GEOTECHNICAL EXPLORATION

PROPOSED RETAIL CENTER
SEC of South Collins Street and Southeast Parkway
Grand Prairie, Texas
ALPHA Report No. W200872
April 30, 2020

Prepared for:

GREG EDWARDS ENGINEERING SERVICES
1621 Amanda Court
Ponder, Texas 76259
Attention: Ms. Delsie Bussell

Prepared By:

ALPHA TESTING
WHERE IT ALL BEGINS

ALPHA TESTING
WHERE IT ALL BEGINS

Geotechnical
Construction Materials
Environmental
TBPE Firm No. 813

5058 Brush Creek Road
Fort Worth, Texas 76119

Tel: 817-496-5600
Fax: 817-496-5608
www.alphatesting.com

April 30, 2020

Greg Edwards Engineering Services
1621 Amanda Court
Ponder, Texas 76259

Attention: Ms. Delsie Bussell

Re: Geotechnical Exploration
Proposed Retail Center
SEC of South Collins Street and Southeast Parkway
Grand Prairie, Texas
ALPHA Report No. W200872

Attached is the report of the geotechnical exploration performed for the project referenced above. This study was authorized by Mr. Greg Edwards on March 17, 2020 and performed in accordance with ALPHA Proposal No. 75555 dated January 31, 2020.

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 geotechnical recommendations to aid in design 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 the final geotechnical exploration, please contact our office.

Sincerely,

ALPHA TESTING, INC.

STATE OF TEXAS
BRIAN J. HOYT
115748
LICENSED
PROFESSIONAL ENGINEER

April 30, 2020

Kimberly Fyffe
Geotechnical Project Manager

Brian J. Hoyt, P.E.
Geotechnical Department Manager

BJH/KLF/klf
Copies: (1-PDF) Client

TABLE OF CONTENTS

ALPHA REPORT NO. W200872

1.0 PURPOSE AND SCOPE

The purpose of this geotechnical exploration is for ALPHA TESTING, INC. (ALPHA) to evaluate for Greg Edwards Engineering Services (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 construction. The field exploration was accomplished by securing subsurface samples from widely spaced test borings performed across the expanse of the site. Engineering analyses were performed from results of the field exploration and results of laboratory tests conducted 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

We understand the project will consist of a single story retail building with a plan area of about 6,400 SF on a site located at the southeast corner of South Collins Street and Southeast Parkway in Grand Prairie, Texas. A site plan illustrating the general outline of the property is provided as Figure 1, the Boring Location Plan, in the Appendix.

At the time the field exploration was performed, the site consisted of an undeveloped tract of land. Review of historical images available from Google Earth® indicates earthwork activities were previously performed at the site. These images also indicate the area around Borings 1 and 3 was used as a staging area for fill stockpiles. No information regarding previous development on the site was provided to us. Cursory visual observations indicate the building pad area is relatively level.

We understand the building/floor slab will be designed for about 1 inch of post-construction seasonal movement. No below grade slabs are planned. Pavement for the project will consist of portland cement concrete (PCC). Grading plans were not provided for this study. For the purpose of our analysis, we have assumed maximum cuts and fills of 2 ft to achieve final grades.

3.0 FIELD EXPLORATION

Subsurface conditions on the site were explored by drilling a total of four (4) test borings. Two (2) test borings were drilled to a depth of about 25 ft each and two (2) test boring was drilled to a depth of about 5 ft. The test borings were performed in general accordance with ASTM D 420 using standard rotary drilling equipment. The approximate location of each test boring is shown on the Boring Location Plan, Figure 1, enclosed in the Appendix. 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 the Log of Boring sheets (boring logs) included in the Appendix. The boring logs contain our Field Technician's and Engineer's interpretation of conditions believed to exist between actual samples retrieved. Therefore, the 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 information 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 on the Log of Boring sheets or summary data sheets enclosed in the Appendix.

5.0 GENERAL SUBSURFACE CONDITIONS

Based on geological atlas maps available from the Bureau of Economic Geology, published by the University of Texas at Austin, the project site lies within the Eagle Ford formation. The Eagle Ford formation is composed predominantly of shale with occasional platy beds of sandstone and limestone. Residual overburden soils associated with the Eagle Ford formation generally consist of clay soils with moderate to very high shrink/swell potential.

Subsurface conditions encountered in Borings 1 and 2 generally consisted of clay and sandy clay extending to the 25 ft termination depth. Clay extended to the 5 ft termination depth of Borings 3 and 4. More detailed stratigraphic information is presented on the Log of Boring sheets.

The clay and sandy clay encountered in the borings are considered relatively impermeable and are expected to have a relatively slow response to water movement. Therefore, several days of observation would be required to evaluate actual groundwater levels within the depths explored. The groundwater level at the site is anticipated to fluctuate seasonally depending on the amount of rainfall, prevailing weather conditions and subsurface drainage characteristics.

Free groundwater was encountered in Borings 1 and 2 at respective depths of about 13 ft and 18 ft below the ground surface on drilling tools during drilling. Groundwater was observed in these borings at depths of about 12 ft and 16 ft, respectively, immediately upon completion of drilling. Borings 3 and 4 were dry. It is common to encounter seasonal groundwater from natural fractures within the clayey matrix, 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 Boring Logs provided in the Appendix.

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). Should the project criteria change, including the building location on the site, our office should conduct a review to determine if modifications to the recommendations are required.

The following design criteria was developed based on the assumption that cuts and fills required to achieve final grade will not exceed 2 ft. Cutting or filling on the site more than 2 ft can alter the recommended design parameters. Therefore, it is recommended our office be provided with a copy of final grading plans to verify appropriate design parameters are utilized for final design.

6.1 Existing Fill Material

Although not encountered in the borings, historical aerial images discussed in Section 2.0 indicate fill exists or previously existed on the site. It is not known if this fill was placed under engineering supervision with compaction records. If compaction records for this fill cannot be obtained, the existing fill should be considered as uncontrolled fill. Uncontrolled fill is generally not considered suitable for support of foundations or floor slabs due to the risk of under-compacted zones resulting in failures of weak soil and/or indeterminate levels of settlement. Any existing uncontrolled fill should be removed from the building pad area and replaced with engineered fill as recommended in Section 6.4 where moisture conditioned soils are required or Section 7.3 where general fill is required. The excavated materials may be suitable for reuse as engineered fill provided they are free of organics, boulders, rubble, and other debris.

The lateral extent, depth and nature of the existing fill are not known. Test pits could be performed prior to construction to verify the presence, lateral extent, depth, and nature of the existing fill materials. ALPHA would be pleased to provide this service if desired.

6.2 Drilled and Underreamed Piers

Our findings indicate the building could be supported using a system of drilled and underreamed piers bearing in sandy clay at a depth of at least 15 ft below final grade.

Groundwater was encountered at depths of about 12 ft to 18 ft below the ground surface in the borings. Therefore, some field adjustments in the depth of the piers may be required in some areas to maintain the bottom of the piers above groundwater seepage. However, for pier constructability, the top of the pier bell should be located beneath any moisture conditioning or water pressure injected soils discussed later in this report. Adjustments in the depths of the piers should be observed in the field by ALPHA personnel. Immediate placement of concrete should also be expected. Test piers should be performed outside the building pad just prior to construction to verify groundwater conditions and constructability of underreamed piers. If underreams are not constructible, straight shaft piers with a diameter equivalent to the underream can be used provided moisture improvement as discussed in Section 6.5 is performed in the building pad.

Drilled and underreamed piers bearing in clay at a depth of at least 15 ft below final grade can be dimensioned using a net allowable end-bearing pressure of 4.0 kips per sq ft and no skin friction component of resistance. This bearing pressure contains a factor of safety of at least 3 considering a general bearing capacity failure. Normal elastic settlement of piers under loading is estimated to be less than about 1 inch.

Each pier should be designed with full length reinforcing steel 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 2.2 kips per sq ft. This soil adhesion is approximated to act uniformly over the portion of the pier shaft in contact with clay soils within 12 ft of final grade. A reduced uplift adhesion of 1.0 kips per sq ft can be used over the portion of the pier in contact with moisture conditioned soil. Uplift adhesion due to soil heave can be neglected over the portion of the pier shaft in contact with non-expansive material.

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.

6.3 Helical Piers (Alternative)

Since underreamed piers are prone to collapse, it could be desirable to use helical piers in lieu of drilled piers.

Helical piers are a manufactured foundation element consisting of a centralized steel shaft and one or more helical bearing plates. The helical plates are formed with a uniform-pitch screw thread, and the pier is installed by rotating it into the ground to the desired depth or refusal. The helical plate(s) provides end bearing resistance due to gravity loads and uplift resistance due to swelling of high shrink-swell active clays such as encountered at the boring locations.

Helical piers should bear at a minimum depth of 15 ft below the ground surface. All helixes should be situated at a depth of at least 15 ft below final grade. Vertical spacing between helixes along the shaft should be least three (3) helix diameters, based on the largest adjacent helix. The minimum helix diameter must be sized to prevent the bearing soils from being over-stressed and to develop sufficient uplift capacity to overcome the potential uplift forces acting on the pier. The helix portion should be at least three (3) times the width of the shaft. The minimum clear spacing between edges of adjacent piers should be at least two (2) helix diameters (based on the larger helix). Normal elastic settlement of helical piers under loading is estimated at less than about 1 inch.

Load capacity of helical piers bearing at a depth of at least 15 ft below the ground surface can be calculated by summing the allowable end bearing pressure of 4 kips per sq ft applied to the lowermost helix and a reduced bearing pressure of 3.4 kips per sq ft applied to any overlying helixes.

Each helical pier should 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 helical 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 2.2 kips per sq ft. This soil adhesion is approximated to act uniformly over the portion of the pier shaft in contact with clay soils within 12 ft of final grade. A reduced uplift adhesion of 1.0 kips per sq ft can be used over the portion of the pier in contact with moisture conditioned soil. Uplift adhesion due to soil heave can be neglected over the portion of the pier shaft in contact with non-expansive material.

From our experience, helical piers are frequently designed and installed by specialty contractors. Helical piers should be designed by a professional engineer and should be installed per the manufacturer's requirements. Helical piers should be load tested to verify the pier is capable of supporting the design load. Load tests can also be utilized to maximize the foundation load, thereby reducing the number of piers. We recommend performing at least one helical pier load test. ALPHA would be pleased to assist in design, implementation, and evaluation of a pier load test if desired.

6.4 Grade Beams

All grade beams connecting piers should be formed and not cast in earthen trenches. Grade beams should be formed with a nominal 12-inch void at the bottom. A reduced void space of 6 inches can be used if the subgrade is improved using moisture conditioning as recommended in Section 6.5. 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.

6.5 Floor Slab and Potential Seasonal Movements

Our findings indicate the floor slab constructed within 2 ft of existing grade could experience soil-related potential seasonal movements of about 6 inches due to shrinking and swelling of active clay soils. Floor slabs supported on uncontrolled fill are also subject to indeterminate levels of settlement.

Potential seasonal movements were estimated in general accordance with methods outlined by Texas Department of Transportation (TxDOT) Test Method Tex-124-E, from results of absorption swell tests and engineering judgment and experience. The 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 methods outlined in Texas Department of Transportation Test Method Tex-124-E. Also, it was assumed a 1 psi surcharge load from the floor slab acts on the subgrade soils. Movements exceeding those predicted herein could occur if the existing soils are exposed to an extended dry period, 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 migration from off-site locations.

In view of these potential seasonal movements, the most positive floor system for the buildings supported on piers is a slab suspended completely above the existing expansive soils. At least 12 inches of void space should be provided between the bottom of the floor slab, or lowest suspended fixture, and top surface of the underlying expansive clays. 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.

We understand it is desired to reduce the potential seasonal movements of the floor slab to about 1 inch. Movements could be reduced to about 1 inch by placing a minimum 2 ft cap of non-expansive fill between the bottom of the floor slab and the top surface of moisture conditioned or water pressure injected soils extending to a depth of 10 below the bottom of the non-expansive fill. Moisture conditioning is described in Section 6.5.1 and water pressure injection is described in Section 6.5.2. Non-expansive fill could consist of select fill or flexible base material as described in Section 7.3. In choosing these methods of foundation movement reduction, the Owner is accepting some post construction seasonal movement of the foundation (about 1 inch).

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. The 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.

6.5.1 Subgrade Improvement Using Moisture-Conditioning

Potential movements could be reduced to about 1 inch by placing a minimum 2 ft cap of non-expansive fill between the bottom of the floor slab and the top surface of moisture conditioned soils extending to a depth of 10 ft below the bottom of the non-expansive fill. Non-expansive fill could consist of select fill or flexible base material as described in Section 7.3.

Moisture-conditioning consists of processing and compacting the specified minimum thickness of on-site soil at a “target” moisture content approximated to be from 5 to 7 percentage points above the material’s optimum moisture content as determined by the standard Proctor method (ASTM D 698). The moisture-conditioned soil should be 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, at least 5 ft beyond the perimeter of the building and below any adjacent flatwork for which it is desired to reduce movements. At building entrances and outward swinging doors, moisture conditioning should extend at least 10 ft beyond the building perimeter. However, non-expansive material should not extend beyond the building limits. If flatwork or paving is not planned adjacent to the structure (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 non-expansive material 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.

The purpose of moisture-conditioning is to reduce the free swell potential of the moisture-conditioned soil to 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.

Moisture conditioning should be monitored and tested on a full-time basis by ALPHA to verify materials tested are placed with the proper degree of moisture and compaction as presented in this report. Field density tests should be performed for each lift of fill placed in each building pad area.

6.5.2 Subgrade Improvement Using Water Pressure Injection (WPI)

As an alternative, installation of 10 ft of water pressure injection (WPI) in conjunction with a 2-ft cap of non-expansive fill could reduce potential ground movements to about 1 inch. Non-expansive material could consist of select fill or flexible base material as discussed in Section 7.3.

Any uncontrolled fill present in the building pad area should be removed and replaced in accordance with Section 6.1 and Section 7.3 prior to water pressure injection.

Water pressure injection improvement procedures:

- Following removal of the necessary thickness of on-site expansive soils to allow for placement of at least 2 ft of non-expansive fill, the exposed subgrade should be water pressure injected (WPI) to a depth of 10 ft below the bottom of the non-expansive fill. The water pressure injection should extend throughout the entire building area, at least 5 ft beyond the perimeter of the building and below any adjacent flatwork for which it is desired to reduce potential movements. At building entrances and outward swinging doors, WPI should extend at least 10 ft beyond the building perimeter. However, the non-expansive fill should not extend beyond the building limits. Recommended specifications for WPI are attached to this report in the appendix.

- If flatwork or paving is not planned adjacent to the structure (i.e. above the injected 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 injected soils. Injected soils should be maintained in a moist condition prior to placement of the required thickness of select, non-expansive material or flatwork.

Performance of post-injection swell testing and moisture content determinations should be employed as final acceptance criteria in engineering analysis to examine accomplishment of intended objectives of the injection treatment. Maximum benefit of these movement reduction procedures can be achieved by employing ALPHA to observe, monitor and test the entire process. Construction specifications for the water pressure injection process are provided in the Appendix.

The purpose of water pressure injection is to pre-swell the existing soils. Satisfactory completion of the injection process is achieved when the desired moisture content and abatement of swell in the injected subgrade clay soils are reached. Acceptance criteria for water pressure injection should be based upon obtaining an average free swell of 1 percent or less in the injected zone. Performance of post-injection swell testing and moisture content determinations should be employed as final acceptance criteria in engineering analysis to verify accomplishment of intended objectives of the injection treatment.

The resulting estimated potential seasonal movement (about 1 inch) was calculated assuming the average free swell of the injected soils does not exceed 1 percent. Further, it is assumed the moisture content of the soil below the injected zone and within the normal zone of seasonal moisture content change varies between a "dry" condition and a "wet" condition as defined by TxDOT Test Method Tex-124-E.

6.6 Slab-on-Grade Foundation (Alternative)

As an alternative, the building could be supported with slab on grade foundations. A slab foundation will be subject to similar potential movements as a grade supported floor slab as discussed in Section 6.5. Subgrade improvement as discussed in Section 6.5 will be required to reduce potential movements of the slab foundation to about 1 inch.

The slab foundation should be designed with exterior and interior grade beams adequate to provide sufficient rigidity to the foundation system. A net allowable soil bearing pressure of 1.5 kips per sq ft may be used for design of the grade beams bearing on a moisture improved subgrade as discussed in Section 6.5. Grade beams should bear a minimum depth of 18 inches below final grade and should have a minimum width of 10 inches.

To reduce cracking as normal movements occur in foundation soils, all grade beams and floor slabs should be adequately reinforced with steel. It is common to experience some minor cosmetic distress to structures with slab-on-grade foundation systems due to normal ground movements. A properly designed and constructed moisture barrier should be placed between the slab and subgrade soils to retard moisture migration through the slabs.

6.6.1 Post-Tensioning Institute, Design of Post-Tensioned Slab-on-Grade

Table B contains information for design of the post-tensioned, slab-on-grade foundations. Design parameters provided below were evaluated based on the conditions encountered in the borings and using information and correlations published by PTI Third Edition and VOLFLO 1.5 computer program provided by Geostructural Tool Kit, Inc. (GTI).

TABLE B
PTI Design Parameters
Potential Seasonal Movement = 1 inch
(After Subgrade Treatment as Described in Section 6.5)

	EDGE LIFT	CENTER LIFT
Edge Moisture Distance, ft (em)	4.0	7.5
Differential Soil Movement, inches (y_m)	1.2 (swell)	1.0 (shrink)

6.7 Subgrade Improvement Considerations for Flatwork

Flatwork planned for the project is subject to potential seasonal movements as discussed for the floor slab in Section 6.5 (about 6 inches). If this level of movement is not acceptable, flatwork can be structurally suspended on drilled piers as discussed in Section 6.2 or Section 6.3. As an alternative, potential movement of flatwork could be reduced to about 1 inch following the recommendations for subgrade improvement discussed in Section 6.5.

6.8 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 the assumption that the subsurface materials below the bottom of the borings were similar to those encountered at the termination depth. 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 D (stiff soil profile) for seismic design at this site.

6.9 Area Pavement

To permit correlation between information from test borings and actual subgrade conditions exposed during construction, 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 used and conventional thickness determination procedures. Pavement joining 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 all 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.

The recommended pavement sections 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 recommended.

6.9.1 Pavement Subgrade Preparation

The exposed surface of the pavement subgrade soil should be scarified to a depth of 6 inches and mixed with a minimum 8 percent hydrated lime (by dry soil weight) in conformance with TxDOT Standard Specification Item 260. Assuming an in-place unit weight of 100 pcf for the pavement subgrade soils, this percentage of lime equates to about 36 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. 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.

Please note, the on-site soils can contain a sufficient quantity of soluble sulfates that can adversely react with hydrated lime. Therefore, before committing to mechanical lime stabilization, samples of the pavement subgrade soil should be tested for the quantity of soluble sulfates. Our office should be contacted regarding evaluation of the quantity of soluble sulfates detected and any special processing/design features that may be applicable.

We recommend subgrade improvement 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.

Improvement 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. Good perimeter surface drainage with a minimum slope of 2 percent away from the pavement is recommended. Normal maintenance of pavement should be expected over the life of the structures.

6.9.2 Portland Cement Concrete (PCC) Pavement

Following subgrade improvement as recommended in Section 6.8.1, PCC (reinforced) pavement sections are recommended in Table B.

TABLE B Recommended PCC Pavement Sections		
Paving Areas and/or Type	**Subgrade Thickness, Inches**	**PCC Thickness, Inches**
Parking Areas Subjected Exclusively to Passenger Vehicle Traffic,	Scarified and Compacted (native), 6	5
Drive Lanes, Fire Lanes, Areas Subject to Light Volume Truck Traffic	Lime Modified, 6	6
Dumpster Traffic Areas, Areas subject to Moderate Volume Truck Traffic,	Lime Modified, 6	7

PCC should have a minimum compressive strength of 3,000 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 drive lanes, fire lanes, and truck areas. Concrete should be designed with 5 ± 1 percent entrained air. Joints in concrete paving should not exceed 15 ft. Reinforcing steel should consist of No. 3 bars placed at 18 inches on-center in two directions.

Improvement of the pavement subgrade is recommended for drive lanes, fire lanes and pavement subject to truck traffic. Improvement of the pavement subgrade is not necessary for pavements subjected exclusively to passenger vehicle traffic, although improvement in these areas would be generally beneficial to the long-term performance of the pavement. Improvement of the subgrade is described in Section 6.8.1.

Alternatively, mechanical improvement of the pavement subgrade could be eliminated by increasing the PCC thickness in the pavement sections presented in Table B by 1 inch. Prior to construction of pavement on unimproved 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 to +3 percentage points of the material's optimum moisture content.

6.10 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 10 ft of the structure should be sloped away from the building to prevent ponding of water around the foundation. Final grades within 10 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 flat-work is common. Normal maintenance should include inspection 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 system:

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

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

- Moisture conditions should be maintained "constant" around the edge of the slab. 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.

- 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.

Trench backfill for utilities should be properly placed and compacted as outlined in Section 7.4 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 structures. 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 GUIDELINES

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 reasonably anticipated until the course of construction. The guidelines 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

Existing fill could be present on the site as discussed in Section 2.0. Although not encountered in the borings, existing fill materials can also contain organics, boulders, rubble, and other debris which could be encountered during site grading and general excavation. The earthwork and excavation contracts should contain provision for removal of unsuitable materials in the existing fill. Test pit excavations performed prior to construction can be used to evaluate the depth, extent and composition of existing fill at this site. ALPHA would be pleased to provide this service if desired.

All areas supporting flatwork, pavement or 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 20 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 proof-rolling 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 re-compacted 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 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 as described in Section 7.3, 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 planning or placing deep fills.

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 clay 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 properly monitored to verify loose, soft or otherwise undesirable materials are removed and foundations will bear on satisfactory material. Soil exposed in the base of all foundation (pier and footing) excavations should be protected against detrimental change in condition, such as surface sloughing or side disturbance, rain or excessive drying. Drilled pier foundations should be completed in one day.

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 piers and slab foundation grade beams should be slightly deepened and cleaned.

All pier shafts should be at least 1.5 ft in diameter to facilitate clean-out of the base and for 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.

Free groundwater was encountered in Borings 1 and 2 at depths of about 12 ft to 18 ft below the ground surface. From our experience, groundwater seepage could be encountered during construction. The risk of encountering groundwater during pier drilling is increased during or after periods of precipitation. Some field adjustments in the depth of the piers may be required in some areas to maintain the bottom of the piers above groundwater seepage. Adjustments in the depths of the piers should be observed in the field by ALPHA personnel. Also, the clay soils encountered at the boring locations are prone to collapse during construction of the underreamed portion of the pier foundation. 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. Temporary casing may be useful for controlling groundwater seepage that could occur in the clay soils. 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.

7.3 Fill Compaction

Select fill used as non-expansive material in the building pad should have a liquid limit less than 35, a plasticity index (PI) not less than about 4 nor greater than 15 and contain no more than 0.5 percent fibrous organic materials, by weight. All select material should contain no deleterious material and should be compacted to a dry density of at least 95 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. The plasticity index and liquid limit of material used as select non-expansive material should be routinely verified during placement using laboratory tests. Visual observation and classification should not be relied upon to confirm the material to be used as select, non-expansive material satisfies the Atterberg-limit criteria.

Flexible base used as non-expansive material in the building pad should consist of material meeting the requirements of TxDOT Standard Specifications Item 247, Type A or D, Grade 1-2 or 3. The flexible base should be compacted to at least 95 percent of standard Proctor maximum dry density (ASTM D 698) and within the range of 2 percentage points below to 2 percentage points above the material's optimum moisture content.

The following recommendations pertain to fill soils placed for general site grading as follows:

- *Outside* the designated building pad areas *if* moisture conditioning will be used as the method for subgrade improvement. Where moisture conditioning is used for subgrade improvement, all fill within the designated building pad areas and associated adjacent areas should meet the requirements of Section 6.5.

- For general grading *including* building areas below the select fill requirement *if* water pressure injection as discussed in Section 6.5 will be used as the method for subgrade improvement.

Clay and sandy clay 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.

Clay with a plasticity index less than 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 such that the largest particle or clod is less than 6 inches prior to compaction.

Where mass fills are deeper than 10 ft, 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.

In general site grading areas where final fill slopes will be four horizontal to one vertical (4:1) or steeper and greater than 5 ft in height, field density and moisture content tests should be performed on each lift.

7.4 Utilities

Where utility lines are deeper than 10 ft, the fill/backfill below 10 ft should be compacted to at least 100 percent of standard Proctor maximum dry density (ASTM D 698) and within –2 to +2 percentage points of the material's optimum moisture content. The portion of the fill/backfill shallower than 10 ft should be compacted as previously outlined. Density tests should be performed on each lift (maximum 12-inch thick) and should be performed as the trench is being backfilled.

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 pavement over utility lines and/or other areas with deep fill.

If utility trenches or other excavations extend to or beyond a depth of 5 ft below construction grade, the contractor or others shall be required to develop an excavation safety plan to protect personnel entering the excavation or excavation vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, is beyond the scope of this study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

7.5 Groundwater

Groundwater was encountered in Borings 1 and 2 at a depth of about 12 ft to 18 ft below the ground surface. From our experience, shallower groundwater seepage could be encountered in excavations for foundations, utilities and other general excavations at this site. 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.

In any areas where cuts are made, attention should be given to possible seasonal water seepage that could occur through natural cracks and fissures in the newly exposed stratigraphy. In these areas 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 final grades 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 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 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.

APPENDIX

SOIL MODIFICATION
WATER PRESSURE INJECTION (WPI)
GUIDELINE SPECIFICATIONS

Purpose

The purpose of this specification is to provide a procedural basis for using water pressure injection as a method to obtain a relatively uniform, moist, pre-swelled zone of soil beneath the floor slab. Specifically, the intent of this procedure is to reduce the average free swell potential of soils within the injected zone to 1 percent or less.

Material

1. Only potable water shall be used during the entire injection process.

2. A non-ionic surfactant (wetting agent) will be added to the water according to manufacturer's recommendations, but, in no case will proportions be less than one part (undiluted) per 3,500 gallons of water.

Application

1. The water pressure injection work shall be accomplished after the site has been brought to near final subgrade elevation and prior to installation of any plumbing, trenches and utilities.

2. The injection vehicle will have a minimum gross weight of 5 tons and shall be capable of making straight vertical penetrations to minimize pressure loss around the injector rods to at least 10 ft.

3. Injections will be continued to "REFUSAL" (until the maximum reasonable quantity of water has been injected into the soil and water is flowing freely at the surface, either out of previous injection holes or from areas where the surface soils have fractured. The amount of water flowing from the areas described above will be approximately equivalent to the volume of water being pumped into the soil. As a minimum, injections should be at least 30 seconds at each injection interval unless altered by the Geotechnical Engineer).

   Note: Loss of water or blow-back around injector pipes does not constitute refusal. Continued loss of water in this manner may indicate inadequate injection equipment or techniques, or in some instances, surficial soils that will not form an adequate seal to contain the water. In either instance, the owner's representative should be contacted and an on-site observation made to determine appropriate steps to achieve adequate injection.

   After completion of water injection, the injection contractor will submit records which reflect the total quantity of water used. The injection contractor will be totally responsible for determining the means and methods of injecting the on-site soils such that the average free swell of soils within the injected zone does not exceed 1 percent.

4. Injection pipe(s) will penetrate the soil in approximately 12 to 18-inch intervals, injecting to refusal at each interval for a total depth of 10 ft or impenetrable material, whichever occurs first. If a seemingly impenetrable layer is encountered, ALPHA must be contacted to evaluate the significance of the lack of penetration with the injector tubes or provide alternate recommendations. A minimum of seven (7) injection intervals will be provided for the 10-ft injection depth. The lower portion of the injection pipe will consist of a hole pattern that will uniformly disperse water throughout the entire depth.

5. Spacing for the injections will not exceed 5 feet on-center each way. Subsequent injections will be offset laterally at one-half the distance in both directions between the original injection centers.

6. Injection pressures should be adjusted to inject the greatest quantity of water possible within a pressure range of 50 - 200 psi pump pressure.

7. After a minimum curing time of 48 hours, the water injected pad shall be tested for moisture content and swell abatement to determine if additional injections with water are necessary. Subsequent water injections will be 5 feet on-center each way and spaced 2½ feet offset in two orthogonal directions from the initial injection.

8. Upon completion of the final water pressure injection, the top surface of the injected pad should be scarified to a depth of at least 6 inches and re-compacted to between 93 and 98 percent of the optimum density, at a moisture content between 2 and 4 percentage points above the optimum values, as defined by ASTM D-698. Compaction tests should be performed at a frequency of one (1) test per 5,000 sq ft with a minimum of two (2) tests.

9. The moisture content of the injected soils will be maintained until the floor slab is placed. Loss of moisture from the surface or sides of the building pad must be prevented by watering or use of a membrane. Any open trenches should be sealed or kept wet to prevent loss of moisture. All trenches should be backfilled with the excavated material. The moisture content of the backfill should be maintained in the range of 2 to 4 percentage points above optimum.

Special Considerations

Several water injections may be required to achieve the desired final moisture content and corresponding soil swell abatement. A minimum 24 hour waiting period should be implemented between water injection passes. Due to variations in the subsurface soils, the number of injection passes required to reduce the swell potential of the injected soils to 1 percent or less is unknown. Hence, the Client should allow for extra construction time on the site considering the time frame required to achieve the desired reduction in swell potential is unknown. Further, the contract with the Injection Contractor should address the situation where more injection passes than predicted are required to achieve the desired result.

Between the time the subgrade is water pressure injected and either the select fill material or plastic sheeting is placed, the upper surface of the injected soil should not be allowed to dry. To allow for adequate pre-swelling of the soils from the injection procedure, concrete for slabs should not be placed above injected areas until at least two (2) weeks following the final water injection. During this two-week period, the surface of the injected soil must be kept moist or covered with plastic sheeting to prevent moisture loss. About 3 to 5 inches of heave can be expected in the building pad during and shortly after completion of the injection process.

Additionally, experience indicates injection adjacent to existing structures (such as, but not limited to, buildings, pavements, grade slabs, and buried utility conduits) can result in swelling of soil in the injected zone as well as those beneath existing nearby structures. Swelling of soil supporting existing structures can result in distress (movement) to existing structures. Therefore, if an existing structure or property line is located within 30 ft of the proposed water injection area, it is recommended a temporary vertical moisture barrier be installed longitudinally between the existing structure and the injected pad to prevent injected water from entering the subgrade of the existing structure. The moisture barrier could consist of a 12-ft deep trench (about 1 ft wide) backfilled with lean concrete or other suitable relatively impermeable material.

Monitoring

A full-time ALPHA technician should be retained and present throughout the injection operations. Moisture content and free swell samples should be taken at 1-foot intervals to the total depth injected from a minimum of one (1) test boring per each 4,000 sq feet of injected area with a minimum of two (2) test borings. The moisture content and shear strength (using a pocket-penetrometer) will be determined for each sample. One-dimension free swell tests (ASTM D 4546-85 Method B) will be performed on selected samples at a frequency of at least three (3) free swell tests per test boring. The free swell tests will be performed with a surcharge equal to the overburden pressure anticipated upon completion of the new structure. Based upon the test results, the current swell potential of the injected soils should be determined by the project Geotechnical Engineer. Acceptance criteria for water pressure injection will be based upon achieving the potential movements indicated in the Geotechnical Exploration. As a guide, an average free swell of 1 percent or less in the injected zone could be used for planning. However, due to variations in the soils across the site, an average free swell of more than 1 percent may be allowable in some areas. Acceptance of soils with average free swells of more than 1 percent should be evaluated by ALPHA. Depending upon the moisture content and the potential swell remaining in the existing injected soils, additional injections with water containing surfactant may be required until these requirements are met.

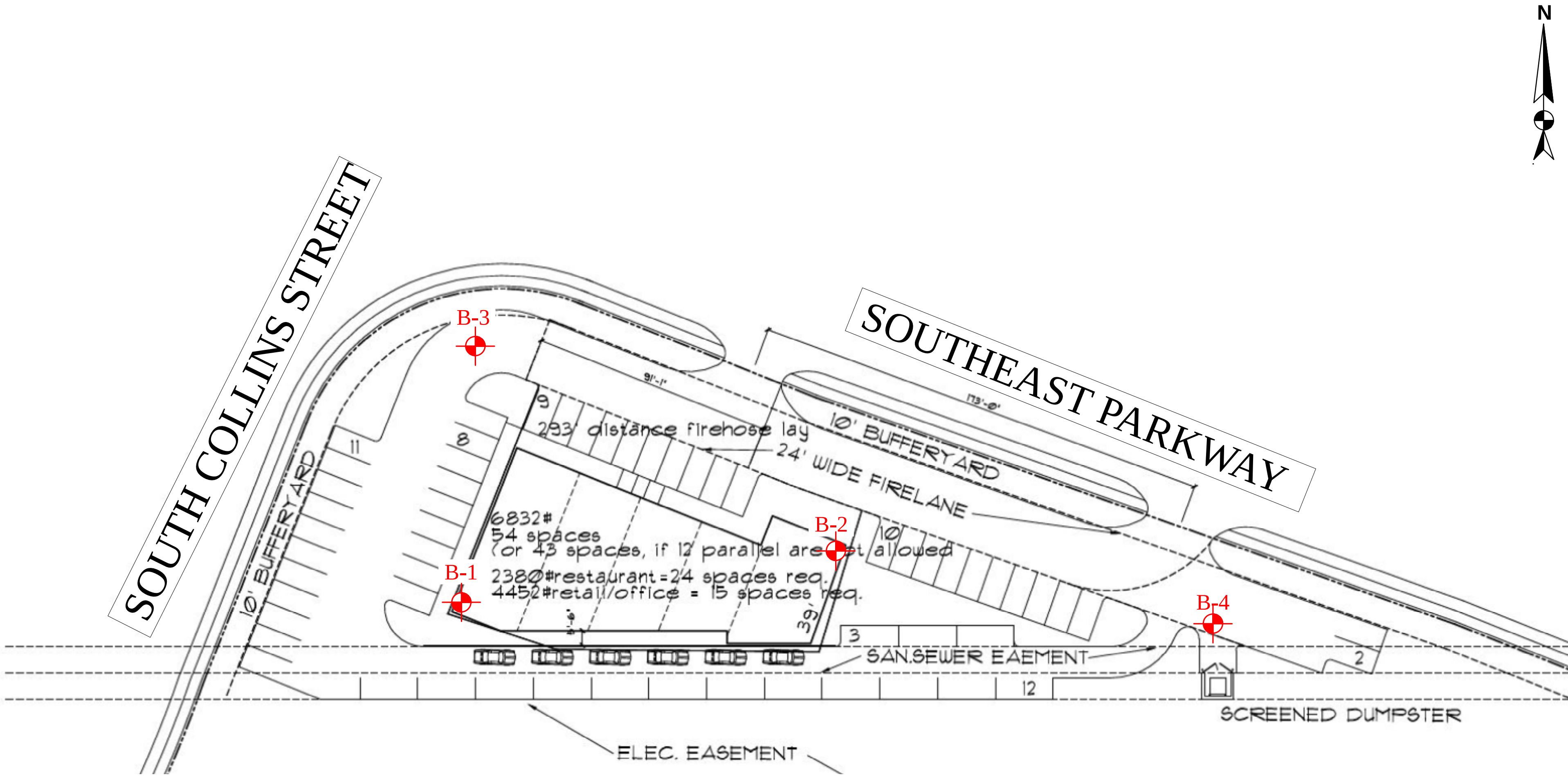
Wet and soft surface conditions resulting from the water injection procedures will require the contractor to provide access to drilling equipment used to obtain the soil samples which verify the injection process. Special track equipment may be required to provide the required access. The contractor will be responsible for providing and operating suitable equipment to permit sampling of the injected soils (test borings) with a standard truck-mounted drilling rig.

A-1 METHODS OF FIELD EXPLORATION

Using standard rotary drilling equipment, four (4) 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 using a handheld GPS unit or by pacing/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 methods used to define them.

Relatively undisturbed soil samples 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 evaluated visually. One representative portion of each sample was sealed in a plastic bag for use in future visual evaluation and possible testing in the laboratory.

Logs of the borings are included in the Appendix. The logs show visual descriptions of subsurface strata encountered in the borings 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.



GEOTECHNICAL EXPLORATION
PROPOSED RETAIL CENTER
SEC OF SOUTH COLLINS STREET
AND SOUTHEAST PARKWAY
GRAND PRAIRIE, TEXAS
ALPHA PROJECT NO. W200872

ALPHA TESTING
WHERE IT ALL BEGINS

FIGURE 1

BORING LOCATION PLAN

APPROXIMATE BORING LOCATION

B-1 METHODS OF LABORATORY TESTING

Representative samples were evaluated 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 compressive strength tests (ASTM D 2166) and pocket-penetrometer tests were conducted on selected soil samples to evaluate the soil shear strength. Results of these laboratory tests are provided on the 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.

Boring No.	Sample Depth (ft)	Pressure (psf, vert.)	Material Description	Liquid Limit	Plastic Limit	Plasticity Index	Initial Moisture	Final Moisture	Free Swell
1	14	1750	Brown SANDY CLAY	60	20	40	19%	22%	0.3%
2	9	1125	Dark Brown CLAY	76	25	51	18%	24%	5.5%

SWELL TEST RESULTS

W200872 - Proposed Retail Center

SEC of S. Collins Street and Southeast Parkway, Arlington, TX

FIGURE 2

ALPHA TESTING
WHERE IT ALL BEGINS

5058 Brush Creek Rd.
Fort Worth, Texas 76119
Phone: 817-496-5600
Fax: 817-496-5608
www.alphatesting.com

BORING NO.: 1
Sheet 1 of 1

PROJECT NO.: W200872

Client: GEES, Inc.
Project: Proposed Retail Center
Start Date: 4/8/2020 **End Date:** 4/8/2020
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Arlington, TX
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 140 / 30

GROUND WATER OBSERVATIONS
▽ On Rods (ft): 13
▼ After Drilling (ft): 12
▼ After ____ Hours (ft):

Depth, feet	MATERIAL DESCRIPTION	Bottom Depth	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
0	Dark Brown CLAY												
1					2.75				22				
3					2.25				23				
5					2.25	3.9		98	24				
7		8.0			3.25				23				
8	Light Brown CLAY with calcareous deposits												
9					4.5+				18	71	23	48	
12	▼	12.0											
12	▽ Brown SANDY CLAY												
14					4.5+				19	60	20	40	0.3
19		22.0			4.25				21				
22	Tan CLAY												
24		25.0			4.25	3.8		105	19	89	28	61	
25	TEST BORING TERMINATED AT 25 FT												

ALPHA TESTING
WHERE IT ALL BEGINS

5058 Brush Creek Rd.
Fort Worth, Texas 76119
Phone: 817-496-5600
Fax: 817-496-5608
www.alphatesting.com

BORING NO.: 2
Sheet 1 of 1

PROJECT NO.: W200872

Client: GEES, Inc.
Project: Proposed Retail Center
Start Date: 4/8/2020 **End Date:** 4/8/2020
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Arlington, TX
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 140 / 30

GROUND WATER OBSERVATIONS
▽ On Rods (ft): 18
▼ After Drilling (ft): 16
After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
		Dark Brown CLAY												
1						2.5				26				
3						2.0	1.6		101	21				
5						2.5				23				
7						3.0				24				
9						4.5				20	76	25	51	5.5
12.0		Tan SANDY CLAY												
14						4.5	3.3		107	21				
16	▼													
18	▽													
19						4.5				19	48	16	32	
24						4.5+				21				
25.0		TEST BORING TERMINATED AT 25 FT												

ALPHA TESTING
WHERE IT ALL BEGINS

5058 Brush Creek Rd.
Fort Worth, Texas 76119
Phone: 817-496-5600
Fax: 817-496-5608
www.alphatesting.com

BORING NO.: 3
Sheet 1 of 1

PROJECT NO.: W200872

Client: GEES, Inc.
Project: Proposed Retail Center
Start Date: 4/8/2020 **End Date:** 4/8/2020
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Arlington, TX
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 140 / 30

GROUND WATER OBSERVATIONS
▽ On Rods (ft): NONE
▼ After Drilling (ft): DRY
▼ After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
		Dark Brown CLAY				2.5				23				
						2.5				25	67	20	47	
5		5.0								25				
		TEST BORING TERMINATED AT 5 FT												
10														
15														
20														
25														

ALPHA TESTING

WHERE IT ALL BEGINS

5058 Brush Creek Rd.
Fort Worth, Texas 76119
Phone: 817-496-5600
Fax: 817-496-5608
www.alphatesting.com

BORING NO.: 4

Sheet 1 of 1

PROJECT NO.: W200872

Client: GEES, Inc.
Project: Proposed Retail Center
Start Date: 4/8/2020 **End Date:** 4/8/2020
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Arlington, TX
Surface Elevation:
West:
North:
Hammer Drop (lbs / in): 140 / 30

GROUND WATER OBSERVATIONS
▽ On Rods (ft): NONE
▼ After Drilling (ft): DRY
▽ After ____ Hours (ft):

Depth, feet	Graphic Log	MATERIAL DESCRIPTION	Sample Type	Recovery % RQD	TX Cone or Std. Pen. (blows/ft, in)	Pocket Penetrometer (tsf)	Unconfined Comp. Strength (tsf)	% Passing No. 200 Sieve	Unit Dry Weight (pcf)	Water Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Swell, %
		Dark Brown CLAY				4.0				23	55	20	35	
						2.75				26				
5		5.0				3.0				24				
		TEST BORING TERMINATED AT 5 FT												
10														
15														
20														
25														

KEY TO SOIL SYMBOLS AND CLASSIFICATIONS

ALPHA TESTING
WHERE IT ALL BEGINS

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

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)

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

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