JACKSON GEOTECHNICAL ENGINEERING, LLC
Consulting Geotechnical Engineers

REPORT OF GEOTECHNICAL EXPLORATION
BUSY BEE
ELLISVILLE, FLORIDA
JGE PROJECT NO. 24-581.1

This item has been digitally signed and sealed by Jeffrey S. Jackson, P.E. on the date adjacent to the seal.

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Prepared for:

Johnson & Johnson, Inc.
197 Southwest Waterford Court
Lake City, Florida 32025

Prepared by:

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January 3, 2025

JACKSON GEOTECHNICAL ENGINEERING, LLC
Consulting Geotechnical Engineers

January 3, 2025

Mr. Kevin Ryan
Johnson & Johnson, Inc.
197 Southwest Waterford Court
Lake City, Florida 32025

Report of Geotechnical Exploration and Engineering Services
Busy Bee
Ellisville, Florida
JGE Project No. 24-581.1

Dear Mr. Ryan:

As requested, Jackson Geotechnical Engineering has completed a geotechnical exploration for the subject project. The exploration was performed to evaluate the general subsurface conditions within the proposed construction areas, and to provide guidelines to facilitate foundation support, earthwork preparation, paving and drainage design.

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

Sincerely:
Jackson Geotechnical Engineering, LLC.

Jeffrey S Jackson

Digitally signed by Jeffrey S Jackson
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cc: Mr. Alex Acree, P.E.
Matthews Design Group

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1.0 PROJECT INFORMATION

1.1 Site Location and Description

The site for the subject project is located within the northwest quadrant of the intersection of Highway 41 and Southwest Howell Road, in Ellisville, Florida. The subject site is cleared, with a few scattered trees. Existing buildings, and adjacent parking lots, are present at the site. Interior driveways to access the existing buildings are present. Interstate 75 bounds the site to the north. Rural residential properties and farmland bounds the site to the west.

1.2 Project Description

Project information was provided to us during correspondence with Mr. Alex Acree, P.E., of Matthews Design Group. We were provided with the Post Development Plan and Master Site Plan (undated), prepared by Matthews DCCM. We were also provided with a Recommended Soils Boring Plan dated March 24, 2023, prepared by IMEG. The provided drawings show the property boundaries of the subject site, layout of the proposed construction, adjacent roadways, and requested boring locations within the proposed building.

We understand the proposed project will consist of constructing a Busy Bee facility at the subject site. In general, the proposed project will include a building, fueling canopies, underground storage tanks, stormwater ponds, parking and drive areas, and possibly retaining walls.

The proposed building will enclose one to two levels of floor space, and occupy approximately 68,000 square feet. Exterior walls of the proposed building will be constructed of concrete tilt-up panels. It is anticipated that interior columns will consist of structural steel. Conventional spread footings will be utilized for support of the proposed structure. Maximum wall and column foundation loads have been provided as 6 klf, and 125 kips, respectively. Floor slabs will be constructed of soil-supported concrete.

We anticipate the fueling canopies will be constructed of steel framing, supported by structural steel columns. The columns will bear on conventional spread footings. Foundations may have to be placed at relatively deep depths to resist uplift and over-turning forces.

Underground storage tanks will be utilized to store fuel. We anticipate the tanks will be manufactured of fiberglass. Tank diameters are not currently known; it is assumed they will be bear approximately 6 to 10 feet below finished grade. The tanks will be anchored to prevent potential hydrostatic uplift.

Retaining walls may be utilized to help achieve finished grades. We have assumed retaining walls will be cantilevered systems. We have also assumed the wall system will include a CMU block wall, or poured-in-place wall, supported on a spread footing.

Three stormwater ponds will be used to satisfy stormwater requirements of the governing authorities. Suitable structural fill soils obtained during pond excavation will be utilized for site development.

Parking and drive areas will be constructed of flexible asphaltic concrete underlain by base course and stabilize subgrade soils. Relatively isolated areas within the proposed facility may incorporate rigid concrete pavement.

We have assumed maximum fill heights will not exceed three to four feet above existing grade. Structural fill will be obtained during on-site pond excavation and/or import.

2.0 FIELD EXPLORATION

2.1 Soil Borings

Jackson Geotechnical Engineering previously conducted a geotechnical exploration and engineering study for the subject project. The results of our exploration and study were conveyed in our Report of Geotechnical Exploration dated May 12, 2023. The site layout has changed significantly since the previous report was issued. As a result, some borings that were originally meant for a particular construction component, may now be located within the area of a different construction component (e.g. a structural boring may now be located in the area of proposed pavement, etc.). In the tables below, we have grouped the borings together based on the various construction components. The locations of the borings provided in the table below were either within the noted construction component, or nearby. We have not presented some borings performed from the previous exploration if they were performed outside the limits of construction, have no (or little) application, or were replaced by other borings.

Construction Component	Borings
Building	B-19 through B-26
Fueling Canopies	B-7, B-8, B-14, B-15, B-26A, PB-2, A-12, A-14
Underground Storage Tanks	B-15, B-19, A-16
Stormwater Ponds	PB-5 through PB-8
Pavement	A-2, A-4 through A-6, A-9, A-11, A-12, A-14 through A-16, A-18 through A-29, PB-1, PB-2, B-2, B-4 through B-6, B-8, B-11 through B-15, B-17, B-18

1. Borings with prefix of B or PB were performed to a depth of 20 to 25 feet
2. Borings with prefix of A were performed to a depth of 6 feet

The borings were located by referencing existing site features, utilizing GPS methods, and should be considered accurate to the degree implied by the methods utilized. The SPT and auger borings were conducted in accordance with ASTM D 1586 and ASTM D 1452, respectively. The locations of the borings, and the subsurface conditions encountered at each boring location, are presented in Appendix A on the Boring Location Plan and Subsurface Profiles, respectively.

2.2 Relatively Undisturbed Soil Samples

Eight relatively undisturbed soil samples were obtained at the locations of Borings PB-5 through PB-8 for the purposes of permeability (hydraulic conductivity) testing. The soil samples were obtained using thin-walled tube sampling techniques (Shelby tube). The Shelby tubes were transported to our laboratory for permeability testing.

2.3 Piezometers

Piezometers were placed in the boreholes of Borings PB-5 through PB-8. The piezometers allowed the water table to be recorded at dates significantly after the completion of borings. The piezometers consisted of slotted PVC pipes. At the ground surface, bentonite pellets were utilized to seal the annular space between the borehole wall and the PVC pipe.

3.0 LABORATORY TESTING

3.1 Index Testing

Soil samples recovered during the field exploration were visually classified in accordance with ASTM D 2488. Additional testing to determine the fines content, natural moisture content, and Atterberg limits of selected soil samples was performed to better define the classification of soils encountered and to provide engineering characteristics of the soils. The results of the testing are presented on the Subsurface Profiles in Appendix A.

3.2 Permeability Testing

Permeability (hydraulic conductivity) tests were conducted on the undisturbed soil samples to estimate the coefficients of permeability of the appropriate soil layers. The coefficient of permeability is a measure of a soil's ability to transmit water under hydraulic loading conditions. It typically is a required input parameter for groundwater modeling, such as dry pond recoveries, background seepage, etc. The laboratory permeability test is typically conducted by placing the undisturbed soil sample in a permeameter, and while in the permeameter, the soil sample is subjected to differential hydraulic loading over a period of time. The volume of water that is transmitted through the soil sample is recorded, and along with the known hydraulic loading conditions, Darcy's law is utilized to calculate the coefficient of permeability. The coefficients of permeability are shown on the Subsurface Profiles at the depths of which the soil samples were obtained.

4.0 GENERAL SUBSURFACE CONDITIONS

4.1 General Soil Profile

The boring locations and general subsurface conditions that were encountered are presented on the Boring Location Plan and Subsurface Profiles. When reviewing these records, it should be understood the soil conditions may change significantly between the boring locations. The following discussion summarizes the general soil conditions encountered.

In general, a surficial layer of sandy soils consisting of fine sand (SP), fine sand with silt (SP-SM), and fine sand with clay (SP-SC) extend from the ground surface to depths ranging between approximately 1 and 8 feet. Below the surficial sandy soil overburden, the soils transition into clayey and silty layers consisting of clayey to very clayey fine sand (SC), silty fine sand (SM), and clay (CH). Relative densities indicated by the SPT testing show the surficial sandy soils exhibit a very loose to medium dense consistency. The underlying clayey to very clayey fine sand (SC), and silty fine sand (SM), typically exhibit loose to dense relative densities. The clay (CH) exhibits firm to very stiff relative density consistencies.

4.2 Groundwater Level

The groundwater level was measured at the boring locations, subsequent to boring completion, at depths varying between 6.7 to 19.3 feet below existing grade. As an exception to the above, the groundwater level was not encountered at the locations of borings B-19 through B-27, PB-7, PB-8, or at the locations of the auger borings within their 6-foot vertical reaches. The depth of the groundwater level encountered at each boring location is presented on the Subsurface Profiles.

The groundwater levels at the pond boring locations (PB-5 through PB-8) were measured on two different dates. The boring locations, dates, and measured groundwater levels are provided in the table below.

Boring Locations	Dates	Groundwater Level
PB-5	12/5/24	18.8
	12/18/24	Not Encountered
PB-6	12/5/24	18.2
	12/18/24	19.3
PB-7	12/5/24	Not Encountered
	12/18/24	Not Encountered
PB-8	12/5/24	Not Encountered
	12/18/24	Not Encountered

The groundwater table will fluctuate depending on seasonal variations, adjacent construction, surface water runoff, etc. Our estimate of the normal seasonal high groundwater level at each applicable boring location is presented on the Subsurface Profiles in Appendix A. Our estimates are based on the results of the soil borings, review of available published literature, and information provided for this study. Should rainfall intensity exceed normal quantities, or should other variables that affect the seasonal high groundwater level be altered, the groundwater profile at the site could change significantly.

5.0 BUILDING AND CANOPY STRUCTURE AREA RECOMMENDATIONS

5.1 General

The following recommendations are made based upon a review of the attached soil test data, our understanding of the proposed construction, and experience with similar projects and subsurface conditions. If the structural loads, construction locations, or grading information change from those discussed previously, we request the opportunity to review and possibly amend our recommendations with respect to those changes.

Please report to us any conditions encountered during construction that were not observed during the performance of the borings. We will review, and provide additional evaluation, as required.

Layers of clayey soils [clayey to very clayey fine sand (SC) and sandy clay (CH)] were present below the surficial layer of sandy soils [fine sand (SP), fine sand with silt (SP-SM), fine sand with clay (SP-SC)]. At some locations, the clayey soils were encountered at relatively shallow depths. To provide firm, uniform foundation bearing conditions, promote conditions favorable for achieving density compaction specifications in the footing excavations, and preclude the possible effects of moderate shrink/swell clayey soils, we recommend maintaining a vertical separation of at least 2 feet between the footing bearing depths and the top of the clayey soils. A vertical separation of at least 2 feet should also be maintained between the bottom of the building floor slabs and the top of the clayey soils (SC, CH). Within the recommended separation zone, only soils consisting of fine sand (SP), fine sand with clay (SP-SC), or fine sand with silt (SP-SM) meeting density specifications should be present/utilized. The recommended separation could be achieved by either (1) elevating site grade with compacted structural fill soil; or (2) over-excavating the clayey soils from below the footings and floor slabs, as required, and replacing with compacted structural backfill.

Clayey soils encountered at the location of the proposed building ranged in depth between 1 and 5 feet below the existing ground surface. In the general area of the proposed fueling canopies, the depths of the clayey soils ranged between 2.25 to 7 feet.

5.2 Structure Foundations

Based on the results of the subsurface exploration, we consider the subsurface conditions at the site favorable for support of the proposed structures when constructed on properly designed shallow foundation systems. Provided the soils are prepared in accordance with the Site Preparation Section of this report, the following parameters may be used for foundation design.

5.2.1 Bearing Pressure

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

5.2.1 Modulus of Subgrade Reaction

The Modulus of Subgrade Reaction is a conceptual relationship defined as the soil pressure divided by soil deflection. It provides a linear relationship between soil resistance and pressure exerted, allowing a simplified method for calculating deflection. It is commonly used for structural design of floor slabs. For floor slab design, we estimate the Modulus of Subgrade Reaction is approximately 125 pci.

5.2.2 Foundation Size

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

5.2.3 Bearing Depth

The exterior foundations should bear at a depth of at least 12 inches below the exterior final grades and the interior footings should bear at a depth of at least 12 inches below the finish floor elevation to provide confinement to the bearing level soils. We recommend stormwater and surface water be diverted away from the building exterior, both during and after construction, to reduce the possibility of erosion adjacent to the exterior footings.

5.2.4 Bearing Material

The foundations may bear on either the compacted suitable in-place natural soils or compacted structural fill. The bearing level soils, after compaction, should exhibit densities of at least 95 percent of the maximum dry density as determined by ASTM D 1557 (Modified Proctor), to the depth described subsequently in the Site Preparation section of the report. In addition to compaction, the bearing soils must exhibit stability and be free of “pumping” conditions.

5.2.5 Settlement Estimates

Post-construction settlement of the structures will be influenced by several interrelated factors, such as (1) subsurface stratification and strength/compressibility characteristics of the bearing soils; (2) footing size, bearing level, applied loads, and resulting bearing pressures beneath the foundations; (3) site preparation and earthwork construction techniques used by the contractor, and (4) external factors, including but not limited to vibration from offsite sources and groundwater fluctuations beyond those normally anticipated for the naturally-occurring site and soil conditions which are present.

Our settlement estimates for the structures are based upon the use of successful adherence to the site preparation recommendations presented later in this report. Any deviation from these recommendations could result in an increase in the estimated post-construction settlement of the structures.

Implementing the recommended earthwork preparation procedures, we expect a significant portion of settlement to be elastic in nature. This settlement is expected to occur relatively quickly, upon application of the loads, during and immediately following construction. Using the recommended maximum bearing pressure, the provided and assumed maximum structural loads, and the field and laboratory test data which we have correlated to the strength and compressibility characteristics of the subsurface soils, we estimate the total settlements of the structures to be approximately one inch or less.

Differential settlement results from differences in applied bearing pressures and the variations in the compressibility characteristics of the subsurface soils. Based on the subsurface conditions as determined by the borings, and the recommended earthwork preparation, it is anticipated that differential settlements will be within tolerable limits.

5.3 Site Preparation for Shallow Foundations

We recommend the following site preparation guidelines for the building and canopy areas:

1. Prior to construction, the location of existing underground utility lines within the construction areas 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 the overlying structures.

2. Implement temporary groundwater control measures, as required. The groundwater should be maintained at least two feet below the depth of excavation required and two feet below compacted surfaces. Based on the groundwater levels encountered at the time of this exploration, temporary groundwater control measures may not be required. However, the groundwater can become elevated during periods of consistent rainfall, which could necessitate groundwater control measures. Temporary groundwater control measures should be the responsibility of the contractor.

3. Strip the proposed construction limits of all grass, roots, topsoil, and other deleterious materials from within, and extending at least 5 feet beyond, the perimeter of the proposed structures. Expect initial clearing and grubbing to average depths of approximately 6 to 12 inches. During stripping operations, roots with a diameter greater than 0.5 inches, stumps,

and roots in a concentrated state, should be completely removed. All demolition debris resulting from the removal of existing structures, pavement, etc. should be disposed of off-site.

As mentioned in Section 5.1 above, maintain a vertical separation of at least 2 feet between the foundation bearing depths and the top of the clayey soils [clayey to very clayey fine sand (SC) and clay (CH)]. Also maintain a vertical separation of at least 2 feet between the floor slab bearing depths and the top of the clayey soils. If elevating fill does not provide the minimum separation, then over-excavate the clayey soils and replace with compacted structural backfill to achieve the recommended vertical clearance. The over-excavation of clayey soils is again discussed in Item 8 below.

4. Compact the exposed surface of sandy soils using a vibratory drum roller having a minimum static, at-drum weight of 7 tons and a drum diameter of at least 5 feet. It is recommended that repeated passes of the roller be made in one direction, followed by repeated passes of the roller in a direction perpendicular to the initial passes. The upper two feet of soils below the exposed surface (after stripping, grubbing, and excavation) should be improved to achieve a minimum compaction requirement of 95% of the Modified Proctor Test (ASTM D 1557). We recommend the soils, at the time of compaction, exhibit moisture contents within 2 percent of the optimum moisture content as determined by the Modified Proctor Test (ASTM D 1557).

   Should the 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 which are then compacted, or (2) the excess moisture content within the disturbed soils 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 be documented with photographs and survey (if deemed necessary). Compaction should cease if deemed detrimental to adjacent structures and Jackson Geotechnical Engineering should be contacted immediately. It is recommended the vibratory roller remain a minimum of 75 feet from existing structures. Within this zone, use of a vibratory roller operating in the static mode (vibration turned off) is recommended.

5. Test the compacted surface within the building areas for density at a minimum of one location per 2,500 square feet. Within the proposed canopy areas, test the compacted surface for density at a minimum frequency of one test per 2,500 square feet (minimum of five locations per structure).

6. Place structural fill in loose lifts not exceeding a thickness of 12 inches and compact until finished subgrade is achieved. Structural fill and backfill is typically defined as 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. 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 foot of the compacted structural fill.

7. Within the building area, perform density tests within each lift of fill at a minimum of one location per 2,500 square feet. Within the proposed canopy areas, perform density tests within each lift of fill at a minimum frequency of one test per 2,500 square feet (minimum of five locations per structure).

8. Excavate, compact and test footing excavations for density to a depth of two below bearing level. We recommend that you test one out of every four column footings and perform one test per every 100 linear feet of wall footing. Compaction operations in confined areas, such as footing excavations, can best be performed with a lightweight vibratory sled or other hand-held compaction equipment.

   As mentioned above, clayey soils located within a vertical distance of two feet below the foundation bearing depths should be excavated. The excavated clayey soils should be replaced with compacted structural backfill meeting the density specifications of Item 6 above.

6.0 PAVEMENT RECOMMENDATIONS

6.1 General

We understand the subject project will utilize flexible asphaltic concrete pavement, and possibly isolated areas of rigid, unreinforced concrete pavement. In the following sections, we have presented our recommendations to guide pavement design and site preparation.

Relatively shallow layers (depth of two feet or less) of clayey soils [clayey fine sand to very clayey fine sand (SC) and clay (CH)] were encountered at the locations of Borings A-4, A-18, A-20, A-26, A-27, and B-11. To provide firm subgrade support for the pavement section, we recommend a vertical separation of at least two feet be maintained between the bottom of the base course and the clayey soils. If possible, this separation should be achieved by elevating the site with structural fill. Alternatively, the clayey soils, if present within two feet of the bottom of base course, could be over-excavated and replaced with compacted structural backfill.

6.2 Pavement Section Recommendations

Our recommendations for pavement sections are presented below. Detailed traffic loading conditions were not available; therefore, we have provided pavement sections which can accommodate loading conditions typical of the subject construction over a design life of 20 years. The light duty pavement sections are based on 500,000 Equivalent Single Axle Loads (ESALs) of 18 kips. The heavy-duty and extra heavy-duty pavement sections are based on 6,000,000 and 20,000,000 ESALs, respectively. If provided with detailed traffic loading conditions, Jackson Geotechnical Engineering can conduct a formal pavement design, and provide refined pavement sections.

Pavement Section	Asphalt/Concrete(1) Thickness (in)	Base Course(2) Thickness (in)	Stabilized(3) Subgrade (in)
Light Duty Asphalt	1.5	6.0	12
Heavy Duty Asphalt	3.0	8.0	12
Extra Heavy Duty Asphalt	4.0	12.0	12
Light Duty Concrete	5.5	4.0	NA
Heavy Duty Concrete	8.5	4.0	NA
Extra Heavy Duty Concrete	10.5	6.0	NA

1) Flexible pavement should consist of SP 9.5 and/or SP 12.5, or equivalent. The use of SP 12.5 is recommended for heavy-duty applications. Rigid pavement should consist of FDOT Class I with a minimum of 3,000 psi concrete in accordance with Section 346 of the FDOT Standard Specifications for Road and Bridge Construction.
2) Base course below flexible asphalt should consist of limerock or crushed concrete. Base course below rigid concrete pavement should consist of crushed concrete. Limerock and crushed concrete should exhibit an LBR of at least 100, and 130, respectively. Limerock and crushed concrete base course materials and gradations should conform to FDOT Standard Specifications for Road and Bridge Construction Sections 911 and 204, respectively.
3) Subgrade should exhibit an LBR of at least 40.

6.3 Site Preparation for Pavements

We recommend the following site preparation guidelines for pavement construction:

1. Strip the proposed construction limits of all grass, roots, topsoil and other deleterious materials from within, and extending at least 3 feet beyond, the proposed pavement limits. Expect initial clearing and grubbing to average depths of approximately 6 to 12 inches.

   As mentioned above, a vertical separation of at least 2 feet should be maintained between the bottom of base course and the top of the clayey soils [clayey to very clayey fine sand (SC) and clay (CH)]. It is preferred the separation be achieved with elevating structural fill. Alternatively, clayey soils, where present, could be over-excavated to a depth of at least two feet below the proposed base course in order to maintain the recommended separation.

2. Compact the exposed surface of sandy soils with a vibratory drum roller until densities of at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557) are achieved within the upper one foot below the exposed surface with the exception that densities of at least 98 percent should be obtained in the upper 12 inches below base course. We recommend the compacted soils exhibit moisture contents within 2 percent of the optimum moisture content as determined by the Modified Proctor Test (ASTM D 1557).

   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 be documented with photographs and survey (if deemed necessary). Compaction should cease if deemed detrimental to adjacent structures and Jackson Geotechnical Engineering should be contacted immediately. It is recommended the vibratory roller remain a minimum of 75 feet from existing structures. Within this zone, use of a vibratory roller operating in the static mode (vibration turned off) is recommended.

   Should the 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 which are then compacted, or (2) the excess moisture content within the disturbed soils allowed to dissipate before recompacting.

3. Test the compacted surface for density at a frequency of not less than one test per 10,000 square feet of pavement area.

4. Place structural fill in loose lifts not exceeding 12 inches and compact until finished subgrade is achieved. Structural fill and backfill is typically defined as 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. 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 foot of the compacted structural fill, with the exception that densities of at least 98 percent should be obtained in the upper 12 inches below base course.

5. Perform density tests within each lift of fill at a frequency of not less than one test per 10,000 square feet of pavement area.

6. Place and compact base course until densities of at least 100 percent of the modified Proctor maximum dry density are achieved.

7. Perform density tests within the base course at a frequency of not less than one test per 10,000 square feet of pavement area.

6.4 Additional Pavement Considerations

6.4.1 Asphaltic Concrete Pavement

Asphaltic concrete mixes should be a current FDOT approved design of the materials actually used. Samples of the materials delivered to the project should be tested to verify that the aggregate gradation and asphalt content satisfies the mix design requirements.

After placement and field compaction, core the wearing surface to evaluate material thickness and to perform laboratory densities. Obtain cores at frequencies of at least one core per 5,000 square feet of placed pavement, or a minimum of two cores per day of production.

6.4.2 Rigid Concrete Pavement

We recommend a detailed jointing plan be prepared in order to limit unnecessary cracking of the concrete. The joints should form square patterns, when possible. Rectangular jointing patterns, when utilized, should not exceed a length to width ratio of 1.3. As a general rule of thumb, the spacing between sawcut joints should not exceed 24 times the thickness of the pavement (e.g. a 6-inch thick pavement would require joints spacings not exceeding 6 x 24 = 144 inches, or 12 feet). The depth of the joint when sawcut should be one-fourth to one-third the thickness of the pavement. Joint spacings exceeding 15 feet, regardless of the pavement thickness, are not recommended. The joints should be sawcut as soon as the concrete has gained enough strength to support personnel and equipment.

It is recommended the joints be completely sealed in order to preclude surface water infiltration into the base course and subgrade. Infiltration of water into the base and subgrade will result in a loss of soil strength, and subsequent pavement deterioration. It is recommended the sealant be silicone-based, manufactured specifically for the subject purpose. The condition of the sealant should be periodically observed during the life of the pavement, and replaced as necessary. The pavement should be periodically inspected for cracking. All cracks should be sealed with the sealant mentioned above. It is imperative that surface water not be allowed to enter into the ground through the pavement, especially for inverted crown pavement sections.

6.4.3 Groundwater Separation

Groundwater, if not maintained below the base course an adequate distance, can result in weakened subgrade and base course soils, and therefore a greatly reduced pavement life. It is recommended the seasonal high groundwater level be maintained at least 18 inches below base course. If the recommended vertical separation cannot be achieved with the proposed finished grades, underdrains can be considered to maintain the groundwater level at the recommended depths.

7.0 UNDERGROUND STORAGE TANKS

7.1 General

We understand underground tanks will be utilized for fuel storage. We anticipate the fuel tanks will be constructed of fiberglass. In the following sections, we have presented our recommendations to guide earthwork preparation for the support of the storage tanks.

Borings B-15, B-19, A-16 were performed within the general area of the proposed underground storage tanks. These borings encountered surficial fine sand within the upper approximate 2.5 to 5 feet. Based on the limited depth of sand, it is expected the tanks will bear in clayey soils [clayey to very clayey fine sand (SC)]. To provide firm and uniform bearing conditions, promote conditions favorable for achieving density compaction specifications in the tank excavations, and preclude the possible effects of moderate shrink/swell clayey soils, we recommend maintaining a vertical separation of at least 2 feet between the bottom of the tanks and the top of the clayey soils. Clayey soils located within the recommended vertical separation zone should be over-excavated and replaced with compacted structural backfill soils. Treatment of clayey soils is again discussed below.

7.2 Site Preparation for Storage Tanks

We recommend the following site preparation guidelines for tank support:

1. Strip the proposed construction limits of all grass, roots, topsoil and other deleterious materials from within, and extending at least 3 feet beyond, the proposed tank area construction limits. Expect initial clearing and grubbing to average depths of approximately 6 to 12 inches. Segregate the stripped organic soils from surficial sandy soils [fine sand (SP) and fine sand with silt (SP-SM]. The segregated sandy soils can be subsequently utilized as structural fill soils. Perform the tank excavation subsequent to stripping.

   As mentioned above, a vertical separation of at least 2 feet should be maintained between the bottom of the tank bearing depths and the top of the clayey soils [clayey to very clayey fine sand (SC) and clay (CH)]. Excavate, as necessary, to achieve the recommended separation.

   Implement temporary groundwater control measures, as necessary. The groundwater control measures should provide relatively dry conditions, conducive to achieving target densities of the backfill soils. Temporary groundwater control measures should be the responsibility of the contractor.

2. Place initial lifts of structural fill on the surface of the excavation and compact. Structural fill and backfill is typically defined as 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. Tracked equipment, or relatively lightweight drum rollers, could be utilized to compact the initial backfill, and avoid disturbance to the underlying clayey soils. Utilize the tracked equipment, lightweight roller, etc., until at least two to three feet of backfill has been placed on the exposed surface. Afterward, vibratory rollers can be utilized until finished subgrade is achieved. Compaction should continue until densities of at least 95 percent of the modified Proctor maximum dry density (ASTM D 1557) are achieved within the structural backfill. We recommend the compacted soils exhibit moisture contents within 2 percent of the optimum moisture content as determined by the Modified Proctor Test (ASTM D 1557).

   To avoid excessive lateral pressure on the tanks during compaction operations, we recommend that densities not exceed 98 percent of the modified Proctor maximum dry density within a lateral distance of 5 feet from the tanks. The use of relatively heavy equipment should be used with caution while within 5 feet of the tanks to avoid excessive lateral earth pressures.

   Should the 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 which are then compacted, or (2) the excess moisture content within the disturbed soils allowed to dissipate before recompacting.

3. Test the compacted backfill to verify target densities were achieved. Density tests should be performed at a minimum of two locations per tank, and at multiple depths, in order to verify target densities were achieved throughout the entirety of the backfill.

8.0 RETAINING WALL RECOMMENDATIONS

8.1 General

As mentioned previously, retaining walls may be utilized in isolated areas of the site. Retaining wall design generally requires the input of soil parameters. The soil parameters are utilized by the designer to estimate the forces which will act against the retaining wall. The designer will need to verify that reasonable global factors of safety are calculated for sliding, overturning, and slope stability, as well as localized, internal factors of safety.

8.2 Design Considerations

Retaining walls will be subjected to lateral pressures as a result of adjacent earth and groundwater. The table below provides the soil unit weights, at-rest, active, and passive earth pressure coefficients to facilitate the design of the retaining wall. We recommend a factor of safety of at least 1.5 be utilized for wall design.

Soil Strata	Dry Unit Weight (pcf)	Buoyant Unit Weight (pcf)	K_o	K_a	K_p	C (psf)	Φ (deg.)
Fill (behind the wall)	105	58	0.49	0.320	3.128	0	31
In-Situ Sandy Soils	105	58	0.50	0.333	3.00	0	30

K_o – coefficient of at-rest lateral earth pressure
K_a – coefficient of active lateral earth pressure
K_p – coefficient of passive lateral earth pressure
C – cohesion
Φ – angle of internal friction

The parameters noted above can be utilized by the wall designer to estimate the total lateral pressures exerted on the wall by the soil. The designer should also include the effects of groundwater by including hydrostatic pressures exerted on the walls. A french drain (or other suitable drain system) can be placed behind the wall near its foundation in order to lower the groundwater and reduce the pressure exerted on the wall. Clean backfill, with adequate permeability in its compacted state, should be utilized within five feet of the wall, therefore encapsulating the french drain system. The designer should specify on the plans the required permeability of the backfill at the compacted density level. The design of the drain system should maintain the water level at depths that would provide an adequate factor of safety against wall failure. The backfill should consist of inorganic fine sand (SP), which by definition has less than five percent silt and/or clay. The drain system should adequately collect the groundwater and positively convey it away from the wall. It is recommended clean-outs be utilized so periodic maintenance of the drainage pipe can be conducted.

In addition to the wall designer verifying an adequate factor of safety against localized failure, he should also verify the design provides an adequate factor of safety against global failure. Global analysis would include showing an adequate factor of safety against failure regarding wall sliding, wall rotation, and slope stability.

8.3 Site Preparation for Retaining Wall

We recommend the following site preparation guidelines:

1. Implement temporary groundwater control measures, as required. The groundwater should be maintained at least two feet below the depth of excavation required and two feet below compacted surfaces. Temporary groundwater control measures should be the responsibility of the contractor.

2. Excavate for construction of the retaining wall footings. Subsequent to performing the footing excavation, compact the exposed surface of sandy soils until the soil within the upper one foot exhibits at least 95 percent of the soil's modified Proctor maximum dry density (ASTM D1557). Compaction operations in confined areas, such as footing excavations, can best be performed with a lightweight vibratory sled or other hand-held compaction equipment.

   To provide uniform and firm foundation bearing conditions, excavate clayey soils [clayey to very clayey fine sand (SC) and clay (CH)] if present within a depth of two feet below the wall foundation bearing level. Replace excavated clayey soils with compacted structural backfill. Structural fill and backfill is typically defined as 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.

3. Perform density tests to confirm target densities were achieved within the compacted soils. A minimum of one density test should be performed for each 100 linear feet of wall.

4. Remove vegetation and topsoil from behind the retaining wall, in preparation for fill placement. Subsequently, compact the exposed surface with tracked equipment, or a drum roller operating in static mode. Compaction should continue until the upper one foot of soils below the exposed surface are improved to achieve a minimum compaction requirement of 95% of the Modified Proctor Test (ASTM D 1557). As an exception, we recommend that densities not exceed 98 percent within a lateral distance of 5 feet from the

wall in order to limit soil pressures exerted on the wall. Maintain heavy equipment a minimum of 5 feet from the wall to limit lateral earth pressures exerted on the wall. We recommend the soils, at the time of compaction, exhibit moisture contents within 2 percent of the optimum moisture content as determined by the Modified Proctor Test (ASTM D 1557).

Should the 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 which are then compacted, or (2) the excess moisture content within the disturbed soils allowed to dissipate before recompacting.

5. Test the compacted surface for density at a minimum of one location for each 2,500 square feet.

6. Place structural fill behind the wall in loose lifts not exceeding a thickness of 12 inches and compact until finished grade is achieved. Structural fill and backfill is typically defined as 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 (if a drain is utilized behind the wall, fill having a maximum fines content of 5 percent should be utilized within at least 5 feet of the wall in order to provide relatively permeable conditions encapsulating the drain pipe). 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 foot of the compacted structural fill. As an exception, we recommend the densities not exceed 98 percent within a lateral distance of 5 feet from the wall in order to limit soil pressures exerted on the wall. Maintain heavy equipment a minimum of 5 feet from the wall to limit lateral earth pressures exerted on the wall.

7. Perform density tests within each lift of fill at a minimum of one location for each 2,500 square feet.

9.0 STORMWATER POND RECOMMENDATIONS

9.1 General

The drainage system includes three stormwater ponds. It is assumed the ponds will be designed according to dry retention/detention criteria. It is anticipated that suitable soils excavated during pond construction will be utilized for site development.

9.2 Detention Systems

The borings in the pond areas were intended, in part, to provide an indication of the suitability of the soils for use as structural fill and backfill. The fine sand (SP), fine sand with silt (SP-SM), and fine sand with clay (SP-SC), as encountered at Borings PB-5 through PB-8, are suitable for use as structural fill and backfill material. The fine sand typically exhibits higher permeability than the fine sand with silt and fine sand with clay, and therefore, is more desirable for use in areas requiring substantial infiltration/percolation potential.

Because the fine sand with silt and fine sand with clay soils inherently retain moisture, strict moisture control will be required to avoid soil instability (pumping) during placement and compaction operations. Density requirements typical of structural soils are very difficult to achieve with silty fine sand (SM), clayey to very clayey fine sand (SC), and clay (CH) due to their plasticity characteristics and extreme nature to retain moisture. Therefore, we do not recommend clayey to very clayey fine sand (SC), silty fine sand (SM), or clay (CH) for use as structural fill materials.

The soils in the proposed pond areas that are below the groundwater level will have moisture contents in excess of the Modified Proctor optimum moisture content and will require stockpiling or spreading to bring the moisture content to within 2 percent of the optimum moisture content corresponding to the required degree of compaction.

9.3 Retention Systems

The drainage system may include dry retention ponds. Retention systems retain the necessary minimum amount of stormwater runoff (treatment volume) during the storm event. The volume retained is treated by infiltration into the ground. The rate of infiltration into the ground is primarily affected by permeability of the soil, vertical height of stored stormwater (hydraulic loading), depth of the aquifer, soil porosity, and vertical distance between the stored runoff and the water table.

Our tested and estimated soil and groundwater parameters that can be utilized for recovery analysis are provided on the Subsurface Profiles in Appendix A. These include the tested permeability values, estimated seasonal-high groundwater levels, estimated effective porosity, and the depth of aquifer observed.

The depth of aquifer is dependent on the depth of the clayey soils observed at the locations of Borings PB-5 through PB-8. Based on our experience with the subject site, the depth of the clayey soils can vary significantly over relatively short lateral distances.

10.0 ADDITIONAL CONSIDERATIONS

Isolated areas within the site are characterized by near-surface clayey soils. Additionally, it is anticipated that excavation into the clayey soils will be required at various locations. The clayey soils typically have a relatively large potential for soil instability (pumping) during earthwork operations, particularly during wetter climatic conditions. Density of soils generally cannot be achieved with instable soils, and significant rutting of sites can occur, making further earthwork operations difficult.

In order to avoid soil instability and pumping conditions, clayey soils should not be compacted directly from the exposed surface. Additionally, rubber-tired equipment should not be operated directly on top of clayey soils exhibiting elevated moisture contents. When plastic, clayey soils are at, or near the surface (e.g. after foundation or tank excavations), a loose layer of structural fill having a thickness of approximately 18 inches should be placed on the exposed surface of the clayey soils and compacted with tracked, or manual, equipment. Provided soil instability is not noted, and density is achieved, subsequent lifts of fill and backfill should be placed and compacted. Depending on the results of the initial compaction operations, the thickness of the lift placed on the exposed surface of the clayey soils may need to be adjusted.

11.0 LIMITATIONS

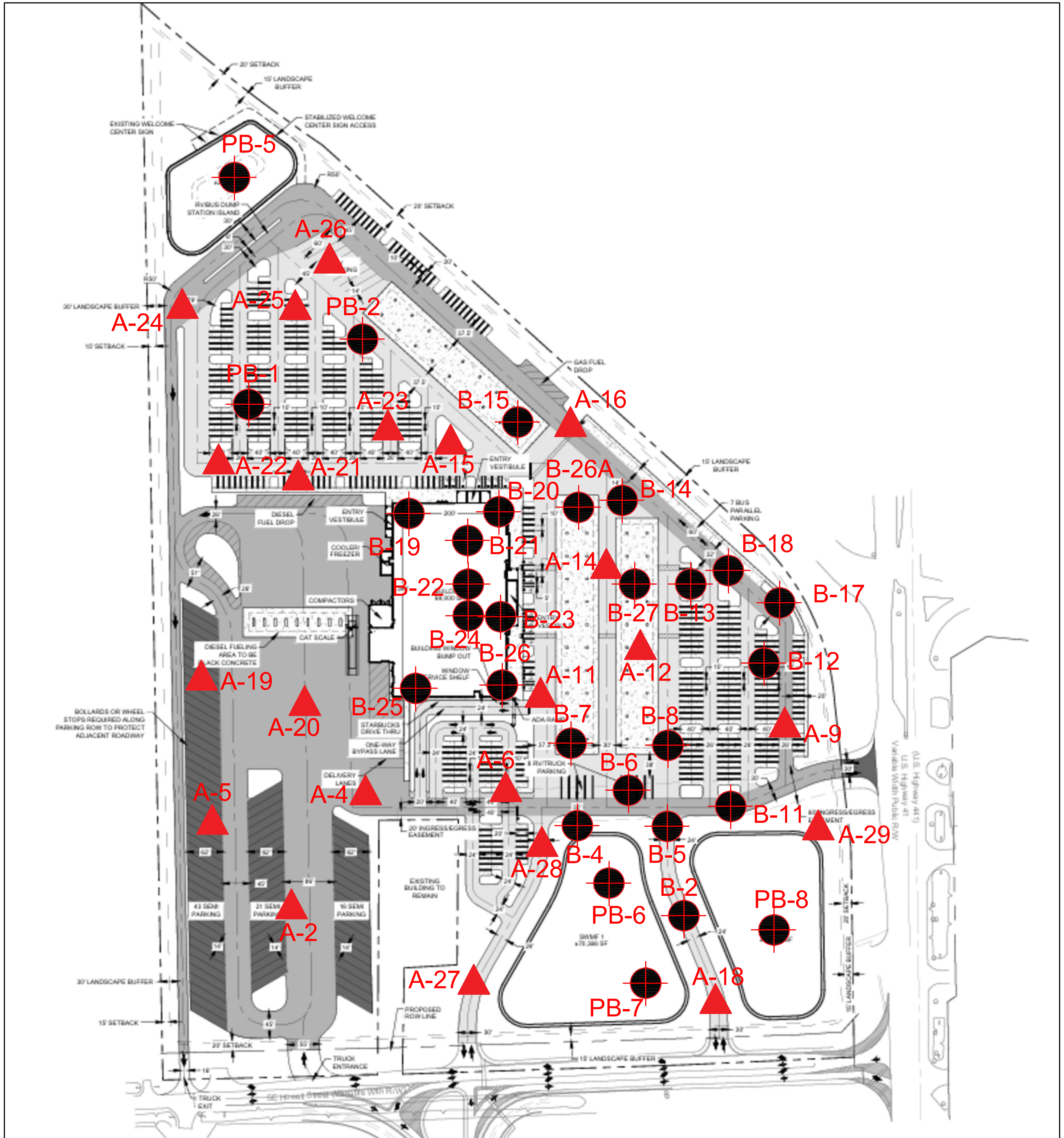
We have conducted the geotechnical engineering in accordance with principles and practices normally accepted in the geotechnical engineering profession. Our analysis and recommendations are dependent on the information provided to us. Jackson Geotechnical Engineering is not responsible for independent conclusions or interpretations based on the information presented in this report.

The recommendations provided in this report are specific to the proposed construction. Significant changes to the site design and construction, as described in this report, would nullify the provided recommendations.

APPENDIX A

BORING LOCATION PLAN

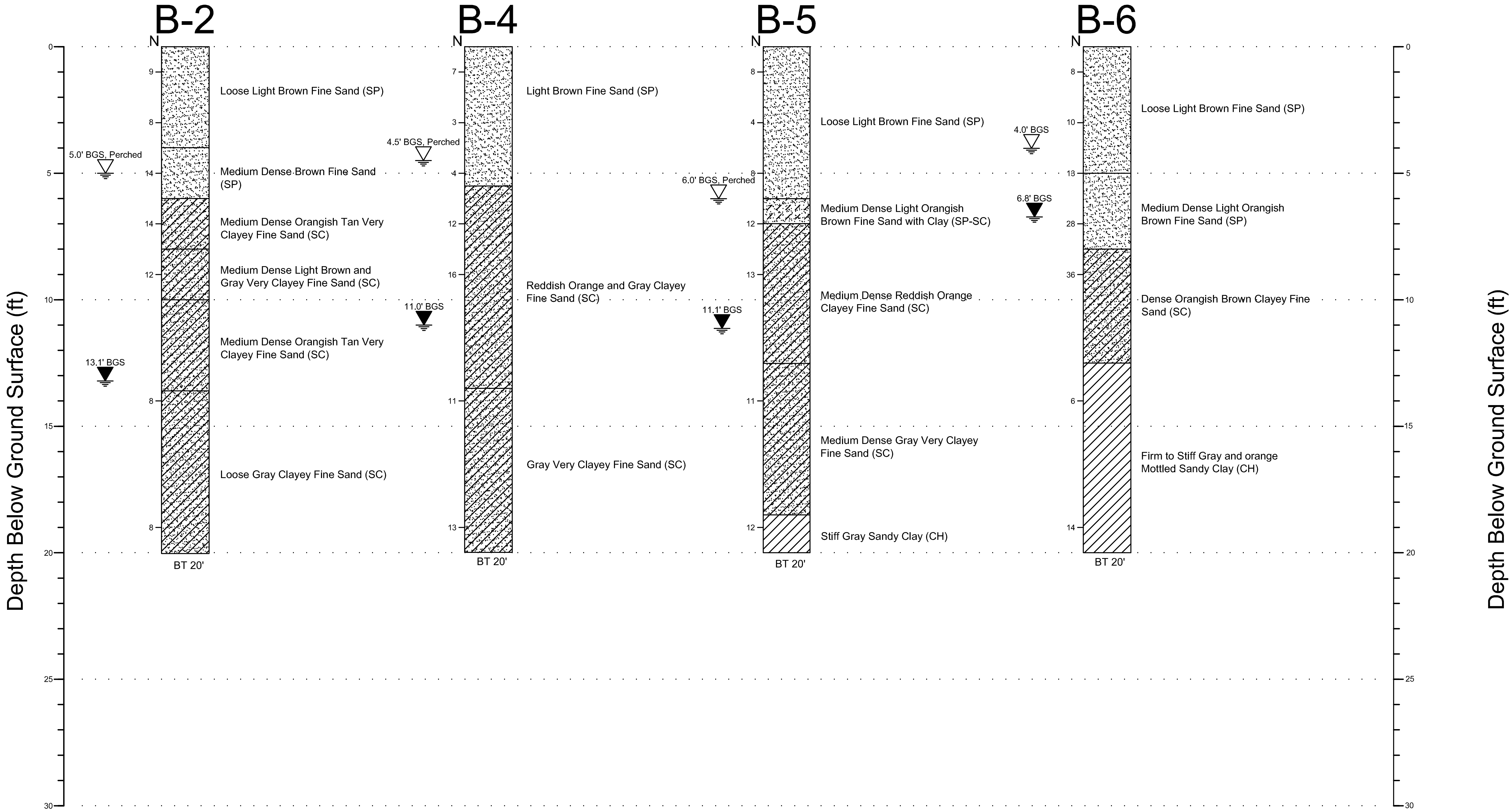
SUBSURFACE PROFILES



SPT Boring Location

Auger Boring Location

Jackson Geotechnical Engineering	
Boring Location Plan	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 1



Fine Sand (SP)

Clay (CH)

Clayey to Very Clayey Fine Sand (SC)

Fine Sand with Clay (SP-SC)

Seasonal High Groundwater Table Estimated Below Existing Ground Surface

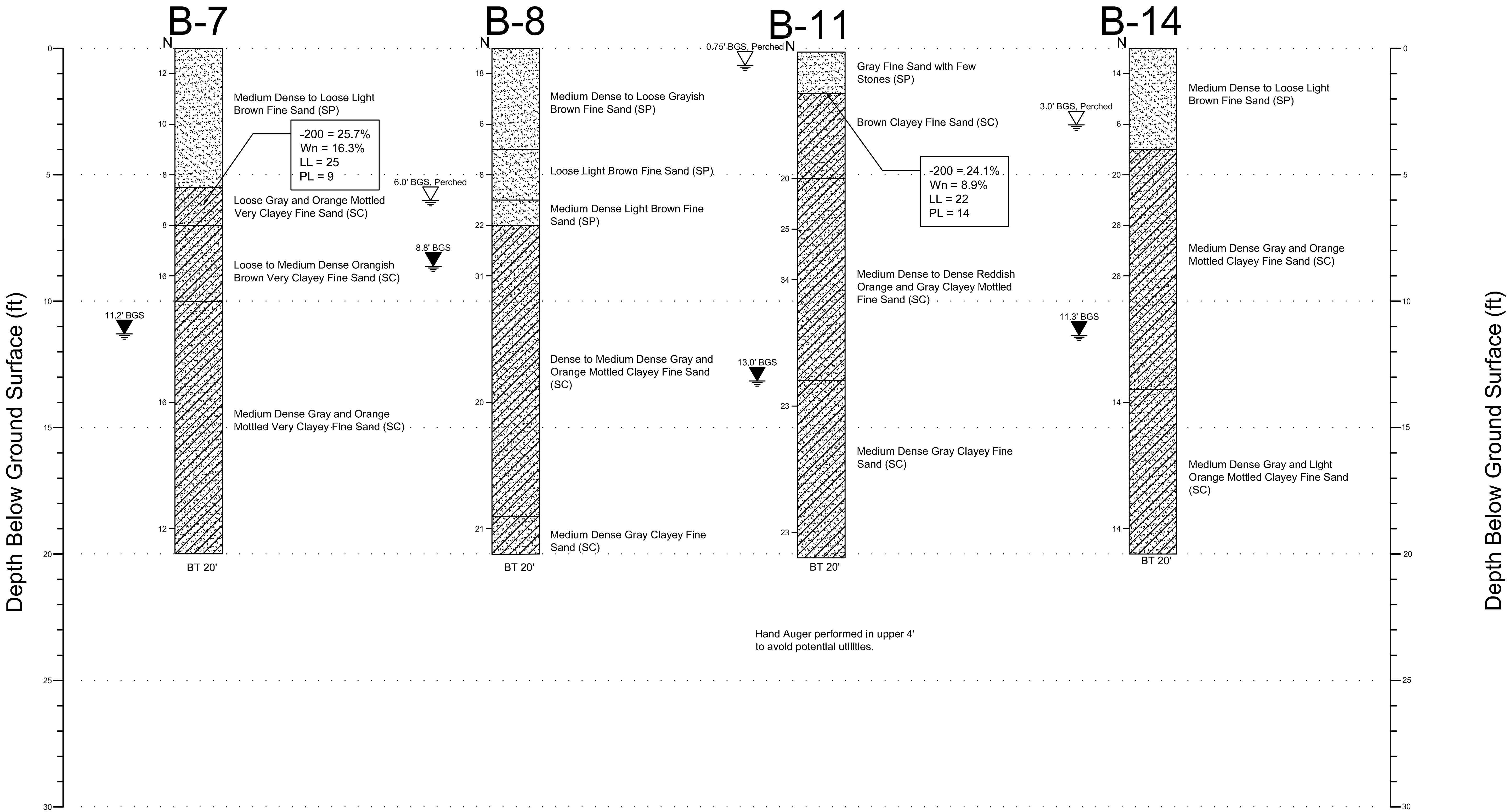
Groundwater Table Measured Below Ground Surface

BGS Below Ground Surface

BT Boring Terminated

N SPT - N Blow Count

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 2



Fine Sand (SP)

Clay (CH)

Clayey to Very Clayey Fine Sand (SC)

Fine Sand with Clay (SP-SC)

Seasonal High Groundwater Table Estimated Below Existing Ground Surface

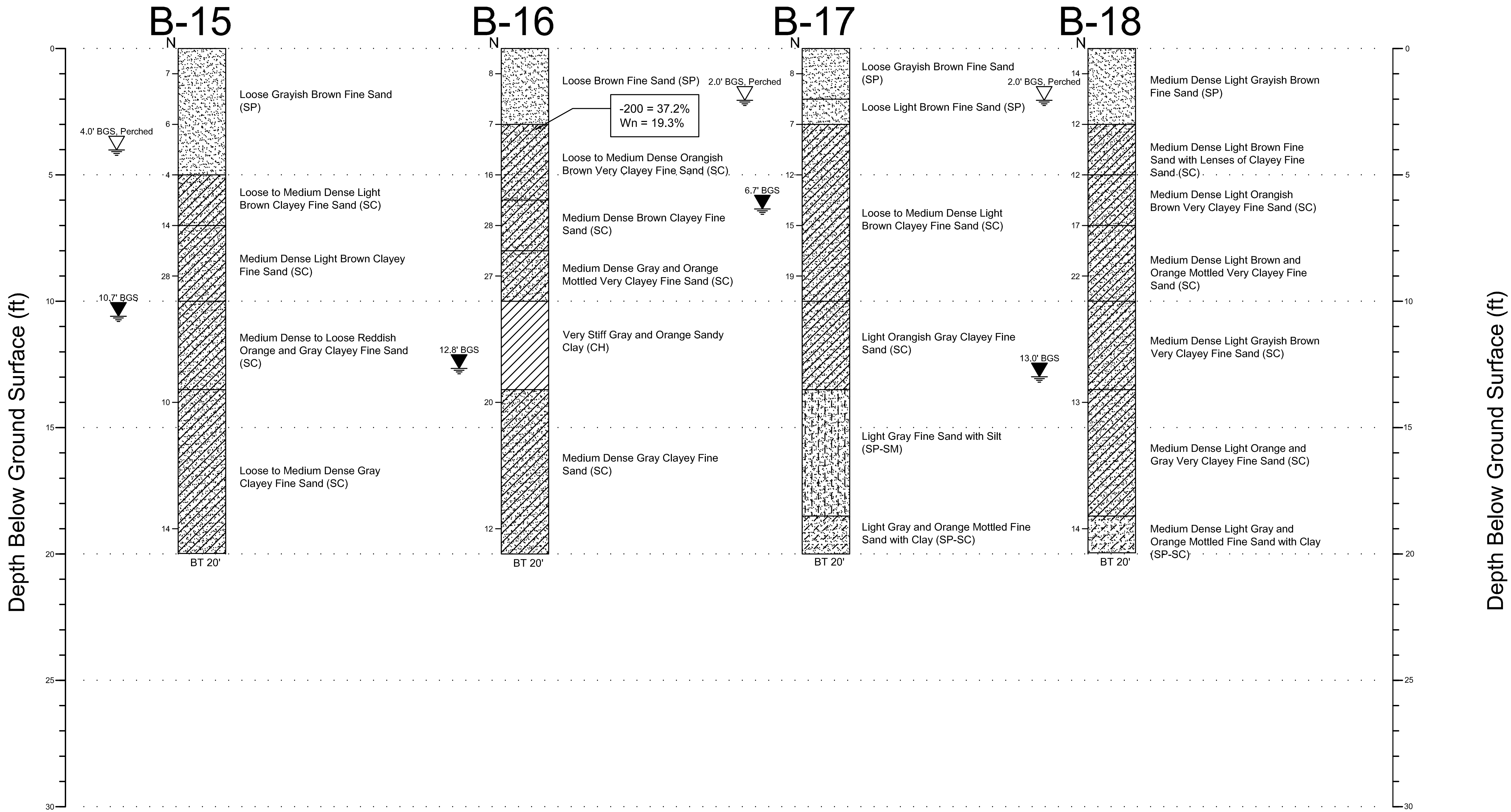
Groundwater Table Measured Below Ground Surface

BGS Below Ground Surface

BT Boring Terminated

N SPT - N Blow Count

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 3



Fine Sand (SP)

Clay (CH)

Clayey to Very Clayey Fine Sand (SC)

Fine Sand with Clay (SP-SC)

Seasonal High Groundwater Table Estimated Below Existing Ground Surface

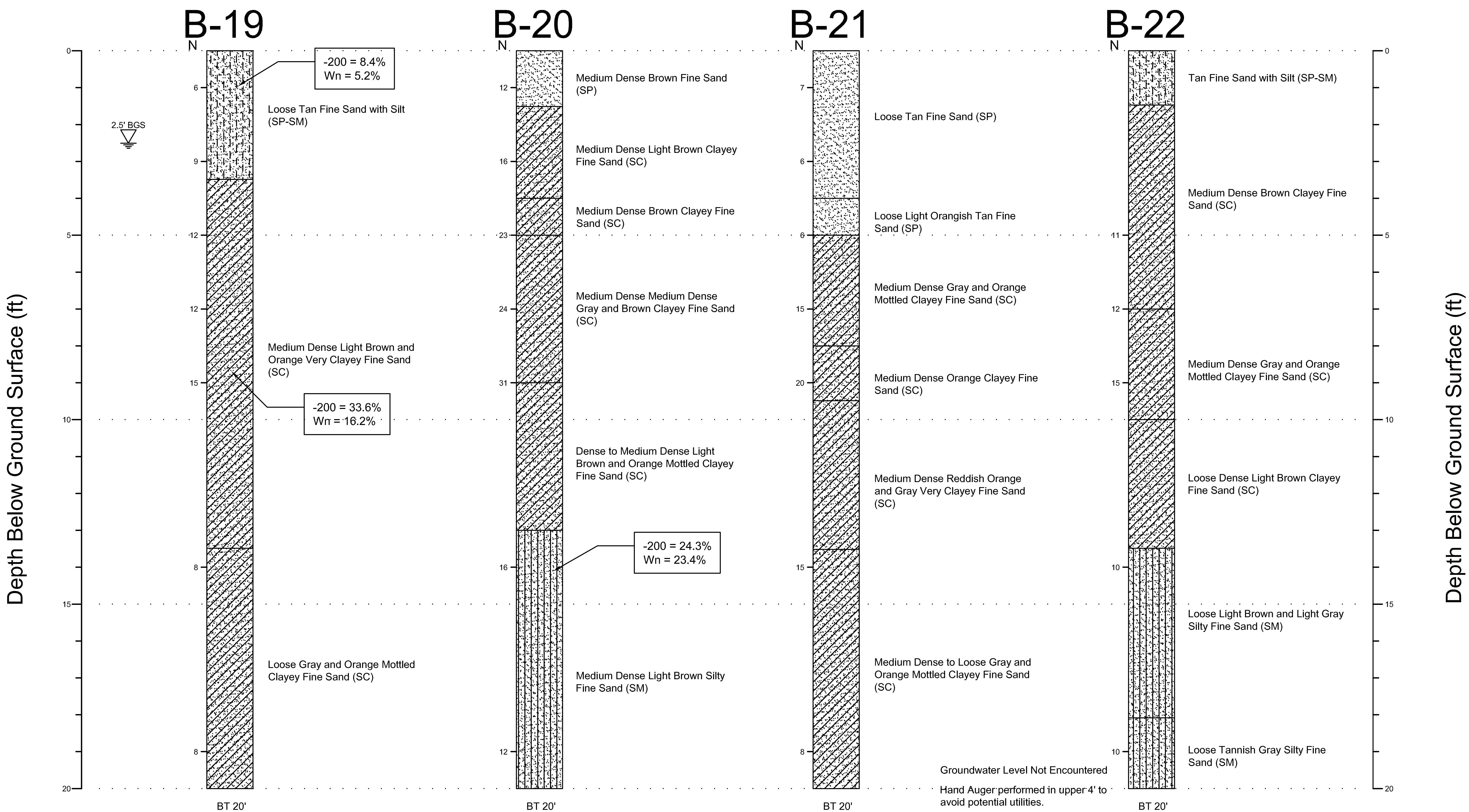
Groundwater Table Measured Below Ground Surface

BGS Below Ground Surface

BT Boring Terminated

N SPT - N Blow Count

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 4



Groundwater Level Not Encountered

Groundwater Level Not Encountered

Groundwater Level Not Encountered

Fine Sand (SP)

Fine Sand with Silt (SP-SM)

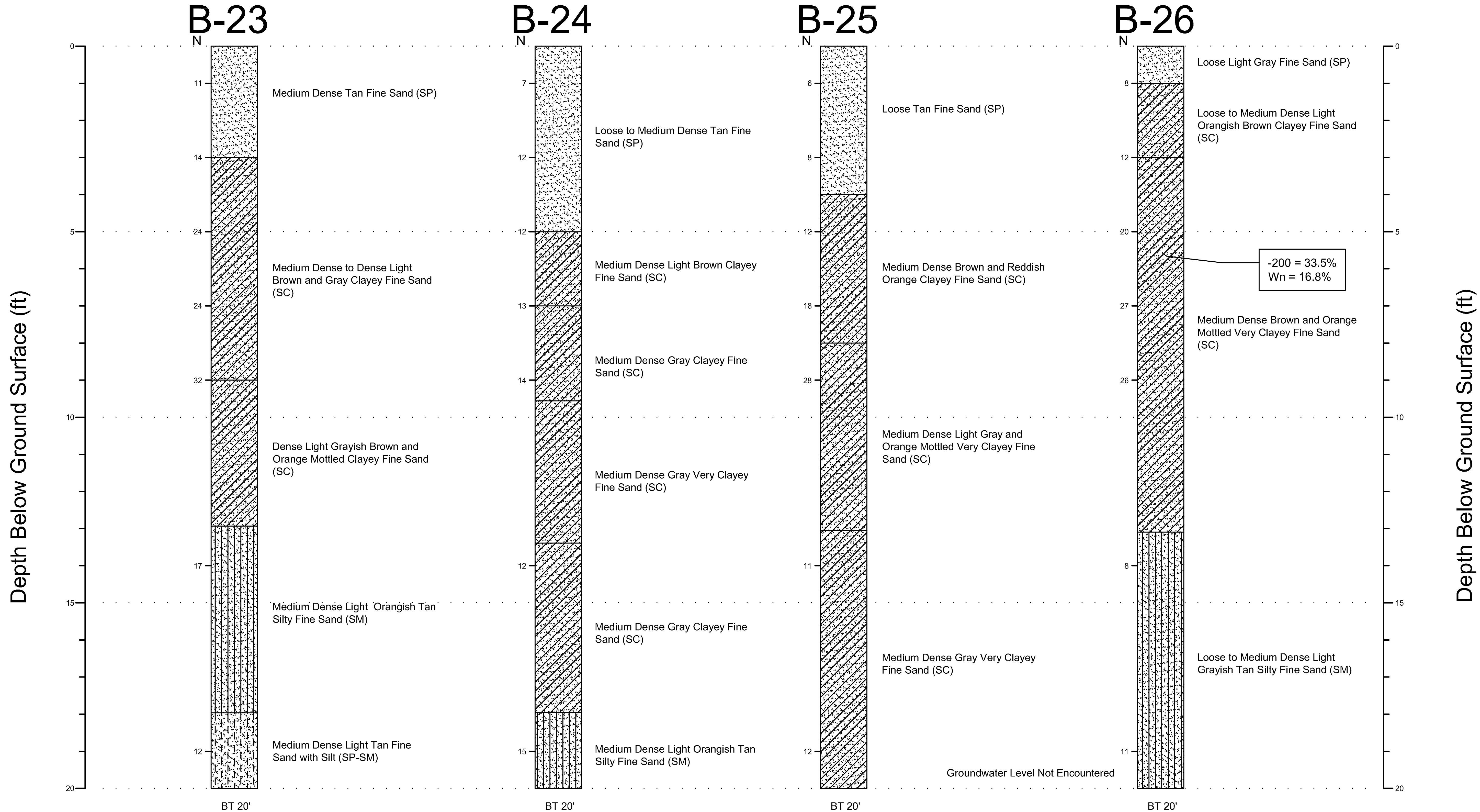
Clayey to Very Clayey Fine Sand (SC)

Silty Fine Sand (SM)

-200	Percentage of Silt/Clay in Sample
Wn	Natural Moisture Content

BT	Boring Terminated
BGS	Below Ground Surface
N	SPT - N Blow Count

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 5



Groundwater Level Not Encountered

Groundwater Level Not Encountered

Groundwater Level Not Encountered

Fine Sand (SP)

Fine Sand with Silt (SP-SM)

Clayey to Very Clayey Fine Sand (SC)

Silty Fine Sand (SM)

-200 Percentage of Silt/Clay in Sample

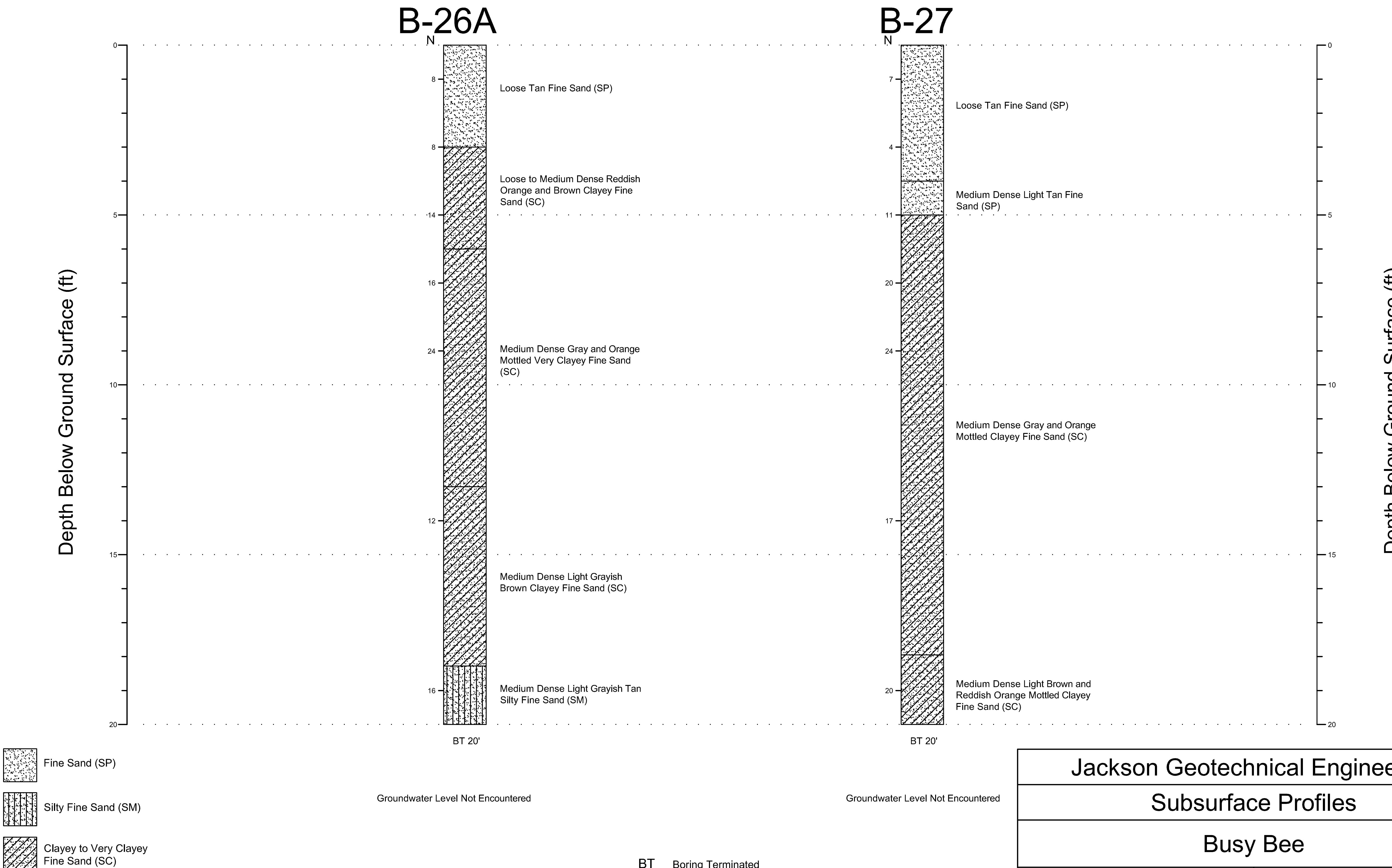
Wn Natural Moisture Content

BT Boring Terminated

BGS Below Ground Surface

N SPT - N Blow Count

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 6

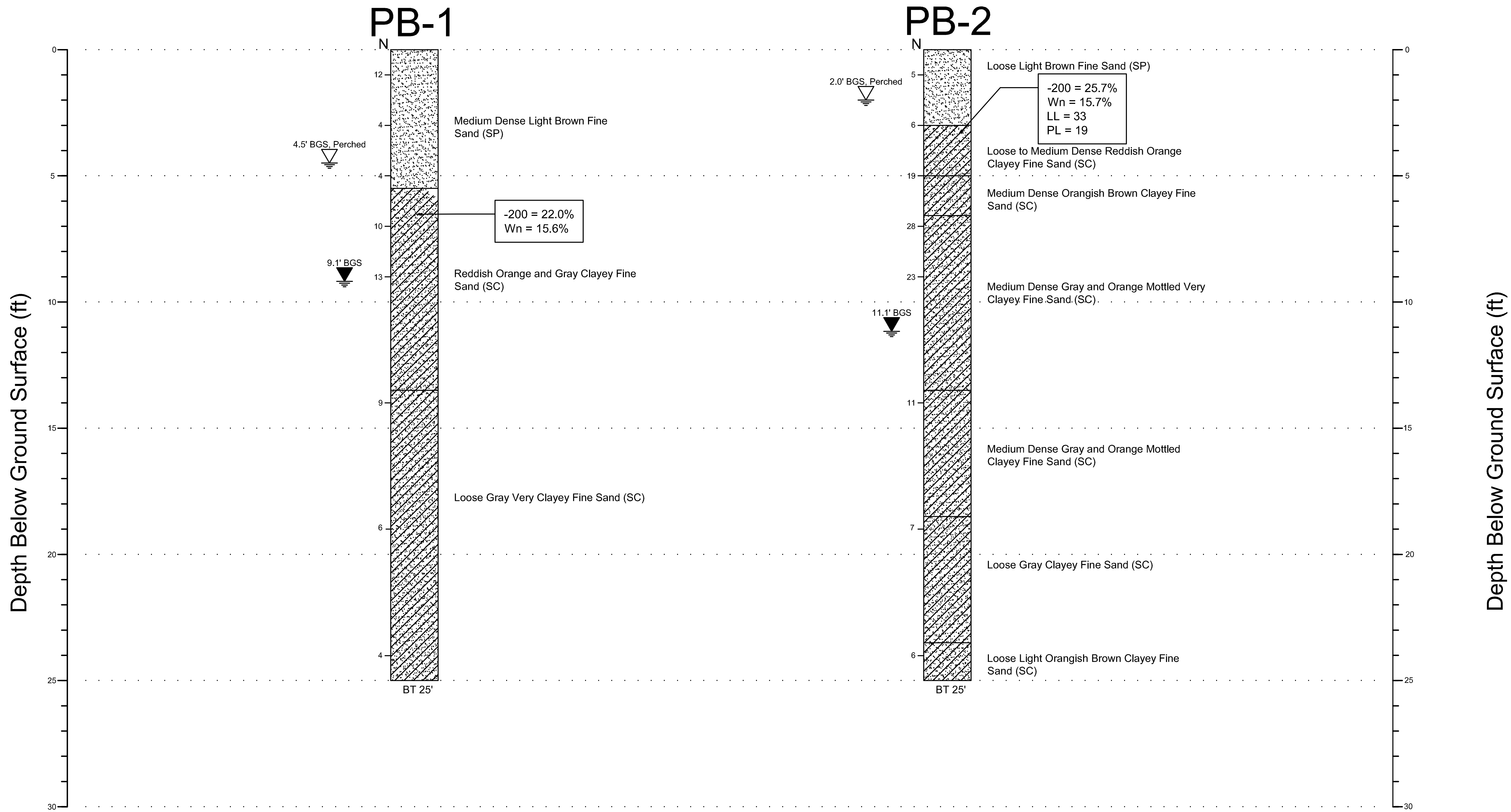


BT Boring Terminated

BGS Below Ground Surface

N SPT - N Blow Count

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 7



Fine Sand (SP)

Fine Sand with Silt (SP-SM)

Clayey to Very Clayey Fine Sand (SC)

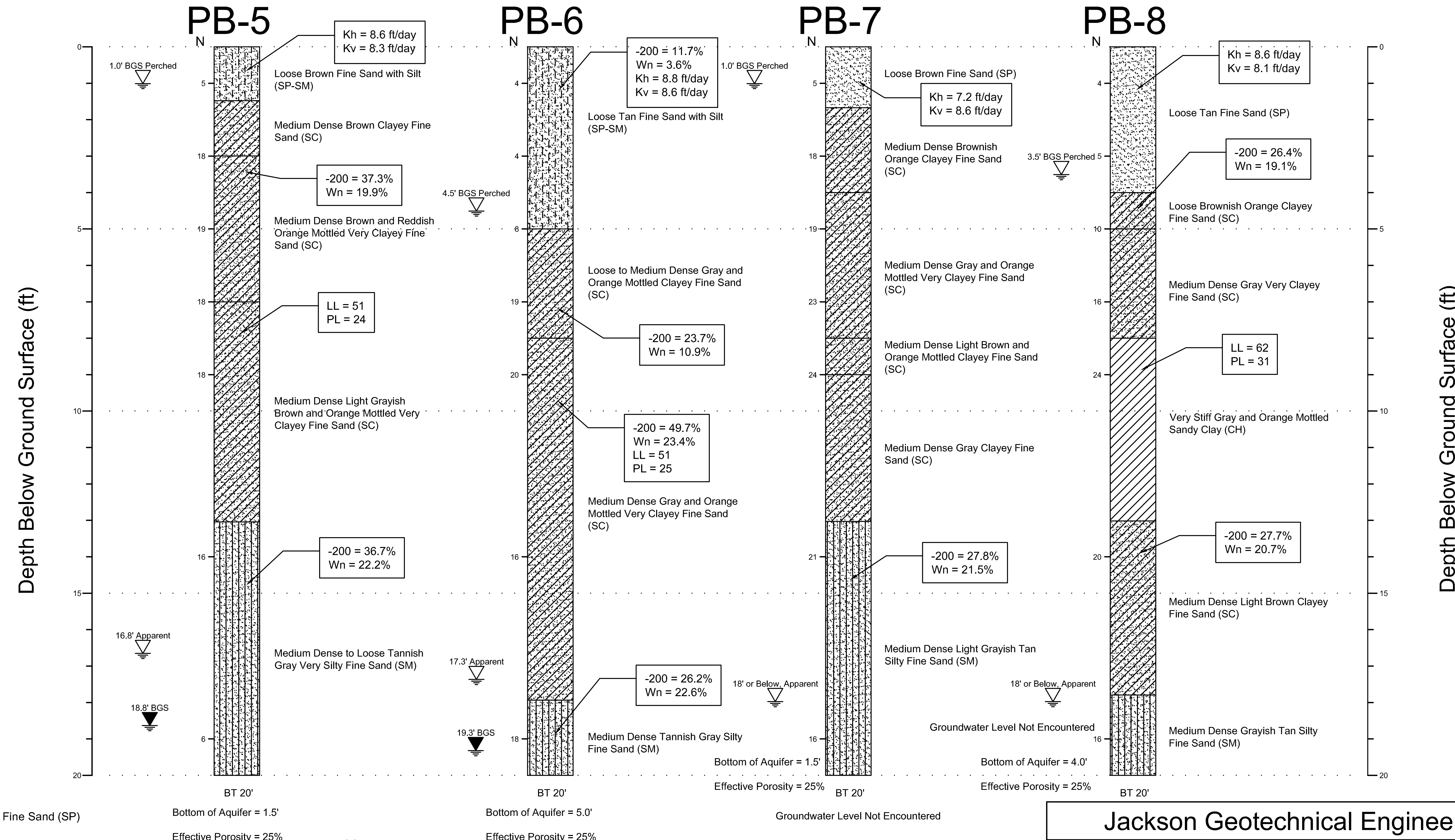
Silty Fine Sand (SM)

Clay (CH)

LL	Liquid Limit
PL	Plastic Limit
-200	Percentage of Silt/Clay in Sample
Wn	Natural Moisture Content

▽	Seasonal High Groundwater Table Estimated Below Existing Ground Surface
▼	Groundwater Table Measured Below Ground Surface
BGS	Below Ground Surface
BT	Boring Terminated
N	SPT - N Blow Count

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
May 8, 2023	Drawn by: MJ
Project No. 24-581	Figure 8



Fine Sand (SP)

Fine Sand with Silt (SP-SM)

Clayey to Very Clayey Fine Sand (SC)

Silty to Very Silty Fine Sand (SM)

Clay (CH)

LL	Liquid Limit
PL	Plastic Limit
-200	Percentage of Silt/Clay in Sample
Wn	Natural Moisture Content
Kh	Coefficient of Horizontal Permeability
Kv	Coefficient of Vertical Permeability

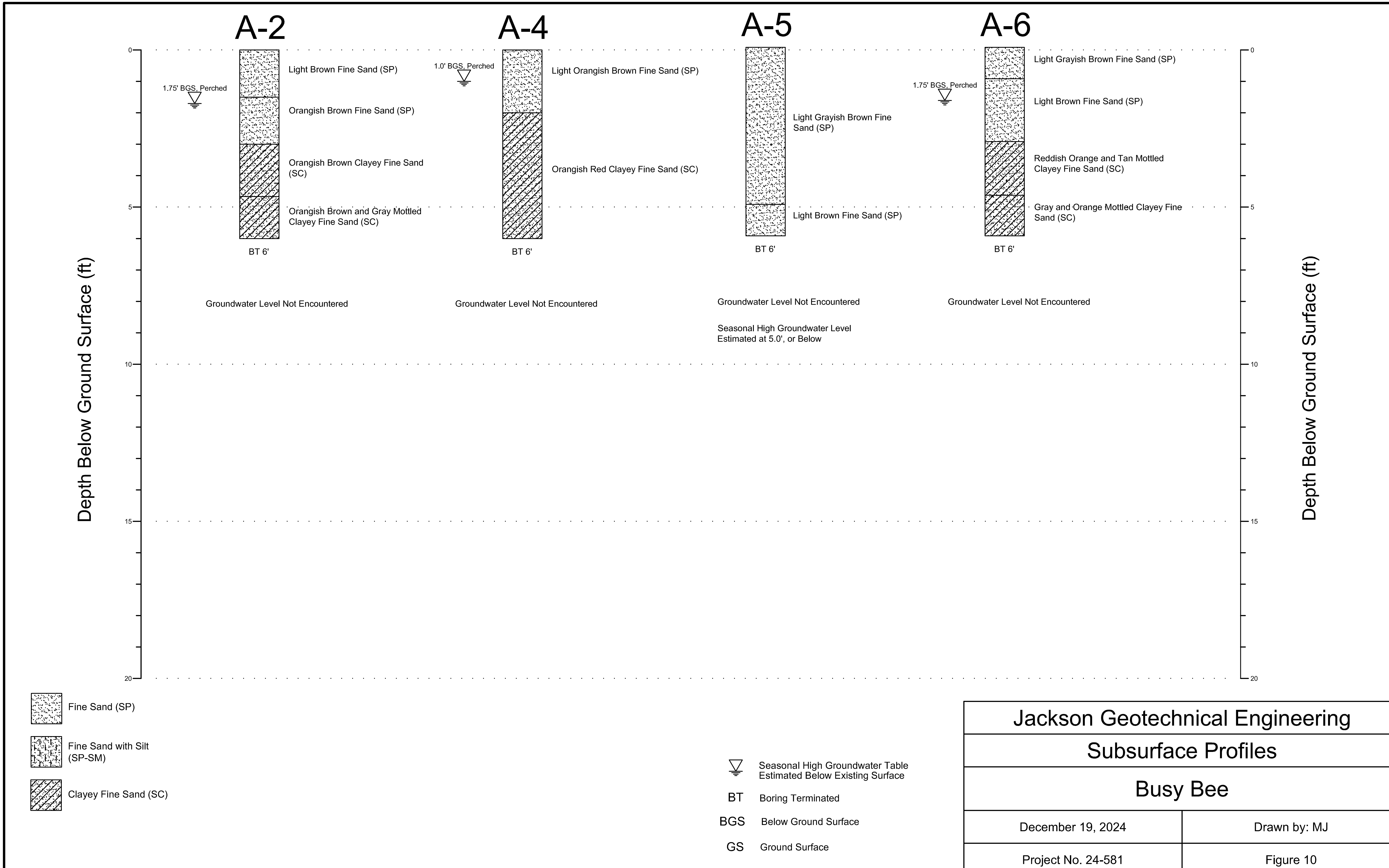
▽	Seasonal High Groundwater Table Estimated Below Existing Surface
▼	Groundwater Table Measured Below Surface
BT	Boring Terminated
BGS	Below Ground Surface
N	SPT - N Blow Count

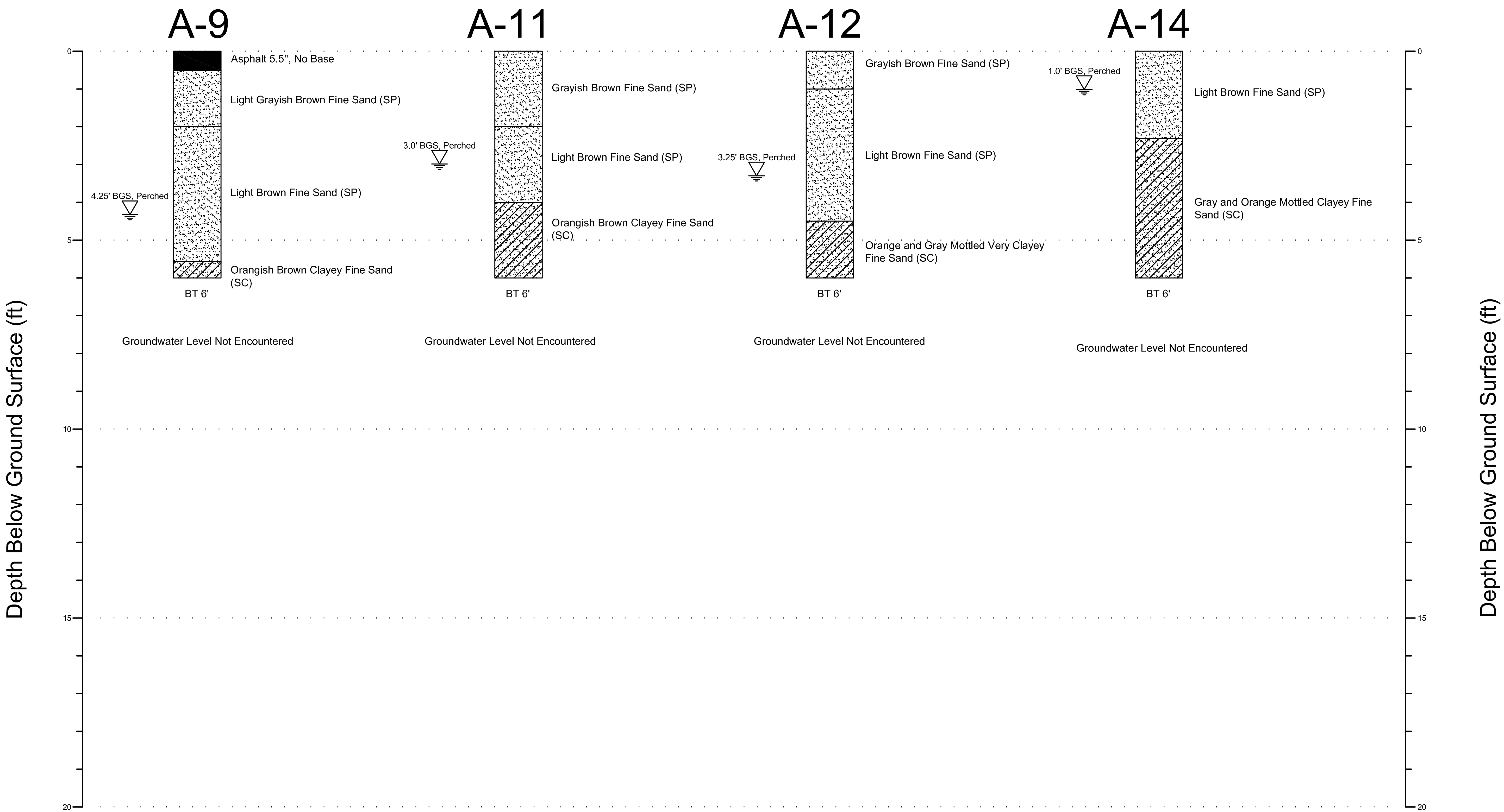
Jackson Geotechnical Engineering

Subsurface Profiles

Busy Bee

December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 9


A-2
A-4
A-5
A-6
Depth Below Ground Surface (ft)
0
5
10
15
20
1.75' BGS, Perched
Light Brown Fine Sand (SP)
Orangish Brown Fine Sand (SP)
Orangish Brown Clayey Fine Sand (SC)
Orangish Brown and Gray Mottled Clayey Fine Sand (SC)
BT 6'
Groundwater Level Not Encountered
1.0' BGS, Perched
Light Orangish Brown Fine Sand (SP)
Orangish Red Clayey Fine Sand (SC)
BT 6'
Groundwater Level Not Encountered
Light Grayish Brown Fine Sand (SP)
Light Brown Fine Sand (SP)
BT 6'
Groundwater Level Not Encountered
Seasonal High Groundwater Level Estimated at 5.0', or Below
Light Grayish Brown Fine Sand (SP)
1.75' BGS, Perched
Light Brown Fine Sand (SP)
Reddish Orange and Tan Mottled Clayey Fine Sand (SC)
Gray and Orange Mottled Clayey Fine Sand (SC)
BT 6'
Groundwater Level Not Encountered
Fine Sand (SP)
Fine Sand with Silt (SP-SM)
Clayey Fine Sand (SC)
Seasonal High Groundwater Table Estimated Below Existing Surface
BT Boring Terminated
BGS Below Ground Surface
GS Ground Surface
Jackson Geotechnical Engineering
Subsurface Profiles
Busy Bee
December 19, 2024
Drawn by: MJ
Project No. 24-581
Figure 10



Fine Sand (SP)

Fine Sand with Silt (SP-SM)

Clayey Fine Sand (SC)

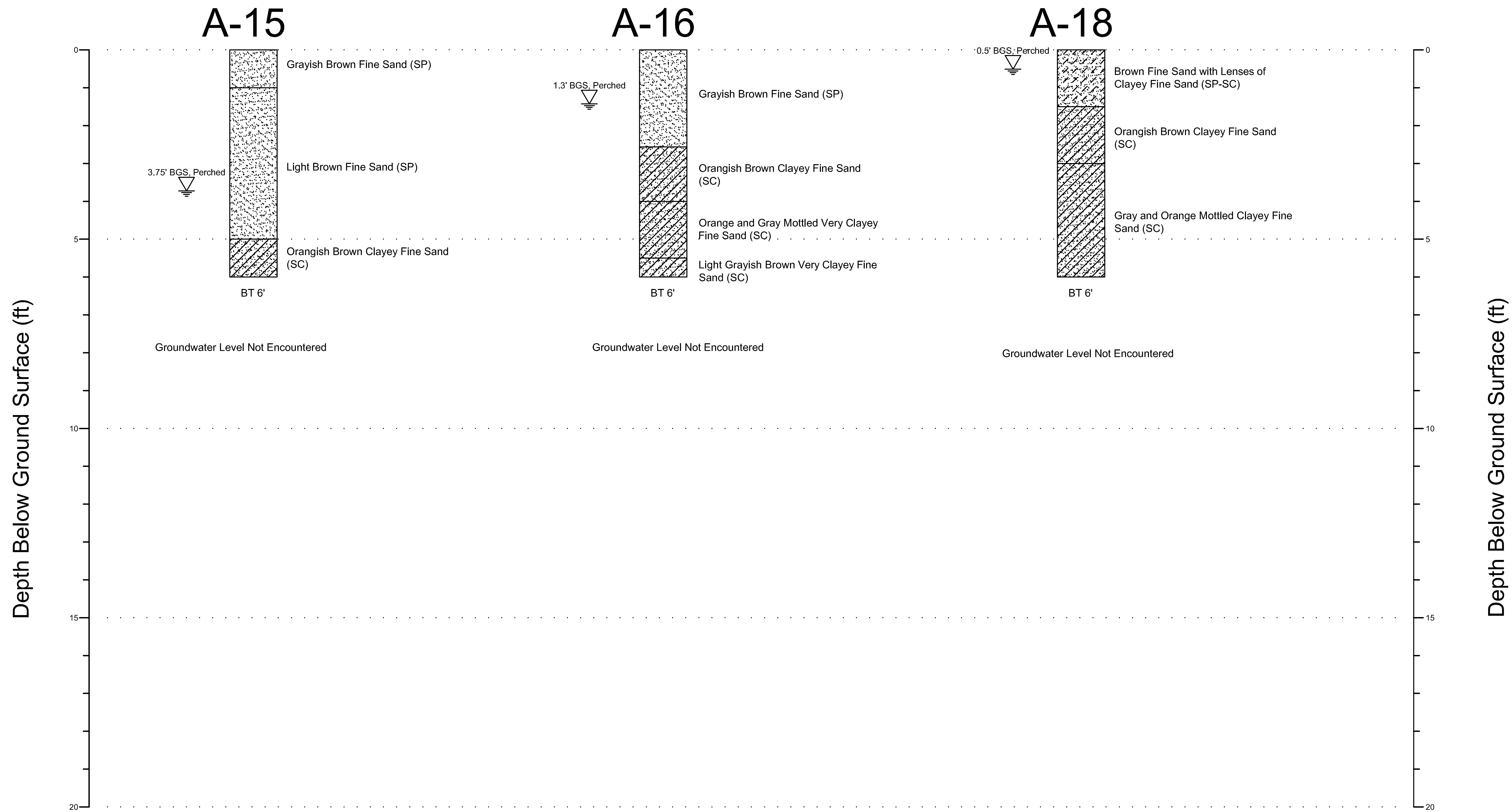
Seasonal High Groundwater Table Estimated Below Existing Surface

BT Boring Terminated

BGS Below Ground Surface

GS Ground Surface

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 11



Fine Sand (SP)

Fine Sand with Silt (SP-SM)

Clayey Fine Sand (SC)

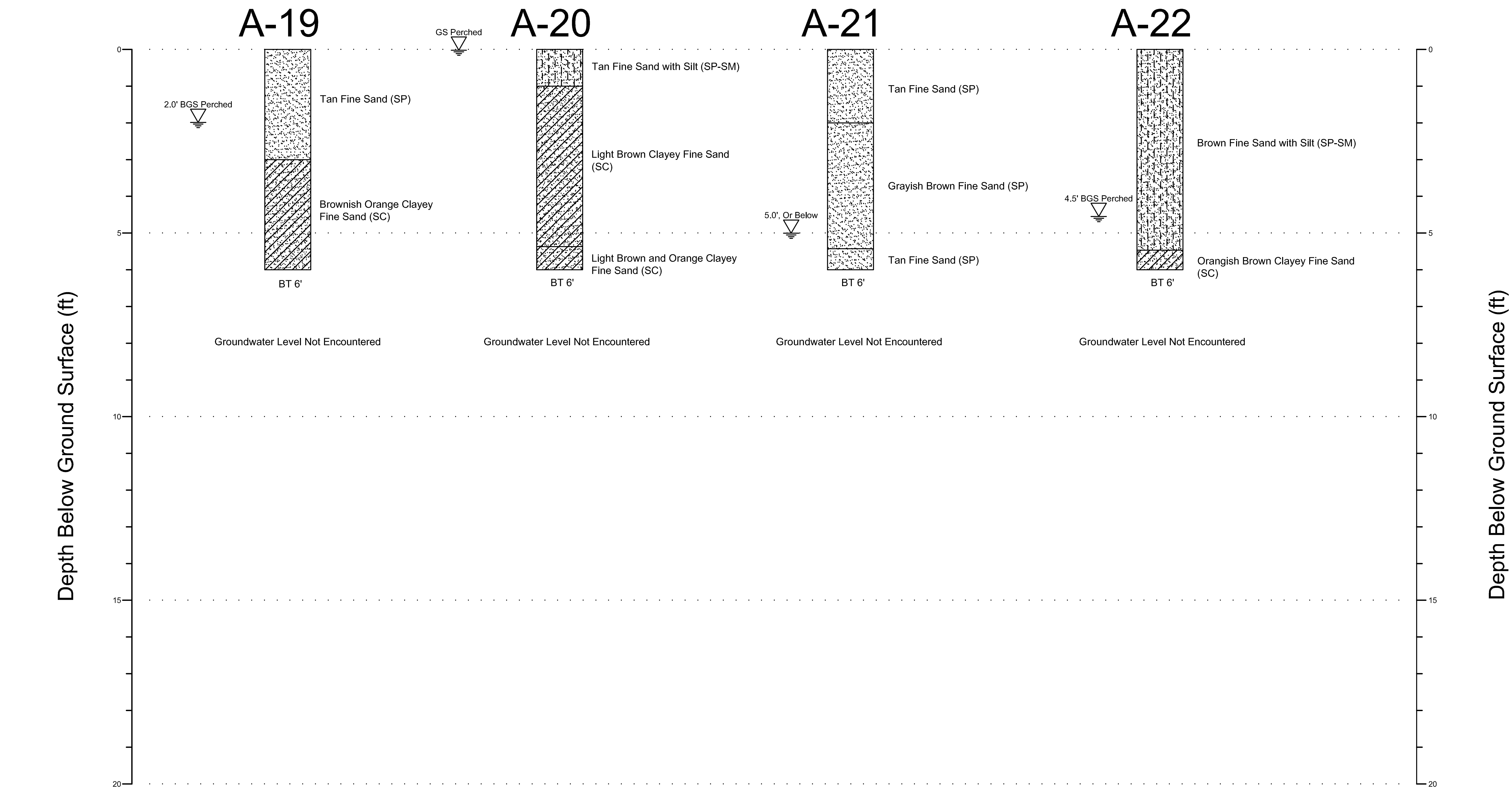
Seasonal High Groundwater Table Estimated Below Existing Surface

BT Boring Terminated

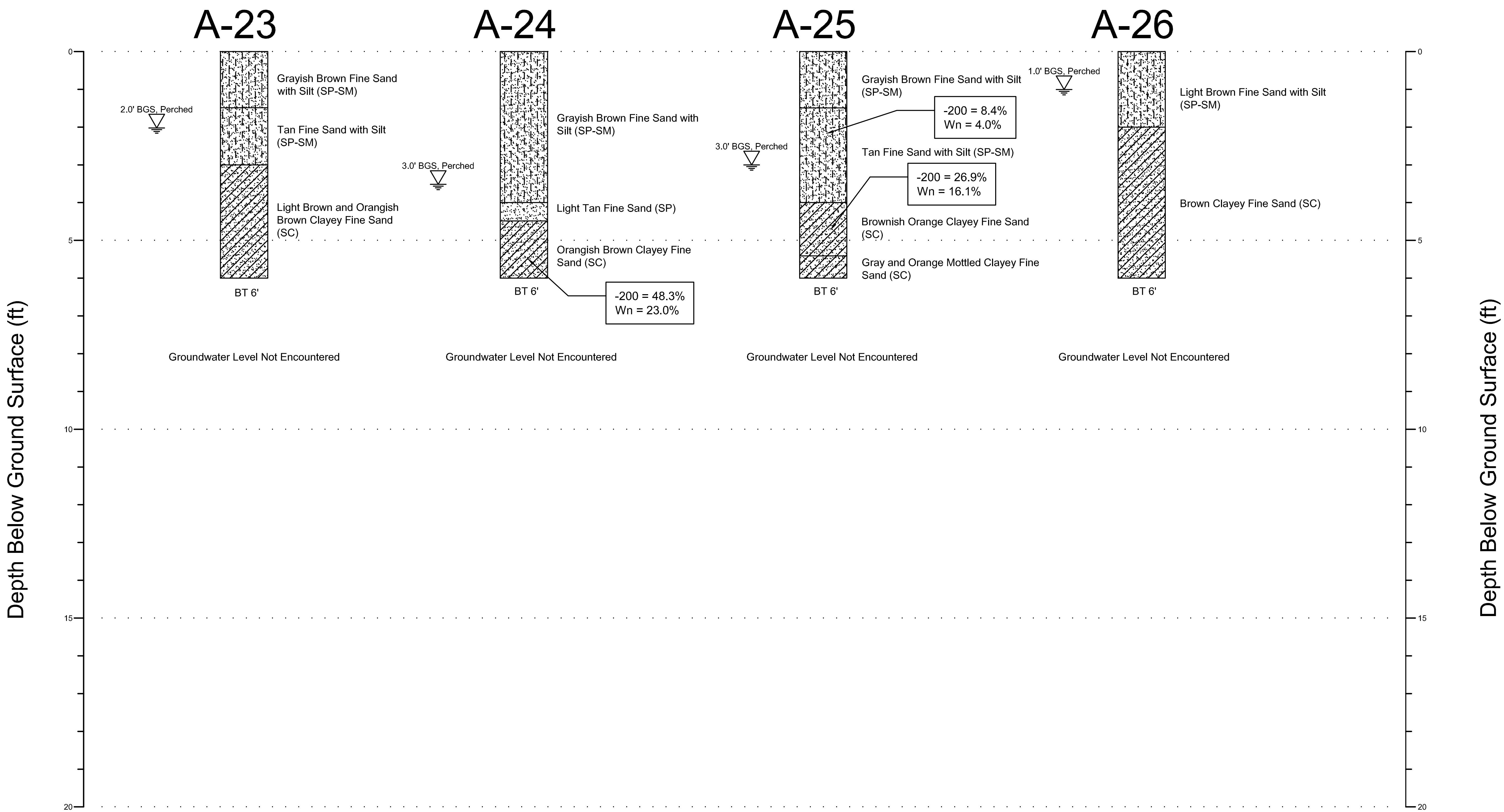
BGS Below Ground Surface

GS Ground Surface

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 12



Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 13



Fine Sand (SP)

Fine Sand with Silt (SP-SM)

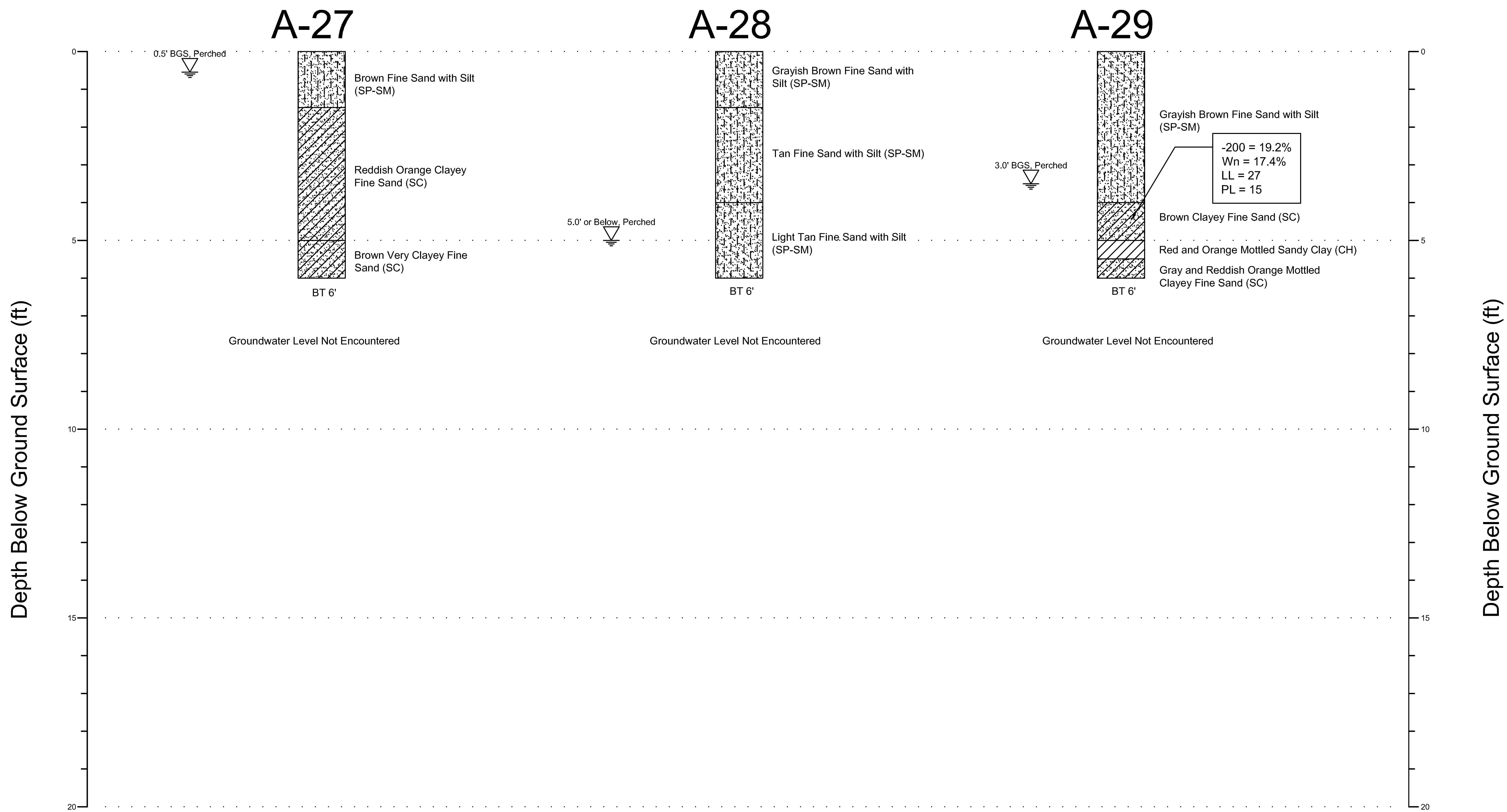
Clayey Fine Sand (SC)

Seasonal High Groundwater Table Estimated Below Existing Surface

BT Boring Terminated

BGS Below Ground Surface

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 14



Fine Sand with Silt (SP-SM)

Clay (CH)

Clayey to Very Clayey Fine Sand (SC)

LL	Liquid Limit
PL	Plastic Limit
-200	Percentage of Silt/Clay in Sample
Wn	Natural Moisture Content

▽	Seasonal High Groundwater Table Estimated Below Existing Surface
BT	Boring Terminated
BGS	Below Ground Surface

Jackson Geotechnical Engineering	
Subsurface Profiles	
Busy Bee	
December 19, 2024	Drawn by: MJ
Project No. 24-581	Figure 15

APPENDIX B

KEY TO SOIL CLASSIFICATION

FIELD AND LABORATORY TEST PROCEDURES

JACKSON GEOTECHNICAL ENGINEERING

Consulting Geotechnical Engineers

KEY TO SOIL CLASSIFICATION

CORRELATION OF PENETRATION WITH RELATIVE DENSITY & CONSISTENCY

SANDS AND GRAVEL	
BLOW COUNT	RELATIVE DENSITY
0-3	VERY LOOSE
4-10	LOOSE
11-30	MEDIUM DENSE
31-50	DENSE
OVER 50	VERY DENSE

SILTS AND CLAYS	
BLOW COUNT	CONSISTENCY
0-2	VERY SOFT
3-4	SOFT
5-8	FIRM
16-30	VERY STIFF
31-50	HARD
OVER 50	VERY HARD

PARTICLE SIZE IDENTIFICATION (UNIFIED CLASSIFICATION SYSTEM)

CATEGORY	*DIMENSIONS*
Boulders	Diameter exceeds 12 inches
Cobbles	3 to 12 inches
Gravel	Coarse – 0.75 to 3 inches in diameter Fine – 4.76 mm to 0.75 inch diameter
Sand	Coarse – 2.0 mm to 4.76 mm diameter Medium – 0.42 mm to 2.0 mm diameter Fine – 0.074 mm to 0.42 mm diameter
Silt and Clay	Less than 0.074 mm (invisible to the naked eye)

MODIFIERS

These modifiers provide our estimate of the amount of minor constituent (sand, silt, or clay size particles) in the soil sample

PERCENTAGE OF MINOR CONSTITUENT	*MODIFIERS*
0% to 5%	No Modifier
5 % to 12 %	With Silt, With Clay
12% to 30%	Silty, Clayey, Sandy
30% to 50%	Very Silty, Very Clayey, Very Sandy

APPROXIMATE CONTENT OF OTHER COMPONENTS (SHELL, GRAVEL, ETC.)	*MODIFIERS*	*APPROXIMATE CONTENT OF ORGANIC COMPONENTS*
0% to 5%	TRACE	1 to 2%
5% to 12%	FEW	2% to 4%
12% to 30%	SOME	4% to 8%
30% to 50%	MANY	>8%

FIELD AND LABORATORY TEST PROCEDURES

Penetration Borings

The penetration borings were made in general accordance with ASTM D 1586-67, "Penetration Test and Split-Barrel Sampling of Soils". Each boring was advanced to the water table by augering and, after encountering the groundwater table, further advanced with a rotary drilling technique that uses a circulating bentonite fluid for borehole flushing and stability. At two-foot intervals within the upper 10 feet and at five-foot intervals thereafter, the drilling tools were removed from the borehole and a split-barrel sampler inserted to the borehole bottom. The sampler was then driven 18 inches into the material using a 140-pound SPT hammer falling, on the average, 30 inches per hammer blow. The number of hammer blows for the final 12 inches of penetration is termed the "penetration resistance, blow count, or N-value". This value is an index to several in-place geotechnical properties of the material tested, such as relative density and Young's Modulus.

After driving the sampler 18 inches (or less, if in hard rock or rock-like material) at each test interval, the sampler was retrieved from the borehole and a representative sample of the material within the split-barrel was placed in a watertight container and sealed. After completing the drilling operations, the samples for each boring were transported to our laboratory where our Geotechnical Engineer examined them in order to verify the driller's field classifications. The samples will be kept in our laboratory for a period of two months after submittal of formal written report, unless otherwise directed by the Client.

Auger Borings

The auger borings were performed using a continuous flight auger attached to a rotary drill rig or manually using a post-hole auger; and thus in general accordance with ASTM D 1452-80, "Soil Investigation and Sampling by Auger Borings". Representative samples of the soils brought to the ground surface by the augering process were placed in watertight containers and sealed. After completing the drilling operations, the samples for each boring were transported to the laboratory where the Geotechnical Engineer examined them in order to verify the driller's field classifications. The samples will be kept in our laboratory for a period of two months after submittal of formal written report, unless otherwise directed by the Client.

Soil Classification

Soil samples obtained from the performance of the borings were transported to our laboratory for observation and review. An engineer, registered in the State of Florida and familiar with local geological conditions, conducted the review and classified the soils in accordance with ASTM 2488. The results of the soil classification are presented on the boring records.

Moisture Content

The moisture content of the sample tested was determined in general accordance with ASTM D 2216. The moisture content is the actual moisture content of the sample as sampled in the field during the performance of the soil boring.

Fines Content

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

Atterberg Limits

The Atterberg limits of a material are the liquid limit (LL), plastic limit (PL) and plastic index (PI) of a soil as determined in general accordance with ASTM D 4318. The liquid limit is the water content (in percent) of the soil at the arbitrarily defined boundary between the semi-liquid and the plastic states. The plastic limit is the water content (in percent) of the soil at the boundary between the plastic and semi-solid states. The plastic or plasticity index is the range of water content over which a soil behaves plastically. Numerically, it is the difference between the liquid limit and the plastic limit.

Constant Head Permeability Test

The coefficient of permeability for the laminar flow of water through granular soils was determined in general accordance with the latest revision of ASTM D 2434. The constant head permeability test is a measure of the quantity of water that flows through a sample contained in a cylinder of known height and diameter in a measured time while maintaining a constant head of water on the sample. The coefficient of permeability is determined by application of the Darcy's Law shown below:

$$k = \frac{Q\,L}{hAt}$$

k = Coefficient of permeability

Q = Quantity of water discharge

L = Length of specimen

h = Constant head of water

A = Cross-sectional area of specimen

t = Total time of discharge

Undisturbed Sampling

Relatively undisturbed samples were obtained in general accordance with the latest revision of ASTM A 1587, "Thin-Walled Tube Sampling of Soils". Manual methods were used to advance the 3-inch O.D. – 16 gauge stainless steel sampler tubes into the soils at the selected depths. After retrieving the samples, the ends were capped and then transported to our laboratory.