Andreyev Engineering, Inc.

SANFORD OFFICE
4055 St. John's Parkway
Sanford, Florida 32771
407-330-7763
Fax: 407-330-7765

▼ Groundwater ▼ Environmental ▼ Geotechnical ▼ Construction Materials Testing

August 12, 2024
Project No.: GPGT-23-0138

To: **Kimley-Horn**
1700 SE 17th Street, Suite 200
Ocala FL 34471

Attention: **Mr. Alan J. Garri, PE**

Subject: **Report, Geotechnical Investigation for Ocala Water Treatment Plant (WTP) No. 2 Site, City of Ocala, Marion County, Florida**

Dear Mr. Garri:

As requested, Andreyev Engineering, Inc. (AEI) has completed a geotechnical investigation for the above referenced project. The geotechnical investigation at the site included all the proposed WTP related structures like Finished Water Ground Storage Tanks, Concentrate Ground Storage Tanks, NF Process Building, Chemical Storage Building, Pump Building and Electrical Buildings and stormwater areas at the site. This report presents the results of our geotechnical investigation along with an evaluation of the soil and groundwater conditions encountered at the location of the proposed structures and stormwater areas. An interim geotechnical report for the proposed 2.0 Million Gallon (MG) Finished Water Ground Storage Tank (GST) #1 to be located at the northeast corner of the site was submitted on April 9, 2024. This report covers recommendations for site preparation and foundation design and construction of the all the proposed structures at the site and the aquifer parameters for the stormwater pond area design.

SITE LOCATION AND PROJECT DESCRIPTION

The proposed Ocala WTP No. 2 expansion will be located on an undeveloped 43-acre site located approximately 500 feet west of the junction of S. Pine Street (US Highway 27) and SE 17th Avenue in Section 32, Township 15 South and Range 22 East. The proposed water treatment plant will include the following structures:

- Two (2) 2.0 MG capacity 100-ft diameter finished water ground storage tanks, Finished Water GST No. 1 and Finished Water GST No. 3.
- One (1) 4.0 MG capacity 140-ft diameter finished water ground storage tank, Finished Water GST No. 2.
- Two (2) 4.5 MG capacity 149-ft diameter concentrate ground storage tanks, Concentrate Water GST No. 1 and Concentrate GST No. 2.
- Concentrate Pump Station Building with 19’ x 38’ footprint.
- Nanofiltration (NF) Process Building with 125’ x 259’ footprint.
- Odor Scrubber Pad with 33’ x 85’ footprint
- Clearwell/Degasifiers with 60’ x 100’ footprint
- Electrical Building No.2 with 22’ x 39’ footprint.
- Chemical Storage and Metering Building with 90’ x 129’ footprint
- High Service Pump Station Room with 63’ x 98’ footprint
- Electrical Building No, 1 with 35’ x 98’ footprint

In addition to the water treatment plant structures, the investigation included two stormwater retention areas in the eastern and southern portions of the site. The existing ground surface at the site is gently sloping upwards towards the north with ground surface elevations ranging from about 100 ft-NAVD88 to 130 ft-NAVD88. The site was an undeveloped wooded area that has been mostly cleared as part of the site development. The site's vicinity is shown on a USGS topographic map on **Figure 1**. The N.R.C.S. web soil survey map of the site is shown on **Figure 2.**

SCOPE OF FIELD EXPLORATION

The scope of services for the entire site included drilling of fifty-nine (59) Standard Penetration Test (SPT) borings to a depth of 10 feet to 85 feet below the ground surface at the locations of the proposed structures at the site. At the locations of the proposed water retention areas, a total of eight (8) borings were drilled to a depth of 10 feet to 25 feet below the ground surface. The boring location plans are shown on **Figures 3A and 3B**. Representative portions of each soil stratum from the soil borings were packaged and sealed for transportation to our laboratory for further examination and visual classification.

SPT borings SB-33, SB-40 and SB-41 indicated raveled soil conditions with 'weight-of-rod' and 'weight-of-hammer' penetrations and as a result sixteen (16) Cone Penetration Tests (CPTs) were carried out to delineate the lateral extent of the raveled soil zones around those boring locations. The CPT sounding locations are shown in **Figures 3A and 3B**.

SOIL AND GROUNDWATER CONDITIONS

The soil types encountered at the boring locations are presented in the form of a soil profile on the attached **Figures 4A thru 4F.** The stratification presented on **Figures 4A thru 4F** is based on visual examination of the recovered soil samples, laboratory testing and the interpretation of the field logs by a geotechnical engineer.

In general, the borings encountered the following soil Strata:

- Light brown to brown fine sand to slightly silty fine sand (Stratum 1).
- Gray to light brown to dark brown silty to clayey fine sand (Stratum 2).
- Grayish green to green sandy clay to clay (Stratum 3).
- Brown to grayish green clayey fine sand to sandy clay with limestone and dolomitic limestone fragments (Stratum 4).
- Highly weathered limestone (lime silt) (Stratum 5).
- Weathered limestone with lime silt and silty clay (Stratum 6).
- Limestone (Stratum 7)

Standard Penetration Tests (SPTs) were carried out in general accordance with ASTM Standard D-1586. Closely spaced SPT tests with split barrel sampling were performed in the upper 10 feet, with successive tests carried out at 5-foot intervals thereafter. The SPT blow counts or "N-Values" are shown adjacent to the boring profiles on **Figures 4A thru 4F.** The "N" values have been empirically correlated with various soil properties and are considered to be indicative of the relative density of cohesionless soils and the consistency of cohesive material. The SPT boreholes were grouted and backfilled with additional bentonite and soil materials.

The thickness of the different soil strata encountered at the boring locations are shown on **Figures 4A thru 4F**. In general, the near surface fine sand to slightly silty fine sand layer (Stratum 1) extended to a depth of 2 to 4 feet below the ground surface and the relative density of the stratum was loose to medium dense. Medium dense to dense silty to clayey fine sand (Stratum 2) was encountered below the surficial fine sand and the thickness of the silty to clayey fine sand layer ranged from 4 feet to about 50 feet.

Grayish green sandy clay to clay (Stratum 3) was encountered below the Stratum 2 clayey fine sand layer. The SPT N-values in the Stratum 3 sandy clay to clay indicated stiff to very stiff consistency of the clayey layer. The thickness of the sandy clay to clay (Stratum 3) varied from 5 feet to about 40 feet.

Brown to grayish green clayey fine sand to sandy clay with limestone fragments (Stratum 4) was encountered below the Stratum 3 clayey sand to sandy clay. The thickness of the clayey fine sand to sandy clay with limestone fragments ranged between 5 to 15 feet. The SPT N-value in Stratum 4 ranged from 11 blow/foot to 50 blows for 6 inches of penetration.

Stratum 5 highly weathered limestone (limesilt) was encountered at SPT borings SB-33, SB-40 and SB-41 as shown in **Figure 4C**. At boring SB-33, Stratum 5 weathered limestone (limesilt) indicated 11 feet of raveled soil condition with 5 feet of (weight-of-rod) WR penetration and 6 feet of (weight-of-hammer) WH condition. At boring SB-41 the Stratum 5 layer indicated 3 feet of raveled soil with WH penetration.

Stratum 6 weathered limestone was encountered above the Stratum 7 limestone. The thickness of the weathered limestone (Stratum 6) ranged from 5 feet to about 25 feet. The SPT N-value in the Stratum 6 weathered limestone ranged between 15 blows/foot to 50 blows/foot.

Stratum 7 limestone was encountered at depths varying from 4 feet to 78 feet below the ground surface. The significant variation in the depths of Stratum 7 limestone is due to the karst topography of the site. The SPT N-values at the Stratum 7 limestone ranged from 54 blow/foot to 50 blows for 1 inch of penetration.

Correlation of the SPT-N values with relative density, unconfined compressive strength and consistency are provided in the following table:

Coarse-Grained Soils		**Fine Grained Soils**		
Penetration Resistance N (blows/ft)	Relative Density of Sand	Penetration Resistance N (blows/ft)	Unconfined Compressive Strength of Clay (tons/ft^2)	Consistency of Clay
0-4	Very Loose	<2	<0.25	Very Soft
4-10	Loose	2-4	0.25-0.50	Soft
10-30	Medium-Dense	4-8	0.50-1.00	Medium
30-50	Dense	8-15	1.00-2.00	Stiff
>50	Very Dense	15-30	2.00-4.00	Very Stiff
		>30	>4.00	Hard

SPT borings SB-33, SB-40 and SB-41 indicated raveled soil conditions with 'weight-of-rod' and 'weight-of-hammer' penetrations and as a result sixteen (16) Cone Penetration Tests (CPTs) were carried out to further delineate the lateral extent of the raveled soil zones around those boring locations. The CPT sounding locations are shown in **Figures 3A and 3B**. Cone Penetration Tests (CPTs) were carried out in general accordance with ASTM Standard D-5778. Cone Penetration Testing with Pore Pressure Measurement (CPTU) is a geotechnical investigation technique designed to evaluate subsurface conditions and geotechnical soil properties. Cone penetrometer tests are a quasi-static penetration test, meaning that the cone is pushed at a slow rate rather than driven with a hammer or rotary drilling. During a cone penetration test (CPT), a cylindrical metal cone is advanced below land surface at a constant and slow rate, normally by a hydraulic press. As the cone is advanced, computerized measurements are made and data is recorded that indicate the various soil properties encountered by the cone.

The CPT is designed to evaluate subsurface conditions based primarily on the resistance to penetration encountered by the cone tip. Resistance measurements are also recorded for the cone sleeve, or shaft. In the case of piezocones, subsurface pore pressure are also measured to assist the evaluation of soil types. The CPT is performed by continuously advancing the cone without withdrawing it from the borehole. This makes a CPT very time-effective when compared to other testing procedures such as Standard Penetration Test (SPT) where the split-spoon sampler must be withdrawn from the borehole at each test interval.

The CPT provides data that can be used to estimate various subsurface properties including soil type and strength. Piezocone penetrometer tests are highly effective for identifying sand, silt, and clay layers, as well as determining pore pressure. These tests are also moderately effective for determining other geotechnical engineering properties including friction angle, undrained shear strength, relative density, constrained modulus, coefficient of consolidation, permeability, horizontal stress, and over consolidation ratios. The CPT sounding locations are shown on **Figures 3A and 3B** and the CPT summary plots using the CPT interpretation software CPeT-IT are shown in **Attachment A.**

Cone Penetration Tests (CPTs) CPT-1 thru CPT-6 were performed around SPT boring SB-33 to delineate the lateral extent of the raveled soil zones. The CPT soundings CPT-1 thru CPT-6 did not indicate the presence of raveled soil with low Constrained Modulus (M) values of say less than 20 tsf. Thus, the raveled soils encountered at SPT boring SB-33 are considered localized over a narrow area and appear to be indicative of the presence of a "Karst Chimney". At present, there are no structures proposed at the location of SB-33, however, if any structure is located at that location in the future, soil stabilization with pressure (cement) grouting will be essential to minimize the possibility of detrimental settlement. As shown in the CPT sounding summary plots in **Attachment A**, CPT soundings CPT-7, CPT-9 and CPT-10 around SPT boring SB-40 and CPT soundings CPT-12, CPT-15 and CPT-16 around SPT boring SB-41 indicated about 15 feet thick raveled soil zones with very low Constrained Modulus (M) values of less than 20 tsf. The very low Constrained Modulus (M) values indicate high compressibility like that of highly compressible peat (muck). At present, there are no structures proposed at the location of SB-40 and SB-41, however, if any structures are located at those locations in the future, soil stabilization with pressure (cement) grouting will be essential to minimize the possibility of detrimental settlement of the structures.

Groundwater Conditions

Groundwater was not encountered within the top ten (10) feet of the SPT borings at the proposed structure locations. Groundwater levels could not be measured below the 10-foot depth at the SPT borings due to the mud rotary drilling method, which uses a thick bentonite drilling slurry to maintain an open borehole.

Groundwater table was encountered at pond boring locations at depths of 16.7 feet to 24.2 feet below the ground surface. Based on the encountered subsurface conditions, nearby lake levels, our local experience, and antecedent rainfall conditions, the normal seasonal high groundwater level at the proposed structure locations is estimated to range in elevation from 80 to 90.0 feet (NAVD88). A temporary perched water table is expected to develop above the sandy clay to clay (Stratum 3)soil layers, after prolonged and intense rains.

NRCS Soil Survey

The U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS) Web Soil Map was reviewed. A portion of the map which depicts the location of the subject site is shown on attached **Figure 2**. The soil map units for the proposed site are: #44 Kendrick Loamy Sand, 0 to 5% slopes and #46 Lockloosa Fine Sand, 0 to 5% slopes. The depth of seasonal highwater table in #44 Kendrick Loamy Sand according to the web soil survey is at a depth greater than 200 cm (6.6 feet) below the ground surface and for #46 Lochloosa Fine Sand is at a depth of 95 cm (3.1 feet) below the ground surface. Soil units #44 Kendrick Sand and #46 Lochloosa Fine Sand are classified as Hydrologic Soil Group “A”, indicating soil with high infiltration rate.

Ground Penetrating Radar (GPR) Survey and Karst Sensitivity

The Ground Penetrating Radar (GPR) Survey of the site was performed by GeoView Inc. along the transect lines shown on **Figure 5**. The GPR survey report is presented in **Attachment B**. The survey did not encounter any “GPR anomaly” and thus the GPR survey did not indicate a potential for sinkhole activity at the site. The Florida Subsidence Incidents Report Map prepared by Florida Geological Survey is shown in **Figure 6**. The incidents report map indicates numerous previously reported sinkholes within 5-mile radius of the site. However, there is no reported sinkhole within ½-mile radius of the site.

Regional Geology

The geologic map of the State of Florida published by the Florida Geological Survey is shown on **Figure 7**. The project area is located in the Ocala Karst District geomorphic region (FDEP Open File Map Series 100, Plate 3). The near surface sediments in the project area are mapped as Middle Miocene Coosawhatchie Formation (Thc) of Hawthorn Group. The Coosawhatchie Formation varies from a light gray to olive gray, poorly consolidated, variably clayey and phosphatic sand to moderately consolidated slightly sandy, silty clay (Scott, 2001).

Ocala Limestone (To) underlies the surface layer of Coosawhatchie Formation at the project area. At the northern part of the Ocala WTP #2 site, Ocala Limestone outcrops at a few isolated locations and at several borings performed for the project, Ocala Limestone was encountered within about 5 feet below the ground surface. The Ocala Limestone consists of nearly pure limestone and occasional dolostones. The upper facies of Ocala Limestone are a white, poorly to well indurated, poorly sorted, very fossiliferous limestone (grainstone, packstone and wackestone). The permeable, highly transmissive carbonates of the Ocala Limestone form an important part of the Floridan Aquifer System (Miller, 1986).

Ocala Limestone is about 100 feet thick in the project area. Avon Park Formation (Tap) underlies the Ocala Limestone and extends to depths of over several hundred feet. Avon Park Formation consists of cream to light-brown or tan, poorly indurated to well indurated, variably fossiliferous limestone. Avon Park Formation is part of the Floridan Aquifer System.

EVALUATION AND RECOMMENDATIONS

General

The following conclusions and recommendations are based on the project characteristics previously described, the data obtained in our field exploration and laboratory testing, and our experience with similar subsurface conditions.

Based on the results of our study, we are of the opinion that the soil and groundwater conditions are suitable for construction of the proposed structures at the site using conventional shallow foundation design. However, it is recommended that a uniform densified engineered soil platform be prepared for the proposed 100 to 149-foot diameter ground storage tanks.

For the calculation of lateral earth pressures, we recommend use of the following parameters: γ_{moist} = 110 lb/ft^3, γ_{sat} = 120 lb/ft^3, c' = 0, ϕ' = 32^o; Rankine active earth pressure coefficient, K_a = 0.307, Rankine passive earth pressure coefficient, K_p = 3.25; and coefficient of lateral earth pressure at rest, K_o = 0.47. For the calculation of safety of foundations against sliding, we recommend the use of a coefficient of friction, f = 0.40.

For seismic design considerations, it may be noted that according to the USGS national seismic hazard map, Central Florida is mapped as a "lowest hazard" area. Based on the available SPT N-values from this investigation, the Seismic Site Classification according to IBC Site Classification system is "Site Class D: Stiff Soil".

Site Preparation

The initial step in site preparation should be the complete removal of all existing topsoil, trees, major root systems and other deleterious materials. Due to the loose soil conditions encountered near the ground surface, we recommend that the tank structure area (i.e., proposed area under the ground storage tanks) be over-excavated to a depth of 4 feet below the proposed foundation bottom elevation. The excavation shall extend a minimum of 5 feet outside the perimeter of the foundation of the structure. The depth of excavation at the building structure areas should be to a depth of 2 feet below the foundation depth. The exposed subgrade should then be proof rolled using a heavy vibratory roller (Dynapac CA-25 or equivalent). A non-vibratory roller should be used within 75 feet of any existing structure. Proofrolling of the tank foundation area should consist of at least ten (10) overlapping passes in each of two perpendicular directions and should be observed by a geotechnical engineer. The purposes of the proof rolling will be to detect any areas where unsuitable soils are present as well to densify the near-surface loose soils. The excavated Stratum 1 fine sand can be used as backfill material for the over-excavated area and the backfill shall be well compacted in uniform 12-inch thick lifts to a minimum of 95% of the soil's modified Proctor maximum dry density (ASTM D-1557).

Please note that at any time during mass grading, over-excavation or site preparation, if a cavity, sinkhole, or karst chimney feature is encountered, then AEI should be notified immediately to review the conditions. Small chimney features should be identified and filled with grout as quickly as possible after identification, to properly fill and seal any potential future surface connections to the subsurface.

Fill Placement

After the over-excavated foundation areas have been backfilled with well compacted clean sand and accepted by the geotechnical engineer, any fill required to bring the site to final grade may be placed and properly compacted. All fill materials should be inorganic, non-plastic, granular soil with less than 10% passing the number 200 sieve. The fill should be placed in level lifts not to exceed 12 inches loose and should be compacted to a minimum of 95 percent (%) of the soil's modified Proctor maximum dry density as determined by ASTM Standard D-1557. In-place density tests should be performed on each lift by an experienced engineering technician working under the direction of a licensed geotechnical engineer to verify that the recommended degree of compaction has been achieved. We suggest a minimum testing frequency of one (1) test per lift per 2,500 square feet of area within structural limits. The fill should extend a minimum of 5 feet beyond tank perimeter line to prevent possible erosion or undermining of foundation soils. Further, fill slopes should not exceed 2 horizontal to 1 vertical (2H:1V). For fill placed in restricted working areas, compaction should be accomplished with lightweight, hand-guided compaction equipment and lift thicknesses should be limited to a maximum of 4 inches loose thickness.

2.0 MG Finished Water Ground Storage Tank (GST) 1 Foundation

SPT borings SB-17, SB-53 and SB-54 were performed at the location of the proposed 100-foot diameter Finished Water ground storage tank GST #1. Based on our test boring results, the proposed tank can be supported by a shallow foundation system (reinforced slab/ring foundation). The ground storage tank foundation system should bear on properly placed and compacted cohesionless (sand) structural fill. As discussed in the site preparation recommendations above, after site stripping and grubbing, and prior to construction of the slab/footing system, over-excavation and replacement needs to be completed to ensure a 4-foot thick buffer (platform) of properly prepared and compacted engineered fill below the proposed bottom of foundation level, based on the elevations of the final grade. The excavated bottom areas should be improved by vibratory compaction, as described earlier in this report, to provide uniform subgrade conditions and densify the encountered subgrade soil. This is intended to limit the total and differential settlements of the tank. The backfill material consisting of clean sand with less than 10% fines may then be placed back and compacted in uniform 12-inch lifts. The Stratum 1 fine sand can be used as backfill. Any fill required to bring the tank foundation to final grade shall be properly compacted in accordance with the recommendations described earlier. Compaction operations should be controlled by the contractor so as to not adversely impact any adjacent structures.

We understand the proposed Finished Water GST #1 will be a 100-foot diameter prestressed concrete tank with a 4-inch-thick membrane floor. We understand that the finished floor level will be at elevation 124 ft- NAVD88. The existing ground surface at the tank location varies between EL 118.5 to 122 ft - NAVD88. With an average fill height of 3.75 feet and the tank load of 2,300 psf, the total foundation load of the tank and new fill is estimated to be 2,750 psf.

The settlement of the 2.0 MG Finished Water GST #1 at boring location SB-54 was calculated for (a) 33.5 feet of sandy soil, (b) consolidation settlement of 5.5 feet of Stratum 3 medium stiff sandy clay to clay between the depths of 8.0 feet and 13.5 feet and (3) consolidation settlement of 5 feet of Stratum 5 soft limesilt between the depths of 33.5 feet and 38.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 19.9 blows/foot. The settlement of the medium stiff sandy clay to clay (Stratum 3) between 8 to 13.5 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.06, OCR = 4.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The settlement of the Stratum 5 limesilt between 33.5 feet to 38.5 feet was calculated assuming: w_n = 35%, e_o = 0.95, C_c = 0.35, C_r = 0.07, OCR = 3.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the tank due to the sand and clay layers is 2.69 inches. Assuming that for clayey soils, the differential settlement is 50% of the calculated total settlement, the calculated differential settlement between the center and perimeter of the finished water tank GST #1 is 1.35 inches. With l = 50.0 feet (distance between center and edge of tank), the calculated angular distortion δ/l is (*1.35*/(50.0x12)) = 0.0022 = 1/446. The tolerable angular distortion of the membrane floor due to differential settlement is generally considered as δ/l = 1/300 and thus the calculated angular distortion is well within the tolerable limits for prestressed concrete tanks.

The prestressed concrete storage tank can be constructed directly on the compacted platform and if perimeter strip or wall foundations are used, the wall footings should be proportioned using a maximum net allowable uniform bearing pressure of 2,500 pounds per square feet. All wall footings (if used) should be embedded a minimum of 24 inches below adjacent compacted grade on both sides and should be a minimum of 3.0 feet in width. This minimum footing size should be used regardless of whether or not the allowable bearing pressure dictates a smaller size. For the design of the membrane floor of the prestressed concrete tank resting on well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Pipe grades and pipe connections within 20 feet of the tanks should be designed considering the expected settlement.

The settlement of the ground storage tank should be monitored during the initial filling of the tank. The tank manufacturer should incorporate settlement monitoring points to permit this operation. Initially, we recommend that 25% loading increments be utilized and held for say one week each until the tanks are 100% full. Monitoring of the settlement points will determine the actual loading frequency. Pipe connections to and under the tanks should be connected after the initial filling.

2.0 MG Finished Water Ground Storage Tank (GST) 3 Foundation

SPT borings SB-16, SB-55 and SB-56 were performed at the location of the proposed 100-foot diameter Finished Water ground storage tank GST #3. Based on our test boring results, the proposed tank can be supported by a shallow foundation system (reinforced slab/ring foundation). The ground storage tank foundation system should bear on properly placed and compacted cohesionless (sand) structural fill. As discussed in the site preparation recommendations above, after site stripping and grubbing, and prior to construction of the slab/footing system, over-excavation and replacement needs to be completed to ensure a 4-foot thick buffer (platform) of properly prepared and compacted engineered fill below the proposed bottom of foundation level, based on the elevations of the final grade. The excavated bottom areas should be improved by vibratory compaction, as described earlier in this report, to provide uniform subgrade conditions and densify the encountered subgrade soil. This is intended to limit the total and differential settlements of the tank. The backfill material consisting of clean sand with less than 10% fines may then be placed back and compacted in uniform 12-inch lifts. The Stratum 1 fine sand can be used as backfill. Any fill required to bring the tank foundation to final grade shall be properly compacted in accordance with the recommendations described earlier. Compaction operations should be controlled by the contractor so as to not adversely impact any adjacent structures.

We understand the proposed Finished Water GST #3 will be a 100-foot diameter prestressed concrete tank with a 4-inch-thick membrane floor. We understand that the finished floor level will be at elevation 124 ft- NAVD88. The existing ground surface at the tank location varies between EL 124 to 128 ft - NAVD88. With an average cut of 2 feet and tank load of 2,300 psf, the net total foundation load of the tank and the 2 ft of cut is estimated to be 2,060 psf.

The settlement of the 2.0 MG Finished Water GST #3 at boring location SB-55 was calculated for (a) 23.5 feet of sandy soil, (b) consolidation settlement of 5 feet of Stratum 3 stiff sandy clay to clay between the depths of 23.5 feet and 28.5 feet and (3) consolidation settlement of 10 feet of Stratum 3 stiff sandy clay to clay between the depths of 33.5 feet and 43.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 15.3 blows/foot. The settlement of the stiff sandy clay to clay (Stratum 3) between 23.5 to 28.5 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.03, OCR = 4.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The settlement of the Stratum 3 clayey sand to clay between 33.5 feet to 43.5 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.03, OCR = 3.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the tank due to the sand and clay layers is

1.23 inches. Assuming that for clayey soils, the differential settlement is 50% of the calculated total settlement, the calculated differential settlement between the center and perimeter of the finished water tank GST #3 is 0.62 inches. With *l* = 50.0 feet (distance between center and edge of tank), the calculated angular distortion δ/l is *(0.62*/(50.0x12)) = 0.0010 = 1/973. The tolerable angular distortion of the membrane floor due to differential settlement is generally considered as δ/l = 1/300 and thus the calculated angular distortion is well within the tolerable limits for prestressed concrete tanks.

The prestressed concrete storage tank can be constructed directly on the compacted platform and if perimeter strip or wall foundations are used, the wall footings should be proportioned using a maximum net allowable uniform bearing pressure of 2,500 pounds per square feet. All wall footings (if used) should be embedded a minimum of 24 inches below adjacent compacted grade on both sides and should be a minimum of 3.0 feet in width. This minimum footing size should be used regardless of whether or not the allowable bearing pressure dictates a smaller size. For the design of the membrane floor of the prestressed concrete tank resting on well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Pipe grades and pipe connections within 20 feet of the tanks should be designed considering the expected settlement.

The settlement of the ground storage tank should be monitored during the initial filling of the tank. The tank manufacturer should incorporate settlement monitoring points to permit this operation. Initially, we recommend that 25% loading increments be utilized and held for say one week each until the tanks are 100% full. Monitoring of the settlement points will determine the actual loading frequency. Pipe connections to and under the tanks should be connected after the initial filling

4.0 MG Finished Water Ground Storage Tank (GST) 2 Foundation

SPT borings SB-14, SB-15, SB-57, SB-58 and SB-59 were performed at the location of the proposed 140-foot diameter Finished Water ground storage tank GST #2. Based on our test boring results, the proposed tank can be supported by a shallow foundation system (reinforced slab/ring foundation). The ground storage tank foundation system should bear on properly placed and compacted cohesionless (sand) structural fill. As discussed in the site preparation recommendations above, after site stripping and grubbing, and prior to construction of the slab/footing system, over-excavation and replacement needs to be completed to ensure a 4-foot thick buffer (platform) of properly prepared and compacted engineered fill below the proposed bottom of foundation level, based on the elevations of the final grade. The excavated bottom areas should be improved by vibratory compaction, as described earlier in this report, to provide uniform subgrade conditions and densify the encountered subgrade soil. This is intended to limit the total and differential settlements of the tank. The backfill material consisting of clean sand with less than 10% fines may then be placed back and compacted in uniform 12-inch lifts. The Stratum 1 fine sand can be used as backfill. Any fill required to bring the tank foundation to final grade shall be properly compacted in accordance with the recommendations described earlier. Compaction operations should be controlled by the contractor so as to not adversely impact any adjacent structures.

We understand the proposed Finished Water GST #2 will be a 140-foot diameter prestressed concrete tank with a 4-inch-thick membrane floor. We understand that the finished floor level will be at elevation 124 ft- NAVD88. The existing ground surface at the tank location varies between EL 120.5 to 127.5 ft - NAVD88. With an average cut/fill heigh of 0 feet and tank load of 2,300 psf, the net total foundation load of the tank is estimated to be 2,300 psf.

The settlement of the 4.0 MG Finished Water GST #2 at boring location SB-59 was calculated for (a) 13.5 feet of sandy soil and (b) consolidation settlement of 20 feet of Stratum 3 stiff sandy clay to clay between the depths of 13.5 feet and 33.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 30.1 blows/foot. The settlement of the stiff sandy clay to clay (Stratum 3) between 13.5 to 33.5 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.03, OCR = 4.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the tank due to the sand and clay layers is 1.16 inches. Assuming that for clayey soils, the differential settlement is 50% of the calculated total settlement, the calculated differential settlement between the center and perimeter of the finished water tank GST #3 is 0.58 inches. With l = 70.0 feet (distance between center and edge of tank), the calculated angular distortion δ/l is (*0.58*/(70.0x12)) = 0.0007 = 1/1454. The tolerable angular distortion of the membrane floor due to differential settlement is generally considered as δ/l = 1/300 and thus the calculated angular distortion is well within the tolerable limits for prestressed concrete tanks.

The prestressed concrete storage tank can be constructed directly on the compacted platform and if perimeter strip or wall foundations are used, the wall footings should be proportioned using a maximum net allowable uniform bearing pressure of 2,500 pounds per square feet. All wall footings (if used) should be embedded a minimum of 24 inches below adjacent compacted grade on both sides and should be a minimum of 3.0 feet in width. This minimum footing size should be used regardless of whether or not the allowable bearing pressure dictates a smaller size. For the design of the membrane floor of the prestressed concrete tank resting on well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Pipe grades and pipe connections within 20 feet of the tanks should be designed considering the expected settlement.

The settlement of the ground storage tank should be monitored during the initial filling of the tank. The tank manufacturer should incorporate settlement monitoring points to permit this operation. Initially, we recommend that 25% loading increments be utilized and held for say one week each until the tanks are 100% full. Monitoring of the settlement points will determine the actual loading frequency. Pipe connections to and under the tanks should be connected after the initial filling

4.5 MG Concentrate Ground Storage Tank (GST) 1 Foundation

SPT borings SB-2, SB-3, SB-48 and SB-49, and CPT soundings CPT-3 and CPT-5 were performed at the location of the proposed 149-foot diameter Concentrate ground storage tank GST #1. Based on our test boring results, the proposed tank can be supported by a shallow foundation system (reinforced slab/ring foundation). The ground storage tank foundation system should bear on properly placed and compacted cohesionless (sand) structural fill. As discussed in the site preparation recommendations above, after site stripping and grubbing, and prior to construction of the slab/footing system, over-excavation and replacement needs to be completed to ensure a 4-foot thick buffer (platform) of properly prepared and compacted engineered fill below the proposed bottom of foundation level, based on the elevations of the final grade. The excavated bottom areas should be improved by vibratory compaction, as described earlier in this report, to provide uniform subgrade conditions and densify the encountered subgrade soil. This is intended to limit the total and differential settlements of the tank. The backfill material consisting of clean sand with less than 10% fines may then be placed back and compacted in uniform 12-inch lifts. The Stratum 1 fine sand can be used as backfill. Any fill required to bring the tank foundation to final grade shall be properly compacted in accordance with the recommendations described earlier. Compaction operations should be controlled by the contractor so as to not adversely impact any adjacent structures.

We understand the proposed Concentrate GST #1 will be a 149-foot diameter prestressed concrete tank with a 4-inch-thick membrane floor. We understand that the finished floor level will be at elevation 118 ft- NAVD88. The existing ground surface at the tank location varies between EL 117 to 122 ft -

NAVD88. With an average cut of 1.5 feet and tank load of 2,300 psf, the net total foundation load of the tank will be 2,120 psf.

The settlement of the 4.5 MG Concentrate GST #1 at boring location SB-2 was calculated for (a) 18.5 feet of sandy soil and (b) consolidation settlement of 15 feet of Stratum 3 stiff sandy clay to clay between the depths of 18.5 feet and 33.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 23 blows/foot. The settlement of the very stiff sandy clay to clay (Stratum 3) between 18.5 to 33.5 feet was calculated assuming: w_n = 25%, e_o = 0.675, C_c = 0.25, C_r = 0.025, OCR = 5.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the tank due to the sand and clay layers is 0.98 inches. Assuming that for clayey soils, the differential settlement is 50% of the calculated total settlement, the calculated differential settlement between the center and perimeter of the Concentrate GST #1 is 0.49 inches. With l = 74.5 feet (distance between center and edge of tank), the calculated angular distortion δ/l is *(0.49*/(74.5x12)) = 0.00055 = 1/1817. The tolerable angular distortion of the membrane floor due to differential settlement is generally considered as δ/l = 1/300 and thus the calculated angular distortion is well within the tolerable limits for prestressed concrete tanks. Settlement of the GST #1 was also calculated based on the CPT sounding results at CPT-3 and CPT-5 using the Cone Penetration Analysis Software CPeT. The output of the settlement calculation is shown in **Attachment A.** The CPeT calculated settlement of GST #1 at CPT-5 is 2.16 inches. Assuming that for clayey soils, the differential settlement is 50% of the calculated total settlement, the CPeT calculated differential settlement between the center and perimeter of the Concentrate GST #1 is 1.08 inches. With l = 74.5 feet (distance between center and edge of tank), the calculated angular distortion δ/l is *(1.08*/(74.5x12)) = 0.0012 = 1/828. The tolerable angular distortion of the membrane floor due to differential settlement is generally considered as δ/l = 1/300 and thus the calculated angular distortion is well within the tolerable limits for prestressed concrete tanks.

The prestressed concrete storage tank can be constructed directly on the compacted platform and if perimeter strip or wall foundations are used, the wall footings should be proportioned using a maximum net allowable uniform bearing pressure of 2,500 pounds per square feet. All wall footings (if used) should be embedded a minimum of 24 inches below adjacent compacted grade on both sides and should be a minimum of 3.0 feet in width. This minimum footing size should be used regardless of whether or not the allowable bearing pressure dictates a smaller size. For the design of the membrane floor of the prestressed concrete tank resting on well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Pipe grades and pipe connections within 20 feet of the tanks should be designed considering the expected settlement.

The settlement of the ground storage tank should be monitored during the initial filling of the tank. The tank manufacturer should incorporate settlement monitoring points to permit this operation. Initially, we recommend that 25% loading increments be utilized and held for say one week each until the tanks are 100% full. Monitoring of the settlement points will determine the actual loading frequency. Pipe connections to and under the tanks should be connected after the initial filling

4.5 MG Concentrate Ground Storage Tank (GST) 2 Foundation

SPT borings SB-35, SB-36, SB-50, SB-51 and SB-52 were performed at the location of the proposed 149-foot diameter Concentrate ground storage tank GST #2. Based on our test boring results, the proposed tank can be supported by a shallow foundation system (reinforced slab/ring foundation). The ground storage tank foundation system should bear on properly placed and compacted cohesionless (sand) structural fill. As discussed in the site preparation recommendations above, after site stripping and grubbing, and prior to construction of the slab/footing system, over-excavation and replacement needs to be completed to ensure a 4-foot thick buffer (platform) of properly prepared and compacted engineered fill below the proposed bottom of foundation level, based on the elevations of the final grade. The excavated bottom areas should be improved by vibratory compaction, as described earlier in this report, to provide uniform subgrade conditions and densify the encountered subgrade soil. This is intended to limit the total and differential settlements of the tank. The backfill material consisting of clean sand with less than 10% fines may then be placed back and compacted in uniform 12-inch lifts. The Stratum 1 fine sand can be used as backfill. Any fill required to bring the tank foundation to final grade shall be properly compacted in accordance with the recommendations described earlier. Compaction operations should be controlled by the contractor so as to not adversely impact any adjacent structures.

We understand the proposed Concentrate GST #2 will be a 149-foot diameter prestressed concrete tank with a 4-inch-thick membrane floor. We understand that the finished floor level will be at elevation 118 ft- NAVD88. The existing ground surface at the tank location varies between EL 113 to 116.5 ft - NAVD88. With an average fill height of 3.25 feet and tank load of 2,300 psf, the net total foundation load of the tank is estimated to be 2,690 psf.

The settlement of the 4.5 MG Concentrate GST #2 at boring location SB-36 was calculated for (a) 28.5 feet of sandy soil, (b) consolidation settlement of 5 feet of Stratum 3 stiff sandy clay to clay between the depths of 28.5 feet and 33.5 feet and (3) consolidation settlement of 16.5 feet of Stratum 3 stiff sandy clay to clay between the depths of 43.5 feet to 60 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sandy layers as 19.9 blows/foot. The settlement of the stiff sandy clay to clay (Stratum 3) between 28.5 to 33.5 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.06, OCR = 4.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The settlement of the stiff sandy clay to clay (Stratum 3) between 43.5 to 60.0 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.03, OCR = 4.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the tank due to the sand and clay layers is 1.42 inches. Assuming that for clayey soils, the differential settlement is 50% of the calculated total settlement, the calculated differential settlement between the center and perimeter of the Concentrate GST #1 is 0.81 inches. With l = 74.5 feet (distance between center and edge of tank), the calculated angular distortion δ/l is *(0.71*/(74.5x12)) = 0.0008 = 1/1267. The tolerable angular distortion of the membrane floor due to differential settlement is generally considered as δ/l = 1/300 and thus the calculated angular distortion is well within the tolerable limits for prestressed concrete tanks.

The prestressed concrete storage tank can be constructed directly on the compacted platform and if perimeter strip or wall foundations are used, the wall footings should be proportioned using a maximum net allowable uniform bearing pressure of 2,500 pounds per square feet. All wall footings (if used) should be embedded a minimum of 24 inches below adjacent compacted grade on both sides and should be a minimum of 3.0 feet in width. This minimum footing size should be used regardless of whether or not the allowable bearing pressure dictates a smaller size. For the design of the membrane floor of the prestressed concrete tank resting on well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Pipe grades and pipe connections within 20 feet of the tanks should be designed considering the expected settlement.

The settlement of the ground storage tank should be monitored during the initial filling of the tank. The tank manufacturer should incorporate settlement monitoring points to permit this operation. Initially, we recommend that 25% loading increments be utilized and held for say one week each until the tanks are 100% full. Monitoring of the settlement points will determine the actual loading frequency. Pipe connections to and under the tanks should be connected after the initial filling

Concentrate Pump Station Foundation

SPT boring SB-34 was drilled at the location of the proposed Concentrate Pump Station Building. Based on the available information, the proposed Concentrate Pump Station structure will have a plan area of 19' x 38' and a distributed foundation loading of 2,500 psf.

The settlement of the proposed mat foundation of the Concentrate Pump Station at boring location SB-34 was calculated for (a) 6 feet of sandy soil and (b) consolidation settlement of 7.5 feet of Stratum 3 stiff sandy clay to clay between the depths of 6 feet and 13.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 12 blows/foot. The settlement of the stiff sandy clay to clay (Stratum 3) between 6 to 13.5 feet was calculated assuming: w_n = 25%, e_o = 0.675, C_c = 0.25, C_r = 0.025, OCR = 5.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the foundation due to the sand and clay layer is 0.93 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement is estimated to be less than 1" and differential settlement of less than 3/4 inch. The allowable bearing capacity of the mat foundation for the Concentrate Pump Station is estimated to be 5,000 psf and thus exceeds the applied load of 2,500 psf. We recommend that the mat foundation for the Concentrate Pump Station should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

NF Process Building Foundation

SPT boring SB-9 thru SB-11, SB-24 thru SB-27 and SB-46 were drilled at the location of the proposed NF Process Building. Based on the available information, the proposed NF Process Building structure will have a plan area of 125' x 259' and a distributed foundation loading of 2,500 psf. We understand that the finished floor level will be at elevation 122 ft- NAVD88. The existing ground surface at the proposed NF Process Building location varies between EL 116 to 125 ft -NAVD88. With an average fill height of 1.5 feet and distributed building load of 2,500 psf, the net total foundation load is estimated to be 2,680 psf.

The settlement of the proposed mat foundation of the NF Process Building at boring location SB-24 was calculated for (a) 23.5 feet of sandy soil and (b) consolidation settlement of 20 feet of Stratum 3 stiff sandy clay to clay between the depths of 23.5 feet and 43.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 14.4 blows/foot. The settlement of the stiff sandy clay to clay (Stratum 3) between 23.5 to 43.5 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.03, OCR = 4.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the foundation due to the sand and clay layer is 1.64 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement of the mat foundation is estimated to be less than the allowable settlement of 2 inches and differential settlement is expected to be less than 1 inch. The allowable bearing capacity of the mat foundation for the NF Process Building is estimated to be 5,000 psf and thus exceeds the applied load of 2,680 psf. We recommend that the mat foundation for the NF Process Building should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Odor Scrubber Pad Foundation

SPT boring SB-44 was drilled at the location of the proposed Odor Scrubber Pad. Based on the available information, the proposed Odor Scrubber Pad structure will have a plan area of 33' x 85' and a distributed foundation loading of 2,500 psf. We understand that the finished floor level will be at elevation 122 ft- NAVD88. The existing ground surface at the proposed Odor Scrubber Pad location varies between EL 127.5 to 129.5 ft -NAVD88. With an average cut of 6.5 feet, the net total foundation load is estimated to be 1,720 psf.

The settlement of the proposed mat foundation of the Odor Scrubber Pad at boring location SB-44 was calculated for 50 feet of sandy soil between the depths of 0 feet and 50 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers within the zone of influence of 23.5 blows/foot. The calculated total settlement of the foundation due to the sand layers is 0.12 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement is estimated to be less than 1" and differential settlement of less than 3/4 inch. The allowable bearing capacity of the mat foundation for the Odor Scrubber Pad is estimated to be 5,000 psf and thus exceeds the applied load of 1,720 psf. We recommend that the mat foundation for the Odor Scrubber Pad should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Clear Well/Degasifiers Foundation

SPT borings SB-5, SB-43 and SB-45 were drilled at the location of the proposed Clear Well/Degasifiers Structure. Based on the available information, the proposed Clear Well/Degasifiers structure will have a plan area of 60' x 100' and a distributed foundation loading of 3,000 psf. We understand that the finished floor level will be at elevation 122.5 ft- NAVD88. The existing ground surface at the proposed Clear Well/Degasifiers varies between EL 124.5 to 129 ft -NAVD88. With an average cut of 4.25 feet and distributed structure load of 3,000 psf, the net total foundation load is estimated to be 2,490 psf.

The settlement of the proposed mat foundation of the Clear Well/Degasifiers at boring location SB-43 was calculated for (a) 18.5 feet of sandy soil and (b) consolidation settlement of 31.5 feet of Stratum 3 stiff sandy clay to clay between the depths of 18.5 feet and 50.0 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 24.3 blows/foot. The settlement of the very stiff sandy clay to clay (Stratum 3) between 18.5 to 50.0 feet was calculated assuming: w_n = 25%, e_o = 0.67, C_c = 0.25, C_r = 0.025, OCR = 5.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the foundation due to the sand and clay layer is 1.47 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement of the mat foundation is estimated to be less than the allowable settlement of 2 inches and differential settlement is expected to be less than 1 inch. The allowable bearing capacity of the mat foundation for the Clearwell/Degasifiers is estimated to be 5,000 psf and thus exceeds the applied load of 2,490 psf. We recommend that the mat foundation for the Clear Well/Degasifiers should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Electrical Building No, 2 Foundation

SPT boring SB-6 was drilled at the location of the proposed Electrical Building 2. Based on the available information, the proposed Electrical Building 2 will have a plan area of 22' x 39' and a distributed foundation loading of 2,500 psf. We understand that the finished floor level will be at elevation 122.5 ft-NAVD88. The existing ground surface at the proposed Electrical Building 2 varies between EL 123.5 to 125 ft -NAVD88. With an average cut of 1.75 feet and distributed structure load of 2,500 psf, the net total foundation load is estimated to be 2,290 psf.

The settlement of the proposed mat foundation of the Electrical Building 2 at boring location SB-6 was calculated for (a) 33.5 feet of sandy soil and (b) consolidation settlement of 5 feet of Stratum 3 stiff sandy clay to clay between the depths of 33.5 feet and 38.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layer within the influence zone as 22 blows/foot. The settlement of the hard sandy clay to clay (Stratum 3) between 33.5 to 38.5 feet was calculated assuming: w_n = 15%, e_o = 0.40, C_c = 0.15, C_r = 0.015, OCR = 8.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the foundation due to the sand and clay layer is 0.22 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement of the mat foundation for the Electrical Building 2 is estimated to be less than the allowable settlement of 2 inches and differential settlement is expected to be less than 1 inch. The allowable bearing capacity of the mat foundation for Electrical Building No, 2 is estimated to be 5,000 psf and thus exceeds the applied load of 2,290 psf. We recommend that the mat foundation for Electrical Building 2 should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Chemical Storage and Metering Building Foundation

SPT borings SB-29, SB-30 and SB-42 were drilled at the location of the proposed Chemical Storage and Metering Building. Based on the available information, the proposed Chemical Storage and Metering Building will have a plan area of 90' x 129' and a distributed foundation loading of 2,500 psf. We understand that the finished floor level will be at elevation 121 ft- NAVD88. The existing ground surface at the proposed Chemical Storage and Metering Building varies between EL 119 to 124.5 ft -NAVD88. With an average cut of 0.75 feet and distributed structure load of 2,500 psf, the net total foundation load is estimated to be 2,410 psf.

The settlement of the proposed mat foundation of the Chemical Storage and Metering Building at boring location SB-29 was calculated for (a) 18.5 feet of sandy soil and (b) consolidation settlement of 15 feet of Stratum 3 stiff sandy clay to clay between the depths of 18.5 feet and 33.5.5 feet and (3) consolidation settlement of 11.5 feet of Stratum 3 stiff sandy clay to clay between the depths of 43.5 feet to 60 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layers as 17.1 blows/foot within the zone of influence. The settlement of the stiff sandy clay to clay (Stratum 3) between 18.5 to 33.5 feet was calculated assuming: w_n = 25%, e_o = 0.675 C_c = 0.25, C_r = 0.025, OCR = 5.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The settlement of the medium sandy clay to clay (Stratum 3) between 38.5 to 50.0 feet was calculated assuming: w_n = 30%, e_o = 0.81, C_c = 0.30, C_r = 0.03, OCR = 4.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the Chemical Storage and Metering Building foundation due to the sand and clay layers is 1.77 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement of the mat foundation for the Chemical Storage and Metering Building is estimated to be less than the allowable settlement of 2 inches and differential settlement is expected to be less than 1 inch. The allowable bearing capacity of the mat foundation for the Chemical Storage and Metering Building is estimated to be 5,000 psf and thus exceeds the applied load of 2,410 psf. We recommend that the mat foundation for the Chemical Storage and Metering Building should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

High Service Pump Station Room

SPT boring SB-18, SB-19 and SB-22 were drilled at the location of the proposed High Service Pump Station Room. Based on the available information, the proposed High Service Pump Station Room will have a plan area of 63' x 98' and a distributed foundation loading of 2,500 psf. We understand that the finished floor level will be at elevation 122 ft- NAVD88. The existing ground surface at the proposed High Service Pump Station Room varies between EL 115 to 117.5 ft -NAVD88. With an average fill height of 5.75 feet and distributed structure load of 2,500 psf, the net total foundation load is estimated to be 3,190 psf.

The settlement of the proposed mat foundation of the High Service Pump Station Room at boring location SB-18 was calculated for (a) 4 feet of sandy soil and (b) consolidation settlement of 9.5 feet of Stratum 3 very stiff sandy clay to clay between the depths of 4 feet and 13.5 feet. The settlement of the sandy layers was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layer as 17.25 blows/foot. The settlement of the very stiff sandy clay to clay (Stratum 3) between 4 to 13.5 feet was calculated assuming: $w_n = 20\%$, $e_o = 0.54$, $C_c = 0.20$, $C_r = 0.02$, OCR = 6.0, $C_v = 0.1$ ft^2/day and $C_\alpha/C_c = 0.04$. The calculated total settlement of the foundation due to the sand and clay layer is 1.27 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement of the mat foundation for the High Service Pump Station Room is estimated to be less than the allowable settlement of 2 inches and differential settlement is expected to be less than 1 inch. The allowable bearing capacity of the mat foundation for the High Service Pump Station is estimated to be 5,000 psf and thus exceeds the applied load of 3,190 psf. We recommend that the mat foundation for the High Service Pump Station Room should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Electrical Building No, 1 Foundation

SPT boring SB-20, SB-21 and SB-47 were drilled at the location of the proposed Electrical Building No. 1. Based on the available information, the proposed Electrical Building 1 will have a plan area of 35' x 98' and a distributed foundation loading of 2,500 psf. We understand that the finished floor level will be at elevation 122 ft- NAVD88. The existing ground surface at the proposed Electrical Building 1 varies between EL 111 to 115.5 ft -NAVD88. With an average fill height of 8.75 feet and distributed structure load of 2,500 psf, the net total foundation load is estimated to be 3,550 psf.

The settlement of the proposed mat foundation of the Electrical Building 1 at boring location SB-47 was calculated for (a) 4 feet of sandy soil and (b) consolidation settlement of 39.5 feet of Stratum 3 stiff sandy clay to clay between the depths of 4 feet and 43.5 feet. The settlement of the sandy layer was calculated using Burland and Burbridge (1985) method, with average SPT N-value, N_{avg} of the sand layer as 17.25 blows/foot. The settlement of the stiff sandy clay to clay (Stratum 3) between 4 to

43.5 feet was calculated assuming: w_n = 25%, e_o = 0.68, C_c = 0.25, C_r = 0.025, OCR = 5.0, C_v = 0.1 ft^2/day and C_α/C_c = 0.04. The calculated total settlement of the foundation due to the sand and clay layer is 1.93 inches.

Assuming our recommendations of over-excavation of 2 feet and replacement with compacted fill described under site preparation section are followed, post-construction total settlement of the mat foundation for the Electrical Building 1 is estimated to be less than the allowable settlement of 2 inches and differential settlement is expected to be less than 1 inch. The allowable bearing capacity of the mat foundation for Electrical Building No. 1 is estimated to be 5,000 psf and thus exceeds the applied load of 3,550 psf. We recommend that the mat foundation for Electrical Building No. 1 should be embedded a minimum of 12 inches below the compacted finished grade. For the design of the mat foundation resting on a well compacted platform, we recommend a modulus of subgrade reaction, K value of 150 pci (lb/in^3).

Stormwater Pond Assessment

A total of eight (8) borings were performed at the proposed two (2) locations of the stormwater retention ponds. Borings DRA-101 and DRA-102 were performed at the proposed East Pond and borings DRA-103 thru DRA-106, AB-105 Rev and AB-106 Rev were performed at the proposed South Pond. The soil profiles at the pond borings are shown on **Figure 4F**. Based on the results of the boring and laboratory tests, the subsoil conditions at the proposed locations are considered suitable for the construction of the stormwater retention ponds. However, we recommend relocation of the proposed South Pond to exclude the areas around borings DRA-105 and DRA-106 where the Stratum 3 low permeability (aquiclude) soil was encountered starting from near the existing ground surface.

Groundwater table at the proposed pond locations was encountered at depths varying from 16.7 feet to 24.2 feet below the existing ground surface with groundwater elevations of about 76.8 to 90.9 feet (NAVD88). However, a temporary perched water table can develop above the Statum 3 clayey sand to clay layer. The Stratum 3 clayey sand to clay indicated fines content (-200) of 55% to 71% and from pond recovery consideration, Stratum 3 is considered to behave as an aquiclude. The seasonal high-water level (SHWL) at the pond boring locations is estimated to be about 1.5 ft higher than the measured elevations. A total of six (6) laboratory falling head permeability tests were performed on tube samples collected from a depth of 2.5 to 3 feet below the ground surface at the pond boring locations. The permeability test results are shown on **Figure 4F**. For recovery analyses of the proposed stormwater ponds, we recommend the following aquifer parameters based the results of the field and laboratory investigations, adjusting for depth and soil variability:

Aquifer Parameters for Pond Recovery Analyses, Ocala WTP 2 Site

Parameter	**Pond Boring DRA-101**	**Pond Boring DRA-102**	***Average of East Pond***	**Pond Boring DRA-103**	**Pond Boring DRA-104**	**Pond Boring DRA-105**	**Pond Boring DRA-106**	**Pond Boring AB-105 Rev**	**Pond Boring AB-106 Rev**	***Average of South Pond***
Bottom of Aquifer Elevation (ft- NAVD 88)	**88.5**	**88.0**	***88.3***	**78.7**	**94.5**	**100.3**	**101.7**	**83.4**	**83.2**	***85.0***
Weighted Average Unsaturated Vertical Hydraulic Conductivity (ft/day)	**4.0**	**4.3**	***4.2***	**7.0**	**3.2**	**9.6**	**0**	**5.0**	**4.6**	***5.0***

Weighted Average Saturated Horizontal Hydraulic Conductivity (ft/day)	**9.0**	**10.0**	***9.5***	**18.7**	**7.5**	**21.5**	**0**	**12.3**	**10.2**	***12.2***
Seasonal High Groundwater Elevation (ft- NAVD 88)	**87.7**	**92.4**	***90.1***	**88.5**	**94.5**	**100.3**	**101.7**	**83.4**	**83.2**	***87.4***
Soil Storage Coefficient	**0.20**	**0.20**	***0.20***	**0.20**	**0.20**	**0.20**	**0.0**	**0.20**	**0.20**	***0.20***

Factors of safety have not been applied to the above weighted average permeability values. For recovery analysis in accordance with water management district rules, a factor of safety of 2 should be applied to the unsaturated vertical permeability to account for long-term performance and siltation of the pond bottom.

To calculate the weighted average permeability values, the saturated vertical hydraulic conductivity (permeability) of Stratum 1 fine sand to slightly silty fine has been assumed to be an average value of 14.7 ft/day, Stratum 2 silty to clayey fine sand has been assumed to have an average saturated vertical permeability of 6.8 ft/day, Stratum 3 sandy clay to clay with fines content (-200) of 55% to 71% has been assumed to behave as an aquiclude from pond recovery consideration and Stratum 4 clayey fine sand to sandy clay with limestone fragments has been assumed to have a permeability of 2 ft/day. Due to presence of Stratum 3 aquiclude near the ground surface at borings DRA-105 and DRA-106, we understand that the location of the proposed South Pond will be relocated away from the DRA-105 and DRA-106 boring locations and these two borings were excluded from the average aquifer parameter calculations for the South Pond. The unsaturated vertical permeability has been assumed to be 2/3 of the measured saturated vertical permeability and saturated horizontal permeability has been assumed to be 1.5 times of the measured saturated vertical permeability. The following formulas were used in the calculation of both the weighted average vertical and horizontal weighted average permeability values.

Weighted Average Vertical Permeability = $$\frac{\sum L}{\frac{L_1}{Kv_1}+\frac{L_2}{Kv_2}+\frac{L_3}{Kv_3}+.........\frac{L_n}{Kv_n}}$$

Weighted Average Horizontal Permeability = $$\frac{Kh_1.L_1+Kh_2.L_2+Kh_3.L_3+....Kh_n.L_n}{\sum L}$$

LIMITATIONS

The geotechnical exploration and recommendations submitted herein are based on the data obtained from the soil borings and CPT soundings presented on **Figures 4A through Figure 4F and Attachment A**. The report does not reflect any variations which may occur between, adjacent to or away from the boring. If the locations of the structures are changed, then additional evaluation and geotechnical investigation may be necessary. Over-excavation to a depth of 4 feet and backfilling with well compacted structural fill has been recommended for the foundation of the proposed ground storage tanks. It should be noted that localized raveled soil conditions at other locations of the site cannot be ruled out due to the karst topography of the site. Also, it is essential to apply the observational approach and monitor structure areas (both during construction and after construction) on a regular basis for any signs of distress. Any future signs of distress should be reported, carefully evaluated, and additional investigations may be necessary to repair and stabilize structures and their subsurface conditions.

CLOSURE

AEI appreciates the opportunity to participate in this project, and we trust that the information herein is sufficient for your present needs. If you have any questions or comments concerning the contents of this report, please do not hesitate to contact our office.

Sincerely,

ANDREYEV ENGINEERING, INC.

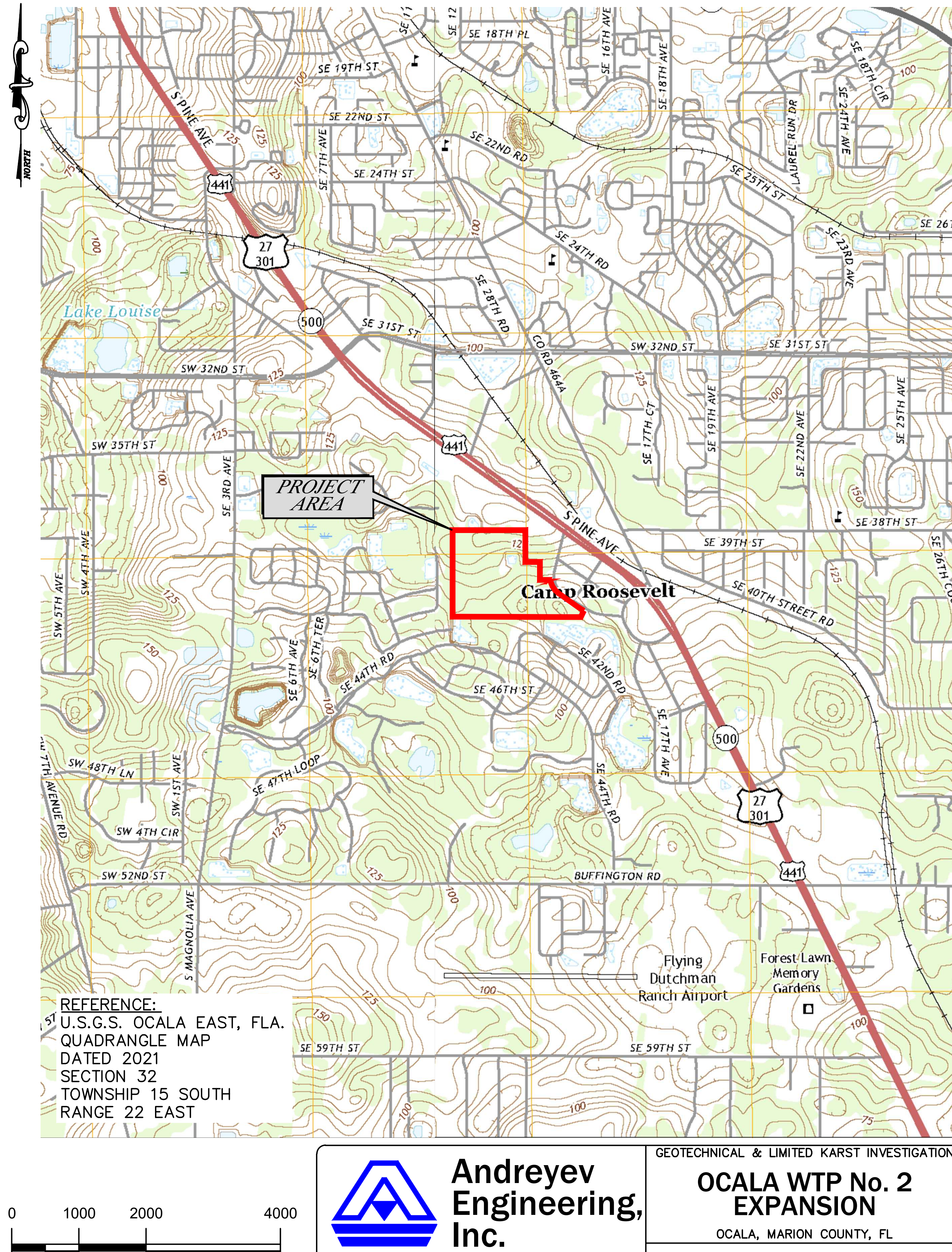
This item has been digitally signed and sealed by Shawkat Ali, P.E. and Raymond Jones, P.E. on 8/12/24

Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies

Shawkat Ali, Ph. D., P.E.
Senior Project Engineer
Florida license No. 52568

Raymond W. Jones, P.E.
Vice President
Florida License No.58079

FIGURES



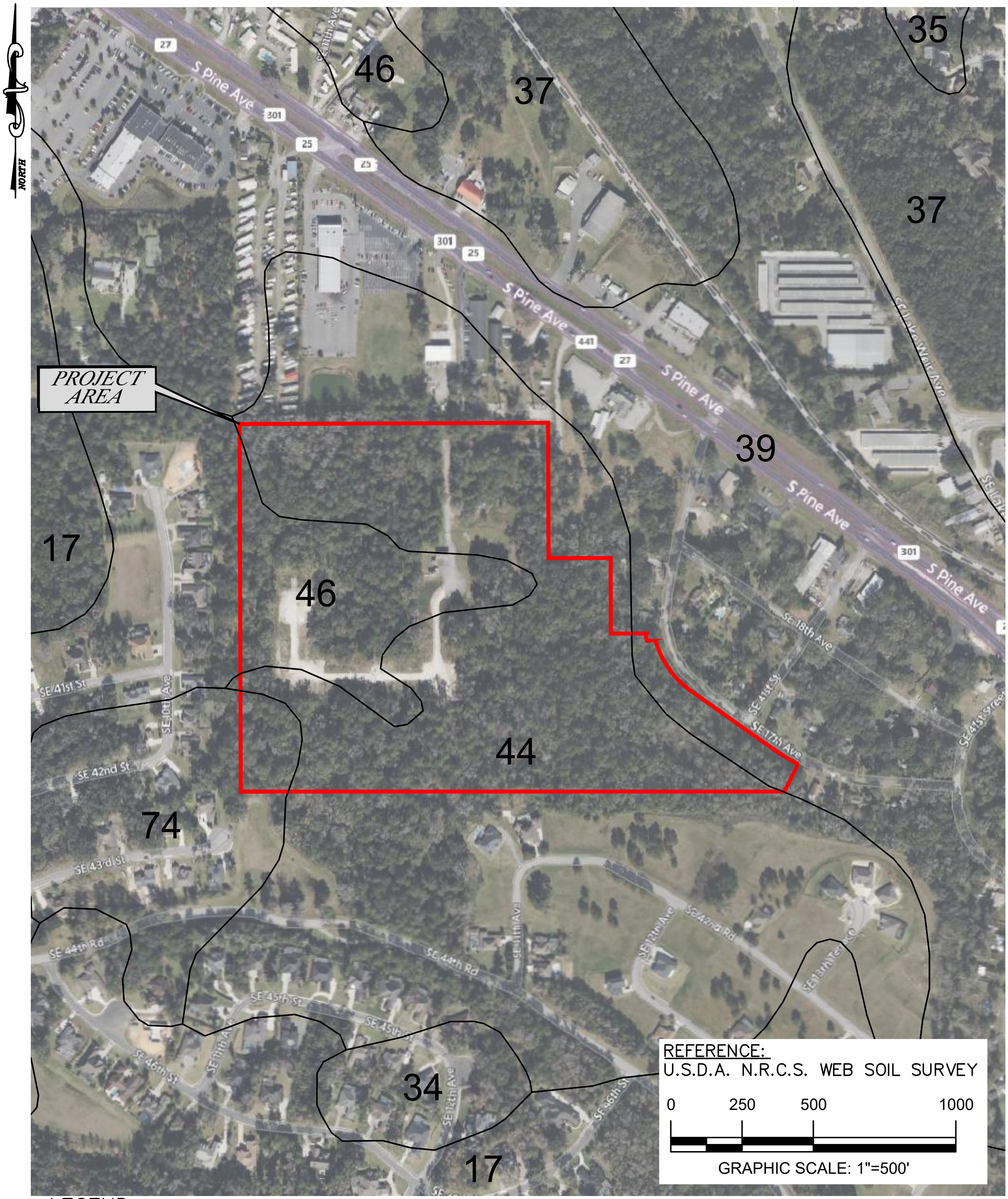
Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

APPROXIMATE SCALE:	DATE: 11/28/23	ENGINEER: SA	U.S.G.S. TOPOGRAPHIC MAP
1"=2000'	PN: GPGT-23-138	DRAWN BY: DLS	FIGURE 1



LEGEND:

39 HAGUE–URBAN LAND COMPLEX, 0 TO 5% SLOPES
44 KENDRICK LOAMY SAND, 0 TO 5% SLOPES
46 LOCHLOOSA FINE SAND, 0 TO 5% SLOPES
74 WACAHOOTA GRAVELLY SAND, GRAVELLY SUBSOIL VARIANT, 2 TO 5% SLOPES

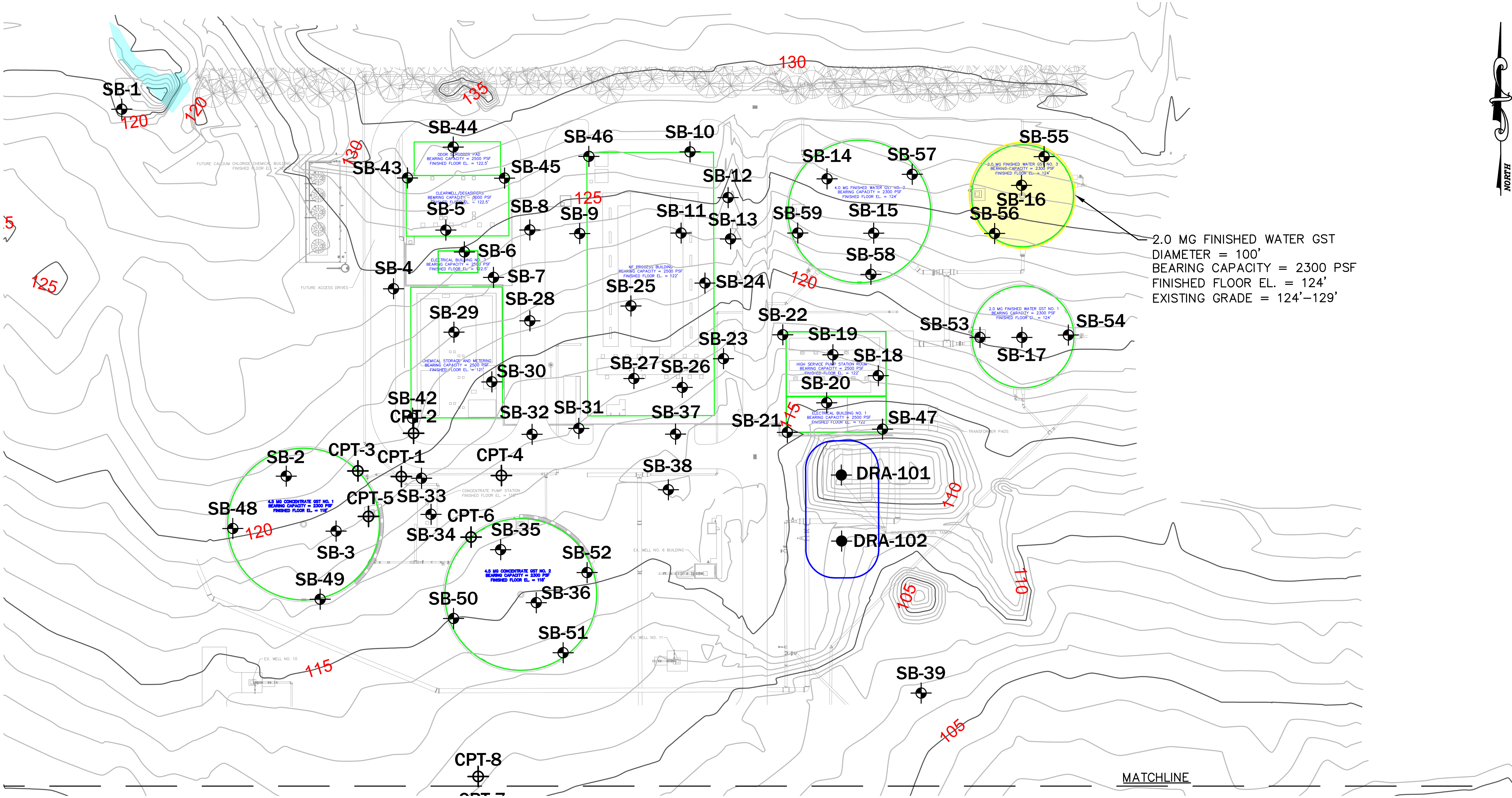
Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

APPROXIMATE SCALE: 1"=500'	DATE: 11/28/23	ENGINEER: SA	N.R.C.S. SOIL SURVEY MAP
	PN: GPGT-23-138	DRAWN BY: DLS	FIGURE 2



LEGEND:

- CPT SOUNDING LOCATION
- APPROXIMATE LOCATION OF SPT BORING
- APPROXIMATE LOCATION OF MACHINE AUGER BORING
- WETLAND & SURFACE WATERS

0 50 100 200
GRAPHIC SCALE: 1"=100'

Andreyev Engineering, Inc.

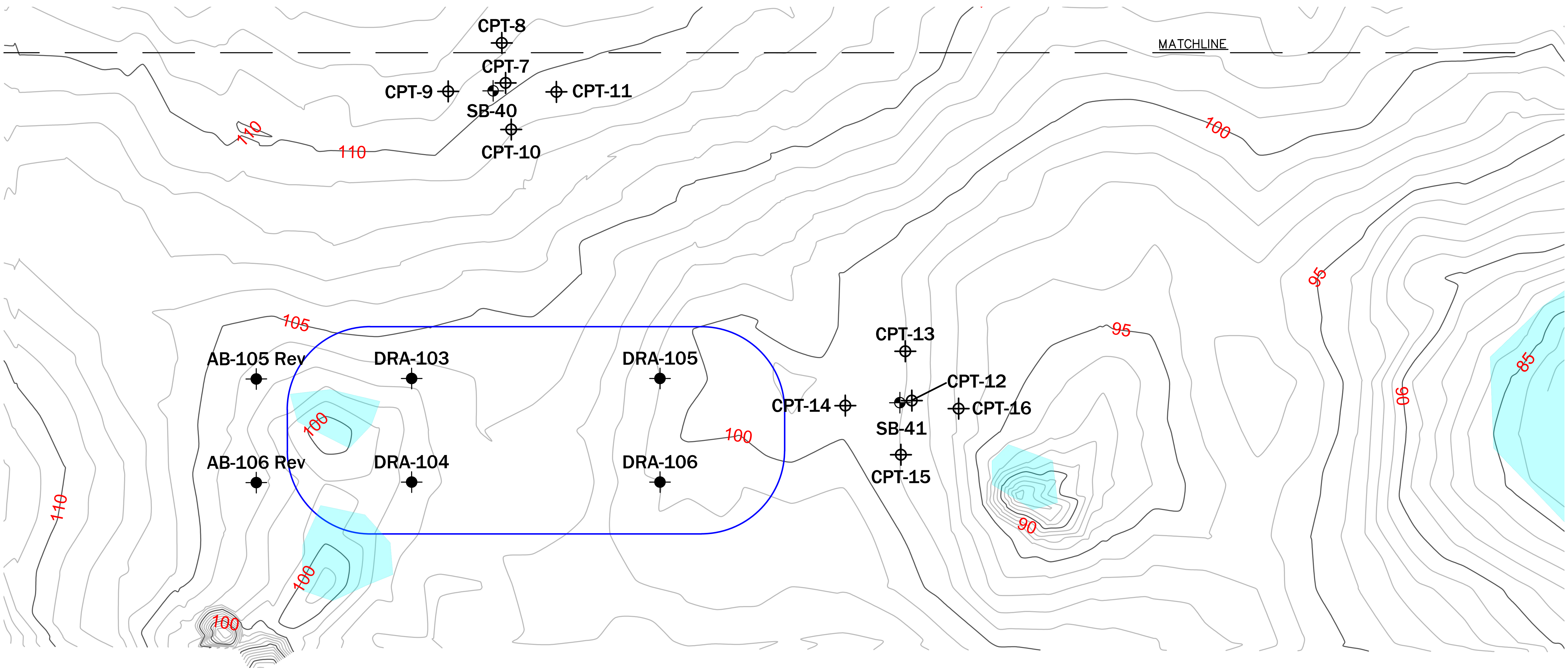
GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

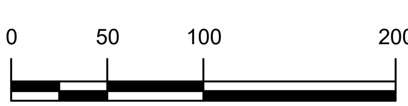
BORING LOCATION PLAN
NORTHERN AREA (WWTP)

APPROXIMATE SCALE:	DATE: 07/16/24	ENGINEER: SA	FIGURE 3A
1"=100'	PN: GPGT-23-138	DRAWN BY: DLS	



LEGEND:

- CPT SOUNDING LOCATION
- APPROXIMATE LOCATION OF SPT BORING
- APPROXIMATE LOCATION OF MACHINE AUGER BORING
- WETLAND & SURFACE WATERS



GRAPHIC SCALE: 1"=100'

Andreyev Engineering, Inc.

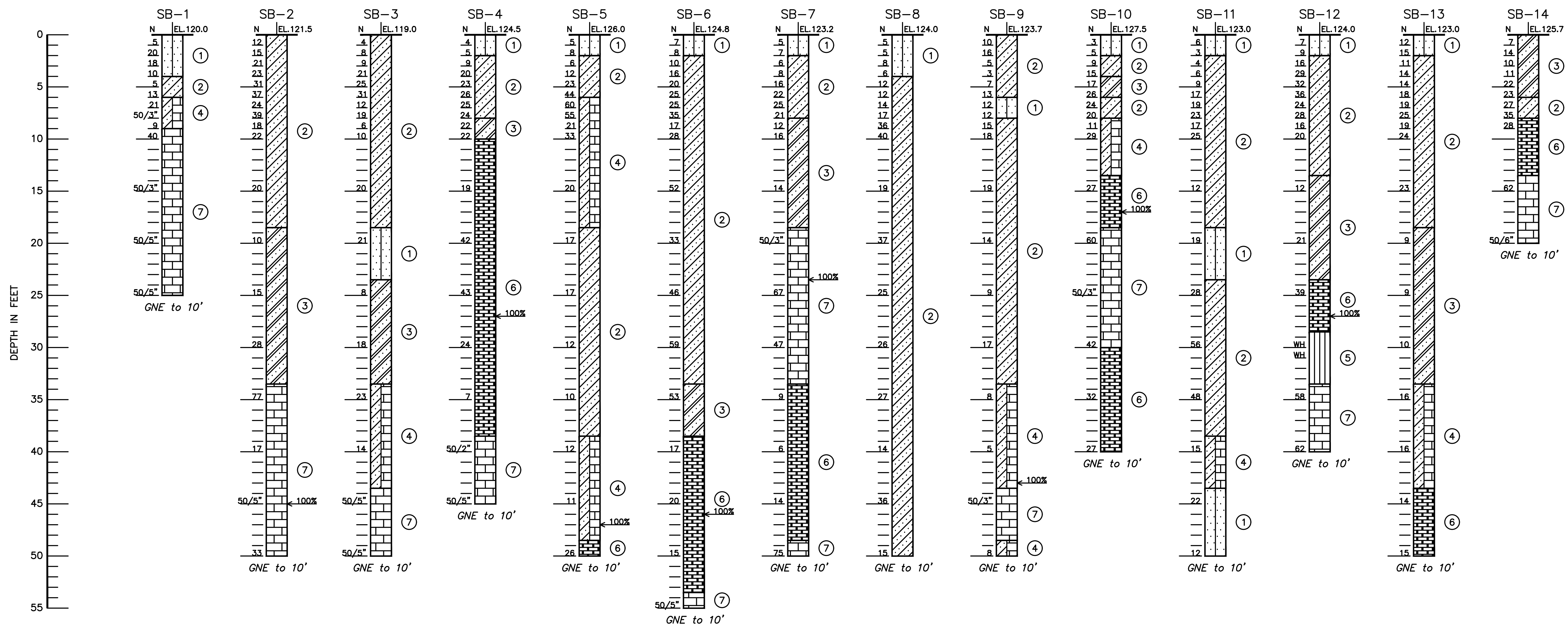
GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

BORING LOCATION PLAN
SOUTHERN AREA (DRA #2)

APPROXIMATE SCALE:	DATE: 07/16/24	ENGINEER: SA	
1"=100'	PN: GPGT-23-138	DRAWN BY: DLS	FIGURE 3B



LEGEND:

(1) LIGHT BROWN TO BROWN FINE SAND TO SLIGHTLY SILTY FINE SAND (SP)(SP–SM)

(2) GRAY TO LIGHT BROWN TO DARK BROWN SILTY TO CLAYEY FINE SAND (SM)(SC)

(3) GRAYISH GREEN TO GREEN SANDY CLAY TO CLAY (CL)

(4) BROWN TO GRAYISH GREEN CLAYEY FINE SAND TO SANDY CLAY WITH LIMESTONE & DOLOMITIC LIMESTONE FRAGMENTS (SC)(CL)

(5) HIGHLY WEATHERED LIMESTONE [LIME SILT]

(6) WEATHERED LIMESTONE WITH LIME SILT & SILTY CLAY

(7) LIMESTONE

(SP) UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL

GNE GROUNDWATER NOT ENCOUNTERED

N STANDARD PENETRATION RESISTANCE, IN BLOWS PER FOOT

WR BORING ADVANCED UNDER STATIC WEIGHT OF DRILL ROD

WH BORING ADVANCED UNDER STATIC WEIGHT OF DRILL HAMMER & ROD

50/1" 50 HAMMER BLOWS TO ADVANCE SAMPLING TOOL ONE INCH

100% LOSS OF DRILLING FLUID CIRCULATION, IN PERCENT

EL. ESTIMATED GROUND SURFACE ELEVATION AT BORING LOCATION (FT–NAVD88) BASED ON 1 FOOT TOPOGRAPHIC DATA PROVIDED BY KIMLEY HORN

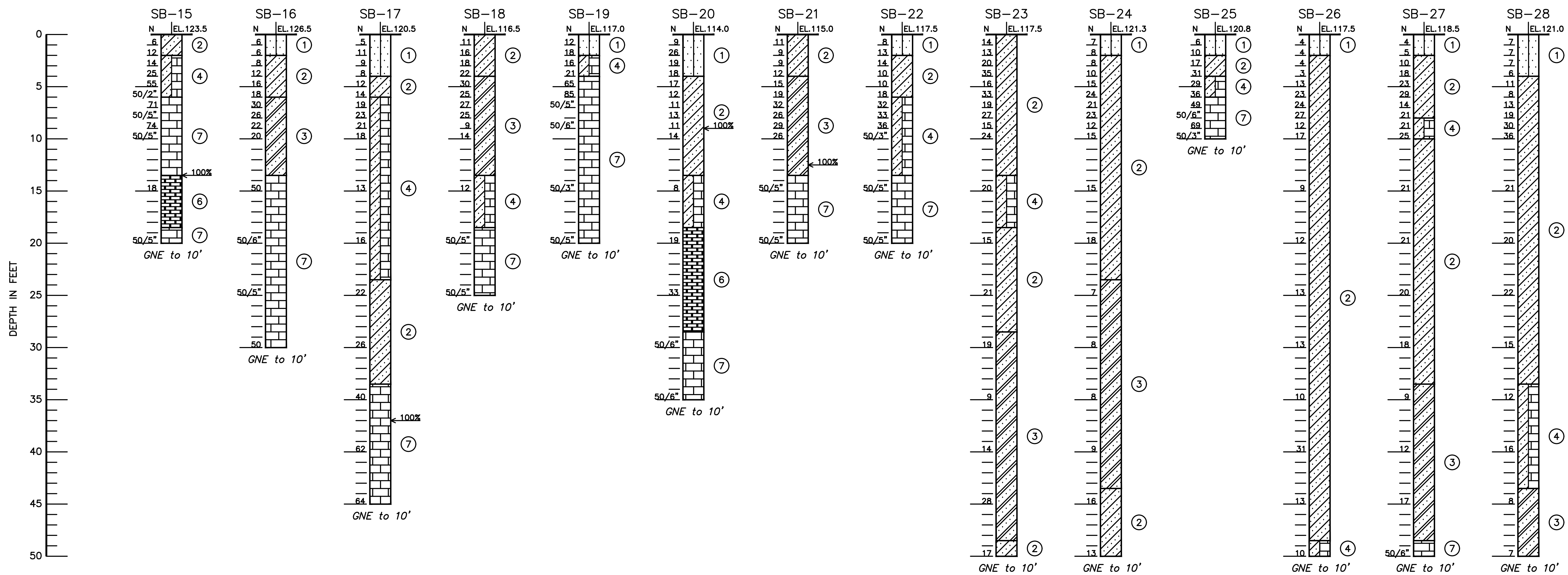
Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

APPROXIMATE SCALE: 1"=10'	DATE: 07/30/24	ENGINEER: SA	SOIL PROFILES
	PN: GPGT–23–138	DRAWN BY: DLS	FIGURE 4A



LEGEND:

(1) LIGHT BROWN TO BROWN FINE SAND TO SLIGHTLY SILTY FINE SAND (SP)(SP–SM)

(2) GRAY TO LIGHT BROWN TO DARK BROWN SILTY TO CLAYEY FINE SAND (SM)(SC)

(3) GRAYISH GREEN TO GREEN SANDY CLAY TO CLAY (CL)

(4) BROWN TO GRAYISH GREEN CLAYEY FINE SAND TO SANDY CLAY WITH LIMESTONE & DOLOMITIC LIMESTONE FRAGMENTS (SC)(CL)

(5) HIGHLY WEATHERED LIMESTONE [LIME SILT]

(6) WEATHERED LIMESTONE WITH LIME SILT & SILTY CLAY

(7) LIMESTONE

(SP) UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL

GNE GROUNDWATER NOT ENCOUNTERED

N STANDARD PENETRATION RESISTANCE, IN BLOWS PER FOOT

WR BORING ADVANCED UNDER STATIC WEIGHT OF DRILL ROD

WH BORING ADVANCED UNDER STATIC WEIGHT OF DRILL HAMMER & ROD

50/1" 50 HAMMER BLOWS TO ADVANCE SAMPLING TOOL ONE INCH

100% LOSS OF DRILLING FLUID CIRCULATION, IN PERCENT

EL. ESTIMATED GROUND SURFACE ELEVATION AT BORING LOCATION (FT–NAVD88) BASED ON 1 FOOT TOPOGRAPHIC DATA PROVIDED BY KIMLEY HORN

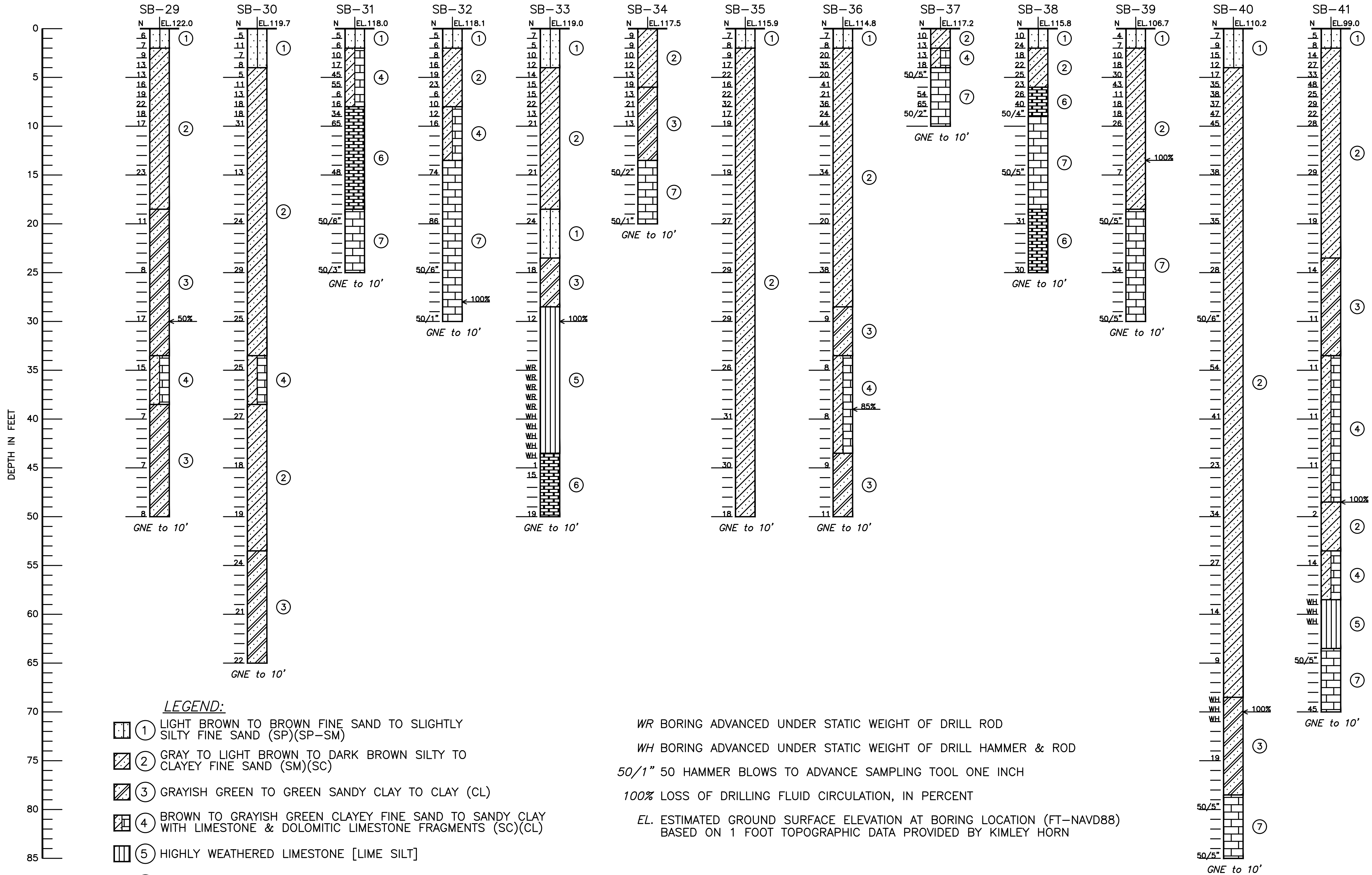
Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

APPROXIMATE SCALE: 1"=10'	DATE: 07/30/24	ENGINEER: SA	SOIL PROFILES
	PN: GPGT–23–138	DRAWN BY: DLS	FIGURE 4B



LEGEND:

1 LIGHT BROWN TO BROWN FINE SAND TO SLIGHTLY SILTY FINE SAND (SP)(SP–SM)

2 GRAY TO LIGHT BROWN TO DARK BROWN SILTY TO CLAYEY FINE SAND (SM)(SC)

3 GRAYISH GREEN TO GREEN SANDY CLAY TO CLAY (CL)

4 BROWN TO GRAYISH GREEN CLAYEY FINE SAND TO SANDY CLAY WITH LIMESTONE & DOLOMITIC LIMESTONE FRAGMENTS (SC)(CL)

5 HIGHLY WEATHERED LIMESTONE [LIME SILT]

6 WEATHERED LIMESTONE WITH LIME SILT & SILTY CLAY

7 LIMESTONE

(SP) UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL

GNE GROUNDWATER NOT ENCOUNTERED

N STANDARD PENETRATION RESISTANCE, IN BLOWS PER FOOT

WR BORING ADVANCED UNDER STATIC WEIGHT OF DRILL ROD

WH BORING ADVANCED UNDER STATIC WEIGHT OF DRILL HAMMER & ROD

50/1" 50 HAMMER BLOWS TO ADVANCE SAMPLING TOOL ONE INCH

100% LOSS OF DRILLING FLUID CIRCULATION, IN PERCENT

EL. ESTIMATED GROUND SURFACE ELEVATION AT BORING LOCATION (FT–NAVD88) BASED ON 1 FOOT TOPOGRAPHIC DATA PROVIDED BY KIMLEY HORN

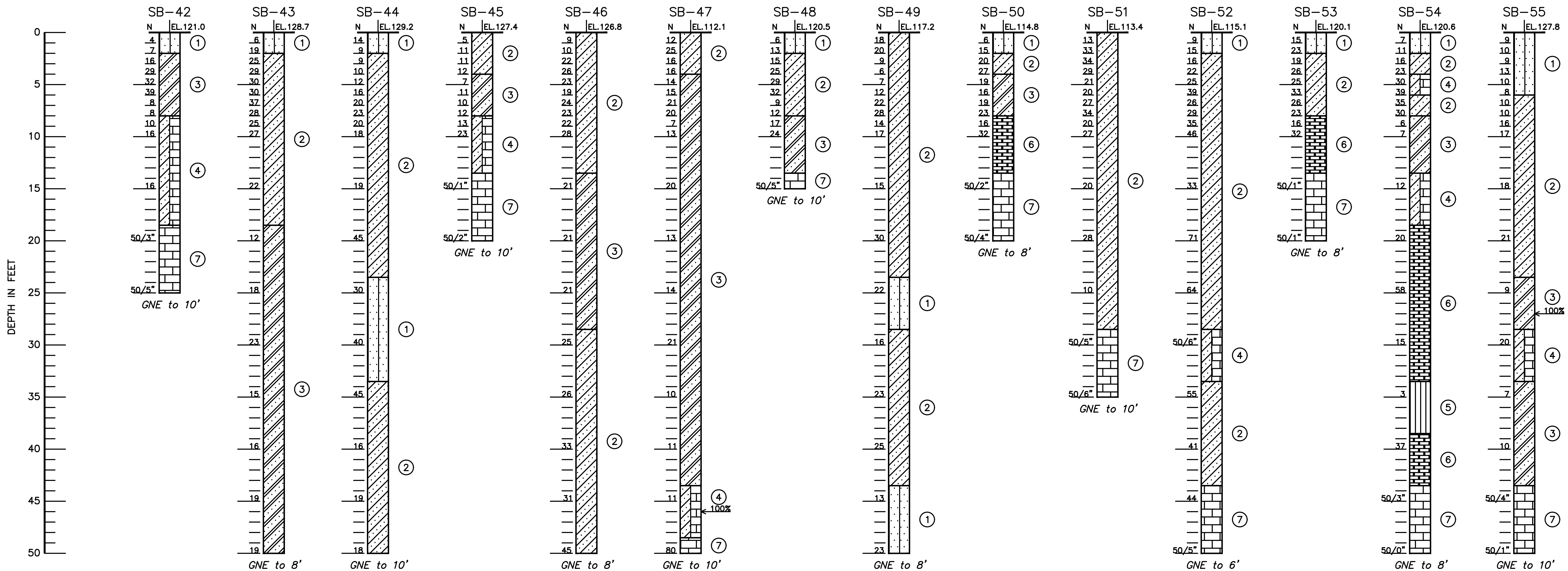
Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

APPROXIMATE SCALE:	DATE: 07/30/24	ENGINEER: SA	SOIL PROFILES
1"=10'	PN: GPGT–23–138	DRAWN BY: DLS	FIGURE 4C



LEGEND:

(1) LIGHT BROWN TO BROWN FINE SAND TO SLIGHTLY SILTY FINE SAND (SP)(SP–SM)

(2) GRAY TO LIGHT BROWN TO DARK BROWN SILTY TO CLAYEY FINE SAND (SM)(SC)

(3) GRAYISH GREEN TO GREEN SANDY CLAY TO CLAY (CL)

(4) BROWN TO GRAYISH GREEN CLAYEY FINE SAND TO SANDY CLAY WITH LIMESTONE & DOLOMITIC LIMESTONE FRAGMENTS (SC)(CL)

(5) HIGHLY WEATHERED LIMESTONE [LIME SILT]

(6) WEATHERED LIMESTONE WITH LIME SILT & SILTY CLAY

(7) LIMESTONE

(SP) UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL

GNE GROUNDWATER NOT ENCOUNTERED

N STANDARD PENETRATION RESISTANCE, IN BLOWS PER FOOT

WR BORING ADVANCED UNDER STATIC WEIGHT OF DRILL ROD

WH BORING ADVANCED UNDER STATIC WEIGHT OF DRILL HAMMER & ROD

50/1" 50 HAMMER BLOWS TO ADVANCE SAMPLING TOOL ONE INCH

100% LOSS OF DRILLING FLUID CIRCULATION, IN PERCENT

EL. ESTIMATED GROUND SURFACE ELEVATION AT BORING LOCATION (FT–NAVD88) BASED ON 1 FOOT TOPOGRAPHIC DATA PROVIDED BY KIMLEY HORN

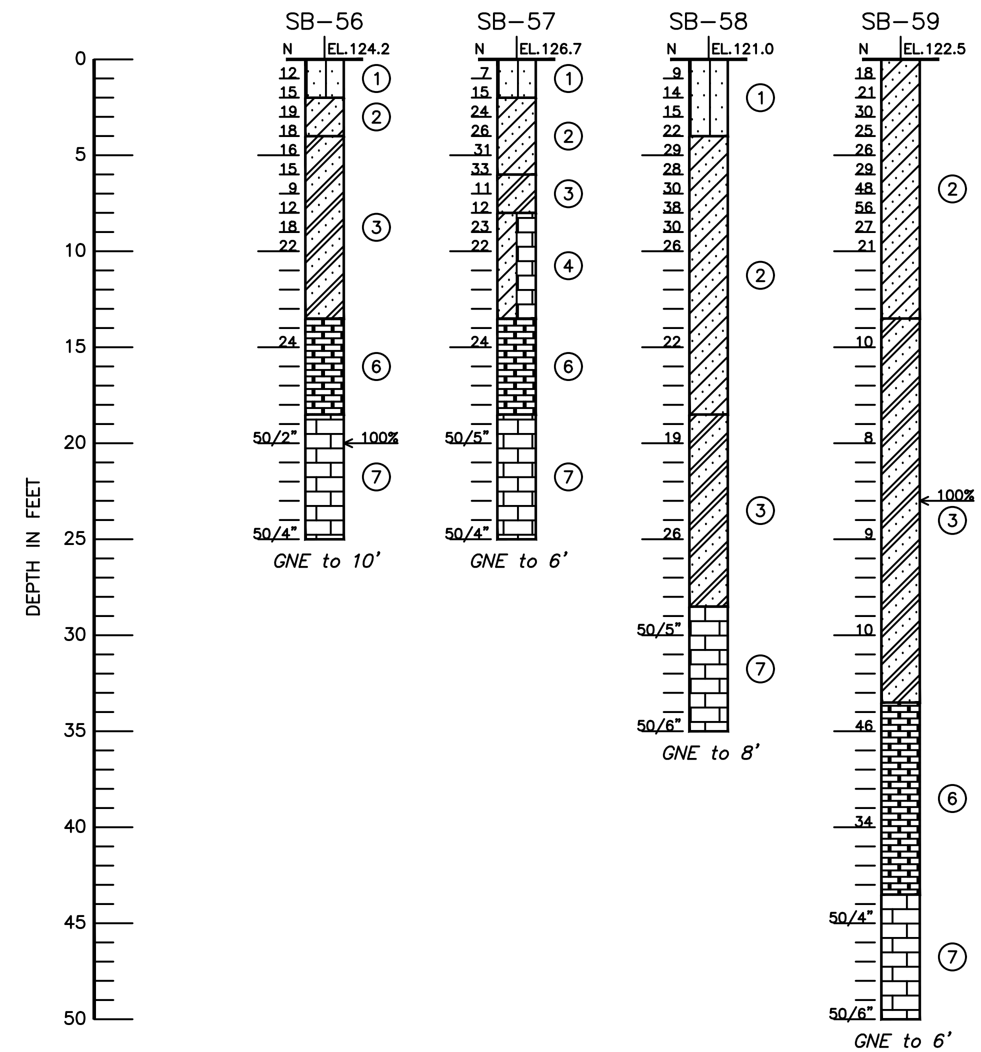
Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

APPROXIMATE SCALE: 1"=10'	DATE: 07/30/24	ENGINEER: SA	SOIL PROFILES
	PN: GPGT–23–138	DRAWN BY: DLS	FIGURE 4D



LEGEND:

1. LIGHT BROWN TO BROWN FINE SAND TO SLIGHTLY SILTY FINE SAND (SP)(SP–SM)
2. GRAY TO LIGHT BROWN TO DARK BROWN SILTY TO CLAYEY FINE SAND (SM)(SC)
3. GRAYISH GREEN TO GREEN SANDY CLAY TO CLAY (CL)
4. BROWN TO GRAYISH GREEN CLAYEY FINE SAND TO SANDY CLAY WITH LIMESTONE & DOLOMITIC LIMESTONE FRAGMENTS (SC)(CL)
5. HIGHLY WEATHERED LIMESTONE [LIME SILT]
6. WEATHERED LIMESTONE WITH LIME SILT & SILTY CLAY
7. LIMESTONE

(SP) UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL

GNE GROUNDWATER NOT ENCOUNTERED

N STANDARD PENETRATION RESISTANCE, IN BLOWS PER FOOT

WR BORING ADVANCED UNDER STATIC WEIGHT OF DRILL ROD

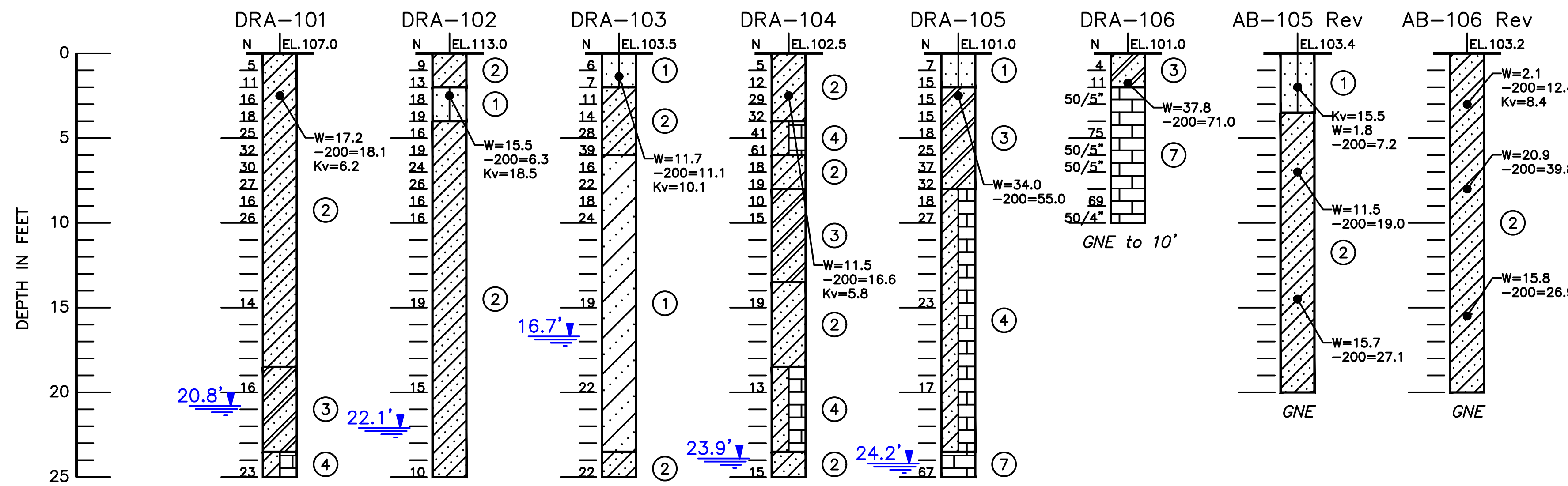
WH BORING ADVANCED UNDER STATIC WEIGHT OF DRILL HAMMER & ROD

50/1" 50 HAMMER BLOWS TO ADVANCE SAMPLING TOOL ONE INCH

100% LOSS OF DRILLING FLUID CIRCULATION, IN PERCENT

EL. ESTIMATED GROUND SURFACE ELEVATION AT BORING LOCATION (FT–NAVD88) BASED ON 1 FOOT TOPOGRAPHIC DATA PROVIDED BY KIMLEY HORN

Andreyev Engineering, Inc.			GEOTECHNICAL & LIMITED KARST INVESTIGATION OCALA WTP No. 2 EXPANSION OCALA, MARION COUNTY, FL
APPROXIMATE SCALE:	DATE: 07/30/24	ENGINEER: SA	SOIL PROFILES
1"=10'	PN: GPGT–23–138	DRAWN BY: DLS	FIGURE 4E



LEGEND:

- (1) LIGHT BROWN TO BROWN FINE SAND TO SLIGHTLY SILTY FINE SAND (SP)(SP–SM)
- (2) GRAY TO LIGHT BROWN TO DARK BROWN SILTY TO CLAYEY FINE SAND (SM)(SC)
- (3) GRAYISH GREEN TO GREEN SANDY CLAY TO CLAY (CL)
- (4) BROWN TO GRAYISH GREEN CLAYEY FINE SAND TO SANDY CLAY WITH LIMESTONE & DOLOMITIC LIMESTONE FRAGMENTS (SC)(CL)
- (5) HIGHLY WEATHERED LIMESTONE [LIME SILT]
- (6) WEATHERED LIMESTONE WITH LIME SILT & SILTY CLAY
- (7) LIMESTONE

(SP) UNIFIED SOIL CLASSIFICATION SYSTEM GROUP SYMBOL

GNE GROUNDWATER NOT ENCOUNTERED

N STANDARD PENETRATION RESISTANCE, IN BLOWS PER FOOT

W MOISTURE CONTENT, IN PERCENT

–200 PERCENT OF FINES PASSING THE U.S. No. 200 SIEVE

WR BORING ADVANCED UNDER STATIC WEIGHT OF DRILL ROD

WH BORING ADVANCED UNDER STATIC WEIGHT OF DRILL HAMMER & ROD

50/1" 50 HAMMER BLOWS TO ADVANCE SAMPLING TOOL ONE INCH

100% LOSS OF DRILLING FLUID CIRCULATION, IN PERCENT

EL. ESTIMATED GROUND SURFACE ELEVATION AT BORING LOCATION (FT–NAVD88) BASED ON 1 FOOT TOPOGRAPHIC DATA PROVIDED BY KIMLEY HORN

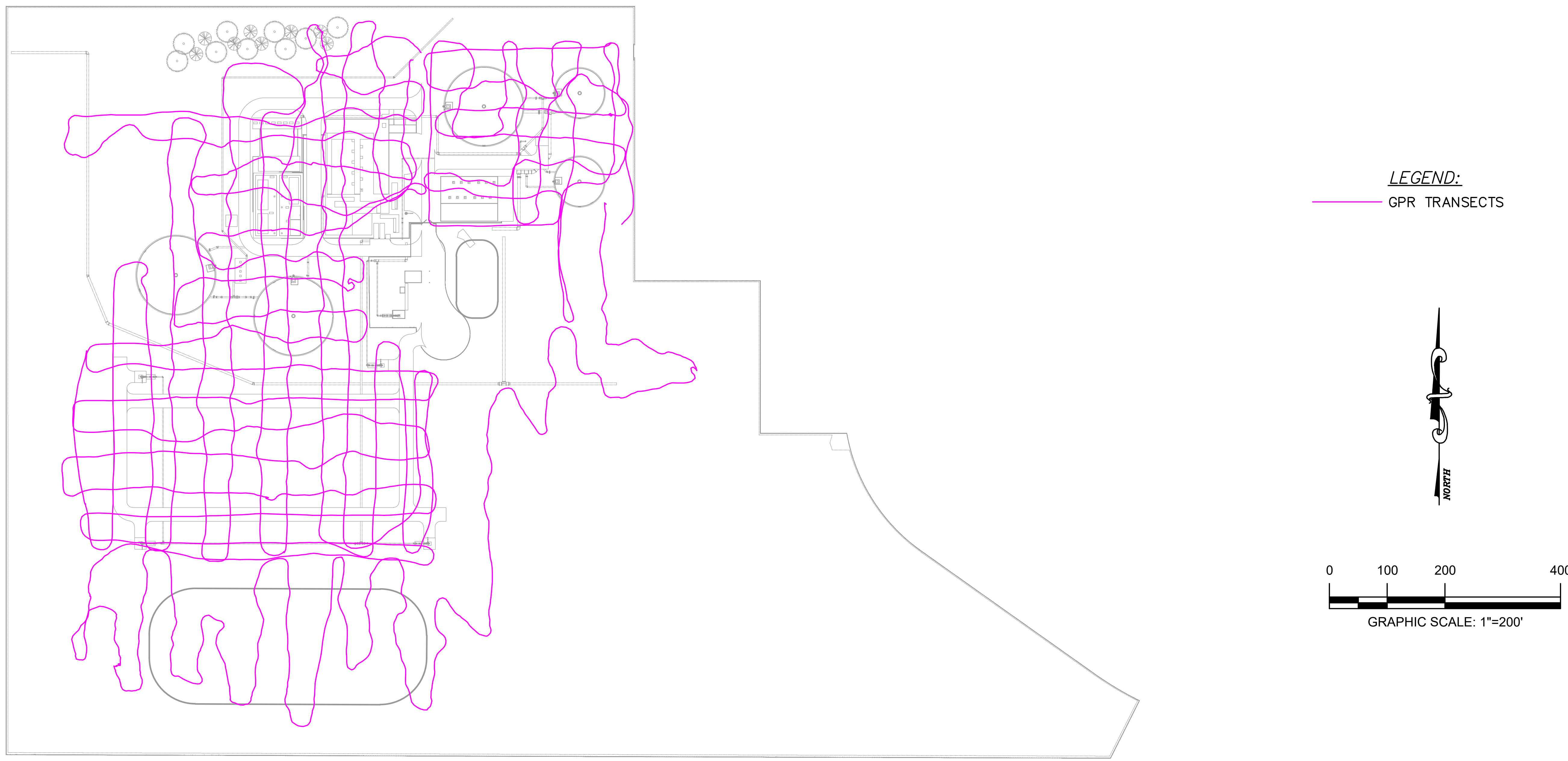
Kv VERTICAL COEFFICIENT OF PERMEABILITY, IN FEET PER DAY

1.0' DEPTH TO GROUNDWATER

NOTE:

PERMEABILITY WITHIN THE STRATUM 2 SOILS COULD VARY SIGNIFICANTLY WITHIN THE LAYER DEPENDING ON VARIATION IN FINES CONTENT, POROSITY, CEMENTATION AND COMPACTION.

Andreyev Engineering, Inc.	GEOTECHNICAL & LIMITED KARST INVESTIGATION OCALA WTP No. 2 EXPANSION OCALA, MARION COUNTY, FL	
APPROXIMATE SCALE: 1"=10'	DATE: 07/30/24	ENGINEER: SA
	PN: GPGT–23–138	DRAWN BY: DLS
	SOIL PROFILES	FIGURE 4F



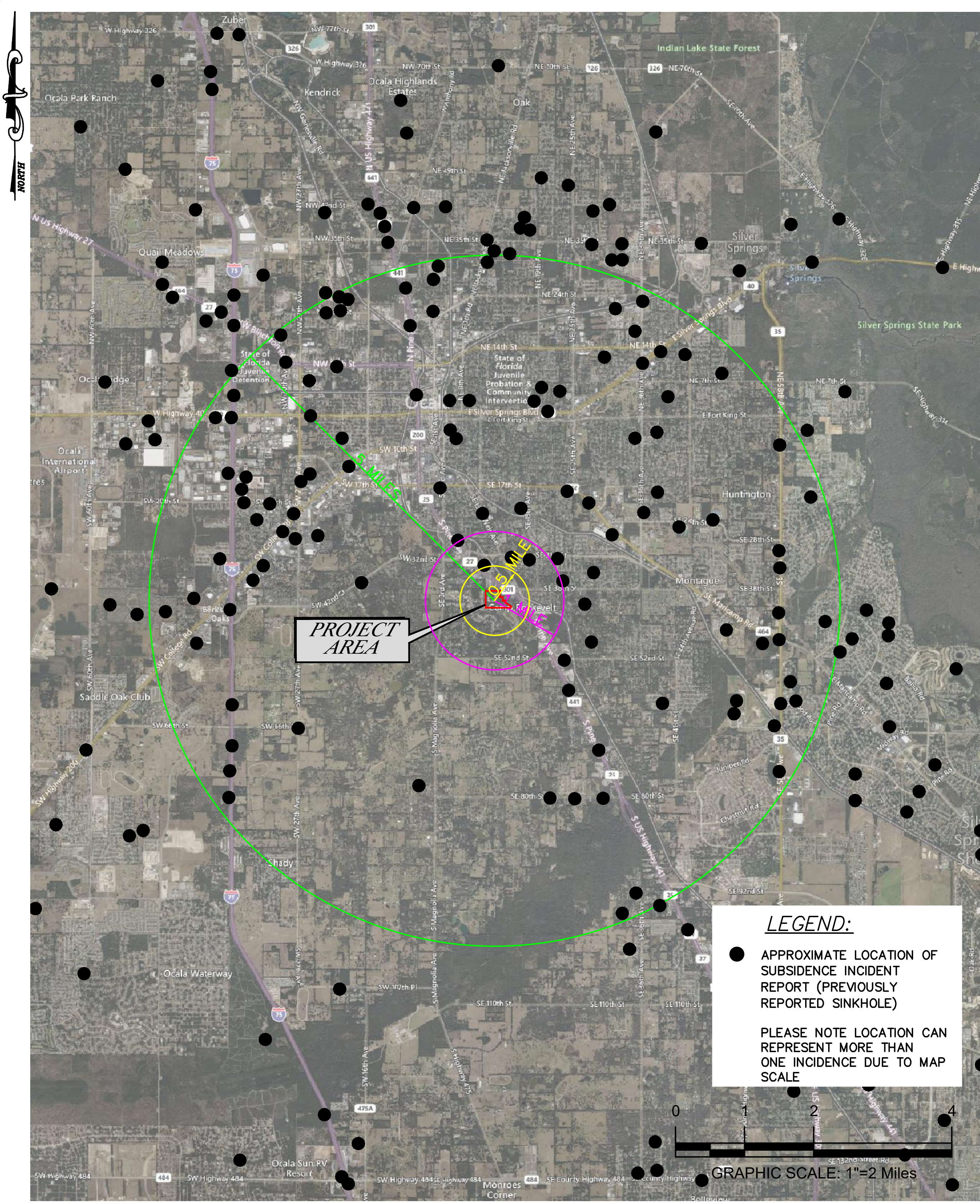
Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

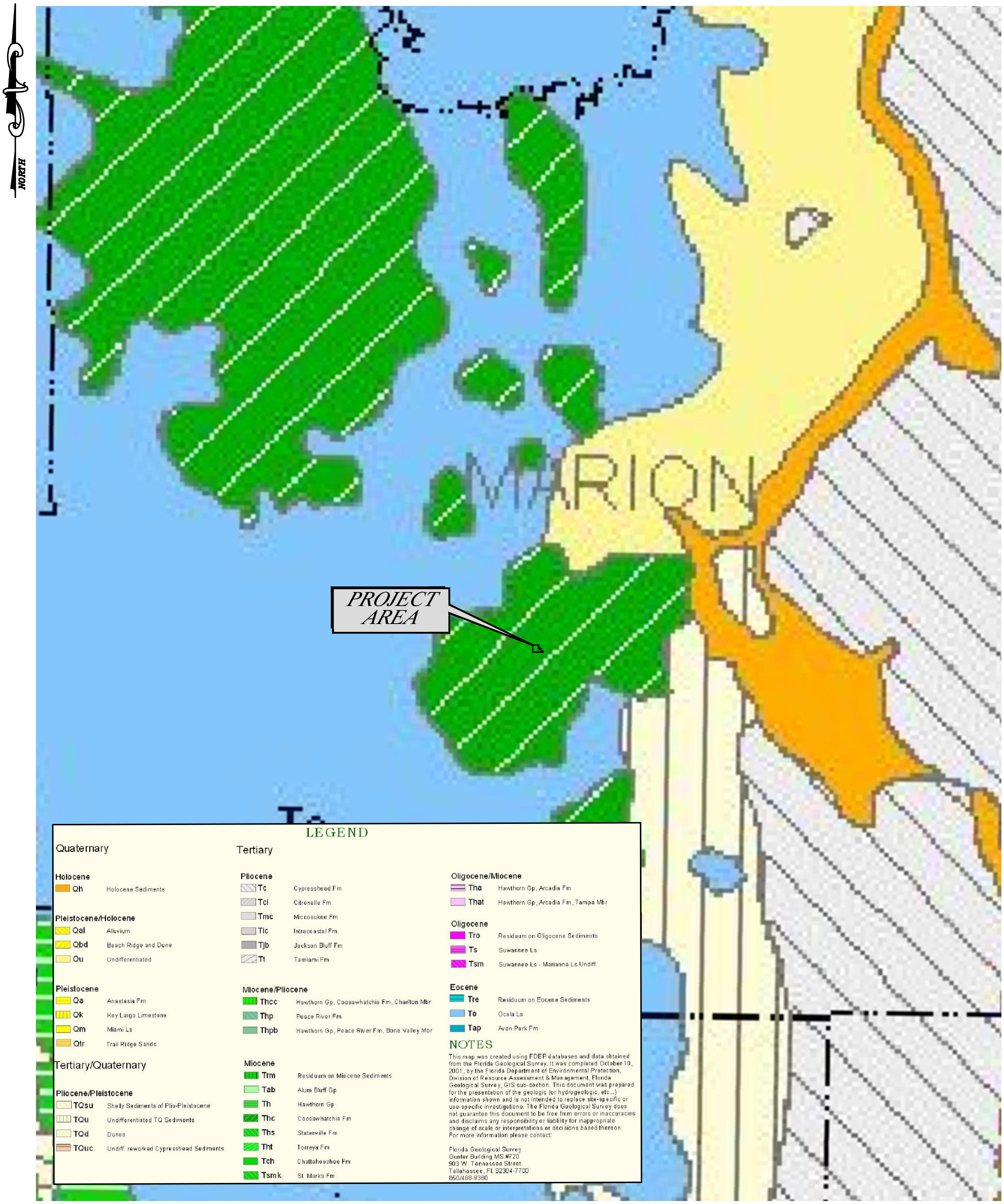
OCALA, MARION COUNTY, FL

APPROXIMATE SCALE: 1"=200'	DATE: 04/04/24	ENGINEER: SA	GPR TRANSECT MAP
	PN: GPGT-23-138	DRAWN BY: DLS	FIGURE 5



REFERENCE:
FLORIDA SUBSIDENCE INCIDENT REPORTS, F.D.E.P., OPEN DATA PORTAL, FLORIDA GEOLOGICAL SURVEY

Andreyev Engineering, Inc.			GEOTECHNICAL & LIMITED KARST INVESTIGATION **OCALA WTP No. 2 EXPANSION** OCALA, MARION COUNTY, FL
APPROXIMATE SCALE:	DATE: 11/28/23	ENGINEER: SA	PREVIOUSLY REPORTED SINKHOLES
1"=2 Miles	PN: GPGT-23-138	DRAWN BY: DLS	FIGURE 6



0 2.5 5 10

GRAPHIC SCALE: 1"=5 Miles

Andreyev Engineering, Inc.

GEOTECHNICAL & LIMITED KARST INVESTIGATION

OCALA WTP No. 2 EXPANSION

OCALA, MARION COUNTY, FL

GEOLOGIC MAP OF THE STATE OF FLORIDA

APPROXIMATE SCALE:	DATE: 04/03/24	ENGINEER: SA
1"=5 Miles	PN: GPGT-23-138	DRAWN BY: DLS

FIGURE 7

ATTACHMENT A

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-1
Total depth: 43.64 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-2
Total depth: 22.11 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-3
Total depth: 14.37 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-4
Total depth: 7.02 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-5
Total depth: 44.03 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-6
Total depth: 14.70 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-7
Total depth: 77.95 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-8
Total depth: 37.66 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-9
Total depth: 68.44 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-10
Total depth: 70.93 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-11
Total depth: 35.96 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-12
Total depth: 60.43 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-13
Total depth: 38.25 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-14
Total depth: 13.32 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

CPT: CPT-15
Total depth: 69.23 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: Ocala WTP #2, Ocala
Location: Ocala, Marion County

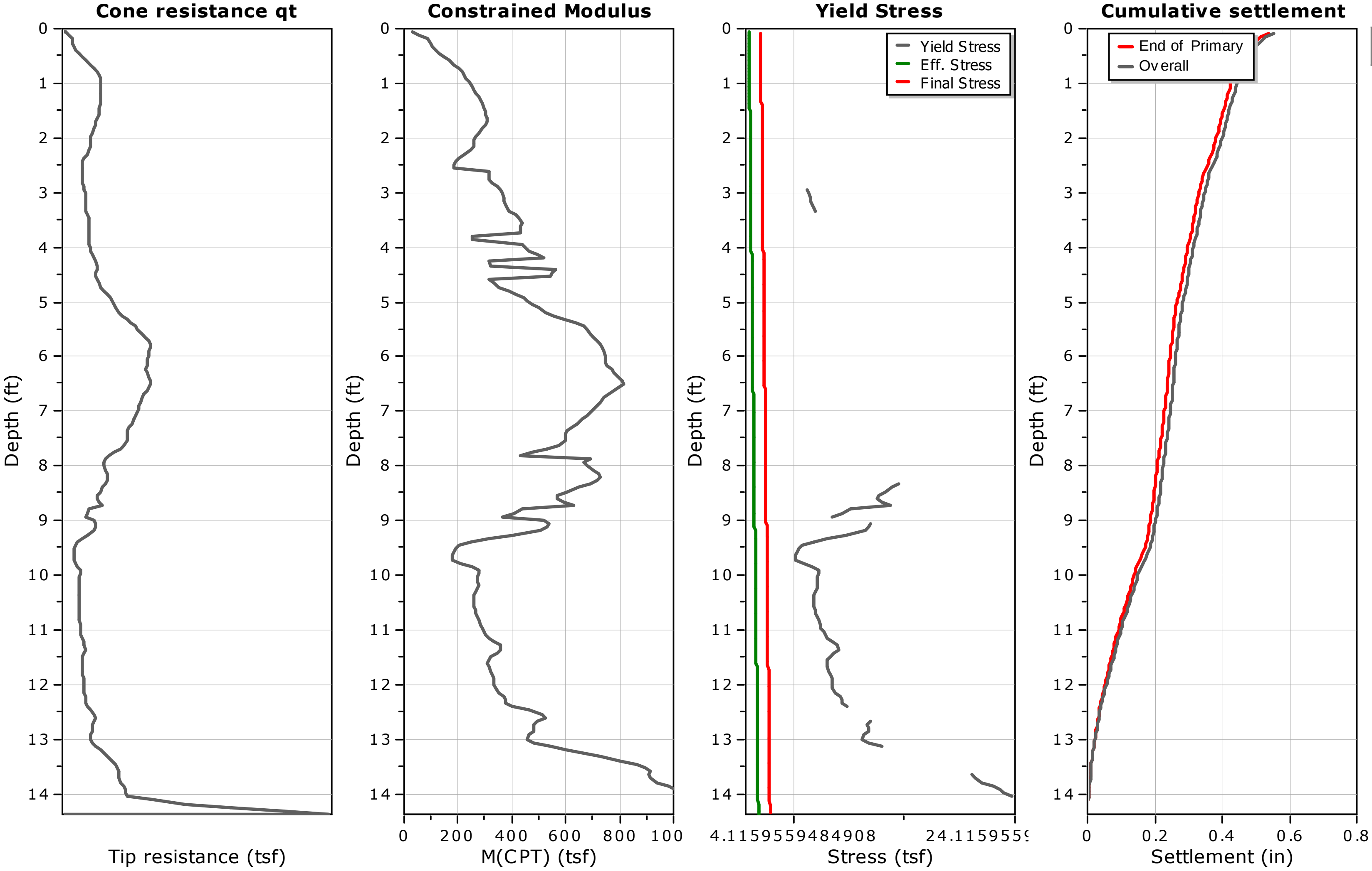
CPT: CPT-16
Total depth: 62.40 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

CPT: CPT-3
Total depth: 14.37 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Project: Ocala WTP #2, Ocala -Settlement Calculation - 4.5 MG Concentrate GST 1
Location: Ocala, Marion County

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Circular
Footing diameter: 149.00 (ft)
L/B: 1.0
Footing pressure: 1.06 (tsf)
Embedment depth: 0.00 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 240 months

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta \sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

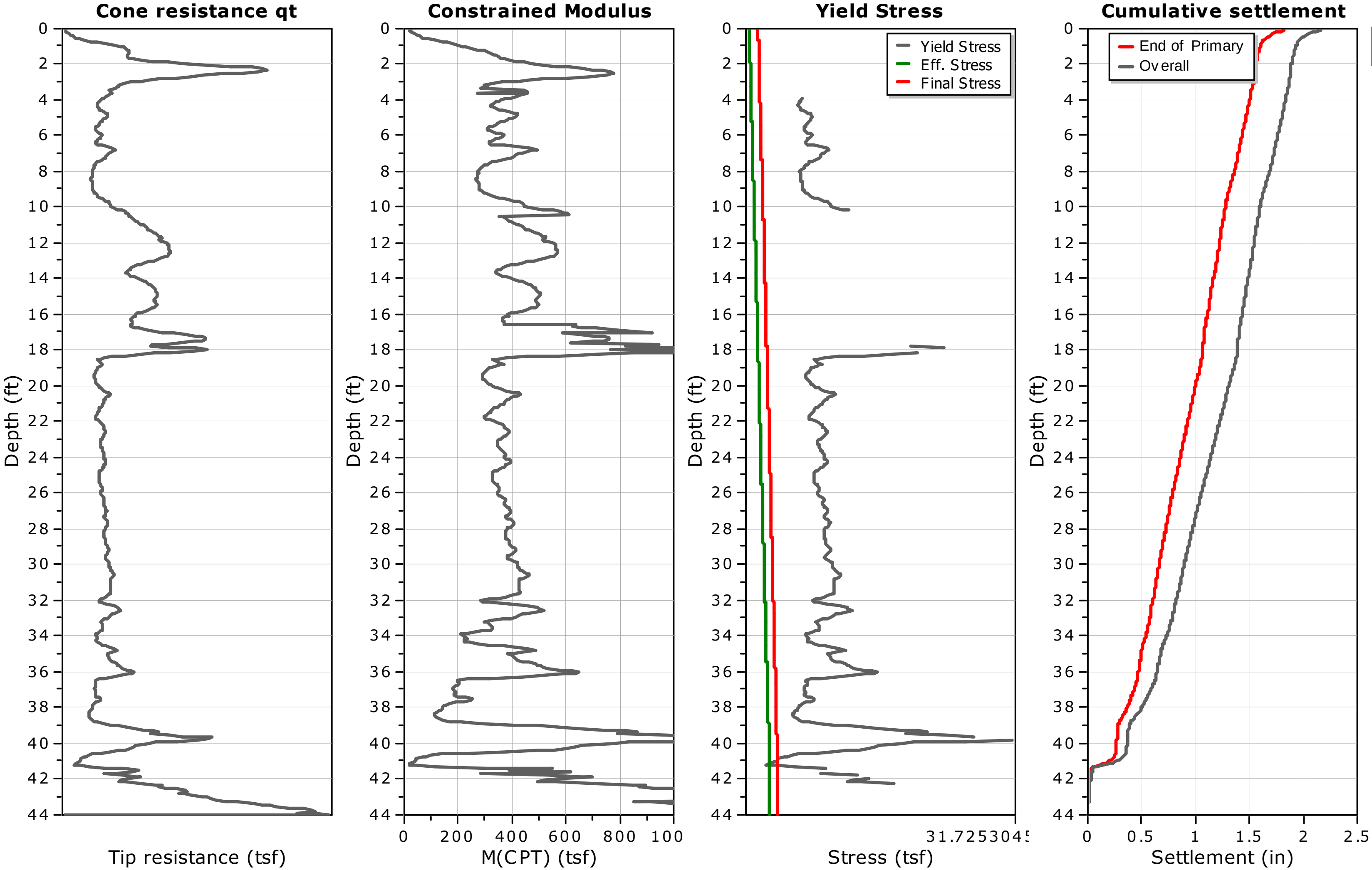
where t_p is the duration of primary consolidation

Andreyev Engineering, Inc.
Geotechnical Engineers
4055 St. John's Parkway
Sanford, Florida

Project: **Ocala WTP #2 - Settlement Calculation - 4.5 MG Concentrate GST 1**
Location: **Ocala, Marion County**

CPT: CPT-5
Total depth: 44.03 ft, Date: 3/18/2024
Surface Elevation: 0.00 ft
Coords: X:0.00, Y:0.00
Cone Type: Geoprobe
Cone Operator: Blackbit Drilling, Inc.

Settlements calculation according to theory of elasticity*



Calculation properties

Footing type: Circular
Footing diameter: 149.00 (ft)
L/B: 1.0
Footing pressure: 1.06 (tsf)
Embedment depth: 0.00 (ft)
Footing is rigid: No
Remove excavation load: No
Apply 20% rule: No
Calculate secondary settlements: Yes
Time period for primary consolidation: 6 months
Time period for second. settlements: 240 months

* Primary settlement calculation is performed according to the following formula:

$$S = \sum \frac{\Delta\sigma_v}{M_{CPT}} \Delta z$$

* Secondary (creep) settlement calculation is performed according to the following formula:

$$S = C_\alpha \cdot \Delta z \cdot \log(t/t_p)$$

where t_p is the duration of primary consolidation

ATTACHMENT B

FINAL REPORT
GEOPHYSICAL INVESTIGATION
OCALA WTP 2 SITE
OCALA, FLORIDA

Prepared for Andreyev Engineering, Inc.
Sanford, FL

Prepared by GeoView Associates, Inc.
St. Petersburg, FL

March 06, 2024

Mr. Shawkat Ali, Ph. D., P.E
Andreyev Engineering, Inc.
4055 St. John's Parkway
Sanford, FL 32771

Subject: Transmittal of Final Report for Geophysical Investigation
Ocala WTP 2 Site
Ocala, Florida
GeoView Project Number 40848

Mr. Ali,

GeoView, Inc. is pleased to submit the final report that summarizes and presents the results of the geophysical investigation performed at the above referenced site. Ground penetrating radar and electrical resistivity imaging were used to help determine the presence of possible karst (sinkhole) features that may be present at the project site. GeoView appreciates the opportunity to have assisted you on this project. If you have any questions or comments about the report, please contact us.

Sincerely,

GEOVIEW ASSOCIATES, INC.

Stephen Scruggs, P.G.
Senior Geophysicist
Florida Professional Geologist Number 2470

5709 First Avenue South ***Tel.: (727) 209-2334***
St. Petersburg, FL 33707 ***Fax: (727) 328-2477***

1.0 Introduction

A geophysical investigation was completed on February 21, 2024, at the Ocala WTP 2 site located at 3778 S Pine Ave in Ocala, Florida. The project site is an undeveloped parcel of land that is being considered for development.

A geophysical investigation, using ground penetrating radar (GPR), was performed across the project site area. The location of the geophysical survey area is provided on Figure 1 (Appendix 1). The purpose of the geophysical investigation was to help characterize near-surface geological conditions and to identify subsurface features that may be associated with karst (sinkhole) activity.

2.0 Description of Geophysical Investigation

The GPR data was collected using a Mala radar system with a 250 MHz antenna and a time range of 195 nanoseconds. This equipment configuration provided an estimated exploration depth of 7 to 10 ft below land surface (bls). The GPR data was digitally recorded for both analysis and archiving purposes.

The GPR survey was completed along a series of previously cleared transects that were spaced on average 50 to 100 feet (ft) apart. The GPR data was acquired using an all-terrain vehicle and position control for the GPR data was provided using a Trimble Geo7x GPS system with sub-meter accuracy. The locations of the GPR transect lines collected within the project site are shown on Figure 1. A description of the GPR technique and the methods employed for geological characterization studies is provided in Appendix A2.2.

3.0 Identification of Possible Sinkhole (Karst) Features Using GPR

The features observed on GPR data that are most commonly associated with karst features are:

- A downwarping of GPR reflector sets, that are associated with suspected lithological contacts, toward a common center. Such features typically have with a bowl or funnel shaped configuration and can be associated with a deflection of overlying sediment horizons caused by the migration of sediments into voids in the underlying limestone. If the GPR reflector sets are sharply downwarping and intersect, they can create a “bow-tie” shaped GPR reflection feature, which often designates the apparent center of the GPR anomaly.

- A localized significant increase in the depth of the penetration and/or amplitude of the GPR signal response. The increase in GPR signal penetration depth or amplitude is often associated with either a localized increase in sand content at depth or decrease in soil density.

- An apparent discontinuity in GPR reflector sets, that are associated with suspected lithological contacts. The apparent discontinuities and/or disruption of the GPR reflector sets may be associated with the downward migration of sediments.

The greater the severity of these features or a combination of these features, the greater the likelihood that the identified feature is a sinkhole. It is not possible based on the GPR data alone to determine if an identified feature is an active karst-related geologic feature.

4.0 Survey Results

Results of the GPR survey indicated the presence of a well-defined, relatively continuous set of GPR reflectors at a depth range of 1 to 4 ft bls. This reflector set is most likely associated with some change in lithological conditions at this depth range.

The GPR reflector sets identified in the GPR investigation were continuous across the accessible areas of the project site. No observed areas of significant downwarping or other indicators of possible sinkhole activity were observed. Accordingly, based on the results of the GPR survey the following is concluded:

1) No indication of potential sinkhole activity was observed within the depth limits of the GPR signal collected across the project site.
2) Soils from the top of the previously discussed GPR reflector set to the maximum depth of penetration of the GPR signal (7 to 10 ft bls) appear to be relatively homogeneous (similar).

A discussion of the limitations of the GPR technique in geological characterization studies is provided in Appendix 2.

APPENDIX 1
FIGURE

EXPLANATION

GPR TRANSECTS

OCALA WTP 2 SITE
3778 S PINE AVENUE
OCALA, FLORIDA

ANDREYEV ENGINEERING, INC.
SANFORD, FLORIDA

PROJECT: 40848
DATE: 03/06/24

GeoView

FIGURE 1
SITE MAP
SHOWING RESULTS
OF GEOPHYSICAL
INVESTIGATION

APPENDIX 2
DESCRIPTION OF GEOPHYSICAL METHODS, SURVEY METHODOLOGIES AND LIMITATIONS

A2.1 On Site Measurements

The positions of the geophysical transect lines were recorded using a Trimble Geo7x Global Positioning System (GPS). These GPS systems typically have an accuracy of 1 to 3 ft.

A2.2 Ground Penetrating Radar

Ground Penetrating Radar (GPR) consists of a set of integrated electronic components which transmits high frequency (200 to 1500 megahertz [MHz]) electromagnetic waves into the ground and records the energy reflected back to the ground surface. The GPR system consists of an antenna, which serves as both a transmitter and receiver, and a profiling recorder that both processes the incoming signal and provides a graphic display of the data. The GPR data can be reviewed as both printed hard copy output or recorded on the profiling recorder's hard drive for later review. GeoView uses Mala and GSSI GPR systems. Geological studies are typically conducted using a 200 to 500 MHz antenna.

A GPR survey is conducted along survey lines (transects), which are measured paths along which the GPR antenna is moved. Electronic marks are placed in the data by the operator at designated points along the GPR transects. These marks allow for a correlation between the GPR data and the position of the GPR antenna on the ground.

A GPR survey provides a graphic cross-sectional view of subsurface conditions. This cross-sectional view is created from the reflections of repetitive short-duration electromagnetic (EM) waves that are generated as the antenna is pulled across the ground surface. The reflections occur at the subsurface contacts between materials with differing electrical properties. The electrical property contrast that causes the reflections is the dielectric permittivity that is directly related to conductivity of a material. The GPR method is commonly used to identify such targets as underground utilities, underground storage tanks or drums, buried debris, voids, rebar or geological features.

The greater the electrical contrast between the surrounding materials (earth or concrete) and target of interest, the greater the amplitude of the reflected return signal. Unless the buried object is metal, only part of the signal energy will be reflected back to the antenna with the remaining portion of the signal continuing to propagate downward to be reflected by deeper features. If there is little or no electrical contrast between the target interest and surrounding earth materials it will be very difficult if not impossible to identify the object using GPR.

The depth of penetration of the GPR signal is reduced as the antenna frequency is increased. However, as antenna frequency is increased the resolution of the GPR data is improved. Therefore, when designing a GPR survey a tradeoff is made between the required depth of penetration and desired resolution of the data. As a rule, the highest frequency antenna that will still provide the desired maximum depth of penetration should be used.

Depth estimates are determined by dividing the time of travel of the GPR signal from the ground surface to the top of the feature by the velocity of the GPR signal. The velocity of the GPR signal is usually obtained from published tables of velocities for the type and condition (saturated vs. unsaturated) of soils underlying the site. The accuracy of GPR-derived depths typically ranges from 20 to 40 percent of the total depth.

A2.3 Limitations

The analysis and collection of GPR data is both a technical and interpretative skill. The technical aspects of the work are learned from both training and experience. Having the opportunity to compare GPR data collected in numerous settings to the results from geotechnical studies performed at the same locations develops interpretative skills for karst studies.

The ability of GPR to collect interpretable information at a project site is limited by the attenuation (absorption) of the GPR signal by underlying soils. Once the GPR signal has been attenuated at a particular depth, information regarding deeper geological conditions will not be obtained. GPR data can only resolve subsurface features that have a sufficient electrical contrast between the feature in question and surrounding earth materials. If an insufficient contrast is present, the subsurface feature will not be identified.

GeoView can make no warranties or representations of geological conditions that may be present beyond the depth of investigation or resolving capability of the GPR equipment or in areas that were not accessible to the geophysical investigation.