Report of Geotechnical Exploration
For

Key Haven Class II Pump Station Upgrade

MAE Project No. 0194-0002
July 25, 2018

Prepared for:

McKIM&CREED
ENGINEERS SURVEYORS PLANNERS

Prepared by:

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July 25, 2018

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Jacksonville, Florida 32256

Reference: Report of Geotechnical Exploration
Key Haven Class II Pump Station Upgrade
Project Location
MAE Project No. 0194-0002

Dear Mr. Anderson:

Meskel & Associates Engineering, PLLC (MAE) has completed a geotechnical exploration for the subject project. Our work was performed in general accordance with our revised proposal dated November 2, 2017. The geotechnical exploration was performed to evaluate the general subsurface conditions at the existing Key Haven Pump Station within the areas of the proposed construction, and to provide recommendations for foundation design and support for the proposed construction, and for site preparation. A summary of our findings and related recommendations are presented below; however, we recommend that you consider this report in its entirety.

In general, the borings encountered either a surficial topsoil layer or pavement structure (asphalt surface/limerock base course), underlain by fine sand with silt (SP-SM) and silty fine sand (SM) to the boring termination depths of 15, 20 and 30 feet below the existing grade. Debris was encountered at two boring locations (B-1 and B-3) between approximate depths of 4 and 6 feet. We recommend that test pits be excavated to verify the nature of the debris and confirm its lateral and vertical limits below slab-on-grade or shallow foundation-supported structures. The relative densities of the encountered soils ranged from very loose (weight-of-hammer) to medium dense. Groundwater was encountered at all three borings at depths ranging from 4 feet 1 inch to 4 feet 4 inches.

Based on our evaluation of the encountered subsurface conditions, it is our opinion that the proposed pump station equipment and transformer may be supported on concrete slab-on-grades and pad foundation systems, respectively, and the wet-well and manhole can be constructed as a cast-in-place concrete structures with concrete slab floors, provided the site preparations provided in this report are followed.

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, PLLC
MAE FL Certificate of Authorization No. 28142

Philip R Mank Digitally signed by Philip R Mank
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1.0 PROJECT INFORMATION

1.1 General

Project information was provided to us by Mr. Emmitt Anderson, with McKim & Creed via several electronic correspondences.

1.2 Project Description

The site for the subject project is located at the existing JEA facility on Key Haven Boulevard, south of its intersection with Key Coral Drive, in Jacksonville, Florida. The general site location is shown on Figure 1.

Based on the provided information and our discussions with Mr. Anderson, it is our understanding that the existing pump station will be demolished and a new pump station, along with new piping and a manhole, will be constructed. We understand that the proposed pump station surface equipment and transformer will be supported on concrete slab-on-grade and pad foundation systems (respectively). We also understand that a base concrete slab will be constructed at the bottom of the proposed wet well and manhole structures. We have assumed the planned wet well will be constructed of cast-in-place concrete. We understand that the depth of the planned wet well (from ground surface to the bottom base footing) is approximately 25 feet below the existing grade (El. -8.2 ft NAVD 88), and the bottom pad for the new manhole will be at about 22 ft below the existing grade (-6.00 NAVD 88). For our geotechnical analysis, we have assumed that the total loads will not exceed 2,000 pounds per square foot (psf) at grade, and the total applied load of the wet well structure will not exceed 1,500 psf.

Grading plans were not provided at the time of our evaluation; however, we have assumed maximum fill heights of no more than 1 to 2 feet above the current grades.

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 21, 2018. The boring locations were determined by us, using the provided plan that showed the proposed construction. GPS coordinates were then obtained by overlaying the provided plan in Google Earth. Our field personnel then located each boring location using a Garmin GPSMAP 78 hand-held GPS receiver. A utility locate request was submitted to the Sunshine State One-Call Center (SSOC) and were coordinated with JEA. Once the utilities were marked and located, our field crew mobilized to the site. Due to subsurface utility constraints, as detected by the ground penetrating radar (GPR) performed by others, boring B-1 had to be relocated from its originally proposed location to a position approximately 35 feet northwest along the planned pipeline. A copy of the plan provided to us was used to show the final approximate boring locations and is included as the *Boring Location Plan*, Figure 2. The boring locations shown should be considered accurate only to the degree implied by the method of layout used.

2.1 SPT Borings

To explore the subsurface conditions within the area of the proposed structures, we located and performed 3 Standard Penetration Test (SPT) borings, drilled to depths of approximately 15, 20 and 30 feet below the existing ground surface, in general accordance with the methodology outlined in ASTM D 1586. Split-spoon soil samples recovered during performance of the borings were visually described in the field and representative portions of the samples were transported to our laboratory for classification and testing.

2.2 Field Permeability Test

Two field permeability tests were performed; one adjacent to boring location B-2 and one adjacent to boring location B-3. The field permeability tests were performed by installing a solid-walled, open-bottom PVC casing snugly fit into a 4-inch diameter, 15 and 10-foot deep augered borehole (respectively). To estimate the vertical permeability rate of the soil, the pipe was left flush with the borehole bottom and to estimate the horizontal permeability rate of the soil, the bottom 1-foot of the pipe was then filled with silica sand or gravel, and the pipe was raised one foot above the bottom of the borehole. For both tests, the pipe was filled to the top with water, and since relatively permeable sandy soils were encountered in the borings, the tests were conducted as "falling head" tests in which the rate of water (head) drop within the pipe was measured over a period of up to 30 minutes. Each test was conducted three times and then averaged to estimate the in-situ permeability rate for the soil conditions at their respective locations.

3.0 LABORATORY TESTING

3.1 Visual Classification

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.

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 General Soil Profile

Graphical presentation of the generalized subsurface conditions is presented on Figure 3. 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.

4.1.1 Manhole Boring - B-1

The boring encountered a surficial topsoil layer approximately 3 inches thick, underlain by loose to medium dense fine sand with silt (SP-SM) to a depth of about 12 feet, followed by loose silty fine sand (SM) to the boring termination depth of 15 feet below the existing ground surface. It should be noted that debris (brick fragments) was encountered amongst the sand soils between approximate depths of 4 and 6 feet.

4.1.2 Wet-Well Boring - B-2

Boring B-2 encountered a pavement structure (1.5 inches of asphalt and 6.25 inches of limerock base course), underlain by medium dense to loose fine sand with silt (SP-SM) to a depth of about 6 feet. Underlying these sands were layers of medium dense, very loose, and then medium dense silty fine sand

(SM) to a depth of about 27 feet, followed by medium dense fine sand with silt (SP-SM) containing trace amounts of gravel (shell fragments) to the boring termination depth of 30 feet below the existing grade. It should be noted that starting at a depth of about 13.5 feet and continuing to about 19 feet, the silty sands were encountered with a very loose (i.e., N = weight-of-hammer and 1 blow-for-12 inches) relative density.

4.1.3 Transformer Pad Boring - B-3

Boring B-3 encountered a surficial topsoil layer approximately 4 inches thick, underlain by loose to medium dense fine sand with silt (SP-SM) to a depth of about 12 feet, followed by very loose silty fine sand (SM) to the boring termination depth of 20 feet below the existing grade. It should be noted that debris (brick fragments) was encountered amongst the sand soils between approximate depths of 4 and 6 feet.

4.2 Groundwater Level

The groundwater level was encountered at each of the boring locations and recorded at the time of drilling at depths varying from 4 feet 1 inch to 4 feet 3 inches 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.

4.3 Field Permeability Test Results

The field permeability tests resulted in the following vertical and horizontal permeabilities:

Test Location	Test Depth (ft)	Measured Permeability (cm/sec)
B-2 - Vertical	15	1.86×10^{-4}
B-2 - Horizontal	14 to 15	1.21×10^{-3}
B-3 - Vertical	10	2.20×10^{-4}
B-3 - Horizontal	9 to 10	2.69×10^{-4}

The measured permeability rates should not be construed to represent the actual permeability rates. For design calculations, we recommend a minimum factor of safety of at least 2 be applied to the above permeability rate values.

4.4 Review of the USDA Web Soil Survey Map

The results of a review of the USDA Soil Survey Conservation Service (SSCS) Web Soil Survey of Duval County are shown in the table below. There is one predominant soil map units at the project sight: Urban land-Hurricane-Albany complex. The soil drainage class, hydrological group, and estimated seasonal high groundwater levels reported in the Soil Survey are as follows:

Map Unit Symbol	**Map Unit Name**	**Drainage Class**	**Hydrologic Group**	**Depth to the Water Table[1] (inches)**
75	Urban land-Hurricane-Albany complex, 0 to 5 percent slopes	Somewhat Poorly Drained	A, A/D	12 to 42

[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 term "complex", as defined by the USDA, refers to a map unit consisting of two or more soils or miscellaneous areas in such an intricate pattern or in such small areas that they cannot be shown separately on the map.

4.5 Seasonal High Groundwater Level

In estimating seasonal high groundwater level, a number of 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 1 to 2 feet above the measured groundwater levels at the time of our field exploration. However, it should be understood that this seasonal high estimate is based on site observations and measurements at the time of our field work and on historical data on the site soil conditions. Changes in onsite stormwater drainage patterns caused by off-site development may cause seasonal high water levels to be higher or lower than historical patterns. The project drainage engineer should be consulted to evaluate the influence of these changes on groundwater levels at the site. In addition, we recommend that piezometers be installed across the site to measure groundwater fluctuations over time.

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.

5.0 DESIGN RECOMMENDATIONS

5.1 General

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

Two borings (B-1 and B-3) encountered soils containing debris (brick fragments) between depths of about 4 and 6 feet bellow the existing grade. We do not recommend supporting structures on soils that contain debris as voids may be present that could lead to intolerable settlement of the overlying structures. We recommend that test pits be excavated within the area of the proposed structure to better explore the nature of the debris and delineate its lateral and vertical extents. A MAE geotechnical engineer or his representative should observe the test pits so that we can provide remedial recommendations, if necessary, based on the encountered conditions.

5.2 Pump Station Foundations Recommendations

Based on the results of our exploration, we consider the subsurface conditions at the site adaptable for support of the proposed pump station equipment on a slab-on-grade foundation, provided that the existing debris is removed, depending on the results of the test pit program, and that surficial topsoil is removed from within the construction area and that these materials are replaced with suitable structural fill material as outlined in Section 6.0.

5.2.1 Bearing Pressure

The maximum allowable net soil bearing pressure for use in slab-on-grade design should not exceed 2,000 psf. The maximum allowable net soil bearing pressure for the wet well base slab should not exceed 1,500 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 slab-on-grade and wet well base slab foundations should be designed based on the maximum load that could be imposed by all loading conditions.

5.2.2 Bearing Depth

The slab-on-grade supporting surface equipment should bear at a depth of at least 12 inches below the exterior final grades. It is recommended that stormwater be diverted away from these slabs to reduce the possibility of erosion beneath the slabs.

5.2.3 Bearing Material

The subgrade soils below the slab-on-grade and the wet well slab should consist of suitable on-site or import structural fill soils. The fine sands (SP) and fine sands with silt (SP-SM) as encountered in the borings are considered suitable onsite soils. These soils should be compacted to at least 95 percent of the soil's modified Proctor Maximum Dry Density (ASTM D-1557) to a depth of at least one foot below the slab bearing levels. Control of the soil's moisture content, particularly for the subgrade soils below the wet well slab, will be necessary to achieve the required level of compaction.

5.3 Transformer Shallow Foundation Design Recommendations

Based on the results of our exploration, we consider the subsurface conditions at the site adaptable for support of the proposed transformer structure when constructed on a properly designed shallow foundation systems. Provided the site preparation and earthwork construction recommendations outlined in Section 6.0 of this report are performed, the following parameters may be used for foundation design.

5.3.1 Bearing Pressure

The maximum allowable net soil bearing pressure for use in shallow foundation design should not exceed 2,000 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.

5.3.2 Foundation Size

We understand that the transformer will be supported on a concrete pad. We recommend the pad have a minimum width of 12 inches. Even though the maximum allowable soil bearing pressure may not be achieved, this width recommendation should control the size of the foundation.

5.3.3 Bearing Depth

Concrete pads should bear at a depth of at least 18 inches below the exterior final grades. It is recommended that stormwater be diverted away from the structures to reduce the potential of erosion of bearing level soils.

5.3.4 Bearing Material

The foundation may bear in either the compacted suitable natural soils or compacted structural fill. The bearing level soils, after compaction, should exhibit densities equivalent to 95 percent of the modified Proctor maximum dry density (ASTM D 1557), to a depth of at least one foot below the foundation bearing levels.

5.3.5 Settlement Estimates

Post-construction settlements of the structure will be influenced by several interrelated factors, such as (1) subsurface stratification and strength/compressibility characteristics; (2) footing size, 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 the shallow foundation supported transformer and circuit breaker structures 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 settlements of the structure.

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/assumed maximum 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 settlements of the structure could 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 the structure should be within tolerable magnitudes.

5.4 Below Grade Structures Design Recommendations

Based on the results of the subsurface explorations, laboratory testing, and provided information, as included in this report, we consider the subsurface conditions at the site adaptable for supporting the proposed pump station wet well and manhole structures when constructed upon properly prepared subgrade soils. Provided the site preparation and earthwork construction recommendations outlined in Section 6.0 of this report are performed, the following parameters may be used for design of below-grade utilities.

5.4.1 Lateral Pressure Design Parameters

In general, walls that have adjacent compacted fill will be subjected to lateral earth pressures. Walls that are restrained at the top and bottom will be subjected to at-rest soil pressures, while walls that are not restrained at the top, and where sufficient movement is anticipated, will be subjected to active earth pressures. Surcharge effects for sloped backfill, point or area loads behind the walls, and adequate drainage provisions should be incorporated in the wall design. Passive resistance, resulting from footing embedment at the wall toe, could be neglected for safer design. The following soil parameters can be used for the project where suitable fill soils, as described in Section 6.5, are placed adjacent to the overflow structure:

- Backfill Soil Unit Weight, Saturated (γ_{sat}) = 115 pcf
- Backfill Soil Unit Weight, Moist (γ_m) = 110 pcf
- Backfill Soil Angle of Internal Friction (ϕ) = 30 degrees
- Coefficient of Active Earth Pressure, k_a = 0.33
- Coefficient of At-Rest Earth Pressure, k_o = 0.5
- Coefficient of Passive Earth Pressure, k_p = 3.0
- Foundation Soil Unit Weight, Saturated (γ_{sat}) = 120 pcf
- Foundation Soil Angle of Internal Friction (ϕ) = 30 degrees

The above parameters are based on sand backfill (SP, SP-SM) placed and compacted behind the vault walls as discussed in Section 6.5, and on compaction of the wall foundation soils as discussed in Section 6.4. A coefficient of friction for poured in-place concrete of 0.45 may be used in the wall design. The wet well structure should be designed to include all temporary construction and permanent traffic and surcharge loads acting on the walls.

5.4.2 Hydrostatic Uplift Resistance

It is anticipated that the buried structure will exert little or no net downward pressure on the soils, rather, the structure may be subject to hydrostatic uplift pressure when empty. Below grade structures should be designed to resist hydrostatic uplift pressures appropriate for their depth below existing grade and the seasonal high groundwater table. Hydrostatic uplift forces can be resisted in several ways including:

- Addition of dead weight to the structure.
- Mobilizing the dead weight of the soil surrounding the structure through extension of the bottom slab outside the perimeter of the structure.

A moist compacted soil unit weight of 110 lb/ft3 may be used in designing the wet well structure to resist buoyancy.

5.5 Reuse of Onsite Soils

Based on the boring results and classification of the soil samples, the fine sands, fine sands with silt, and silty fine sands (SP, SP-SM, SM) as encountered at the boring locations, are considered suitable for use as fill soil. However, it should be noted that the SM soils (i.e., soils with more than 10 to 12 percent passing the No. 200 sieve) will be more difficult to compact due to their natural tendency to retain soil moisture and will require drying. It should be anticipated that if the SM soils are not properly dewatered prior to excavation, drying of these soils to obtain the proper moisture content for compaction may take approximately 2 to 3 weeks, if weather permits. Depending on the anticipated time for completing the site work portion of the project and the drying time required to preclude pumping and yielding of these soils during placement and compaction operations, these soils may be considered unsuitable for use as fill material. The soils containing surficial organic material (e.g., topsoil) will require removal and are considered unsuitable for use as structural fill. The organic soils could be used in landscape berms. In addition, soil containing debris is not considered suitable for structural fill. Any debris laden soils should be stockpiled a safe distance from the construction area, as to not be confused with any soils intended for reuse, and removed from the site.

Due to the typically high groundwater levels at this site, it should be anticipated the soils will have moisture contents in excess of the modified Proctor optimum moisture content and will require stockpiling or spreading to bring the moisture content within 2 percent of the soil's optimum moisture content corresponding to the required degree of compaction.

6.0 SITE PREPARATION AND EARTHWORK RECOMMENDATIONS

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

6.1 Clearing and Stripping

Prior to construction, and subsequent to the clearing and removal of all debris associated with the demolition of the existing pump station, 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 concrete pad and slab-on-grade foundations plus a minimum additional margin of 5 feet, should be stripped of all surface vegetation, stumps, debris, organic topsoil, or other deleterious materials. 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.

Based on the results of our field exploration, it should be anticipated that 6 to 12 inches of topsoil and soils containing significant amounts of organic materials may be encountered across the site. The actual depths of unsuitable soils and materials should be determined by Meskel & Associates Engineering using visual observation and judgment during earthwork operations. Any topsoils removed from the construction areas can be stockpiled and used in areas to be grassed.

6.2 Supplemental Test Pit Exploration

We recommend that a supplemental test pit exploration be performed. The intent of this recommendation is to better define the nature of the debris as encountered at boring locations B-1 and B-3, and to better estimate the vertical and lateral extents of the debris. A geotechnical engineer or his representative from MAE should be present to document the encountered conditions and provide recommendations for removal, if required.

6.3 Removal /Replacement and Dewatering Program

The heaviest concentration of debris was encountered in boring B-1 between depths of about 4 to 6 feet below existing grade; however, we note that it is possible that debris laden soils requiring removal may exist at deeper depths at locations away from the borings. Subsequent to the supplemental test pit exploration, materials identified for removal should be excavated from within and to a distance of at least 5 feet beyond the planned foundation peripheries. We note that dewatering will be required to facilitate the removal and replacement process. Our personnel should be present to confirm that all debris materials are removed, and to perform in-place density testing of newly placed backfill to confirm that the recommended degree of compaction is achieved prior to placements of additional lifts. The excavation should be adequately sloped or braced to comply with applicable regulations for safe worker entry. Stockpiles of soil should be placed a sufficient distance from the excavation edges to preclude surcharging the excavation sides, potentially causing slope failures.

Outside areas with debris, temporary groundwater control measures may be required to facilitate the densification of soils within the upper 2 feet below the stripped surface. Should groundwater control measures become necessary, dewatering methods should be determined by the contractor. We recommend the groundwater control measures, if necessary, remain in place until compaction of the existing soils is completed. The dewatering method should be maintained until backfilling 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 required to evaluate compliance with the regulations.

6.4 Surface Compaction

The exposed surface areas outside of the excavation should be compacted with a vibratory drum roller having a minimum static, at-drum weight, on the order of 3 tons. 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 95 percent of the modified Proctor maximum dry density (ASTM D 1557) have been achieved within the upper 2 feet of the compacted natural soils at the sites. Prior to compaction, proof-rolling of these areas with a loaded dump truck is recommended to locate any unforeseen soft areas or unsuitable surface or near-surface soils.

Should the surface soils experience pumping and soil strength loss during the compaction operations, compaction work should be immediately terminated. The disturbed soils should be removed and backfilled with dry structural fill soils, which are then compacted, or 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). Compaction should cease if deemed detrimental to adjacent structures, and Meskel & Associates Engineering should be contacted immediately. It is recommended that the vibratory roller remain a minimum of 50 feet from existing structures. Within this zone, use of a track-mounted bulldozer or a vibratory roller, operating in the static mode, is recommended.

6.5 Compaction of Excavation Bottom and Backfilling

Once the clearing and stripping has been completed, and subsequent to the clearing and removal of all debris associated with the demolition of the existing pump station, excavation for the wet well and manhole structures, and associated pipelines, may commence. The excavations should extend at least 3 feet in all directions outside the lateral dimensions of the structure. Once the wet well, manhole and pipeline excavations have achieved their target depths, backfill placement can commence. The temporary dewatering method should remain in-place to facilitate compaction of the bottom soils for the wet well and manhole slabs, and to facilitate the backfilling operation. The bottom soils for the wet well slabs should be compacted to 95 percent of their modified Proctor maximum dry density for a depth of 12 inches below subgrade elevation. If very loose silty sands, as encountered at boring location B-2 between depths of about 15 and 20 feet, are encountered at the wet well or manhole slab subgrade elevations, then we recommend the excavation continue at least an additional 12 inches and be backfilled with a graded aggregate such as ASTM C33 Gradation 67 stone as specified in the JEA Water/Wastewater Standards. The excavation bottom soils should be overlain with a filter fabric to act as a separation layer between the very loose silty soils and the stone backfill. The fabric should continue up the sides of the excavation to separate the soil backfill from the adjacent silty soils. The stone should be placed in 2 lifts of equal thickness but with no lift greater than 12 inches thick, with each lift compacted to form a stable working surface.

Backfill soil placed against the sides of the structure above the subgrade stone should consist of sand soils as defined in Section 6.6 below. The backfill should be placed in maximum 6-inch lifts, with each lift compacted with hand-held equipment as defined in Section 6.6. Backfill placed more than 5 feet away from the structure walls may be placed in lifts up to 12 inches in thickness, with each lift compacted with appropriate compaction equipment to achieve the same level of compaction. Dewatering should remain in place until the level of backfill is at least 2 feet above the groundwater table at the time of construction.

6.6 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. The lift thickness should be reduced to 8 inches if the roller operates in the static mode or if track-mounted compaction equipment is used. If hand-held compaction equipment is used, the lift thickness should be further reduced to 6 inches.

Structural fill is defined as a non-plastic, inorganic, granular soil having less than 10 percent material passing the No. 200 mesh sieve and containing less than 4 percent organic material. The fine sand and slightly silty or clayey fine sand, without roots, as encountered in the borings, are suitable as fill materials and, with proper moisture control, should densify using conventional compaction methods. 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 95 percent of the modified Proctor maximum dry density (ASTM D 1557) have been achieved within each lift of the compacted structural fill.

We recommend that material excavated from the wet well and manhole pits and pipeline trenches, which will be reused as backfill, be stockpiled a safe distance from the excavations and in such a manner that promotes runoff away from the open trenches and limits saturation of the materials.

6.7 Foundation Areas

The foundation bearing level soils, after compaction, should exhibit densities equivalent to 95 percent of the modified Proctor maximum dry density (ASTM D 1557), to a depth of one foot below the bearing level. For confined areas, such as the footing excavations, 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 2000 pounds.

6.8 Excavation Protection

Excavation work for the pump station construction will be required to meet OSHA Excavation Standard Subpart P regulations for Type C Soils. The use of excavation support systems will be necessary where there is not sufficient space to allow the side slopes of the excavation to be laidback to at least 2H:1V (2 horizontal to 1 vertical) to provide a safe and stable working area and to facilitate adequate compaction along the sides of the excavation.

The method of excavation support should be determined by the contractor but can consist of a trench box, drilled-in soldier piles with lagging, interlocking steel sheeting or other methods. The support structure should be designed according to OSHA sheeting and bracing requirements by a Florida registered Professional Engineer.

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 bearing levels in the footing excavations. The density tests are considered necessary to verify that satisfactory compaction operations have been performed. We recommend density testing be performed as listed below:

- one location for every 5,000 square feet of pad and slab foundation areas
- one test per lift of backfill placed against the wet well walls
- one test per lift of backfill placed against the manhole walls
- one test per 100 feet of pipe length per lift of backfill

8.0 REPORT LIMITATIONS

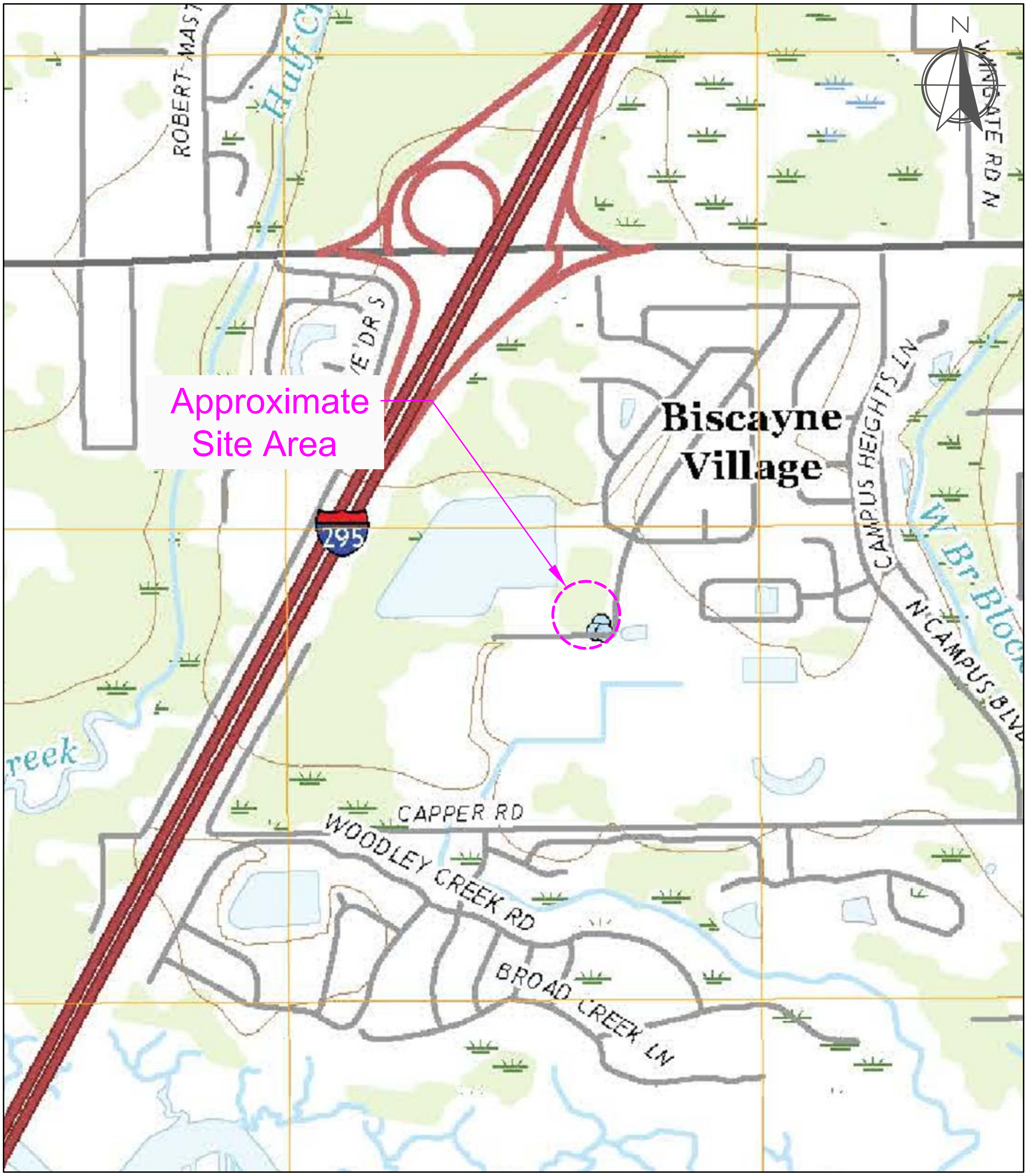
This report has been prepared for the exclusive use of McKim & Creed and the JEA for specific application to the design and construction of the Key Haven Class II Pump Station Upgrade project. An electronically signed and sealed version, and a version of our report that is signed and sealed in blue ink, 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 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 away from the boring locations and/or at depths below the boring termination depths. Subsurface conditions and water 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.

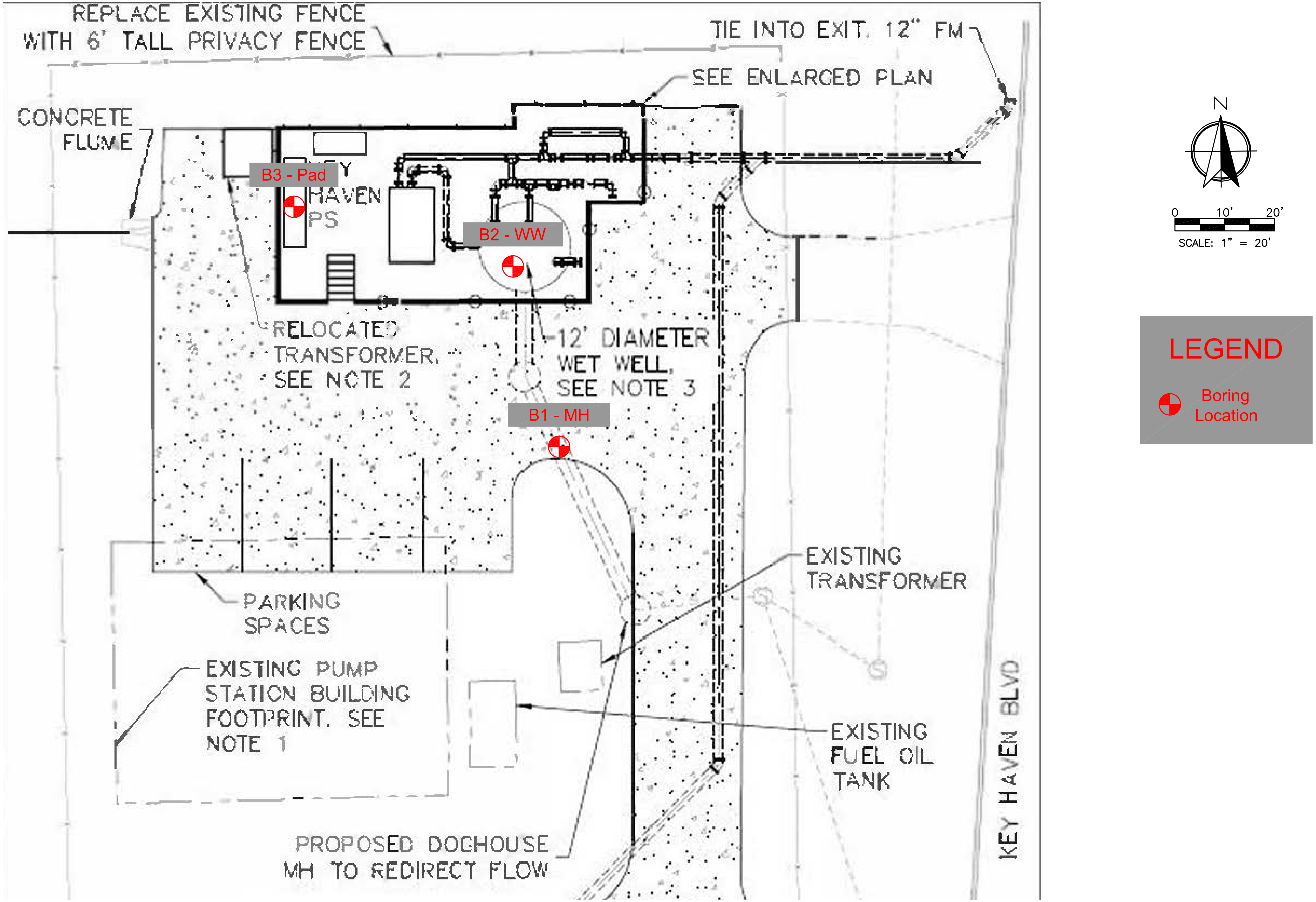
If changes in the design or location of the structures occur, the conclusions and recommendations contained in this report may need to be modified. We recommend that these changes be provided to us for our consideration. MAE is not responsible for conclusions, interpretations, opinions or recommendations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

Figures



Site Location Map

PREPARED BY	PROJECT NAME	
Meskel & Associates Engineering, LLC FL Certificate of Authorization No. 28142 8936 Western Way, Suite 12, Jacksonville, FL 32256	*Key Haven Class II Pump Station Upgrade* *Jacksonville, Florida*	
	REFERENCE	SCALE
	USGS Trout River, FL Quadrangle	NTS
PREPARED FOR	MAE PROJECT NO.	FIGURE NO.
McKim & Creed, Inc.	0194-0002	1



NOTE: Class Two Pump Station Layout Plan, dated June 8, 2018, was provided by Mckim & Creed, Inc.

REVISIONS

DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

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

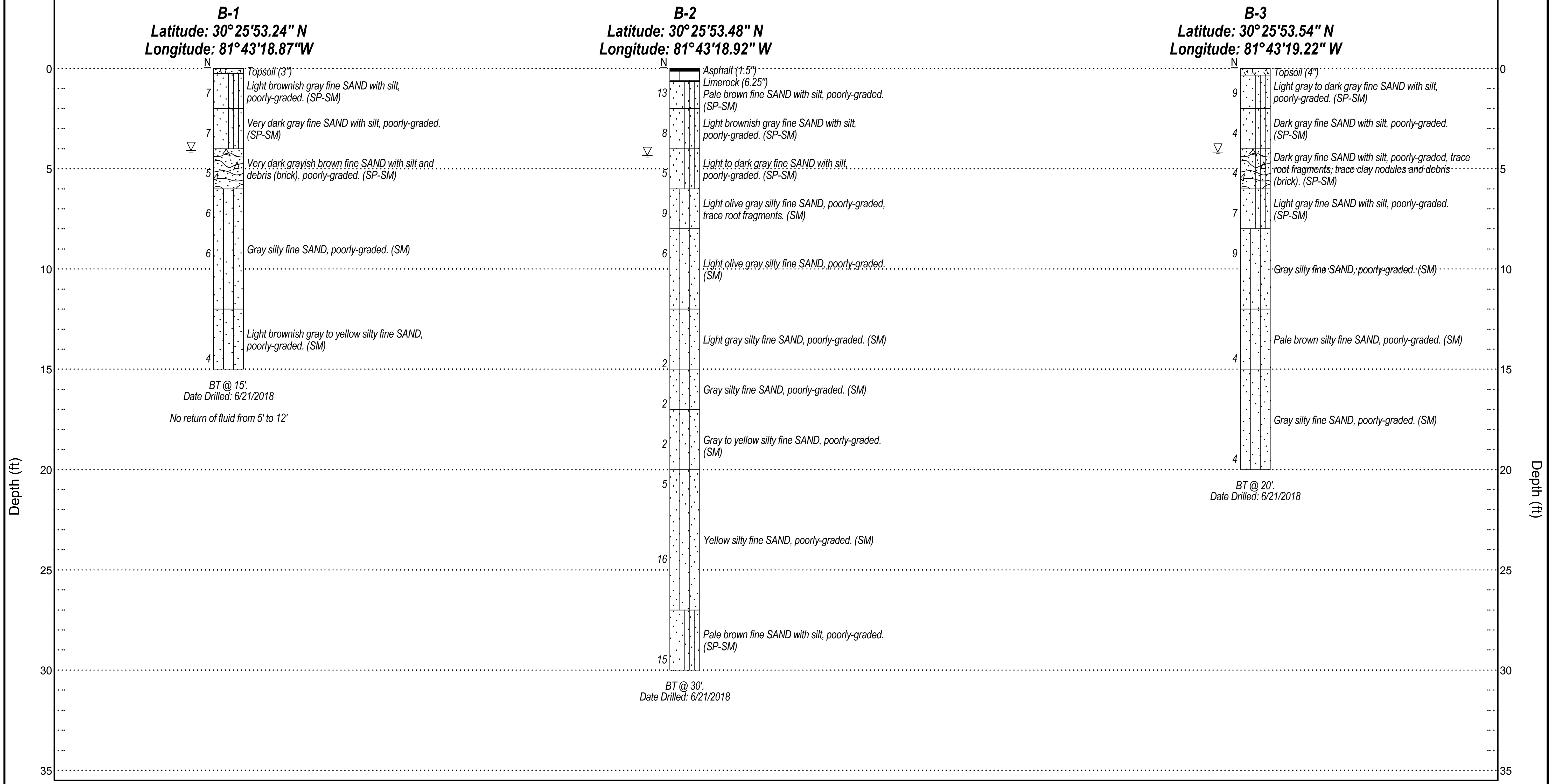
CLIENT: McKim & Creed, Inc.

DATE:	MAE PROJECT NO.
07/19/2018	0194-0002

SHEET TITLE: BORING LOCATION PLAN

PROJECT NAME: *Key Haven Class II Pump Station Upgrade*
Jacksonville, Florida

FIGURE NO. 2



Legend

▨	Topsoil	▥	Fine Sand with Silt	▨	Fine SAND with Debris
▥	Silty Fine Sand	■	Asphalt	▦	Limestone

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

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

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

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FL Certificate of Authorization No. 28142
8936 Western Way, Suite 12, Jacksonville, FL 32256

McKim & Creed

DATE:	MAE PROJECT NO.
7/25/2018	0194-0002

SHEET TITLE:
Generalized Soil Profile

PROJECT NAME:
JEA Key Haven Pump Station
Jacksonville, Florida

FIGURE NO.
3

Appendix A

Meskel & Associates Engineering, PLLC
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BORING B-1

PAGE 1 OF 1

PROJECT NO. 0194-0002

PROJECT NAME JEA Key Haven Pump Station

PROJECT LOCATION Jacksonville, Florida **CLIENT** McKim & Creed

DATE STARTED 6/21/18 **COMPLETED** 6/21/18 **LATITUDE** 30°25'53.24" N **LONGITUDE** 81°43'18.87"W

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

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		Topsoil (3")												
	1	Light brownish gray fine SAND with silt, poorly-graded.	SP-SM		3 3 4 4	7								
	2	Very dark gray fine SAND with silt, poorly-graded.	SP-SM		4 4 3 3	7								
		∇												
5	3	Very dark grayish brown fine SAND with silt and debris (brick), poorly-graded.	SP-SM		3 3 2 2	5								
	4	Gray silty fine SAND, poorly-graded.	SM		3 3 3 3	6								
10	5				2 3 3 3	6								
		Light brownish gray to yellow silty fine SAND, poorly-graded.	SM											
15	6				1 2 2	4								

Bottom of borehole at 15 feet.

NOTES

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 4.08 ft *∇ **END OF DAY** ---

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Meskel & Associates Engineering

BORING B-2

PAGE 1 OF 2

PROJECT NO. 0194-0002

PROJECT NAME JEA Key Haven Pump Station

PROJECT LOCATION Jacksonville, Florida

CLIENT McKim & Creed

DATE STARTED 6/21/18 **COMPLETED** 6/21/18

LATITUDE 30°25'53.48" N **LONGITUDE** 81°43'18.92" W

DRILLING CONTRACTOR MAE, PLLC

DRILLING METHOD Standard Penetration Test

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		Asphalt (1.5")												
		Limerock (6.25")												
	1	Pale brown fine SAND with silt, poorly-graded.	SP-SM		7 8 5	13								
	2	Light brownish gray fine SAND with silt, poorly-graded.	SP-SM		5 4 4 3	8								
5	3	∇ Light to dark gray fine SAND with silt, poorly-graded.	SP-SM		2 2 3 2	5								
	4	Light olive gray silty fine SAND, poorly-graded, trace root fragments.	SM		2 4 5 5	9								
10	5	Light olive gray silty fine SAND, poorly-graded.	SM		2 3 3 5	6								
		Light gray silty fine SAND, poorly-graded.	SM											
15	6				1 1 1 1	2								
	7	Gray silty fine SAND, poorly-graded.	SM		1 1 1 1	2								
	8	Gray to yellow silty fine SAND, poorly-graded.	SM		1 1 1 1	2								
20	9				1 2 3 3	5								
		Yellow silty fine SAND, poorly-graded.	SM											
25	10				3 8 8	16								

NOTES

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 4.33 ft *∇ **END OF DAY** ---

(Continued Next Page)

BORING B-2

PAGE 2 OF 2

PROJECT NO. 0194-0002

PROJECT NAME JEA Key Haven Pump Station

PROJECT LOCATION Jacksonville, Florida **CLIENT** McKim & Creed

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
25		Yellow silty fine SAND, poorly-graded. *(continued)*	SM											
	11	Pale brown fine SAND with silt, poorly-graded.	SP-SM		5 7 8	15								
30		Bottom of borehole at 30 feet.												

NOTES

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 4.33 ft *∇ **END OF DAY** ---

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BORING B-3

PAGE 1 OF 1

PROJECT NO. 0194-0002

PROJECT NAME JEA Key Haven Pump Station

PROJECT LOCATION Jacksonville, Florida

CLIENT McKim & Creed

DATE STARTED 6/21/18 **COMPLETED** 6/21/18

LATITUDE 30°25'53.54" N **LONGITUDE** 81°43'19.22" W

DRILLING CONTRACTOR MAE, PLLC

DRILLING METHOD Standard Penetration Test

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		Topsoil (4")												
	1	Light gray to dark gray fine SAND with silt, poorly-graded.	SP-SM		3 4 5 5	9								
	2	Dark gray fine SAND with silt, poorly-graded.	SP-SM		2 2 2 2	4								
5	3	∇ Dark gray fine SAND with silt, poorly-graded, trace root fragments, trace clay nodules and debris (brick).	SP-SM		2 2 2 1	4								
	4	Light gray fine SAND with silt, poorly-graded.	SP-SM		1 3 4 6	7								
10	5	Gray silty fine SAND, poorly-graded.	SM		2 4 5 4	9								
15	6	Pale brown silty fine SAND, poorly-graded.	SM		1 2 2	4								
20	7	Gray silty fine SAND, poorly-graded.	SM		1 2 2	4								
		Bottom of borehole at 20 feet.												

NOTES

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 4.17 ft

*∇ **END OF DAY** ---

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, “Standard Test Method for Standard Penetration Test (SPT) 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 a geotechnical engineer to verify the field descriptions and classify the soil, and to select samples for laboratory testing.

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	Less than 5%
Organic Soils	5% to 20%
Highly Organic Soils (Muck)	20% to 75%
PEAT	Greater than 75%

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

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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%

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