GEOTECHNICAL EXPLORATION

NEW RESIDENCE
351 Eagle Point Road
Van Alstyne, Texas
ALPHA Report No. G210510
April 22, 2021

Prepared for:

MR. FRED LEAL
[redacted]

Prepared By:

ALPHA TESTING

WHERE IT ALL BEGINS

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Geotechnical
Construction Materials
Environmental
TBPE Firm No. 813

2209 Wisconsin Street
Dallas, Texas 75229

Tel: 972-620-8911
Fax: 972-620-1302
www.alphatesting.com

April 22, 2021

Mr. Fred Leal
[redacted]

Re: Geotechnical Exploration
New Residence
351 Eagle Point Road
Van Alstyne, Texas
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Attached is the report of the geotechnical exploration performed for the referenced project. This study was authorized by Mr. Fred Leal on February 15, 2021 and performed in accordance with ALPHA Proposal No. 82115 dated February 1, 2021.

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 a single-family residence.

ALPHA TESTING, INC. appreciates the opportunity to be of service on this project. If we can be of further assistance, please contact our office.

Sincerely,

ALPHA TESTING, INC.

HAddula

STATE OF TEXAS
HARSHA R. ADDULA
120198
LICENSED
PROFESSIONAL ENGINEER
4/22/21

Harsha R. Addula, P.E.
Geotechnical Department Manager

Mark L. McKay, P.E.
Associate Principal

Copy: (1) Mr. Fred Leal

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ALPHA REPORT NO. G210510

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 construction. The field exploration was accomplished by securing subsurface samples from widely spaced test borings performed across the expanse of the subject building pad. 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 were developed from information obtained in test borings depicting subsurface conditions only at the specific test boring locations and at the particular time designated on the Log of Borings (boring logs). Subsurface conditions at other locations may differ from those observed at the test boring locations, and subsurface conditions at test 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 test 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

The project site is located at 351 Eagle Point Road in Van Alstyne, Texas. The approximate locations of the test borings are shown on the Boring Location Plan – Figure 1, enclosed in the Appendix. At the time of the field exploration, the site was relatively open. According to the topographic map provided by the client on April 21, 2021, the site in the vicinity of the proposed building slopes downward toward the west with a maximum surficial elevation change of about 12 ft (Elevation 85 to 73).

Present plans provide for the construction of a single-family residential structure. The proposed structure is anticipated to create light loads to be carried by the foundation. Based on our conversations with the Client, we understand the residential structure will be supported using a post-tensioned, slab-on-grade foundation system for potential seasonal movements of 3 inches. No below-grade slabs are planned.

Grading plans were not available at the time of this investigation. Based on our conversations with the client, we have assumed cuts and fills of up to 4 ft to establish the final grade in the building area.

Retaining walls may be planned in some areas of the site. These retaining walls should be properly designed and constructed. Specific design parameters (including global stability) for retaining walls are considered beyond the scope of this study. If design parameters for retaining walls are desired, our office should be contacted.

3.0 FIELD EXPLORATION

Using standard rotary drilling equipment, subsurface conditions were explored by drilling two (2) widely spaced test borings (B-1 to a depth of 25 ft and B-2 to a depth of 20 ft) in general accordance with ASTM D 420. 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 Log of Borings (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, 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 on Log of Borings, enclosed in the Appendix.

5.0 GENERAL SUBSURFACE CONDITIONS

Based on the Geologic Atlas of Texas (Sherman Sheet) from the Texas Bureau of Economic Geology, published by the University of Texas at Austin, the project site is mapped in the Austin Chalk formation. The Austin Chalk generally consists of massive gray unweathered limestone, overlain by tan weathered limestone. Residual overburden soils associated with Austin Chalk formation generally consist of clays with very high shrink/swell potential. The materials encountered at the borings appear consistent with the Austin Chalk formation.

Within the 20 and 25-ft depths explored at the site, subsurface materials encountered in the test borings generally consists of clay (CH) soils to a depth of about 10 ft underlain by gray shaly limestone to the termination depths of 20 ft and 25 ft. The letters in parentheses represent the soils' classification according to the Unified Soil Classification System (ASTM D 2488). More detailed stratigraphic information is presented on the Log of Borings, included in the Appendix.

During the field exploration, free groundwater was not encountered on drilling tools or in the open boreholes immediately upon completion of drilling the borings. These observations provide an indication of the groundwater conditions present at the time the borings were drilled. It is common to detect shallow seasonal groundwater in the fill soils, within the native clayey matrix, near the soil/rock (shaly limestone) interface or from fractures in the rock, particularly during or after periods of precipitation. 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. 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 Borings, 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). If project criteria should change, 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.

The design criteria given were developed assuming final grade is established within 4 ft of existing grade using on-site soil or similar imported soil with a plasticity index (PI) of 50 or less. Further cutting and filling on the site beyond that assumed or utilizing soils with a PI higher than 50 may require modifications of the foundation design parameters. Therefore, if changes to the project plans and specifications are made our office must be contacted to verify the appropriate design parameters are utilized for final foundation design.

6.1 Potential Seasonal Movements

Based on the subsurface conditions encountered at this site and considering final grade within 4 ft of existing grades, our findings indicate slab-on-grade foundation systems for the residential structure could experience soil-related seasonal movements (potential vertical rise, PVR) in the order of 5 to 6 inches based on dry soil conditions that may exist prior to construction.

The potential seasonal movement was 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 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.

Potential movements could be reduced to not more than 3 inches by installation of 10 ft of moisture conditioned soil in conjunction with placement of 6 to 8-mil plastic sheeting. Alternately, potential movements could be reduced to not more than 3 inches if the upper 10 ft of on-site clays (below final grade) are chemically injected and then covered with 6 to 8-mil plastic sheeting. These methods of subgrade improvement are described in detail in Sections 6.1.1 and 6.1.2.

In choosing these methods of slab movement reduction, the Client is accepting some post-construction movement of slab-on-grade foundations (about 3 inches). Therefore, the Client understands and acknowledges that in the geographical region covered by this report, a PVR of 4½ inches is considered a reasonable compromise between foundation design and construction cost and the amount of allowable movement of the foundation.

Please note, improvement of the subgrade soils (using moisture conditioning and/or WPI, including placement of plastic sheeting) are intended only for the designated building pad, plus

5 ft beyond the building pad, and not the entire residential lot. Accordingly, planned residential structures must be exclusively constructed within the building pad areas designated on the project grading plan. The purpose of placement of the plastic sheeting referenced is to maintain the moisture of the underlying subgrade soils relatively the same from the time the plastic sheeting is placed through the time the foundation is placed. This plastic sheeting is not intended as a moisture barrier component for the actual foundation. Any such requirements should be addressed by the designer of the foundation, and should be followed by the builder. Prior to building on the improved designated building pad area, a surveyor should verify the lateral extent of the plastic sheeting and to confirm that no portion of the new residences will extend beyond the limits of the designated building pad areas. If any part of the slab footprint extends beyond the designated building pad, ALPHA should be contacted for additional design recommendations.

6.1.1 Subgrade Improvement Utilizing Moisture Conditioning

Movements of grade-supported floor slab or slab-on-grade foundations can be reduced to the desired level of 3 inches by moisture conditioning the upper 10 ft of soil below final grade in conjunction with placement of plastic sheeting.

Please note: Based on the subsurface conditions encountered in the borings and proposed final grade, it appears shaly limestone will be encountered within 10 ft below final grade in some areas of the proposed building. It is not necessary to over-excavate the shaly limestone to install the recommended moisture-conditioned soils. During construction, *qualified geotechnical personnel should verify the presence of shaly limestone in all areas where moisture conditioning does not extend to the intended depth.*

Moisture-conditioning consists of over-excavating (where necessary) and/or filling with on-site soil that is compacted at a “target” moisture content at least 5 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 between 93 and 98 percent of standard Proctor maximum dry density. Moisture conditioning with on-site soil should extend throughout the entire designated building pad areas, pool deck, under all adjoining flatwork and at least 5 ft beyond the perimeter of the building pad and pool deck areas. In major entrance areas, moisture conditioning should extend at least 10 ft beyond the perimeter of the building pad areas. Upon completion of moisture conditioning, the entire building pad areas plus 5 ft beyond the building pad areas should be covered with plastic sheeting. Moisture-conditioned soils should be maintained in a moist condition prior to placement of the moisture barrier, pavement or flatwork.

It is the intent of the moisture-conditioning process to reduce the swell potential of the moisture-conditioned soils to about 1 percent or less. Additional laboratory tests (i.e., standard Proctors, absorption swell tests, etc.) should be conducted during construction to verify that 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 swell potential of the processed soil to about 1 percent or less.

The resulting PVR (3 inches) 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.

Installation of moisture-conditioned soils should be observed and tested on a full-time basis by a representative of ALPHA to verify the moisture-conditioned soils are placed with the proper lift thickness, moisture content and density.

6.1.2 Subgrade Improvement Utilizing Chemical Injection

Movement of the grade-supported floor slab or slab-on-grade foundations can be reduced to about 3 inches by chemically injecting the soils to a depth of 10 ft below final grade or to the top of the shaly limestone in conjunction with placement of plastic sheeting.

Gray shaly limestone was encountered at a depth of about 10 ft below existing grade. Based on the subsurface conditions encountered in the borings and proposed final grade, it appears shaly limestone will be encountered within 10 ft below final grade in some areas of the proposed building. If injector tubes encounter penetration refusal due the presence of shaly limestone, injection may be terminated at that depth. However, it is recommended an Engineer from ALPHA verify the presence of shaly limestone in all areas where injection does not extend to the intended depth. This verification will require test borings and laboratory testing (i.e., free swell tests), in some instances.

The chemical injection should extend throughout the entire building pad area and at least 5 ft beyond the perimeter of the building pad. Upon completion of chemical injection, the entire injected area (including 5 ft beyond the designated building pad) should be covered with a heavy plastic sheeting (minimum 6 to 8 mil thickness plastic sheeting). The plastic sheeting should be placed above the injected soils within one (1) week after completion of improvement for long-term maintenance of the injected soil. Then, 8 to 12 inches of compacted clay soil cover should be provided over the plastic sheeting to protect the plastic long-term from deterioration due to exposure to sunlight. The use of sandy soils (soil with a plasticity index less than 20) above the plastic should be avoided. Injected soils should be maintained in a moist condition prior to placement of the plastic sheeting.

Chemical injection is performed by injecting the clayey soils with a proprietary chemical specifically formulated for long-term reduction of shrink-swell capacity in expansive clayey soils. The Client should obtain appropriate documentation from the manufacturer indicating the chemical is environmentally safe, long lasting and the injection process will not affect any adjacent existing structures. The injection contractor should provide references, and references should be obtained and verified. Also, chemical injection proposals should only be considered from injection contractors whose chemicals and processes have been shown to be effective through studies at major U.S. research universities.

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 injection should be based upon obtaining an average swell of about 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 examine accomplishment of intended objectives of the injection treatment.

Moisture content and swell samples should be taken at 1-ft intervals to the total depth injected from a minimum of 1 to 2 test borings. A minimum of three (3) swell tests should be performed for each 10-ft boring drilled to evaluate the 10-ft injection. Maximum benefit of these movement reduction procedures can be achieved by employing ALPHA to observe, monitor and test the entire process.

The PVR (about 3 inches) was calculated assuming the average swell of the injected soils does not exceed about 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 Tex-124-E.

6.2 Slab-On-Grade Foundation

Slab-on-grade foundation should be designed with exterior and interior grade beams adequate to provide sufficient rigidity to the foundation system. A net allowable bearing pressure of 1.5 ksf may be used for design of all grade beams bearing subgrade improved soils placed as recommended in Section 6.1. Grade beams should bear a minimum depth of 18 inches below final grade and should have a minimum width of 10 inches for bearing capacity considerations.

To reduce cracking as normal movements occur in foundation soils, all grade beams and the floor slab should be adequately reinforced with steel (conventional reinforcing steel and/or post-tension reinforcement). 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 slab.

6.2.1 Post-Tensioned, Slab-on-Grade Foundation Design Parameters

Design parameters for a post-tensioned, slab-on-grade foundation system are provided in Table A. These parameters were evaluated based on the conditions encountered in the test 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 A
Post-Tensioned, Slab-on-Grade Foundation Design Parameters
PVR = 3 inches

	EDGE LIFT	CENTER LIFT
Edge Moisture Distance, ft (e_m)	4.3	7.0
Differential Soil Movement, inches (y_m)	1.9	1.4

6.3 Independent, Drilled, Straight-Shaft Piers

If desired, drilled straight-shaft piers could be utilized below the slab-on-grade foundation system. These drilled, straight-shaft piers should bear at least 2 ft into the gray shaly limestone, and have a minimum length of 8 ft. Deeper penetrations will be required to develop skin friction and/or uplift resistance. As discussed in Section 5.0, gray shaly limestone was encountered at a depth of about 10 ft below the existing ground surface, in the test borings.

Piers bearing at least 2 ft into the gray shaly limestone can be dimensioned using a net allowable end-bearing pressure of 50.0 ksf and skin friction (in compression) of 7.0 ksf. The skin friction component should be applied only to the portion of the shaft located at least 2 ft below the surface of the gray shaly limestone and the portion of the shaft below any temporary casing. Further, the minimum clear spacing between piers should be at least two pier shaft diameters to develop the full load carrying capacity from skin friction. Closer spacing will result in a reduction of the skin friction. The skin friction will vary linearly from the full value at a clear spacing of two (2) diameters to 50 percent of the design value with no clear spacing. The bearing capacity contains a factor of safety of at least three (3) considering a general bearing capacity failure and the skin friction value has a factor of safety of at least two (2). Normal elastic settlement of piers under loading is estimated at less than about ½ inch.

Each pier shaft should be reinforced with suitable tension steel over its entire length and should be embedded a sufficient distance below the surface of the gray shaly limestone to adequately resist potential uplift (tensile) forces due to potential soil swell (soil-to-pier adhesion) 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. For piers extending through clay soils, it is estimated this uplift adhesion will not exceed about 2.0 ksf. This soil adhesion is approximated to act uniformly over the upper 12 ft of the pier shaft in contact with clayey soils. Uplift adhesion due to soil heave can be neglected over the portion of the pier shaft in contact with shaly limestone.

The uplift resistance of each pier can be computed using an allowable skin friction value of 5.5 ksf acting uniformly over the portion of the shaft extending into the gray shaly limestone. The top 2 ft of gray shaly limestone should be neglected in computing the uplift resistance of each pier. Also, the uplift resistance should be neglected for any portion of the shaft above the bottom of temporary casing. This uplift resistance value has a factor of safety of at least two (2).

Due to the difference in potential movement between the piers and the slab-on-grade foundation, piers should be independent and not structurally connected to the slab-on-grade foundation. In addition, the structural engineer should appropriately design the slab-on-grade to consider the possibility portions of the slab are supported only by the piers in the event the on-site soils shrink/settle and pull away from the slab. Post Tension Institute (PTI) design parameters provided in Section 6.2 are valid only for slab-on-grade systems fully supported on soil and do not take into consideration any portion of the slab or grade beams being fully or partially suspended on piers due to shrinkage or swelling of on-site soils.

6.4 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 residences 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, sidewalks or other flatwork adjacent to the structure, a positive seal must be maintained between the structure and the flatwork to minimize seepage of water into the underlying supporting soils. Post-construction movement of pavement and other 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 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 their mature height due to their significant moisture demand upon maturing.

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

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

- Architectural design of the foundation slabs 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 Sections 7.3 and 7.4 and in accordance with requirements of local municipal 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 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 structure.

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

Shaly limestone was encountered at a depth of about 10 ft below the existing ground surface at the boring locations, and considering the proposed final grade shaly limestone may be encountered during general excavation for utilities. From our experience, this shaly limestone can be hard and difficult to excavate, and difficulty excavating this material can increase with depth. Rock excavation methods (including, but not limited to rock teeth, rippers, jack hammers, or sawcutting) may be required to remove shaly limestone. The contractor selected should have experience with excavation in this shaly limestone.

All areas supporting foundations, pavement, flatwork, or areas to receive 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.

- The exposed subgrade should be further evaluated by proof-rolling with a heavy pneumatic tired roller, loaded dump truck or similar equipment weighing approximately 10 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 proof-roll should be removed and replaced with well-compacted material as outlined in Section 7.3.

- Any undesirable material (organic material, wet, soft, or loose soil) exposed during the proof-roll 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-key width of five (5) ft. This should provide a good contact between the existing soils and fill materials, reduce potential sliding planes, and allow relatively horizontal lift placements.

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 properly monitored to verify loose, soft, or otherwise undesirable materials are removed and foundations will bear on satisfactory material. Surface runoff should be drained away from excavations and not allowed to pond in the bottom of the excavation. Concrete for foundations should be placed as soon as practical after the excavation is made. Drilled piers should be excavated and concrete placed the same 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, straight shaft drilled piers should be slightly widened and deepened to provide a fresh penetration surface, or a new (deeper) full penetration should be provided. Grade beam excavations for slab foundations should be slightly deepened and cleaned, in order to provide a fresh bearing surface.

All pier shafts should have a minimum diameter of 1 ft 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 log,
- 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 and steel reinforcement.

From our experience, groundwater seepage could be encountered during pier installation, and the risk of encountering seepage is increased during or after periods of precipitation. Temporary casing should be anticipated to control groundwater seepage that could occur in the clayey matrix or fill material, near the soil/rock (shaly limestone) interface or within fractures in the rock. Casing should be seated in the shaly limestone below the depth of seepage, and all water and loosened material should be removed from the cased excavation before starting the design penetration. 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.

Groundwater can also occur within fractures in the bearing stratum for drilled, straight-shaft piers and this may require extending the casing and deepening the piers. From our experience with similar soil and rock conditions, sometimes groundwater cannot be controlled by the use of casing, and underwater placement of pier concrete may be required. Special mix designs are usually required for tremied or pumped concrete. Proper concreting procedures should include placement of concrete from the bottom to the top of the pier using a sealed tremie or pumped concrete. The tremie should be maintained at least 5 ft into the wet concrete during placement. It is recommended a separate bid item be provided for casing and underwater concrete placement on the contractor’s bid schedule. Pier drilling contractors experienced in similar soil and groundwater conditions should be utilized for this project.

7.3 Fill Compaction

The following recommendations pertain to fill soils placed for general site grading including the main residence as follows:

- *Outside* the designated building pad area *if* moisture conditioning will be used as the method for subgrade improvement. Where moisture conditioning is utilized for subgrade improvement, 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 6.1.

- For general grading *including* the building pad areas *if* chemical injection will be used as a method for subgrade improvement.

Clay soils used for general fill with a PI 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 of the material’s optimum moisture.

In cases where fills are more than 10 ft deep, 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. Even if fill is properly compacted as discussed, 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 areas with deep fills and/or wall backfill.

Clay fill should be processed and the largest particle or clod should be less than 6 inches prior to compaction. 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

In cases where utility lines are more than 10 ft deep, 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 thickness) 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 pavements 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 Free Groundwater

Free groundwater was not present in any of the borings. From our experience with similar soils, groundwater seepage could be encountered at shallower depths in excavations for foundations, utility conduits and other general excavations. The risk of encountering 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 to establish final grades, attention should be given to possible seasonal water seepage that could occur through natural cracks and fissures in the newly exposed stratigraphy. *From our experience, seasonal seepage could occur where shaly limestone is near the final site grade.* Subsurface drains may be required in these areas to intercept seasonal groundwater seepage. The need for these or other de-watering devices at the site should be carefully addressed during construction. Our office could be contacted to visually observe the 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 same.

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 in 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 and 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

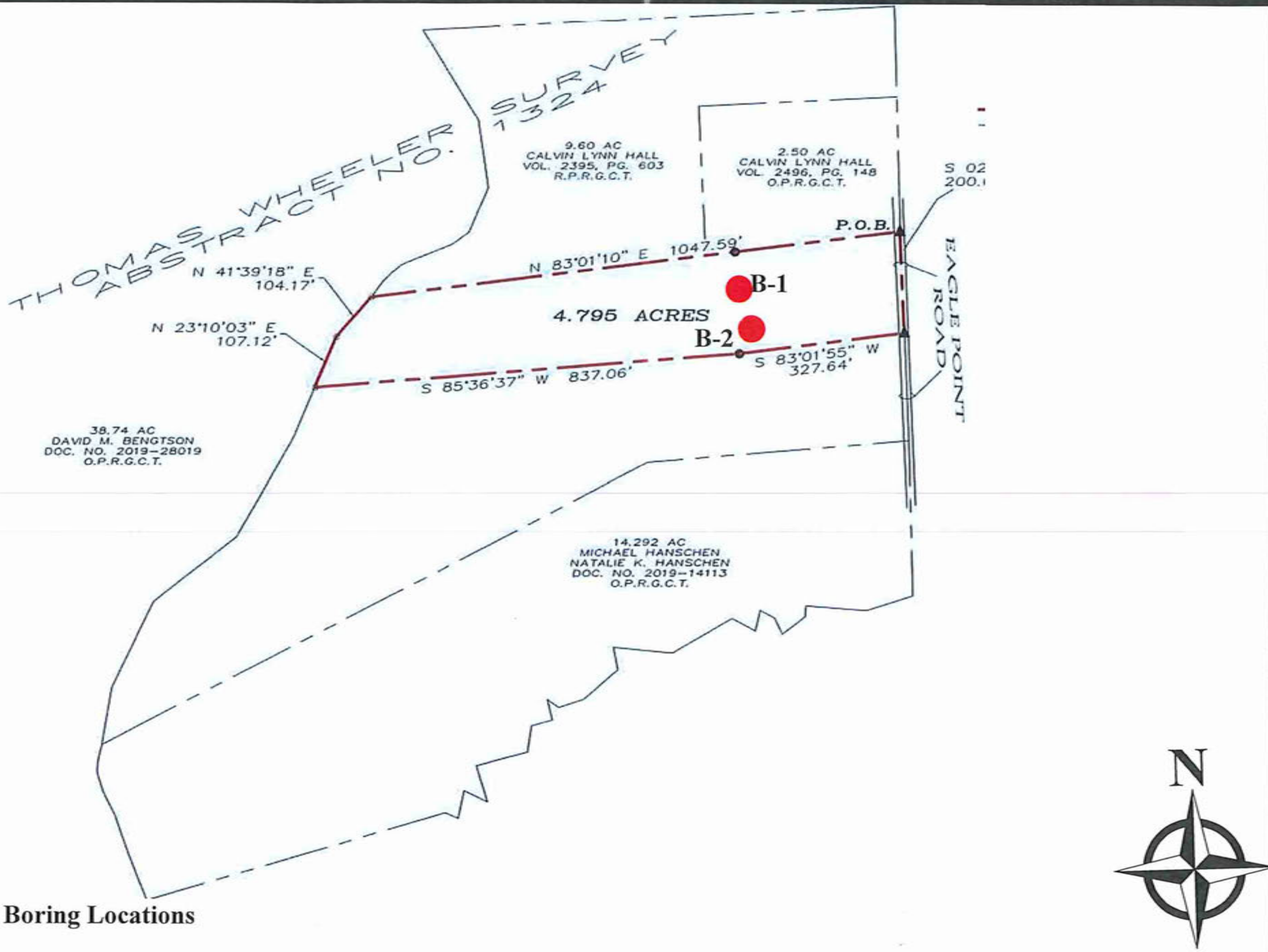
A-1 METHODS OF FIELD EXPLORATION

Using standard rotary drilling equipment, two (2) widely spaced test borings were performed for this geotechnical exploration at the approximate locations shown on the Boring Location Plan – Figure 1. The test borings were located in the field using a handheld GPS device or by 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 the test borings shown on the Boring Location Plan – Figure 1 are considered accurate only to the degree implied by the methods used to define them.

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.

A modified version of the Texas Cone Penetration (TCP) test was used to assess the apparent in-place strength characteristics of rock type materials. A 3-inch diameter steel cone driven by a 170-pound hammer dropped 24 inches (340 ft-pounds of energy) is the basis for TxDOT strength correlations. In this case, ALPHA modified the procedure by using a 140-pound hammer dropped 30-inches (350 ft-pounds of energy) for completion of the field test. 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 were recorded on the field logs and are shown on the Log of Borings as “TX Cone” (reference: TxDOT Test Method TEX 132-E, as modified).

Boring logs are included in the Appendix. The boring 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.



Approximate Boring Locations

Geotechnical Exploration
New Residence
351 Eagle Point Road
Van Alstyne, Texas
ALPHA Report No. G210510

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Boring Location Plan
Figure 1

B-1 METHODS OF LABORATORY TESTING

Representative samples were observed 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) and Atterberg-limit tests (ASTM D 4318) were performed on selected samples. In addition, pocket-penetrometer tests were conducted on selected soil samples to evaluate soil shear strength. Results of the previously described laboratory tests are provided on the accompanying Log of Borings.

In addition to the Atterberg-limit tests, the expansive properties of the clay soils were further analyzed by an absorption swell test (ASTM D 4546). 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 swell and total moisture gain calculated. Results of the absorption swell test are provided on the Swell Test Data – Figure 2, included in this Appendix.

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2209 Wisconsin St.
Dallas, Texas 75229
Phone: 972-620-8911
Fax: 972-620-1302
www.alphatesting.com

BORING NO.: 1
Sheet 1 of 1

PROJECT NO.: G210510

Client: [redacted]
Project: New Residence
Start Date: 3/24/2021 **End Date:** 3/24/2021
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Van Alstyne, 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	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, %
0	Tan CLAY												
					2.0								
					2.0				30				
5					4.5				29	81	28	53	1.3
					4.5				24				
					4.5				28				
10	10.0												
	Gray SHALY LIMESTONE												
15				100/ 0.25"					18				
20				100/ 0.75"					18				
25	25.0			100/ 0.25"									
	TEST BORING TERMINATED AT 25 FT												

MATERIAL DESCRIPTION

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Fax: 972-620-1302
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BORING NO.: 2
Sheet 1 of 1

PROJECT NO.: G210510

Client: [redacted]
Project: New Residence
Start Date: 3/24/2021 **End Date:** 3/24/2021
Drilling Method: CONTINUOUS FLIGHT AUGER

Location: Van Alstyne, 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, %
		Tan CLAY				3.5								
						3.0				26	68	23	45	
5						4.5				29				
						4.5				24	72	24	48	1.6
						4.5				25				
10		10.0												
		Gray SHALY LIMESTONE												
15					100/ 0.75"					15				
20		20.0			100/ 0.25"									
		TEST BORING TERMINATED AT 20 FT												
25														

ALPHA TESTING

WHERE IT ALL BEGINS

KEY TO SOIL SYMBOLS AND CLASSIFICATIONS

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

BORING 1

ABSORPTION SWELL DATA

SAMPLE DEPTH, ft	1 - 2	3 - 4	6 - 7	7 - 8 AT
INITIAL MOISTURE CONTENT, %	32.3	29.6	24.5	28.9
FINAL MOISTURE CONTENT, %	36.1	31.3	24.8	30.1
PERCENT SWELL	1.2	0.6	0.0	0.3

LABORATORY DATA

Sample Depth, ft	Moisture Content, %	Pocket Pen.,tsf
0-1	35.0	2.00
1-2	32.3	2.75
2-3	31.8	3.00
3-4	29.6	3.25
4-5	29.6	3.00
5-6	33.7	2.75
6-7	24.5	3.00
7-8	28.9	2.50
8-9	24.0	3.50
9-10	26.1	3.25

Fred Leal **McKinney, Texas**		**Date Received** **03/18/22**
Client Delivered Soil Sample (Swell & Moisture Content) **Alpha Lab Dallas, Texas**		**220909**