GEOTECHNICAL ENGINEERING STUDY
CASTLEBERRY HIGH SCHOOL ADDITIONS
215 CHURCHILL ROAD
CASTLEBERRY ISD
FORT WORTH, TEXAS

Presented To:

Castleberry Independent School District

April 2024

PROJECT NO. 1029-24-03

CMJ Engineering

April 29, 2024
Report No. 1029-24-03

Castleberry Independent School District
5228 Ohio Garden Road
Fort Worth, Texas 76114

Attn: Mr. Lenny Lasher, Assistant Superintendent

GEOTECHNICAL ENGINEERING STUDY
CASTLEBERRY HIGH SCHOOL ADDITIONS
215 CHURCHILL ROAD
CASTLEBERRY ISD
FORT WORTH, TEXAS

Dear Mr. Lasher:

Submitted here are the results of a geotechnical engineering study for the referenced project. This investigation was performed in general accordance with CMJ Estimate No. 23-9226 (Revised) dated December 27, 2023. The geotechnical services were authorized via P.O. Number 2752400236, executed by Ms. DeAnne M. Page, Executive Director of Financial Services on February 2, 2024.

Engineering analyses and recommendations are contained in the text section of the report. Results of our field and laboratory services are included in the appendix of the report. We would appreciate the opportunity to be considered for providing the construction material testing services during the construction phase of this project.

We appreciate the opportunity to be of service to Castleberry Independent School District and its consultants. Please contact us if you have any questions or if we may be of further service at this time.

Respectfully submitted,
CMJ ENGINEERING, INC.
TEXAS FIRM REGISTRATION NO. F-9177

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APPENDIX A

1.0 INTRODUCTION

1.1 General

The project site is located at the existing Castleberry High School campus in Fort Worth, Texas. The project as currently planned, will consist of the planned Phases 1A and 1B, consisting of new administration/CTE wing, new gymnasium (storm shelter), new core, connector to the existing school, band hall addition, plaza, service yard, and new student parking. The total new building footprint area is on the order of 65,500 square feet comprised of one, two, and three-story components, and split-level construction. Anticipated dead loads range from 20 to 150 kips and anticipated live loads range from 10 to 125 kips. New access drives and fire lanes are also planned. Site retaining walls are also anticipated with heights up to 10 feet, particularly in the new plaza area. New shotput and discus pads are also planned. Plate A.1, Plan of Borings, presents the approximate locations of the exploration borings.

1.2 Purpose and Scope

The purpose of this geotechnical engineering study has been to determine the general subsurface conditions, evaluate the engineering characteristics of the subsurface materials encountered, and develop recommendations for the type or types of foundations suitable for the project.

To accomplish its intended purposes, the study has been conducted in the following phases: (1) drilling sample borings to determine the general subsurface conditions and to obtain samples for testing; (2) performing laboratory tests on appropriate samples to determine pertinent engineering properties of the subsurface materials; and (3) performing engineering analyses, using the field and laboratory data to develop geotechnical recommendations for the proposed construction.

The design is currently in progress and the locations and/or elevations of the structures could change. Once the final design is near completion (80-percent to 90-percent stage), it is recommended that CMJ Engineering, Inc. be retained to review those portions of the construction documents pertaining to the geotechnical recommendations, as a means to determine that our recommendations have been interpreted as intended.

1.3 Report Format

The text of the report is contained in Sections 1 through 12. All plates and large tables are contained in Appendix A. The alpha-numeric plate and table numbers identify the appendix in which they appear. Small tables of less than one page in length may appear in the body of the text and are numbered according to the section in which they occur.

Units used in the report are based on the English system and may include tons per square foot (tsf), kips (1 kip = 1,000 pounds), kips per square foot (ksf), pounds per square foot (psf), pounds per cubic foot (pcf), and pounds per square inch (psi).

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

Subsurface materials at the project site were explored with eighteen (18) vertical soil borings. Borings B-1 through B-11 were drilled to depths of 40 to 55 feet below existing grades in the area of the proposed buildings, plazas, and retaining walls. Borings B-12 through B-16 were drilled to a depth of 8 feet for proposed drives, fire lanes, parking, and service yard, and Borings B-17 and B-18 were drilled to a depth of 15 feet for proposed shotput and discuss pads. The borings were drilled using continuous flight augers with a truck-mounted drilling rig. Borings were drilled at the approximate locations shown on the Plan of Borings, Plate A.1. The boring logs are included on Plates A.4 through A.21 and keys to classifications and symbols used on the logs are provided on Plates A.2 and A.3. Ground surface elevations shown on the boring logs are approximate as interpreted from the grading plan, Sheet CG2, design development document dated April 18, 2024 as prepared by RLK Engineering.

Undisturbed samples of cohesive soils were obtained with nominal 3-inch diameter thin-walled (Shelby) tube samplers at the locations shown on the logs of borings. The Shelby tube sampler consists of a thin-walled steel tube with a sharp cutting edge connected to a head equipped with a ball valve threaded for rod connection. The tube is pushed into the soil by the hydraulic pulldown of the drilling rig. The soil specimens were extruded from the tube in the field, logged, tested for consistency with a hand penetrometer, sealed, and packaged to limit loss of moisture.

The consistency of cohesive soil samples was evaluated in the field using a calibrated hand penetrometer. In this test a 0.25-inch diameter piston is pushed into the relatively undisturbed sample at a constant rate to a depth of 0.25 inch. The results of these tests, in tsf, are tabulated at respective sample depths on the logs. When the capacity of the penetrometer is exceeded, the value is tabulated as 4.5+.

Disturbed samples of the noncohesive granular or stiff to hard cohesive materials were obtained utilizing a nominal 2-inch O.D. split-barrel (split-spoon) sampler in conjunction with the Standard Penetration Test (ASTM D1586). This test employs a 140-pound hammer that drops a free fall vertical distance of 30 inches, driving the split-spoon sampler into the material. The number of blows required for 18 inches of penetration is recorded and the value for the last 12 inches, or the penetration obtained from 50 blows, is reported as the Standard Penetration Value (N) at the appropriate depth on the logs of borings.

To evaluate the relative density and consistency of the harder formations, a modified version of the Texas Cone Penetration test was performed at selected locations. Texas Department of Transportation (TxDOT) Test Method Tex-132-E specifies driving a 3-inch diameter cone with a 170-pound hammer freely falling 24 inches. This results in 340 foot-pounds of energy for each blow. This method was modified by utilizing a 140-pound hammer freely falling 30 inches. This results in 350 foot-pounds of energy for each hammer blow. In relatively soft materials, the penetrometer cone is driven 1 foot and the number of blows required for each 6-inch penetration is tabulated at respective test depths, as blows per 6 inches on the log. In hard materials (rock or rock-like), the penetrometer cone is driven with the resulting penetrations, in inches, recorded for the first and second 50 blows, a total of 100 blows. The penetration for the total 100 blows is recorded at the respective testing depths on the boring logs.

Groundwater observations during and after completion of the borings are shown on the upper right of the boring log. Upon completion of the borings, the bore holes were backfilled with soil cuttings and plugged at the surface by hand tamping.

2.2 Laboratory Testing

Laboratory soil tests were performed on selected representative samples recovered from the borings. In addition to the classification tests (liquid limits, plastic limits, and particle size analyses), moisture content, unit weight, and unconfined compressive strength tests were performed. Results of the laboratory classification tests, moisture content, unit weight, and unconfined compressive strength tests conducted for this project are included on the boring logs. Particle size analysis results are presented on Plates A.22 through A.24.

Free swell tests were performed on specimens from selected samples of the soils. These tests were performed to help in evaluating the swell potential of near-surface soils in the area of the proposed structures. The results of the swell tests are presented on Plate A.25

Analytical tests to aid in evaluation of corrosive potential of the on-site soils were performed on selected samples recovered from the borings. The results of the analytical testing are tabulated on Plate A.26.

The above laboratory tests were performed in general accordance with applicable ASTM procedures, or generally accepted practice.

3.0 SUBSURFACE CONDITIONS

3.1 Soil Conditions

Specific types and depths of subsurface strata encountered at the boring locations are shown on the boring logs in Appendix A. Note that depths on the borings refer to the depth from the existing grade or ground surface present at the time of the investigation, and the boundaries between the various soil types are approximate.

Fill soils are present at the surface in Boring B-3 extending to a depth of 9 feet below existing grade. The fills consist of dark brown, brown, and light reddish brown silty clayey sands, clayey sands, and sandy clays containing ironstone nodules, calcareous nodules, gravel, and asphalt fragments.

Natural soils encountered consist of dark brown, brown, light brown, reddish brown, light reddish brown, tan, and gray silty clays, sandy clays, sands, silty sands, clayey sands, and silty clayey sands. The various soils contain iron stains, ironstone nodules, iron seams, calcareous nodules, and gravel. Sand layers are noted within the clayey sands in Boring B-1 below a depth of 10 feet. A 6-inch thick sandstone seam was noted within the silty clayey sands in Boring B-17 at a depth of 1½ feet, and a 1-foot thick fractured sandstone layer was encountered in Boring B-17 at a depth of 2 feet below existing grade.

The various soils encountered in the borings had tested Liquid Limits (LL) ranging from 17 to 47 with Plasticity Indices (PI) ranging from 6 to 31 and are classified as SC, SM, SC-SM, and CL by the USCS. The various clayey soils were generally firm to hard in consistency with pocket penetrometer readings of 1.5 to over 4.5 tsf. Tested unit weight values varied from 105 to 122 pcf and tested unconfined compressive strength values were from 1,560 to 8,220 psf. Select strength tests and pocket penetrometer readings reflect more granular materials, indicating higher in-situ strengths than the tested values.

Brown, light brown, reddish brown, light reddish brown, and tan sands and silty sands were next encountered in the borings at depths of 1 to 14 feet extending to depths of 4 to 32 feet below existing grade. Sand was present at the surface in Borings B-8 through B-11 and B-15. The sands contain iron stains, iron seams, ironstone nodules, calcareous nodules, and gravel. Clay seams are noted within the sands in Boring B-5 below a depth of 8½ feet. Sandy clay layers are noted within the silty sands in Boring B-10 below a depth of 8 feet, and sandy clay seams are noted within the sands in Boring B-15 below a depth of 5 feet. These sands and silty sands were loose to dense in consistency with Standard Penetration (N) values of 6 to 33 hammer blows for 1 foot of penetration.

Tan or tan and gray limestone was next encountered in Borings B-4, B-6, B-7, B-8, B-10, and B-11 at depths of 18 to 32 feet below existing grade. These limestones are considered moderately hard to hard (rock basis) with Texas Cone Penetrometer (THD) values of 1¼ to 2½ inches of penetration for 100 hammer blows.

Gray limestone was next encountered in Borings B-1 through B-11 at depths of 19 to 35 feet extending through termination at depths of 40 to 55 feet below existing grade. The gray limestone often contains shale seams and layers and occasionally contains shaly limestone seams. The gray limestone is considered moderately hard to very hard (rock basis) with Texas Cone Penetrometer (THD) values of ⅜ to 3½ inches of penetration for 100 hammer blows.

The Atterberg Limits tests indicate the various soils encountered at this site vary from generally stable to moderately active with respect to moisture induced volume changes. Active clays can experience volume changes (expansion or contraction) with fluctuations in their moisture content.

3.2 Groundwater Observations

The borings were drilled using continuous flight augers in order to observe groundwater seepage during drilling. Groundwater seepage was encountered during drilling in Borings B-1 through B-3, B-5, B-6, B-8 through B-12, B-14, B-17, and B-18 at depths of 2 to 25 feet with water levels of 4 to 41 feet measured at drilling completion in these borings. Boring B-6 was dry at completion. In addition, borehole cave-in was observed during drilling in Borings B-1 through B-3 and B-8 through B-11 at depths of 10 to 27 feet. No seepage was encountered during drilling or at completion in Borings B-4, B-7, B-13, B-15, and B-16. Table 3.2-1 summarizes the water level data as encountered in the borings.

TABLE 3.2-1

Groundwater Observations

Boring No.	Seepage During Drilling (ft.)	Water at Completion (ft.)
B – 1	15 w/ cave-in at 21	21
B – 2	19 w/ cave-in at 21	22
B – 3	25 w/ cave-in at 22, 27	41
B – 4	Dry	Dry
B – 5	14	25
B – 6	13	Dry
B – 7	Dry	Dry
B – 8	5 w/ cave-in at 25	18
B – 9	10 w/ cave-in at 14, 17, 27	31
B – 10	3 w/ cave-in at 10, 12, 21, 25	4
B – 11	10 w/ cave-in at 20	17
B – 12	4	7
B – 13	Dry	Dry
B – 14	2	7
B – 15	Dry	Dry
B – 16	Dry	Dry
B – 17	7	4
B – 18	4	13

While it is not possible to accurately predict the magnitude of subsurface water fluctuation that might occur based upon these short-term observations, it should be recognized that groundwater conditions will vary with fluctuations in rainfall. Seepage near the observed levels should be anticipated throughout the year.

Fluctuations of the groundwater level can occur due to seasonal variations in the amount of rainfall; site topography and runoff; hydraulic conductivity of soil strata; and other factors not evident at the time the borings were performed. During wet periods of the year seepage can occur in the more granular soils, joints in the clays, or atop/within the tan limestones. The possibility of groundwater level fluctuations should be considered when developing the design and construction plans for the project.

4.0 EXISTING FILLS

Existing fills were present at the surface in Boring B-3 extending to a depth of 9 feet below existing grade. Samples of the fills were reasonably dense and free of significant voids. However, in the absence of documented density control, the possibility of undercompacted zones or voids exists. Removal and replacement of all the fill following the recommendations in subsequent sections of this report is the only method eliminating the risk of unusual settlement.

Methods less extreme than complete removal are discussed in the Foundation Recommendations and Pavements sections of this report. These methods are intended to represent a reasonable approach for construction of on-grade elements and paving; however, they will not eliminate the risk of unexpected movements in some areas.

5.0 FOUNDATION RECOMMENDATIONS

5.1 General Foundation Considerations

Two independent design criteria must be satisfied in the selection of the type of foundation to support the proposed structures. First, the ultimate bearing capacity, reduced by a sufficient factor of safety, must not be exceeded by the bearing pressure transferred to the foundation soils. Second, due to consolidation or expansion of the underlying soils during the operating life of the structures, total and differential vertical movements must be within tolerable limits.

Shallow or near surface footings could be subject to differential movements due to possible indeterminate settlement of the existing fills. The most positive foundation system for the proposed structures would be situated below the fills and below the zone of seasonal moisture variations. In addition, the anticipated column loads indicate a deep foundation system transferring column loads to a suitable bearing stratum is considered the most positive foundation system. Due to the anticipated column loads and possible indeterminate settlement of the existing fills, straight drilled reinforced concrete shafts penetrating the gray limestone with shale seams and layers and shaly limestone seams offer a positive foundation system and are recommended.

Care must be taken not to disturb the foundation system of the existing structures. Differential movements between the additions and the existing structures should be anticipated independent of the type of foundation system used for the addition, unless both are structurally suspended atop similar deep foundation systems.

5.2 Straight Shaft Design Parameters

5.2.1 *Design Criteria*

Recommendations and parameters for the design of cast-in-place straight-shaft drilled piers are outlined below. Specific recommendations for the construction and installation of the straight drilled piers are included in the following section, and shall be followed during construction.

Bearing Stratum	Gray LIMESTONE, w/ shale seams and layers, and shaly limestone seams
Depth of Bearing Stratum:	Approximately 19 to 35 feet below existing grades (El. 545 to 567)
Required Penetration/Depth:	All piers should penetrate into the bearing stratum a minimum of 2 feet. Deeper penetrations may be required to develop additional skin friction and/or uplift resistance.
Allowable End Bearing Capacity:	30,000 psf
Allowable Skin Friction:	Applicable below a minimum penetration of 2 feet into the gray limestone and below any temporary casing; 4,800 psf for compressive loads and 3,600 psf for tensile loads.

The above values contain a safety factor of three (3). Drilled shafts should extend through any clay or weathered seams and bear only in unweathered gray limestone. Penetrations greater than the minimum penetration may be required to develop additional skin friction and/or uplift resistance.

It should be anticipated that groundwater seepage and caving soils will be encountered above the bearing stratum during installation of all straight shafts. Temporary casing will likely be required for proper installation of all shafts; however, in the event the casing cannot seal off the groundwater, underwater/slurry concrete placement techniques would be necessary to properly install the shafts. *In underwater/slurry concrete placement techniques, end bearing is neglected and the shaft design is based entirely on skin friction. This will require deeper penetrations*. **Test shafts are recommended to determine the need for underwater/slurry concrete placement, or if a temporary casing is capable of adequately sealing off groundwater.**

In order to develop full load carrying capacity in skin friction, adjacent shafts should have a minimum center-to-center spacing of 2.5 times the diameter of the larger shaft. Closer spacing may require some reductions in skin friction and/or changes in installation sequences. Closely spaced shafts should be examined on a case-by-case basis. As a general guide, the design skin friction will vary linearly from the full value at a spacing of 2.5 diameters to 50 percent of the design value at 1.0 diameter.

During construction one of the more important responsibilities of the pier excavation contractor and the construction materials inspection laboratory will be to verify the presence of the bearing materials encountered during construction.

Settlements for properly installed and constructed straight shafts in the gray limestone with shale seams and layers and shaly limestone seams will be primarily elastic and are estimated to be ¾ inch or less.

5.2.2 *Soil Induced Uplift Loads*

The drilled shafts could experience tensile loads as a result of post construction heave in the site soils. The magnitude of these loads varies with the shaft diameter, soil parameters, and particularly the in-situ moisture levels at the time of construction. In order to aid in the structural design of the reinforcement, the reinforcement quantity should be adequate to resist tensile forces based on soil adhesion equal to 750 psf acting over the upper 10 feet of the pier shaft. This load must be resisted by the dead load on the shaft, continuous vertical reinforcing steel in the shaft, and a shaft adhesion developed within the bearing strata as previously discussed for straight shafts.

5.2.3 *Lateral Load Design Values*

Drilled shaft design parameters for use with LPile based on our laboratory test results are presented in the table below together with our recommended design stratigraphy. The design depth interval is referenced from present existing grades. For the limestone, the “Weak Rock” (Reese) model is suitable for use with LPile. Where the ground surface is exposed surrounding a drilled shaft a p-modification factor of 0.1 is appropriate at the surface increasing linearly to the full value at a depth of 10 feet. This reduction is because of the potential for shrinkage cracks forming along the sides of the drilled shafts.

TABLE 5.2.3-1 – RECOMMENDED LATERAL LOAD DESIGN VALUES

<table>
<tr><th>Soil Type</th><th>Design Depth Interval (ft)</th><th>Design Total Unit Weight (pcf)</th><th colspan="2">Design Strength</th><th>Design $\varepsilon 50$ or k_{rm}</th><th>k-value (pci)</th></tr>
<tr><td rowspan="2">Sands, Silty Sands, Clayey Sands, Silty Clayey Sands
(Sand, Reese)</td><td rowspan="2">10-35</td><td rowspan="2">120</td><td>Cohesion (psi)</td><td>Friction Angle (degrees)</td><td rowspan="2">-</td><td rowspan="2">60</td></tr>
<tr><td>0</td><td>30</td></tr>
<tr><td rowspan="2">Gray Limestone w/ Shale Seams and Layers
(Estimated RQD = 75%)</td><td rowspan="2">35+</td><td rowspan="2">140</td><td colspan="2">Uniaxial Compressive Strength (psi)</td><td rowspan="2">0.0001</td><td rowspan="2">E=1x10^5 psi</td></tr>
<tr><td colspan="2">500</td></tr>
</table>

5.2.4 *Drilled Shaft Construction Considerations*

Drilled pier construction should be monitored by a representative of the geotechnical engineer to observe, among other things, the following items:

- Identification of bearing material
- Adequate penetration of the shaft excavation into the bearing layer
- The base and sides of the shaft excavation are clean of loose cuttings
- If seepage is encountered, whether it is of sufficient amount to require the use of temporary steel casing. If casing is needed it is important that the field representative observe that a high head of plastic concrete is maintained within the casing at all times during their extraction to prevent the inflow of water

Precautions should be taken during the placement of reinforcing steel and concrete to prevent loose, excavated soil from falling into the excavation. Concrete should be placed as soon as practical after completion of the drilling, cleaning, and observation. Excavation for a drilled pier should be filled with concrete before the end of the workday, or sooner if required to prevent deterioration of the bearing material. Prolonged exposure or inundation of the bearing surface with water will result in changes in strength and compressibility characteristics. If delays occur, the drilled pier excavation should be deepened as necessary and cleaned, in order to provide a fresh bearing surface.

Shaft excavations should be maintained in the dry. It should be anticipated that groundwater seepage and caving soils will be encountered during installation of all straight shafts and that seepage rates will likely be sufficient to require the use of temporary casing for installation of all straight shafts. The casing should be seated in the bearing stratum with all water and most loose material removed prior to beginning the design penetration. Care must then be taken that a sufficient head of plastic concrete is maintained within the casing during extraction. If the water cannot be controlled, we recommend the concrete be placed by a tremie or by using a concrete pump. If this method is utilized, end bearing should be neglected and the shaft design based entirely on skin friction. *In this case deeper penetrations will be required.* **Test shafts are recommended to determine the need for underwater/slurry concrete placement, or if a temporary casing is capable of adequately sealing off groundwater**.

Tremied or pumped-in concrete for straight shafts should take place as continuously as possible until the concrete placement is complete. The bottom of the discharge pipe should always be kept below the surface of the concrete.

Before tremied or pumped-in concrete is used, care should be taken to ensure that the water is at a stabilized level and muck is removed to as low a level as possible, which will provide for a thin water solution to be displaced during concrete placement. The pipe or tremie is to be plugged when inserted into the pier and lowered until it is resting on the bottom of the hole. It should be filled with concrete and then lifted off the bottom about 1 foot. The concrete should then be placed in a continuous operation until all water is forced out of the hole. The tremie or pipe must always have about 5 feet of pipe into the concrete. Once the water is forced from the pier, the remaining concreting operation will be the same as for a cased hole.

The concrete should have a slump of 6 inches plus or minus 1 inch. Concrete for use in underwater/slurry placements may have a slump of 8 inches plus or minus 1 inch. Where underwater concrete placement techniques are not utilized, the concrete should be placed in a manner to prevent the concrete from striking the reinforcing cage or the sides of the excavation. Concrete should be tremied to the bottom of the excavation to control the maximum free fall of the plastic concrete to less than 10 feet.

In addition to the above guidelines, the specifications from the Association of Drilled Shaft Contractors Inc. "Standards and Specifications for the Foundation Drilling Industry" as Revised 1999 or other recognized specifications for proper installation of drilled shaft foundation systems should be followed.

5.3 Grade Beams and Floor Slabs

5.3.1 *General*

The design of ground-supported grade beams and floor slab support depends on the magnitude of movement that these structural components can tolerate. The potential magnitude of these movements varies with the subsurface conditions over the site. Potential vertical movements were evaluated using the TxDOT Potential Vertical Rise (PVR) Method, and the results of our laboratory-testing program. Based on subsurface conditions encountered and planned Finished Floor Elevations as depicted on the referenced grading plan Sheet CG2 (Level 0 FF = 568.3, Level 0.5 FF = 576.35, Level 1 FF = 588.7) it is estimated that post-construction movements are on the order of 1 inch or less. If grade beams and floor slabs can tolerate movements on the order of 1 inch, grade beams and floor slabs may be placed atop the prepared grade without special soil conditioning.

If such movements are not tolerable, the most positive method of preventing slab distress due to swelling soils and differential soil movement is to structurally suspend the interior slab. Support of the structural floor is provided by the drilled piers. Due to the expansion potential of the site clays, it is recommended that the suspended floor slab and associated grade beams be constructed on carton forms with a minimum 6-inch void space or crawl space. Consult this office for additional recommendations if a suspended floor system is selected.

All fill required to establish finished grade must consist of non-expansive select fill with a Liquid Limit less than 35 and a Plasticity Index (PI) between 4 and 16. The select fill should be compacted in maximum 9-inch loose lifts at minus 2 to plus 3 percentage points of the soil's optimum moisture content at a minimum of 95 percent of Standard Proctor density (ASTM D698).

Select fills placed at or below 10 feet below finished grade should be compacted to 100 percent of Standard Proctor density.

5.3.2 *Connector Building – Existing Fill Considerations*

Floor slabs placed on-grade will be subject to movement as a result of possible indeterminate settlement of the existing fills present at this location (Boring B-3). Reductions in anticipated movements can be achieved by using methods developed in this area to reduce the potential for on-grade slab movements. A more commonly used method consists of re-working the existing fill soils. The materials encountered at this location appear to consist of select fill soils as specified in the previous report section. Therefore, it is recommended that the connector building subgrade be prepared in accordance with report Section 5.3.2.1 prior to foundation installation.

5.3.2.1 *Reworking of Near-Surface Soils with Select Fill – Connector Building*

In general, the procedure is performed as follows:

1. Remove all existing pavements, surface vegetation, trees and associated root mats, organic topsoil and any other deleterious material.
2. Excavate to a minimum of 2 feet below existing grade. The exposed subgrade should be proof rolled using a heavy (25-ton minimum) pneumatic tired roller making several passes over the subgrade. Any soft or spongy areas should be overexcavated to firm materials and backfilled following the recommendations provided in report Section 9.0. The proof rolling operations should be observed by the project geotechnical engineer or his/her representative. Scarify the exposed clay subgrade at the base of the excavation to a depth of 8 inches, adjust the moisture, and compact between minus 2 to plus 3 percentage points above optimum moisture to a minimum of 95 percent Standard Proctor density (ASTM D698). More granular materials may need to be compacted closer to optimum moisture at the discretion of the geotechnical engineer.
3. Complete pad fill using on-site or imported sandy clay/clayey sand non-expansive select fill with a Liquid Limit less than 35 and a Plasticity Index (PI) between 5 and 16. Site excavated soils meeting select fill material specifications may also be used to complete pad fill. The select fill should be compacted in maximum 9-inch loose lifts at minus 2 to plus 3 percentage points of the soil's optimum moisture content at a minimum of 95 percent of Standard Proctor density (ASTM D698). Field density tests should be taken as each lift of select fill material is placed. Each lift should be compacted, tested, and approved before another lift is added. Over-compaction should not be allowed. The select fill should be placed within 48 hours of completing the installation of the moisture conditioned soils.

6.0 EXPANSIVE SOIL CONSIDERATIONS

6.1 Site Drainage

An important feature of the project is to provide positive drainage away from the proposed buildings. If water is permitted to stand next to or below the structures, excessive soil movements (heave) can occur. This could result in differential floor slab or foundation movement.

A well-designed site drainage plan is of utmost importance and surface drainage should be provided during construction and maintained throughout the life of the structures. Consideration should be given to the design and location of gutter downspouts, planting areas, or other features which would produce moisture concentration adjacent to or beneath the structures or paving. Consideration should be given to the use of self-contained, watertight planters. Joints next to the structures should be sealed with a flexible joint sealer to prevent infiltration of surface water. Proper maintenance should include periodic inspection for open joints and cracks and resealing as necessary.

Rainwater collected by the gutter system should be transported by pipe to a storm drain or to a paved area. If downspouts discharge next to the structures onto flatwork or paved areas, the area should be watertight in order to eliminate infiltration next to the building.

6.2 Additional Design Considerations

The following information has been assimilated after examination of numerous projects constructed in active soils throughout the area. It is presented here for your convenience. If these features are incorporated in the overall design of the project, the performance of the structures should be improved.

- Special consideration should be given to completion items outside the building area, such as stairs, sidewalks, signs, etc. They should be adequately designed to sustain the potential vertical movements mentioned in the report.
- Roof drainage should be collected by a system of gutters and downspouts and transmitted away from the structures where the water can drain away without entering the building subgrade.
- Sidewalks should not be structurally connected to the building. They should be sloped away from the building so that water will drain away from the structures.

- The paving and the general ground surface should be sloped away from the buildings on all sides so that water will always drain away from the structures. Water should not be allowed to pond near the building after the slab has been placed.

- Trees and deep rooted shrubs should not be used as landscaping around the structure perimeter as the root systems can lead to desiccation of the subgrade soils. Any trees should be planted at a distance from the building such that the building will not fall within the drip line of the mature plants (usually one to one-and-one-half times the mature height of the tree). If existing tree removal is not an acceptable option, a vertical root barrier, extending to a minimum depth of 4 feet, should be constructed around the perimeter of the foundation in proximity to the area described above.

- Every attempt should be made to limit the extreme wetting or drying of the subsurface soils since swelling and shrinkage will result. Standard construction practices of providing good surface water drainage should be used. A positive slope of the ground away from the foundation should be provided to carry off the run-off water both during and after construction.

- Backfill for utility lines or along the perimeter beams should consist of on-site material so that they will be stable. If the backfill is too dense or too dry, swelling may form a mound along the ditch line. If the backfill is too loose or too wet, settlement may form a sink along the ditch line. Either case is undesirable since several inches of movement is possible and floor cracks are likely to result. The soils should be processed using the previously discussed compaction criteria.

7.0 BELOW GRADE AREAS & RETAINING WALLS

7.1 Permanent Basement Walls

7.1.1 *General*

Below grade walls will either be single formed, double formed or a combination of the two. The type of construction affects the lateral earth pressures acting on the basement walls. Design parameters are provided below for both single and double formed walls.

7.1.2 *Single Formed Wall*

A lateral earth pressure, expressed as an equivalent fluid pressure, of 100 psf/ft is recommended for a rigid single formed wall with a drained condition and a level backfill. Surcharge loads should be included in the wall design where appropriate, as previously discussed.

7.1.3 *Double Formed Wall*

Recommended lateral earth pressures, expressed as equivalent fluid pressures, are presented below for a rigid double formed wall with a drained condition and a level backfill behind the top of the wall. The equivalent fluid pressure for an undrained condition should be used if a drainage system is not present to remove water trapped in the backfill and behind the wall. Pressures are provided for an at-rest and active earth pressure conditions. In order to allow for an active condition the top of the wall(s) must deflect on the order of 0.4 percent.

TABLE 7.1.3-1 – EQUIVALENT FLUID PRESSURES

Backfill Material	At-Rest Equivalent Fluid Pressure (pcf)		Active Equivalent Fluid Pressure (pcf)	
	Drained	Undrained	Drained	Undrained
Excavated on-site soil fill materials	75	100	60	90
Select fill or on-site soils meeting material specifications	65	90	50	85
Free draining granular backfill material	50	90	35	80

For the select fill or free draining granular backfill, these values assume that a “full” wedge of the material is present behind the wall. The wedge is defined where the wall backfill limits extend outward at least 2 feet from the base of the wall and then upward on a 1H:2V slope. For narrower backfill widths of granular or select fill soils, the equivalent fluid pressures for the on-site soils should be used.

Surcharge loads must be included in the wall design where appropriate, as previously discussed. Piping and electrical conduits through the fill should be designed for potential soil loading due to fill settlement.

Excavated On-Site Soil: For wall backfill areas with site-excavated materials, or similar imported materials all oversized fragments larger than four inches in maximum dimension should be removed from the backfill materials prior to placement. The backfill should be free of all organic and deleterious materials, and should be placed in maximum 8-inch compacted lifts at a minimum of 95 percent of Standard Proctor density (ASTM D698) within a moisture range of plus to minus three (3) percentage points of optimum moisture content. Compaction within five feet of the walls should be accomplished using hand compaction equipment, and should be between 90 and 95 percent of the Standard Proctor Density.

Select Fill (on-site or imported): All wall select backfill should consist of clayey sand and/or sandy clay material with a Plasticity Index of 16 or less, with a Liquid Limit not exceeding 35. The select fill should be placed in maximum 8-inch lifts and compacted to between 95 and 100 percent of Standard Proctor density (ASTM D698) within a moisture range of plus to minus 3 percentage points of the optimum moisture content. Compaction within five feet of the walls should be accomplished using hand compaction equipment and should be compacted between 90 and 95 percent of the Standard Proctor Density.

Free Draining Granular Backfill: All free draining granular wall backfill material should be a crushed stone, sand/gravel mixture, or sand/crushed stone mixture. The material should have less than 3 percent passing the No. 200 sieve and less than 30 percent passing the No. 40 sieve. The minus No. 40 sieve material should be non-plastic. Granular wall backfill should not be water jetted during installation.

7.1.4 *Additional Lateral Pressures*

The location and magnitude of permanent surcharge loads (if present) should be determined, and the additional pressure generated by these loads such as the weight of construction equipment and vehicular loads that are used at the time the structures are being built must also be considered in the design. The effect of this or any other surcharge loading may be accounted for by adding an additional uniform load to the full depth of the side walls equivalent to one-half of the expected vertical surcharge intensity for select backfill materials, or equal to the full vertical surcharge intensity for clay backfill. The equivalent fluid pressures, given here, do not include a safety factor. Analysis of surcharge loads (if any) should be performed on a case-by-case basis. This is not included in the scope of this study. These services can be provided as additional services upon request.

7.2 Retaining Walls

If the retaining walls are sensitive to movements, we recommend they be supported on a deep foundation system as previously discussed. If differential movements as are acceptable, the retaining wall foundations can be supported on footings founded in the natural soils at least 2 feet below existing grade.

The retaining wall foundations may be designed for an allowable bearing pressure of 2.0 ksf. Soils existing in a soft to firm state should be evaluated on a case-by-case basis. Close inspection of soil strength should be conducted by a geotechnical engineer to allow designation and removal of soft soils not meeting the bearing capacity stated above. The base of all excavated footings should be inspected by a geotechnical engineer or geotechnician under his or her supervision to assure that the bottom is firm, level and free of loose soil material and/or debris.

It should be noted that retaining wall foundations are typically subjected to non-uniform pressure across the foundation, and possibly negative pressure (separation of foundation from soil) under a portion of the foundation, due to the overturning moment induced by the lateral earth pressures. The allowable foundation pressures given above are for the maximum pressure induced by the foundation loads, and not the average pressure under the foundation base.

The horizontal bases of the footings will develop resistance to sliding by means of a combination of friction and adhesion (for cohesive foundation materials). Given the primarily sandy nature of the foundation materials, adhesion should be neglected and an ultimate friction factor of 0.45 may be used to calculate sliding resistance of the footings bearing on site soils.

Sliding resistance may be increased in areas where keyways are present beneath the wall footings. The vertical earth-formed sides of keyways will resist lateral forces by developing passive earth pressures. A passive lateral earth pressure coefficient of 2.0 should be used for passive resistance calculations where passive resistance is developed against a vertical earth-formed side of a keyway, based on a soil unit weight of 125 pcf, per foot of footing height.

Foundations for the retaining walls designed in accordance with these recommendations will have a minimum factor of safety of 3 with respect to a bearing capacity failure, and should experience a total settlement of 1 inch or less and a differential settlement of ½ inch or less, after construction.

Lateral earth pressures on retaining walls will depend on a variety of factors, including the type of soils behind the wall, the condition of the soils, and the drainage conditions behind the wall. Recommended lateral earth pressures expressed as equivalent fluid pressures, per foot of wall height, presented in Table 7.1.3-1 for a double formed wall with a level backfill behind the top of the wall are appropriate for retaining walls. The equivalent fluid pressure for an undrained condition should be used if a drainage system is not present to remove water trapped in the backfill and behind the wall. Pressures are provided for at-rest and active earth pressure conditions. Rigid walls are not anticipated to develop enough movement to mobilize active earth pressures. In order to allow for an active condition, the top of the wall(s) must deflect on the order of 0.4 percent. Surcharge loads must be included in the wall design where appropriate, as previously discussed.

7.3 Wall Backfill Settlement

Settlement of the wall backfill should be anticipated. Piping and conduits through the fill should be designed for potential soil loading due to fill settlement. Floor slabs, sidewalls, and pavements over fills may also settle. Backfill compacted to the density recommended above is anticipated to settle on the order of 0.2 to 0.5 percent of the fill thickness.

7.4 Wall Drainage

The equivalent fluid pressures for a single formed wall assume a drained condition. Equivalent fluid pressures for a drained condition were also provided for a double formed wall. Drained conditions must incorporate drainage behind the below grade wall to prevent the development of hydrostatic pressures.

A vertical drain is necessary for single formed walls. This drain may consist of manufactured products such as “Enka-Drain”, “Miradrain”, or other similar systems. The vertical drain should be connected to a permanent perimeter drainage system that is located 12 or more inches lower than the adjacent below grade slab.

For double-formed walls a perimeter drain should be provided. The bottom of the drain should be situated a minimum of 12 inches lower than the adjacent below grade floor slab. The perimeter drain should be a perforated or slotted drain with a minimum pipe diameter of 4 inches and be wrapped in filter fabric for protection against infiltration. Accessible clean-outs should be provided.

For retaining walls drainage could be provided using a collector pipe or weep holes near the base of the retaining wall, with a maximum spacing of 15 feet. Drains should be properly filtered to minimize the potential for erosion through these drains, and /or the plugging of drain lines.

8.0 SEISMIC CONSIDERATIONS

Based on the conditions encountered in the borings for the above referenced project the IBC-2021 site classification is TYPE D for seismic evaluation.

9.0 EARTHWORK

9.1 Site Preparation and Material Requirements

The project site should be stripped of vegetation, roots, old construction debris, and other organic material. It is estimated that the depth of stripping will be on the order of 4 to 6 inches. The actual stripping depth should be based on field observations with particular attention given to old drainage areas, uneven topography, and excessively wet soils. The stripped areas should be observed to determine if additional excavation is required to remove weak or otherwise objectionable materials that would adversely affect the fill placement or other construction activities.

The subgrade should be firm and able to support the construction equipment without displacement. Soft or yielding subgrade should be corrected and made stable before construction proceeds. The subgrade should be proof rolled to detect soft spots, which if exist, should be excavated to provide a firm and otherwise suitable subgrade. Proof rolling should be performed using a heavy pneumatic tired roller, loaded dump truck, or similar piece of equipment weighing a minimum of 25 tons. The proof rolling operations should be observed by the project geotechnical engineer or his/her representative.

The on-site soils are suitable for use in site grading. Imported general fill (not to be used below building structures) material should be clean soil with a Liquid Limit less than 50 and no rock greater than 4 inches in maximum dimension. The fill materials should be free of vegetation and debris. Spoils from excavations may be used for site grading and general fill, provided 50 percent of the crushed material passes the No. 4 sieve and no particles are greater than 4 inches in maximum dimension.

It is noted that the surficial soils consisted of more granular clayey sands, silty clayey sands, silty sands, and sands. This type of material is difficult to compact, and can be difficult from a trafficability standpoint, particularly when wet. Also, during periods of inclement weather these surface soils can become saturated and subject to pumping. This may require undercutting to a firm subgrade and blending them with more clayey soils or low quantities of cement or removing them entirely.

9.2 Placement and Compaction

Fill material should be placed in loose lifts not exceeding 8 inches in uncompacted thickness. The uncompacted lift thickness should be reduced to 4 inches for structure backfill zones requiring hand-operated power compactors or small self-propelled compactors. The fill material should be uniform with respect to material type and moisture content. Clods and chunks of material should be broken down and the fill material mixed by disking, blading, or plowing, as necessary, so that a material of uniform moisture and density is obtained for each lift. Water required for sprinkling to bring the fill material to the proper moisture content should be applied evenly through each layer.

The fill material should be compacted to a density ranging from 95 to 100 percent of maximum dry density as determined by ASTM D698, Standard Proctor. In conjunction with the compacting operation, the fill material should be brought to the proper moisture content. The moisture content for general earth fill should range from 2 percentage points below optimum to 5 percentage points above optimum (-2 to +5). These ranges of moisture contents are given as maximum recommended ranges. For some soils and under some conditions, the contractor may have to maintain a more narrow range of moisture content (within the recommended range) in order to consistently achieve the recommended density.

Field density tests should be taken as each lift of fill material is placed. As a guide, one field density test per lift for each 5,000 square feet of compacted area is recommended. For small areas or critical areas the frequency of testing may need to be increased to one test per 2,500 square feet. A minimum of 2 tests per lift should be required. The earthwork operations should be observed and tested on a continuing basis by an experienced geotechnician working in conjunction with the project geotechnical engineer.

Each lift should be compacted, tested, and approved before another lift is added. The purpose of the field density tests is to provide some indication that uniform and adequate compaction is being obtained. The actual quality of the fill, as compacted, should be the responsibility of the contractor and satisfactory results from the tests should not be considered as a guarantee of the quality of the contractor's filling operations.

If fill is to be placed on existing slopes that are steeper than five horizontal to one vertical, then the fill materials should be benched into the existing slopes in such a manner as to provide a good contact between the two materials and allow relatively horizontal lift placement.

Permanent slopes at the site should be as flat as practical to reduce creep and occurrence of shallow slides. The following slope angles are recommended as maximums.

TABLE 9.2-1 Maximum Permanent Slope Angles

Height (ft.)	Horizontal to Vertical
0 – 3	1:1
3 – 6	2:1
6 – 9	3:1
> 9	4:1

The above angles refer to the total height of a slope. Site improvement should be maintained away from the top of the slope to reduce the possibility of damage due to creep or shallow slides.

9.3 Trench Backfill

Trench backfill for pipelines or other utilities should be properly placed and compacted. Overly dense or dry backfill can swell and create a mound along the completed trench line. Loose or wet backfill can settle and form a depression along the completed trench line. Distress to overlying structures, pavements, etc. is likely if heaving or settlement occurs. On-site soil fill material is recommended for trench backfill. Care should be taken not to use free draining granular material, to prevent the backfilled trench from becoming a french drain and piping surface or subsurface water beneath structures, pipelines, or pavements. If a higher class bedding material is required for the pipelines, a lean concrete bedding will limit water intrusion into the trench and will not require compaction after placement. The soil backfill should be placed in approximately 4- to 6-inch loose lifts. The density and moisture content should be as recommended for fill in Section 9.2, Placement and Compaction, of this report. A minimum of one field density test should be taken per lift for each 150 linear feet of trench, with a minimum of 2 tests per lift.

9.4 Excavation

The side slopes of excavations through the overburden soils should be made in such a manner to provide for their stability during construction. Existing structures, pipelines or other facilities, which are constructed prior to or during the currently proposed construction and which require excavation, should be protected from loss of end bearing or lateral support.

Temporary construction slopes and/or permanent embankment slopes should be protected from surface runoff water. Site grading should be designed to allow drainage at planned areas where erosion protection is provided, instead of allowing surface water to flow down unprotected slopes.

Trench safety recommendations are beyond the scope of this report. The contractor must comply with all applicable safety regulations concerning trench safety and excavations including, but not limited to, OSHA regulations.

9.5 Acceptance of Imported Fill

Any soil imported from off-site sources should be tested for compliance with the recommendations for the particular application and approved by the project geotechnical engineer prior to the materials being used. The owner should also require the contractor to obtain a written, notarized certification from the landowner of each proposed off-site soil borrow source stating that to the best of the landowner's knowledge and belief there has never been contamination of the borrow source site with hazardous or toxic materials. The certification should be furnished to the owner prior to proceeding to furnish soils to the site. Soil materials derived from the excavation of underground petroleum storage tanks should not be used as fill on this project.

9.6 Soil Corrosion Potential

Various analytical laboratory tests were performed on selected soil samples. These tests include soluble sulfate, pH, and electrical resistivity. The tests indicate that the subsurface soils are generally nearly non-corrosive to buried ductile iron, cast iron, steel and galvanized pipe and mildly corrosive to corrosive to buried concrete.

The results of these tests are attached on Plate A.26. Standard construction practice for protecting buried pipes and similar facilities in contact with these soils should be used.

9.7 Erosion and Sediment Control

All disturbed areas should be protected from erosion and sedimentation during construction, and all permanent slopes and other areas subject to erosion or sedimentation should be provided with permanent erosion and sediment control facilities. All applicable ordinances and codes regarding erosion and sediment control should be followed.

9.8 Utilities

Care should be taken that utility cuts are not left open for extended periods, and that the cuts are properly backfilled. Backfilling should be accomplished with properly compacted on-site soils, rather than granular materials.

Trench excavations should be sloped or braced in the interest of safety. Attention is drawn to OSHA Safety and Health Standards (29 CFR 1926/1910), Subpart P, regarding trench excavations greater than 5 feet in depth.

10.0 PAVEMENTS

10.1 Pavement Subgrade Preparation

10.1.1 *General*

Subgrade soils are expected to consist of more granular clayey sands, silty sands, silty clayey sands, and sands. Cuts may expose moderately plastic sandy clays in isolated areas. The success of the pavement subgrade is subgrade soil strength and control of water. Adequate subgrade performance can be achieved by modifying or stabilizing existing soils used to construct the pavement subgrade.

The performance of the pavement for this project depends upon several factors including: the characteristics of the supporting soil; the magnitude and frequency of wheel load applications; the quality of construction materials; the contractor's placement and workmanship abilities; and the desired period of design life.

Pavement sections are susceptible to edge distress as edge support deteriorates over time. Therefore, care must be taken to provide and maintain proper edge support. In conjunction with a stabilized subgrade underlying the pavement, it is recommended that the stabilized subgrade extend a minimum of 12 inches beyond each side of the proposed pavement. Maintenance should be provided when edge support deteriorates.

10.1.2 *Pavement Subgrade Treatment*

Pavement performance is impacted by many factors far beyond what is normally included in engineering design. Wherein pavement analyses should include establishing an appropriate thickness of asphalt concrete or Portland Cement concrete and appropriate subgrade remediation/stabilization, other factors such as location of trees adjacent to the existing paving and water conditions in grassed areas adjacent to curbing impact the performance of the pavement.

Lime stabilization is not recommended because the predominate surface soils are granular and lime will not react with most of them. The most conventional option is cement modification, which serves to improve and maintain their support value. Treatment of these soils with cement will improve their subgrade characteristics to support area paving.

In lieu of a cement stabilized subgrade for pavement consisting of Portland cement concrete, the recommended PCC pavement thicknesses presented in Section 10.2 may be increased by 1 inch, and placed atop a properly compacted subgrade.

Alternatively, in lieu of a cement stabilized subgrade, a flexible base meeting TxDOT Item 247, Type A, Grade 1/2 may be utilized on an equal basis and placed atop a properly compacted subgrade. The option of using a flexible base in lieu of cement stabilizing the subgrade presents a relatively quick, straight forward solution to preparing the subgrade prior to pavement placement.

Prior to cement stabilization or compaction, the subgrade should be proofrolled with heavy pneumatic equipment, with particular attention given to areas of existing fill. Any soft or pumping areas should be undercut to a firm subgrade and properly backfilled as described in the Earthwork section. The subgrade should be scarified to a minimum depth of 6 inches and uniformly compacted to a minimum of 95 percent of ASTM D698, to -2 to +4 percentage points of the optimum moisture content determined by that test. It should then be protected and maintained in a moist condition until the pavement is placed. The presence of iron seams, ironstone nodules, calcareous nodules, and gravel in the surficial soils can complicate mixing of the soil and cement.

We recommend a minimum of 5 percent Portland cement be used to modify the subgrade soils. The amount of cement required to stabilize the subgrade should be on the order of 23 pounds per square yard for a 6-inch depth based on a soil dry unit weight of 100 pcf. The cement should be thoroughly mixed and blended with the upper 6 inches of the subgrade (TxDOT Item 275 or similar standard).

The Portland cement should meet the requirements of Item 275 in the Texas Department of Transportation (TxDOT) Standard Specifications for Construction of Highways, Streets and Bridges, 2014 Edition.

The stabilized subgrade should be scarified to a minimum depth of 6 inches and uniformly compacted to a minimum of 98 percent of ASTM Standard Test Method for Moisture-Density Relations of Soil-Cement Mixtures (ASTM D558), to minus 3 to plus 1 percentage points of the optimum moisture content determined by that test. Cement treatment should extend beyond exposed pavement edges to reduce the effects of shrinkage and associated loss of subgrade support. It should then be protected and maintained in a moist condition until the pavement is placed via curing compound or sprinkling. Providing proper curing of the cement treated subgrade cannot be understated. Failure to properly cure the cement treated subgrade can result in undue shrinkage cracking.

We recommend that subgrade stabilization extend to at least one foot beyond pavement edges to aid in reducing pavement movements and cracking along the curb line due to seasonal moisture variations after construction. Each construction area should be shaped to allow drainage of surface water during earthwork operations, and surface water should be pumped immediately from each construction area after each rain and a firm subgrade condition maintained. Water should not be allowed to pond in order to prevent percolation and subgrade softening, and cement should be added to the subgrade after removal of all surface vegetation and debris.

The Texas Transportation Institute has performed studies to reduce “block cracks” common to cement-treated base materials. Microcracking is the application of several vibratory roller passes to a cement-treated base after a short curing stage, typically after one to three days, to create a fine network of cracks. Microcracking is one technique to help reduce the risk of shrinkage cracks in the cement-treated base. The goal of microcracking is to form a network of fine cracks and prevent the wider, more severe cracks from forming. Proper moisture control during cement placement/mixing and curing are also key factors to reducing shrinkage cracking.

After placement and satisfactory compaction of the cement treated subgrade, the base should be moist cured by sprinkling for 48 to 72 hours before microcracking. If performing construction during winter months when average daily temperatures are 60^{o} F or below, moist cure the base at least 96 hours before microcracking. Microcracking should be performed with the same (or equivalent tonnage) steel wheel vibratory roller used for compaction. A minimum 12-ton roller should be used.

Typically three full passes (one pass is down and back) with the roller operating at maximum amplitude and traveling approximately 2 to 3 mph will satisfactorily microcrack the section. After satisfactory completion of microcracking, the subgrade should be moist cured by sprinkling to a total cure time of at least 72 hours from the day of placement.

Surface drainage is critical to the performance of this pavement. Water should be allowed to exit the pavement surface quickly. All pavement construction should be performed in accordance with the following procedures.

10.2 Pavement Sections

The project may include the construction of parking lots and/or drives. At the time of this investigation, site paving plans or vehicle traffic studies were not available. Therefore, several rigid and flexible pavement sections are presented for a 20-year design life based on our experience with similar facilities for Light-Duty Parking Areas, Medium-Duty Parking Areas and Drives, and Medium- to Heavy-Duty Drives. In general, these areas are defined as follows:

> Light-Duty Parking Areas are those lots and drives subjected almost exclusively to passenger cars, with an occasional light- to medium-duty truck (2 to 3 per week)
>
> Medium-Duty Parking Areas and Drives are those lots subjected to a variety of light-duty vehicles to medium-duty vehicles and an occasional heavy-duty truck or 85-kip fire truck (1 to 2 per week).
>
> Medium- to Heavy-Duty Drives are those drives subjected to a variety of light to heavy-duty vehicles. These pavements include areas subject to significant truck and 85-kip fire apparatus traffic or trash vehicles.

We recommend that rigid pavements be utilized at this project whenever possible, since they tend to provide better long-term performance when subjected to significant slow moving and turning traffic.

If asphaltic concrete pavement is used, we recommend a full depth asphaltic concrete section having a minimum total thickness of 5 inches for light-duty parking areas, 6 inches for medium-duty parking areas and drives, and 8 inches for medium- to heavy-duty drives. A minimum surface course thickness of 2 inches is recommended for asphaltic concrete pavements.

If Portland cement concrete pavement is used, a minimum thickness of 5 inches of concrete is recommended for light-duty parking areas, 6 inches for medium-duty parking areas and drives, and 7 inches for medium- to heavy-duty areas.

A California Bearing Ratio or other strength tests were not performed because they were not within the scope of our services on this project. A subgrade modulus of 100 psi was considered appropriate for the near-surface soils. If heavier vehicles are planned, the above cross sections can be confirmed by performing strength tests on the subgrade materials once the traffic characteristics are established. Periodic maintenance of pavement structures normally improves the durability of the overall pavement and enhances its expected life.

The above sections should be considered minimum pavement thicknesses and higher traffic volumes and heavy trucks may require thicker pavement sections. Additional recommendations can be provided after traffic volumes and loads are known. Periodic maintenance should be anticipated for minimum pavement thickness. This maintenance should consist of sealing cracks and timely repair of isolated distressed areas.

10.3 Pavement Material Requirements

Reinforced Portland Cement Concrete: Reinforced Portland cement concrete pavement should consist of Portland cement concrete having a 28-day compressive strength of at least 3,500 psi. The mix should be designed in accordance with the ACI Code 318 using 3 to 6 percent air entrainment. The pavement should be adequately reinforced with temperature steel and all construction joints or expansion/contraction joints should be provided with load transfer dowels. The spacing of the joints will depend primarily on the type of steel used in the pavement. We recommend using No. 3 steel rebar spaced at 18 inches on center in both the longitudinal and transverse direction. Control joints formed by sawing are recommended every 12 to 15 feet in both the longitudinal and transverse direction. The cutting of the joints should be performed as soon as the concrete has “set-up” enough to allow for sawing operations.

Hot Mix Asphaltic Concrete Surface Course: Item 340, Type D, Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, 2014 Edition.

Hot Mix Asphaltic Concrete Base Course: Item 340, Type A or B, Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, 2014 Edition.

Cement Stabilized Subgrade: Cement treatment for base course (road mix) - Item 275, Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, 2014 Edition.

Flexible Base: Crushed Stone Flexible Base – Item 247, Type A, Grade 1/2, Texas Department of Transportation Standard Specifications for Construction of Maintenance of Highways, Streets, and Bridges, 2014 Edition.

10.4 General Pavement Considerations

The design of the pavement drainage and grading should consider the potential for differential ground movement due to future soil swelling on the order of 1 inch. In order to minimize rainwater infiltration through the pavement surface, and thereby minimizing future upward movement of the pavement slabs, all cracks and joints in the pavement should be sealed on a routine basis after construction.

11.0 CONSTRUCTION OBSERVATIONS

In any geotechnical investigation, the design recommendations are based on a limited amount of information about the subsurface conditions. In the analysis, the geotechnical engineer must assume the subsurface conditions are similar to the conditions encountered in the borings. However, quite often during construction anomalies in the subsurface conditions are revealed. Therefore, it is recommended that CMJ Engineering, Inc. be retained to observe earthwork and foundation installation and perform materials evaluation during the construction phase of the project. This enables the geotechnical engineer to stay abreast of the project and to be readily available to evaluate unanticipated conditions, to conduct additional tests if required and, when necessary, to recommend alternative solutions to unanticipated conditions. Until these construction phase services are performed by the project geotechnical engineer, the recommendations contained in this report on such items as final foundation bearing elevations, proper soil moisture condition, and other such subsurface related recommendations should be considered as preliminary.

It is proposed that construction phase observation and materials testing commence by the project geotechnical engineer at the outset of the project. Experience has shown that the most suitable method for procuring these services is for the owner or the owner's design engineers to contract directly with the project geotechnical engineer. This results in a clear, direct line of communication between the owner and the owner's design engineers and the geotechnical engineer.

12.0 REPORT CLOSURE

The boring logs shown in this report contain information related to the types of soil encountered at specific locations and times and show lines delineating the interface between these materials. The logs also contain our field representative's interpretation of conditions that are believed to exist in those depth intervals between the actual samples taken. Therefore, these boring logs contain both factual and interpretive information. Laboratory soil classification tests were also performed on samples from selected depths in the borings. The results of these tests, along with visual-manual procedures were used to generally classify each stratum. Therefore, it should be understood that the classification data on the logs of borings represent visual estimates of classifications for those portions of each stratum on which the full range of laboratory soil classification tests were not performed. It is not implied that these logs are representative of subsurface conditions at other locations and times.

With regard to groundwater conditions, this report presents data on groundwater levels as they were observed during the course of the field work. In particular, water level readings have been made in the borings at the times and under conditions stated in the text of the report and on the boring logs. It should be noted that fluctuations in the level of the groundwater table can occur with passage of time due to variations in rainfall, temperature and other factors. Also, this report does not include quantitative information on rates of flow of groundwater into excavations, on pumping capacities necessary to dewater the excavations, or on methods of dewatering excavations. Unanticipated soil conditions at a construction site are commonly encountered and cannot be fully predicted by mere soil samples, test borings or test pits. Such unexpected conditions frequently require that additional expenditures be made by the owner to attain a properly designed and constructed project. Therefore, provision for some contingency fund is recommended to accommodate such potential extra cost.

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our field investigation and further on the assumption that the exploratory borings are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the borings at the time they were completed. If, during construction, different subsurface conditions from those encountered in our borings are observed, or appear to be present in excavations, we must be advised promptly so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between submission of this report and the start of the work at the site, if conditions have changed due either to natural causes or to construction operations at or adjacent to the site, or if structure locations, structural loads or finish grades are changed, we urge that we be promptly informed and retained to review our report to determine the applicability of the conclusions and recommendations, considering the changed conditions and/or time lapse.

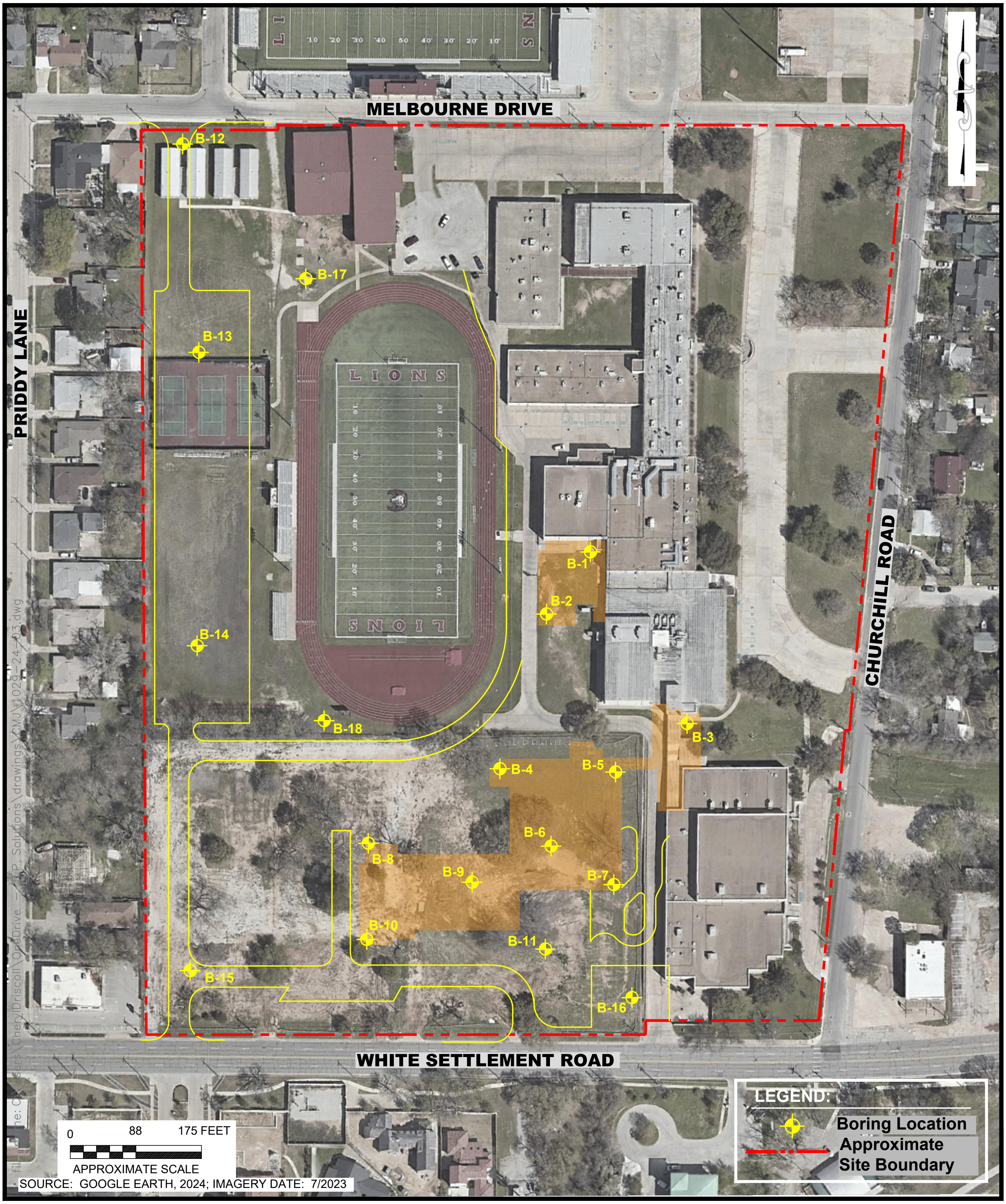
Further, it is urged that CMJ Engineering, Inc. be retained to review those portions of the plans and specifications for this particular project that pertain to earthwork and foundations as a means to determine whether the plans and specifications are consistent with the recommendations contained in this report. In addition, we are available to observe construction, particularly the compaction of structural fill, or backfill and the construction of foundations as recommended in the report, and such other field observations as might be necessary.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater or air, on or below or around the site.

This report has been prepared for use in developing an overall design concept. Paragraphs, statements, test results, boring logs, diagrams, etc. should not be taken out of context, nor utilized without a knowledge and awareness of their intent within the overall concept of this report. The reproduction of this report, or any part thereof, supplied to persons other than the owner, should indicate that this study was made for design purposes only and that verification of the subsurface conditions for purposes of determining difficulty of excavation, trafficability, etc. are responsibilities of the contractor.

This report has been prepared for the exclusive use of Castleberry Independent School District and their consultants for specific application to design of this project. The only warranty made by us in connection with the services provided is that we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, expressed or implied, is made or intended. These recommendations should be reviewed once a grading plan is finalized.

* * * *



SOURCE: GOOGLE EARTH, 2024; IMAGERY DATE: 7/2023

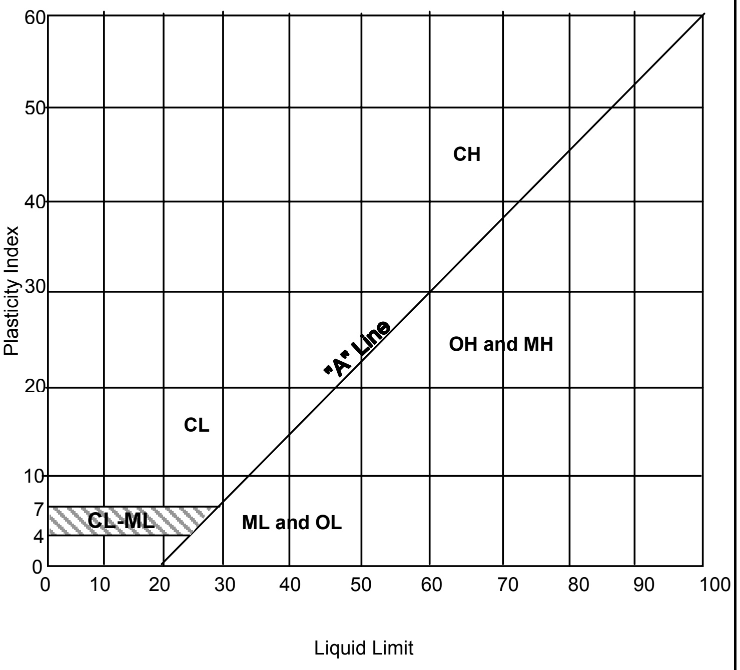
CMJ Engineering

CMJ Project No. 1029-24-03

PLAN OF BORINGS

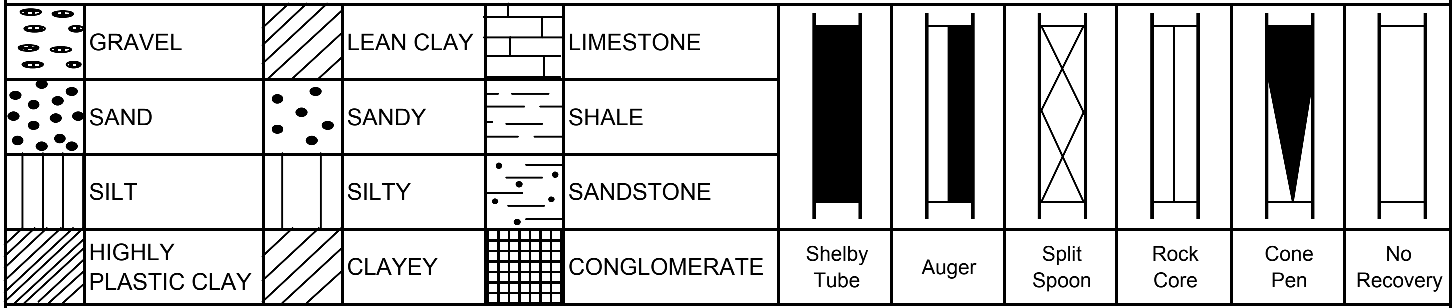
Castleberry High School Additions
215 Churchill Road - Castleberry I.S.D.
Fort Worth, Texas

Plate
A.1

Major Divisions			Grp. Sym.	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (more than half of the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent..........................GW, GP, SW, SP More than 12 percent........................GM, GC, SM, SC 5 to 12 percent..........................*Borderline* cases requiring dual symbols	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4: $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures		Liquid and Plastic limits below "A" line or P.I. greater than 4	Liquid and plastic limits plotting in hatched zone between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Liquid and Plastic limits above "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6: $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands; gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures		Liquid and Plastic limits below "A" line or P.I. less than 4	Liquid and plastic limits plotting between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Liquid and Plastic limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	 Plasticity Chart		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, and lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
	Highly Organic soils		Pt	Peat and other highly organic soils			

UNIFIED SOIL CLASSIFICATION SYSTEM **PLATE A.2**

SOIL OR ROCK TYPES



TERMS DESCRIBING CONSISTENCY, CONDITION, AND STRUCTURE OF SOIL

Fine Grained Soils (More than 50% Passing No. 200 Sieve)

Descriptive Item	Penetrometer Reading, (tsf)
Soft	0.0 to 1.0
Firm	1.0 to 1.5
Stiff	1.5 to 3.0
Very Stiff	3.0 to 4.5
Hard	4.5+

Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)

Penetration Resistance (blows/foot)	Descriptive Item	Relative Density
0 to 4	Very Loose	0 to 20%
4 to 10	Loose	20 to 40%
10 to 30	Medium Dense	40 to 70%
30 to 50	Dense	70 to 90%
Over 50	Very Dense	90 to 100%

Soil Structure

Calcareous	Contains appreciable deposits of calcium carbonate; generally nodular
Slickensided	Having inclined planes of weakness that are slick and glossy in appearance
Laminated	Composed of thin layers of varying color or texture
Fissured	Containing cracks, sometimes filled with fine sand or silt
Interbedded	Composed of alternate layers of different soil types, usually in approximately equal proportions

TERMS DESCRIBING PHYSICAL PROPERTIES OF ROCK

Hardness and Degree of Cementation

Very Soft or Plastic	Can be remolded in hand; corresponds in consistency up to very stiff in soils
Soft	Can be scratched with fingernail
Moderately Hard	Can be scratched easily with knife; cannot be scratched with fingernail
Hard	Difficult to scratch with knife
Very Hard	Cannot be scratched with knife
Poorly Cemented or Friable	Easily crumbled
Cemented	Bound together by chemically precipitated material; Quartz, calcite, dolomite, siderite, and iron oxide are common cementing materials.

Degree of Weathering

Unweathered	Rock in its natural state before being exposed to atmospheric agents
Slightly Weathered	Noted predominantly by color change with no disintegrated zones
Weathered	Complete color change with zones of slightly decomposed rock
Extremely Weathered	Complete color change with consistency, texture, and general appearance approaching soil

KEY TO CLASSIFICATION AND SYMBOLS **PLATE A.3**

CMJ ENGINEERING INC.

Project No.	Boring No.	Project
1029-24-03	B-1	**Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**

Location: **See Plate A.1**

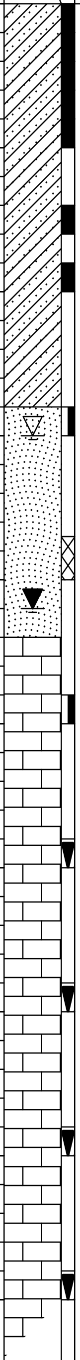
Water Observations: **Seepage at 15' with borehole cave-in at 21' during drilling; water at 21' at completion**

Completion Depth: **45.0'**

Completion Date: **3-11-24**

Surface Elevation: **589.0**

Type: **B-53, w/ CFA**



Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		CLAYEY SAND, reddish brown, w/ iron seams, ironstone nodules, and gravel			4.0					12		
					2.25					12		
					1.5					13		
					1.25	44	26	14	12	17		
					3.5					14	118	2650
5												
					2.5					13		
					4.5+					15		
10		- w/ sand layers below 10'										
	575.0	**SAND**, tan, w/ iron seams, ironstone nodules, and gravel, medium dense								12		
15												
		- reddish brown below 18.5'										
					21	9				5		
20												
	567.0	- cave-in at 21' during drilling										
		LIMESTONE, gray, hard to very hard										
25												
					100/1"							
30												
					100/0.625"							
35		- w/ shale seams below 35'										
					100/0.5"							
40												
					100/0.375"							
45	544.0											

CMJ ENGINEERING INC.

Project No.	Boring No.	Project
1029-24-03	**B-2**	**Castleberry High School Additions 215 Churchill Road - Fort Worth, Texas**

Location: **See Plate A.1**

Water Observations: **Seepage at 19' with borehole cave-in at 21' during drilling; water at 22' at completion**

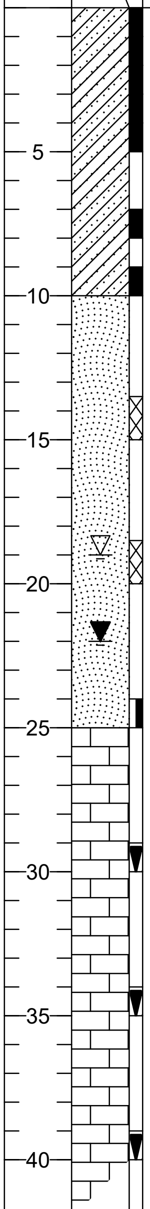
Completion Depth: **40.0'**

Completion Date: **3-11-24**

Surface Elevation: **588.0**

Type: **B-53, w/ CFA**

Stratum Description



CLAYEY SAND, reddish brown and light reddish brown, w/ iron seams and ironstone nodules

578.0

SAND, light brown, w/ gravel, medium dense

- cave-in at 21' during drilling

563.0

LIMESTONE, gray, w/ shale seams and layers, hard to very hard

548.0

REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		1.0					10		
		0.75					6		
		4.0					14		
		4.5+	35	29	15	14	8	109	
		2.5					8		
		2.5					8	105	2330
		4.5+					8		
		22							
		29							
		100/1.25"							
		100/1.5"							
		100/0.75"							

Project No. **1029-24-03**

Boring No. **B-3**

Project **Castleberry High School Additions**
215 Churchill Road - Fort Worth, Texas

Location **See Plate A.1**

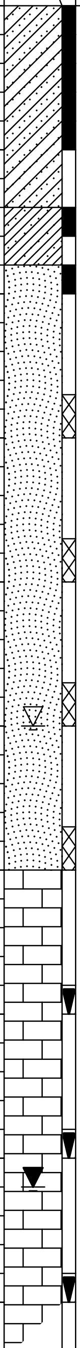
Water Observations **Seepage at 25' with borehole cave-in at 22' and 27' during drilling; water at 41' at completion**

Completion Depth **45.0'**

Completion Date **3-11-24**

Surface Elevation **575.0**

Type **B-53, w/ CFA**



Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SILTY CLAYEY SAND, dark brown, brown, and light reddish brown, w/ ironstone nodules, gravel, and asphalt fragments (FILL)			2.0					13		
					4.5+					9	118	2300
					0.75		20	13	7	9		
					0.5					10		
		- w/ calcareous nodules below 4'			0.25					4		
5												
	568.0	**SANDY CLAY / CLAYEY SAND**, light reddish brown, w/ iron seams, ironstone nodules, calcareous nodules, and asphalt fragments, hard (FILL)			4.5+	53	29	12	17	9		
	566.0				0.25					4		
10		**SAND**, tan and light reddish brown, w/ calcareous nodules and gravel, loose to medium dense										
					9	10				2		
15												
					14							
20												
		- cave-in at 22' during drilling										
					24							
25												
		- cave-in at 27' during drilling										
		- dense below 28.5'			33							
30	545.0	**LIMESTONE**, gray, w/ shale seams and layers, hard to very hard										
					100/1.25"							
35												
					100/0.5"							
40												
	530.0				100/0.75"							
45												

CMJ ENGINEERING INC.

Project No. **1029-24-03**

Boring No. **B-4**

Project **Castleberry High School Additions**
215 Churchill Road - Fort Worth, Texas

Location **See Plate A.1**

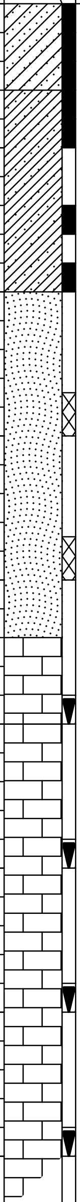
Water Observations **Dry during drilling; dry at completion**

Completion Depth **40.0'**

Completion Date **3-13-24**

Surface Elevation **586.0**

Type **CME 55, w/ CFA**



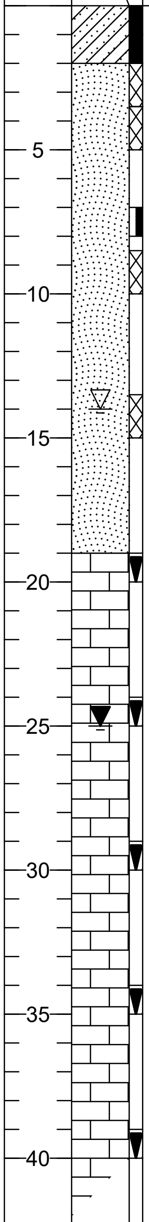
Depth, Ft.	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
	CLAYEY SAND, dark brown, w/ iron seams and ironstone nodules			2.0					20		
	- reddish brown below 1'			0.5					10		
	583.0			0.5					15		
	SANDY CLAY, light reddish brown, w/ iron seams and ironstone nodules, firm to stiff			1.5					16	115	1560
5	- very stiff to hard below 4'			3.75	57	45	14	31	16	113	
				4.5+					16		
10	- w/ calcareous nodules below 9' 576.0			3.25					16		
15	**SAND**, light brown, medium dense			16							
20	- w/ gravel below 18'			20							
25	564.0 **LIMESTONE**, tan and gray, moderately hard 561.0			100/2.25"							
30	**LIMESTONE**, gray, w/ shaly limestone seams, moderately hard to hard			100/1.75"							
35				100/2"							
40	546.0			100/1.25"							

LOG OF BORING 1029-24-03.GPJ CMJ.GDT 4/19/24

LOG OF BORING NO. **B-4** **PLATE A.7**

CMJ ENGINEERING INC.

Project No. **1029-24-03**	Boring No. **B-5**	Project **Castleberry High School Additions 215 Churchill Road - Fort Worth, Texas**
Location **See Plate A.1**		Water Observations **Seepage at 14' during drilling; water at 25' at completion**
Completion Depth **40.0'**	Completion Date **3-13-24**	
Surface Elevation **577.0**		Type **CME 55, w/ CFA**



Depth, Ft.	Elev.	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SILTY CLAYEY SAND, brown and reddish brown, w/ iron stains and ironstone nodules			0.5					13		
	575.0				1.0		22	16	6	5		
		SAND, light brown, loose to medium dense			7							
5					8							
10		- w/ clay seams below 8.5'			11							
15		- w/ gravel below 13.5'			20							
	558.0											
20		**LIMESTONE**, gray, moderately hard to hard			100/3.5"							
25					100/2"							
30					100/1.75"							
35		- w/ shale seams and shaly limestone seams below 32'			100/2.5"							
40	537.0				100/2"							

LOG OF BORING NO. **B-5**

PLATE A.8

Project No. **1029-24-03**

Boring No. **B-6**

Project **Castleberry High School Additions**
215 Churchill Road - Fort Worth, Texas

CMJ ENGINEERING INC.

Location **See Plate A.1**

Water Observations **Seepage at 13' during drilling; dry at completion**

Completion Depth **45.0'**

Completion Date **3-13-24**

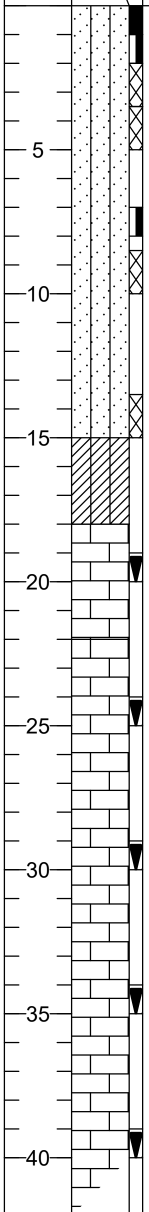
Surface Elevation **583.0**

Type **CME 55, w/ CFA**

Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
	581.0	**CLAYEY SAND**, light brown, w/ iron stains and ironstone nodules			0.25					4		
		SAND, light brown, loose to medium dense			6							
5					10							
10					12							
		- w/ gravel below 13'										
15	566.0				18							
20	561.0	**SILTY CLAY**, light brown, w/ ironstone nodules, calcareous nodules, and calcareous deposits, very stiff			4.0		47	16	31	17		
25	556.0	**LIMESTONE**, tan and gray, moderately hard to hard			100/2"							
30		**LIMESTONE**, gray, moderately hard to hard			100/2"							
35					100/1.25"							
40					100/1.75"							
		- w/ shale seams and shaly limestone seams below 41'										
45	538.0				100/2"							

LOG OF BORING NO. **B-6**

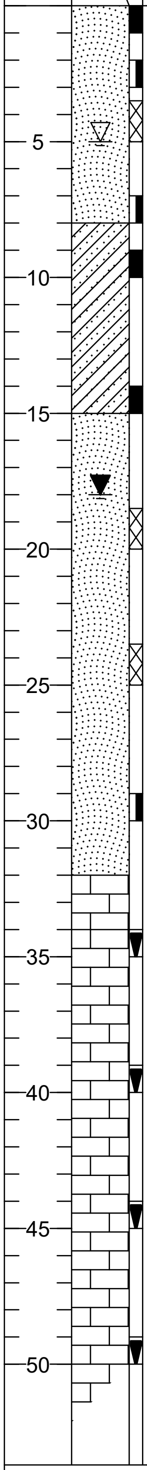
Project No. **1029-24-03**	Boring No. **B-7**	Project **Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**
Location **See Plate A.1**		Water Observations **Dry during drilling; dry at completion**
Completion Depth **40.0'**	Completion Date **3-14-24**	
Surface Elevation **577.0**		Type **CME 55, w/ CFA**



Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SILTY SAND, brown, w/ iron stains and ironstone nodules, loose to medium dense			0.5		NP	NP	NP	6		
		- reddish brown below 2'			9							
					9							
					11	18				8		
	562.0	- w/ gravel below 14'			12							
		SILTY CLAY, light brown										
	559.0	**LIMESTONE**, tan, hard			100/1.25"							
	555.0	**LIMESTONE**, gray, hard to very hard			100/1.25"							
					100/1"							
					100/1.125"							
	537.0				100/1"							

Project No. 1029-24-03	Boring No. **B-8**	Project **Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**
Location **See Plate A.1**		Water Observations **Seepage at 5' with borehole cave-in at 25' during drilling; water at 18' at completion**
Completion Depth **50.0'**	Completion Date **3-13-24**	
Surface Elevation **591.0**	Type **B-53, w/ CFA**	

CMJ ENGINEERING INC.



Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SAND, brown, w/ iron stains and ironstone nodules, loose to medium dense			0.25					5		
5		- reddish brown below 3.5'			10	10				8		
10	583.0	**SILTY CLAYEY SAND**, light reddish brown, w/ iron seams, ironstone nodules, and calcareous nodules			1.5					15	117	2800
15					3.0	33	22	15	7	12	115	
	576.0	**SAND**, tan and brown, medium dense										
20					13							
25		- w/ gravel below 23.5' - cave-in at 25' during drilling			23							
30												
	559.0	**LIMESTONE**, tan										
35	557.0	**LIMESTONE**, gray, w/ shale seams, very hard			100/0.875"							
40					100/0.75"							
45					100/0.625"							
50	541.0				100/0.5"							

CMJ ENGINEERING INC.

Project No.	Boring No.	Project
1029-24-03	**B-9**	**Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**

Location **See Plate A.1**

Water Observations **Seepage at 10' with borehole cave-in at 14', 17', and 27' during drilling; water at 31' at completion**

Completion Depth **50.0'**

Completion Date **3-13-24**

Surface Elevation **588.0**

Type **B-53, w/ CFA**

Depth, Ft.	Elev.	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SILTY SAND, reddish brown, w/ iron seams and ironstone nodules, loose			0.5					8		
5		- light reddish brown below 3.5'			9	20				12		
	580.0	**CLAYEY SAND**, light reddish brown, w/ iron seams, ironstone nodules, and calcareous nodules			3.0	46	26	14	12	15	115	
10	576.0	**SAND**, tan and brown, medium dense										
15		- cave-in at 14' during drilling			14							
		- cave-in at 17' during drilling										
20		- w/ gravel below 20'										
25	560.0	- cave-in at 27' during drilling										
30		**LIMESTONE**, gray, w/ shale seams and layers, hard to very hard			100/1.25"							
35					100/0.5"							
40					100/0.375"							
45					100/0.625"							
50	538.0				100/0.375"							

LOG OF BORING NO. **B-9**

PLATE A.12

CMJ ENGINEERING INC.

Project No. **1029-24-03**

Boring No. **B-10**

Project **Castleberry High School Additions**
215 Churchill Road - Fort Worth, Texas

Location **See Plate A.1**

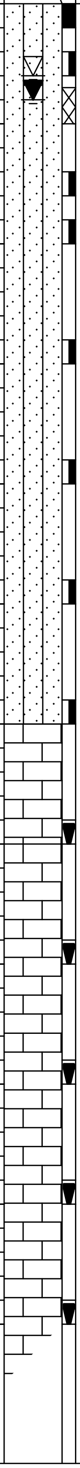
Water Observations
Seepage at 3' with borehole cave-in at 10', 12', 21', and 25' during drilling; water at 4' at completion

Completion Depth **55.0'**

Completion Date **3-13-24**

Surface Elevation **590.0**

Type **B-53, w/ CFA**



Depth, Ft.		Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SILTY SAND, light reddish brown, w/ iron seams and ironstone nodules, loose			0.25		NP	NP	NP	6		
5					8	16				19		
		- w/ sandy clay layers below 8'										
10										15		
		- cave-in at 10' during drilling										
		- cave-in at 12' during drilling										
15												
20												
		- cave-in at 21' during drilling - w/ gravel below 22'										
25		- cave-in at 25' during drilling										
30	560.0	**LIMESTONE**, tan, moderately hard										
35	555.0				100/2.5"							
		LIMESTONE, gray, w/ shale seams and layers, very hard										
40					100/0.5"							
45					100/0.75"							
50					100/0.375"							
55	535.0				100/0.375"							

LOG OF BORING 1029-24-03.GPJ CMJ.GDT 4/19/24

Project No.	Boring No.	Project
1029-24-03	**B-11**	**Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**

Location **See Plate A.1**

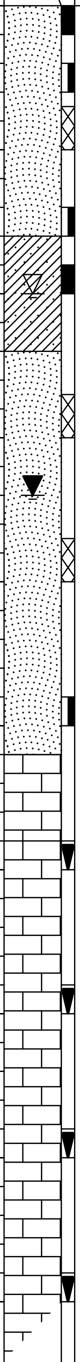
Water Observations **Seepage at 10' with borehole cave-in at 20' during drilling; water at 17' at completion**

Completion Depth **45.0'**

Completion Date **3-14-24**

Surface Elevation **589.0**

Type **B-53, w/ CFA**



Stratum Description

Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
0		**SAND**, brown, w/ iron stains and ironstone nodules, loose			0.25					6		
5					7	11				6		
8	581.0	**SANDY CLAY**, light brown and reddish brown, w/ iron seams, ironstone nodules, and calcareous nodules, stiff										
10					2.0		27	12	15	15	119	2200
12	577.0	**SAND**, light reddish brown, w/ iron stains and ironstone nodules, medium dense										
15					15	9				4		
20					18							
20		- w/ gravel below 20' - cave-in at 20' during drilling										
26	563.0	**LIMESTONE**, tan										
29	560.0	**LIMESTONE**, gray, w/ shale seams, hard to very hard										
30					100/1"							
35					100/0.375"							
40					100/0.75"							
45	544.0				100/0.375"							

Project No.	Boring No.	Project
1029-24-03	**B-12**	**Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**

CMJ ENGINEERING INC.

Location: **See Plate A.1**

Water Observations: **Seepage at 4' during drilling; water at 7' at completion**

Completion Depth: **8.0'**

Completion Date: **3-12-24**

Surface Elevation: **590.0**

Type: **B-53, w/ CFA**

Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
	589.0	**CLAYEY SAND**, dark brown, w/ iron stains and ironstone nodules			1.0	17				10		
	586.0	**SAND**, tan										
5		**SANDY CLAY / CLAYEY SAND**, light brown and reddish brown, w/ iron seams and ironstone nodules, stiff			2.75					15	118	2310
	582.0				2.75		29	13	16	15		

LOG OF BORING NO. **B-12**

PLATE A.15

Project No.	Boring No.	Project
1029-24-03	**B-13**	**Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**

Location **See Plate A.1**

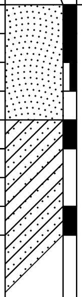
Water Observations **Dry during drilling; dry at completion**

Completion Depth **8.0'**

Completion Date **3-12-24**

Surface Elevation **591.0**

Type **B-53, w/ CFA**



Depth, Ft.	Symbol	Samples	Stratum Description		REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
				SAND, tan, w/ iron stains and ironstone nodules			0.5	9				6		
			587.0											
5				**CLAYEY SAND**, reddish brown, w/ iron stains and ironstone nodules			1.25					13		
			583.0				0.75					12		

CMJ ENGINEERING INC.

Project No. **1029-24-03**

Boring No. **B-14**

Project **Castleberry High School Additions**
215 Churchill Road - Fort Worth, Texas

Location **See Plate A.1**

Water Observations **Seepage at 2' during drilling; water at 7' at completion**

Completion Depth **8.0'**

Completion Date **3-12-24**

Surface Elevation **591.0**

Type **B-53, w/ CFA**

Depth, Ft.	Symbol	Samples		Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
	∇	■		**SAND**, brown, w/ iron stains and ironstone nodules - tan below 1'			0.75	8				7		
			587.0											
5		■		**SANDY CLAY**, light brown, reddish brown, and gray, w/ iron seams and ironstone nodules, stiff to very stiff			3.0		32	12	20	16		
	▼	■	583.0				2.75					15	117	4700

Project No. **1029-24-03**	Boring No. **B-15**	Project **Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**
Location **See Plate A.1**		Water Observations **Dry during drilling; dry at completion**
Completion Depth **8.0'**	Completion Date **3-14-24**	
Surface Elevation **591.0**		Type **B-53, w/ CFA**

Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SAND, tan, w/ iron stains and ironstone nodules, medium dense			0.25	10				6		
					15							
5		- w/ sandy clay seams below 5'										
	583.0											

CMJ ENGINEERING INC.

Project No. **1029-24-03**

Boring No. **B-16**

Project **Castleberry High School Additions**
215 Churchill Road - Fort Worth, Texas

Location **See Plate A.1**

Water Observations **Dry during drilling; dry at completion**

Completion Depth **8.0'**

Completion Date **3-14-24**

Surface Elevation **573.0**

Type **CME 55, w/ CFA**

Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
		SILTY CLAYEY SAND, brown, light brown, and light reddish brown, w/ iron stains and ironstone nodules			0.75	15	17	11	6	8		
	571.0				0.75					5		
		SAND, light brown, loose			7							
5					8							
	565.0				9							

CMJ ENGINEERING INC.

Project No. **1029-24-03**

Boring No. **B-17**

Project **Castleberry High School Additions**
215 Churchill Road - Fort Worth, Texas

Location **See Plate A.1**

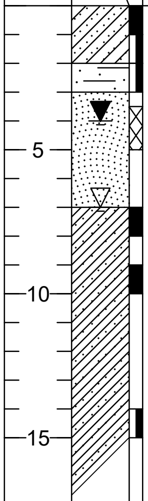
Water Observations **Seepage at 7' during drilling; water at 4' at completion**

Completion Depth **15.0'**

Completion Date **3-12-24**

Surface Elevation **591.0**

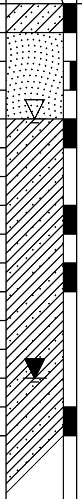
Type **B-53, w/ CFA**



Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
0		**SILTY CLAYEY SAND**, dark brown and brown, w/ iron stains, ironstone nodules, and gravel - 6-inch thick sandstone seam at 1.5'			1.25	14	26	20	6	9		
2.0	589.0	**SANDSTONE**, tan, fractured, w/ gravel layers										
3.0	588.0	**SAND**, tan, medium dense			15							
7.0	584.0	**SANDY CLAY**, light reddish brown, w/ iron seams, ironstone nodules, and calcareous nodules, hard			4.5+	68	32	12	20	15	114	
		- reddish brown below 9'			4.5+					14	122	8220
15.0	576.0											

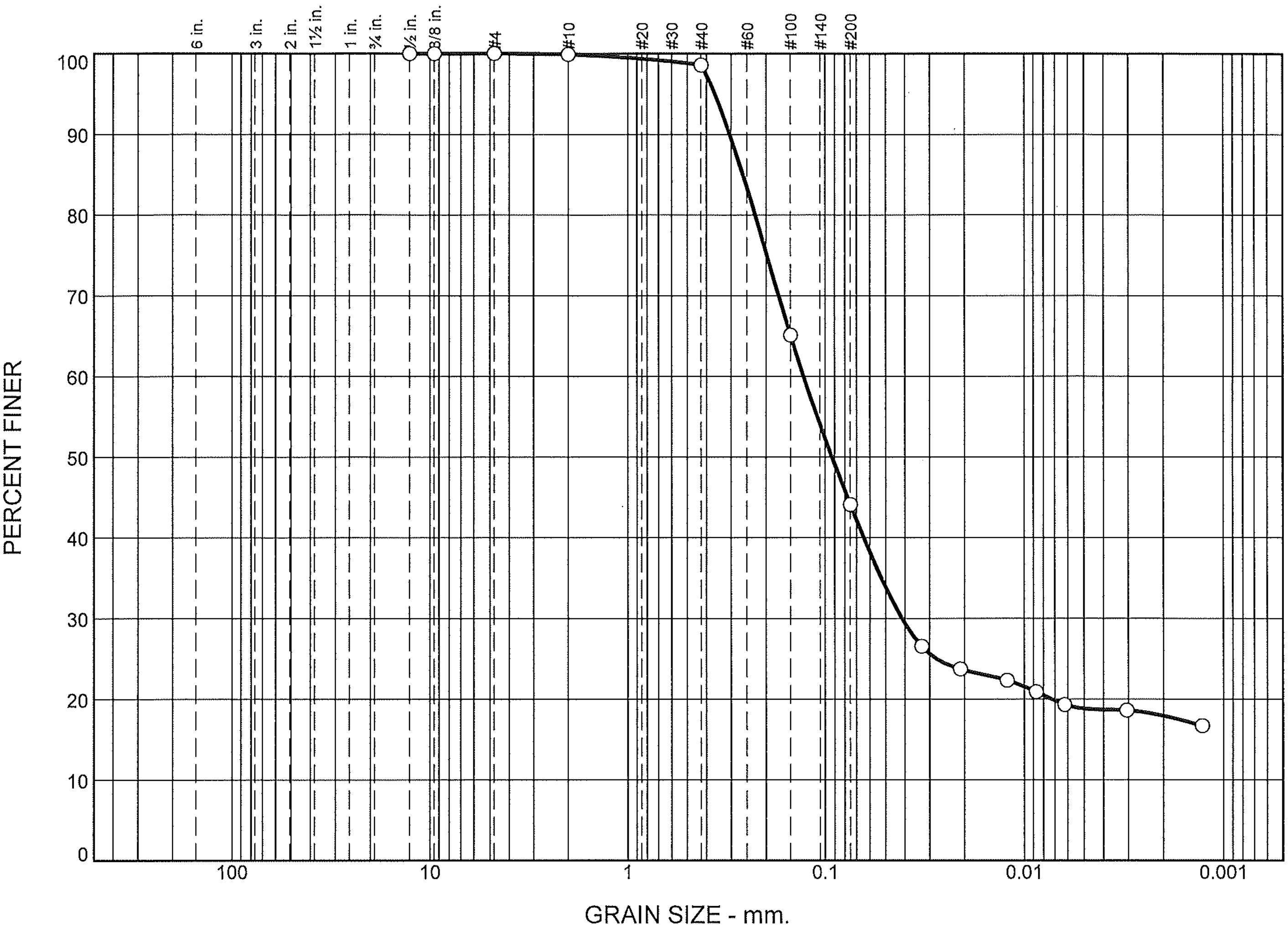
CMJ ENGINEERING INC.

Project No. **1029-24-03**	Boring No. **B-18**	Project **Castleberry High School Additions** **215 Churchill Road - Fort Worth, Texas**
Location **See Plate A.1**		Water Observations **Seepage at 4' during drilling; water at 13' at completion**
Completion Depth **15.0'**	Completion Date **3-12-24**	
Surface Elevation **591.0**	Type **B-53, w/ CFA**	



Depth, Ft.	Symbol	Samples	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
			590.0 **CLAYEY SAND**, brown, w/ iron stains and ironstone nodules			1.0					12		
			587.0 **SAND**, tan										
			SANDY CLAY / CLAYEY SAND, light brown, reddish brown, and gray, w/ iron seams and ironstone nodules, stiff to very stiff			2.25					18		
5													
						2.75					18	114	2090
						3.0		25	12	13	11		
10													
			576.0 - hard below 14'			4.5+					16		
15													

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.1	1.3	54.5	25.3	18.8

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	26	14	0.2617	0.1287	0.0926	0.0413				

	MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○			SC	

Project No. 1029-24-03 **Client:** Castleberry I.S.D.

Project: Castleberry HS Add.-215 Churhill Rd-Fort Worth, TX

○ **Depth:** 3-4 **Sample Number:** B-1

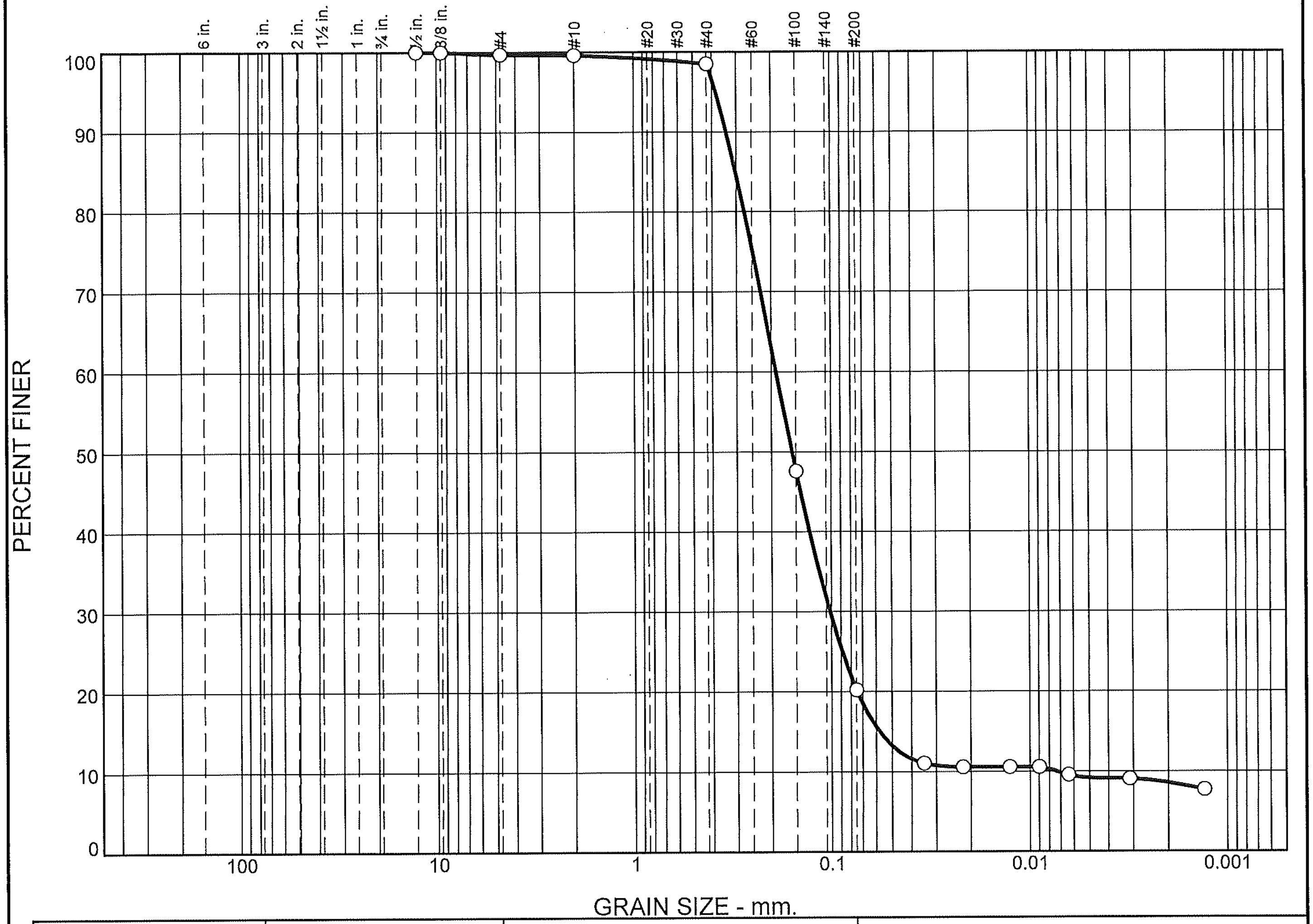
Remarks:

CMJ ENGINEERING, INC.

Fort Worth, Texas

PLATE A.22

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.3	0.1	1.1	78.3	11.0	9.2

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.3027	0.1891	0.1571	0.1017	0.0577	0.0073	7.46	25.80

MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○			

Project No. 1029-24-03 **Client:** Castleberry I.S.D.
Project: Castleberry HS Add.-215 Churhill Rd-Fort Worth, TX

○ **Depth:** 3.5-5 **Sample Number:** B-9

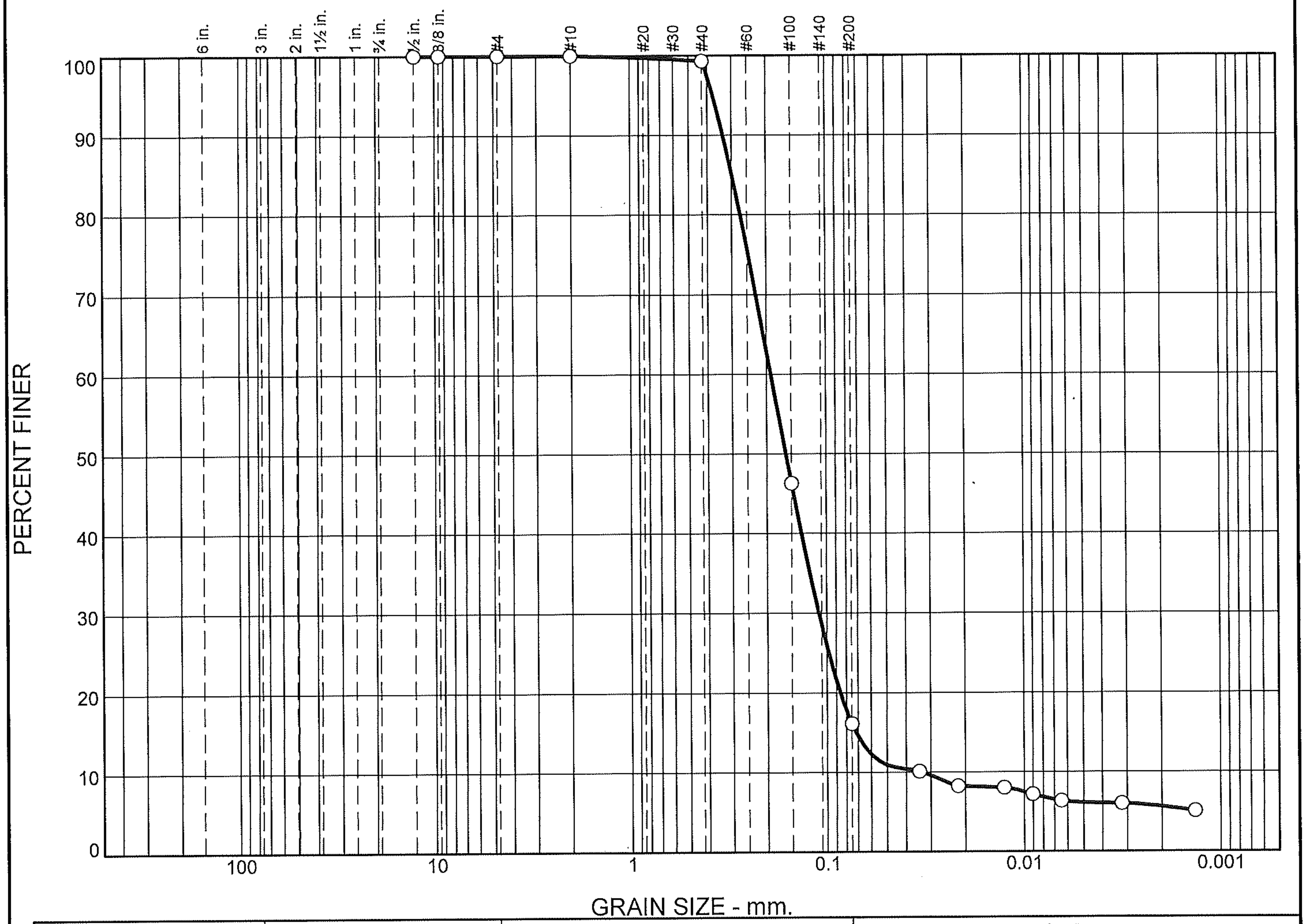
Remarks:

CMJ ENGINEERING, INC.

Fort Worth, Texas

PLATE A.23

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○	0.0	0.0	0.0	0.0	0.7	83.2	10.0	6.1

☒	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○			0.2988	0.1901	0.1601	0.1097	0.0715	0.0332	1.91	5.73

	MATERIAL DESCRIPTION	TEST DATE	USCS	NM
○				

Project No. 1029-24-03 **Client:** Castleberry I.S.D.
Project: Castleberry HS Add.-215 Churhill Rd-Fort Worth, TX

○ **Depth:** 3.5-5 **Sample Number:** B-10

Remarks:

CMJ ENGINEERING, INC.

Fort Worth, Texas

PLATE A.24

FREE SWELL TEST RESULTS

Project: Castleberry High School Additions
215 Churchill Road – Fort Worth, Texas

Project No.: 1029-24-03

Boring No.	Depth Interval (ft.)	Sample Description	Liquid Limit	Plastic Limit	Plasticity Index	Moisture Content %		Percent Swell (%)
			LL	PL	PI	Initial	Final	
B-2	3 – 4	Clayey Sand	29	15	14	8.3	15.9	0.0
B-4	4 – 5	Sandy Clay	45	14	31	16.4	18.1	0.3
B-8	14 – 15	Silty Clayey Sand	22	15	7	11.8	14.1	0.0
B-9	9 – 10	Clayey Sand	26	14	12	15.3	15.9	0.0
B-17	7 – 8	Sandy Clay	32	12	20	14.9	15.8	0.0

Free swell tests performed at approximate overburden pressure

ANALYTICAL TEST RESULTS

Project:	Castleberry High School Additions 215 Churchill Road – Fort Worth, Texas
Project No.:	1029-24-03

Boring No.	Depth (ft.)	Soluble Sulfate (ppm)	pH	Resistivity (ohm-cm)
B-1	9 – 10	<100	6.92	5,860
B-4	7 – 8	<100	7.80	2,690
B-12	4 – 5	<100	5.96	14,950