edge of pavements.
7. Four settlement plates shall be installed by Contractor forces at an elevation that is 3 feet above the ambient surface water level in the lake that exists at the onset of backfilling. The settlement plates shall conform to the requirements shown on the Florida Department of Transportation Design Standard Index No. 540. The plates shall be located within the footprint of the areas of the site that are to be paved. The Contractor shall use hand labor and plate tamping equipment to place and compact the fill within 3 feet horizontal feet of the riser pipe.

8. The surcharge shall be left in place for a minimum period of 30 days. Subsidence induced by the surcharge should be monitored by Contractor forces using settlement plates that are surveyed by a professional surveyor mapper. The settlement plate readings shall begin coincident with the installation of the settlement plates. The frequency of reading the plates shall be every 5 vertical feet of fill (site grading fill plus surcharge), or weekly, whichever comes first, for the duration of the surcharge, with settlement monitoring reports submitted to the Owner-Engineer as they are obtained. The readings shall have an accuracy of 0.01 feet.
9. The removal of the surcharge will be determined by the Engineer with the following parameters involved in making this decision:
 - The time rate of settlement during the fill placement and during the 30 day surcharge period shall depend on the shape of the time – settlement curve as evaluated by the Engineer.
 - The settlement is not expected to continue beyond the 30 day surcharge period, however there is the potential for this to occur.
 - Contractor shall approach the settlement time issue by allowing a surcharge period of 60 days duration with an allowance for early removal of the surcharge after a minimum of 30 days, provided the time – settlement curve data supports this decision.
10. Building footprint areas, namely the Facility Maintenance Area, Warehouse Storage-Facility, Administration building and Fire Water Storage Tank areas, shall be treated with ground improvement taking the form of vibro-replacement. The treatment shall extend to depths equivalent to the full depth of the lake backfill, or twice the footing widths for column footings and four footing widths for continuous foundations, measured from the base of the footings, whichever is deeper. For the Fire Water Storage tank structure, the treatment shall extend to a depth of 50 feet. Crushed limestone shall be employed as add fill in the vibro-replacement points over the intervals to be treated. Foundations resting on vibro-replacement treated profile components may be designed for a net allowable bearing pressure of 6,000 pounds per square foot (psf) for dead plus normal live loads.

The vibro-replacement operations shall be accomplished with a 150-horsepower (minimum) vibroflot having a centrifugal force of not less than 22 tons. Add fill used in the compaction points shall consist of crushed limestone meeting the gradational requirements established for Number 57 coarse aggregate within the Florida Department of Transportation *Standard Specifications for Roadways and Bridges*.

Granular soils over the treated interval afford an average cone penetrometer tip resistance of not less than 80 tons per square foot when tested anywhere in the compaction pattern.

Vibro-replacement operations shall be performed by an experienced ground-improvement contractor using a plan that he prepares. The plan shall utilize center-to-center spacings for the vibro-replacement points that are normally used in granular soils to achieve 80 percent relative density. Before adoption, the ground improvement plan shall be reviewed by the geotechnical engineer and structural engineer for conformance with the intent of the foundation design.

Alternative to vibro-replacement, the Contractor may select the use of sand backfill as the add fill for this work, a method known as vibro-compaction. Add fill for vibro-compaction shall consist of relatively clean sands having not more than 5 percent passing the US Standard No. 200 Sieve, and a Suitability Number less than 20. Depths of treatment and required densification shall be as stated above for vibro-replacement methodology. A design bearing pressure of 6,000 psf may be utilized for vibro-replacement methods.