GEOTECHNICAL EXPLORATION

on

TWO MEDICAL OFFICE BUILDINGS
N. Beach Street and Heritage Glen Drive
Fort Worth, Texas
ALPHA Report No. G152751

Prepared for:

CASTLE DEVELOPMENT GROUP
P. O. Box 96255
Southlake, Texas 76092
Attention: Mr. Jesse Gober
December 31, 2015

Prepared By:

ALPHA TESTING, INC.
2209 Wisconsin Street, Suite 100
Dallas, Texas 75229
December 31, 2015

Castle Development Group
P. O. Box 96255
Southlake, Texas 76092
Attention: Mr. Jesse Gober

Re: Geotechnical Exploration
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Attached is the report of the geotechnical exploration performed for the project referenced above. This study was authorized by Mr. Jesse Gober on November 23, 2015 and performed in accordance with ALPHA Proposal No. 49999-rev dated November 19, 2015.

This report contains results of field explorations and laboratory testing and an engineering interpretation of these with respect to available project characteristics. The results and analyses were used to develop recommendations to aid design and construction of foundations and pavement.

ALPHA TESTING, INC. appreciates the opportunity to be of service on this project. If we can be of further assistance, such as providing materials testing services during construction, please contact our office.

Sincerely,

ALPHA TESTING, INC.

Harsha R. Addula P.E.
Senior Geotechnical Engineer

Mark L. McKay, P.E.
Geotechnical Department Manager

HAR/MLM/pc
Copies: (1) Client
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1.0 PURPOSE AND SCOPE

The purpose of this geotechnical exploration is for ALPHA TESTING, INC. (“ALPHA”) to evaluate for the “Client” some of the physical and engineering properties of subsurface materials at selected locations on the subject site with respect to formulation of appropriate geotechnical design parameters for the proposed new construction. The field exploration was accomplished by securing subsurface samples from widely spaced test borings performed across the proposed new construction areas. Engineering analyses were performed from results of the field exploration and results of laboratory tests performed on representative samples.

Also included are general comments pertaining to reasonably anticipated construction problems and recommendations concerning earthwork and quality control testing during construction. This information can be used to evaluate subsurface conditions and to aid in ascertaining construction meets project specifications.

Recommendations provided in this report were developed from information obtained in test borings depicting subsurface conditions only at the specific boring locations and at the particular time designated on the logs. Subsurface conditions at other locations may differ from those observed at the boring locations, and subsurface conditions at boring locations may vary at different times of the year. The scope of work may not fully define the variability of subsurface materials and conditions that are present on the site.

The nature and extent of variations between borings may not become evident until construction. If significant variations then appear evident, our office should be contacted to re-evaluate our recommendations after performing on-site observations and possibly other tests.

2.0 PROJECT CHARACTERISTICS

It is proposed to construct two (2) new medical office buildings on a site located west of N. Beach Street and about 1,550 ft north of Heritage Glen Drive in Fort Worth, Texas. A site plan illustrating the general outline of the property is provided as Figure 1, the Boring Location Plan, in the Appendix of this report. At the time of the field exploration, the site was relatively open with a few scattered trees. An existing creek borders along the northern and western portion of the proposed new construction. Visible fill was observed on the site. According to online maps available from the North Central Texas Council of Governments (found at www.dfwmaps.com), the topography of the site generally slopes downward toward the creek with a maximum change in surface elevation of about 10 ft (about Elev. 700 to 690).

Present plans consist of constructing two (2) new single story buildings (6,000 and 8,000 SF) with associated paving and driveways. Based on our conversations with the Client, it is anticipated the new structures will be supported using drilled pier foundation systems. Pavement will consist of Portland cement concrete (PCC). Grading plans were not available for this study. For the purpose of our analysis, we have assumed maximum cuts and fills of about 2 ft to achieve final grade in the building pad areas.
Retaining walls may be planned in some areas of the site. These walls should be properly designed and constructed. Specific design parameters (including global stability) for retaining walls are considered beyond the scope of this study.

3.0 FIELD EXPLORATION

Subsurface conditions were explored by drilling a total of six (6) test borings in general accordance with ASTM Standard D 420. Borings 1, 2 and 3 were drilled in the building areas to a depth of 25 ft each. Borings 4, 5 and 6 were drilled in the paving areas to a depth of 5 ft each. The approximate location of each test boring is shown on the Boring Location Plan, Figure 1, enclosed in the Appendix of this report. Details of drilling and sampling operations are briefly summarized in Methods of Field Exploration, Section A-1 of the Appendix.

Subsurface types encountered during the field exploration are presented on Log of Boring sheets included in the Appendix of this report. The boring logs contain our Field Technician's and Engineer's interpretation of conditions believed to exist between actual samples retrieved. Therefore, these boring logs contain both factual and interpretive information. Lines delineating subsurface strata on the boring logs are approximate and the actual transition between strata may be gradual.

4.0 LABORATORY TESTS

Selected samples of the subsurface materials were tested in the laboratory to evaluate their engineering properties as a basis in providing recommendations for foundation design and earthwork construction. A brief description of testing procedures used in the laboratory can be found in Methods of Laboratory Testing, Section B-1 of the Appendix. Individual test results are presented either on Log of Boring sheets or on summary data sheets also enclosed in the Appendix.

5.0 GENERAL SUBSURFACE CONDITIONS

The project site lies in an area of undivided Pawpaw Formation, Weno Limestone and Denton Clay near the undivided Grayson Marl and Main Street Limestone Formation. The undivided Pawpaw/Weno/Denton Clay generally consists of limestone, clay shale, and/or shale. Residual soils generally consist of highly plastic clays and shaly clays. These residual soils typically have high seasonal shrink-swell potential with changes in soil moisture content. The Grayson Marl and Main Street Formation consists of variable thicknesses of moderate to high plasticity clay soils over the limestone and/or shale associated with this formation.

Subsurface materials at the borings consists of clayey (CH/CL) fill material to the boring termination depth of 5 ft in Borings 4, 5 and 6 and to depths of about 6 to 7 ft in Borings 1, 2 and 3 followed by native clay/shaly clay (CH), limestone and clay shale to the boring termination depth of 25 ft. Tan and/or gray limestone was encountered in Borings 1, 2 and 3 at depths of about 15 to 17 ft below existing grade and extended to depths of about 18 to 22 ft followed by clay shale to the boring termination depth of 25 ft. The existing fill material in the borings was
identified by visual observations. The letters in parenthesis represent the soils' classification according to the Unified Soil Classification System (ASTM D 2488). More detailed stratigraphic information is presented on the Log of Boring sheet attached to this report.

Most of the subsurface materials are relatively impermeable and are anticipated to have a relatively slow response to water movement. Therefore, several days of observation will be required to evaluate actual groundwater levels within the depths explored. Also, the groundwater level at the site is anticipated to fluctuate seasonally depending on the amount of rainfall, prevailing weather conditions and subsurface drainage characteristics.

During field explorations, groundwater was encountered in Boring 1 during drilling and immediately upon completion of the boring at a depth of about 24 ft below existing grade. In Boring 2, groundwater was encountered during drilling at a depth of about 6 ft below existing grade; however, the borehole was dry immediately upon completion of the boring. Free groundwater was not encountered in the other borings. It is common to detect shallower seasonal groundwater from within the fill material, in the native clayey matrix, in or above the limestone/clay shale, particularly during or after periods of precipitation. If more detailed groundwater information is required, monitoring wells or piezometers can be installed.

Further details concerning subsurface materials and conditions encountered can be obtained from the Log of Boring sheets provided in the Appendix of this report.

6.0 DESIGN RECOMMENDATIONS

The following design recommendations were developed on the basis of the previously described Project Characteristics (Section 2.0) and General Subsurface Conditions (Section 5.0). If project criteria should change, including building locations on the site, our office should conduct a review to determine if modifications to the recommendations are required. Further, it is recommended our office be provided with a copy of the final plans and specifications for review prior to construction.

Design criteria given in this report were developed assuming the floor slabs are constructed within 2 ft of existing grade. Substantial cutting and filling on the site (more than 2 ft) can alter the recommended foundation design parameters. Therefore, it is recommended our office be contacted before performing other cutting and filling on site to verify the appropriate design parameters are utilized for final foundations design.

6.1 Existing Fill

Based on visual examination of the samples obtained, the upper 6 to 7 ft of clayey material in Borings 1, 2 and 3 is considered fill. The fill material in Borings 4, 5 and 6 extended to at least 5 ft, the maximum depth explored. Also, deeper fill could exist in other areas. Composition of the fill was evaluated based on samples retrieved from 6-inch maximum diameter boreholes.
Subgrade improvement method consisting of moisture conditioning (as discussed below in Section 6.3.1) to reduce potential movements would over-excavate and replace all existing fill in the building areas. Pavement areas in existing fill should be properly prepared and tested as discussed in Section 7.1 of this report.

Any rock pieces or concrete/brick encountered in the fill could be re-used as fill provided the material is sorted and processed to a size less than 4 inches and there are sufficient fines to prevent nested voids in the backfill. Although not encountered at the borings, uncontrolled fill can contain large concrete pieces, rubble, organic material, wood, reinforcement steel, plastic and other debris. This debris is not suitable for reuse as replacement fill and should be removed from the site. Test pits could be performed prior to construction to assess the lateral extent, depth and nature of the existing fill. ALPHA TESTING would be pleased to assist with a test pit program if desired.

6.2 Drilled and Underreamed Piers

The structural frame of the proposed new buildings could be supported using a system of drilled and underreamed piers. We recommend these piers bear in clayey soils and/or on top of the limestone/clay shale at a depth of about 16 ft below final grade.

Please note, groundwater was encountered in Boring 2 during drilling at a depth of about 6 ft below existing grade; however, the borehole was dry immediately upon completion of the boring. In addition, groundwater was encountered in Boring 1 at a depth of about 24 ft below existing grade. Adjustments in the depths of the piers may be required in some areas to maintain the bottom of the piers above limestone, and/or groundwater; however, underreamed piers should be constructed below the depth of moisture improvement. Adjustments in the depths of the piers should be observed in the field by ALPHA. Due to the presence of groundwater and relatively deep moisture improvement, it is recommended test piers be installed to determine the feasibility of constructing under-reamed piers at this site.

Drilled and underreamed piers can be dimensioned using a net allowable end bearing pressure of 5.0 kips per sq ft and no skin friction component of resistance. The above bearing pressure contains a factor of safety of at least three (3) considering a general bearing capacity failure. Normal elastic settlement of piers under loading is estimated to be about 1 inch.

Each pier should contain full length steel reinforcing and be designed to resist the uplift pressure (soil-to-pier adhesion) due to potential soil swell along the shaft from post construction heave and other uplift forces applied by structural loadings. The magnitude of uplift adhesion due to soil swell along the pier shaft cannot be defined accurately and can vary according to the actual in-place moisture content of the soils during construction. It is estimated this uplift adhesion will not exceed about 2.0 kips per sq ft. This soil adhesion is approximated to act uniformly over the upper 10 ft of the pier shaft in contact with clayey soils. If subgrade soils are improved as recommended in Section 6.2 to reduce potential floor slab movements to about 1 inch, this uplift adhesion will not exceed about 1.2 kips per sq ft to a depth of 10 ft. Uplift adhesion due to soil heave can
be neglected over the portion of the pier shaft in contact with any non-expansive material used to reduce potential movement in the building pad area.

The uplift force due to swelling of active clays should be resisted by the underreamed portion of the pier. The underreamed portion should be at least two (2) and not exceeding three (3) times the diameter of the shaft. The minimum clear spacing between edges of adjacent piers should be at least one (1) underream diameter, based on the larger underream.

All grade beams connecting piers should be formed and not cast in earthen trenches. Grade beams should be formed with a nominal 8-inch void at the bottom. Commercially available cardboard box forms (cartons) are made for this purpose. The cardboard cartons should extend the full length and width of the grade beams. Prior to concrete placement, the cartons should be inspected to verify they are firm, properly placed, and capable of supporting wet concrete. Some type of permanent soil retainer, such as pre-cast concrete panels, must be provided to prevent soils adjacent to grade beams from sloughing into the void space at the bottom of the grade beams. Additionally, backfill soils placed adjacent to grade beams must be compacted as outlined in Section 7.3 of this report.

6.3 Floor Slab for Pier-Supported Structures

Considering the subsurface conditions encountered at this site, floor slabs for the proposed new buildings could experience soil-related potential movements on the order of about 4 inches if constructed within 2 ft of existing grade.

These potential seasonal movements were estimated using results of absorption swell tests, in general accordance with methods outlined by Texas Department of Transportation (TxDOT) Test Method Tex-124-E and engineering judgment and experience. Estimated movements were calculated assuming the moisture content of the in-situ soil within the normal zone of seasonal moisture content change varies between a "dry" condition and a "wet" condition as defined by Tex-124-E. Also, it was assumed a 1 psi surcharge load from the slab acts on the subgrade soils. Movements exceeding those predicted above could occur if positive drainage of surface water is not maintained or if soils are subject to an outside water source, such as leakage from a utility line or subsurface moisture migration from off-site locations.

In view of these potential seasonal movements, the most positive floor system for the building is a slab suspended completely above the existing highly expansive soils. A 12-inch void space should be provided between the bottom of the slab (and lowest suspended fixture/utility) and top surface of the underlying expansive clays. Cardboard carton forms or a deeper crawl space can be used to create the minimum void space. A ventilated crawl space is preferred. Provisions should be made for (a) adequate drainage of the under-floor space and (b) differential movement of utility lines, including areas where the utility penetrates through the grade beam and/or where the utility penetrates below grade areas.
If some slab movement is tolerable (about 1 inch), an alternate method is for the floor system of the buildings to consist of a concrete slab designed to bear uniformly on improved soils. Considering the existing fill encountered in the borings, the most appropriate method of subgrade improvement at this site includes over-excavating a portion of the existing clayey soils and then re-placing the same over-excavated soils at elevated moisture levels in conjunction with select, non-expansive material. The extent (or depth) of the subgrade improvement for the planned building is discussed below in more detail in Section 6.3.1. In choosing this method of floor slab movement reduction, the Owner is accepting some post construction seasonal movement of floor slabs (1 inch).

Movement of the floor slab could be reduced to about 1 inch by placing at least 2 ft of select, non-expansive material between the bottom of the floor slab and the top surface of 8 ft of moisture-conditioned soil.

If a soil-supported floor slab is utilized for the planned building, consideration should be given to a "floating" (fully ground supported, and not structurally connected to walls or foundations) floor slab. This can reduce the risk of cracking and displacement of the floor slab due to differential movements between the slab and foundations. A floor slab doweled into perimeter grade beams can develop a plastic hinge (crack) parallel to and approximately 5 to 10 ft inside the building perimeter. Differential movements can still occur between the grade beam and a “floating” floor slab. The structural engineer should determine the need for connections between the slab and structural elements and determine if control joints to limit cracking are needed. A properly designed and constructed moisture barrier should be placed between the slab and subgrade soils to retard moisture migration through the slab.

### 6.3.1 Subgrade Improvement Utilizing Moisture-Conditioned Soil

Movement of the floor slabs could be reduced to about 1 inch by placing at least 2 ft of non-expansive material between the bottom of the floor slab and the top surface of 8 ft of moisture-conditioned soil. Moisture conditioning will also serve to re-work the existing fill material. Any remaining uncontrolled fill below the depth of moisture conditioning should be removed and replaced per the recommendations in Section 7.3.

Moisture-conditioning consists of over-excavating the site soils, then processing and compacting the specified minimum thickness of soil at a “target” moisture content approximated to be at least 5 percentage points (with a higher limit of 7 percentage points) above the material’s optimum moisture content as determined by the standard Proctor method (ASTM D 698). Materials with relatively lower plasticity index values (clayey soil with a plasticity index of 25 or less) may need to be placed at moisture contents at about 3 percentage points above the material’s optimum moisture content as determined by the standard Proctor method (ASTM D 698) to allow compaction.
The moisture-conditioned soil should be placed in 8-in thick loose lifts and compacted to a dry density of 93 to 97 percent of standard Proctor maximum dry density. Moisture conditioning of the on-site soil should extend throughout the entire building pad area and at least 5 ft beyond the perimeter of the building (including adjoining flatwork). In entrance areas, the moisture conditioning process should extend at least 10 ft beyond the perimeter of the building. However, select, non-expansive material should not extend beyond the building limits. If flatwork or paving is not planned adjacent to the structures (i.e., above the moisture-conditioned soils), a moisture barrier consisting of a minimum of 10 mil plastic sheeting with 8 to 12 inches of soil cover should be provided above the moisture conditioned soils. Moisture-conditioned soils should be maintained in a moist condition prior to placement of the required thickness of select, non-expansive material, plastic sheeting or flatwork.

The resulting estimated potential seasonal movement (about 1 inch) was calculated assuming the moisture content of the moisture-conditioned soil varies between the "target" moisture content and the "wet" condition while the deeper undisturbed in-situ soil within the normal zone of seasonal moisture content change varies between the "dry" condition and the "wet" condition as defined by methods outlined in TxDOT Test Method Tex-124-E.

Please note, it is the intent of the moisture-conditioning process described above to reduce the free swell potential of the moisture-conditioned soil to about 1 percent or less. Additional laboratory tests (i.e., standard Proctors, absorption swell tests, etc.) should be conducted during construction to verify the "target" moisture content for moisture-conditioning (estimated at 5 percentage points above the material’s optimum moisture content as defined by ASTM D 698) is sufficient to reduce the free swell potential of the processed soil to 1 percent or less. In addition, it is recommended samples of the moisture-conditioned material be routinely obtained during construction to verify the free swell of the improved material is 1 percent or less.

Installation of moisture-conditioned soils should be monitored and tested on a full-time basis by a representative of ALPHA TESTING, INC., to verify the soils tested were placed with the proper lift thickness, moisture content, and degree of compaction.

6.4 Seismic Considerations

The Site Class for seismic design is based on several factors that include soil profile (soil or rock), shear wave velocity, and strength, averaged over a depth of 100 ft. Since our borings did not extend to 100-foot depths, we based our determinations on previous experience in the area considering the subsurface materials encountered at the termination depths of the deepest boring. Based on Section 1613.3.2 of the 2012 International Building Code and Table 20.3-1 in the 2010 ASCE-7, we recommend using Site Class C (very dense soil and soft rock) for seismic design at this site.
6.5 Flatwork

Flatwork, pavement and any other soil-supported structural elements will be subjected to the same level of movement as discussed in Section 6.3. If this level of movement is not acceptable, flatwork could be supported on drilled pier foundations as described in Section 6.1 above. As an alternative, subgrade improvements as described in Section 6.3 could be considered for reduction in soil movements in any areas where post-construction movements would be critical.

6.6 Pavement

Clayey fill soils encountered near the existing ground surface or similar materials used as engineered fill for site grading will probably constitute the subgrade for most parking and drive areas. Pavements can generally tolerate more movement than building structures. Therefore, it is recommended the existing subsurface materials be improved prior to construction as recommended in Section 7.1 and below. A qualified Geotechnical Engineer should be retained to provide subgrade monitoring and testing during construction. If there is any change in project criteria, the recommendations contained in this report should be reviewed by our office.

Calculations used to determine the required pavement thickness are based only on the physical and engineering properties of the materials and conventional thickness determination procedures. Pavement joining the buildings should be constructed with a curb and the joint between the building and curb should be sealed. Related civil design factors such as subgrade drainage, shoulder support, cross-sectional configurations, surface elevations, reinforcing steel, joint design and environmental factors will significantly affect the service life and must be included in preparation of the construction drawings and specifications, but were not included in the scope of this study.

Normal periodic maintenance will be required for all pavement to achieve the design life of the pavement system.

Please note, the recommended pavement sections provided below are considered the minimum necessary to provide satisfactory performance based on the expected traffic loading. In some cases, City minimum standards for pavement section construction may exceed those provided below.

6.6.1 Pavement Subgrade Preparation

After final subgrade elevation is achieved, the exposed surface of the pavement subgrade soil should be scarified to a depth of 6 inches and mixed with a minimum of 7 percent hydrated lime (by dry soil weight) in conformance with TxDOT Standard Specifications Item 260. Assuming an in-place unit weight of 100 pcf for the pavement subgrade soils, this percentage of lime equates to about 32 lbs of lime per sq yard of treated subgrade. The actual amount of lime required should be confirmed by additional laboratory tests (ASTM C 977 Appendix XI) prior to construction.
It is recommended lime stabilization procedures extend at least 1 ft beyond the edge of the pavement to reduce effects of seasonal shrinking and swelling upon the extreme edges of pavement. The soil-lime mixture should be compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 0 to 4 percentage points above the mixture's optimum moisture content. In all areas where hydrated lime is used to stabilize subgrade soil, routine Atterberg-limit tests should be performed to verify the resulting plasticity index of the soil-lime mixture is at/or below 15.

Mechanical lime stabilization of the pavement subgrade soil will not prevent normal seasonal movement of the underlying untreated materials. Pavement and other flatwork will have the same potential for movement as slabs constructed directly on the existing undisturbed soils. Therefore, good perimeter surface drainage with a minimum slope of 2 percent away from the pavement is recommended. The use of sand as a leveling course below pavement supported on expansive clays should be avoided. Normal maintenance of pavement should be expected over the life of the structures.

### 6.6.2 Portland-Cement Concrete (PCC) Pavement

Following subgrade improvement as recommended in Section 6.6.1 above, the following PCC (reinforced) pavement sections are recommended.

*Note: Lime treatment of the pavement subgrade is not necessary for pavements subjected exclusively to passenger vehicle traffic, although lime treatment in these areas would be generally beneficial to the long-term performance of the pavement and improve constructability. Prior to construction of pavement on untreated clay subgrade soil, the exposed subgrade should be scarified to a depth of at least 6 inches and compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 1 percentage point below to 3 percentage points above the material's optimum moisture content.

Portland-cement concrete should have a minimum compressive strength of 3,000 lbs per sq inch (psi) at 28 days in parking areas subjected exclusively to passenger vehicle traffic. We recommend a minimum compressive strength of 3,500 psi at 28 days for the street, drive lanes, fire lanes, and truck areas. Concrete should be designed with 5 ± 1 percent entrained air. Joints in concrete paving should not
Reinforcing steel should consist of No. 3 bars placed at 18 inches on-center in two directions.

Alternately, mechanical lime stabilization of the pavement subgrade could be eliminated by increasing the PCC thickness in the pavement sections presented above by 1 inch. Prior to construction of pavement on untreated clay subgrade soil, the exposed subgrade should be scarified to a depth of at least 6 inches and compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 1 percentage point below to 3 percentage points above the material's optimum moisture content.

6.7 Drainage and Other Considerations

Adequate drainage should be provided to reduce seasonal variations in the moisture content of foundation soils. All pavement and sidewalks within 5 ft of the structure should be sloped away from the building to prevent ponding of water around the foundations. Final grades within 5 ft of the structure should be adjusted to slope away from the structure at a minimum slope of 2 percent. Maintaining positive surface drainage throughout the life of the structure is essential.

In areas with pavement or sidewalks adjacent to the new structure, a positive seal must be maintained between the structure and the pavement or sidewalk to minimize seepage of water into the underlying supporting soils. Post-construction movement of pavement and flatwork is common. Normal maintenance should include examination of all joints in paving and sidewalks, etc. as well as resealing where necessary.

Several factors relate to civil and architectural design and/or maintenance, which can significantly affect future movements of the foundation and floor slab systems:

1. Preferably, a complete system of gutters and downspouts should carry runoff water a minimum of 5 feet from the completed structure.

2. Large trees and shrubs should not be allowed closer to the foundations than a horizontal distance equal to roughly one-half of their mature height due to their significant moisture demand upon maturing.

3. Moisture conditions should be maintained "constant" around the edge of the slabs. Ponding of water in planters, in unpaved areas, and around joints in paving and sidewalks can cause slab movements beyond those predicted in this report.

4. Planter box structures placed adjacent to the building should be provided with a means to assure concentrations of water are not available to the subsoil stratigraphy.

5. Architectural design of the floor slab and slab foundations should avoid additional features such as wing walls as extensions of the slab.
Trench backfill for utilities should be properly placed and compacted as outlined in Section 7.3 of this report and in accordance with requirements of local City standards. Since granular bedding backfill is used for most utility lines, the backfilled trench should not become a conduit and allow access for surface or subsurface water to travel toward the new structure. Concrete cut-off collars or clay plugs should be provided where utility lines cross building lines to prevent water from traveling in the trench backfill and entering beneath the structures.
7.0 GENERAL CONSTRUCTION PROCEDURES AND RECOMMENDATIONS

Variations in subsurface conditions could be encountered during construction. To permit correlation between test boring data and actual subsurface conditions encountered during construction, it is recommended a registered Professional Engineering firm be retained to observe construction procedures and materials.

Some construction problems, particularly degree or magnitude, cannot be anticipated until the course of construction. The recommendations offered in the following paragraphs are intended not to limit or preclude other conceivable solutions, but rather to provide our observations based on our experience and understanding of the project characteristics and subsurface conditions encountered in the borings.

7.1 Site Preparation and Grading

All areas supporting the floor slabs, pavement, flatwork and areas to receive new fill should be properly prepared.

After completion of the necessary stripping, clearing, and excavating and prior to placing any required fill, the exposed soil subgrade should be carefully evaluated by probing and testing. Any undesirable material (organic material, wet, soft, or loose soil) still in place should be removed.

The exposed soil subgrade should be further evaluated by proof-rolling with a heavy pneumatic tired roller, loaded dump truck or similar equipment weighing approximately 25 tons to check for pockets of soft or loose material hidden beneath a thin crust of possibly better soil.

Proof-rolling procedures should be observed routinely by a Professional Engineer, or his designated representative.

Any undesirable material (organic material, wet, soft, or loose soil) exposed during the proofroll should be removed and replaced with well-compacted material as outlined in Section 7.3.

Prior to placement of any fill, the exposed soil subgrade should then be scarified to a minimum depth of 6 inches and recompressed as outlined in Section 7.3.

If fill is to be placed on existing slopes (natural or constructed) steeper than six horizontal to one vertical (6:1), the fill materials should be benched into the existing slopes in such a manner as to provide a minimum bench-key width of five (5) feet. This should provide a good contact between the existing soils and new fill materials, reduce potential sliding planes, and allow relatively horizontal lift placements.

Even if fill is properly compacted, fills in excess of about 10 ft are still subject to settlements over time of up to about 1 to 2 percent of the total fill thickness. This should be considered when designing utility lines under pavements.
Slope stability analysis of embankments (natural or constructed) was not within the scope of this study.

The contractor is responsible for designing any excavation slopes, temporary sheeting or shoring. Design of these structures should include any imposed surface surcharges. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods and sequencing of construction operations. The contractor should also be aware that slope height, slope inclination or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state and/or federal safety regulations, such as OSHA Health and Safety Standard for Excavations, 29 CFR Part 1926, or successor regulations. Stockpiles should be placed well away from the edge of the excavation and their heights should be controlled so they do not surcharge the sides of the excavation. Surface drainage should be carefully controlled to prevent flow of water over the slopes and/or into the excavations. Construction slopes should be closely observed for signs of mass movement, including tension cracks near the crest or bulging at the toe. If potential stability problems are observed, a geotechnical engineer should be contacted immediately. Shoring, bracing or underpinning required for the project (if any) should be designed by a professional engineer registered in the State of Texas.

Due to the nature of the clayey soils found near the surface at the borings, traffic of heavy equipment (including heavy compaction equipment) may create pumping and general deterioration of shallow soils. Therefore, some construction difficulties should be anticipated during periods when these soils are saturated.

7.2 Foundation Excavations

All foundation excavations should be monitored to verify foundations bear on suitable material. The bearing stratum exposed in the base of all foundation excavations should be protected against any detrimental change in conditions. Surface runoff water should be drained away from excavations and not allowed to collect. All concrete for foundations should be placed as soon as practical after the excavation is made. Drilled piers should be excavated and concrete placed within the same day after the design penetration is begun.

Prolonged exposure of the bearing surface to air or water will result in changes in strength and compressibility of the bearing stratum. Therefore, if delays occur, underreamed pier excavations should be slightly widened and cleaned to provide a fresh bearing surface.

All pier shafts should be at least 1.5 ft in diameter to facilitate clean-out of the base and proper monitoring. Concrete placed in pier holes should be directed through a tremie, hopper, or equivalent. Placement of concrete should be vertical through the center of the shaft without hitting the sides of the pier or reinforcement to reduce the possibility of segregation of aggregates. Concrete placed in piers should have a minimum slump of 5 inches (but not greater than 7 inches) to avoid potential honey-combing.
Observations during pier drilling should include, but not necessarily be limited to, the following items:

- Verification of proper bearing strata and consistency of subsurface stratification with regard to boring logs,
- Confirmation the minimum required penetration into the bearing strata is achieved,
- Complete removal of cuttings from bottom of pier holes,
- Proper handling of any observed water seepage and sloughing of subsurface materials,
- No more than 2 inches of standing water should be permitted in the bottom of pier holes prior to placing concrete, and
- Verification of pier diameter, underream size and steel reinforcement.

Groundwater was encountered in Boring 2 during drilling at a depth of about 6 ft below existing grade. In addition, groundwater was encountered in Boring 1 at a depth of about 24 ft below existing grade. Adjustments in the depths of the piers may be required in some areas to maintain the bottom of the piers in clayey soils above the groundwater and/or limestone; however, underreams should be constructed below the depth of moisture improvement. Adjustments in the depths of the piers should be observed in the field by ALPHA. Due to the presence of groundwater and relatively deep moisture improvement, it is recommended test piers be installed to determine the feasibility of constructing under-reamed piers at this site.

Temporary casing may be useful for controlling groundwater seepage that could occur in the clayey soils. Immediate placement of concrete after constructing the underream and/or the use of submersible pumps may be adequate to control underream collapse and/or seepage. As casing is extracted, care should be taken to maintain a positive head of plastic concrete and minimize the potential for intrusion of water seepage. It is recommended a separate bid item be provided for casing on the contractors' bid schedule.

We should be contacted for further review and evaluation if groundwater seepage and/or underream collapse occurs during pier installation.

### 7.3 Fill Compaction

**Select, Non-Expansive Fill:** Materials used as select, non-expansive fill should have a liquid limit less than 35, a plasticity index (PI) not less than 5 nor greater than 15. All select, non-expansive fill should contain no deleterious material and should be compacted to a dry density of at least 98 percent standard Proctor maximum dry density (ASTM D 698) and within the range of 1 percentage point below to 3 percentage points above the material's optimum moisture content. (Note: The plasticity index and liquid limit of
material used as select, non-expansive fill should be verified during fill placement using laboratory tests. Visual observation and classification should not be relied upon to confirm the material to be used as select, non-expansive fill satisfies the above Atterberg-limit criteria.

Atterberg-limits tests to verify the select, non-expansive fill shall be performed at a frequency of at least one test per every 2 ft (thick) per every 5,000 sq ft. Atterberg-limit tests shall be staggered between various lifts within each 5,000 sq ft.

**Flexible Base Material:** Flexible Base material used as non-expansive fill for the building pad area should meet the requirements of TxDOT Item 247, Type A, Grade 1 or 2. Processed concrete meeting TxDOT Item 247, Grade 1 or 2, Type D is also acceptable for as non-expansive material. The material should be compacted to a minimum 95 percent of standard Proctor maximum dry density (ASTM D 698) and within 3 percentage points of the material's optimum moisture content.

The following recommendations pertain to fill soils placed for general site grading outside the designated building pad areas. All fill within the designated building pad areas, plus at least 5 ft outside the limits of the building pad areas, should meet the requirements of Section 6.3.1 discussed earlier.

Clayey soils with a plasticity index equal to or greater than 25 should be compacted to a dry density between 93 and 98 percent of standard Proctor maximum dry density (ASTM D 698). The compacted moisture content of the clays during placement should be within the range of 2 to 6 percentage points above optimum.

Clayey materials with a plasticity index below 25 should be compacted to a dry density of at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 1 percentage point below to 3 percentage points above the material's optimum moisture content.

Clayey materials used as fill should be processed and the largest particle or clod should be less than 6 inches prior to compaction.

In cases where either mass fills or utility lines are more than 10 ft deep, the fill/backfill below 10 ft should be compacted to at least 98 percent of standard Proctor maximum dry density (ASTM D 698) and within 2 percentage points of the material's optimum moisture content. The portion of the fill/backfill shallower than 10 ft should be compacted as outlined above.

Compaction should be accomplished by placing fill in about 8-inch thick loose lifts and compacting each lift to at least the specified minimum dry density. Field density and moisture content tests should be performed on each lift. As a guide, one test per 2,500 sq ft per lift is recommended in the building area. Utility trench backfill should be tested at a rate of one test per lift per each 300 lineal feet of trench.
7.4 **Groundwater**

Groundwater was encountered in Boring 1 at a depth of about 24 ft below existing grade and at a depth of about 6 ft in Boring 2. From our experience with similar soils, groundwater seepage could be encountered in relatively shallow excavations for foundations, utility conduits, and other general excavations. The risk of seepage increases with depth of excavation and during or after periods of precipitation. Standard sump pits and pumping may be adequate to control seepage on a local basis. Where sump pits and pumping are not adequate to control seepage, supplemental dewatering measures (such as, but not limited to, submersible pumps in slotted casings and wellpoints) may be required.

In any areas where cuts are made to establish final grades for the site, attention should be given to possible seasonal water seepage that could occur through natural cracks and fissures in the newly exposed stratigraphy. In these cases, subsurface drains may be required to intercept seasonal groundwater seepage. The need for these or other dewatering devices should be carefully addressed during construction. Our office could be contacted to visually observe the final grade to evaluate the need for such drains.
8.0 LIMITATIONS

Professional services provided in this geotechnical exploration were performed, findings obtained, and recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. The scope of services provided herein does not include an environmental assessment of the site or investigation for the presence or absence of hazardous materials in the soil, surface water or groundwater. ALPHA, upon written request, can be retained to provide these services.

ALPHA TESTING, INC. is not responsible for conclusions, opinions or recommendations made by others based on this data. Information contained in this report is intended for the exclusive use of the Client (and their designated design representatives), and is related solely to design of the specific structures outlined in Section 2.0. No party other than the Client (and their designated design representatives) shall use or rely upon this report in any manner whatsoever unless such party shall have obtained ALPHA’s written acceptance of such intended use. Any such third party using this report after obtaining ALPHA’s written acceptance shall be bound by the limitations and limitations of liability contained herein, including ALPHA’s liability being limited to the fee paid to it for this report. Recommendations presented in this report should not be used for design of any other structures except those specifically described in this report. In all areas of this report in which ALPHA may provide additional services if requested to do so in writing, it is presumed that such requests have not been made if not evidenced by a written document accepted by ALPHA. Further, subsurface conditions can change with passage of time. Recommendations contained herein are not considered applicable for an extended period of time after the completion date of this report. It is recommended our office be contacted for a review of the contents of this report for construction commencing more than one (1) year after completion of this report. Non-compliance with any of these requirements by the Client or anyone else shall release ALPHA from any liability resulting from the use of, or reliance upon, this report.

Recommendations provided in this report are based on our understanding of information provided by the Client about characteristics of the project. If the Client notes any deviation from the facts about project characteristics, our office should be contacted immediately since this may materially alter the recommendations. Further, ALPHA TESTING, INC. is not responsible for damages resulting from workmanship of designers or contractors. It is recommended the Owner retain qualified personnel, such as a Geotechnical Engineering firm, to verify construction is performed in accordance with plans and specifications.
A-1 METHODS OF FIELD EXPLORATION

Using standard rotary drilling equipment, a total of three six (6) test borings were performed for this geotechnical exploration at the approximate locations shown on the Boring Location Plan, Figure 1. The test boring locations were staked by either pacing or taping and estimating right angles from landmarks which could be identified in the field and as shown on the site plan provided during this study. The locations of test borings shown on the Boring Location Plan are considered accurate only to the degree implied by the method used to locate the borings.

Relatively undisturbed samples of the cohesive subsurface materials were obtained by hydraulically pressing 3-inch O.D. thin-wall sampling tubes into the underlying soils at selected depths (ASTM D 1587). These samples were removed from the sampling tubes in the field and examined visually. One representative portion of each sample was sealed in a plastic bag for use in future visual examinations and possible testing in the laboratory.

The Texas Cone Penetration (TCP) test was used to assess the apparent in-place strength characteristics of the rock type materials. The TCP test consists of a 3-inch diameter steel cone driven by a 170-pound hammer dropped 24 inches (340 ft-pounds of energy) and is the basis for TxDOT strength correlations. Depending on the resistance (strength) of the materials, either the number of blows of the hammer required to provide 12 inches of penetration, or the inches of penetration of the cone due to 100 blows of the hammer are recorded on the field logs and are shown on the Log of Boring sheets as “TX Cone” (reference: TxDOT Test Method TEX 132-E).

Logs of all borings are included in the Appendix of this report. The logs show visual descriptions of subsurface strata encountered using the Unified Soil Classification System. Sampling information, pertinent field data, and field observations are also included. Samples not consumed by testing will be retained in our laboratory for at least 14 days and then discarded unless the Client requests otherwise.
B-1 METHODS OF LABORATORY TESTING

Representative samples were examined and classified by a qualified member of the Geotechnical Division and the boring logs were edited as necessary. To aid in classifying the subsurface materials and to determine the general engineering characteristics, natural moisture content tests (ASTM D 2216), Atterberg-limit tests (ASTM D 4318) and dry unit weight determinations were performed on selected samples. In addition, unconfined compression (ASTM D 2166) and pocket-penetrometer tests were conducted on selected soil samples to evaluate the soil shear strength. Results of all laboratory tests described above are provided on the accompanying Log of Boring sheets.

In addition to the Atterberg-limit tests, the expansive properties of the clay soils were further analyzed by absorption swell tests. The swell test is performed by placing a selected sample in a consolidation machine and applying either the approximate current or expected overburden pressure and then allowing the sample to absorb water. When the sample exhibits very little tendency for further expansion, the height increase is recorded and the percent free swell and total moisture gain calculated. Results of the absorption swell tests are provided on the Swell Test Data sheet, Figure 2 included in this appendix.
<table>
<thead>
<tr>
<th>Boring No.</th>
<th>1</th>
<th>2</th>
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<td>3</td>
<td>14</td>
<td>5</td>
</tr>
<tr>
<td>Dry Unit Weight, pcf</td>
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<td>101</td>
<td>103</td>
<td>105</td>
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<td>47</td>
</tr>
<tr>
<td>Plastic Limit</td>
<td>19</td>
<td>18</td>
<td>20</td>
<td>20</td>
</tr>
<tr>
<td>Plasticity Index</td>
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<td>22</td>
<td>35</td>
<td>27</td>
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<tr>
<td>Initial Moisture Content</td>
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<td>21%</td>
<td>20%</td>
<td>17%</td>
</tr>
<tr>
<td>Final Moisture Content</td>
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<td>24%</td>
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<td>20%</td>
</tr>
<tr>
<td>Free Swell</td>
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<td>0.1%</td>
<td>0.0%</td>
<td>0.0%</td>
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### MATERIAL DESCRIPTION

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<th>Depth, feet</th>
<th>Graphic Log</th>
<th>Material Description</th>
<th>Sample Type</th>
<th>Recovery %</th>
<th>TX Cone or Std. Pen. (blows/ft), RQD</th>
<th>Pocket Penetrometer (tsf)</th>
<th>Unconfined Comp. Strength (tsf)</th>
<th>% Peaking No. 200 sieve</th>
<th>Unit Dry Weight (pcf)</th>
<th>Water Content, %</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
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<td></td>
<td>7.0</td>
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<td></td>
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<tr>
<td>10</td>
<td>13.0</td>
<td>Dark Brown CLAY</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>15.0</td>
<td>Brown CLAY</td>
<td>4.5+</td>
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<td>47</td>
<td>19</td>
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</tr>
<tr>
<td>20</td>
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<td>Tan LIMESTONE with clay seams and layers</td>
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<td>51</td>
<td>19</td>
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<td>22.0</td>
<td>Gray LIMESTONE with shale seams</td>
<td>100/ 0.25&quot;</td>
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<td>Gray CLAY SHALE</td>
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**TEST BORING TERMINATED AT 25 FT**
**MATERIAL DESCRIPTION**

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<thead>
<tr>
<th>Depth, feet</th>
<th>On Rods (ft)</th>
<th>After Drilling (ft)</th>
<th>After Hours (ft)</th>
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<td>Dry</td>
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</tr>
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<tr>
<td>25</td>
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</tr>
</tbody>
</table>

**GROUND WATER OBSERVATIONS**

- **Sample Type**: TX Cone or Std. Pen. (Bowland)
- **Recovery % RQD**: 6.0
- **Recovery %**: 2.75
- **Liquid Limit**: 2.5
- **Plastic Limit**: 0.25
- **Plasticity Index**: 13

**Depth, feet**

- **5**: Dark Brown, Brown, and Tannish Brown CLAY with gravel and sand-FILL
- **10**: Dark Brown CLAY
- **15**: -brown to tan and gray below 12'
- **18**: Tan LIMESTONE
- **20**: Gray CLAY SHALE
- **25**: TEST BORING TERMINATED AT 25 FT

**PROJECT NO.**: G152751

**Location**: Fort Worth, Texas

**Start Date**: 12/17/2015

**End Date**: 12/17/2015

**Drilling Method**: CONTINUOUS FLIGHT AUGER

**Surface Elevation**: 6.0

**Hammer Drop (lbs / in)**: 170 / 24
### GROUND WATER OBSERVATIONS

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<thead>
<tr>
<th>Depth, feet</th>
<th>On Rods (ft):</th>
<th>After Drilling (ft):</th>
<th>After Hours (ft):</th>
</tr>
</thead>
<tbody>
<tr>
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<td>None</td>
<td>Dry</td>
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### MATERIAL DESCRIPTION

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<tr>
<th>Depth, feet</th>
<th>Sample Type</th>
<th>Recovery % RQD</th>
<th>TX Cone or Std. Pen. (Blows/in)</th>
<th>Penetrometer (tsf)</th>
<th>Unconfined Comp. Strength (tsf)</th>
<th>% Peeling</th>
<th>No. 200 Sieve</th>
<th>Unit Dry Weight (pcf)</th>
<th>Water Content, %</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
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</tr>
<tr>
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**TEST BORING TERMINATED AT 25 FT**
<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Graph</th>
<th>Log</th>
<th>GROUND WATER OBSERVATIONS</th>
<th>Material Description</th>
<th>Recovery %</th>
<th>TX Cone or Std. Pen. (blows/ft, in)</th>
<th>Penetrometer (tsf)</th>
<th>Unconfined Comp. Strength (tsf)</th>
<th>% Peeling No. 200 Sieve</th>
<th>Unit Dry Weight (pcf)</th>
<th>Water Content, %</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
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<tbody>
<tr>
<td>5</td>
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<td></td>
<td>None</td>
<td>Dark Brown, Brown, and Tannish Brown CLAY with sand and gravel-FILL -reddish tan layer 2-4’</td>
<td>4.5+</td>
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<td>1.0</td>
<td>10</td>
<td>17</td>
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<td>17</td>
<td>23</td>
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</tbody>
</table>

TEST BORING TERMINATED AT 5 FT
### MATERIAL DESCRIPTION

- Dark Brown and Brown CLAY with sand and gravel-FILL
  - calcareous deposits below 2'
  - trace of brick below 4'

### GROUND WATER OBSERVATIONS

- None
- Dry

### Water Content, %

<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Sample Type</th>
<th>Recovery % RQD</th>
<th>TX Cone or Std. Pen. (blows/ft, in)</th>
<th>Pocket Penetrometer (psi)</th>
<th>Unconfined Comp. Strength (tsf)</th>
<th>% Peeling No. 200 Sieve</th>
<th>Unit Dry Weight (pcf)</th>
<th>Water Content, %</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
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</thead>
<tbody>
<tr>
<td>5</td>
<td>5.0</td>
<td>4.0</td>
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**LOG OF BORING NO.: 6**  
**PROJECT NO.: G152751**

**Client:** Castle Development Group  
**Location:** Fort Worth, Texas

**Project:** Two Medical Office Buildings (6,000 & 8,000 SF)  
**Surface Elevation:**

**Start Date:** 12/17/2015  
**End Date:** 12/17/2015

**Drilling Method:** CONTINUOUS FLIGHT AUGER  
**Hammer Drop (lbs / in):** 170 / 24

<table>
<thead>
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<th>Depth, feet</th>
<th>Graphic Log</th>
<th>GROUND WATER OBSERVATIONS</th>
<th>MATERIAL DESCRIPTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td></td>
<td>On Rods (ft): None</td>
<td>Dark Brown, Brown, and Tan CLAY with sand, gravel, and calcareous deposits—FILL</td>
</tr>
<tr>
<td>5</td>
<td></td>
<td>After Drilling (ft): Dry</td>
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</tr>
<tr>
<td>5</td>
<td></td>
<td>After Hours (ft):</td>
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</tr>
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**LOG OF BORING TERMINATED AT 5 FT**

<table>
<thead>
<tr>
<th>Depth, feet</th>
<th>Sample Type</th>
<th>Recovery %</th>
<th>RQD</th>
<th>TX Cone or Std. Pen. (Bowani ft)</th>
<th>Recovery %</th>
<th>RQD</th>
<th>Test Penetrometer (tbf)</th>
<th>Recovery %</th>
<th>RQD</th>
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</table>
SOIL & ROCK SYMBOLS

(CH), High Plasticity CLAY
(CL), Low Plasticity CLAY
(SC), CLAYEY SAND
(SP), Poorly Graded SAND
(SW), Well Graded SAND
(SM), SILTY SAND
(ML), Silt
(MH), Elastic Silt
LIMESTONE
SHALE / MARL
SANDSTONE
(GP), Poorly Graded GRAVEL
(GW), Well Graded GRAVEL
(GC), CLAYEY GRAVEL
(GM), SILTY GRAVEL
(OL), ORGANIC SILT
(OH), ORGANIC CLAY
FILL

RELATIVE DENSITY OF COHESIONLESS SOILS (blows/ft)

- VERY LOOSE: 0 TO 4
- LOOSE: 5 TO 10
- MEDIUM: 11 TO 30
- DENSE: 31 TO 50
- VERY DENSE: OVER 50

SHEAR STRENGTH OF COHESIVE SOILS (tsf)

- VERY SOFT: LESS THAN 0.25
- SOFT: 0.25 TO 0.50
- FIRM: 0.50 TO 1.00
- STIFF: 1.00 TO 2.00
- VERY STIFF: 2.00 TO 4.00
- HARD: OVER 4.00

RELATIVE DEGREE OF PLASTICITY (PI)

- LOW: 4 TO 15
- MEDIUM: 16 TO 25
- HIGH: 26 TO 35
- VERY HIGH: OVER 35

RELATIVE PROPORTIONS (%)

- TRACE: 1 TO 10
- LITTLE: 11 TO 20
- SOME: 21 TO 35
- AND: 36 TO 50

SAMPLING SYMBOLS

- SHELBY TUBE (3" OD except where noted otherwise)
- SPLIT SPOON (2" OD except where noted otherwise)
- AUGER SAMPLE
- TEXAS CONE PENETRATION
- ROCK CORE (2" ID except where noted otherwise)

PARTICLE SIZE IDENTIFICATION (DIAMETER)

- BOULDERS: 8.0" OR LARGER
- COBBLES: 3.0" TO 8.0"
- COARSE GRAVEL: 0.75" TO 3.0"
- FINE GRAVEL: 5.0 mm TO 3.0"
- COURSE SAND: 2.0 mm TO 5.0 mm
- MEDIUM SAND: 0.4 mm TO 5.0 mm
- FINE SAND: 0.07 mm TO 0.4 mm
- SILT: 0.002 mm TO 0.07 mm
- CLAY: LESS THAN 0.002 mm