Report of Geotechnical Exploration
For

JEA Northside Switchyard
4377 Heckscher Drive
Jacksonville, Florida

MAE Project No. 0057-0007
June 30, 2017

Prepared for:

WorleyParsons
resources & energy

JEA
Building Community®

Prepared by:

MAE
Meskel & Associates Engineering
Geotechnical ◤ Environmental ◤ Inspection ◤ Testing

8936 Western Way, Suite 12
Jacksonville, Florida 32256
Phone (904) 519-6990
Fax (904) 519-6992

June 30, 2017

Mr. Russell Coby, P.E.
WorleyParsons
13901 Sutton Park Drive South, Suite 210
Jacksonville, Florida 32224

Meskel & Associates Engineering
Geotechnical ▸ Environmental ▸ Inspection ▸ Testing

P. Rodney Mank, State of Florida, Professional Engineer, License No. 41986. This item has been electronically signed and sealed by P. Rodney Mank, P.E. on 06/30/2017 using a Digital Signature. Printed copies of this document are not considered signed and sealed and the signature must be verified on any electronic copies.

Reference: Report of Geotechnical Exploration
JEA Northside Switchyard
4377 Heckscher Drive
Jacksonville, Florida
MAE Project No. 0057-0007

Dear Mr. Coby:

Meskel & Associates Engineering, LLC has completed a geotechnical exploration for the subject project. Our work was performed in general accordance with our proposal dated May 22, 2017. Authorization was provided through your Task Order No. EO-JEA-00188-MAE. The geotechnical exploration was performed to evaluate the general subsurface conditions within the area of the proposed substation improvements, to provide recommendations for foundation support and design, and to provide recommendations for site preparation during construction. A summary of our findings and related recommendations are presented below; however, we recommend that you consider this report in its entirety.

As further discussed in this report, the soil borings encountered a surficial layer of topsoil, underlain by loose to very dense fine sands (SP), fine sands with silt (SP-SM) and silty fine sands (SM) to the SPT boring termination depths of 40 feet below existing ground surface. Groundwater was encountered at depths of 6 feet 8 inches and 7 feet below the existing ground surface.

Based on the provided loading conditions for the proposed 230 kV Circuit Breaker and 230 kV PT structures, the encountered subsurface conditions are considered adaptable for their support on conventional shallow foundations provided the earthwork recommendations made in this report are followed. Our drilled shaft analysis using the supplied axial, lateral and moment loading conditions shows that a 7-foot diameter drilled shaft constructed to a depth of 20 feet below existing grade will be satisfactory.

We appreciate this opportunity to be of service as your geotechnical consultant on this phase of the project. If you have any questions, or if we may be of any further service, please contact us.

Sincerely,
MESKEL & ASSOCIATES ENGINEERING, LLC
MAE FL Certificate of Authorization No. 28142

Philip R Mank Digitally signed by Philip R Mank
Date: 2017.06.30 15:55:52 -04'00'

W. Josh Mele, E.I.
Staff Engineer

P. Rodney Mank, P.E.
Principal Engineer
Licensed, Florida No. 41986

Distribution: Mr. Russell Coby, P.E. – WorleyParsons 1 PDF

TABLE OF CONTENTS

1.0 PROJECT INFORMATION

1.1 General

Project information was provided to us by you during preparation of the field exploration program via several emails and telephone conversations. We were also provided with a document titled *Geotechnical Service Request*, which outlined the requested services and showed the requested soil boring locations.

1.2 Project Description

The site for the subject project is located at 4377 Heckscher Drive in Jacksonville, Florida. The general site location is shown on Figure 1.

Based on the provided information and our discussions, it is our understanding the proposed project will include modifications to the existing bus and a circuit breaker, and the construction of a new A-Frame and communication equipment. We understand that the A-Frame will be supported on drilled shafts and the communication equipment is anticipated to be founded on shallow foundations. The proposed loading conditions as shown in the provided document are as follows:

Structure	Axial (kip)	Shear (kip)	Moment (k-ft)	Typical Dimensions (ft)
A-Frame	160	20	400	7-foot Diameter
230kV Circuit Breaker	18	20	134	12x7 Mat
230kV PT	2.5	1.0	8.2	5x5 Footing

If actual project information varies from these conditions, then the recommendations in this report may need to be re-evaluated. Any changes in these conditions should be provided so the need for re-evaluation of our recommendations can be assessed prior to final design.

2.0 FIELD EXPLORATION

A field exploration was performed on June 15, 2017. The requested boring locations as shown in the document provided to us were staked at the project site by our field representative. The locations were adjusted to provide a minimum 20-foot clearance from any overhead power lines. Note that B-2 had to be relocated several feet from the requested location to maintain this clearance. A JEA representative then located and marked all underground utilities near the boring locations. The boring locations were then moved as needed to avoid any conflicts. The final boring locations are shown on the *Boring Location Plan*, Figure 2, which is a copy of the aerial provided to us. The locations as shown should be considered approximate.

2.1 SPT Borings

Two Standard Penetration Test (SPT) borings (B-1 and B-2) were performed at the locations shown on Figure 2. The borings were manually advanced using a hand-held bucket auger to a depth of 6 feet below existing grade to avoid potential utility conflicts. The borings were then continued as an

8936 Western Way, Suite 12
Jacksonville, Florida 32256
Phone: (904)519-6990 Fax: (904)519-6992

SPT boring to a depth of 40 feet below the existing ground surface. The SPT portion of each boring was continuously sampled to a depth of 15 feet, and thereafter sampled every 5 feet in general accordance with the methodology outlined in ASTM D-1586. The bucket auger and split-spoon soil samples recovered during performance of the borings were visually described in the field by the field crew, and representative portions of the samples were transported to our laboratory for classification and testing. A summary of the field procedures is included in Appendix A.

2.2 Auger Borings

Two auger borings (A-1 and A-2) were located within 10 feet of the approximate area of each SPT boring, as shown on the *Boring Location Plan*, Figure 2. They were advanced continuously with a flight auger to a depth of approximately 25 feet below the existing ground surface in general accordance with the methodology outlined in ASTM D-1452. The purpose of these borings was to collect soil samples for resistivity testing in the laboratory. The soil samples obtained from the augers were visually described in the field by the field crew. Representative soil samples were returned to our laboratory for classification and testing. A summary of the field procedures is included in Appendix A.

3.0 LABORATORY TESTING

Representative soil samples obtained during our field exploration were visually classified by a geotechnical engineer using the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. A *Key to the Soil Classification System* is included in Appendix A.

3.1 Index Tests

Quantitative laboratory testing was performed on selected samples of the soils encountered during the field exploration to better define the composition of the soils encountered, and to provide data for correlation to their anticipated strength and compressibility characteristics. The laboratory testing determined the percent fines and the natural moisture content of the selected soil samples. The results of the laboratory testing are shown in the *Summary of Laboratory Test Results* included in Appendix B. Also, these results are shown on the *Generalized Soil Profiles* (Figures 3 and 4), and on the *Log of Boring* records at the respective depths from which the tested samples were recovered.

3.2 Resistivity Tests

As requested in the Geotechnical Services Request document, several composite soil samples from the auger borings were selected for resistivity testing. These soil samples were obtained at depths of approximately 2, 5, 10, 15, 20 and 25 feet below existing grade at both auger boring locations. The resistivity tests were conducted in general accordance with Florida Standard Test Method, FM 5-551. The test results are presented in Section 4.6 below.

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 Regional Geology

The study area is located within the Jacksonville Basin in Duval County, Florida. The near-surface geology consists of Plio-Pleistocene unconsolidated sands overlying Pliocene undifferentiated sandy clays and clayey sands to approximately El. -50 feet. Below the undifferentiated sediments, sands, silts and clays of the Hawthorn Group (Marl formation) are present. The Ocala Group is the first formation of the Floridan Aquifer, underlying the Hawthorn Group, which consists of a thick sequence of consolidated carbonate rocks. The Hawthorn Group acts as an aquiclude and separates the shallow water table from the artesian Floridan aquifer within the Ocala Group and lower units. The published potentiometric level for the Floridan aquifer at the project location is on the order of about El. 35 feet.

The Jacksonville Basin formed during the Eocene Period as the Ocala Group (limestone) was deposited. The Ocala Limestone is up to 300 feet thick and unconformably overlies the Avon Park Limestone. Erosion and dissolution of the Ocala Group occurred in late Eocene-early Miocene Periods before the Hawthorn Group was deposited as the sea transgressed. Carbonates from the marine transgression, mixed with clastics from the north, were deposited into the Jacksonville Basin. The Jacksonville Basin gently dips to the northeast and the deepest area of the basin coincides with the thickest Hawthorn sediments which occur near the mouth of the St. Johns River. Regionally, the downwarping of the Jacksonville Basin coincided with the St. Johns Platform Uplift (south of the basin), the Sanford High, and the Ocala Uplift (west and southwest of the basin) in late Eocene-early Miocene Periods.

The Hawthorn Group consists of a highly variable mixture of quartz sand, silt, clay, carbonates and phosphates, and is approximately 400 feet thick in the study area. The Hawthorn Group can be divided into three generalized units. The upper Hawthorn is primarily poorly consolidated dolomites and dolosilts with a mixture of clastics and phosphate. The middle unit is mostly clastic, and the lower unit is predominately dolomite. Occasionally, a lower unit of the Hawthorn will act as part of the Floridan aquifer. Beds of a single component (pure clay) do occur in the Hawthorn but are the exception to a widely varying lithology. Phosphate is nearly always present throughout the Hawthorn Group.

An unconformity exists between the Miocene Hawthorn Group and the overlying undifferentiated sandy clays and clayey sands from the Pliocene. These undifferentiated sediments often contain reworked phosphate from the Hawthorn near the contact. Shell beds and limestone were deposited on top of the Hawthorn in some areas prior to the major regression that occurred during late Miocene Period. The shell beds are primarily in eastern Clay County, and reworked phosphate from the Hawthorn is commonly present. The post-Hawthorn Jacksonville Limestone occurs as a thin horizon in the Jacksonville Downtown area and east Clay County.

4.2 General Soil Profile

Graphical presentation of the generalized subsurface conditions is presented on the *Generalized Soil Profiles* sheets, Figures 3-4. Detailed boring records are included in Appendix A. When reviewing these records, it should be understood that the soil conditions will vary between the boring locations.

In general, the borings encountered a surficial gravel fill layer between 14 and 18-inches in thickness, underlain by loose to very dense fine sands (SP), fine sands with silt (SP-SM) and silty fine sands (SM) to the SPT boring termination depths of approximately 40 feet below existing grade. The corresponding augur borings associated with each SPT location encountered fine sands (SP) and fine sands with silt (SP-SM) to their termination depths of approximately 25 feet below existing grade.

4.3 Groundwater Level

The groundwater level was encountered at each of the boring locations. The groundwater depths were recorded at the time of drilling at depths of 6 feet 8 inches and 7 feet below the existing ground surface. However, it should be anticipated that the groundwater levels will fluctuate seasonally and with changes in climate. As such, we recommend that the water table be verified prior to construction. Measured groundwater levels are shown the boring profiles and boring logs.

4.4 Review of the USDA Web Soil Survey Map

The results of a review of the USDA Soil Survey Conservation Service (SCS) Web Soil Survey of Duval County are shown in the table below. The predominant soil map unit at the project sight is Urban land; however, included is another soil map unit shown to the east of the boring locations on the project site. The soil drainage class, hydrological group, and estimated seasonal high groundwater levels reported in the Soil Survey are as follows:

Soil No.	Soil Type	Drainage Class	Hydrologic Group	Depth to the Water Table(1) (inches)
7	Arents, nearly level	Somewhat Poorly Drained	A	18 to 36
69	Urban land(2)	---	---	---

(1) The "Water table" above refers to a saturated zone in the soil which occurs during specified months, typically the summer wet season. Estimates of the upper limit shown in the Web Soil Survey are based mainly on observations of the water table at selected sites and on evidence of a saturated zone, namely grayish colors (redoximorphic features) in the soil. A saturated zone that lasts for less than a month is not considered a water table.

(2) The Urban land classification does not have an associated soil type, drainage class, hydrologic group, and estimated seasonal high groundwater levels typically reported in the Soil Survey.

4.5 Seasonal High Groundwater Level

In estimating seasonal high groundwater level, several factors are taken into consideration including antecedent rainfall, soil redoximorphic features (i.e., soil mottling), stratigraphy (including presence of hydraulically restrictive layers), vegetative indicators, effects of development, and relief points such as drainage ditches, low-lying areas, etc.

Based on our interpretation of the current site conditions, including the boring logs and review of published data, we estimate the seasonal high groundwater levels at the site to be generally 2 to 3 feet above the water levels measured at the time of our field work.

It is possible that higher groundwater levels may exceed the estimated seasonal high groundwater level as a result of significant or prolonged rains. Therefore, we recommend that design drawings and specifications account for the possibility of groundwater level variations, and construction planning should be based on the assumption that such variations will occur.

4.6 Resistivity Test Results

Resistivity tests were performed on soil samples obtained from both auger locations. These soils were sampled from depths of 2, 5, 10, 15, 20, and 25 feet below existing grade. The sample depths and test results are shown below:

Boring No.	Sample Depth (ft)	Resistivity (Ohm-cm)
A-1	2	38,000
A-1	5	39,000
A-1	10	20,000
A-1	15	25,000
A-1	20	36,000
A-1	25	31,000
A-2	2	52,000
A-2	5	61,000
A-2	10	22,000
A-2	15	19,000
A-2	20	17,000
A-2	25	24,000

5.0 DESIGN RECOMMENDATIONS

5.1 General

The following evaluations and recommendations are based on the provided project information as presented in this report, the results of the field exploration and laboratory testing performed, and the construction techniques recommended in this section and in Section 6.0 below. If the described project conditions are incorrect or changed, or if subsurface conditions encountered during construction are different from those reported, MAE should be notified so that these recommendations can be re-evaluated and revised, if necessary. We recommend that MAE review the final project foundation plans and earthwork specifications to verify that the recommendations in this report have been properly interpreted and implemented.

5.2 Shallow Foundations

Based on the results of our exploration, we consider the encountered subsurface conditions adaptable for support of the proposed 230kV Circuit Breaker and 230kV PT structures on properly designed shallow foundation systems. The site preparation and earthwork construction recommendations outlined in Section 6.0 of this report should be implemented to provide a uniform soil subgrade and reduce the potential for excessive total and differential settlements.

5.2.1 Bearing Pressure

The maximum allowable net soil bearing pressure for use in shallow foundation design should not exceed 3,000 pounds per square foot (psf). Net bearing pressure is defined as the soil bearing pressure at the foundation bearing level in excess of the natural overburden pressure at that level. The foundations should be designed based on the maximum load that could be imposed by all loading conditions.

A modulus of subgrade reaction of 200 pci should be used for mat foundation design. The mat foundation bearing soils should be compacted to at least 98 percent of the soil's modified Proctor maximum dry density, to a depth of at least 2 feet below the foundation bearing level.

5.2.2 Foundation Size

The minimum widths recommended for any isolated spread or strip footings are 24 inches and 18 inches, respectively. Even though the maximum allowable soil bearing pressure may not be achieved, these width recommendations should control the size of the foundations.

5.2.3 Bearing Depth

The exterior foundations should bear at a depth of at least 18 inches below the exterior final grades to provide confinement to the bearing level soils. It is recommended that stormwater be diverted away from the structure exteriors during construction to reduce the possibility of erosion beneath the exterior footings.

5.2.4 Bearing Material

The foundations may bear in either the compacted suitable natural soils or compacted structural fill. The bearing level soils, after compaction, should exhibit densities equivalent to 98 percent of the modified Proctor maximum dry density (ASTM D 1557), to a depth of at least two feet below the foundation bearing levels.

5.2.5 Settlement Estimates

Post-construction settlements of each structure will be influenced by several interrelated factors, such as (1) subsurface stratification and strength/compressibility characteristics; (2) footing size and mat foundation dimensions, bearing level, applied loads, and resulting bearing pressures beneath the foundations; and (3) site preparation and earthwork construction techniques used by the contractor. Our settlement estimates for each structure are based on the use of site preparation/earthwork construction techniques as recommended in Section 6.0 of this report. Any deviation from these recommendations could result in an increase in the estimated post-construction settlement of the structures.

Due to the sandy nature of the near-surface soils, we expect the majority of settlement to occur in an elastic manner and fairly rapidly during construction. Using the recommended maximum bearing pressure, the supplied structural loads, and the field and laboratory test data that we have correlated to geotechnical strength and compressibility characteristics of the subsurface soils, we estimate that total settlement of each structure to be on the order of one inch or less.

Differential settlements result from differences in applied bearing pressures and variations in the compressibility characteristics of the subsurface soils. Because of the general uniformity of the subsurface conditions and the recommended site preparation and earthwork construction techniques outlined in Section 6.0, we anticipate that differential settlements of each structure to be less than one-half inch.

5.3 Drilled Shaft

Based on the subsurface conditions encountered in the soil borings and the provided loading information, it is our opinion that the encountered subsurface conditions are suitable for supporting the proposed A-Frame structure on a drilled shaft foundation. To estimate the allowable load of the

8936 Western Way, Suite 12
Jacksonville, Florida 32256
Phone: (904)519-6990 Fax: (904)519-6992

shaft, the computer software All-Pile, produced by CivilTech Software, was used to model the shaft and encountered subsurface conditions. The soil design properties used for the drilled shaft analysis are shown in Table 1 in Appendix C. Our analyses encompassed a variety of different shaft sizes, in order to determine the required diameter and embedment depth needed to provide the required foundation support. For the analyses, all loads were applied at the top ground surface level of a free-standing shaft, and the design groundwater level was assumed to be at the existing ground surface.

Based on the encountered subsurface conditions and the axial, lateral and moment loading conditions provided, our analysis shows that a 7-foot (84-inch) diameter drilled shaft constructed to a depth of 20 feet below existing grade will be satisfactory. The results of the vertical and lateral analyses of the 84-inch diameter drilled shaft is presented in Appendix C. Soil parameters used in the analyses are shown on the output included in the appendix.

5.3.1 Drilled Shaft Analysis

The allowable drilled shaft capacity is a function of the structural strength of the concrete and the strength of the supporting soil. The following discussion refers to the allowable soil capacity. The concrete strength should be selected based on the actual pile loads. The lesser of the concrete or soil strength will govern the maximum allowable pile capacity. A minimum concrete strength of 4,000 psi is recommended.

Our analysis of the 84-inch diameter drilled shaft was based on the anticipated design loads as provided to us and the encountered subsurface conditions. In addition, we understood the settlement tolerances per shaft to be one inch under vertical (compressive) load, and one inch laterally under the design shear load.

Borings B-1and B-2 were used to model the site soil profile. In summary, both borings encountered medium dense fine sands (SP) and fine sands with silt (SP-SM) to depths of approximately 12 feet below existing grade, underlain by alternating layers of loose to medium dense fine sands with silt (SP-SM) to about 33 feet depth, and then dense to very dense fine sands with silt (SP-SM) and silty fine sands (SM) to the boring termination depth of 40 feet below existing grade.

Based on the results of the borings, both side friction and end bearing (tip resistance) were considered as contributing to the allowable capacity of the shaft for our analysis. We also chose to use safety factors of 2.5 for side friction and 3.0 for end bearing (tip resistance) in the compression load analysis, and 2.5 in tension (uplift). Using these limits, an 84-inch diameter shaft will achieve the design vertical loads with a shaft embedment depth of 20 feet below existing grade. At that depth, the estimated shaft settlement is 0.6-inch. This settlement estimate is based upon the use of (1) the field test data obtained during our geotechnical exploration, which has been correlated to geotechnical strength and compressibility characteristics of the subsurface soils beneath the site, and (2) published theoretical and empirical methods of settlement analysis for deep foundations bearing on soils similar to those at this site. The lateral analysis for the 84-inch diameter shaft resulted in a lateral deflection of 0.31-inch. Therefore, it appears that an 84-inch diameter drilled shaft embedded at a depth of 20 feet below existing grade will support the proposed loading conditions.

Appendix C includes output for allowable compressive and tensile capacities versus shaft depth for the 84-inch diameter drilled shaft. The appendix also includes the results of the lateral analysis for this shaft diameter.

5.3.2 Shaft Group Effects

We recommend a minimum shaft spacing to shaft diameter ratio (S/D) of 3.0. Using this minimum S/D ratio, we anticipate that any capacity reductions due to group effects of individual shafts which are installed within a group of piles should be small and therefore should be considered insignificant in the design of the foundation system.

5.4 Construction Considerations

5.4.1 Construction Techniques

Drilled shafts should be installed by an experienced contractor having at least 5 years of continuous experience constructing drilled shafts. The auger used to drill the shaft should be drilled continuously to the shaft tip elevation while maintaining plumbness throughout the excavation process. The shafts should be constructed using the wet method. The mineral slurry should be dense enough to prevent cave-in of the side soils that would reduce the area of the shaft.

Once the shaft has reached the design bottom elevation, the concrete should be placed inside the shaft using the tremie method. If temporary casing is used, a minimum concrete head of 5 feet must be maintained between the top of the concrete and the top of the casing as it is being withdrawn to reduce the possibility of cave-in of the side soils.

5.4.2 Installation Sequences

Once the shaft concrete has been placed, adjacent shafts located within six shaft diameters, center-to-center, should not be constructed until the concrete has achieved its initial set, which typically occurs approximately 24 hours after shaft construction. This time delay allows the "green" concrete in the recently constructed shaft to harden, and helps reduce the possible loss of concrete into adjacent shafts during their construction process.

5.4.3 Steel Placement

After augering to the desired shaft tip elevation and the auger removed, the steel reinforcement can be placed into the shaft prior to concrete placement. The alignment and concrete cover of the rebar, when placed by this method, can vary. Accordingly, it is recommended that centralizers be placed every 5 feet to ensure proper concrete coverage.

5.4.4 Quality Control

Since the drilled shaft is a "cast-in-place" foundation, the quality of the shaft construction is dependent upon the skill, experience, and techniques used by the foundation contractor. We recommend that a geotechnical engineer or an experienced engineering technician, acting under the supervision and direction of the geotechnical engineer, observe and monitor the drilled shaft installations. Duties should consist of, but not be limited to, the following:

1. Verify that the shafts are augered to the design tip bearing level and that a vertical orientation is maintained during augering.
2. Monitor the slurry conditions as the auger is drilled to the shaft tip elevation (i.e. “wet method”), and that the concrete tremie operation is continuous and maintains the proper concrete head if temporary casing is used, to help verify that a continuous shaft cross-section is maintained during construction.
3. Record the volume of concrete required to construct the shaft.
4. Obtain samples of the fresh concrete to mold compressive strength cylinders to make sure the compressive strength meets design requirements.
5. Monitor the installation of steel reinforcement to verify that the size, length, configuration, and placement of the steel conform to the job specifications.

In addition to the above inspection duties during construction, we recommend that the shaft be inspected post-construction to make sure that the shaft cross-section is maintained through the entire length of the shaft. We recommend that cross-hole sonic logging or thermal integrity testing be used to verify shaft integrity.

We recommend that our firm be provided the opportunity to make a general review of the foundation plans and the foundation construction and earthwork specifications. If necessary, we will suggest any modifications that may be required to verify that our recommendations have been properly interpreted and implemented. We feel that our experience with drilled shaft construction could be helpful in preparing specifications for the foundation construction of this project.

6.0 SITE PREPARATION AND EARTHWORK RECOMMENDATIONS

Site preparation as outlined in this section should be performed to provide more uniform foundation bearing conditions for shallow foundations, and to reduce the potential for post-construction settlements of the planned structures.

6.1 Clearing and Stripping

Prior to construction, the location of existing underground utility lines within the construction area should be established. Provisions should then be made to relocate interfering utilities to appropriate locations. It should be noted that, if underground pipes are not properly removed or plugged, they may serve as conduits for subsurface erosion, which may subsequently lead to excessive settlement of overlying structures.

The "footprint" of the proposed construction areas, plus a minimum additional margin of 5 feet, should be stripped of all surface vegetation, stumps, debris, organic topsoil, or other deleterious materials, as well as any existing slabs-on-grade or pavements (surface and base courses). During grubbing operations, roots with a diameter greater than 0.5-inch, stumps, or small roots in a concentrated state, should be grubbed and completely removed.

The actual depths of unsuitable soils and materials should be determined by MAE using visual observation and judgment during earthwork operations. Any topsoil removed from the construction area can be stockpiled and used subsequently in areas to be grassed.

6.2 Temporary Groundwater Control

Based on the groundwater level encountered at the time of our field exploration, and the estimated Seasonal High Groundwater level, we do not anticipate the need for a dewatering system given our understanding of the proposed construction. However, should groundwater control measures become necessary, dewatering methods should be determined by the contractor. We recommend the groundwater control measure remain in place until compaction of the existing soils is completed and until backfilling/site filling has reached a height of 2 feet above the groundwater level at the time of construction. The site should be graded to direct surface water runoff from the construction area.

Note that discharge of produced groundwater to surface waters of the state from dewatering operations or other site activities is regulated and requires a permit from the State of Florida Department of Environmental Protection (FDEP). This permit is termed a Generic Permit for the Discharge of Produced Groundwater From Any Non-Contaminated Site Activity. If discharge of produced groundwater is anticipated, we recommend sampling and testing of the groundwater early in the site design phase to prevent project delays during construction. MAE can provide the sampling, testing, and professional consulting services required to evaluate compliance with the regulations.

8936 Western Way, Suite 12
Jacksonville, Florida 32256
Phone: (904)519-6990 Fax: (904)519-6992

6.3 Compaction

After completing the clearing and stripping operations, and installing the temporary groundwater control measures if required, the exposed surface soils should be compacted with a vibratory drum roller operating in the static mode having a minimum static, at-drum weight, on the order of 10 tons. Typically, the soils should exhibit moisture contents within ±2 percent of the modified Proctor optimum moisture content (ASTM D 1557) during the compaction operations. Compaction should continue until densities of at least 98 percent of the modified Proctor maximum dry density (ASTM D 1557) have been achieved within the upper 24 inches below the compacted surface.

Should the bearing level soils experience pumping and soil strength loss during the compaction operations, compaction work should be immediately terminated, and (1) the disturbed soils should be removed and backfilled with dry structural fill soils, which are then compacted, or (2) the excess moisture content within the disturbed soils should be allowed to dissipate before recompacting.

Care should be exercised to avoid damaging any nearby structures while the compaction operation is underway. Prior to commencing compaction, occupants of adjacent structures should be notified, and the existing conditions of the structures should be documented with photographs and survey (if deemed necessary). Due to the proximity of the existing electrical substation structures, the vibratory roller should be operated in the static mode. Alternatively, a track mounted bulldozer may be used if compaction as recommended alone can be achieved.

6.4 Structural Backfill and Fill Soils

Any structural backfill or fill required for site development should be placed in loose lifts not exceeding 12 inches in thickness and compacted by the use of the above described vibratory drum roller operating in the static mode, or by track-mounted compaction equipment. If hand-held compaction equipment is used, the lift thickness should be reduced to 6 inches.

Structural fill is defined as a non-plastic, inorganic, granular soil containing less than 12 percent material passing the No. 200 mesh sieve and containing less than 4 percent organic material. The SP and SP-SM soils as encountered at the soil boring locations are considered suitable for use as structural fill. It should be noted that soils with more than 12 percent passing the No. 200 sieve will be more difficult to compact, due to their nature to retain soil moisture, and may require drying. Typically, the material should exhibit moisture contents within ±2 percent of the modified Proctor optimum moisture content (ASTM D-1557) during the compaction operations. Compaction should continue until densities of at least 98 percent of the modified Proctor maximum dry density (ASTM D-1557) have been achieved within each lift of the compacted structural fill.

6.5 Foundation Areas

After satisfactory placement and compaction of the required structural fill, the shallow foundation areas may be excavated to the planned bearing levels. The foundation bearing level soils, after compaction, should exhibit densities equivalent to 98 percent of the modified Proctor maximum dry density (ASTM D-1557), to a depth of 2 feet below the bearing level. For confined areas (if encountered), any additional compaction operations can probably best be performed by the use of a lightweight vibratory sled or roller having a total weight on the order of 500 to 2,000 pounds.

7.0 QUALITY CONTROL TESTING

A representative number of field in-place density tests should be made in the upper 2 feet of compacted natural soils, in each lift of compacted backfill and fill, and in the upper 12 inches below the foundation bearing levels. The density tests are considered necessary to verify that satisfactory compaction operations have been performed. We recommend density testing be performed at a minimum of one location within the 230kV PT structure, and a minimum of 2 locations within the Circuit Breaker mat slab area.

8.0 REPORT LIMITATIONS

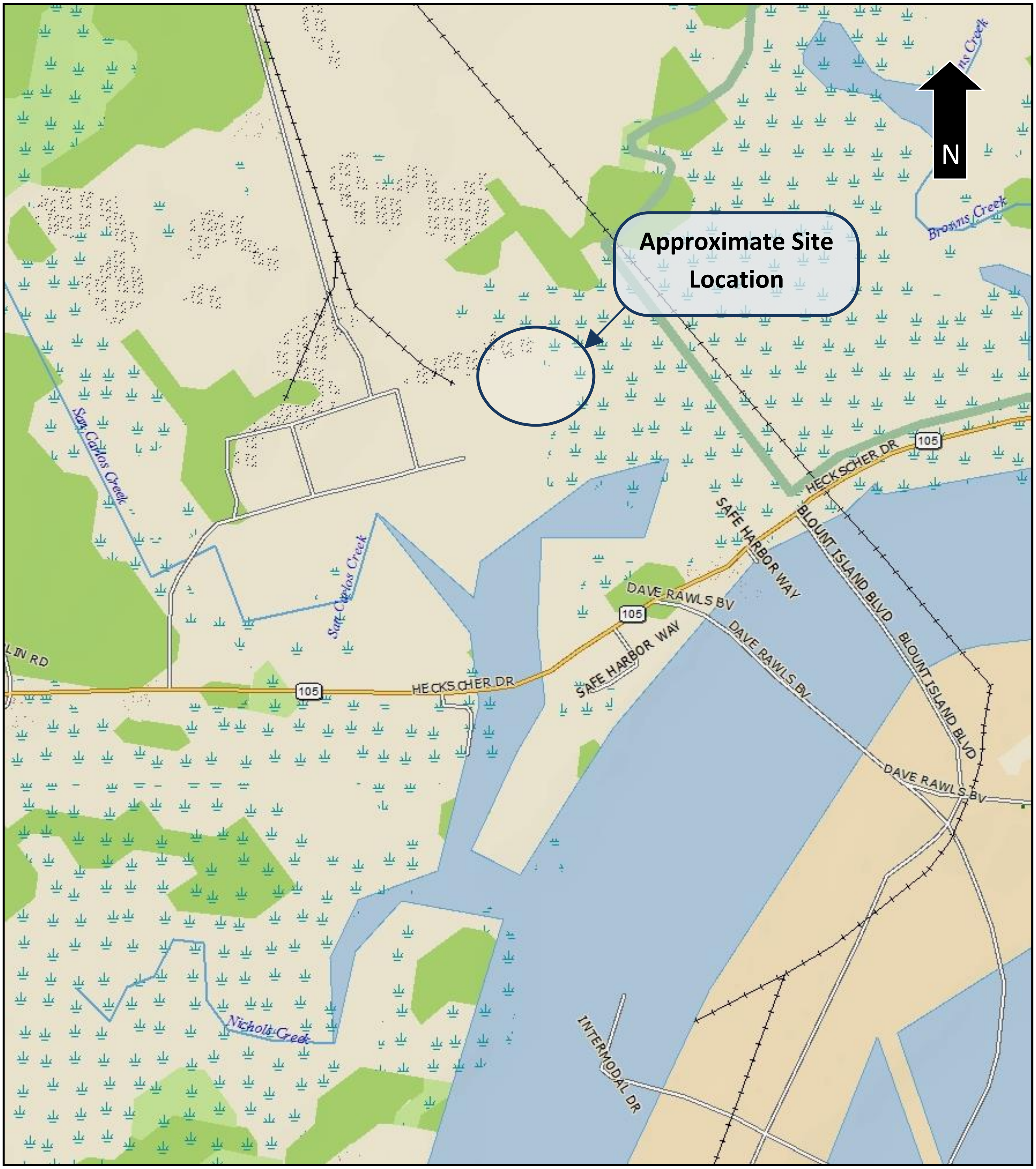
This report has been prepared for the exclusive use of WorleyParsons and the JEA for specific application to the design and construction of the JEA Northside Switchyard project. An electronically signed and sealed version, and a version of our report that is signed and sealed in blue ink, may be considered an original of the report. Copies of an original should not be relied on unless specifically allowed by MAE in writing. Our work for this project was performed in accordance with generally accepted geotechnical engineering practice. No warranty, express or implied, is made.

The analyses and recommendations contained in this report are based on the data obtained from the borings performed for this project. This testing indicates subsurface conditions only at the specific locations and times, and only to the depths explored. These results do not reflect subsurface variations that may exist between the boring locations and/or at depths below the boring termination depths. Subsurface conditions and groundwater levels at other locations may differ from conditions occurring at the tested locations. In addition, it should be understood that the passage of time may result in a change in the conditions at the tested locations. If variations in subsurface conditions from those described in this report are observed during construction, the recommendations in this report must be re-evaluated.

The scope of our services did not include any environmental assessment or testing for the presence or absence of hazardous or toxic materials in the soil, groundwater, or surface water within or beyond the subject site. Any statements made in this report, and/or notations made on the generalized soil profiles or boring logs, regarding odors or other potential environmental concerns are based on observations made during execution of our scope of services and as such are strictly for the information of our client. No opinion of any environmental concern of such observations is made or implied. Unless complete environmental information regarding the site is already available, an environmental assessment is recommended.

If changes in the design or location of the structures occur, the conclusions and recommendations contained in this report may need to be modified. We recommend that these changes be provided to us for our consideration. MAE is not responsible for conclusions, interpretations, opinions or recommendations made by others based on the data contained in this report.

Figures

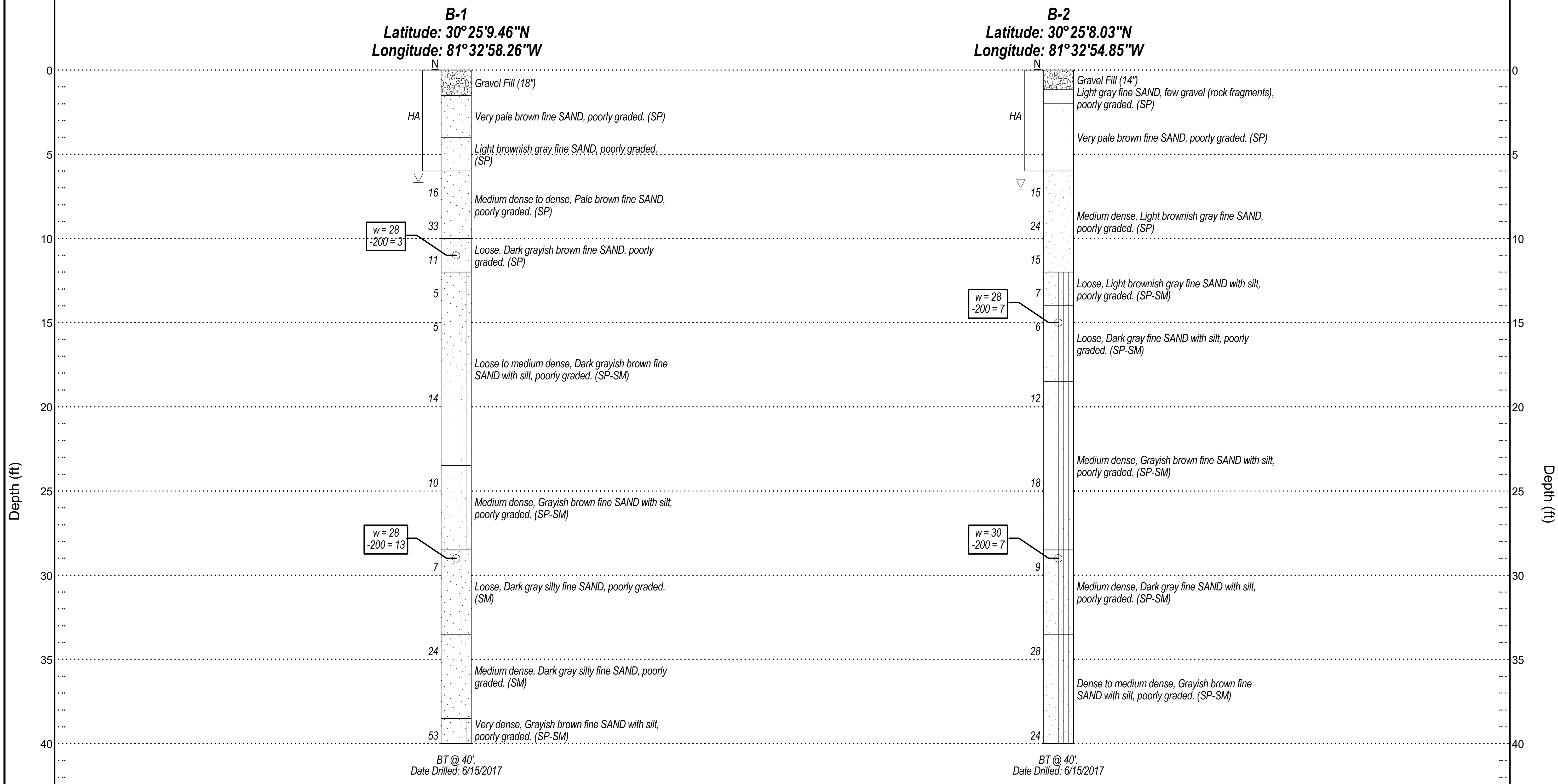


Site Location Map

PREPARED BY	PROJECT NAME	
MAE Meskel & Associates Engineering	**JEA Northside Switchyard Jacksonville, Florida**	
	REFEREENCE	SCALE
	Delorme XMap 7.0	NTS
PREPARED FOR	MAE PROJECT NO.	**FIGURE NO.**
WorleyParsons	0057-0007	1

Boring Location Plan

PREPARED BY	PROJECT NAME	
Meskel & Associates Engineering, LLC FL Certificate of Authorization No. 28142 8936 Western Way, Suite 12, Jacksonville, FL 32256	JEA Northside Switchyard Jacksonville, Florida	
	REFERENCE	SCALE
	GOOGLE EARTH	NTS
PREPARED FOR	MAE PROJECT NO.	FIGURE NO.
WorleyParsons	0057-0007	2



Legend

Gravel Fill
Fine Sand
Fine Sand with Silt
Silty Fine Sand

N	Standard Penetration Resistance, Blows/Foot
BT	Boring Terminated at Depth Below Existing Grade
(SP)	Unified Soil Classification System (USCS)
▽	Depth to Groundwater at Time of Drilling
w	Natural Moisture Content (%)
-200	% Passing No. 200 U.S. Standard Sieve
HA	Boring Advanced by hand-held bucket auger due to possible underground utilities.

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

P. RODNEY MANK, P.E. P.E. NO.: 41986
MAE
Meskel & Associates Engineering
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12, Jacksonville, FL 32256

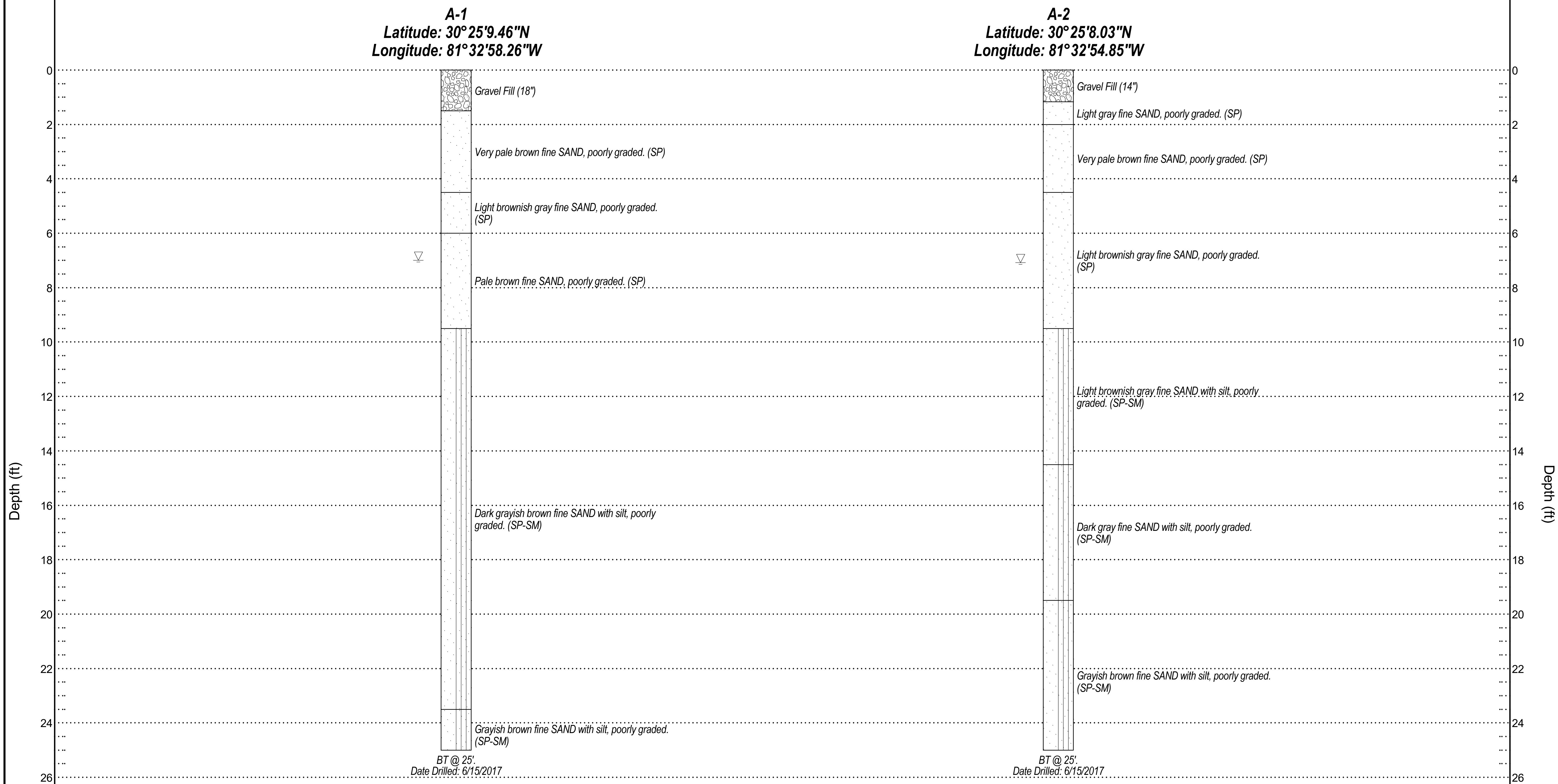
WorleyParsons

DATE:	MAE PROJECT NO.
6/30/2017	0057-0007

SHEET TITLE: Generalized Soil Profiles

PROJECT NAME: JEA Northside Switchyard
Jacksonville, Florida

FIGURE NO. 3



Legend

Gravel Fill

Fine Sand

Fine Sand with Silt

(SP) Unified Soil Classification System (USCS)

Depth to Groundwater at Time of Drilling

BT Boring Terminated at Depth Below Existing Grade

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

P. RODNEY MANK, P.E. P.E. NO.: 41986

MAE
Meskel & Associates Engineering
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12, Jacksonville, FL 32256

WorleyParsons

DATE:	MAE PROJECT NO.
6/30/2017	0057-0007

SHEET TITLE: Generalized Soil Profiles

PROJECT NAME: JEA Northside Switchyard
Jacksonville, Florida

FIGURE NO. 4

Appendix A

Meskel & Associates Engineering, LLC
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12
Jacksonville, FL 32256
P: (904)519-6990 F: (904)519-6992

Meskel & Associates Engineering

BORING B-1

PROJECT NO. 0057-0007

PROJECT NAME JEA Northside Switchyard

PROJECT LOCATION Jacksonville, Florida **CLIENT** WorleyParsons

DATE STARTED 6/15/17 **COMPLETED** 6/15/17 **LATITUDE** 30°25'9.46"N **LONGITUDE** 81°32'58.26"W

DRILLING CONTRACTOR MAE, LLC **DRILLING METHOD** Standard Penetration Test

LOGGED BY P.R.Young **CHECKED BY** W. Josh Mele **GROUND ELEVATION** — **HAMMER TYPE** Automatic

DEPTH (ft)	SAMPLE DEPTH NUMBER	MATERIAL DESCRIPTION	USCS	GRAPHIC LOG	BLOW COUNTS	N-VALUE	MOISTURE CONTENT (%)	FINES CONTENT (%)	ORGANIC CONTENT (%)	LIQUID LIMIT	PLASTICITY INDEX	POCKET PEN. (tsf)	RECOVERY % (RQD)	REMARKS
0	1	Gravel Fill (18")												
	2	Very pale brown fine SAND, poorly graded.	SP											
5	3	Light brownish gray fine SAND, poorly graded.	SP											
	4	∇ Medium dense to dense, Pale brown fine SAND, poorly graded.	SP		6 7 9 15	16								
	5				8 15 18 21	33								
10	6	Loose, Dark grayish brown fine SAND, poorly graded.	SP		3 5 6 7	11	28	3						
	7	Loose to medium dense, Dark grayish brown fine SAND with silt, poorly graded.	SP-SM		1 2 3 6	5								
15	8				1 2 3 3	5								
	9				5 8 6	14								
20														

NOTES *Boring Advanced by hand-held bucket auger to 6 feet due to possible underground utilities.*

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 6 ft 8 in *∇ **END OF DAY** ---

(Continued Next Page)

Meskel & Associates Engineering, LLC
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12
Jacksonville, FL 32256
P: (904)519-6990 F: (904)519-6992

BORING B-1

PROJECT NO. 0057-0007

PROJECT NAME JEA Northside Switchyard

PROJECT LOCATION Jacksonville, Florida **CLIENT** WorleyParsons

DEPTH (ft)	SAMPLE DEPTH NUMBER	MATERIAL DESCRIPTION	USCS	GRAPHIC LOG	BLOW COUNTS	N-VALUE	MOISTURE CONTENT (%)	FINES CONTENT (%)	ORGANIC CONTENT (%)	LIQUID LIMIT	PLASTICITY INDEX	POCKET PEN. (tsf)	RECOVERY % (RQD)	REMARKS
20		Loose to medium dense, Dark grayish brown fine SAND with silt, poorly graded. *(continued)*	SP-SM											
25	10	Medium dense, Grayish brown fine SAND with silt, poorly graded.	SP-SM		3 4 6	10								
30	11	Loose, Dark gray silty fine SAND, poorly graded.	SM		4 4 3	7	28	13						
35	12	Medium dense, Dark gray silty fine SAND, poorly graded.	SM		5 7 17	24								
40	13	Very dense, Grayish brown fine SAND with silt, poorly graded.	SP-SM		15 25 28	53								
		Bottom of borehole at 40 feet.												

NOTES

GROUND WATER LEVELS

▽ **AT TIME OF DRILLING** 6 ft 8 in *▽ **END OF DAY** ---

Meskel & Associates Engineering, LLC
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12
Jacksonville, FL 32256
P: (904)519-6990 F: (904)519-6992

Meskel & Associates Engineering

BORING B-2

PROJECT NO. 0057-0007

PROJECT NAME	JEA Northside Switchyard		
PROJECT LOCATION	Jacksonville, Florida	**CLIENT**	WorleyParsons
DATE STARTED	6/15/17 **COMPLETED** 6/15/17	**LATITUDE**	30°25'8.03"N **LONGITUDE** 81°32'54.85"W
DRILLING CONTRACTOR	MAE, LLC	**DRILLING METHOD**	Standard Penetration Test
LOGGED BY	P.R.Young **CHECKED BY** W. Josh Mele	**GROUND ELEVATION**	— **HAMMER TYPE** Automatic

DEPTH (ft)	SAMPLE DEPTH NUMBER	MATERIAL DESCRIPTION	USCS	BLOW COUNTS	N-VALUE	MOISTURE CONTENT (%)	FINES CONTENT (%)	ORGANIC CONTENT (%)	LIQUID LIMIT	PLASTICITY INDEX	POCKET PEN. (tsf)	RECOVERY % (RQD)	REMARKS
0	1	Gravel Fill (14")											
		Light gray fine SAND, few gravel (rock fragments), poorly graded.	SP										
	2, 3	Very pale brown fine SAND, poorly graded.	SP										
5	4	▽ Medium dense, Light brownish gray fine SAND, poorly graded.	SP	3-6-9-11	15								
	5			10-11-13-15	24								
10	6			4-7-8-11	15								
	7	Loose, Light brownish gray fine SAND with silt, poorly graded.	SP-SM	3-3-4-4	7								
15	8	Loose, Dark gray fine SAND with silt, poorly graded.	SP-SM	2-3-3-4	6	28	7						
20	9	Medium dense, Grayish brown fine SAND with silt, poorly graded.	SP-SM	4-5-7	12								

NOTES *Boring Advanced by hand-held bucket auger to 6 feet due to possible underground utilities.*

GROUND WATER LEVELS

▽ **AT TIME OF DRILLING** 7 ft 0 in   ▽ **END OF DAY** ---

(Continued Next Page)

Meskel & Associates Engineering, LLC
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12
Jacksonville, FL 32256
P: (904)519-6990 F: (904)519-6992

Meskel & Associates Engineering

BORING B-2

PROJECT NO. 0057-0007

PROJECT NAME JEA Northside Switchyard

PROJECT LOCATION Jacksonville, Florida **CLIENT** WorleyParsons

DEPTH (ft)	SAMPLE DEPTH NUMBER	MATERIAL DESCRIPTION	USCS	GRAPHIC LOG	BLOW COUNTS	N-VALUE	MOISTURE CONTENT (%)	FINES CONTENT (%)	ORGANIC CONTENT (%)	LIQUID LIMIT	PLASTICITY INDEX	POCKET PEN. (tsf)	RECOVERY % (RQD)	REMARKS
20														
25	10	Medium dense, Grayish brown fine SAND with silt, poorly graded. *(continued)*	SP-SM		4 8 10	18								
30	11				2 4 5	9	30	7						
		Medium dense, Dark gray fine SAND with silt, poorly graded.	SP-SM											
35	12				9 12 16	28								
		Dense to medium dense, Grayish brown fine SAND with silt, poorly graded.	SP-SM											
40	13				9 12 12	24								
		Bottom of borehole at 40 feet.												

NOTES

GROUND WATER LEVELS

AT TIME OF DRILLING 7 ft 0 in

END OF DAY ---

NEW MAE LOG LAT/LONG-EOD - NEW TEMPLATE 7-30-12.GDT - 6/26/17 13:59 - F:\GINT\GINT FILES\PROJECTS\0057-0007\JEA NORTHSIDE SWITCHYARD.GPJ

Meskel & Associates Engineering, LLC
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12
Jacksonville, FL 32256
P: (904)519-6990 F: (904)519-6992

Meskel & Associates Engineering

BORING A-1

PROJECT NO. 0057-0007

PROJECT NAME JEA Northside Switchyard

PROJECT LOCATION Jacksonville, Florida **CLIENT** WorleyParsons

DATE STARTED 6/15/17 **COMPLETED** 6/15/17 **LATITUDE** 30°25'9.46"N **LONGITUDE** 81°32'58.26"W

DRILLING CONTRACTOR MAE, LLC **DRILLING METHOD** Flight Auger

LOGGED BY P.R.Young **CHECKED BY** W. Josh Mele **GROUND ELEVATION** — **HAMMER TYPE** —

DEPTH (ft)	SAMPLE DEPTH NUMBER	MATERIAL DESCRIPTION	USCS	GRAPHIC LOG	BLOW COUNTS	N-VALUE	MOISTURE CONTENT (%)	FINES CONTENT (%)	ORGANIC CONTENT (%)	LIQUID LIMIT	PLASTICITY INDEX	POCKET PEN. (tsf)	RECOVERY % (RQD)	REMARKS
0		Gravel Fill (18")												
	1	Very pale brown fine SAND, poorly graded.	SP											
5	2	Light brownish gray fine SAND, poorly graded.	SP											
		▽ Pale brown fine SAND, poorly graded.	SP											
10	3	Dark grayish brown fine SAND with silt, poorly graded.	SP-SM											
15	4													
20	5													
	6	Grayish brown fine SAND with silt, poorly graded.	SP-SM											
25														

Bottom of borehole at 25 feet.

NOTES

GROUND WATER LEVELS

▽ **AT TIME OF DRILLING** 7 ft 0 in *▽ **END OF DAY** ---

Meskel & Associates Engineering, LLC
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12
Jacksonville, FL 32256
P: (904)519-6990 F: (904)519-6992

BORING A-2

PROJECT NO. 0057-0007

PROJECT NAME JEA Northside Swithcyard

PROJECT LOCATION Jacksonville, Florida **CLIENT** WorleyParsons

DATE STARTED 6/15/17 **COMPLETED** 6/15/17 **LATITUDE** 30°25'8.03"N **LONGITUDE** 81°32'54.85"W

DRILLING CONTRACTOR MAE, LLC **DRILLING METHOD** Flight Auger

LOGGED BY P.R.Young **CHECKED BY** W. Josh Mele **GROUND ELEVATION** — **HAMMER TYPE** —

DEPTH (ft)	SAMPLE DEPTH NUMBER	MATERIAL DESCRIPTION	USCS	GRAPHIC LOG	BLOW COUNTS	N-VALUE	MOISTURE CONTENT (%)	FINES CONTENT (%)	ORGANIC CONTENT (%)	LIQUID LIMIT	PLASTICITY INDEX	POCKET PEN. (tsf)	RECOVERY % (RQD)	REMARKS
0		Gravel Fill (14")												
	1	Light gray fine SAND, poorly graded.	SP											
		Very pale brown fine SAND, poorly graded.	SP											
5	2	∇ Light brownish gray fine SAND, poorly graded.	SP											
10	3	Light brownish gray fine SAND with silt, poorly graded.	SP-SM											
15	4	Dark gray fine SAND with silt, poorly graded.	SP-SM											
20	5, 6	Grayish brown fine SAND with silt, poorly graded.	SP-SM											
25														

Bottom of borehole at 25 feet.

NOTES

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 7 ft 1 in *∇ **END OF DAY** ---

KEY TO BORING LOGS – USCS

Soil Classification

Soil classification of samples obtained at the boring locations is based on the Unified Soil Classification System (USCS). Coarse grained soils have more than 50% of their dry weight retained on a #200 sieve. Their principal descriptors are: sand, cobbles and boulders. Fine grained soils have less than 50% of their dry weight retained on a #200 sieve. They are principally described as clays if they are plastic and silts if they are slightly to non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

BORING LOG LEGEND

Symbol	Description
N	Standard Penetration Resistance, the number of blows required to advance a standard spoon sampler 12" when driven by a 140-lb hammer dropping 30".
WOR	Split Spoon sampler advanced under the weight of the drill rods
WOH	Split Spoon sampler advanced under the weight of the SPT hammer
50/2”	Indicates 50 hammer blows drove the split spoon 2 inches; 50 Hammer blows for less than 6-inches of split spoon driving is considered “Refusal”.
(SP)	Unified Soil Classification System
-200	Fines content, % Passing No. 200 U.S. Standard Sieve
w	Natural Moisture Content (%)
OC	Organic Content (%)
LL	Liquid Limit
PI	Plasticity Index
NP	Non-Plastic
PP	Pocket Penetrometer in tons per square foot (tsf)

MODIFIERS

SECONDARY CONSTITUENTS (Sand, Silt or Clay)	
Trace	Less than 5%
With	5% to 12%
Sandy, Silty or Clayey	12% to 35%
Very Sandy, Very Silty or Very Clayey	35% to 50%

ORGANIC CONTENT	
Trace	2% or less
Few	3% to 5%
Little	5% to 10%
With	Greater than 10%

MINOR COMPONENTS (Shell, Rock, Debris, Roots, etc.)	
Trace	Less than 5%
Few	5% to 10%
Little	15% to 25%
Some	30% to 45%

RELATIVE DENSITY (Coarse-Grained Soils)

Relative Density	N-Value *
Very Loose	Less than 3
Loose	3 to 8
Medium Dense	8 to 24
Dense	24 to 40
Very Dense	Greater than 40

CONSISTENCY (Fine-Grained Soils)

Consistency	N-Value *
Very Soft	Less than 1
Soft	1 to 3
Firm	3 to 6
Stiff	6 to 12
Very Stiff	12 to 24
Hard	Greater than 24

RELATIVE HARDNESS (Limestone)

Relative Hardness	N-Value *
Soft	Less than 50
Hard	Greater than 50

* Using Automatic Hammer

FIELD EXPLORATION PROCEDURES

Standard Penetration Test (SPT) Borings

The Standard Penetration Test (SPT) boring(s) were performed in general accordance with the latest revision of ASTM D 1586, "Penetration Test and Split-Barrel Sampling of Soils." The borings were advanced by rotary drilling techniques. A split-barrel sampler was inserted to the borehole bottom and driven 18 to 24 inches into the soil using a 140 pound hammer falling an average of 30 inches per hammer blow. The number of hammer blows for the final 12 inches of penetration (18" sample) or for the sum of the middle 12 inches of penetration (24" sample) is termed the "penetration resistance, blow count, or N-value." This value is an index to several in-situ geotechnical properties of the material tested, such as relative density and Young's Modulus.

After driving the sampler, it was retrieved from the borehole and representative samples of the material within the split-barrel were containerized and sealed. After completing the drilling operations, the samples for each boring were transported to the laboratory where they were examined by our engineer in order to verify the field descriptions.

Flight Auger Borings

The auger boring(s) were performed mechanically by the use of a continuous-flight auger attached to a drill rig in general accordance with the latest revision of ASTM D 1452, "Soil Investigation and Sampling by Auger Borings." Representative samples of the soils brought to the ground surface by the augering process were visually described in the field, and representative portions of the samples were obtained for further evaluation by a geotechnical engineer.

Unified Soil Classification System (USCS)
(from ASTM D 2487)

Major Divisions			Group Symbol	Typical Names
Coarse-Grained Soils More than 50% retained on the 0.075 mm (No. 200) sieve	**Gravels** 50% or more of coarse fraction retained on the 4.75 mm (No. 4) sieve	Clean Gravels	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		Gravels with Fines	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands 50% or more of coarse fraction passes the 4.75 (No. 4) sieve	Clean Sands	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		Sands with Fines	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
Fine-Grained Soils More than 50% passes the 0.075 mm (No. 200) sieve	**Silts and Clays** Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock four, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly/sandy/silty/lean clays
			OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays Liquid Limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck, and other highly organic soils

Prefix: G = Gravel, S = Sand, M = Silt, C = Clay, O = Organic
Suffix: W = Well Graded, P = Poorly Graded, M = Silty, L = Clay, LL < 50%, H = Clay, LL > 50%

MAE
Meskel & Associates Engineering

Appendix B

Meskel & Associates Engineering, LLC
FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12
Jacksonville, FL 32256
P: (904)519-6990 F: (904)519-6992

Meskel & Associates Engineering

SUMMARY OF LABORATORY TEST RESULTS

PROJECT NO. 0057-0007

PROJECT NAME JEA Northside Switchyard

DATE. 6/26/2017

PROJECT LOCATION Jacksonville, Florida

CLIENT WorleyParsons

Borehole	Sample No.	Approx. Depth (ft)	%<#200 Sieve	Water Content (%)	Organic Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification	Comments
B-1	6	11	3	28	---	---	---	---	SP	
B-1	11	29	13	28	---	---	---	---	SM	
B-2	8	15	7	28	---	---	---	---	SP-SM	
B-2	11	29	7	30	---	---	---	---	SP-SM	

Note: "---" Untested Parameter

LABORATORY TEST PROCEDURES

Percent Fines Content

The percent fines or material passing the No. 200 mesh sieve of the sample tested was determined in general accordance with the latest revision of ASTM D 1140. The percent fines are the soil particles in the silt and clay size range.

Natural Moisture Content

The water content of the tested sample was determined in general accordance with the latest revision of ASTM D 2216. The water content is defined as the ratio of “pore” or “free” water in a given mass of material to the mass of solid material particles.

Appendix C

TABLE 1

DRILLED SHAFT SOIL DESIGN PARAMETERS

JEA Northside Switchyard

4377 Heckscher Drive, Jacksonville, Florida

MAE Project No. 0057-0007

Soil Type	Typical Depth (ft)		Average N-Value	Effective Unit Weight[1] (pcf)	Friction Angle, ϕ (Degrees)	Deformation Modulus, E_D (ksi) [5]	Recommended Earth Pressure Coefficients		
	From	To					At Rest $(K_o)^2$	Active $(K_a)^3$	Passive $(K_p)^4$
SP	0	5	15	55	32	1.5	0.47	0.31	3.3
SP	5	12	18	60	33	1.8	0.46	0.29	3.4
SP-SM	12	18.5	6	50	30	0.59	0.50	0.33	3.0
SP-SM	18.5	28.5	12	50	31	1.2	0.48	0.32	3.1
SM	28.5	34	8	50	30	0.55	0.50	0.33	3.0
SM	34	40	24	65	34	1.8	0.44	0.28	3.5

Notes:

1. The groundwater level was assumed to be at the ground surface for design purposes

2. $K_o = 1 - \sin(\phi)$

3. $K_a = \tan^2(45 - \phi / 2)$

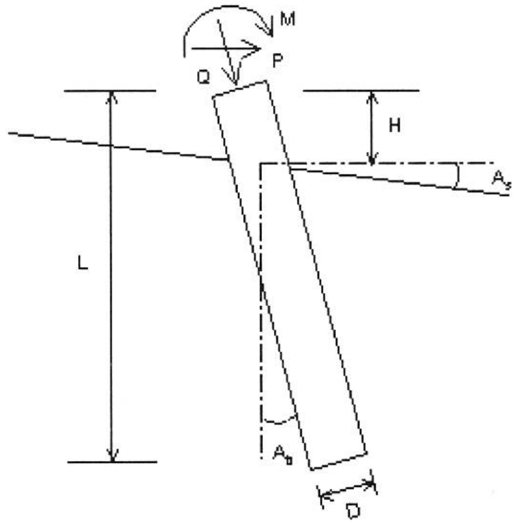
4. $K_p = \tan^2(45 + \phi / 2)$

5. Deformation Modulus estimated from Figure 1-8 in FAD 5.1 User's Guide, December 2015 edition

Meskel & Associates Engineering

VERTICAL ANALYSIS

Figure 1



Drilled Shaft (dia >24 in. or 61 cm)

Loads:

Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 160.0 -kp
Shear Load, P= 20.0 -kp
Moment, M= 400.0 -kp-f

Profile:

Pile Length, L= 20.0 -ft
Top Height, H= 0 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	55.0	32.0	0.00	51.9	46.29	15
5	60.2	33.0	0.00	62.2	51.54	18
12	50.1	30.0	0.00	18.5	24.26	6
18.5	50.1	31.0	0.00	38.2	38.42	12
28.5	50.1	30.0	0.00	26.0	30.24	8
34	65.1	34.0	0.00	77.8	58.64	24

Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	84	5541.8	263.9	2443920.3	3605	5.773
20.0						

Vertical capacity:

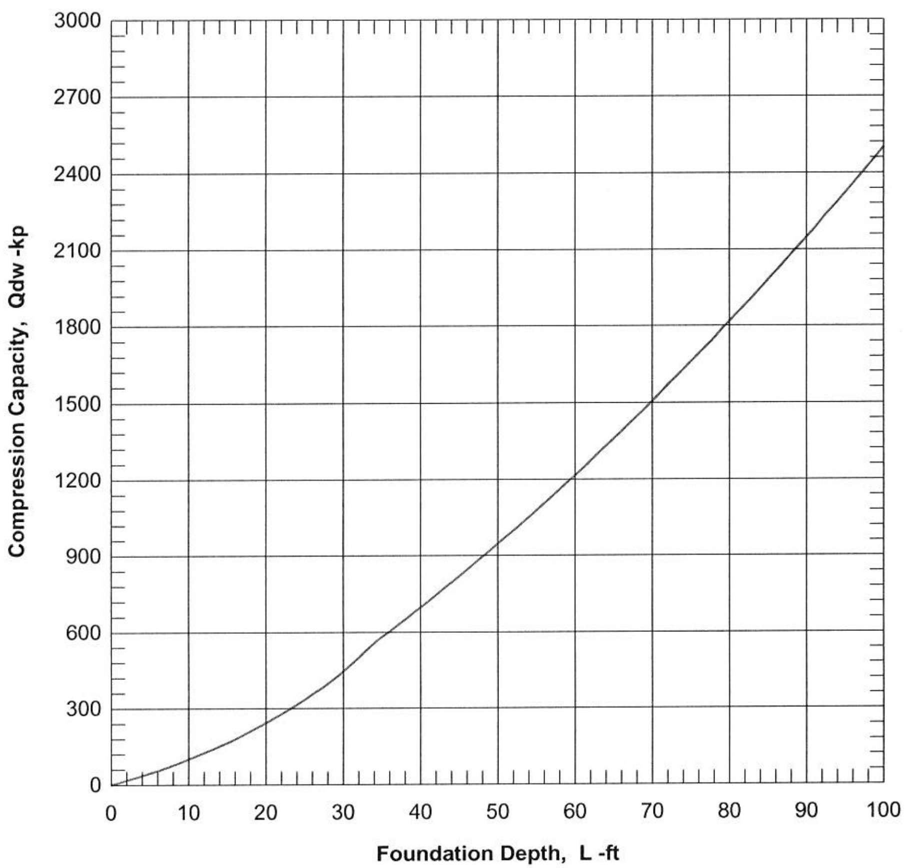
Weight above Ground= 0.00 Total Weight= 67.43-kp *Soil Weight is not included
Side Resistance (Down)= 80.652-kp Side Resistance (Up)= 46.087-kp
Tip Resistance (Down)= 633.094-kp Tip Resistance (Up)= 0.000-kp
Total Ultimate Capacity (Down) Qult= 713.745-kp Total Ultimate Capacity (Up)= 113.518-kp
Total Allowable Capacity (Down) Qallow= 243.292-kp Total Allowable Capacity (Up) Qallow= 85.866-kp
OK! Qallow > Q

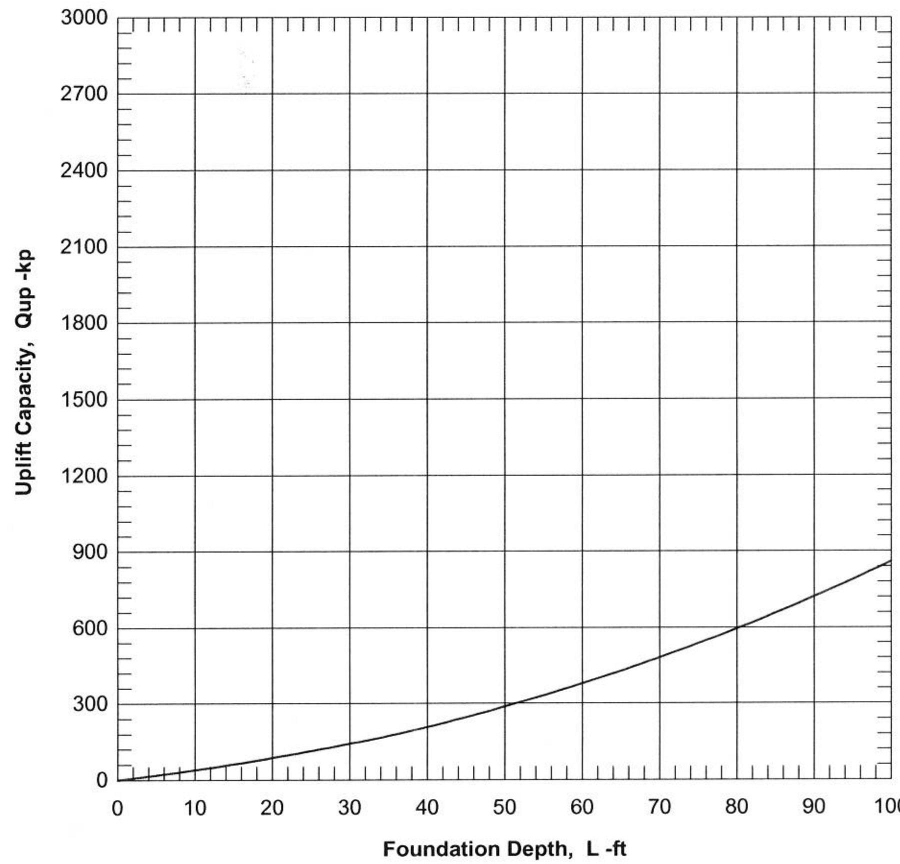
Settlement Calculation:

At Q= 160.00-kp Settlement= 0.58275-in
At Xallow= 1.00-in Qallow= 224.98659-kp

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.

ALLOWABLE CAPACITY vs FOUNDATION DEPTH





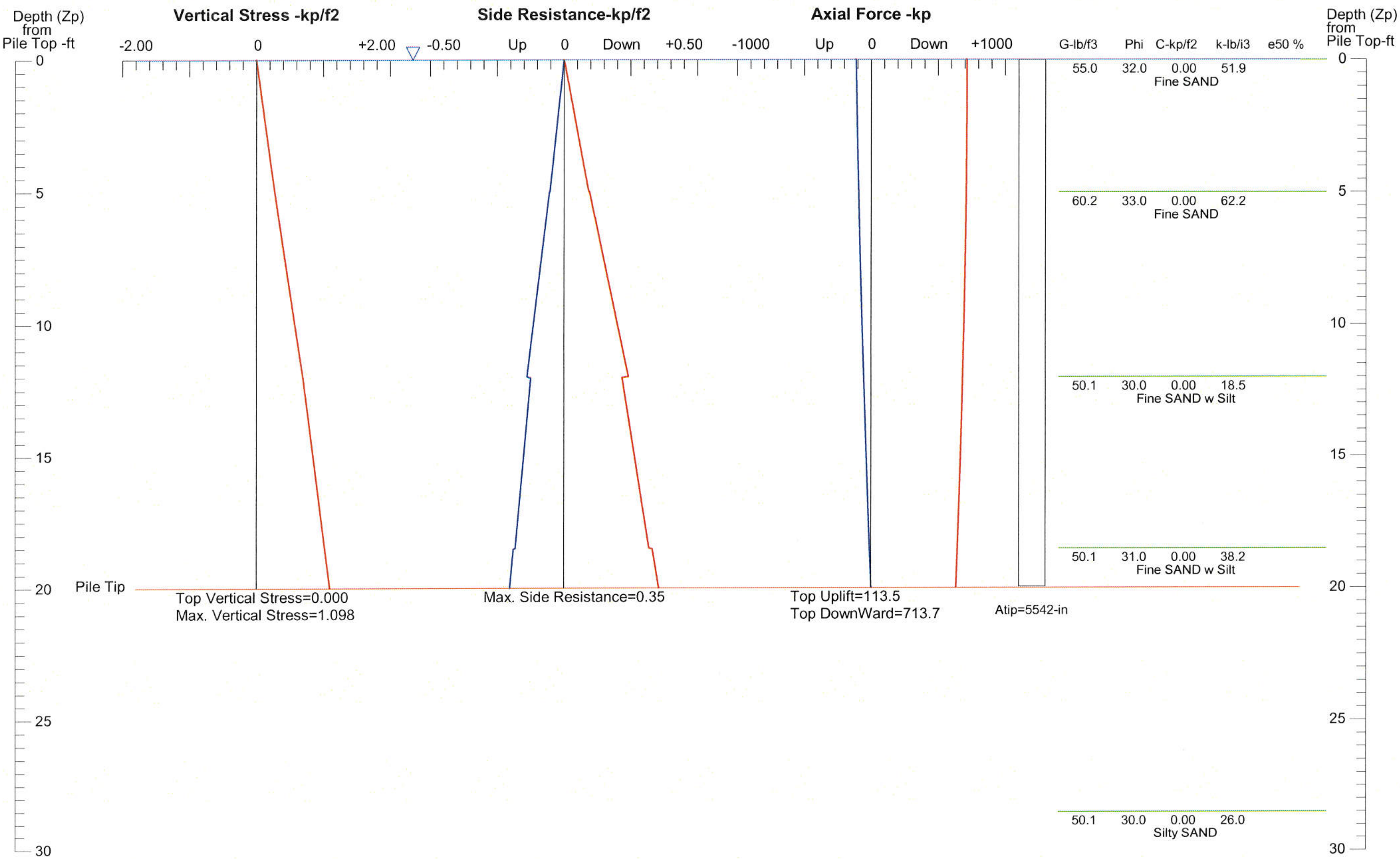
CivilTech Software

JEA Northside Switchyard - 84-in Shaft
MAE No. 0057-0007

Figure 1

SOIL STRESS, SIDE RESISTANCE, & AXIAL FORCE vs DEPTH

Based on Ultimate Load Condition



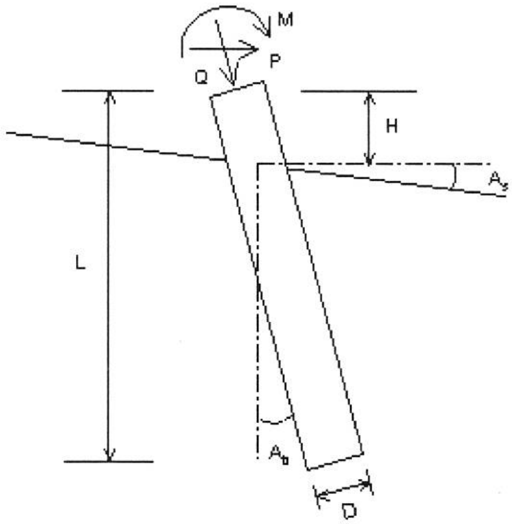
CivilTech Software

JEA Northside Switchyard - 84-in Shaft
MAE No. 0057-0007

Figure 1

LATERAL ANALYSIS

Figure 2



Drilled Shaft (dia >24 in. or 61 cm)

Loads:

Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 160.0 -kp
Shear Load, P= 20.0 -kp
Moment, M= 400.0 -kp-f

Profile:

Pile Length, L= 20.0 -ft
Top Height, H= 0 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	55.0	32.0	0.00	51.9	46.29	15
5	60.2	33.0	0.00	62.2	51.54	18
12	50.1	30.0	0.00	18.5	24.26	6
18.5	50.1	31.0	0.00	38.2	38.42	12
28.5	50.1	30.0	0.00	26.0	30.24	8
34	65.1	34.0	0.00	77.8	58.64	24

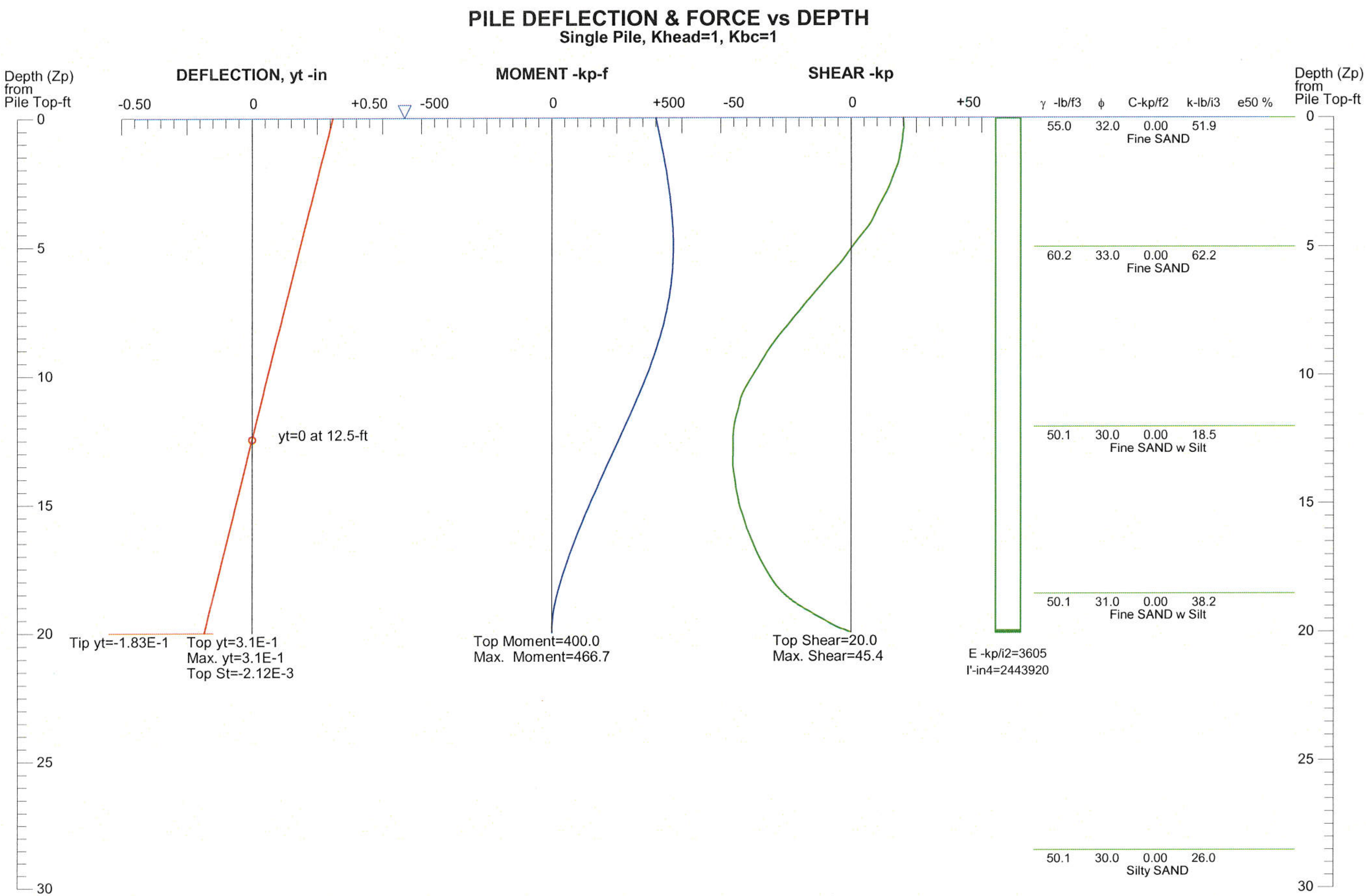
Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	84	5541.8	263.9	2443920.3	3605	5.773
20.0						

Single Pile Lateral Analysis:

Top Deflection, yt= 0.31000-in
Max. Moment, M= 466.67-kp-f
Top Deflection Slope, St= -0.00212
OK! Top Deflection, 0.3100-in is less than the Allowable Deflection= 1.00-in

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.
The Max. Moment calculated by program is an internal force from the applied load conditions. Structural engineer has to check whether the pile has enough capacity to resist the moment with adequate factor of safety. If not, the pile may fail under the load conditions.



CivilTech Software

JEA Northside Switchyard - 84-in Shaft
MAE No. 0057-0007

Figure 2