Geotechnical Engineering Report

Texoma Parkway Building

Sherman, Texas

October 28, 2022

D&S ENGINEERING LABS

D&S ENGINEERING LABS

October 28, 2022

Huitt-Zollars, Inc.
5430 LBJ Freeway, Suite 1500
Dallas, Texas 75240
Attn: John Ho. P.E., NSPE, Vice President
Phone: 214-934-5635
Email: jho@huitt-zollars.com

GEOTECHNICAL INVESTIGATION
D&S ENGINEERING #G22-2228
TEXOMA PARKWAY BUILDING
SHERMAN, TEXAS

Mr. Ho,

As requested, D&S Engineering Labs, LLC has completed the Geotechnical Investigation for the above referenced project. This investigation was conducted in accordance with Proposal No. GP22-2228 dated July 22, 2022. Authorization to proceed was received on August 3, 2022.

We appreciate the opportunity to provide professional geotechnical engineering services to you. We are available to discuss any questions which may arise regarding this report. Please do not hesitate to call when we can provide any additional services.

Sincerely,

D&S Engineering Labs, LLC

STATE OF TEXAS
IBRAHIM A. BAAYEH
132175
LICENSED
PROFESSIONAL ENGINEER

10-28-2022

Saurav Sinha

Saurav Sinha, P.E.
Senior Geotechnical Engineer

Ibrahim A. Baayeh, P.E.
Director of Geotechnical Engineering

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION
TEXOMA PARKWAY BUILDING
SHERMAN, TEXAS

1.0 PROJECT DESCRIPTION

This report presents the results of the geotechnical investigation conducted for the proposed administration building that is to be located at 6104 Texoma Parkway in Sherman, Texas. The project consists of design and construction of an approximately 11,000 sq-ft administration building and associated pavements. We understand that the new building addition will be constructed along the east side of the existing building.

The project site was covered with short, maintained grasses and pavements. Based on visual site observations and available USGS topographic maps, the site appears to slope gently towards the west with an elevation change across the proposed development on the order of about 3 feet. Grading plans were not available at the time of this report; therefore, we anticipate finished grades will be at or near existing grades. Recommended design parameters provided herein should be expected to change should there be significant quantities of cut or fill; therefore, we recommend that this office be permitted to review final grading and design plans prior to construction to confirm and/or revise the conclusions and recommendations provided herein

2.0 PURPOSE AND SCOPE

The purpose of this investigation was to:

- Identify the subsurface stratigraphy and groundwater conditions present at the site.
- Evaluate the physical and engineering properties of the subsurface soil and bedrock strata for use in the geotechnical analyses.
- Provide geotechnical recommendations for use in the design and construction of the proposed structures and related site work.

The scope of this investigation consisted of:

- Drilling and sampling of five (5) soil borings were advanced across the site. Three (3) were advanced within the footprint of the proposed structure area to depths of about 25 feet. Two (2) borings were advanced to depths of about 10 feet within the proposed pavement areas.
- Laboratory testing of selected soil and bedrock samples obtained during the field investigation.
- Preparation of a Geotechnical Report that includes:

- Evaluation of potential soil heave through Potential Vertical Movement (PVM) estimates.
- Recommendations for the design and construction of the foundations.
- Recommendations for pavements and pavement subgrade preparation.
- Recommendations for earthwork.

3.0 FIELD AND LABORATORY INVESTIGATION

3.1 General

The borings were advanced using a truck-mounted drilling rig, equipped with continuous hollow-stem flight augers. Undisturbed samples of cohesive soils and weathered bedrock strata were obtained using 3-inch diameter tube samplers that were advanced into the soils in 1-foot increments by the continuous thrust of a hydraulic ram located on the drilling equipment. After sample extrusion, an estimate of the material stiffness of each cohesive soil sample was obtained in the field using a hand penetrometer.

Subsurface materials were also intermittently tested in-situ using cone penetration tests in order to determine their resistance to penetration. For this test, a 3-inch diameter steel cone is driven by the energy of a 170-pound hammer falling freely from a height of 24 inches and striking an anvil located at the top of the drill string. Depending on the resistance of the soil materials, either the number of blows of the hammer required to provide 12 inches of penetration is recorded (as two increments of 6 inches each), or the inches of penetration of the cone resulting from 100 blows of the hammer are recorded (as two increments of 50 blows each).

All samples obtained were extruded in the field, placed in plastic bags to minimize changes in the natural moisture condition, labeled according to the appropriate boring number and depth, and placed in protective cardboard boxes for transportation to the laboratory. The approximate locations of the borings performed at the site are shown on the boring location map that is included in Appendix A. The specific depths, thicknesses and descriptions of the strata encountered are presented on the individual Boring Log illustrations, which are also included in Appendix A. The approximate surface elevations shown on the boring logs were interpolated from the USGS topoview website, which provides spot elevations and contours in 10-foot intervals. Strata boundaries shown on the boring logs are approximate.

3.2 Laboratory Testing

Laboratory tests were performed to identify the relevant engineering characteristics of the subsurface materials encountered and to provide data for developing engineering design parameters. The subsurface materials recovered during the field exploration were initially logged in the field by the drill crew and were later described by a Staff Engineer after the samples arrived in the laboratory. These descriptions were later refined by a Geotechnical Engineer based on results of the laboratory tests performed. All recovered soil samples were classified and described in part using the Unified Soil Classification System (USCS) and other accepted procedures.

In order to determine soil characteristics and to aid in classifying the soils, classification testing was performed on selected samples as requested by the Geotechnical Engineer. Classification testing was performed in general accordance with the following ASTM testing standards:

- Moisture Content ASTM D2216
- Atterberg Limits ASTM D4318
- Percent of Particles Finer than No 200 Sieve ASTM D1140

Additional tests were performed to aid in evaluating strength, volume change, and chemical characteristics, which consisted of the following:

- Overburden Swells ASTM D4546
- Unconfined Compressive Strength of Soil Samples ASTM D2166
- Soluble Sulfates TEX-145-E

The results of these tests are presented at the corresponding sample depths on the appropriate Boring Log illustrations. The classification tests are described in more detail in Appendix B (General Description of Procedures).

3.2.1 Overburden Swell Tests

Selected samples of the near-surface cohesive soils were subjected to overburden swell tests. For this test, a sample is placed in a consolidometer and is subjected to the estimated in-situ overburden pressure. The sample is then inundated with water and allowed to swell. Moisture contents are determined both before and after completion of the test. Test results are recorded as the percent swell, with initial and final moisture content. The results from overburden swell tests performed indicate a high potential for swell with changes in soil moisture content.

3.2.2 Unconfined Compression Tests

Unconfined compressive strength testing was performed on selected samples of the cohesive soils. These tests were performed in general accordance with ASTM D2166. During each test, a cylindrical specimen is subjected to an axial load that is applied at a constant rate of strain until either failure or a large strain (i.e., greater than 15 percent) occurs. Once the test is completed, the unit weight of the sample is determined based on the moisture content.

3.2.3 Soluble Sulfates

Soluble sulfate tests were performed on representative samples obtained. These results are provided in Appendix A (Boring Logs and Supporting Data). Subgrade materials in some areas of Texas has experienced sulfate-induced heave treatment with calcium-based additives such as lime. In general, a sulfate level less than 3,000 ppm is considered to have an acceptably low potential for sulfate induced heaving. The result of the sulfate test performed on representative near-surface soil sample from test boring in this study indicate values of about 100 ppm, and thus should be considered to pose a minimal risk of sulfate-induced heaving after lime treatment. However, additional sulfate testing is recommended to be performed prior to construction for the subgrade to receive lime treatment.

4.0 SITE CONDITIONS

4.1 Stratigraphy

Based upon a review of the recovered samples, as well as the Geologic Atlas of Texas, Sherman Sheet, this site is in an area characterized by soil and bedrock strata associated with the Eagle Ford Formation. The Eagle Ford Formation is typically composed of clay soils overlying shale bedrock.

At the ground surface within Borings B1, B3, P1 and P2 fill materials consisting primarily of sandy lean clays. The fill materials encountered were generally very stiff to hard in consistency, light brown and reddish tan in color, and contained few to some aggregate fragments. The fill materials extended to depths of about 2 to 7 feet below current site grades.

Below the fill materials within Borings B1, B3, P1 and P2, and at the ground surface within Boring B2 native soils consisting of fat and lean clays were encountered. The native clays present were generally stiff to hard in consistency, various shades of brown and reddish tan in color, variably shaley in nature, and contained varying amounts of iron oxide stains, calcareous and ferrous nodules, gypsum and sand. The clay soils extended to at least the maximum depths explored of about 10 to 25 feet below current site grades.

4.2 Groundwater

Groundwater seepage was observed within Borings B1 and B2 during drilling at depths of about 24 feet and measured at depths of about 24 feet upon completion of drilling activities. Groundwater seepage was not encountered within the remaining borings either during or upon the completion of drilling operations. It should be noted groundwater levels may fluctuate with seasonal and annual variations in rainfall and may vary as a result of development and landscape irrigation. Groundwater is often contained within the joints, fractures and other rock mass defects present in bedrock strata. When intercepted, these defects can produce appreciable amounts of water for a period of time, especially if those defects are extensive and well inter-connected.

5.0 ENGINEERING ANALYSIS

5.1 Estimated Potential Vertical Movement (PVM)

Potential Vertical Movement (PVM) was evaluated utilizing a variety of different methods for predicting movement, as described in Appendix B, and based on our experience and professional opinion. An active zone or seasonal moisture variation depth of about 10 feet were considered in our analysis.

The near-surface soils at this site were generally found to be in a dry to wet moisture condition at the time of our field investigation. Based upon the results of our analysis, the site is estimated to possess a PVM of about 3 inches at the soil moisture conditions existing at the time of the filed investigation. If the near surface soils are allowed to dry appreciably to significant depth prior to or during construction, the potential for post-construction vertical movement may increase. Please note that dry, average, and wet are relative terms based on moisture content and plasticity.

6.0 FOUNDATION RECOMMENDATIONS

The near-surface soils present at the site currently have a moderate to high potential for post-construction vertical movement with changes in soil moisture content. We have assumed that the structural loads will be typical for the type and size of the building proposed. Final determination of the foundation type to be utilized for this project should be made by the Structural Engineer based on loading, economic factors, and risk tolerance.

If movements on the order of 1-inch are not considered tolerable, an underreamed shaft (belled pier) foundation system or straight shaft drilled pier may be utilized with either a soil supported floor slab or structurally supported grade beams and suspended floor slab. This pier-and-beam type foundation system provides the least risk of soil-related post-construction vertical movements as the structure is isolated from the underlying expansive soils.

Alternatively, if some soil-related movement is considered acceptable, structural support for the proposed new retail building addition may be provided utilizing a combination of shallow isolated and/or continuous footings used in conjunction with a soil supported floor slab. Based on the type of structure and the estimated degree of potential soil movement, we recommend that the subgrade be prepared as outlined in the Earthwork Recommendations section of this report to provide a uniform building pad and limit soil-related movements to the order of 1-inch or less for all shallow, ground supported foundation options provided in this report.

Please note that shallow foundations and soil-supported floor systems may experience some vertical movement with changes in soil moisture content. Non-load bearing walls, partitions, and other elements bearing on the floor slab will reflect these movements should they occur. However, with appropriate design, adherence to good construction practices and appropriate post-construction maintenance, these potential movements can be reduced. The majority of the movement is expected to occur within 10 feet of the perimeter of the building, and any walls bearing on the slab in the areas of movement may exhibit distress.

6.1 Underream Shafts (Belled Piers) Foundation Recommendations

Structural loads for the new building may be supported on auger-excavated, reinforced concrete underreamed drilled shafts founded in the lean clay soils and/or weathered shale strata at a minimum depth of 14 feet below existing or finished grades, whichever is deeper. Underreamed piers supporting structural loads may be designed using an allowable end bearing pressure of 6,000 pounds per square foot (psf). Such piers should have a shaft diameter of at least 18 inches. The bell diameter should not exceed 3.0 times the shaft diameter, and the minimum clearance between the edges of bells should be equal to twice the diameter of the largest adjacent bell. For uplift considerations, piers should not be spaced closer than two underream diameters (edge to edge) based on the diameter of the larger underream. Closer pier spacings may result in reduced uplift capacity. We should be contacted to review closer pier spacings on a case-by-case basis.

There will be no reduction in uplift resistance contribution from the weight of soil above bell and weight of drilled pier.

The piers should contain sufficient steel reinforcement to resist the uplift pressures that will be exerted by the near surface soils. These pressures are approximated to be on the order of 1,000 psf of shaft area over the upper 10 feet of any shaft in contact with near surface overburden soils. Typically, ½ percent of steel by cross-sectional area is sufficient for this purpose (ACI 318). Uplift forces acting on individual shafts will be resisted by the vertical shaft load plus the weight of a conical wedge of soil above the underream. This weight of soil should be taken as a wedge extending upward from the base of the underream at an angle of 40 degrees from vertical.

Drilled-and-underreamed drilled shaft foundations designed and constructed in accordance with the information provided in this report will have a Factor of Safety of at least 2.5 against shear failure and may experience potential settlements of fractions of an inch. We recommend drilling test piers prior to finalizing the foundation design to assess the feasibility of this foundation type.

6.2 Straight-Sided Drilled Shafts

The new structures may be supported on auger-excavated, straight-sided, reinforced concrete drilled shafts at minimum depths of about 15 feet below existing grades. We recommend that straight-sided drilled piers for structural loads be a minimum of 18 inches in diameter.

Straight-sided drilled shafts may be designed to transfer imposed loads into the bearing strata using a combination of end-bearing and skin friction as outlined in Table 2 below. The drilled piers should have adequate embedment depth to develop the required axial and uplift/lateral resistance.

The allowable side frictions noted in Table 3 may be taken from a minimum depth of 10 feet below existing or finished grades, or from the bottom of any temporary casing used, whichever is deeper, to resist both axial loading and uplift.

Table 2. Recommended Drilled Shaft Design Parameters

Material	Depth Below Existing Grades (ft)	Allowable Side Friction (psf)	Allowable End Bearing (psf)
NATIVE CLAYS	10 - 15	800	-
NATIVE CLAYS	>15	1,000	10,000

The shafts should be provided with sufficient steel reinforcement throughout their length to resist potential uplift pressures that will be exerted. For the near surface soils, we recommend using an uplift pressure of 800 psf over an average depth of about 10 feet. Typically, one-half (½) of a percent of steel by cross-sectional area is sufficient for this purpose (ACI 318). However, the final amount of reinforcement required should be determined based on the information provided herein, and should be the greater of that determination, or ACI 318. Uplift forces acting on individual shafts will be resisted by the dead weight of the structure, plus the concrete-to-bearing stratum adhesion acting on that portion of the shaft that is in intimate contact with the bearing stratum from a depth of 10 feet below existing or finished grades, or from the bottom of any temporary casing used, whichever is deeper.

There is no reduction in allowable capacities for shafts in proximity to each other. However, for a two-shaft system, there is an 18 percent reduction in the available perimeter area for side friction capacity for shafts in contact (tangent). The area reduction can be extrapolated linearly to zero at one shaft diameter clear spacing. Please contact this office if other close proximity geometries need to be considered.

We anticipate that a straight-sided drilled pier foundation system designed and constructed in accordance with the information provided in this report will have a factor of safety in excess of 2.5 against shear failure and may experience settlements of small fractions of an inch.

6.2.1 Drilled Shaft Construction Considerations

Groundwater seepage was observed within Borings B1 and B2 during drilling at depths of about 24 feet and measured at depths of about 24 feet upon completion of drilling activities. Groundwater seepage was not encountered within the remaining borings either during or upon the completion of drilling operations. The amount of water present in rock mass defects may fluctuate over time. In many cases, the rate of seepage would suggest that groundwater may be controlled using conventional pumps. In the event that excessive groundwater seepage is encountered that cannot be controlled with conventional pumps, sumps, or other means, or in the event that excessive sidewall sloughing occurs, temporary casing will be necessary and should be available locally. Concrete should be onsite during drilling operations, to facilitate placement immediately after drilling of each shaft is complete.

The installation of all drilled piers should be observed by experienced geotechnical personnel during construction to verify compliance with design assumptions including: 1) verticality of the shaft excavations, 2) minimum pier diameter and depth, 3) identification of the bearing stratum, 4) correct amount of reinforcement, 5) proper removal of loose material, and 6) that groundwater seepage, if encountered, is properly controlled. D&S would be pleased to provide these services in support of this project.

During construction of the drilled shafts, care should be taken to avoid creating an oversized cap ("mushroom") near the ground surface that is larger than the shaft diameter. These "mushrooms" provide a resistance surface that near-surface soils can heave against. If near-surface soils are prone to sloughing, a condition which can result in "mushrooming", the tops of the shafts should be formed in the sloughing soils using cardboard or other smooth circular forms equal to the diameter of the shaft.

Concrete used for the shafts should have a slump of 8 inches $\pm$ 1 inch. Individual shafts should be excavated in a continuous operation and concrete should be placed as soon as after completion of the drilling as is practical. All pier holes should be filled with concrete within 8 hours after completion of drilling. If under-reamed piers will be used, concrete should be placed within 4 hours after completion of drilling the under-ream. In the event of equipment breakdown, any uncompleted open shaft should be backfilled with soil to be redrilled at a later date. This office should be contacted when shafts have reached the target depth but cannot be completed.

6.2.2 Pier-Supported Grade Beams

For pier-supported grade beams, a minimum void space of 8 inches should be provided beneath all grade beams. If the floor slab and/or other portions of the foundation will be suspended in addition to the grade beams, it is recommended an 8-inch void be maintained below all suspended portions of the foundation. Two methods are typically utilized for constructing a suspended floor slab system. These include constructing a crawl space using pan and joist type construction utilizing either concrete or steel beams and raising the floor slab well above the underlying expansive soils or using cardboard carton forms to create a void.

If a pan and joist system is used, and if utility lines are suspended beneath the slab, the void space clearance should be increased to either a minimum of 2 feet to provide for access to these lines, or to provide a minimum of 12 inches clearance below the lowest suspended utility. Flexible connections or oversized penetration sleeves should be considered in the design of utilities to accommodate potential future movements of soil supported utility lines, especially where these lines approach or enter stationary elements or structures. If a crawl space is employed, provisions should be made for positive drainage of the crawl space floor. Sufficient ventilation should also be provided where construction with metal beams and joists is planned, to limit corrosion of the metal components.

Structural cardboard carton forms (void boxes) may also be used to provide the required voids beneath the grade beams and slabs; however, trapezoidal void boxes should not be used. Care should be taken to assure that the void boxes are not allowed to become wet or are crushed prior to or during concrete placement and finishing operations. Masonite or other protective material should be placed on top of the carton forms per manufacturer recommendations to reduce the risk of crushing the cardboard forms during concrete placement and finishing operations. We strongly recommend that side retainers be placed along the grade beam carton forms to prevent soil from infiltrating the void space after the carton forms deteriorate.

The bottom of all grade beam and slab excavations should be essentially free of any loose or soft material prior to the placement of concrete. All grade beams and floor slabs should be adequately reinforced to minimize cracking as normal movements occur in the foundation soils. Required fill under the void boxes may be any clean soil and should be compacted in accordance with the Earthwork Recommendation section in this report. If needed, a thin (less than 3-inches thick) leveling bed of lean concrete or flowable fill may be used.

If grade beams are formed, the exterior side of the grade beams around the structure should be carefully backfilled with on-site clayey soils. The backfill soils should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and should be placed at a moisture content that is at least two (2) percentage point above the optimum moisture content, as determined by the same test. This fill should extend the full depth of the grade beam plus void space and should extend a minimum distance of two feet away from the exterior grade beam perimeter.

6.3 Shallow Foundations

Alternatively, if potential post-construction movements on the order of 1-inch can be tolerated, a shallow footing foundation system may be utilized for support of the new structure. If a shallow foundation system is selected, we recommend that structural loads for the building be supported on reinforced concrete, monolithic shallow footings founded at a depth of at least 24 inches below final exterior grade in reworked soils. The shallow footings should be a minimum of 18 inches in width for strip footings and may be designed using a net allowable bearing pressure of 2,000 pounds per square foot (psf) when bearing on a properly prepared subgrade as outlined in the Earthwork Recommendations section of this report.

The bottom of all footing excavations should be essentially free of any loose or soft material prior to the placement of concrete. All footings and floor slabs should be adequately reinforced to minimize cracking, as normal movements will occur in the foundation soils.

If footings are formed, the exterior sides of the footings around the structure should be carefully backfilled with clay soils. The backfill soils should be compacted to at least 95 percent of the maximum dry density for the backfill material as determined by ASTM D698 (standard Proctor method). The backfill should also be placed at a moisture content that is at least two (2) percentage points above the optimum moisture content, as determined by that same test. This fill should extend to the full depth of the footing and should extend a minimum distance of two feet away from the exterior footing perimeter.

All footings or footing segments should be constructed in a relatively seamless operation, with excavation activities and placement of the reinforcement steel and concrete occurring within the same day. In the event that concrete cannot be placed in newly excavated footings within 5 days of excavating, the base of the excavated footing should be deepened a few inches and be covered with a thin seal of lean concrete. We recommend a representative of a qualified geotechnical engineer observe all footing excavations prior to placing concrete to verify bearing stratum competence. Any footing excavations left open overnight should be observed by the geotechnical representative prior to placing concrete in order to determine stratum degradation and the amount of additional depth of excavation required. D&S would be pleased provide these services in support of this project.

6.4 Soil-Supported Floor System

A soil-supported floor slab in conjunction with a pier and beam system or shallow foundation system may be utilized for this project if some post-construction movements are considered acceptable. A ground-supported floor system has an increased risk of potential vertical movement resulting from subgrade soil volume changes, which may occur as a result of changes in soil moisture content. The majority of such movement typically occurs in the perimeter 10 feet of buildings with slabs constructed with Finished Floor Elevations near those of the final exterior grades. Any walls bearing on the slab in the areas of movement may exhibit distress. We recommend that the subgrade be prepared according to the Earthwork Recommendations section of this report in order to reduce the potential for post-construction movement to about 1-inch. The floor slab should be doweled to the grade beams at the locations of exterior doors in order to prevent vertical steps from forming at these high-traffic areas.

In order to reduce the effects of seasonal moisture fluctuations and subsequent possible soil movement beneath soil-supported floor slabs as described above, consideration is often given to the installation of a horizontal barrier around the perimeter of the structure. This barrier may be in the form of an independent barrier, such as a minimum 5-foot-wide sidewalk. The joints between the building and any sidewalks and pavements should be sealed and the seals inspected periodically and re-sealed as necessary through the life of the structure.

We understand that sidewalks are not always practical or desired around the full perimeters of some facilities. Where landscaping will be present adjacent to building perimeters, diligent post-construction maintenance should be employed to prevent excessive wetting or drying of those adjacent soils, while maintaining a consistent soil moisture condition around the foundations as much as possible throughout the year.

7.0 EARTHWORK RECOMMENDATIONS

The near-surface soils present have a moderate potential for post-construction vertical movement with changes in subsurface soil moisture changes. We have the following earthwork recommendations to provide a uniform building pad and limit the potential for post-construction movements to the order of 1-inch for all ground-supported foundation options provided in this report. Recommendations for preparation of a structurally suspended floor slab building pad are also provided in the following sections. Please note that more stringent tolerances limiting potential post-construction vertical movement will require more extensive effort.

7.1 Soil Preparation for Shallow Foundations and Soil-Supported Floor Slabs

- Strip the site of all vegetation and remove any remaining organic or deleterious material including tree roots, root balls and matted roots. Typically, 4 to 6 inches are sufficient for this purpose.

- After stripping and performing any necessary grade cuts, excavate to a common elevation at least 5 feet below existing or finished pad elevation, whichever is deeper. The excavation should extend at least 5 feet beyond the perimeter of the new structure. The excavated soil may be stockpiled for future reuse.

- Prior to placement of any grade-raise fill, scarify, rework, and recompact the exposed stripped subgrade to a depth of 12 inches. The scarified and reworked soils should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and placed at a moisture content that is at least two (2) percentage points above the optimum moisture content (+2%), as determined by the same test.

- Within 24 hours of recompacting the reworked excavated exposed subgrade, begin fill operations with debris-free imported soil to no higher than 12 inches below the bottom of finished subgrade elevation. The fill soil should be spread across the building pad in a uniform thickness atop the scarified and recompacted layer. The fill soil should be placed in maximum 8-inch compacted lifts, be compacted to at least 95 percent of the maximum dry density as determined by ASTM D698 (standard Proctor) and be placed at a moisture content that is at least two (2) percentage points above the optimum moisture content, as determined by that same test (≥+2%). Grade raise fill within the building pad areas may be onsite soils or imported materials having a Liquid Limit (LL) of 50 or less, a Plasticity Index (PI) of 30 or less, a minimum of 30% of the material passing a No. 200 mesh sieve and be essentially free of particles in excess of 4 inches in their longest direction.

- Provide a minimum of 12 inches of select fill on top of the re-worked fill. Select fill should have a liquid limit less than 35, a plasticity index between 6 and 18, and a minimum of 30 percent passing the No. 200 sieve. The select fill should be placed in maximum 6-inch compacted lifts and be compacted to at least 95 percent of the maximum density as determined by ASTM D698, and to a moisture content of optimum or greater as determined by that same test.

- The moisture content of the subgrade should be maintained up to the time of concrete placement. Depending on the speed of the earthwork layers, on hot or windy days, sprinkling with water atop the subgrade may be required, to maintain the compaction moisture content.

- Water should not be allowed to pond on the prepared subgrade either during fill placement, or after reaching final subgrade elevation. To that end, the subgrade surfaces should be shaped to shed water to the edges of the respective pads.

- Place a minimum 15-mil thick vapor barrier beneath all floor slabs (Stego or equivalent). All seams and penetrations through the barrier should be sealed in accordance with the manufacturer’s requirements.

- Each lift of fill placed should be tested for moisture content and degree of compaction by a testing laboratory at the rate of one (1) test per 3,000 square feet, with a minimum of two tests performed per lift within the building pad, one (1) test per lift per 100 linear feet for flatwork areas, and one (1) test per lift per 100 linear feet of utility trench backfill. D&S would be pleased to provide these services in support of this project.

7.2 Soil Preparation for Structurally Suspended Floor Slabs

- Strip the site of all vegetation and remove any remaining organic or deleterious material including tree roots, root balls and matted roots within the new building areas. Typically, 4 to 6 inches are sufficient for this purpose.

- After stripping and performing any required cuts, scarify, rework, and recompact the exposed bottom of the excavated subgrade to a depth of 12 inches. The scarified and reworked soils should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and to a moisture content that is at least two (2) percentage points above the optimum moisture content (≥+2%), as determined by the same test. This procedure should extend at least 5 feet beyond the perimeter of the new structure or 2 feet beyond the sidewalk, whichever is greater.

- After scarifying and recompacting, begin required fill operations using debris-free on-site or imported soil to no higher than the bottom of the void boxes. The grade-raise soil fill should be placed in maximum 12-inch compacted lifts and should be compacted in similar fashion to the scarified soils noted above. Grade raise fill within the building pad areas may be on site material or imported material having a maximum Liquid Limit (LL) of 50, a maximum Plasticity Index (PI) of 30, a minimum of 30% of material passing a No. 200 mesh sieve and be essentially free of particles in excess of 4 inches in their longest direction.

- Place a minimum 15-mil thick vapor barrier beneath all floor slabs (Stego or equivalent). The barrier should be securely bonded to the underside of the floor slab during concrete placement and curing and maintain that bond throughout the design life of the structure. All seams and penetrations through the barrier should be sealed in accordance with the manufacturer’s requirements

7.3 Additional Considerations

In order to minimize the potential for post-construction vertical movement, consideration should be given to the following:

- Trees or shrubbery with a mature height greater than 6 feet and/or that require excessive amounts of water should not be planted near structures or flatwork.

- Trees should not be planted closer than the mature tree's height from structures or flatwork.
- Water should not be allowed to pond next to the foundations. Rainfall roof runoff should be collected and conveyed to downspouts. Downspouts should be directed to discharge at least 5 feet away from the foundations.
- The moisture content of subgrade soils that are in proximity to the structures should be maintained as close as possible to a consistent level throughout the year. However, we strongly recommend that excessive watering near foundations be avoided.
- Exposed ground should be sloped at a minimum 5 percent away from the building for at least 10 feet beyond the perimeter of the building.

8.0 PAVEMENT RECOMMENDATIONS

8.1 General

The pavement design recommendations provided herein are derived from the subgrade information that was obtained from our geotechnical investigation, design assumptions based on project information, our experience with similar projects in this area, and on the guidelines and recommendations of the American Concrete Pavement Association (ACPA). It is ultimately the responsibility of the Civil Engineer of Record and/or other design professionals who are responsible for pavement design to provide the final pavement design and associated specifications for this project.

8.2 Behavior Characteristics of Expansive Soils Beneath Pavement

Near-surface soils at this site are considered to generally have a low to moderate potential for appreciable volume change with changes in soil moisture content. The moisture content can be stabilized to some degree in these soils by covering them with an impermeable surface, such as pavement. However, if moisture is introduced as a result of surface water percolation or poor drainage, the soils can heave and/or soften, causing distress to pavements in contact with the soil in the form of cracks.

The edges of pavement are particularly prone to moisture variations, and so these areas therefore often experience the most distress. When cracks appear on the surface of the pavement, these openings can allow moisture to enter the pavement subgrade, which can lead to further weakening of the pavement section as well as accelerated failure of the pavement surface.

In order to minimize the potential impacts of expansive soil on paved areas and to improve the long-term performance of the pavement, we have the following recommendations:

- Design a crowned pavement with edge slopes. A minimum slope of five percent within the first 5 feet from the edge of the pavement is considered ideal.

- Subgrade treatments intended to reduce the soil's potential for vertical movement or to increase the subgrade stability should extend to at least two (2) feet beyond the back of curbs or edges of pavements.

- Avoid long areas of low-sloping roadway and adjust slopes to provide maximum drainage away from pavement edges.

8.3 Subgrade Strength Characteristics

We recommend that a California Bearing Ratio (CBR) value of 3 be used in the design with a corresponding resilient modulus of 4,100 psi. For compacted lime treated soils or compacted aggregate base course, we also recommend a resilient modulus of 20,000 psi.

8.4 Pavement Subgrade Preparation Recommendations

The anticipated subgrade soils in the pavement areas will be generally fat clay soils which have poor subgrade characteristics and can become soft and pump with an increase in moisture content. A commonly used method to reduce the potential for pumping, improve the strength properties of the subgrade soils, provide a working platform, and provide a uniform subgrade in this area is to treat the soils with lime or to install an aggregate base layer.

- Remove all surface vegetation, including tree root balls and root mats, pavements, and similar unsuitable materials from within the limits of the project. Typically, 6 to 12 inches is sufficient for this purpose.

- Perform any cut operations as needed. We anticipate that excavation of overburden soils can be accomplished with conventional earthwork equipment and methods.

- After stripping and performing necessary cuts, the exposed subgrade should be proof rolled. Proof rolling should consist of rolling the entire pavement subgrade in mutually perpendicular directions with a heavily loaded, tandem-axle dump truck weighing at least 25 tons or other approved equipment capable of applying similar loading conditions. Any soft, wet, or weak soils that are observed to rut or pump excessively during proof rolling should be removed and replaced with well-compacted, on-site clayey material as outlined below. The proof rolling operation should be performed under the observation of a qualified geotechnical engineer. D&S would welcome the opportunity to perform these services for this project.

- After proof rolling, all exposed surfaces in areas to receive fill should be scarified and reworked to a depth of 12 inches. The soils should then be recompacted to a minimum of 95 percent of the maximum dry density obtained in accordance with ASTM D698 (standard Proctor), and to a moisture content that is at least one (1) percentage points (≥+1%) above the material's optimum moisture content, as determined by the same test.

- Fill may be derived from on-site or may be imported as long as the materials are essentially free of organic materials and particles in excess of 4 inches their maximum direction. The fill should be placed in maximum 8-inch compacted lifts, compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and be placed at a moisture content that is at least one percentage point (≥+1%) above the optimum moisture content, as determined by the same test. Prior to compaction, each lift of fill should first be processed throughout its thickness to break up and reduce clod sizes and blended to achieve a material of uniform density and moisture content. Once blended, compaction should be performed with a heavy tamping foot roller. Once compacted, if the surface of the embankment is too smooth, it may not bond properly with the succeeding layer. If this occurs, the surface of the compacted lift should be roughened and loosened by dicing before the succeeding layer is placed

- Water required to bring the fill material to the proper moisture content should be applied evenly through each layer. Any layers that become significantly altered by weather conditions should be reprocessed in order to meet recommended requirements. On hot or windy days, the use of water spraying methods may be required in order to keep each lift moist prior to placement of the subsequent lift. Furthermore, the subsurface soils should be kept moist prior to placing the pavement by water sprinkling or spraying methods.

- Fill materials should be placed on a properly prepared subgrade as outlined above. The combined excavation, placement, and spreading operation should be performed in such a manner as to obtain blending of the material, and to assure that, once compacted, the materials, will have the most practicable degree of compaction and stability. Materials obtained from on-site should be mixed and not segregated.

- Soil imported from off-site sources should be tested for compliance with the recommendations herein and approved by the project geotechnical engineer prior to being used as fill. Imported materials should consist of lean clays (maximum Plasticity Index of 30) that are essentially free of organic materials and particles larger than 4 inches in their maximum dimension.

- Place aggregate base in accordance with the recommendations outlined in subsequent sections.

- Water should not be allowed to pond within or adjacent to the pavement corridor once grading and compaction/testing have begun, particularly once aggregate base is placed. To that end, the pavement subgrade surface should be shaped in a way that will allow water to shed from one or more edges of the prepared subgrade.

- Field density and moisture content testing should be performed at the rate of one test per lift per 100 linear feet of roadway for drive aisles, and at the rate of one test per lift per 5,000 square feet in pavement parking areas.

8.4.1 Lime Treatment

- After completion of proof rolling and any grade raise fills, mix lime slurry into the prepared subgrade soil after scarifying to the lime treatment depth to achieve a treated subgrade with an estimated 6 percent lime by dry weight measure. However, the final amount of lime used should be determined once subgrade preparation is nearly complete. The amount of lime used should be sufficient to reduce the Plasticity Index of the soil to 15 or below (Atterberg Lime series) or to increase pH of the soil-lime mixture to 12.4 (pH series). To account for potential construction and site variability, an additional 1 to 2 percent lime should be added to these test quantities. The hydrated lime should be applied only in an area where the initial mixing operations can be completed the same working day. The area of lime treated subgrade should extend a minimum of 2 feet beyond the back of curbs or edges.

- Hydrated lime should be applied such that mixing operations can be completed during the same working day. The hydrated lime should be placed by the slurry method, meaning that the hydrated lime should be mixed with water in trucks or in tanks and applied as a thin water suspension or slurry. The distributor truck or tank should be equipped with an agitator, which will maintain the lime and water in a uniform mixture. The material and hydrated lime should be thoroughly mixed by a rotary mixer or other device to obtain a homogeneous, friable mixture of material and lime that is free from clods and left to cure from one to seven days.

- Within our experience, we have found that a curing period of 48 to 72 hours is adequate for these sulfate bearing soils at this site. During the curing period, the material should be kept moist at least 3% above optimum. After the specified “mellowing duration”, the soil-lime mixture should be remixed and tested for sufficient pulverization and mixing in accordance with TxDOT Item 260. After the required curing time, the material should be uniformly mixed using a rotary mixer capable of reducing the size of the particles so that, when all non-slaking aggregates retained on a no. 4 sieve are removed, the remainder of the material shall meet the following requirements when tested dry by laboratory sieves:

- Minimum passing 1-3/4 inch sieve = 100%
- Minimum passing no. 4 sieve = 60%

- After achieving the required gradation, the treated soil-lime mixture should then be immediately compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and placed at a moisture content that is above the optimum moisture content, as determined by the same test. During compaction, it may be necessary to allow the lime treated soils to dry to achieve density but at a higher moisture content as possible.

- During the interval of time between application and mixing, the hydrated lime should not be exposed to the open air for a period exceeding six hours.

- To reduce the potential for subgrade soil moisture changes at the edges of pavements, the lime treated subgrade should extend a minimum of 2 feet past the back of the roadway curbs.

- Field density and moisture content testing should be performed at the rate of one (1) test per lift per 5,000 square feet of lime treated soils in pavement areas.

8.4.2 Aggregate Base

Aggregate base may be placed over the prepared subgrade in accordance with the following recommendations prior to placing the pavement.

- Aggregate base should be TxDOT Type A or D and meeting the gradation, durability, and plasticity requirements of TxDOT Item 247 Grade 1-2 or better (2014). Aggregate base material should be uniformly compacted to a minimum of 95% of the maximum standard Proctor dry density (ASTM D698) and placed at a moisture content that is sufficient to achieve density.

- After proof rolling, and prior to the placement of aggregate base, the exposed subgrade beneath pavement areas should be scarified and reworked to a depth of 12 inches, moisture added or removed as required, and the subgrade soils recompacted to a minimum of 95 percent of the maximum dry density of the materials obtained in accordance with ASTM D698 (standard Proctor test) and that is at or above the material's optimum moisture content, as determined by the same test. The rework and aggregate base should extend to at least 24-inches beyond the outside edges of curbs.

- Field density and moisture content testing should be performed at the rate of one test per lift per 100 linear feet of roadway for drive aisles, and at the rate of one test per 5,000 square feet in pavement parking areas.

8.5 Rigid Pavement

We recommend that Portland Cement Concrete Pavement for this site have a minimum thickness of 6 inches for parking areas of automobiles and all light duty trucks over 6-inches of lime treated soil or aggregate base. Concrete thickness should be increased to 7 inches for fire lanes, dumpster areas, and drive lanes receiving up to 25 loaded heavy (18-wheel) trucks per day over 8 inches of lime treatment or aggregate base. Actual traffic loading, frequency, and intensity may require an increase in these minimum recommendations, as may local governing design criteria. We have the following concrete mix design recommendations:

The more stringent of the recommendations in this report or the design criteria for the governing agency should be followed.

- Recommended minimum 28-day design compressive strength: 3,500 psi with nominal aggregate size no greater than 1 inch.
- 15 to 20 percent fly ash may be used with the approval of the Civil Engineer of record.
- Curing compound should be applied within one hour of finishing operations.

8.6 Pavement Joints and Cutting

The performance of concrete pavement depends to a large degree on the design, construction, and long-term maintenance of concrete joints. The following recommendations and observations are offered for consideration by the Civil Engineer and/or pavement Designer-of-Record.

The concrete pavements should have adequately spaced contraction joints to control shrinkage cracking. Past experience indicates that reinforced concrete pavements with sealed contraction joints on a 12 to 15-foot spacing, cut to a depth of one-quarter to one-third of the pavement thickness, have generally exhibited less uncontrolled post-construction cracking than pavements with wider spacing. The contraction joint pattern should divide the pavement into panels that are approximately square where the panel length should not exceed 25 percent more than the panel width. Saw cut, post placement formed contraction joints should be saw cut as soon as the concrete can support the saw cutting equipment and personnel and before shrinkage cracks appear, on the order of 4 to 6 hours after concrete placement.

Isolation joints should be used wherever the pavement will abut a structural element subject to a different magnitude of movement, e.g., light poles, retaining walls, existing pavement, stairways, entryway piers, building walls, or manholes.

In order to minimize the potential differential movement across the pavement areas, all joints including contraction, isolation and construction joints should be sealed to minimize the potential for infiltration of surface water. Rubberized asphalt, silicone or other suitable flexible sealant may be used to seal the joints. Maintenance should include periodic inspection of these joints and resealed as necessary.

8.7 Pavement Reinforcing Steel

We recommend that a minimum of 0.1 percent of steel be used for all concrete pavements. For a 6 to 7-inch thick concrete pavement section, this reinforcement ratio is approximately equivalent to No. 3 bars spaced at 18-inches center-to-center both ways. Reinforcement requirements may increase depending on specific traffic loading and design life parameters.

9.0 OTHER CONSTRUCTION

9.1 Utility and Service Lines

Backfill for utility lines should consist of on-site material and should be placed in accordance with the following recommendations. The on-site fill soil should be placed in maximum 6-inch compacted lifts, compacted to a minimum of 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and placed at a moisture content that is at least the optimum moisture content, as determined by that same test. It is not uncommon to realize some settlement along the trench backfill. Therefore, the final lift of backfill may be mounded somewhat to accommodate such settlements, where possible. We also recommend that the utility trenches be visually inspected during the excavation process to ensure that undesirable fill that was not detected by the test borings does not exist at the site. This office should be notified immediately if any such fill is detected.

Utility lines connected to the structure may experience differential movement in response to changing moisture conditions in expansive soil. These movements may result in damage to the lines, especially at connections. Flexible connections may be considered to account for potential differential movement between the building and utilities.

Utility excavations should be sloped so that water within excavations will flow to a low point away from the active construction where it can be removed from before backfilling. Compaction of bedding material should not be water jetted. Compacted backfill above the utilities should be on-site clays to limit the percolation of surface water.

9.2 Exterior Flatwork

Concrete flatwork should include high tensile steel reinforcement to reduce the formation and size of cracks. Flatwork should also include frequent and regularly spaced expansion/control joints and dowels to limit vertical offsets between neighboring flatwork slabs. Structure entrances should either be part of the structure or designed to tolerate vertical movement without inhibiting access. The moisture content of the subgrade should be maintained up to the time of concrete placement. If subgrade soils are allowed to dry below the levels recommended herein, additional moisture conditioning of the soils may be required. These recommendations are intended to reduce possible distress to exterior flatwork but will not prevent movement and/or vertical offsets between slabs.

9.3 Surface Drainage

Proper drainage is critical to the performance and condition of building foundations and flatwork. Positive surface drainage should be provided that directs surface water away from these elements. Where possible, we recommend that exterior grades slope away from foundations at the rate of five (5) percent in the first five (5) feet, and preferably ten (10) feet away. The slopes should direct water away from the structure and these grades should be maintained throughout construction and the life of the structure.

The location of gutter downspouts should be designed such that these items will not create moisture concentrations at or beneath the structure or flatwork. Downspouts should discharge well away from the structure and should not be allowed to erode surface soil.

Moisture related issues can be positively addressed by constructing continuous exterior flatwork that extends to the building line. Where this occurs, the joints created at the interface of the flatwork and building line should be sealed with a flexible joint sealer to prevent the infiltration of water. Open cracks that may develop in the flatwork should also be sealed. The joint and any cracks that develop should be resealed as they become apparent and should be part of a periodic inspection and maintenance program.

9.4 Landscaping

Landscaping against and around the exterior of the structure can adversely affect subgrade moisture resulting in localized differential movements if not properly maintained. Landscaping elements (such as edging) should not prohibit or slow the drainage of water that could result in water ponding next to foundations or edges of flatwork. When feasible, irrigation lines and heads should not be placed in close proximity to the foundation to prevent the collection of water near the foundation or flatwork, particularly in the event of leaking lines or sprinkler heads.

Trees (if planned) should not be placed in proximity to the structure or movement sensitive flatwork, as trees are known to cause localized soil shrinkage due to desiccation of the soil by the root system, possibly leading to differential movements of the structure in excess of those anticipated herein. The desiccation zone varies by tree, but trees should not be planted closer to structures than the mature tree height, and in no case, should the dripline of the mature tree extend closer than 10-feet of rooflines. To the extent practical, it is recommended that trees scheduled for removal (where required) in the vicinity of the proposed structure and pavements be removed as far in advance of slab construction as possible, ideally by several months or longer. This will tend to restore a more favorable soil moisture equilibrium which will, in turn, tend to minimize the potential for greater than anticipated post-construction ground movements. A moist but not overly wet soil condition should be maintained at all times in all landscaped areas near the building after construction to minimize soil volume changes caused by changing soil moisture conditions.

9.5 Site Grading

Clay cut and fill slopes should be gentle and preferably should not exceed 4 horizontals to 1 vertical (4H:1V).

Excess water ponding on and beside roadways, sidewalks, and ground-supported slabs can cause unacceptable heave of these structures. To reduce this potential heave, good surface drainage should be established. In addition, final grades in the vicinity of structures, pavements, and flatwork should provide for positive drainage away from these elements.

9.6 Excavations

Excavations greater than 5 feet in height/depth should be in accordance with OSHA 29CFR 1926, Subpart P. Temporary construction slopes should incorporate excavation protection systems or should be sloped back. Where the excavation does not extend close to building lines, these areas may be laid back. Borings from this investigation indicated that the soils may be classified as Type B per OSHA regulations up to 10 feet from existing grades. Where space allows, temporary slopes should be sloped at 1 horizontal to 1 vertical (1H:1V) or flatter.

Where excavation slopes greater than five (5) feet in height cannot be laid back, these areas will require installation of a temporary retention system or shoring to protect the existing construction, restrain the subsurface soils and maintain the integrity of the excavation. We recommend that monitoring points be established around the retention system and that these locations be monitored during and after the excavation activities to confirm the integrity of the retention system.

The slopes and temporary retention system should be verified by and designed by the contractor's engineer and should not be surcharged by traffic, construction equipment, or permanent structures. The slopes and temporary retention system should be adequately maintained and periodically inspected to ensure the safety of the excavation and surrounding property.

10.0 SEISMIC CONSIDERATION

The seismic site classification is based on the 2018 International Building Code (IBC) and is a classification of the site based on the type of soil encountered at the site and their engineering properties. Based on the general geologic information gathered in accordance with Table 20.3-1 of ASCE 7-10, we recommend that Soil Site Class “D” be used at this site.

11.0 LIMITATIONS

The professional geotechnical engineering services performed for this project, the findings obtained, and the recommendations prepared were accomplished in accordance with currently accepted geotechnical engineering principles and practices.

Variations in the subsurface conditions are noted at the specific boring locations for this study. As such, all users of this report should be aware that differences in depths and thicknesses of strata encountered can vary between the boring locations. Statements in the report as to subsurface conditions across the site are extrapolated from the data obtained at the specific boring locations. The number and spacing of the exploration borings were chosen to obtain geotechnical information for the design and construction of multi-family residential structures. If there are any conditions differing significantly from those described herein, D&S should be notified to re-evaluate the recommendations contained in this report.

Recommendations contained herein are not considered applicable for an indefinite period of time. Our office must be contacted to reevaluate the contents of this report if construction does not begin within a one-year period after completion of this report.

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.

All contractors referring to this geotechnical report should draw their own conclusions regarding excavations, construction, etc. for bidding purposes. D&S is not responsible for conclusions, opinions or recommendations made by others based on these data. The report is intended to guide preparation of project specifications and should not be used as a substitute for the project specifications.

Recommendations provided in this report are based on our understanding of information provided by the Client to us regarding the scope of work for this project. If the Client notes any differences, our office should be contacted immediately since this may materially alter the recommendations.

This report has been prepared for the exclusive use of our client for specific applications to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless D&S reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A - BORING LOGS AND SUPPORTING DATA

BORING LOCATIONS ARE INTENDED FOR GRAPHICAL REFERENCE ONLY

D&S ENGINEERING LABS

N.T.S.

PLAN OF BORINGS

TEXOMA PARKWAY BUILDING

SHERMAN

TEXAS

SHEET NO.

G1

DATE DRILLED

October 13, 2022

BORING LOCATIONS ARE INTENDED FOR GRAPHICAL REFERENCE ONLY

D&S ENGINEERING LABS

N.T.S.

PLAN OF BORINGS

TEXOMA PARKWAY BUILDING

SHERMAN

TEXAS

SHEET NO.

G2

DATE DRILLED

October 13, 2022

D&S ENGINEERING LABS

KEY TO SYMBOLS AND TERMS

LITHOLOGIC SYMBOLS

ARTIFICIAL	Asphalt
	Aggregate Base
	Concrete
	Fill
SOIL	CH: High Plasticity Clay
	CL: Low Plasticity Clay
	GP: Poorly-graded Gravel
	GW: Well-graded Gravel
	SC: Clayey Sand
	SP: Poorly-graded Sand
	SW: Well-graded Sand
ROCK	Limestone
	Mudstone
	Shale
	Sandstone
	Weathered Limestone
	Weathered Shale
	Weathered Sandstone

CONSISTENCY OF SOILS

CONSISTENCY: FINE GRAINED SOILS		
Consistency	SPT (blowcounts)	PP (tsf)
Very Soft	0 - 2	< 0.25
Soft	3 - 4	0.25 - 0.5
Medium Stiff	5 - 8	0.5 - 1.0
Stiff	9 - 15	1.0 - 2.0
Very Stiff	16 - 30	2.0 - 4.0
Hard	> 30	> 4.0

CONDITION OF SOILS

CONDITION: COARSE GRAINED SOILS			
Condition	SPT (blowcounts)	TCP (values)	Relative Density (%)
Very Loose	0 - 4	< 8	0 - 15
Loose	5 - 10	8 - 20	15 - 35
Medium Dense	11 - 30	20 - 80	35 - 65
Dense	31 - 50	80 - 5 in./100	65 - 85
Very Dense	> 50	0 in. - 5 in./100	85 - 100

SECONDARY COMPONENTS

QUANTITY DESCRIPTORS	
Trace	< 5% of sample
Few	5% to 10%
Little	10% to 25%
Some	25% to 35%
With	> 35%

RELATIVE HARDNESS OF ROCK MASS

Designation	Description
Very Soft	Can be carved with a knife. Can be excavated readily with point of pick. Pieces 1" or more in thickness can be broken by finger pressure. Readily scratched with fingernail.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows with the pick point. Small, thin pieces can be broken by finger pressure.
Medium Hard	Can be grooved or gouged 1/4" deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1" maximum size by hard blows with the point of a pick.
Moderately Hard	Can be scratched with knife or pick. Gouges or grooves 1/4" deep can be excavated by hard blow of the point of a pick. Hand specimens can be detached by a moderate blow.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach a hand specimen.
Very Hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows from a hammer or pick.

WEATHERING OF ROCK MASS

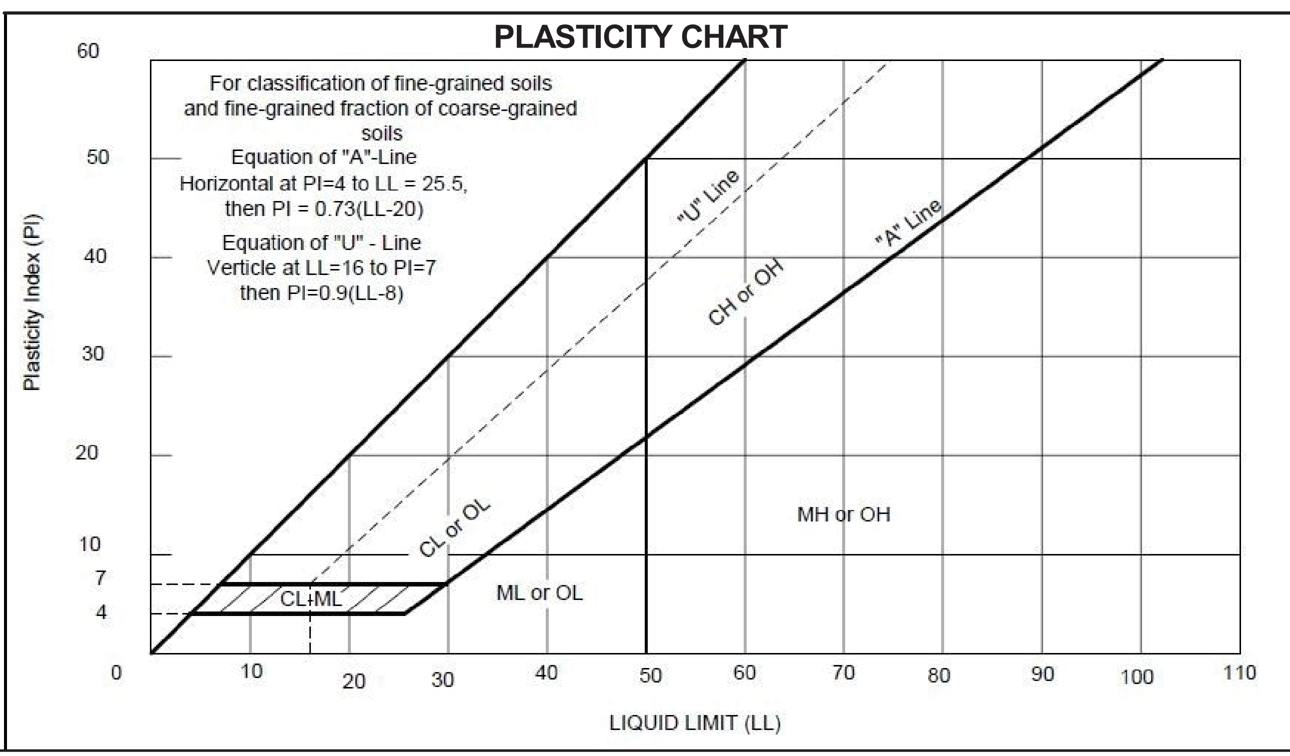
Designation	Description
Fresh	No visible sign of weathering
Slightly weathered	Penetrative weathering on open discontinuity surfaces, but only slight weathering of rock material
Moderately weathered	Weathering extends throughout rock mass, but the rock material is not friable
Highly weathered	Weathering extends throughout rock mass, and the rock material is partly friable
Completely weathered	Rock is wholly decomposed and in a friable condition but the rock texture and structure are preserved
Residual Soil	A soil material with the original texture, structure, and mineralogy of the rock completely destroyed

UNIFIED SOIL CLASSIFICATION SYSTEM

ADAPTED FROM ASTM D 2487

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS				GROUP SYMBOL	GROUP NAME
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS RETAINED ON THE NO. 200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LESS THAN 5% FINES)	$Cu \geq 4 \text{ and } 1 \leq Cc \leq 3$	**GW**	WELL-GRADED GRAVEL
			$Cu < 4 \text{ and/or } [Cc < 1 \text{ or } Cc > 3]$	**GP**	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES (MORE THAN 12% FINES)	Fines classify as ML or MH	**GM**	SILTY GRAVEL
			Fines classify as CL or CH	**GC**	CLAYEY GRAVEL
	SANDS MORE THAN 50% OF COARSE FRACTION PASSING THE NO. 4 SIEVE	CLEAN SANDS (LESS THAN 5% FINES)	$Cu \geq 6 \text{ and } 1 \leq Cc \leq 3$	**SW**	WELL-GRADED SAND
			$Cu < 6 \text{ and/or } [Cc < 1 \text{ or } Cc > 3]$	**SP**	POORLY-GRADED SAND
		SANDS WITH FINES (MORE THAN 12% FINES)	Fines classify as ML or MH	**SM**	SILTY SAND
			Fines classify as CL or CH	**SC**	CLAYEY SAND
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL PASSES THROUGH THE NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50	INORGANIC	PI > 7 and plots on or above "A" line	**CL**	LEAN CLAY
			PI < 4 or plots below "A" line	**ML**	SILT
		ORGANIC	$\frac{Liquid\ limit - oven\ dried}{Liquid\ limit - not\ dried} < 0.75$	**OL**	ORGANIC CLAY ORGANIC SILT
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50	INORGANIC	PI plots on or above "A" line	**CH**	FAT CLAY
			PI plots below "A" line	**MH**	ELASTIC SILT
		ORGANIC	$\frac{Liquid\ limit - oven\ dried}{Liquid\ limit - not\ dried} < 0.75$	**OH**	ORGANIC CLAY ORGANIC SILT
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR			**PT**	PEAT



BORING LOG

D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building
CLIENT: Huitt-Zollars, Inc.
PROJECT NUMBER: G22-2228
START DATE: 10/13/2022 FINISH DATE: 10/13/2022
LOGGED BY: Dalton Hubbard (D&S)
LOCATION: Sherman, TX
GPS COORDINATES: N33.698160, W96.574994
GROUND ELEVATION: Approx. 725 feet
DRILL METHOD: Hollow Stem Flight Auger
DRILLED BY: Markevian Smith (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- ▽ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Material Description	REC (%) RQD (%)	MC (%)	LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+	**FILL: SANDY LEAN CLAY (CL);** very stiff to hard; light brown, reddish tan; few to some aggregate fragments		2.9									
	S	4.5+												
	S	4.5+			15.8									
	S	4.5+												
5	S	4.5+	4.5 ft		16.7									
	T	10, 10	**FAT CLAY (CH);** very stiff to hard; light brown, reddish tan; shaley; few to little iron oxide stains, ferrous and calcareous nodules; trace gypsum and sand (720.5 ft)											
	S	4.5+			24.3	60	24	36	87			5.3	102.0	
	S	3.0			21.2								104.2	4.9
10	T	8, 10												
15	S	4.5+												
	T	11, 10												
20	S	1.0												
	T	13, 11												
	▽													
25	S	2.5												
	T	25, 25	26.0 ft											

End of boring at 26.0' (699.0 ft)

Notes:
-seepage at 24 feet during drilling
-water at 24 feet upon completion

30

35

BORING LOG

D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building
CLIENT: Huitt-Zollars, Inc.
PROJECT NUMBER: G22-2228
START DATE: 10/13/2022 FINISH DATE: 10/13/2022
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Sherman, TX
GPS COORDINATES: N33.697996, W96.574828
GROUND ELEVATION: Approx. 725 feet
DRILL METHOD: Hollow Stem Flight Auger
DRILLED BY: Markevian Smith (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- ∇ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+		**LEAN CLAY (CL);** very stiff to hard; brown, light brown; few ferrous nodules											
	S	4.5+				15.9	49	21	28				11.5	115.3	
	S	2.5				20.5									
	S	2.0													
5	S	3.0				23.1									
	T	5, 7													
	S	3.0				26.4									
10	S	4.0		-light brown, reddish tan; shaley; few to little iron oxide stains, ferrous and calcareous nodules; trace gypsum; sandy below 8 feet		14.7				64					
	T	7, 8													
15	S	3.5													
	T	8, 9													
20	S	3.0													
	T	9, 9													
	∇														
25	S	1.0													
	T	16, 17		26.0 ft											
				End of boring at 26.0' 699.0 ft											
30															
35															

Notes:
-seepage at 24 feet during drilling
-water at 24 feet upon completion

D&S ENGINEERING LABS

BORING LOG

B3

PROJECT: Texoma Parkway Building
CLIENT: Huitt-Zollars, Inc.
PROJECT NUMBER: G22-2228
START DATE: 10/13/2022 FINISH DATE: 10/13/2022
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Sherman, TX
GPS COORDINATES: N33.698168, W96.574649
GROUND ELEVATION: Approx. 724 feet
DRILL METHOD: Hollow Stem Flight Auger
DRILLED BY: Markevian Smith (D&S)

Legend:
■ S-Shelby Tube
☒ N-Standard Penetration
T-Texas Cone Penetration
C-Core
B-Bag Sample
∇ - Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Material Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	AU			**FILL: SANDY LEAN CLAY (CL);** very stiff to hard; light brown, reddish tan; few to some aggregate fragments											
	S	4.0		2.0 ft		18.6									
	S	4.0		722.0 ft **LEAN CLAY (CL);** stiff to very stiff; brown, light brown; few ferrous nodules		20.1	48	23	25						
	S	3.5													
5	S	3.0				21.5				85					
	T	4, 5													
	S	3.5													
	S	2.5													
10	S	2.0				21.1									
	T	8, 6													
15	S	4.5+		-light brown, reddish tan; shaley; few to little iron oxide stains, ferrous and calcareous nodules; trace gypsum below 14 feet		16.3				88				114.4	11.3
	T	12, 12													
20	S	3.0													
	T	5, 10													
25	S	3.5													
	T	19, 21		26.0 ft											
				End of boring at 26.0' 698.0 ft Notes: -dry during drilling -dry upon completion											
30															
35															

BORING LOG

D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building
CLIENT: Huitt-Zollars, Inc.
PROJECT NUMBER: G22-2228
START DATE: 10/13/2022 FINISH DATE: 10/13/2022
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Sherman, TX
GPS COORDINATES: N33.698406, W96.574806
GROUND ELEVATION: Approx. 726 feet
DRILL METHOD: Cont. Push
DRILLED BY: Markevian Smith (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Graphic Log	Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+		**FILL: SANDY LEAN CLAY (CL);** very stiff to hard; light brown, reddish tan; few to some aggregate fragments		10.3									
	S	4.5+		4.0 ft		17.8									
5	S	4.5+		**FAT CLAY (CH);** very stiff to hard; light brown, brown; shaley; trace to few iron oxide stains, ferrous and calcareous nodules (722.0 ft)		20.3	55	23	32						
	S	3.5													
	S	3.0				20.7									
10	S	2.5		10.0 ft		29.3									
				End of boring at 10.0' (716.0 ft)											

Notes:
-dry during drilling
-dry upon completion

15
20
25
30
35

BORING LOG

D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building
CLIENT: Huitt-Zollars, Inc.
PROJECT NUMBER: G22-2228
START DATE: 10/13/2022 FINISH DATE: 10/13/2022
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Sherman, TX
GPS COORDINATES: N33.697883, W96.575022
GROUND ELEVATION: Approx. 723 feet
DRILL METHOD: Cont. Push
DRILLED BY: Markevian Smith (D&S)

Legend:
- S-Shelby Tube
- N-Standard Penetration
- T-Texas Cone Penetration
- C-Core
- B-Bag Sample
- Water Encountered

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	Description	REC (%) RQD (%)	MC (%)	Atterberg Limits LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0	S	4.5+	**FILL: SANDY LEAN CLAY (CL);** very stiff to hard; light brown; few to some aggregate fragments		4.0									
	S	NT												
5	S	4.5+			16.7	48	22	26						
	S	NT	7.0 ft / 716.0 ft											
	S	4.0	**FAT CLAY (CH);** very stiff to hard; light brown, reddish tan; shaley; trace to few iron oxide stains, ferrous and calcareous nodules		23.6									
10	S	4.5+	10.0 ft / 713.0 ft											

End of boring at 10.0'

Notes:
-dry during drilling
-dry upon completion

15
20
25
30
35

SOLUBLE SULFATE CONTENT RESULTS

TEX 145-E

D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building

PROJECT NUMBER: G22-2228

CLIENT: Huitt-Zollars, Inc.

LOCATION: Sherman, TX

Boring Number:	Depth (feet):	Soil Description	Soluble Sulfate Content (ppm)
B2	1-2	**LEAN CLAY (CL);** brown, light brown	100

SWELL TEST RESULTS

D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building

PROJECT NUMBER: G22-2228

CLIENT: Huitt-Zollars, Inc.

LOCATION: Sherman, TX

Boring Number	Depth feet	Initial Moisture Content, %	Applied Pressure, psf	Vertical Swell, %
B1	7-8	24.3	791	5.3
B2	1-2	15.9	130	11.5

UNCONFINED COMPRESSION TEST

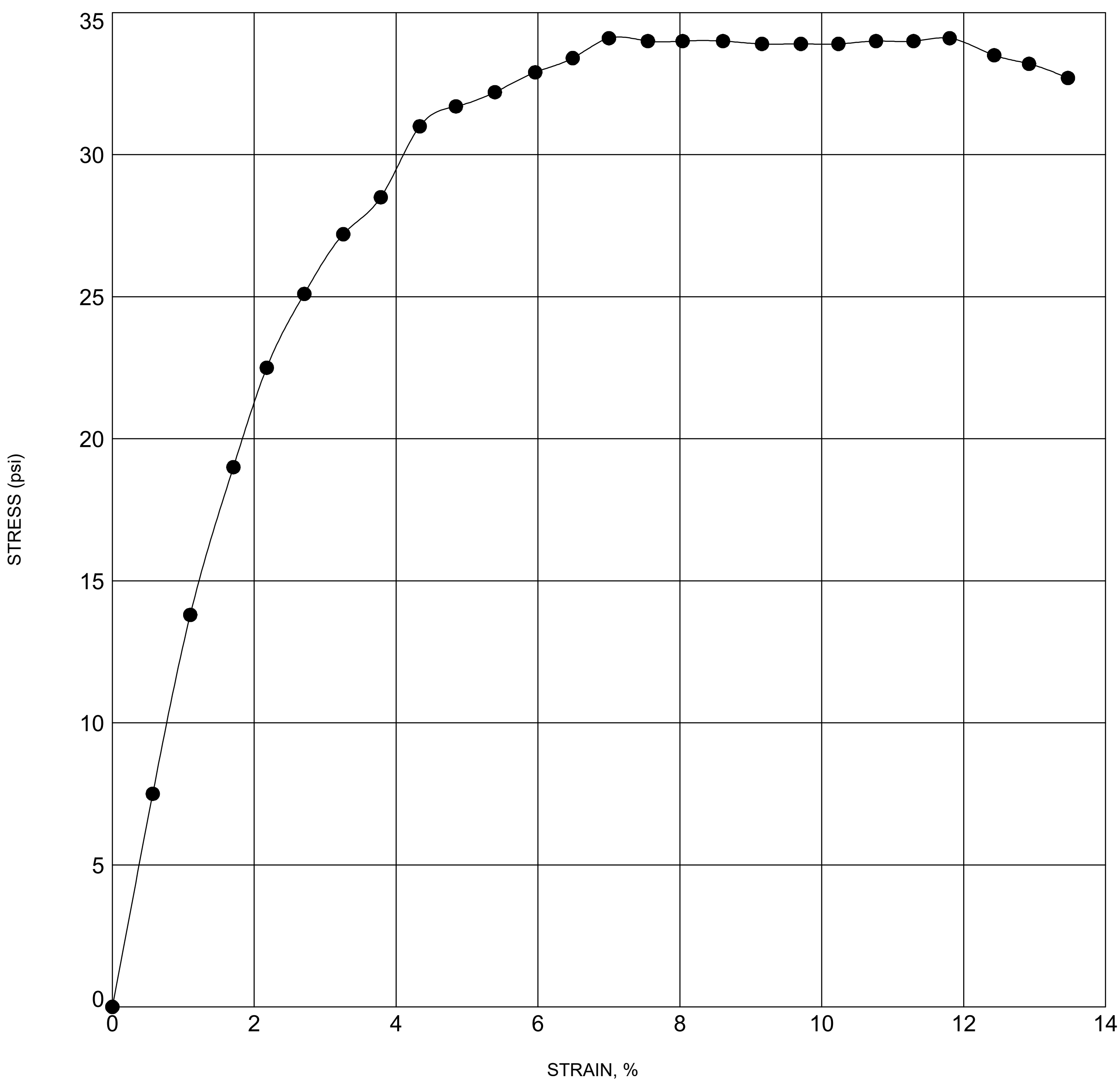
D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building

CLIENT: Huitt-Zollars, Inc.

PROJECT NUMBER: G22-2228

LOCATION: Sherman, TX

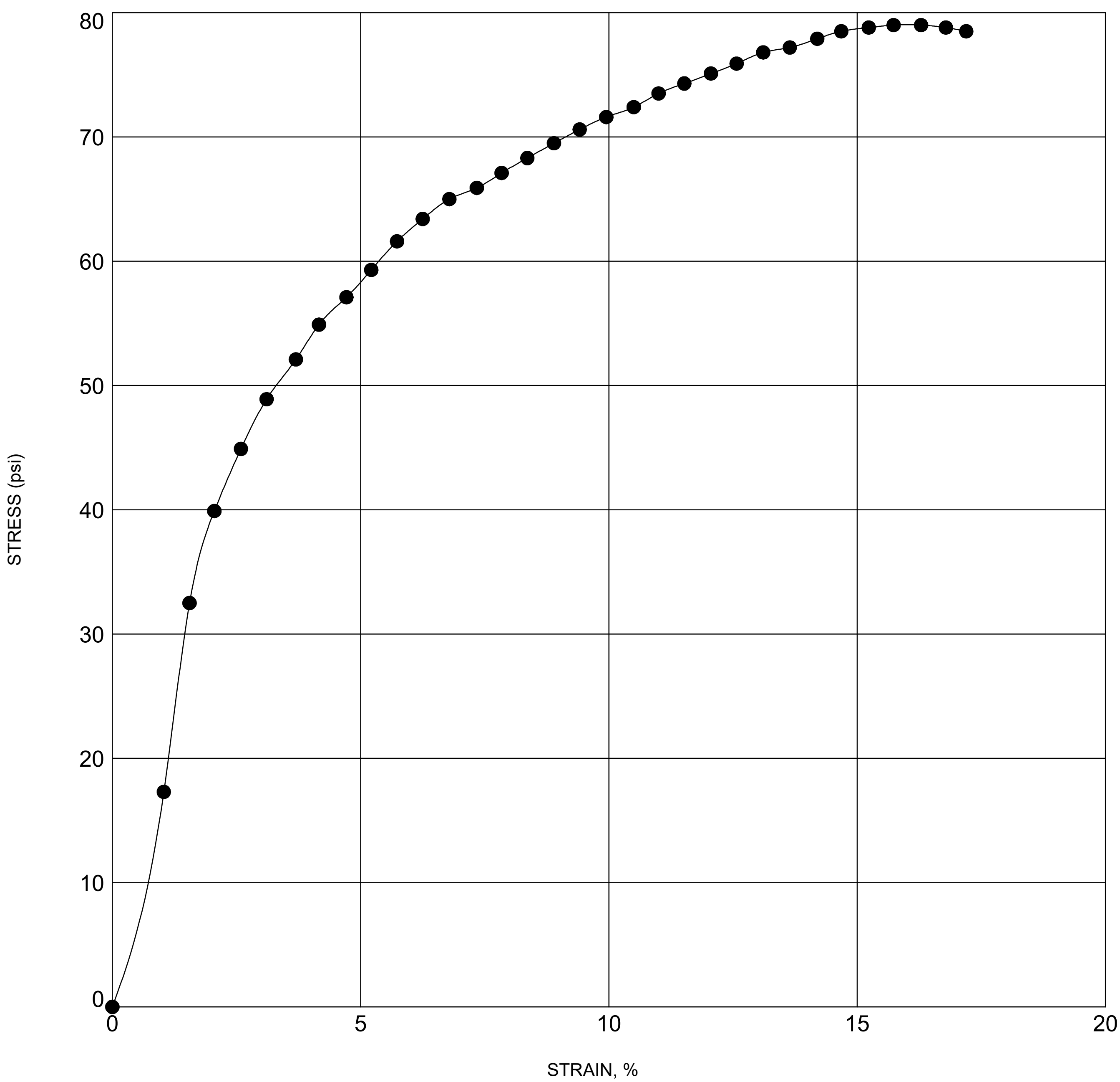


Borehole	Depth	Description	γ_d	MC%
B1	9.0	**FAT CLAY (CH);** very stiff to hard; light brown, reddish tan; shaley	104.2	21.2

UNCONFINED COMPRESSION TEST

D&S ENGINEERING LABS

PROJECT: Texoma Parkway Building
CLIENT: Huitt-Zollars, Inc.
PROJECT NUMBER: G22-2228
LOCATION: Sherman, TX



	Borehole	Depth	Description	γ_d	MC%
●	**B3**	**14.0**	**LEAN CLAY (CL);** stiff to very stiff; brown, light brown	**114.4**	**16.3**

APPENDIX B - GENERAL DESCRIPTION OF PROCEDURES

ANALYTICAL METHODS TO PREDICT MOVEMENT

CLASSIFICATION TESTS

Classification testing is perhaps the most basic, yet fundamental tool available for predicting potential movements of clay soils. Classification testing typically consists of moisture content, Atterberg Limits, and Grain-size distribution determinations. From these results a general assessment of a soil's propensity for volume change with changes in soil moisture content can be made.

Moisture Content

By studying the moisture content of the soils at varying depths and comparing them with the results of Atterberg Limits, one can estimate a rough order of magnitude of potential soil movement at various moisture contents, as well as movements with moisture changes. These tests are typically performed in accordance with ASTM D2216.

Atterberg Limits

Atterberg limits determine the liquid limit (LL), plastic limit (PL), and plasticity index (PI) of a soil. The liquid limit is the moisture content at which a soil begins to behave as a viscous fluid. The plastic limit is the moisture content at which a soil becomes workable like putty, and at which a clay soil begins to crumble when rolled into a thin thread (1/8" diameter). The PI is the numerical difference between the moisture constants at the liquid limit and the plastic limit. This test is typically performed in accordance with ASTM D4318.

Clay mineralogy and the particle size influence the Atterberg Limits values, with certain minerals (e.g., montmorillonite) and smaller particle sizes having higher PI values, and therefore higher movement potential.

A soil with a PI below about 15 to 18 is considered to be generally stable and should not experience significant movement with changes in moisture content. Soils with a PI above about 30 to 35 are considered to be highly active and may exhibit considerable movement with changes in moisture content.

Fat clays with very high liquid limits, weakly cemented sandy clays, or silty clays are examples of soils in which it can be difficult to predict movement from classification testing alone.

Grain-size Distribution

The simplest grain-size distribution test involves washing a soil specimen over the No. 200 mesh sieve with an opening size of 0.075 mm (ASTM D1140). This particle size has been defined by the engineering community as the demarcation between coarse-grained and fine-grained soils. Particles smaller than this size can be further distinguished between silt-size and clay-size particles by use of a Hydrometer test (ASTM D422). A more complete grain-size distribution test that uses sieves to relative amount of particles according is the Sieve Gradation Analysis of Soils (ASTM D6913). Once the characteristics of the soil are determined through classification testing, a number of movement prediction techniques are available to predict the potential movement of the soils. Some of these are discussed in general below.

TEXAS DEPARTMENT OF TRANSPORTATION METHOD 124-E

The Texas Department of Transportation (TxDOT) has developed a generally simplistic method to predict movements for highways based on the plasticity index of the soil. The TxDOT method is empirical and is based on the Atterberg limits and moisture content of the subsurface soil. This method generally assumes three different initial moisture conditions: dry, "as-is", and wet. Computation of each over an assumed depth of seasonal moisture variation (usually about 15 feet or less) provides an estimate of potential movement at each initial condition. This method requires a number of additional assumptions to develop a potential movement estimate. As such, the predicted movements generally possess large uncertainties when applied to the analysis of conditions under building slabs and foundations. In our opinion, estimates derived by this method should not be used alone in determination of potential movement.

SWELL TESTS

Swell tests can lead to more accurate site-specific predictions of potential vertical movement by measuring actual swell volumes at in situ initial moisture contents. One-dimensional swell tests are almost always performed for this measurement. Though swell is a three-dimensional process, the one-dimensional test provides greatly improved potential vertical movement estimates than other methods alone, particularly when the results are "weighted" with respect to depth, putting more emphasis on the swell characteristics closer to the surface and less on values at depth.

POTENTIAL VERTICAL MOVEMENT

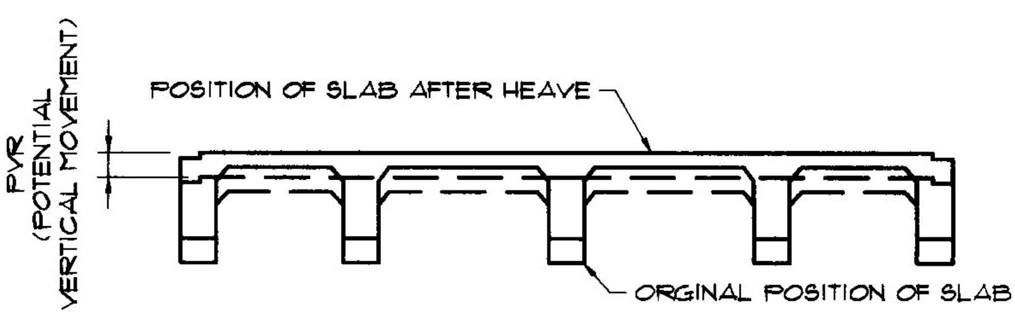
A general index for movement is known as the Potential Vertical Rise (PVR). The actual term PVR refers to the TxDOT Method 124-E mentioned above. For the purpose of this report the term Potential Vertical Movement (PVM) will be used since PVM estimates are derived using multiple analytical techniques, not just TxDOT methods.

It should be noted that all slabs and foundations constructed on clay or clayey soils have at least some risk of potential vertical movement due to changes in soil moisture contents. To eliminate that risk, slabs, and foundation elements (e.g., grade beams) should be designed as structural elements physically separated by some distance from the subgrade soils (usually 6 to 12 inches).

In some cases, a floor slab with movements as little as 1/4 of an inch may result in damage to interior walls, such as cracking in sheet rock or masonry walls, or separation of floor tiles. However, these cracks are often minor, and most people consider them 'livable'. In other cases, movement of one inch may cause significant damage, inconvenience, or even create a hazard (trip hazard or others).

Vertical movement of clay soils under slab on grade foundations due to soil moisture changes can result from a variety of causes, including poor site grading and drainage, improperly prepared subgrade, trees, and large shrubbery located too close to structures, utility leaks or breaks, poor subgrade maintenance such as inadequate or excessive irrigation, or other causes. A sampling of more common moisture control procedures to reduce the potential for movement due to these causes is presented in Appendix B.

PVM is generally considered to be a measurement of the change in height of a foundation from the elevation it was originally placed. Experience and generally accepted practice suggest that if the PVM of a site is less than one inch, the associated differential movement will be minor and acceptable to most people.



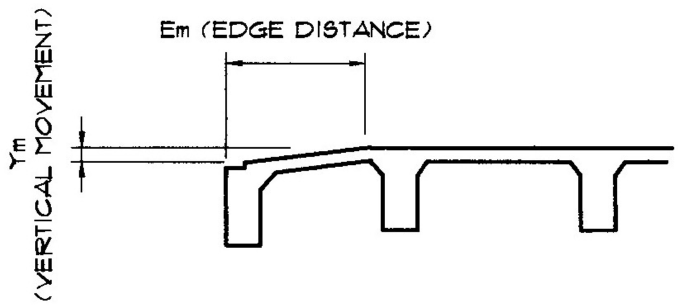
POTENTIAL VERTICAL RISE

SETTLEMENT

Settlement is a measure of a downward movement due to consolidation of soil. This can occur from improperly placed fill (uncompacted or under-compacted), loose native soil, or from large amounts of unconfined sandy material. Properly compacted fill may settle approximately 1 percent of its depth, particularly when fill depths exceed 10 feet.

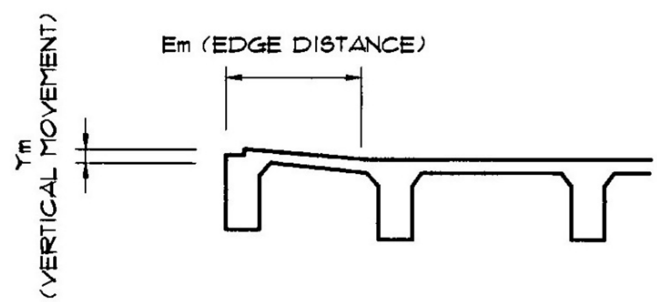
EDGE AND CENTER LIFT MOVEMENT (y_m)

The Post-Tensioning Institute (PTI) has developed a parameter of movement defined as the differential movement (y_m) estimated using the change in soil surface elevation in two locations separated by a distance e_m within which the differential movement will occur; e_m being measured from the exterior of a building to some distance toward the interior. All calculations for this report are based on the modified PTI procedure in addition to our judgment as necessary for specific site conditions. The minimum movements given in the PTI are for climatic conditions only and have been modified somewhat to account for site conditions which may increase the actual parameters.



CENTER LIFT PARAMETERS

"Center lift" occurs when the center, or some portion of the center of the building, is higher than the exterior. This can occur when the soil around the exterior shrinks, or the soil under the center of the building swells, or a combination of both occurs.



EDGE LIFT PARAMETERS

"Edge lift" occurs when the edge, or some portion of the exterior of the building, is higher than the center. This can occur when the soil around the exterior swells. It is not uncommon to have both the center lift and the edge lift phenomena occurring on the same building, in different areas.

SPECIAL COMMENTARY ON CONCRETE AND EARTHWORK

RESTRAINT TO SHRINKAGE CRACKS

One of the characteristics of concrete is that during the curing process shrinkage occurs and if there are any restraints to prevent the concrete from shrinking cracks can form. In a typical slab on grade or structurally suspended foundation there will be cracks due to interior beams and piers that restrict shrinkage. This restriction is called Restraint to Shrinkage (RTS). In post tensioned slabs, the post tensioning strands are slack when installed and must be stressed at a later time. The best procedure is to stress the cables approximately 30% within one to two days of placing the concrete. Then the cables are stressed fully when the concrete reaches greater strength, usually in 7 days. During this time before the cables are stressed fully, the concrete may crack more than conventionally reinforced slabs. When the cables are stressed, some of the cracks will pull together. These RTS cracks do not normally adversely affect the overall performance of the foundation. It should be noted that for exposed floors, especially those that will be painted, stained, or stamped, these cracks may be aesthetically unacceptable. Any tile which is applied directly to concrete or over a mortar bed over concrete has a high probability of minor cracks occurring in the tile due to RTS. It is recommended if tile is used to install expansion joints in appropriate locations to minimize these cracks.

UTILITY TRENCH EXCAVATION

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

FIELD SUPERVISION AND DENSITY TESTING

Field density and moisture content determinations should be made on each lift of fill at a frequency of one (1) test per lift per 3,000 square feet of fill, with a minimum of two (2) tests per lift within each building pad, one (1) test per lift per 100 linear feet of grade beam and/or footing perimeter backfill, one (1) test per lift per 100 linear feet of utility trench backfill, and one (1) test per lift per 5,000 square feet of pavement area. Supervision by the field technician and the project engineer is required. Some adjustments in the test frequencies may be required based upon the general fill types and soil conditions at the time of fill placement.

It is recommended that all site and subgrade preparation, proof rolling, and pavement construction be monitored by a qualified engineering firm. Density tests should be performed to verify proper compaction and moisture content of any earthwork. Inspection should be performed prior to and during concrete placement operations. D&S would be pleased to assist you on this project.

14805 Trinity Boulevard, Fort Worth, Texas 76155
Geotechnical 817.529.8464   Corporate 903.420.0014
www.dsenglabs.com
Texas Engineering Firm Registration # F-12796
Oklahoma Engineering Firm Certificate of Authorization CA 7181

D&S ENGINEERING LABS