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This document is not intended to be utilized as a Geotechnical Baseline Report.

Variations

The subsurface information submitted in this document is based upon data obtained from investigative methods as completed at the approximate locations as indicated in the respective attachments. This document does not reflect variations which may occur between investigation locations. The nature and extent of variations between the investigation locations may not become evident until construction is performed.

Water levels, as described in the respective appendices, reflect only those conditions that existed at the time that those particular subsurface investigations were performed. Fluctuations or changes in water levels and groundwater conditions can be influenced by sources outside the site investigated, by seasonal rainfall, and by changes in drainage conditions in and around the Site. Fluctuations can occur between the time of investigation and the time of construction.

APPENDIX A

Report Title:	***Report of Geotechnical Exploration for JEA Church Street – 69kV Cable Relocation***
Report Date:	**December 19, 2025**
Report Author:	**Meskel & Associates Engineering, PLLC**
Report Recipient:	**Burns & McDonnell Engineering Co.**
Additional Recipient(s):	
Project Number:	**190016**

Report of Geotechnical Exploration
For

JEA Church Street – 69kV Cable Relocation

MAE Project No. 0508-0003
December 19, 2025

Prepared for:

9400 Ward Parkway
Kansas City, MO 64114

Prepared by:

3728 Philips Highway, Suite 208
Jacksonville, Florida 32207
Phone (904) 519-6990
Fax (904) 519-6992

Meskel & Associates Engineering
Geotechnical Engineering · Materials Testing · CEI Services
Environmental Consulting · Private Provider Services

December 19, 2025

Burns & McDonnell
9400 Ward Parkway
Kansas City, MO 64114

Attention: Mr. Tyler R. McArthur, P.E., ENV SP

Reference: Report of Geotechnical Exploration
JEA Church Street – 69kV Cable Relocation
Jacksonville, Florida
MAE Project No. 0508-0003

Dear Mr. McArthur:

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 October 7, 2025. The geotechnical exploration was performed to evaluate the general subsurface conditions along the proposed cable relocation, to provide recommendations for design.

As further discussed in this report, the borings encountered an asphalt concrete layer to varying depths of approximately 3 to 7 inches below ground surface, boring B-1 is underlain by a limerock base approximately 6 inches. Below the asphalt concrete and limerock base, the boring encountered dense to very loose fine sand (SP), fine sand with silt (SP-SM), fine sand with clay (SP-SC), silty fine sand (SM), and clayey fine sand (SC), underlain by soft to hard weathered limestone to the boring termination depth of 60 feet below the existing ground surface. Groundwater was encountered at all the boring locations at depths varying from 4 feet to 7 feet 7 below the existing site grades, with normal seasonal high levels estimated at 2 to 3 feet higher.

Based on our findings, it is our opinion that the encountered soils are adaptable for supporting the proposed cable relocation using open-cut and HDD construction techniques given the recommendations provided in this report are followed. In addition, the predominant near-surface fine sand (SP, SP-SM, SP-SC/A-3) soils may be used as bedding and structure backfill. The site preparation recommendations provided in this report should be followed to provide uniform subgrade and backfill conditions during cable installation.

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 Registry No. 28142

Brett H. Harbison, State of Florida, Professional Engineer, License No. 74679. This item has been electronically signed and sealed by Brett H. Harbison, P.E. on 12/19/2025 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.

Daisy Peña-Ross

Daisy Peña-Ross
Staff Engineer

Brett Harbison, P.E.
Principal Engineer
Licensed, Florida No. 74679

Distribution: Mr. Tyler R. McArthur, P.E. – Burns & McDonnell 1 pdf

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1.0 PROJECT INFORMATION

1.1 General

Project information was provided to us by Mr. Tyler R. McArthur, P.E., ENV SP, with Burns & McDonnell. Project details were provided to us via a proposal request email on July 24, 2025. We were provided with the following project documents for review and reference:

- Exhibit titled "Geotechnical Engineering Services", prepared by Burns & McDonnell on October 7, 2025.

1.2 Project Description

The site for the subject project is located along the north side of Church Street West from the intersection at Eaverson Street to east of Johnson Street in Jacksonville, Duval County, Florida. The general site location is shown in Figure 1.

Based on the information provided and our discussions with Burns & McDonnell, it is our understanding that the subject project includes the relocation of an existing underground transmission cable along Church Street to accommodate the Interstate-95 improvements at its Church Street crossing. The cable relocation will utilize open-cut excavation. The trench will be excavated to a depth of 10 to 12 feet below existing grade. Manhole, also excavated to a depth of 10 to 12 feet, will likely be installed along the cable route. No change to the existing ground surface is anticipated. Following construction, any disturbed pavement will be replaced using matching materials. The duct bank housing the transmission cable is expected to be 2 to 4 feet wide.

Trenchless excavation methods may be used in lieu of traditional open cut excavation to avoid conflicts with existing utilities. Horizontal Directional Drilling (HDD) will likely be the technology used. The HDD bore will extend no more than 50 feet below the ground surface.

A riser transmission pole is situated near boring location B-3. Burns & McDonnell will assess the existing pole's structural integrity to determine its suitability for supporting the new transmission line. While the precise loading conditions of the existing pole are currently unknown, similar structures typically experience the following approximate loading conditions: axial load of 20 kips, shear load of 35 kips, and bending moment of 2080 kip-ft.

If actual project details vary from those described above, 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 during October 2 and October 3, 2025. An additional field exploration was performed on November 4, 2025, to collect additional bulk samples for thermal resistivity testing. The boring locations were selected and the locations shown on the exhibit titled "Geotechnical Engineering Services", prepared by Burns & McDonnell. The boring coordinates were downloaded from Google Earth and were marked in the field by MAE personnel. Prior to starting our field exploration, a utility locate request was submitted to the Sunshine 811 Call Center. Once the site utilities were marked and cleared, our field crew mobilized to the site. GPS coordinates for the final boring location were obtained in the field at the time of drilling using a Garmin GPSMAP78 hand-held receiver. The boring location as shown on the *Boring Location Plan* sheets, Figures 2, should be considered approximate based on the method of layout and the equipment used.

2.1 SPT Borings

To explore the subsurface conditions within the area of the proposed project, we located and performed 3 Standard Penetration Test (SPT) borings along the pipeline alignment to depths of approximately 60 feet below the existing ground surface, in general accordance with the methodology outlined in ASTM D 1586. Additional split-spoon samples were obtained where the SPT N-value was less than 4 blows per foot. Split-spoon soil samples recovered during performance of the boring were visually described in the field and representative portions of the samples were transported to our laboratory for further testing and classification. Once the boring is complete the borehole is backfilled from bottom to top with cement-bentonite grout. A summary of the field procedures is in Appendix A.

3.0 LABORATORY TESTING

3.1 Soil 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.

3.2 Index Testing

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 passing US No. 200 sieve (percent fines), natural moisture content, Atterberg Limit and organic content of selected soil samples. Grain size distribution (sieve analysis) tests were also performed on selected soil samples. The results of the laboratory testing are shown in the *Summary of Laboratory Index Test Results*, Table 1, included in Appendix B. Also, these results are shown on the *Generalized Soil Profiles*, Figures 3, and on the *Boring Logs* records at the respective depths from which the tested samples were recovered. A summary of the laboratory test procedures is also included in Appendix B.

3.3 Corrosion Series Tests

Eleven soil corrosion series tests were performed on bulk soil samples at varying depths obtained from each boring location to determine the environmental classification. The sample was classified in accordance with FDOT procedures contained in Chapter 1.3.2.1 of the January 2026 edition of the *FDOT Structures Design Guidelines*. The sample depth and test results are included on the *Summary of Corrosion Series and Electrical Resistivity Test Results,* Table 2, included in Appendix B.

3.4 Modified Proctor Tests

Eleven bulk samples of the existing soils within the boring locations were collected for determination of the maximum dry density and optimum moisture content of the existing soil. The samples were obtained at borings B-1 and B-3 at depths of 5 and 8 feet below the existing ground surface, and at boring B-2 at depths of 5, 8, 20, 30, 40, 50, and 60 feet below the existing ground surface. The Florida Method of Tests FM 1-T-180 was used to determine the maximum dry density and optimum moisture content of the composite sample. The test results are presented in Appendix C.

3.5 Thermal Resistivity Tests

Eleven bulk samples of the existing soils within the boring locations were collected, containerized, stored, and shipped to GeoTherm to determine the thermal resistivity of the soil. The samples were obtained at borings B-1 and B-3 at depths of 5 and 8 feet below the existing ground surface. The samples were obtained at boring B-2 at depths of 5, 8, 20, 30, 40, 50, and 60 feet below the existing ground surface. The test results are presented in Appendix E.

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 General Soil Profile

Graphical presentation of the generalized subsurface conditions is presented on the Generalized Soil Profiles sheet, Figures 3. Detailed boring records are included in Appendix A. When reviewing the soil profile and boring log, it should be understood that the soil conditions will vary outside this boring location.

In general, the borings encountered a pavement section consisting of an asphaltic concrete layer to varying depths of approximately 3 to 7 inches below ground surface. Boring B-1 is underlain by a limerock base approximately 6 inches. Limerock base was not observed within borings B-2 and B-3. Below the asphalt concrete and limerock base, the boring encountered dense to very loose fine sand (SP), fine sand with silt (SP-SM), fine sand with clay (SP-SC), silty fine sand (SM), and clayey fine sand (SC), underlain by soft to hard weathered limestone to the boring termination depth of 60 feet below the existing ground surface. Occasional samples encountered trace to few organic fines and trace shell fragments and weathered limestone.

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 to 7 feet 7 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 re-measured prior to construction. Estimated groundwater levels are shown in the boring profiles and boring logs.

4.3 Review of the USDA Web Soil Survey Map

A review of the USDA Soil Survey Conservation Service (SSCS) Web Soil Survey (WSS) of Duval County show the following soil map unit(s) at the project site as tabulated below. 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)
69	Urban Land[2]	---	---	---
75	Urban land-Hurricane-Albany complex[3], 0 to 5 percent slopes	Somewhat poorly drained	A	24 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 Urban land classification does not have an associated soil type, drainage class, hydrologic group, and estimated seasonal high groundwater level typically reported in the WSS.

3. 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.

4.4 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 as shown in the Generalized Soil Profiles sheets, Figures 3.

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.

4.5 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. 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. 50.

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 Group 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.

5.0 DESIGN RECOMMENDATIONS

5.1 General

The following geotechnical engineering evaluation and recommendations are based on the results of the field and laboratory testing performed, our experience with similar soil conditions, and our understanding of the project information provided. If the project information presented in this report is incorrect, or if the project details change prior to final design, then MAE should be contacted so that these recommendations can be reviewed. Also, the discovery of any site or subsurface conditions during construction that deviate from the data presented herein should be reported to us for evaluation. We recommend that we be provided the opportunity to review the plans and earthwork specifications before construction to verify that our recommendations have been properly interpreted and implemented.

5.2 Direct Embed Recommendations

Based on the results of the field exploration, we consider the subsurface conditions along the structures replacement locations to be favorable for support of the proposed transmission pole structure near Boring B-3 when supported upon a properly designed deep foundation system consisting of either a spun-concrete or steel direct embedded foundation with either rock or concrete backfill.

The estimated soil parameters that can be used for design of the foundations are presented in Appendix C. The parameters were estimated based on empirical correlations between the results of the SPT borings, using corrected N-values as noted in the FDOT's Soils and Foundations Handbook, and various soil properties.

For design purposes, we recommend that the groundwater level be assumed to be at existing grade. We recommend the following values be utilized for design:

Crushed Rock Backfill Properties: FDOT No. 57 Stone

- Total Unit Weight 115 PCF
- Friction Angle 34 degrees
- Deformation Modulus 4.0 KSI

5.3 Below Grade Utility Support 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 proposed below-grade 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 structures, such as manholes, vaults, and duct banks.

5.3.1 Allowable Soil Bearing Pressure

The maximum allowable net soil bearing pressure for below grade structures 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 structure should be designed based on the maximum load that could be imposed by all loading conditions.

The structures should bear in either compacted approved 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 D1557/AASHTO T-180), to a depth of at least one foot below the bearing level.

5.3.2 Lateral Pressure Design Parameters

In general, structures 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, such as traffic loading, and adequate drainage provisions should be incorporated in the design. Passive resistance, resulting from footing embedment at the wall toe, could be neglected for safer design.

The following soil parameters can be used where suitable fill soils are placed and compacted adjacent to structures as described in Section 6.0:

- Backfill Soil Unit Weight, Saturated (γ_{sat}) = 120 pcf
- Backfill Soil Unit Weight, Moist (γ_m) = 115 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, Moist (γ_m) = 115 pcf*

  *Assumes optimum moisture at 95% Modified Proctor
- Foundation Soil Unit Weight, Submerged (γ_m) = 58 pcf**

  **Applicable where footings will be subjected to standing water
- Foundation Soil Angle of Internal Friction (ϕ) = 30 degrees

The above parameters are based on sand backfill (SP/A-3) placed and compacted against structures as discussed in Section 6.0 below. A coefficient of friction between concrete and sand of 0.3 for precast concrete or 0.45 for cast-in-place concrete may be used in the design. The structures should be designed to include all temporary construction and permanent traffic and surcharge loads acting on the structures.

For the design of lateral loads on below-grade structures, we recommend that the groundwater level be assumed to be at the ground surface. Lateral pressure distributions in accordance with the above do not take into account forces from construction equipment, wheel loads or other surcharge loads. To account for this loading, a pressure equal to 0.5 times the anticipated surface surcharge should be applied over the full height of all walls.

5.3.3 Resisting Lateral Forces

Horizontal forces that act on duct banks and/or manhole structures can be resisted to some extent by the earth pressures that develop in contact with the buried perpendicular face of the block structure, and by shearing resistance mobilized along the block structures base and subgrade interface. Allowable passive earth pressure resistance may be determined using the following equivalent fluid densities; a factor of safety of 3 was already taken into consideration for these values:

- Above Water Table - Equivalent Fluid Density 100 lb/ft^3
- Below Water Table - Equivalent Fluid Density 58 lb/ft^3

It is assumed the block structures are surrounded by well compacted structural backfill, as described in

Section 6.0 below, extending at least 5 feet horizontally beyond the vertical bearing face. In addition, it is presumed that the block structures can withstand horizontal movements on the order of 0.5-inch before mobilizing full passive resistance.

The allowable sliding shearing resistance mobilized along the base of the block structure may be determined by the following formula:

$$P = {}^{1}/_{3}V \tan ({}^{2}/_{3}\,\phi)$$

Where: P = Allowable shearing resistance force
V = Net vertical force (total weight of block and soil overlying the structure minus hydrostatic uplift forces)
ϕ = Angle of internal friction = 30°

The following unit weights can be used to calculate the weight of the overburden soil:

- Compacted Moist Soil 115 lb/ft^3
- Saturated Soil 120 lb/ft^3

5.3.4 Hydrostatic Uplift Resistance

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

- Addition of dead weight to the structure; and
- Mobilizing the dead weight of the soil surrounding the structure through extension of footings outside the perimeter of the structure.

A moist compacted soil unit weight of 115 lb/ft^3 may be used in designing structures to resist buoyancy.

6.0 SITE PREPARATION AND EARTHWORK RECOMMENDATIONS

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

6.1 Clearing

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 pipelines or structures.

During the excavation process, pavement section materials such as asphalt and limerock should be stockpiled a safe distance from the construction areas to be removed from the site. We do not recommend use of any of the pavement materials as pipe bedding or backfill within the cable relocation excavations. Any topsoil and surficial organic soils should not be reused as backfill material within the cable relocation excavations. However, they may be stockpiled and used subsequently in areas to be grassed.

6.2 Temporary Groundwater & Surface Water Control

The groundwater level was encountered at the boring locations at depths varying from 4 feet to 7 feet 7 inches below the existing ground surface at the time of our exploration. Because of the need for excavation to a depth of 10 to 12 feet below existing grade, followed by compaction of the bedding and backfill soils, it may be necessary to install temporary groundwater control measures to dewater the area to facilitate the excavation and compaction processes.

Groundwater control measures should be determined by the contractor but can consist of sumps or wellpoints (or a combination of these or other methods) capable of lowering the groundwater level to at least 2 feet below the required depth of excavation. The dewatering system should not be decommissioned until excavation, compaction, and fill placement is complete, and sufficient deadweight exists on the structures to prevent uplift. During excavation of the trench, surface water during rainfall events should be diverted or captured and re-routed to avoid impacts to the excavation.

6.3 Open-Cut Cable Relocation Recommendations

Where open-cut installation of the cable relocation will occur, no change to the existing ground surface elevation is anticipated. The trench and manholes will be excavated to a depth of 10 to 12 feet below existing grade. The duct bank housing the transmission cable is expected to be 2 to 4 feet wide. Based on the results of the subsurface exploration and laboratory testing as discussed in this report, we consider the subsurface conditions encountered in the borings to be adaptable for supporting this portion of the proposed cable relocation when constructed by open-cut methods upon properly prepared subgrade soils.

The borings encountered an asphalt concrete section underlain by fine sands , fine sands with silt, and fine sand with clay (SP, SP-SM, SP-SC/A-3) throughout the boring profiles. These soils are suitable as pipe bedding and are suitable for use as pipe backfill soil. These soils should be placed and/or compacted as discussed in Section 6.4 below.

Silty fine sands (SM/A-2-4) were encountered at all borings, these soils may be encountered at or near the excavation depth along the cable route. The silty soils are not recommended for use as pipe bedding or backfill due to their affinity for moisture, which makes them difficult to place and compact. Where silty fine sands (SM/A-2-4) are encountered they should be removed to a minimum depth of 12 inches below the duct bank and should be replaced with approved sand (SP, SP-SM, SP-SC/A-3) backfill. Alternatively, they could be blended with the approved sand soils such that the blended soil meets the structural backfill recommendations and comprise fine sand (SP, SP-SM, SP-SC/A-3) soil with less than 10 percent fines.

Clayey fine sands (A-2-6/SC) were encountered at all borings. Boring B-1 predominately encountered clayey fine sands at or near the excavation depth, beginning at a depth of approximately 8 feet to approximately 43.5 feet below existing grade. At Boring B-2, these soils are encountered at or near the excavation depth, beginning at 4 feet to 10 feet and at 23.5 feet to 48.5 feet below the existing ground surface. At Boring B-3 these soils are encountered at 28.5 feet to 38.5 feet below the existing ground surface. Given the expected trench depth and the depths where these clayey soils were encountered, it is expected that these soils will be encountered during open-cut excavation. If clay or clayey soils are encountered, they are not recommended for pipe bedding and should be excavated to a depth of at least 12 inches below the duct bank elevation, respectively, and should be replaced with properly compacted approved structural fill soil as described herein. The purpose of this is to provide more uniform bearing conditions, and to reduce the potential for post construction settlements of the cable. Following removal, clayey soils (SC or A-2-6) soils should be stockpiled for disposal separately from the fine sand (SP, SP-SM, SP-SC/A-3) soils and from silty fine sand (SM/A-2-4) soils to be blended with approved sand soils for backfill, and should be replaced with approved structural backfill soil.

Alternatively, a graded aggregate conforming to ASTM No. 67 stone may be used as pipe bedding where clayey or silty soils are present at duct bank elevation. However, if the graded aggregate is used as the pipe bedding material, then the unsuitable soils only require excavation to a depth of 12 inches below the duct bank elevation. Where the bottom of the ASTM No. 67 stone will be on top of the finer-grained soils, a non-woven geotextile should be placed at the gravel-clayey/silty soil interface to function as a separation layer. This fabric will help reduce the potential for infiltration of the soil fines into the gravel material. The stone should be placed in equal lifts of 6 inches or less, with each lift compacted to form a stable working surface.

6.4 Structural Backfill and Fill Soils

Imported structural fill should consist of 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 sands and fine sands with silt and clay (SP, SP-SM, SP-SC/A-3) 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 D1557/AASHTO T-180) during the compaction operations. Compaction should continue until densities of at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557/AASHTO T-180) have been achieved within each lift of the compacted structural fill.

We recommend that soil excavated which will be reused as backfill, be stockpiled a safe distance from the excavations in a manner that promotes runoff away from the open trenches and limits saturation of the materials.

6.5 Compaction of Pipe Backfill

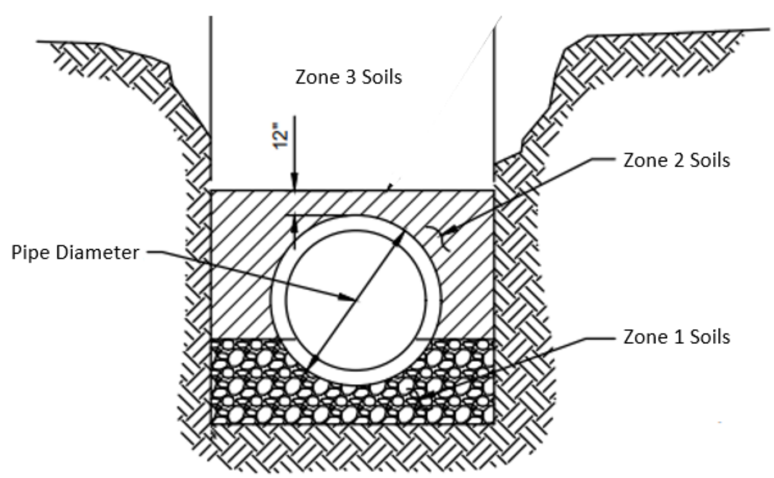
After installing the temporary groundwater control measures, achieving the required depth of excavation, and removing any unsuitable soil as described in Section 6.3, the exposed sand soil surface (Zone 1 soils) should be evaluated by the Contractor to ensure the undisturbed soil is (SP, SP-SM, SP-SC/A-3) material. In areas where existing silty (SM/A-2-4) or clayey (SC/A-2-6) soils are over-excavated, suitable backfill (SP, SP-SM, SP-SC/A-3 soils) should meet the compaction criteria shown in the table below (or alternatively ASTM No. 67 Stone may be used as discussed in Section 6.3).

Zone 1 consists of undisturbed soils or approved backfill/bedding below the pipe.

Zone 2 consists of backfill soils from below the pipe to 1 foot above the top of pipe.

Zone 3 consists of backfill soils from 1 foot above the top of pipe to the ground surface or bottom of stabilized subgrade for a pavement section.

Typically backfill soils should exhibit moisture contents within ±2 percent of the Proctor optimum moisture content and should meet the compaction requirements shown in the table below.



Open Cut Trench Detail

Location	**Material Type**	**Minimum Compaction Requirement**		**Maximum Lift Thickness**
		Test Type	**Requirement**	
CITY OF JACKSONVILLE EASEMENTS (ROADWAY <u>SHOULDER</u> & <u>UNDER PAVEMENT</u>)				
<u>Zone 1</u>: Below Pipe	Undisturbed Soil (SP, SP-SM, SP-SC/A-3)	∗ (1)	∗ (1)	∗ (1)
	Select Fill (SP-, SP-SM, SP-SC/A-3)	ASTM D1557/AASHTO T-180	95% Max. Dry Density	6 inches
	ASTM No. 67 Stone (2)	N/A	N/A	N/A
	Flowable Fill (3)	N/A	N/A	N/A
<u>Zone 2</u>: Up to 1-Foot Over Top of Pipe	Select Fill (SP, SP-SM, SP-SC/A-3)	ASTM D1557/AASHTO T-180	98% Max. Dry Density	6 inches
<u>Zone 3</u>: Over 1-Foot From Top of Pipe to Grade	Select Fill (SP, SP-SM, SP-SC/A-3)		95% Max. Dry Density	12 inches

1. Undisturbed Soil below the pipe must not consist of unsuitable soil as described in Section 6.3. If unsuitable soil is present, this material should be over-excavated and replaced as described in Section 6.3.
2. In lieu of the A-3 soil material for backfill and bedding, granular backfill shall consisting of well-graded crushed stone or crushed gravel meeting the requirements of ASTM Designation C33, Gradation 67 (3/4-inch to No. 4) may be used.
3. Flowable fill may be used in accordance with Section IX.4 of the JEA Water & Wastewater Standards.

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 removed and backfilled with dry structural fill soils that are then compacted, or (2) the excess moisture content within the disturbed soils allowed to dissipate before recompacting.

Structural backfill placed within the cable relocation excavation, and in areas in which over-excavation of unsuitable soils is required below the duct bank elevation, should be placed in loose lifts not exceeding the lift thicknesses shown in the table above and compacted using hand or mechanically-operated compaction equipment.

Structural backfill placed around structures should be placed in 6-inch lifts and compacted with hand-operated compaction equipment. Heavy compaction equipment should not be used within 5 feet of structures to prevent overstressing of the structure walls.

Structural backfill is defined as a non-plastic, granular soil having less than 10 percent material passing the No. 200 mesh sieve and containing less than 4 percent organic material. The sand soils (SP, SP-SM, SP-SC/A-3) meeting the properties given above, as encountered in the borings, may be used as backfill. The (SM/A-2-4) soils as encountered in the borings can be blended with the (SP, SP-SM, SP-SC/A-3) soils provided the blended soil meets the structural backfill recommendations given above.

The backfill soils should exhibit moisture contents within ±2 percent of the Modified Proctor optimum moisture content during the compaction operations. Compaction should continue until density requirements in the table above have been achieved within each lift of compacted structural backfill.

We recommend that soil excavated from the cable relocation trenches that 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 excavated soil.

Care should be exercised to avoid damaging any nearby structures while the compaction operations are underway. Compaction should cease if deemed detrimental to adjacent structures.

6.6 Horizontal Directional Drilling (HDD) Recommendations

Based on the information provided we understand that trenchless excavation methods, such as HDD, may be used in lieu of traditional open cut excavation to avoid conflicts with existing utilities. The HDD bore will extend no more than 50 feet below the existing ground surface.

The borings predominantly encountered medium dense to very loose fine sandy soils throughout most of the subsurface profiles. Soft to hard weathered limestone were also encountered in the borings beginning at a depth of 38.5 feet to 60 feet below the existing ground surface. Therefore, hard drilling is anticipated within the depths of the borings. The contractor performing the HDD operations in this area should anticipate difficult drilling operations and be prepared to use specialized equipment or procedures as necessary. Additionally, the HDD operations should be performed in accordance with Section 555 of the *Florida Department of Transportation (FDOT) Standard Specifications for Road and Bridge Construction 2026*. Any soil conditions encountered that are not consistent with those contained within this report should be reported to MAE for our evaluation.

We recommend the general subsurface conditions as described above and in Section 4.0 be provided to the HDD contractor for use in design of the HDD bore.

6.7 Excavation Protection

Excavation work for the cable relocation will be required to meet OSHA Excavation Standard Subpart P regulations for Type C Soils. The use of excavation support systems for trenches that are 5 feet in depth or deeper will be necessary where there is not sufficient space to allow the side slopes of the excavation to be laidback to at least 1.5H:1V (1.5 horizontal to 1 vertical) to provide a safe and stable working area and to facilitate adequate compaction along the sides of the excavation. Trenches that are less than 5 feet deep may have unsupported, vertical walls if a competent person as defined by OSHA determines that a protective system is not required. In addition, it should be anticipated that an excavation support system may be necessary to protect adjacent existing structures, pavement and/or utilities that are located along the proposed cable route.

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 licensed Professional Engineer. Where cable relocation excavations and the construction of excavation support systems are within 50 feet of existing structures, the existing structures should be monitored for adverse reactions to construction vibrations and dewatering activities.

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 of structures and the pipeline excavations. The density tests are considered necessary to verify that satisfactory compaction operations have been performed.

We recommend density testing on the Pipeline be performed as shown in the table below:

MINIMUM DENSITY TESTING INTERVAL RECOMMENDATIONS (ft)[1]		
Test Location	**JEA ROW & Easements (Under Pavement)**	**JEA ROW & Easements (Not Under Pavement)**
Below pipe	150	150
Up to 1-ft over top of pipe	150	3000
Over 1-ft from top of pipe to grade	150	3000

1. Recommendations obtained from the JEA Water/Wastewater Standards 2023, Section 408

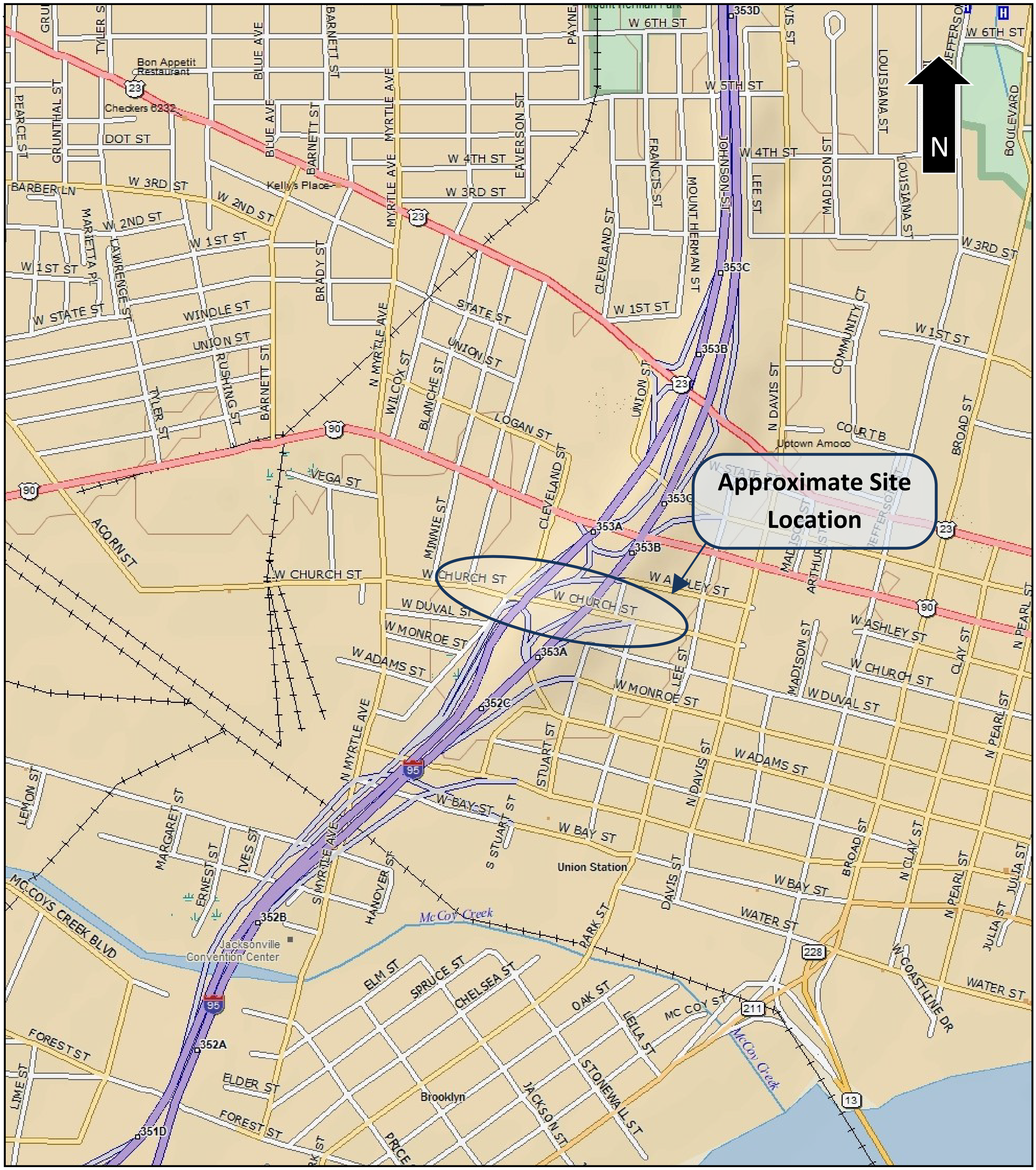
8.0 REPORT LIMITATIONS

This report has been prepared for the exclusive use of Burns & McDonnell for specific application to the design of the *JEA Church Street 69kV Cable Relocation*. 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, then the recommendations in this report must be re-evaluated.

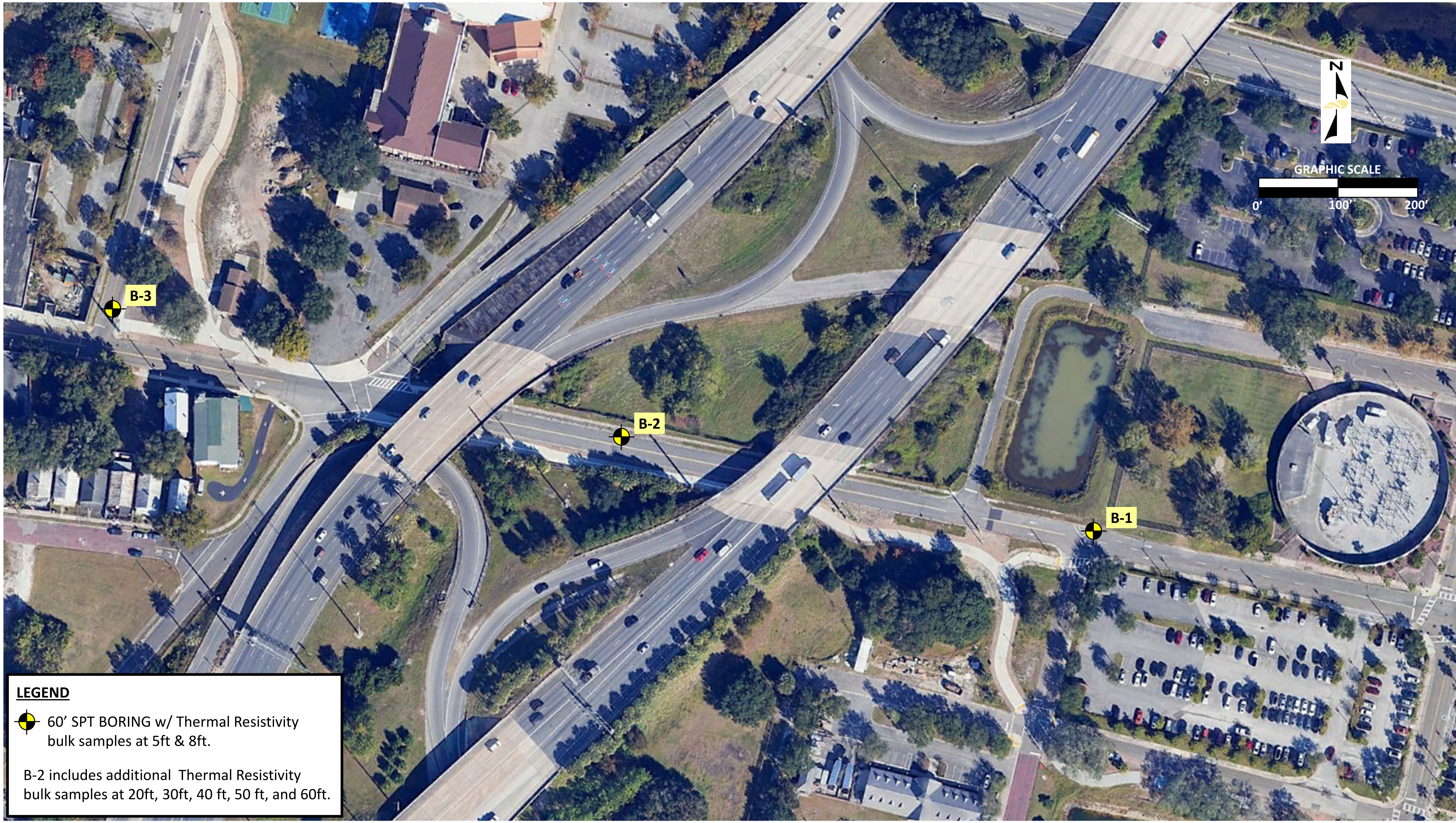
The scope of our services did not include any environmental assessment or testing for the presence or absence of hazardous or toxic materials in the soil, groundwater, or surface water within or beyond the subject site. Any statements made in this report, and/or notations made on the generalized soil 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 cable route 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

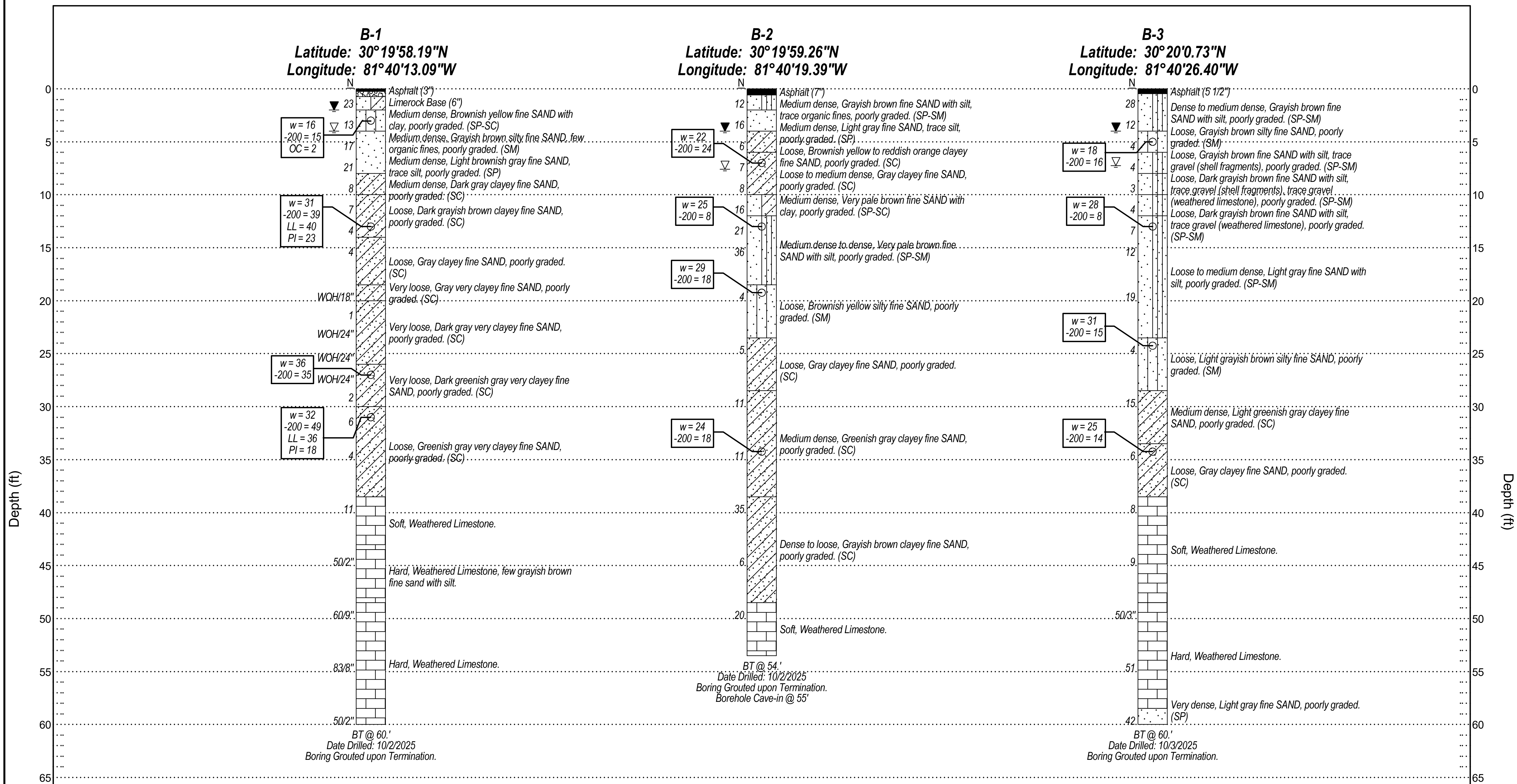


Site Location Map

PREPARED BY	PROJECT NAME		
MAE Meskel & Associates Engineering	JEA Church Street 69kV Cable Relocation Jacksonville, Florida		
	REFEREFERENCE	SCALE	
	Delorme XMap 7.0	NTS	
PREPARED FOR	MAE PROJECT NO.	**FIGURE NO.**	
Burns and McDonnell	0508-0003	1	



Project Manager: DPR	Project No. 0508-0003	MAE Meskel & Associates Engineering	3728 PHILIPS HIGHWAY, SUITE 208, JACKSONVILLE, FL 32207	BORING LOCATION PLAN		FIG NO.
Drawn by: DPR	Scale: AS SHOWN		PH. (904) 519-6990 • FAX (904) 519-6992 • www.MeskelEngineering.com	JEA CHURCH STREET 69KV CABLE RELOCATION JACKSONVILLE, FLORIDA		2
Checked by: BH	File Name: 0508-0003.BLP					
Approved by: BH	Date: 10/22/2025					



Legend

Asphalt	Limerock Base	Fine Sand with Clay
Silty Fine Sand	Fine Sand	Clayey Fine Sand
Limestone	Fine Sand with Silt	

N	Standard Penetration Resistance, Blows/Foot	(SP)	Unified Soil Classification System (USCS)	w	Natural Moisture Content (%)	LL	Liquid Limit
BT	Boring Terminated at Depth Below Existing Grade	∇	Depth to Groundwater at Time of Drilling	-200	% Passing No. 200 U.S. Standard Sieve	PI	Plasticity Index
		▼	Estimated Seasonal High Groundwater Level	OC	Organic Content (%)		

REVISIONS					
DATE	BY	DESCRIPTION	DATE	BY	DESCRIPTION

BRETT H. HARBISON, P.E. P.E. NO.: 74679

MAE
Meskel & Associates Engineering
FL. Registry No. 28142
3728 Philips Highway, Suite 208, Jacksonville, FL 32207

Burns & McDonnell

DATE:	MAE PROJECT NO.
10/14/2025	0508-0003

SHEET TITLE: Generalized Soil Profiles

PROJECT NAME: JEA Church St 69kV Cable Relocation
Duval County, Florida

FIGURE NO. 3

Appendix A

Meskel & Associates Engineering, PLLC
FL. Registry No. 28142
3728 Philips Highway, Suite 208
Jacksonville, FL 32207
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BORING B-1

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation
PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell
DATE STARTED 10/2/2025 **COMPLETED** 10/2/2025 **LATITUDE** 30°19'58.19"N **LONGITUDE** 81°40'13.09"W
DRILLING CONTRACTOR MAE, PLLC **DRILLING METHOD** Standard Penetration Test
LOGGED BY D.Hayward **CHECKED BY** D.Pena **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		Asphalt (3")												
		Limerock Base (6")												
	1	Medium dense, Brownish yellow fine SAND with clay, poorly graded.	SP-SC		- 11 12 11	23								
	2	Medium dense, Grayish brown silty fine SAND, few organic fines, poorly graded.	SM		5 5 8 9	13	16	15	2.0					
5	3	Medium dense, Light brownish gray fine SAND, trace silt, poorly graded.	SP		6 7 10 10	17								
	4				6 8 13 12	21								
10	5	Medium dense, Dark gray clayey fine SAND, poorly graded.	SC		5 5 3 4	8								
	6	Loose, Dark grayish brown clayey fine SAND, poorly graded.	SC		2 3 4 4	7								
	7				1 2 2 1	4	31	39		40	23			
15	8	Loose, Gray clayey fine SAND, poorly graded.	SC		1 2 2 2	4								
20	9	Very loose, Gray very clayey fine SAND, poorly graded.	SC		WOH	WOH /18"								
	10	Very loose, Dark gray very clayey fine SAND, poorly graded.	SC		WOH WOH 1 1	1								
	11				WOH	WOH /24"								
25					WOH									

NOTES Boring Grouted upon Termination.

GROUND WATER LEVELS
AT TIME OF DRILLING 4.00 ft
ESHGWT 2.00 ft

(Continued Next Page)

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BORING B-1

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation

PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell

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	12	Very loose, Dark gray very clayey fine SAND, poorly graded. *(continued)*	SC		WOH ↓	WOH /24"								
	13	Very loose, Dark greenish gray very clayey fine SAND, poorly graded.	SC		WOH ↓	WOH /24"	36	35						
	14				1 1 1	2								
30	15	Loose, Greenish gray very clayey fine SAND, poorly graded.	SC		1 2 4 4	6	32	49		36	18			
	16				2 2 2	4								
35														
	17	Soft, Weathered Limestone.			3 6 5	11								
40														
	18	Hard, Weathered Limestone, few grayish brown fine sand with silt.			50/2" - -	50/2"								
45														
	19	Hard, Weathered Limestone.			12 10 50/3"	60/9"								
50														

NOTES Boring Grouted upon Termination.

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 4.00 ft

▼ **ESHGWT** 2.00 ft

(Continued Next Page)

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BORING B-1

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation

PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell

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
55	20	Hard, Weathered Limestone. *(continued)*			16 33 50/2"	83/8"								
60	21				50/2" - -	50/2"								
		Bottom of borehole at 60 feet.												

NOTES Boring Grouted upon Termination.

GROUND WATER LEVELS

AT TIME OF DRILLING 4.00 ft

ESHGWT 2.00 ft

NEW MAE LOG LAT/LONG-EOD_ESHGWT - NEW TEMPLATE 7-30-12.GDT - 10/14/25 14:15 - F:\GINT\GINT FILES\PROJECTS\0508-0003\JEA CHURCH ST 69KV.GPJ

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BORING B-2

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation
PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell
DATE STARTED 10/2/2025 **COMPLETED** 10/2/2025 **LATITUDE** 30°19'59.26"N **LONGITUDE** 81°40'19.39"W
DRILLING CONTRACTOR MAE, PLLC **DRILLING METHOD** Standard Penetration Test
LOGGED BY D.Hayward **CHECKED BY** D.Pena **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		Asphalt (7")												
	1	Medium dense, Grayish brown fine SAND with silt, trace organic fines, poorly graded.	SP-SM		7 2 10 8	12								
	2	Medium dense, Light gray fine SAND, trace silt, poorly graded.	SP		5 9 7 6	16								
5	3	Loose, Brownish yellow to reddish orange clayey fine SAND, poorly graded.	SC		2 3 3 4	6								
	4	Loose to medium dense, Gray clayey fine SAND, poorly graded.	SC		2 3 4 5	7	22	24						
10	5				2 2 6 7	8								
	6	Medium dense, Very pale brown fine SAND with clay, poorly graded.	SP-SC		6 8 8 10	16								
	7	Medium dense to dense, Very pale brown fine SAND with silt, poorly graded.	SP-SM		7 8 13 23	21	25	8						
15	8				7 15 21 27	36								
20	9	Loose, Brownish yellow silty fine SAND, poorly graded.	SM		1 2 2	4	29	18						
25	10	Loose, Gray clayey fine SAND, poorly graded.	SC		1 2 3	5								

NOTES Boring Grouted upon Termination.
Borehole Cave-in at 55 feet.

GROUND WATER LEVELS
AT TIME OF DRILLING 7.58 ft
ESHGWT 4.00 ft

(Continued Next Page)

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BORING B-2

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation

PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell

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		Loose, Gray clayey fine SAND, poorly graded. *(continued)*	SC											
30	11	Medium dense, Greenish gray clayey fine SAND, poorly graded.	SC		2 4 7	11								
35	12				4 5 6	11	24	18						
40	13	Dense to loose, Grayish brown clayey fine SAND, poorly graded.	SC		6 14 21	35								
45	14				5 1 5	6								
50	15	Soft, Weathered Limestone.			6 12 8	20								

NOTES Boring Grouted upon Termination.
Borehole Cave-in at 55 feet.

GROUND WATER LEVELS

∇ **AT TIME OF DRILLING** 7.58 ft

ESHGWT 4.00 ft

(Continued Next Page)

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BORING B-2

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation

PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell

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
		Soft, Weathered Limestone. *(continued)*												
		Bottom of borehole at 54 feet.												

NOTES Boring Grouted upon Termination.
Borehole Cave-in at 55 feet.

GROUND WATER LEVELS

AT TIME OF DRILLING 7.58 ft

ESHGWT 4.00 ft

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BORING B-3

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation
PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell
DATE STARTED 10/3/2025 **COMPLETED** 10/3/2025 **LATITUDE** 30°20'0.73"N **LONGITUDE** 81°40'26.40"W
DRILLING CONTRACTOR MAE, PLLC **DRILLING METHOD** Standard Penetration Test
LOGGED BY D.Hayward **CHECKED BY** D.Pena **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		Asphalt (5 1/2")												
	1	Dense to medium dense, Grayish brown fine SAND with silt, poorly graded.	SP-SM		12 16 12 8	28								
	2				5 6 6 4	12								
5	3	Loose, Grayish brown silty fine SAND, poorly graded.	SM		2 2 2 2	4	18	16						
	4	Loose, Grayish brown fine SAND with silt, trace gravel (shell fragments), poorly graded.	SP-SM		2 2 2 2	4								
10	5	Loose, Dark grayish brown fine SAND with silt, trace gravel (shell fragments), trace gravel (weathered limestone), poorly graded.	SP-SM		1 2 1 1	3								
	6	Loose, Dark grayish brown fine SAND with silt, trace gravel (weathered limestone), poorly graded.	SP-SM		1 2 2 2	4								
	7	Loose to medium dense, Light gray fine SAND with silt, poorly graded.	SP-SM		2 2 5 4	7	28	8						
15	8				3 6 6 5	12								
20	9				8 7 12	19								
25	10	Loose, Light grayish brown silty fine SAND, poorly graded.	SM		3 2 2	4	31	15						

NOTES Boring Grouted upon Termination.

GROUND WATER LEVELS
∇ **AT TIME OF DRILLING** 7.25 ft
▼ **ESHGWT** 4.00 ft

(Continued Next Page)

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BORING B-3

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation

PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell

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		Loose, Light grayish brown silty fine SAND, poorly graded. *(continued)*	SM											
30	11	Medium dense, Light greenish gray clayey fine SAND, poorly graded.	SC		3 5 10	15								
35	12	Loose, Gray clayey fine SAND, poorly graded.	SC		2 3 3	6	25	14						
40	13	Soft, Weathered Limestone.			2 3 5	8								
45	14				4 4 5	9								
50	15	Hard, Weathered Limestone.			50/3" - -	50/3"								

NOTES Boring Grouted upon Termination.

GROUND WATER LEVELS

AT TIME OF DRILLING 7.25 ft

ESHGWT 4.00 ft

(Continued Next Page)

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BORING B-3

PROJECT NO. 0508-0003

PROJECT NAME JEA Church St 69kV Cable Relocation

PROJECT LOCATION Duval County, Florida **CLIENT** Burns & McDonnell

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
55	16	Hard, Weathered Limestone. *(continued)*			13 22 29	51								
60	17	Very dense, Light gray fine SAND, poorly graded.	SP		10 19 23	42								
		Bottom of borehole at 60 feet.												

NOTES Boring Grouted upon Termination.

GROUND WATER LEVELS

AT TIME OF DRILLING 7.25 ft

ESHGWT 4.00 ft

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." The relative density descriptions have been modified from ASTM D-1586 due to recommendations from FDOT Based on an auto hammer efficiency study. Values have been adjusted by a factor of 1.24. 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.

Once the boring is complete and the groundwater level is measured, the borehole is backfilled from bottom to top with cement-bentonite grout.

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

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 and sandy silts with low to medium plasticity, rock flour
			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 and sandy silts with high plasticity
			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, C=Clayey, L =Low Plastic, LL < 50%, H = High Plastic, LL > 50%

Appendix B

Table 1
Summary of Laboratory Index Test Results
JEA Church St. 69kV Cable Relocation
Jacksonville, Florida
MAE Project No.: 0508-0003

			Gradation Test, % Passing												
Boring No.	**Sample No.**	**Approximate Test Depth[1] (ft)**	**No. 4**	**No. 10**	**No. 20**	**No. 40**	**No. 60**	**No. 100**	**No. 200**	**Natural Moisture Content (%)**	**Organic Content (%)**	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**	**USCS [2] Classification**
B-1	2	2 to 4	100	100	99	98	96	67	15	16	2.0	---	---	---	SM
B-1	7	12 to 14	100	100	100	100	99	84	39	31	---	40.0	17.0	23.0	SC
B-1	13	26 to 28	100	100	100	99	95	62	35	36	---	---	---	---	SC
B-1	15	30 to 32	100	100	100	100	99	81	49	32	---	36.0	18.0	18.0	SC
B-2	4	6 to 8	100	100	100	99	97	54	24	22	---	---	---	---	SC
B-2	7	12 to 14	100	100	100	100	99	35	8	25	---	---	---	---	SP-SM
B-2	9	18.5 to 20	100	100	100	100	99	74	18	29	---	---	---	---	SM
B-2	12	33.5 to 35	100	100	99	97	88	39	18	24	---	---	---	---	SC
B-3	3	4 to 6	100	98	96	95	94	59	16	18	---	---	---	---	SM
B-3	7	12 to 14	100	100	100	100	96	39	8	28	---	---	---	---	SP-SM
B-3	10	23.5 to 25	100	100	99	98	93	44	15	31	---	---	---	---	SM
B-3	12	33.5 to 35	100	98	92	74	55	19	14	25	---	---	---	---	SC

(1) Feet below existing ground surface.

(2) Unified Soil Classification System.

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Table 2
Summary of Corrosion Series Test Results
JEA Church St. 69kV Cable Relocation
Jacksonville, Florida
MAE Project No.: 0508-0003

Approximate Sample Location	GPS Coordinates Latitude and Longitude		Approximate Test Depth (ft)[1]	pH	Resistivity (ohm-cm)	Chlorides (ppm)	Sulfates (ppm)	Environmental Classification	
								Steel Substructure	Concrete Substructure
B-1	30°19'58.19"N	81°40'13.09"W	5	7.94	12,000	180	9	Slighty Aggressive	Slightly Aggressive
B-1	30°19'58.19"N	81°40'13.09"W	8	7.97	4,000	180	36	Moderately Aggressive	Moderately Aggressive
B-2	30°19'59.26"N	81°40'19.39"W	5	7.87	18,000	120	6	Slighty Aggressive	Slightly Aggressive
B-2	30°19'59.26"N	81°40'19.39"W	8	7.91	6,900	75	12	Slighty Aggressive	Slightly Aggressive
B-2	30°19'59.26"N	81°40'19.39"W	20	5.98	8,700	90	33	Extremely Aggressive	Extremely Aggressive
B-2	30°19'59.26"N	81°40'19.39"W	30	6.27	2,000	135	231	Moderately Aggressive	Moderately Aggressive
B-2	30°19'59.26"N	81°40'19.39"W	40	8.18	1,300	240	81	Moderately Aggressive	Moderately Aggressive
B-2	30°19'59.26"N	81°40'19.39"W	50	8.65	4,000	150	126	Moderately Aggressive	Slightly Aggressive
B-2	30°19'59.26"N	81°40'19.39"W	60	8.55	6,000	75	171	Slighty Aggressive	Slightly Aggressive
B-3	30°20'0.73"N	81°40'26.40"W	5	8.53	8,500	150	18	Slighty Aggressive	Slightly Aggressive
B-3	30°20'0.73"N	81°40'26.40"W	8	8.2	4,700	120	33	Moderately Aggressive	Slightly Aggressive

(1) Feet below existing ground surface.

(2) Environmental Classification based on FDOT Structures Design Guidelines Section 1.3.2, Subsection C.1

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.

Atterberg Limits

The Atterberg Limits consist of the Liquid Limit (LL) and the Plastic Limit (PL). The LL and PL were determined in general accordance with the latest revision of ASTM D 4318. The LL is the water content of the material denoting the boundary between the liquid and plastic states. The PL is the water content denoting the boundary between the plastic and semi-solid states. The Plasticity Index (PI) is the range of water content over which a soil behaves plastically and is denoted numerically by the difference between the LL and the PL. The water content of the sample tested was determined in general accordance with the latest revision of ASTM D 2216. The water content is defined as the ration of "pore" or "free" water in a given mass of material to the mass of solid material particles.

Organic Loss on Ignition (Percent Organics)

The organic loss on ignition or percent organic material in the sample tested was determined in general accordance with ASTM D 2974. The percent organics is the material, expressed as a percentage, which is burned off in a muffle furnace at 455±10 degrees Celsius.

Gradation

The particle size analysis or gradation of the sample tested was determined in general accordance with latest revision of ASTM D 422. This test procedure determines the grain size distribution of the tested sample by passing the sample through a standard set of nested sieves.

Appendix C

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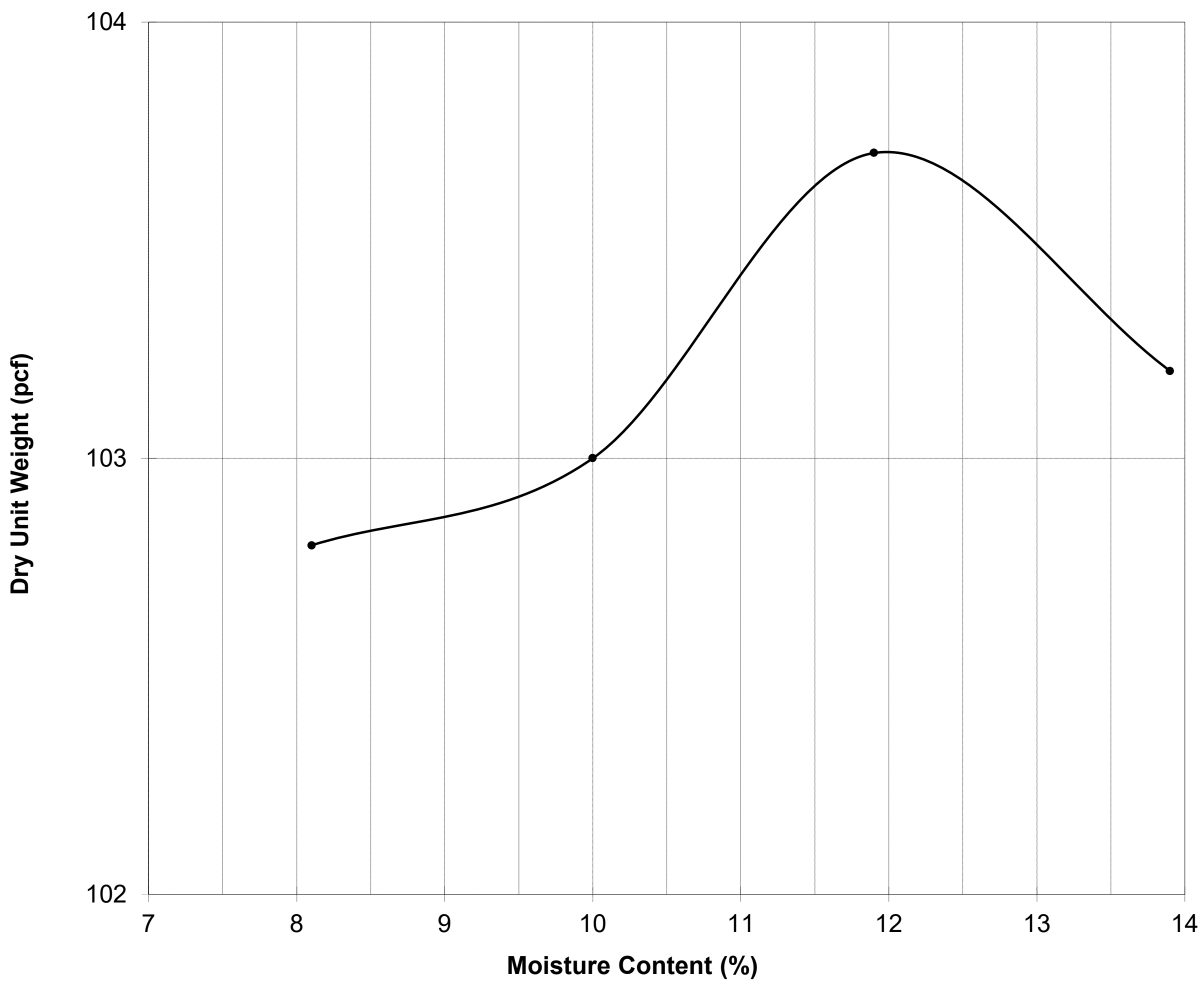
		Report Date:	**11/17/2025**
Project Name:	**JEA Church St. 69kV**	**Test Date:**	**11/12/2025**
Sample Location:	**B-1 5'**	**Sample Date:**	**11/4/2025**
Description:	**Gray Fine Sand with Silt**	**Project Number:**	**0508-0003**
		MDR Number:	**1**

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	**103.6**	**Soil Classification:**	**SP-SM**
Optimum Moisture, %:	**12.0**	**Percent Fines:**	**9%**



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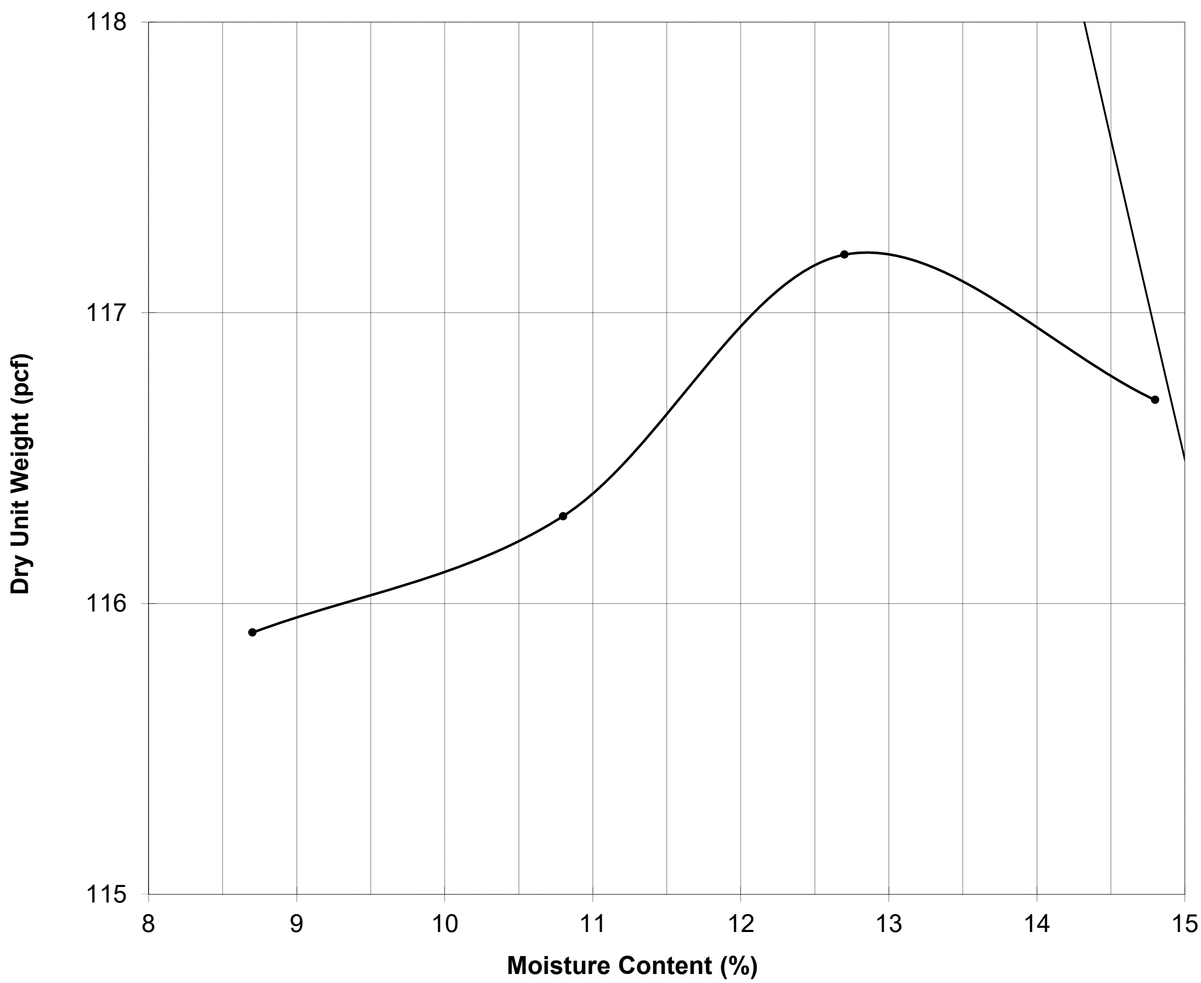
		Report Date:	**11/17/2025**
Project Name:	**JEA Church St. 69kV**	**Test Date:**	**11/12/2025**
Sample Location:	**B-1 8'**	**Sample Date:**	**11/4/2025**
Description:	**Light Brown Clayey Fine Sand**	**Project Number:**	**0508-0003**
		MDR Number:	**2**

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	**117.2**	**Soil Classification:**	**SC**
Optimum Moisture, %:	**12.8**	**Percent Fines:**	**27%**



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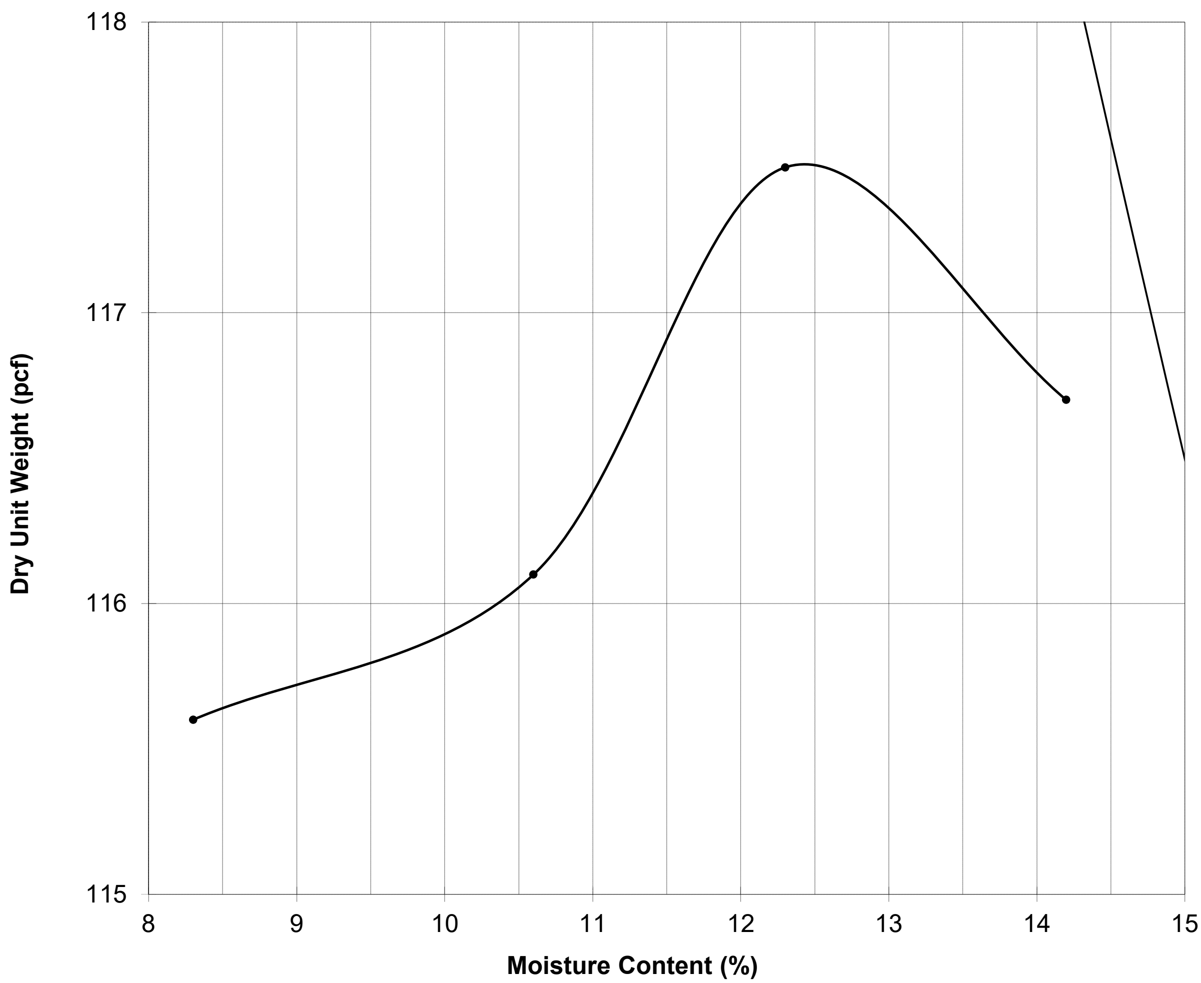
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kV	Test Date:	11/12/2025
Sample Location:	B-3 5'	Sample Date:	11/4/2025
Description:	Light Brown Silty Fine Sand	Project Number:	0508-0003
		MDR Number:	3

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	117.5	Soil Classification:	SM
Optimum Moisture, %:	12.3	Percent Fines:	13%



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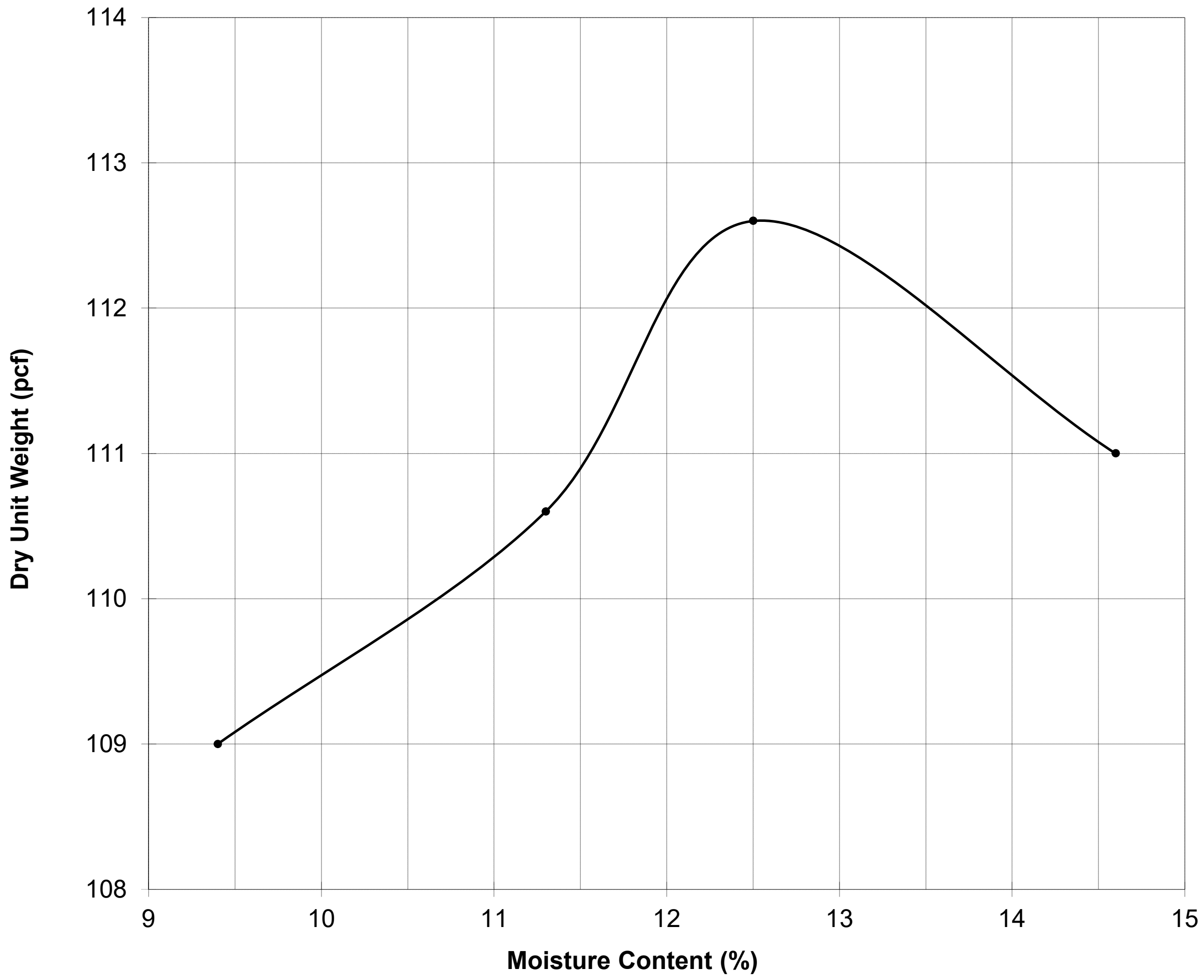
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kV	Test Date:	11/11/2025
Sample Location:	B-3 8'	Sample Date:	11/4/2025
Description:	Light Brown Fine Sand with Silt	Project Number:	0508-0003
		MDR Number:	4

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	112.7	Soil Classification:	SP-SM
Optimum Moisture, %:	12.5	Percent Fines:	7%



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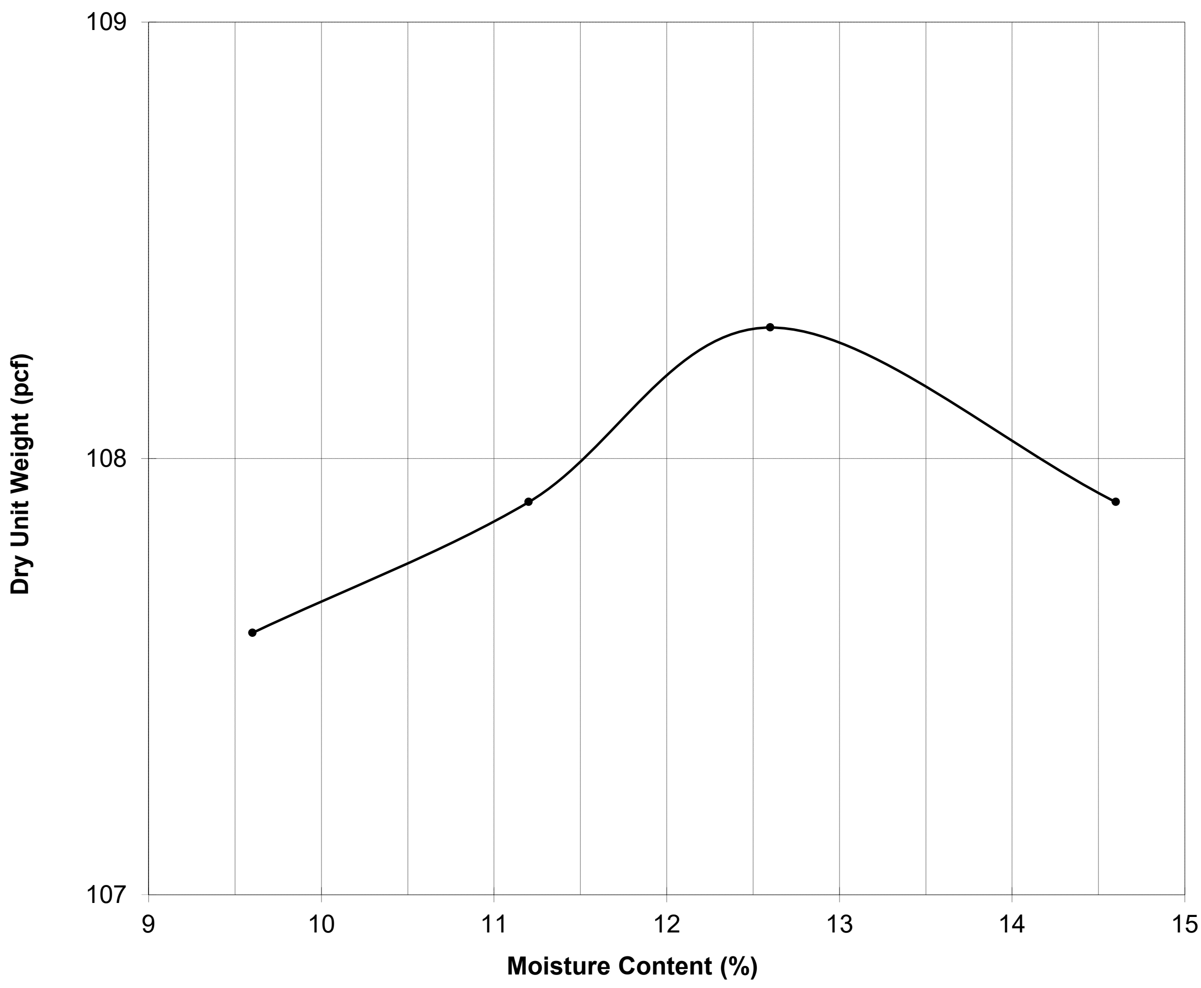
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kv	Test Date:	11/11/2025
Sample Location:	B-2 5'	Sample Date:	11/4/2025
Description:	Tan Clayey Fine Sand	Project Number:	0508-0003
		MDR Number:	5

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	108.3	Soil Classification:	SC
Optimum Moisture, %:	12.6	Percent Fines:	19%



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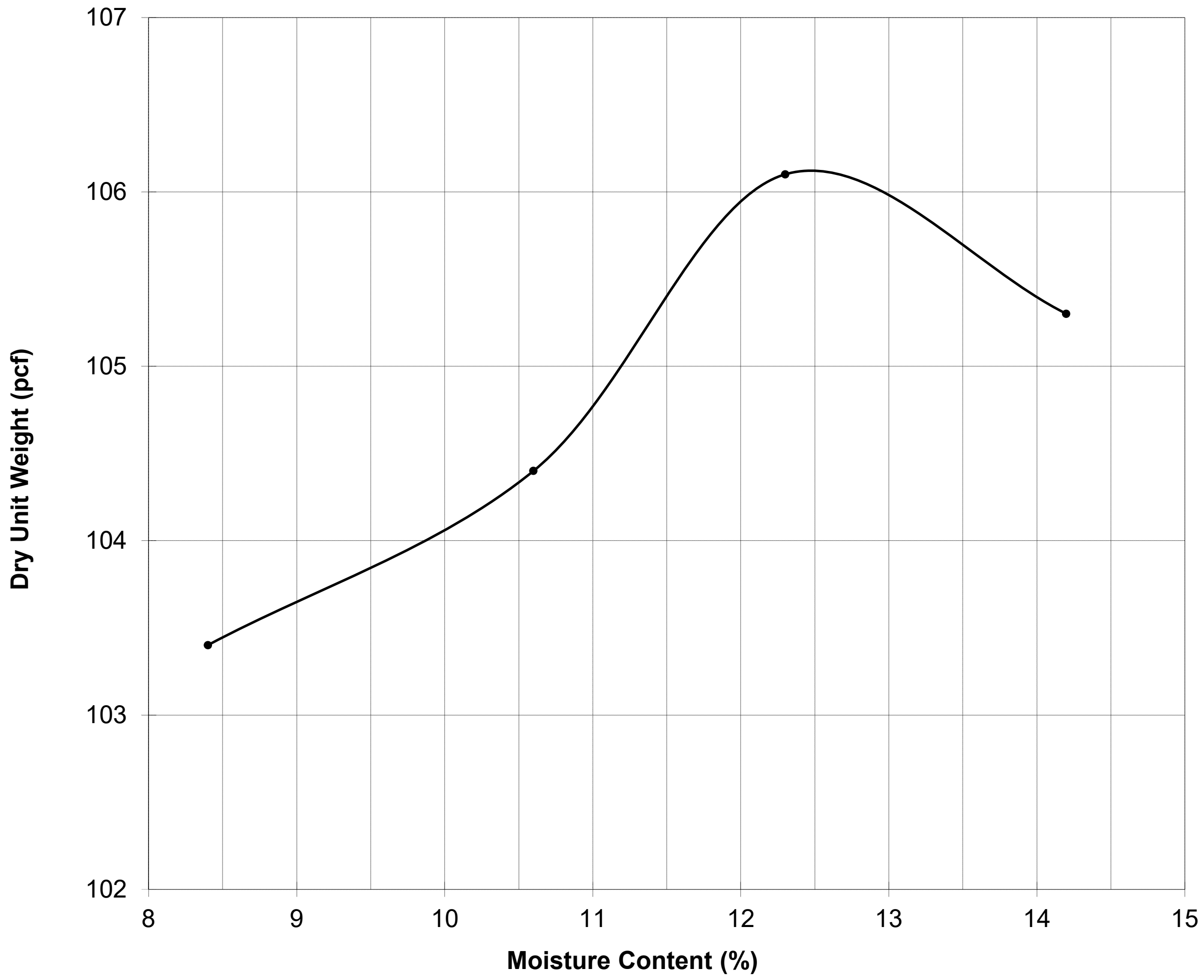
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kV	Test Date:	11/11/2025
Sample Location:	B-2 8'	Sample Date:	11/4/2025
Description:	Tan and Orange Fine Sand	Project Number:	0508-0003
		MDR Number:	6

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	106.1	Soil Classification:	SP
Optimum Moisture, %:	12.4	Percent Fines:	4%



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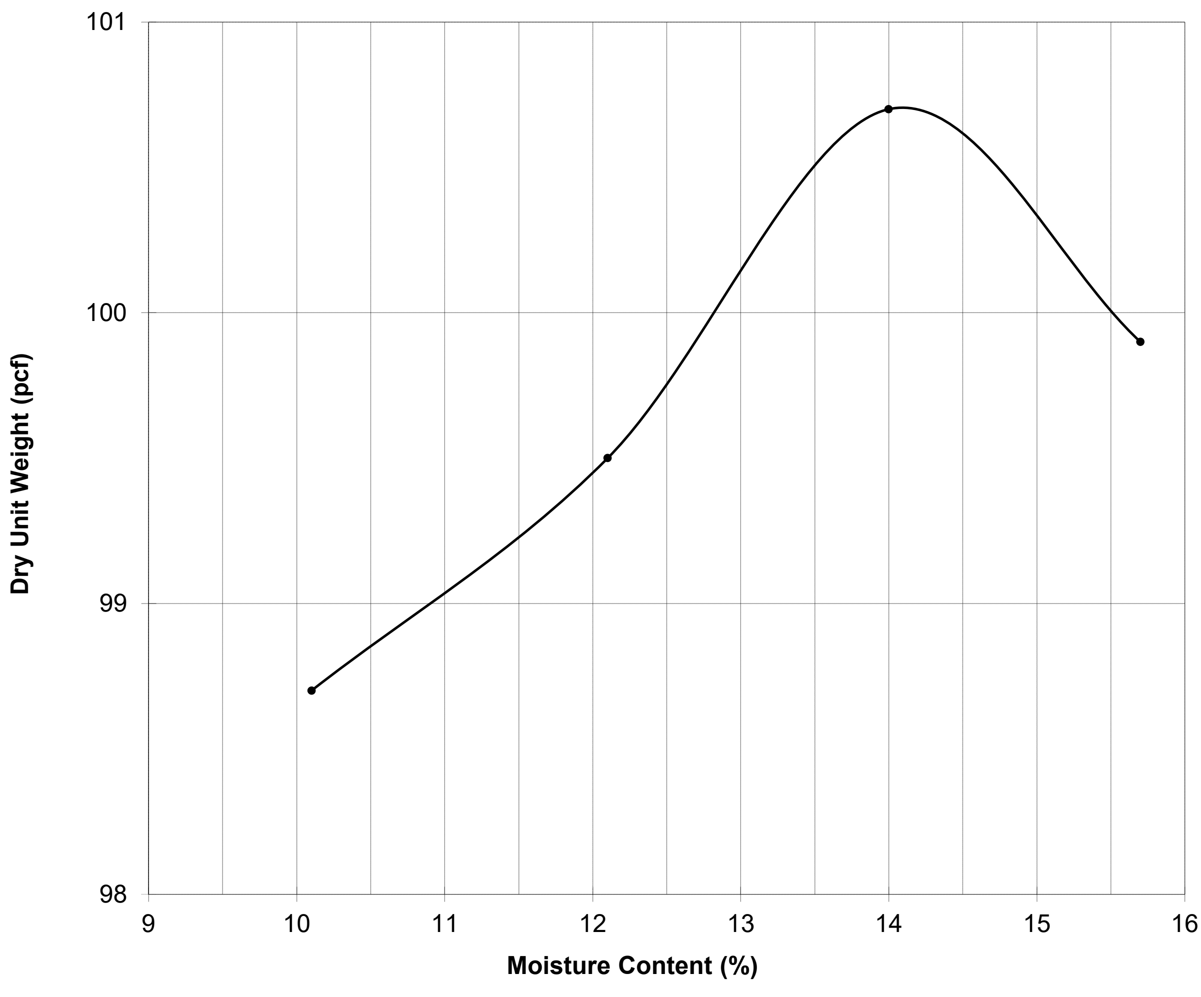
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kV	Test Date:	11/13/2025
Sample Location:	B-2 20'	Sample Date:	11/4/2025
Description:	Tan Fine Sand	Project Number:	0508-0003
		MDR Number:	7

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	100.8	Soil Classification:	SP
Optimum Moisture, %:	14.1	Percent Fines:	2%



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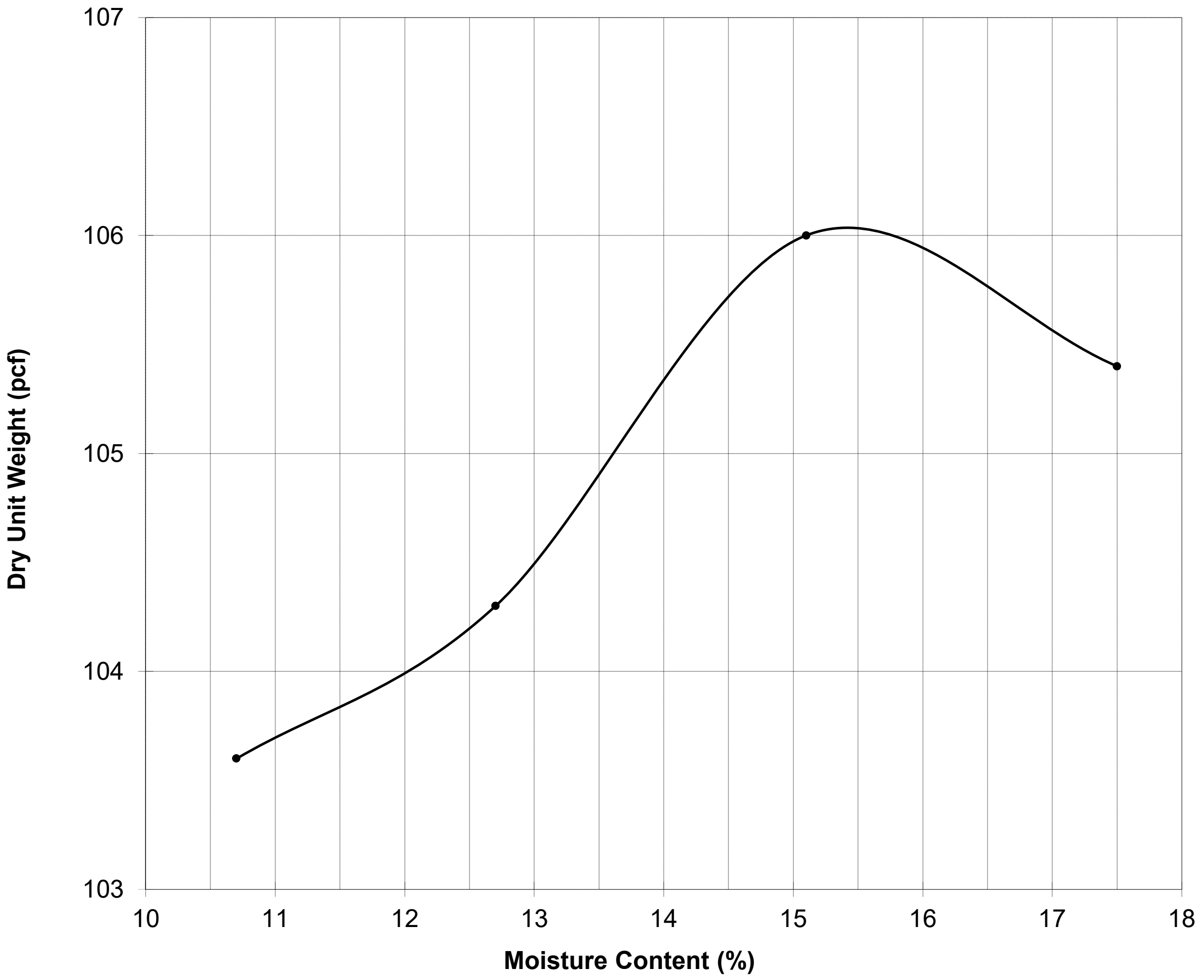
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kV	Test Date:	11/13/2025
Sample Location:	B-2 30'	Sample Date:	11/4/2025
Description:	Tan Clayey Fine Sand	Project Number:	0508-0003
		MDR Number:	7

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	106.1	Soil Classification:	SC
Optimum Moisture, %:	15.3	Percent Fines:	27%



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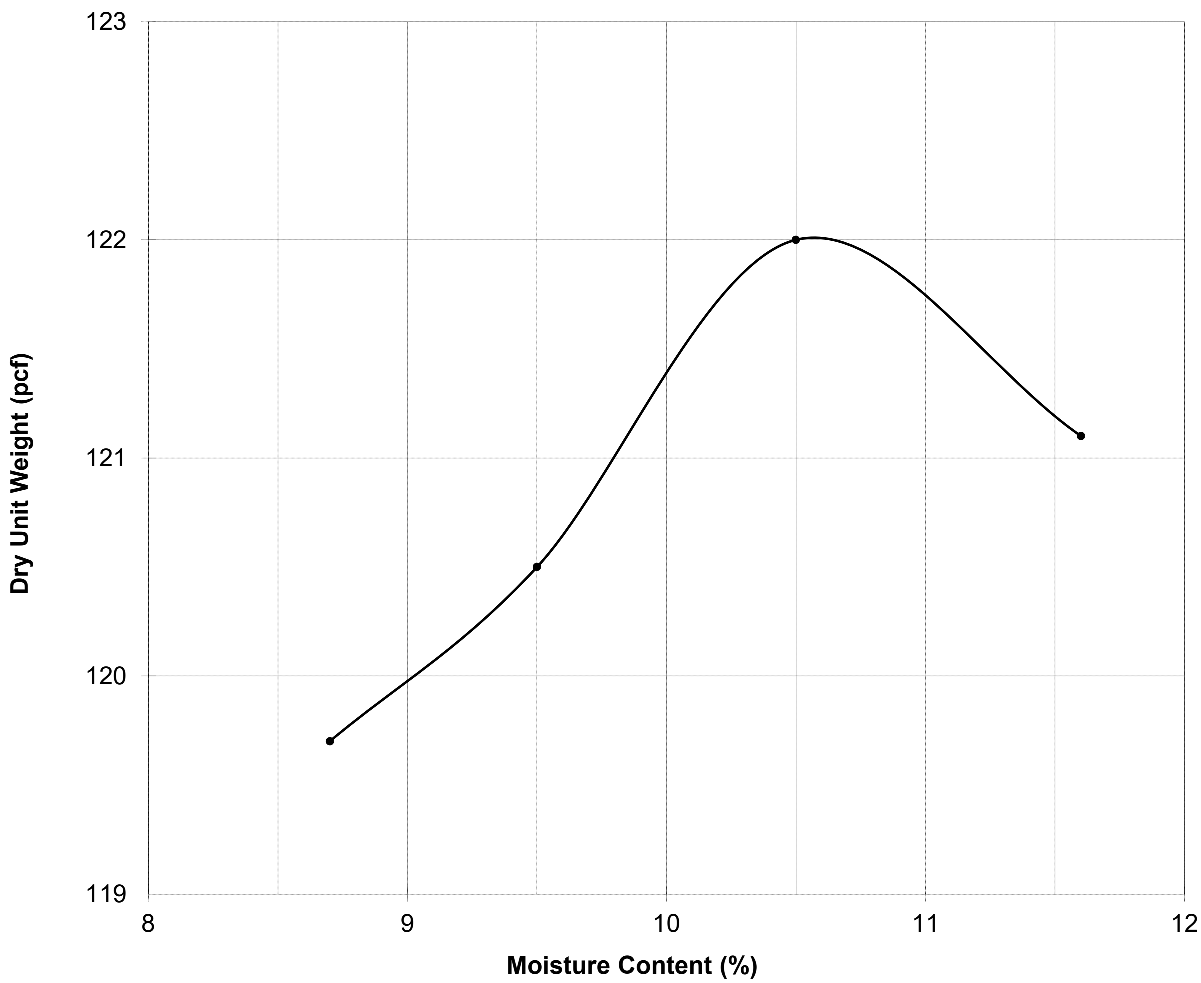
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kV	Test Date:	11/13/2025
Sample Location:	B-2 40'	Sample Date:	11/4/2025
Description:	Dark Greenish to Gray Clayey Fine Sand	Project Number:	0508-0003
		MDR Number:	9

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	122.0	Soil Classification:	SC
Optimum Moisture, %:	10.6	Percent Fines:	40%



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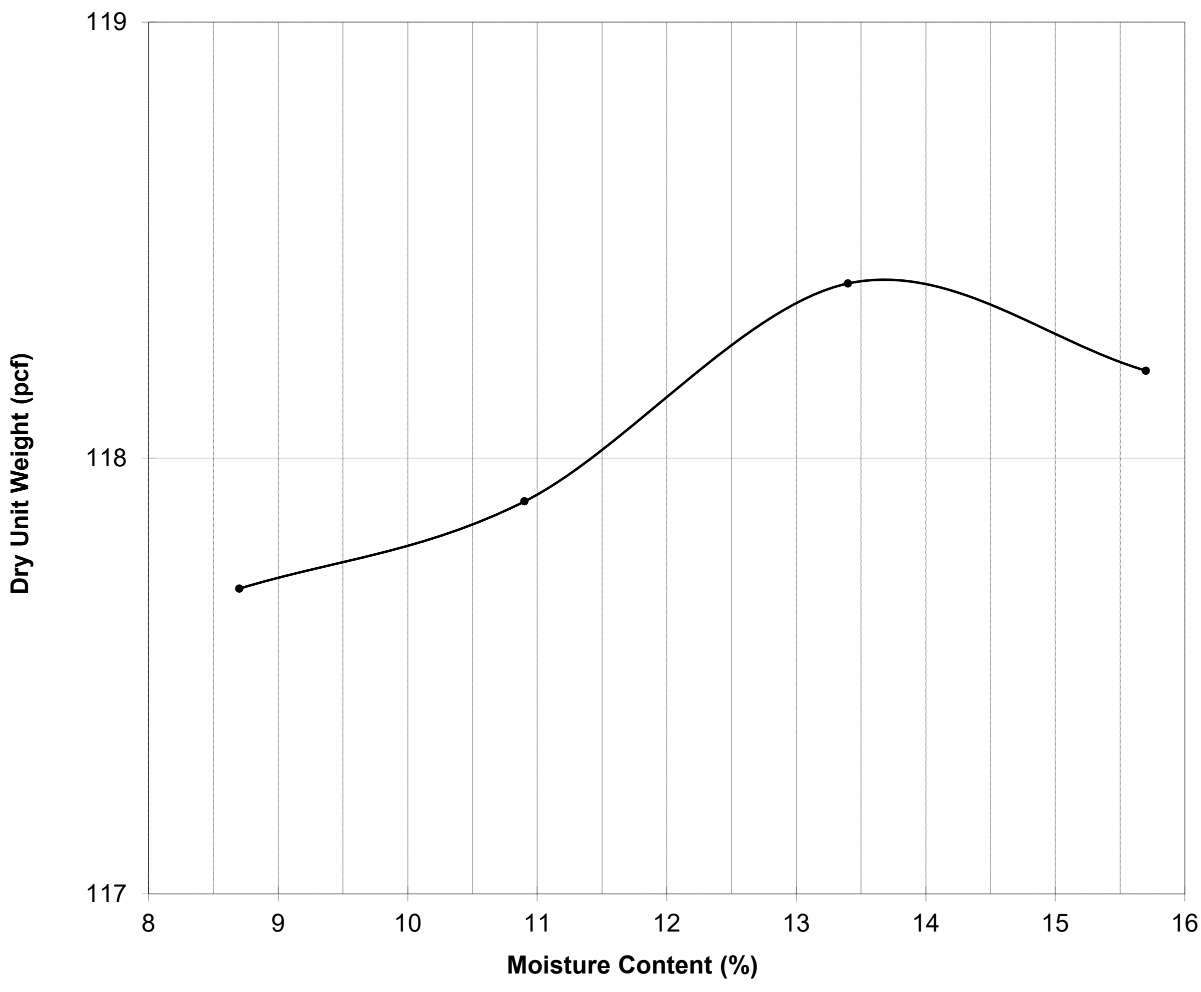
		Report Date:	**11/17/2025**
Project Name:	**JEA Church St. 69kV**	**Test Date:**	**11/13/2025**
Sample Location:	**B-2 50'**	**Sample Date:**	**11/4/2025**
Description:	**Light Brown Fine Sand with Clay**	**Project Number:**	**0508-0003**
		MDR Number:	**10**

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	**118.4**	**Soil Classification:**	**SP-SC**
Optimum Moisture, %:	**13.7**	**Percent Fines:**	**11%**



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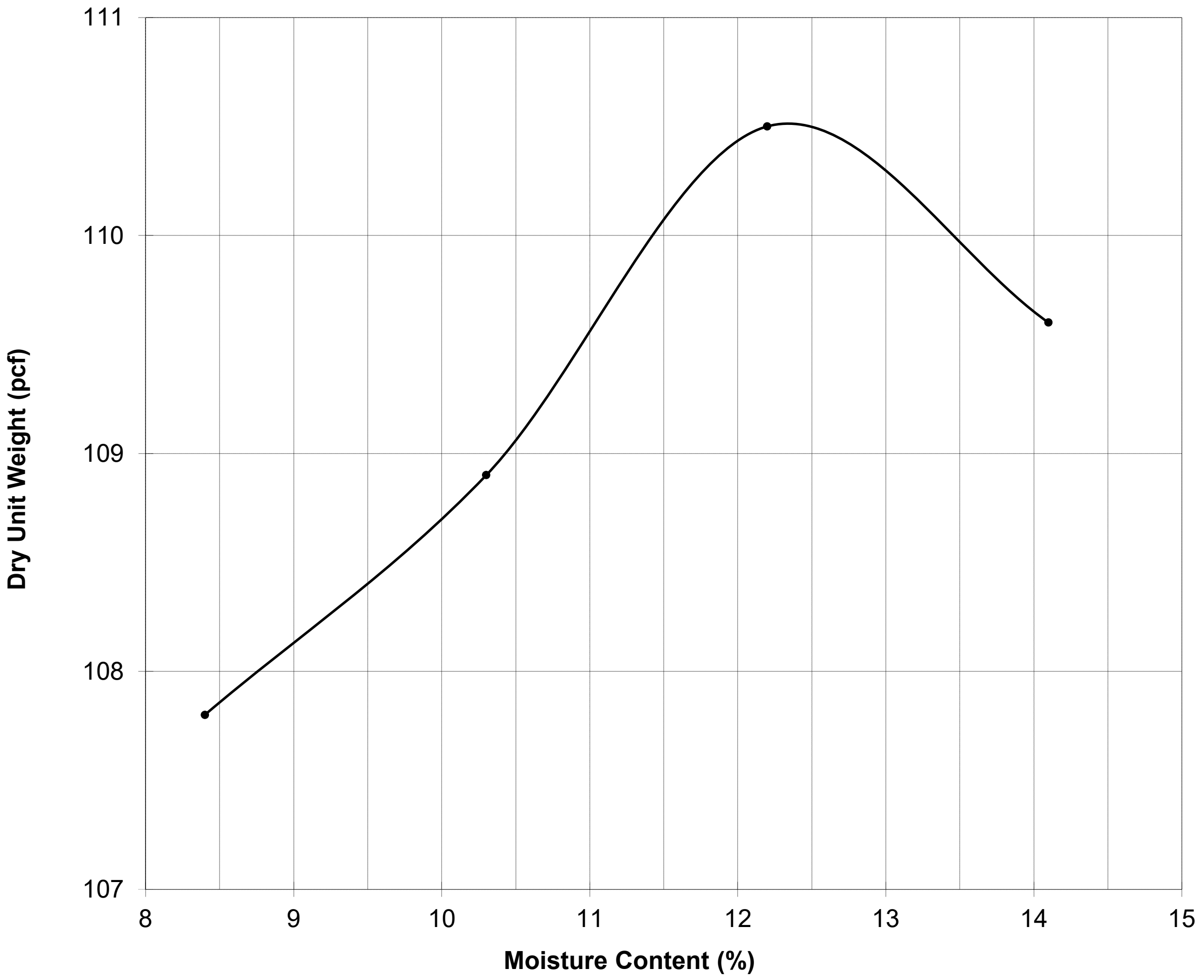
		Report Date:	11/17/2025
Project Name:	JEA Church St. 69kV	Test Date:	11/11/2025
Sample Location:	B-2 60'	Sample Date:	11/4/2025
Description:	Tan Fine Sand	Project Number:	0508-0003
		MDR Number:	11

REPORT ON MOISTURE-DENSITY RELATIONSHIP OF SOIL (AASHTO T-180 Method D)
SUMMARY OF TEST RESULTS

Maximum Dry Density, pcf:	110.5	Soil Classification:	SP
Optimum Moisture, %:	12.4	Percent Fines:	4%



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Appendix D

Soil Parameters for Lateral Analysis
JEA Church St 69kV Cable Relocation
Jacksonville, Florida
MAE Project No.: 0508-0003

Boring No.	Layer No.	Approximate Layer Thicknesses Depth (ft) from	to	Soil: Soil Type	USCS Soil Classification	Lateral Soil Model [1]	Average SPT N-Value, Corrected	ƒ Angle of Internal Friction (degree)	c Cohesion (lb/ft^2)	γ_{sat} Saturated Unit Wt. (lb/ft^3)	γ_E Effective Unit Wt. (lb/ft^3)	k Lateral Subgrade Modulus (lb/in^3)	Deformation Modulus (ksi)	Rock Mass Rating (RMR)	Allowable Side Shear, Comp. ($kips/ft^2$) [2]	Allowable End Bearing, Comp. ($kips/ft^2$) [2]
B1-25	1	0	2	SAND	SP	**1**	**29**	35	0	125	63	86	2.9	N/A	0.04	11.4
	2	2	4	SAND	SM	**1**	**16**	32	0	125	63	48	0.9	N/A	0.11	6.4
	3	4	8	SAND	SP	**1**	**24**	34	0	125	63	71	2.4	N/A	0.22	9.4
	4	8	10	SAND	SC	**1**	**10**	30	0	117	55	30	0.6	N/A	0.30	4.0
	5	10	18.5	SAND	SC	**1**	**6**	30	0	113	50	17	0.3	N/A	0.41	2.5
	6	18.5	30	SAND	SC	**1**	**WOH**	26	0	105	43	5	0.005	N/A	0.54	0.0
	7	30	38.5	SAND	SC	**1**	**6**	30	0	113	50	17	0.3	N/A	0.62	2.5
	8	38.5	43.5	LIMESTONE	---	**1**	**14**	31	0	117	55	41	284.3	24.0	0.67	5.5
	9	43.5	60	LIMESTONE	---	**1**	**100**	40	0	125	63	250	1709.6	60.0	0.73	49.6
B2-25	1	0	4	SAND	SP	**1**	**17**	32	0	122	59	52	1.7	N/A	0.07	6.9
	2	4	10	SAND	SC	**1**	**9**	30	0	116	53	26	0.5	N/A	0.23	3.5
	3	10	14	SAND	SP	**1**	**24**	34	0	125	63	71	2.4	N/A	0.35	9.4
	4	14	18.5	SAND	SP	**1**	**45**	38	0	125	63	141	4.5	N/A	0.45	17.9
	5	18.5	23.5	SAND	SM	**1**	**5**	29	0	111	49	13	0.3	N/A	0.53	2.0
	6	23.5	28.5	SAND	SC	**1**	**6**	30	0	113	50	17	0.3	N/A	0.59	2.5
	7	28.5	38.5	SAND	SC	**1**	**14**	31	0	122	60	41	0.8	N/A	0.68	5.5
	8	38.5	43.5	SAND	SC	**1**	**43**	38	0	130	68	137	2.4	N/A	0.75	17.4
	9	43.5	48.5	SAND	SC	**1**	**7**	30	0	114	52	21	0.4	N/A	0.77	3.0
	10	48.5	60	LIMESTONE	---	**1**	**25**	34	0	125	63	74	628.8	36.0	0.33	9.9

Soil Parameters for Lateral Analysis
JEA Church St 69kV Cable Relocation
Jacksonville, Florida
MAE Project No.: 0508-0003

Boring No.	Layer No.	Approximate Layer Thicknesses Depth (ft) from	to	Soil Type	USCS Soil Classification	Lateral Soil Model [1]	Average SPT N-Value, Corrected	f Angle of Internal Friction (degree)	c Cohesion (lb/ft^2)	γ_{sat} Saturated Unit Wt. (lb/ft^3)	γ_E Effective Unit Wt. (lb/ft^3)	k Lateral Subgrade Modulus (lb/in^3)	Deformation Modulus (ksi)	Rock Mass Rating (RMR)	Allowable Side Shear, Comp. ($kips/ft^2$) [2]	Allowable End Bearing, Comp. ($kips/ft^2$) [2]
B3-25	1	0	2	SAND	SP	**1**	**35**	37	0	125	63	107	3.5	N/A	0.04	13.9
	2	2	4	SAND	SP	**1**	**15**	32	0	119	56	45	1.5	N/A	0.11	6.0
	3	4	6	SAND	SM	**1**	**5**	29	0	111	49	13	0.3	N/A	0.17	2.0
	4	6	14	SAND	SP	**1**	**6**	30	0	108	45	17	0.5	N/A	0.28	2.5
	5	14	23.5	SAND	SP	**1**	**20**	33	0	125	62	60	2.0	N/A	0.46	7.9
	6	23.5	28.5	SAND	SM	**1**	**5**	29	0	111	49	13	0.3	N/A	0.57	2.0
	7	28.5	33.5	SAND	SC	**1**	**19**	33	0	128	66	56	1.1	N/A	0.64	7.4
	8	33.5	38.5	SAND	SC	**1**	**7**	30	0	114	52	21	0.4	N/A	0.69	3.0
	9	38.5	48.5	LIMESTONE	---	**1**	**11**	31	0	114	52	33	284.3	24.0	0.73	4.5
	10	48.5	53.5	LIMESTONE	---	**1**	**100**	40	0	125	63	250	1709.6	60.0	0.75	49.6
	11	53.5	58.5	LIMESTONE	---	**1**	**63**	40	0	125	63	184	1709.6	60.0	0.76	25.3
	12	58.5	60	SAND	SP	**1**	**52**	38	0	125	63	164	5.2	N/A	0.76	20.8

Notes:

1. Lateral Soil Model No.: 1. Sand (Reese or O'Neill), 5. Stiff Clay Below Water Table (Reese).

2. Factor of Safety for Allowable Side Shear = 2.0. Factor of Safety for End Bearing = 3.0.

Appendix E

Geotherm

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December 16, 2025

Meskel & Associates Engineering, LLC
3728 Philips Highway Ste 208
Jacksonville, FL 32207
Attn: Daisy Pena-Ross

Thermal Resistivity Report
JEA Church St 69kV Cable Relocation – Jacksonville, FL (Project No. 0508-0003)

The following is the report of thermal dryout characterization tests conducted on eleven (11) bulk samples of native soil from the referenced project sent to our laboratory.

Thermal Resistivity Tests: The samples were tested at the 'optimum' moisture content and at 85% of the modified Proctor dry density ***provided by Meskel & Associates***. The tests were conducted in accordance with the **IEEE standard 442-2017**. The results are tabulated below and the thermal dry out curves are presented in **Figures 1 to 4**.

Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Effort (%)	Description (Meskel & Assc.)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
				Wet	Dry		
B-1	5	95	Gray Fine Sand with Silt	46	137	12	98
B-1	8	95	Light Brown Clayey Fine Sand	40	109	13	111
B-2	5	95	Tan Clayey Fine Sand	40	126	13	103
B-2	8	95	Tan and Orange Fine Sand	44	132	12	101
B-2	20	95	Tan Fine Sand	45	150	14	96
B-2	30	95	Tan Clayey Fine Sand	42	128	15	101

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Sample ID, Description, Thermal Resistivity, Moisture Content and Density

Sample ID	Depth (ft)	Effort (%)	Description (Meskel & Assc.)	Thermal Resistivity (°C-cm/W)		Moisture Content (%)	Dry Density (lb/ft^3)
				Wet	Dry		
B-2	40	95	Dark Greenish to Gray Clayey Fine Sand	49	132	11	116
B-2	50	95	Light Brown Fine Sand with Clay	38	129	14	113
B-2	60	95	Tan Fine Sand	43	142	12	105
B-3	5	95	Light Brown Silty Fine Sand	36	124	12	112
B-3	8	95	Light Brown Fine Sand with Silt	36	134	13	107

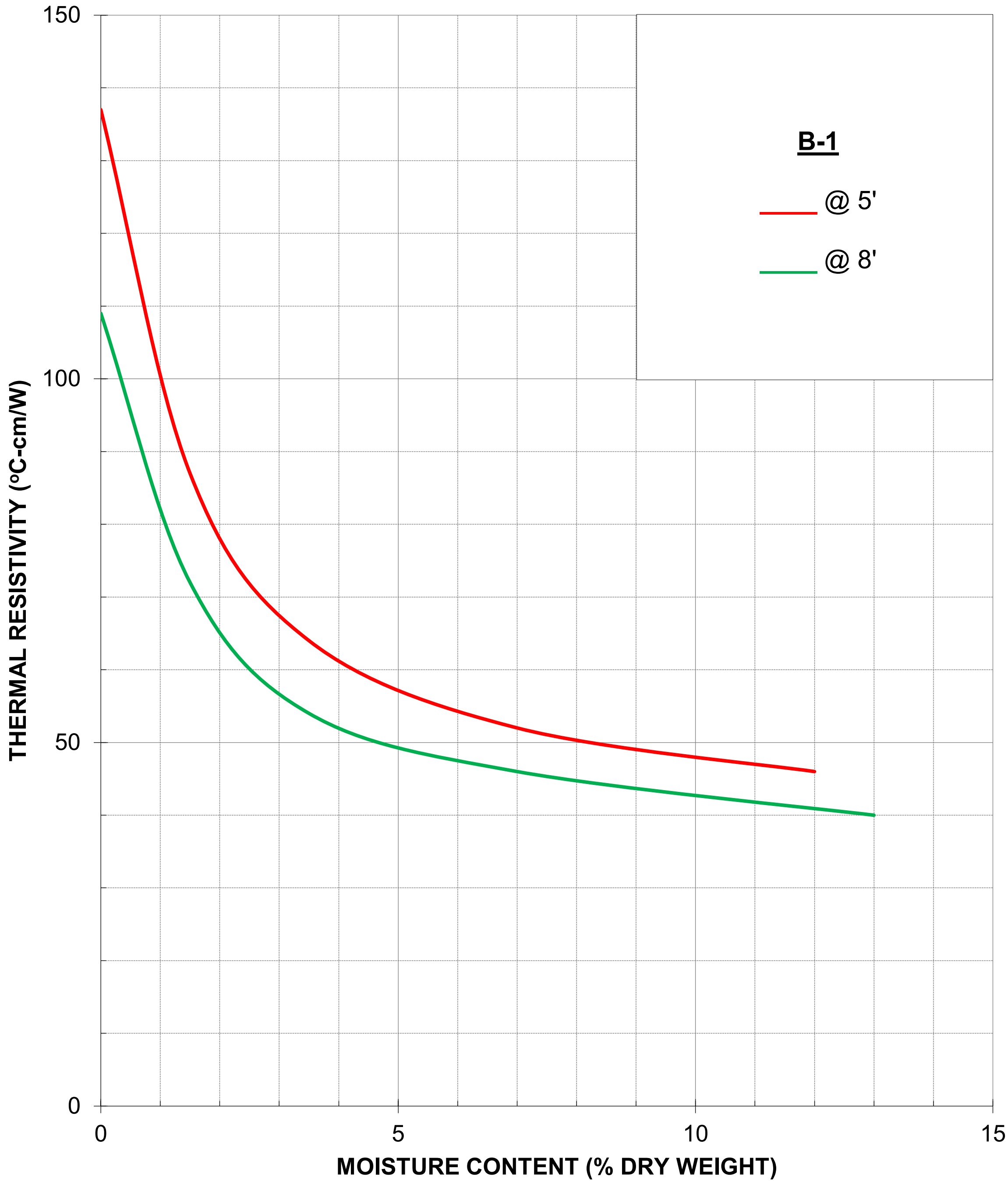
Comments: The thermal characteristic depicted in the dryout curves apply for the soils at their respective test dry density.

Please contact us if you have any questions or if we can be of further assistance.

Geotherm USA, LLC

Nimesh Patel

THERMAL DRYOUT CURVES



Meskel & Associates Engineering (Project No. 0508-0003)

JEA Church St 69kV Cable Relocation – Jacksonville, FL

Thermal Resistivity Report

December 2025 **Figure 1**

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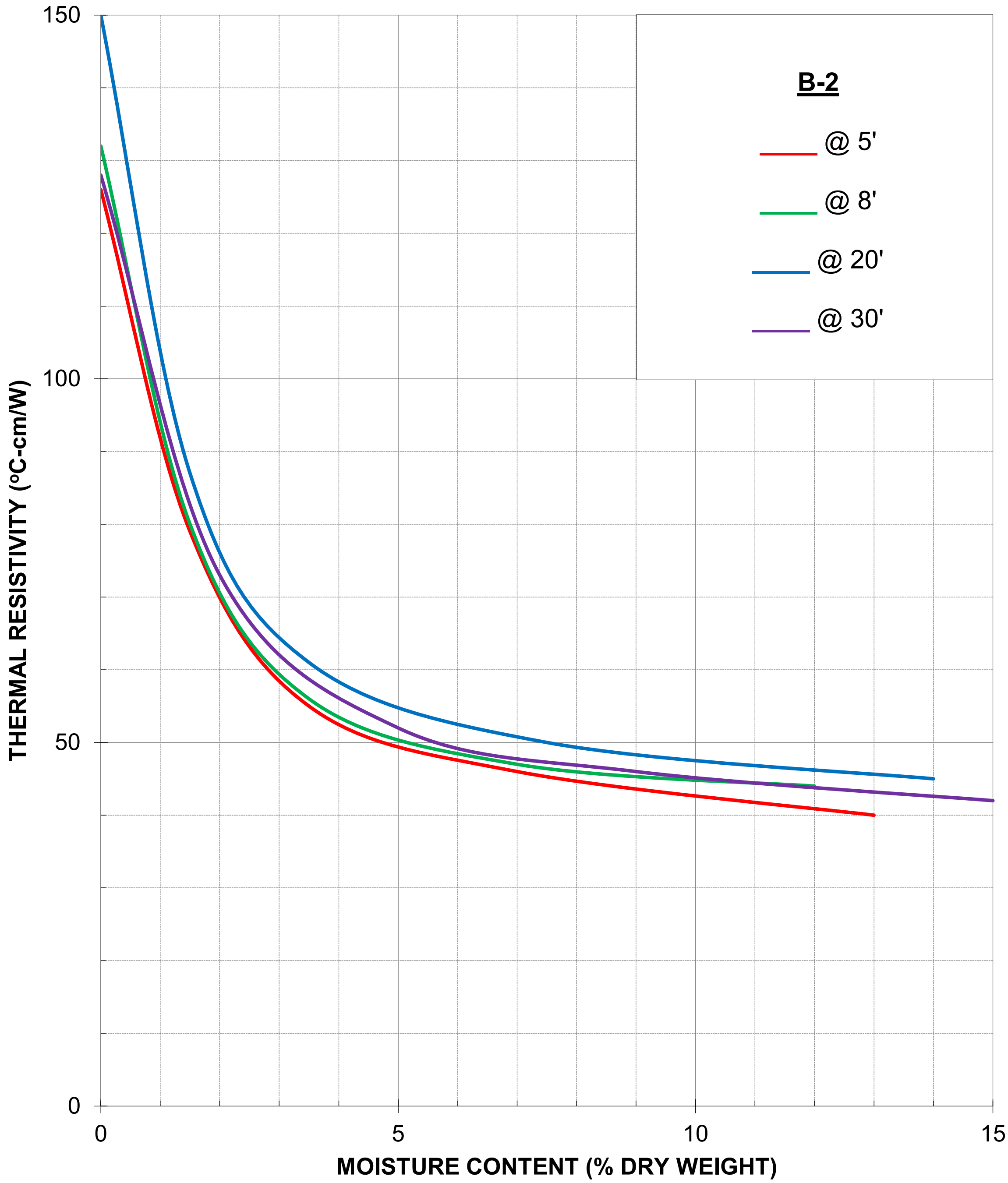
THERMAL DRYOUT CURVES



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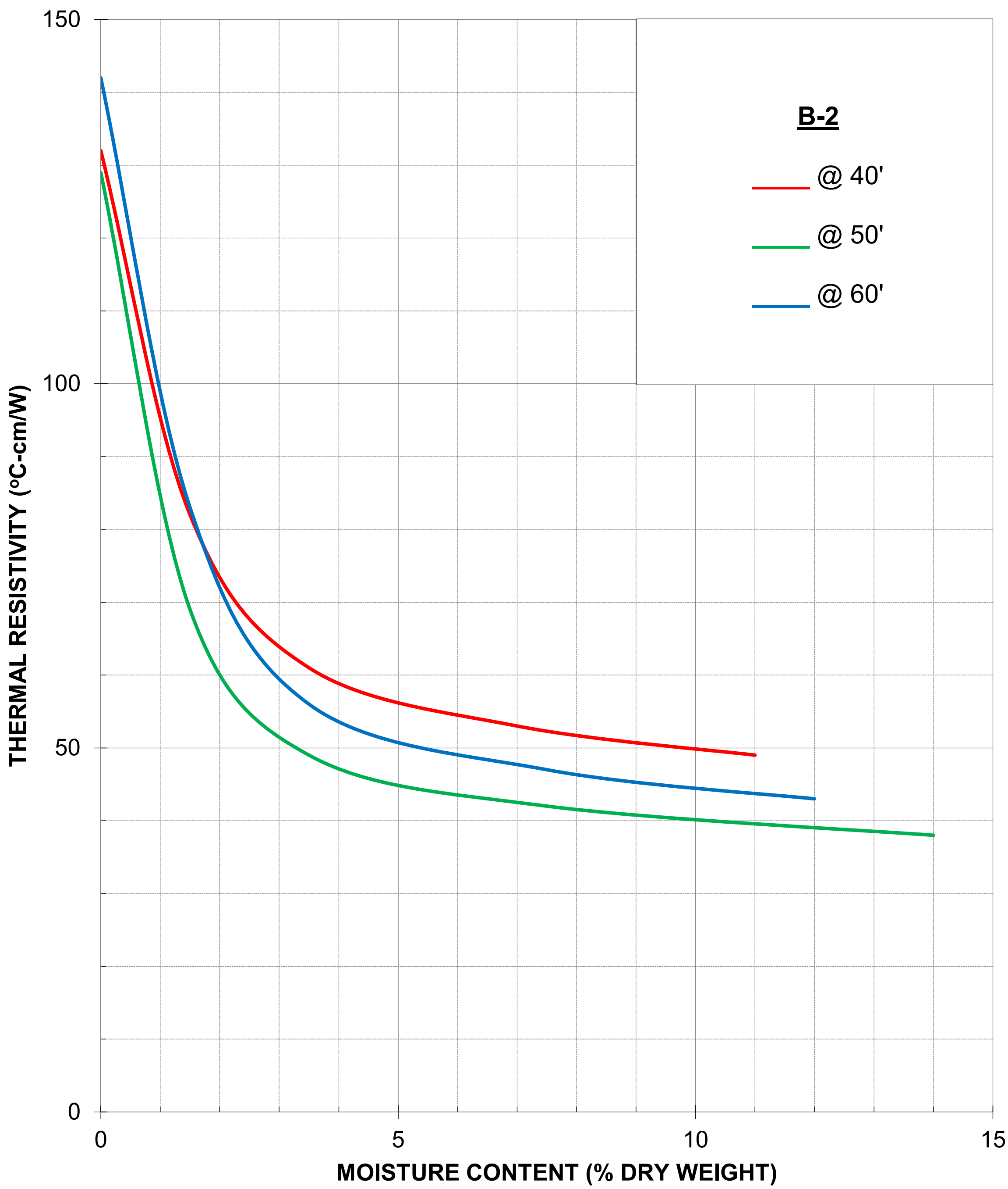
Thermal Resistivity Report

December 2025 **Figure 2**

THERMAL DRYOUT CURVES



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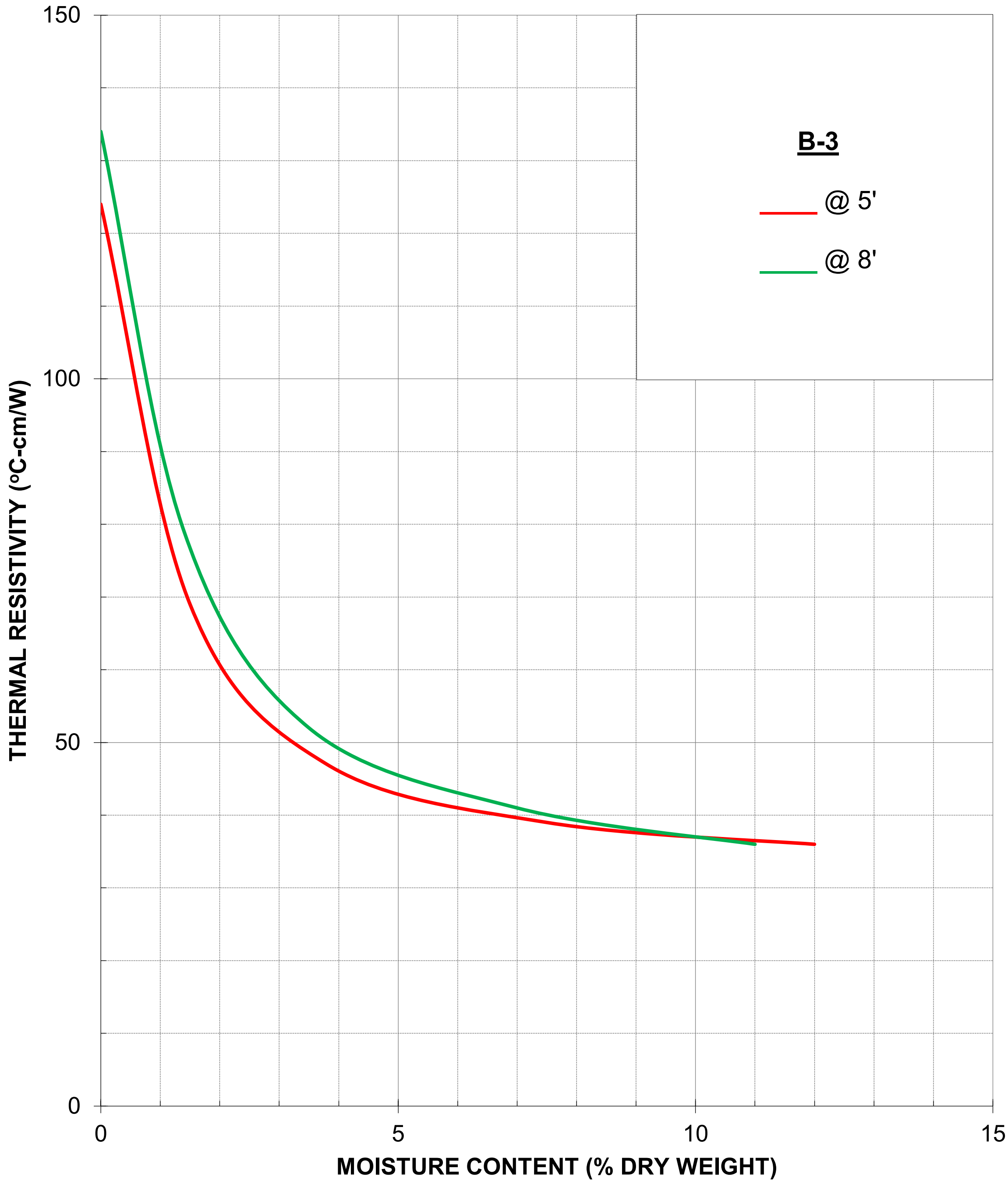
Thermal Resistivity Report

December 2025 **Figure 3**

Geotherm

THERMAL DRYOUT CURVES



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JEA Church St 69kV Cable Relocation – Jacksonville, FL

Thermal Resistivity Report

December 2025 **Figure 4**