Geotechnical Engineering Report

Toliver RV Dealership At Alliance

Fort Worth, Texas

November 18, 2020

D&S ENGINEERING LABS

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GEOTECHNICAL INVESTIGATION
D&S ENGINEERING #G20-2194
TOLIVER RV DEALERSHIP AT ALLIANCE
FORT WORTH, TEXAS 77354

Mr. Powell,

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. GP20-2194 dated October 7, 2020. Authorization to proceed was received October 13, 2020.

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

11-18-2020

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GEOTECHNICAL INVESTIGATION
TOLIVER RV DEALERSHIP AT ALLIANCE
FORT WORTH, TEXAS 77354

1.0 PROJECT DESCRIPTION

This report presents the results of the geotechnical investigation for the proposed Toliver RV Dealership project. The site is located along east of I-35W highway approximately 2,700 feet south of its intersection with Eagle Parkway in Fort Worth, Texas. We understand that the project consists of the construction of a new building and associated pavements.

The site is currently an open space covered with small to medium height grass. Based on the available NCTCOG topographic maps (www.dfwmaps.com), the site slopes from northwest corner to southeast corner of the property and the elevation change within the building footprints is on the order of about 4 feet. We anticipate cuts and fills on the order of about 2 feet to achieve the final building pads elevation.

2.0 PURPOSE AND SCOPE

The purpose of this investigation was to:

- Identify the subsurface stratigraphy 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 of foundations for the new structure and associated pavements.

The scope of this investigation consisted of:

- Drilling and sampling a total of eight (8) borings. Five (5) borings (B1 through B5) were advanced within the footprint of the proposed building to depths of about 30 feet. Three (3) borings (P1 through P3) were drilled to depths of about 5 to 10 feet within the pavement areas.
- Laboratory testing of selected soil and bedrock samples obtained during the field investigation.
- Preparation of a Geotechnical Report that includes the following:
    - Evaluation of Potential Vertical Movement (PVM)
    - Recommendations for the design of foundations
    - Recommendations for earthwork
    - Recommendations for pavement and pavement subgrade preparations

3.0 FIELD AND LABORATORY INVESTIGATION

3.1 General

The borings were advanced utilizing truck-mounted drilling equipment outfitted with continuous hollow stem flight augers. Undisturbed samples of cohesive soils and weathered bedrock were obtained using 3-inch diameter tube samplers, which 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 and weathered bedrock 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 and bedrock 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 estimated from the NCTCOG topographic map website which provides elevations at 2-feet 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 by the field crew and were further described by a staff engineer in the testing 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. Bedrock strata were described using standard geologic nomenclature.

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

- Moisture Content ASTM D2216
- Atterberg Limits ASTM D4318

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

- Unconfined Compressive Strength ASTM D2166
- Overburden Swell Testing
- Soluble Sulfates TEX-145-E

The results of these tests are presented at the corresponding sample depths on the appropriate Boring Log illustrations or summary tables, Appendix A.

3.2.1 Unconfined Compression Tests

Unconfined compressive strength testing was performed on selected samples of the weathered shale bedrock. These tests were performed in general accordance with ASTM D2166 for soil samples. 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.

3.2.2 Overburden Swell Tests

Selected samples of the near-surface soil were subjected to overburden swell testing. For this test, a sample is placed in a consolidometer and subjected to the estimated overburden pressure. The sample is then inundated with water and is allowed to swell. The moisture content of the sample is determined both before and after completion of the test. Test results are recorded, including the percent swell and the initial and final moisture contents.

3.2.3 Soluble Sulfates

Sulfate tests were performed on samples obtained from the field investigation, the results of which are provided in Appendix A (Boring Logs and Supporting Data). Subgrade materials in some areas of Texas have experienced sulfate-induced heave after 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 results of the sulfate tests performed on representative near-surface soil samples from test borings in this study indicate sufficiently low sulfate concentrations with values of about 100 ppm, and thus should be considered to pose a very low risk of sulfate-induced heaving after treatment in certain areas.

4.0 SITE CONDITIONS

4.1 Stratigraphy

Based upon a review of the recovered samples, as well as the Geologic Atlas of Texas, Dallas Sheet, this site is characterized by soil and bedrock strata associated with the undivided Pawpaw Formation, Weno Limestone and Denton Clay Formation. Subsurface bedrock of these formations typically consists of limestone, shale, calcareous shales (locally referred to as marl) and sandstone. Residual soils derived from the bedrock are clays that are moderately to highly plastic.

At the surface within the borings, native fat and lean clay soils are present. The clay soils present are generally stiff to very stiff in consistency, dark brown and brown in color, and contain varying amounts of calcareous and ferrous nodules. The native clays extend to depths of about 2 to 6 feet.

The overburden soils are underlain by weathered shale bedrock strata. The weathered shale strata present are generally very soft to soft in rock hardness, various shades of gray, light brown and brown in color, contains varying amounts of limestone and calcareous laminations, are fissile in structure and are calcareous in content. The weathered shale strata present extend to depths of about 17 to 23 feet within the building borings and to maximum depths explored of about 5 to 10 feet within the pavement borings.

Within Borings B1 through B5, fresh shale bedrock strata are present beneath the weathered shale strata. The fresh shale bedrock strata present are generally medium hard in rock hardness, dark gray and gray in color, are calcareous in content, and contain varying amounts of limestone seams. The fresh shale bedrock strata extended at least to the maximum depths explored of about 30 feet within the borings.

4.2 Groundwater

Groundwater seepage was not encountered either during drilling or upon the completion of drilling operations within the borings advanced. Groundwater levels should be anticipated to fluctuate with seasonal and annual variations in rainfall, and also 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 different methods for predicting movement, as described in Appendix B, and based on our experience and professional opinion.

At the time of our field investigation, the overburden soils were found to be very dry to average in moisture condition. Based upon the results of our analysis, the site is estimated to possess a PVM of about 3 to 8.5 inches at the soil moisture conditions existing at the time of the field 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 will increase. 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 have very high potential for post-construction vertical movement with changes in soil moisture content. Considering the type of structure, reported loading intensity and the subsurface soil and groundwater conditions, we recommend that the new building be supported on a drilled shaft foundation system using a structurally supported floor slab system. The following paragraphs provide design recommendations and construction considerations for deep foundation design and construction.

6.1 Straight-sided Drilled Shafts

The structural loads for the new building may be supported on auger excavated, straight sided, reinforced concrete drilled shafts. Drilled shafts should be founded in medium hard, fresh shale encountered at depths of 17 to 23 feet below existing site grades. Drilled shafts may be designed to transfer imposed loads into the bearing stratum using a combination of end-bearing and skin friction. The allowable side friction may be used to resist both axial loading and uplift.

We recommend drilled shafts be a minimum of 18 inches in diameter and penetrate a minimum of 3 feet into the gray/dark gray fresh shale bedrock to utilize the full amount of allowable end bearing. The allowable side frictions to resist both axial loading and uplift may be taken from below a depth of 10 feet or from the bottom of any temporary casing used, whichever is deeper, to resist both axial loading and uplift. As there is appreciable strain-compatibility between the weathered and the fresh shales, the side friction for the different shale strata may be included in the shaft design for shafts extending into the fresh shale.

Table 1. Drilled Shaft Allowable Bearing

Material	Depth Below Current Grades (ft)	Allowable Side Friction (psf)	Allowable End Bearing (psf)
Weathered Shale	10 - 20	1,200	-
Fresh Shale, medium hard	>20	5,000	30,000

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, these pressures are approximated to be on the order of 1,500 psf of shaft area over an average depth of 10 feet. Often, 1/2 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 bearing stratum-to-concrete adhesion acting on that portion of the shaft that is in contact with the shale bedrock below the bearing strata.

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 Underreamed Shafts (Belled Piers)

Structural loads for the new building may be supported on auger-excavated, reinforced concrete underreamed drilled shafts founded in the shale bedrock at a minimum depth of 15 feet below final exterior grade.

Drilled-and-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 2.75 times the shaft diameter, and the minimum clearance between the edges of bells should be 5 feet. If the location of piers requires less clearance between bells than 5 feet, this office should be contacted for recommendations.

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 1,500 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 under-ream at an angle of 40 degrees from vertical.

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 3 against shear failure and should experience settlement of about ¾-inch or less.

6.3 Drilled Shaft Construction Considerations

Groundwater seepage was not encountered either during drilling or upon the completion of drilling operations within the borings advanced. Groundwater levels should be anticipated to fluctuate with seasonal and annual variations in rainfall, and also 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. Temporary casing should be locally available 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.

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 excavation, 2) identification of the bearing stratum, 3) minimum pier diameter and depth, 4) correct amount of reinforcement, 5) proper removal of loose material, and 6) that groundwater seepage, if present, is properly controlled.

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 circular forms equal to the diameter of the shaft.

Concrete used for the shafts should have a slump of 8 inches ± 1 inch. Individual shafts should be excavated in a continuous operation and concrete should be placed as soon after completion of the drilling as is practical. All shafts should be filled with concrete within 8 hours after completion of drilling. 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.

6.4 Pier-Supported Grade Beams

For pier-supported grade beams, a minimum void space of 16 inches should be provided beneath all grade beams (and the floor slab if structurally suspended). If a structurally suspended floor slab system is utilized, 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 6 inches clearance below the lowest suspended utility, whichever is greater. 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, in order to limit corrosion of the metal components.

Structural cardboard carton forms (void boxes) or StormVoid™ carton forms may be used to provide the required voids beneath the grade beams; 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 to reduce the risk of crushing the cardboard forms during concrete placement and finishing operations. If required by the carton form manufacturer, the protection should be placed per manufacturer recommendations. 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 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 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 for the backfill material as determined by ASTM D698 (standard Proctor). The fill should be placed at a moisture content that is at least three (3) percent above the optimum moisture content, as determined by that same test. This fill should extend the full depth of the grade beam and void box and should extend a minimum distance of three feet away from the exterior grade beam perimeter.

6.5 Lateral Load Parameters

The following soils and rock geo-parameters for lateral analysis of drilled shafts for use in LPILE® or other lateral load software. These values are based on the stratigraphy, laboratory data and experience, and were selected to conservatively approximate the subsurface conditions across the site. The recommended model layers are “Stiff Clay w/o Free Water” and “Weak Rock”. The depth ranges are based on the borings drilled. In view of the nature and characteristics of the materials present, we recommend that the lateral resistance parameters be neglected for the uppermost 3 feet of soil materials to account for seasonal and annual cyclic variations in soil desiccation and contraction, and potential future erosion. However, unit weight in this zone can be considered in the design, and the lateral loads may be resolved at the top of the ground surface.

Table 2. Subsurface Materials

Stratum	Depth (ft)	Software Material Designation	Unit Weight (pcf)
FAT CLAY / weathered shale	3 – 6	Stiff Clay w/o Free Water	115
SHALE, weathered	6 - 20	Stiff Clay w/o Free Water	120
SHALE, fresh, medium hard	> 20	Weak Rock	130

Table 3. Recommended Geotechnical Parameters

Stratum	Depth (ft)	Undrained Cohesion (psf)	Unconfined Compressive Strength (psi)	Modulus (psi)	RQD	Strain Factor ε_{50}
FAT CLAY / weathered shale	3 – 6	500	NA	NA	NA	0.015
SHALE, weathered	6 - 20	2,000	NA	NA	NA	0.007
SHALE, fresh, medium hard	> 20	NA	150	10,000	90	0.0003

7.0 EARTHWORK RECOMMENDATIONS

The near-surface soils present have significant potential for post-construction vertical movement with changes in subsurface soil moisture changes. We have the following earthwork recommendations for a structurally-supported floor slab system in conjunction with a pier and beam foundation system.

7.1 Soil Preparation for Structurally Suspended Floor Slabs

- Strip the site of all vegetation, organic soil, and deleterious material within the new building areas. Typically, 4 to 6 inches is 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 between 92 and 96 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and to a moisture content that is at least three (3) percent above the optimum moisture content, as determined by the same test. This procedure should extend at least 5 feet beyond the perimeter of the new structure.

- 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 retarder or barrier should be securely bonded to the underside of the floor slab to promote continued contact after concrete placement. All seams and penetrations through the barrier should be sealed in accordance with the manufacturer's requirements.

7.2 Additional Considerations

The following are considered to be best practices to minimize the potential for post-construction vertical movement.

- Where possible, 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 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.

- Exterior grades should slope away from foundations and flatwork to the maximum degree possible. Ideally, the grade should slope away from foundation at a minimum rate of five (5) percent within the first 10 feet of the foundation's perimeter.

- Water should not be allowed to pond next to structure foundations, pavements or other flatwork. 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. We strongly recommend that excessive watering near foundations be avoided.

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 seal the final pavement plans and associated specifications for this project.

8.2 Behavior Characteristics of Expansive Soils Beneath Pavement

Near-surface soils anticipated to be present once grading operations are completed at this site are considered to have high potential for volume change with changes in soil moisture content. Increased moisture content can result in reduced soil stiffness. 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 by surface or subsurface water, poor drainage or the addition of excessive irrigation after periods of no moisture, the soil strength can reduce causing distress to pavements as traffic travels over it.

The edges of pavements are particularly prone to moisture variations, therefore, these areas often experience the most distress (cracking, displacement, etc.). 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 excessive water entering pavement subgrades 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 should be extended to at least 2 feet beyond the back of curbs or edges of pavements.

8.3 Subgrade Strength Characteristics

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

8.4 Pavement Subgrade Preparation Recommendations

The anticipated subgrade soils will be clay soils in the proposed paving areas. These soils can become weak with appreciable increases 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 is to treat the soils with lime. For the pavement subgrade soils, we generally recommend treating the soils with lime to improve the soil characteristics and increase the long-term performance of the subgrade. As an alternative to lime treatment, a layer of compacted aggregate base may also be used.

The following recommendations discuss subgrade preparation and the two subgrade preparation alternatives.

8.4.1 Soil Preparation

- Strip the site of all vegetation and remove any remaining organic or deleterious material under the planned paved areas. Typically, 6 to 8 inches is sufficient for this purpose.

- Cut as needed to required pavement subgrade elevation.

The soils beneath the pavement are estimated to possess a PVM of about 8 inches at the soil moisture conditions existing at the time of the field investigation. Consideration may be given to reduce the PVM to on the order of 4.5 inches by undercutting and moisture conditioning the subgrade an additional two feet. This will help to reduce potential pavement movements.

We anticipate that excavation of overburden soils can be accomplished with conventional earthwork equipment and methods.

- Prior to the placement of fill, aggregate base, or treatment with lime, scarify, rework, and recompact the exposed stripped subgrade to a depth of 12 inches. The scarified soils should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and at a moisture content that is at or above the optimum moisture content, as determined by the same test.

- 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 more than about 1/2-inch or pump excessively (exhibiting “waving” action) during proof rolling should be removed and replaced with well-compacted, on-site clayey material or allowed to dry 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.

- Following proof rolling, the upper one foot of exposed soil should be scarified and recompacted. The scarified fill should be compacted to at least 95 percent of the maximum dry density, as determined by ASTM D698 (standard Proctor), and at a moisture content that is at or above the optimum moisture content, as determined by the same test.

- In areas to receive fill, fill may be derived from on-site or may be imported. 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 placed at a moisture content that is at or 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 discing 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.

- Water should not be allowed to pond on the prepared surface once lime treatment is performed or aggregate base has been placed. To that end, the lime-treated or aggregate base subgrade surface should be shaped in a way that will allow water to shed to one or more edges of the prepared subgrade.

- Field density and moisture content testing should be performed at the rate of one test per 10,000 square feet in pavement parking areas. These tests are necessary to determine if the recommended moisture and compaction requirements have been attained.

- Surface grading adjacent to the edges of pavements and other flatwork should be sloped away from the edges to the maximum degree possible. Where minimum recommended slopes of adjacent surface grades cannot be achieved, the edges of pavements should be thickened a minimum of 2-feet wide along each edge.

8.4.2 Lime Treatment

Once the subgrade has been brought to rough final grade the surface should be treated with lime in accordance with the following recommendations.

- 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 a minimum of 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 below 15 (Atterberg Lime series), or to increase pH of the soil-lime mixture to 12.4 (pH series). To account for error, 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 roadway 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 7 days is needed for these sulfate bearing soils at this site. During the curing period, the material should be kept moist at least 5% above optimum. Sprinkling may be required to maintain a moisture of at least 5% above optimum during the curing period. 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" sieve: 100%
    - Minimum passing No. 4 sieve: 60%

- After sufficiently re-mixed, the soil and lime mixture should be compacted should be reworked to bring the moisture content down close to optimum and achieve 95% of the maximum dry density.

- 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 test per 10,000 square feet in pavement parking areas.

8.4.3 Aggregate Base

As an alternative to lime treatment, an aggregate base layer 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 meet the gradation, durability and plasticity requirements of TxDOT Item 247 Grade 1-2 or better (2014). Aggregate base material should be uniformly compacted in maximum 6-inch compacted lifts 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 10,000 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 5 inches for light-duty automobile parking over 6-inches of lime treated or aggregate base. Concrete thickness should be increased to 6 inches for medium duty fire lanes and drive lanes over 8 inches of lime treatment or aggregate base. For dumpster areas and heavy duty areas, the concrete thickness should be 7 inches over 8 inches of lime treatment or aggregate base. Actual traffic loading, frequency, and intensity may require an increase in these minimum recommendations. We have the following concrete mix design recommendations:

- Recommended minimum 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

A minimum of 0.1 percent of steel be used for all concrete pavements. We recommend No. 3 bars reinforcement spaced at 18-inches on-center each way for light and medium duty areas. For dumpster and heavy-duty areas, we recommend No. 4 bars reinforcement spaced at 18-inches on-center each way. Reinforcement requirements may increase depending on specific traffic loading and design life parameters.

8.8 Surface Drainage

Proper drainage is critical to the performance of the paved areas. Positive surface drainage should be provided that directs surface water away from pavement edges. Where possible, we recommend that a slope of at least 5 percent be provided. The slopes should direct water away from the structure and should be maintained throughout construction and the life of the structure.

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. Fill 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. Field density and moisture content testing should be performed at the rate of one test per 100 linear feet in utility trenches. It is not uncommon to realize some settlement along the trench backfill. 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 the expansive soil. These movements may result in damage to the lines, especially at connections. Oversized penetration sleeves or flexible connections should be considered to account for potential differential movement between the building and utilities. Alternatively, utilities should extend below structural elements when crossing to enter a structure footprint.

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 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. Utility trenches extending under structures should include fat clay or concrete cut-off collars at the perimeter/edge to prevent the transmission of water along trench lines.

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.

The concrete flatwork should have adequately spaced contraction joints to control shrinkage cracking. Past experience indicates that reinforced concrete flatwork with sealed contraction joints on a 4 to 10-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. Rubberized asphalt, silicone or other suitable flexible sealant could be used to seal the joints. 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.

9.3 Surface Drainage

Proper drainage is critical to the performance and condition of the building foundation, pavements and flatwork. Positive surface drainage should be provided that directs surface water away from the building, pavements and flatwork. Where possible, we recommend that the exterior grades slope away from foundations at the rate of five (5) percent in the first five (5), 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, and other features, 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. The joints created at the interface of the flatwork and building line must 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 must be maintained and resealed as needed and should be part of a periodic inspection and maintenance program.

However, 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.

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. If used, landscaping should be kept as far away from the foundation as possible and positive drainage away from the structure should be installed and 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 in localized soil shrinkage due to desiccation of the soil by the root system, possibly leading to differential movements of the structure. The desiccation zone varies by tree, but trees should not be planted closer to structures than the mature tree height, and in no cases, should the drip-line of the mature tree extend closer than 10-feet of rooflines. 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. To the extent practical, it is recommended that trees scheduled for removal (where required) in the vicinity of the proposed structure 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.

9.5 Site Grading

Expansive clay cut and fill slopes should be gentle and preferably should not exceed about 4 horizontal 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 and Excavation Difficulties

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. Where space allows, temporary slopes should be sloped at 1.5 horizontal to 1 vertical (1.5H: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 insure the safety of the excavation and surrounding property.

10.0 SEISMIC CONSIDERATION

North Central Texas is generally regarded as an area of low seismic activity. Based on the boring log data and general geologic information gathered, we recommend that Soil Site Class "C" 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 a lightly to moderately loaded structure foundations and associated pavements. 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.

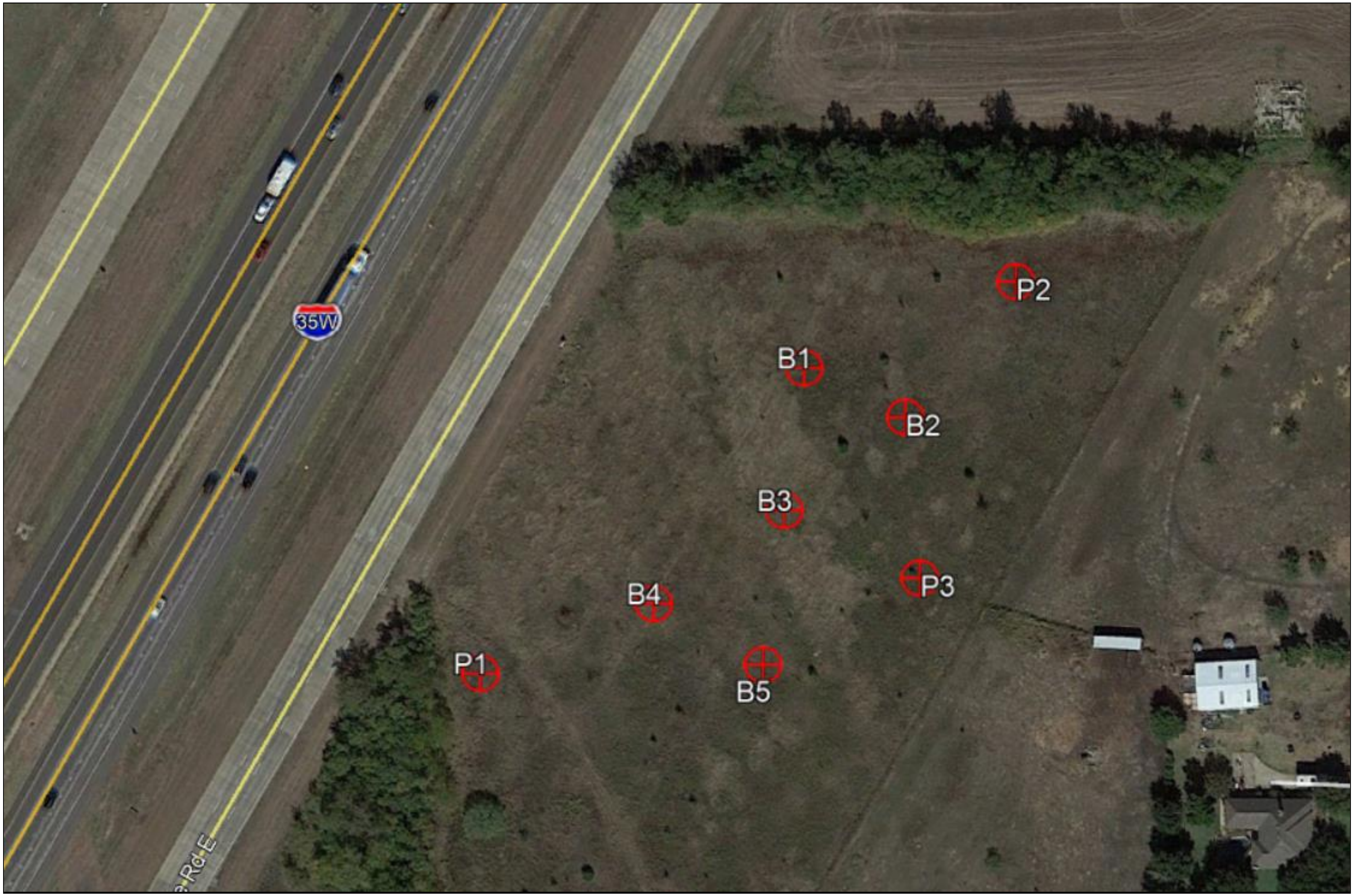
Recommendations contained herein are not considered applicable for an indefinite period of time. Our office must be contacted to re-evaluate the contents of this report if construction does not begin within a one-year period after completion of this report.

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.

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

TOLIVER RV DEALERSHIP AT ALLIANCE

FORT WORTH

TEXAS

SHEET NO.

G1

DATE DRILLED

October 31, 2020

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 (# blows/ft)	UCS (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 (# blows/ft)	TCP (#blows/ft)	Relative Density (%)
Very Loose	0 - 4	< 8	0 - 15
Loose	5 - 10	8 - 20	15 - 35
Medium Dense	11 - 30	20 - 60	35 - 65
Dense	31 - 50	60 - 100	65 - 85
Very Dense	> 50	> 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

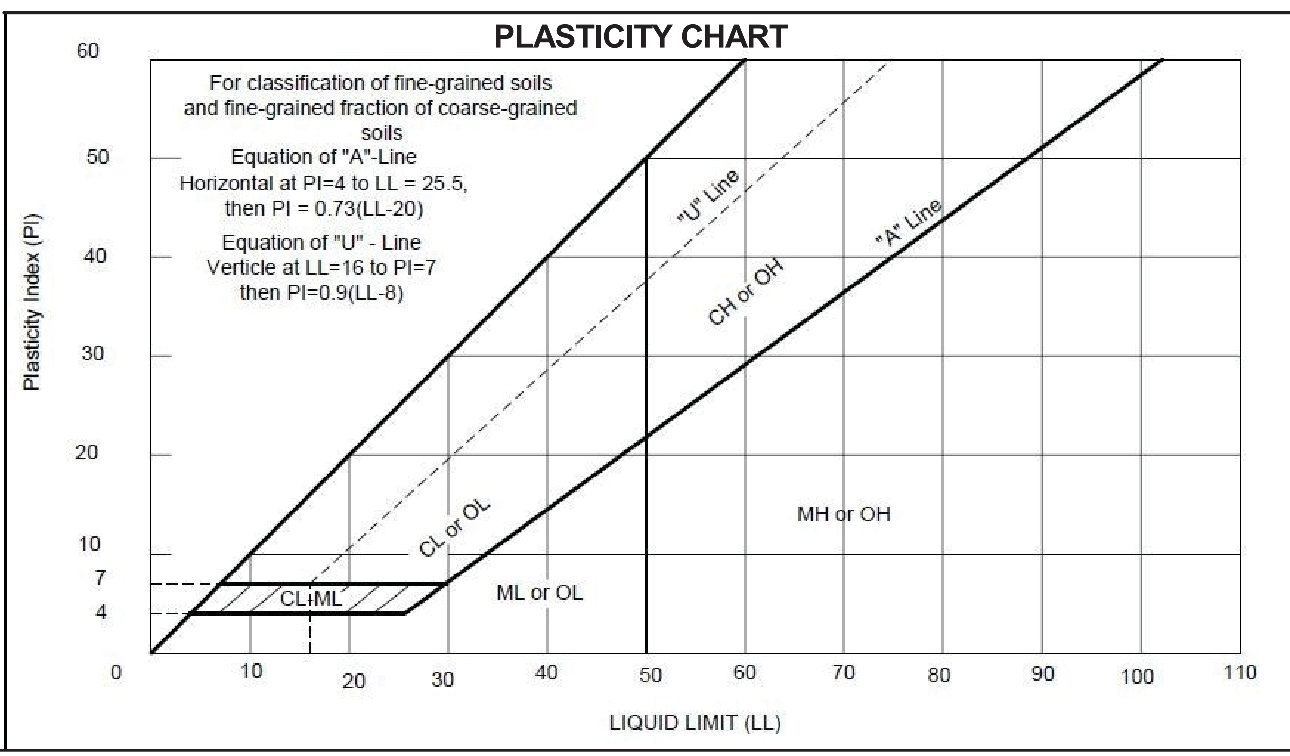
D&S ENGINEERING LABS

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)	$Cu \geq 4$ and $1 \leq Cc \leq 3$	GW	WELL-GRADED GRAVEL
			$Cu < 4$ and/or $[Cc < 1$ 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)	$Cu \geq 6$ and $1 \leq Cc \leq 3$	SW	WELL-GRADED SAND
			$Cu < 6$ and/or $[Cc < 1$ 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	$\frac{\text{Liquid limit} - \text{oven dried}}{\text{Liquid limit} - \text{not dried}} < 0.75$	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	$\frac{\text{Liquid limit} - \text{oven dried}}{\text{Liquid limit} - \text{not dried}} < 0.75$	OH	ORGANIC CLAY ORGANIC SILT
HIGHLY ORGANIC SOILS	PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR			PT	PEAT



BORING LOG

PROJECT: Toliver RV Dealership at Alliance
CLIENT: Dunaway Associates
PROJECT NUMBER: G20-2194
START DATE: 10/31/2020 FINISH DATE: 10/31/2020
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Fort Worth, TX
GPS COORDINATES: N32.993683, W97.296449
GROUND ELEVATION: Approx. 657 feet
DRILL METHOD: Hollow Stem Flight Auger
DRILLED BY: Octavio Herrera (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+	**FAT CLAY (CH);** very stiff; dark brown, brown; trace to few calcareous nodules											
	S	4.5+			24.8	56	25	31				4.0	96.1	
	T	8,8												
5	NR													
	T	5,7	6.0 ft											
	S	4.0	651.0 ft **SHALE;** highly to completely weathered; very soft; light brown; calcareous; fissile; frequent calcareous laminations		22.6									
	T	6,6												
10	S	2.5	10.0 ft		22.0	54	18	36						
	T	19,17	647.0 ft **SHALE;** slightly to moderately weathered; very soft; dark gray, brown, light brown; calcareous; fissile; frequent calcareous laminations											
15	S	4.5+			21.7								105.5	11.4
	T	18,21	19.0 ft											
20	B		638.0 ft **SHALE;** fresh; medium hard; dark gray; calcareous											
	T	50=1.0" 50=0.5"	-Limestone seam at 22-23'											
25	B													
	T	50=0.25" 50=0.0"												
30	B		30.1 ft											
	T	50=0.75" 50=0.25"	626.9 ft End of boring at 30.1'											
35														

Notes:
-dry during drilling
-dry upon completion

BORING LOG

B2

D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance
CLIENT: Dunaway Associates
PROJECT NUMBER: G20-2194
START DATE: 10/31/2020 FINISH DATE: 10/31/2020
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Fort Worth, TX
GPS COORDINATES: N32.993579, W97.296195
GROUND ELEVATION: Approx. 656 feet
DRILL METHOD: Hollow Stem Flight Auger
DRILLED BY: Octavio Herrera (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; dark brown; few calcareous nodules; trace ferrous nodules											
	S	4.5+		2.0 ft		15.7	48	20	28						
	T	9,12		**SHALE;** highly to completely weathered; very soft; light brown; calcareous; fissile; frequent calcareous laminations (654.0 ft)											
	S	4.5+													
5	S	4.5+				13.5	52	17	35				11.0	123.3	
	T	10,14													
	S	4.5+				17.0									
	T	14,21													
10	S	4.5				22.7								100.4	2.7
	T	13,19													
				14.0 ft											
15	S	4.5+		**SHALE;** slightly to moderately weathered; very soft to soft; light brown, gray; calcareous (642.0 ft)		15.8								114.0	15.9
	T	50=3.0" 50=2.5"													
				17.0 ft											
				SHALE; fresh; medium hard; dark gray, gray; calcareous (639.0 ft)											
20	B														
	T	50=1.0" 50=0.5"													
25	B														
	T	50=0.25" 50=0.0"													
30	B			30.1 ft											
	T	50=0.5" 50=0.25"		End of boring at 30.1' (625.9 ft)											
35															

Notes:
-dry during drilling
-dry upon completion

BORING LOG

D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance	LOCATION: Fort Worth, TX
CLIENT: Dunaway Associates	GPS COORDINATES: N32.993383, W97.296499
PROJECT NUMBER: G20-2194	GROUND ELEVATION: Approx. 656 feet
START DATE: 10/31/2020 FINISH DATE: 10/31/2020	DRILL METHOD: Hollow Stem Flight Auger
LOGGED BY: Dalton Hubbard (D&S)	DRILLED BY: Octavio Herrera (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	2.25	**FAT CLAY (CH);** stiff to very stiff; dark brown; trace calcareous nodules		27.8									
	S	4.5+												
	S	4.5+												
	T	1,2												
5	S	4.5+			20.7	61	21	40				3.9	103.4	
	T	5,4	6.0 ft											
	S	4.5+	**SHALE;** highly to completely weathered; very soft; light brown; calcareous; frequent calcareous laminations; fissile (650.0 ft)		16.3								118.4	5.7
	T	13,8												
	S	3.5												
10	S	3.5			27.8	51	20	31						
	T	13,15												
			14.0 ft											
15	S	4.5	**SHALE;** slightly to moderately weathered; soft; dark gray, light brown; calcareous (642.0 ft)		17.9								104.5	3.0
	T	22,40												
			19.0 ft											
20	B		**SHALE;** fresh; medium hard; dark gray (637.0 ft)											
	T	50=0.5" 50=1.0"												
			-Limestone seam at 22-23'											
25	B													
	T	50=0.25" 50=0.0"												
30	B		30.1 ft											
	T	50=1.0" 50=0.5"	End of boring at 30.1' (625.9 ft)											
35														

Notes:
-dry during drilling
-dry upon completion

BORING LOG

PROJECT: Toliver RV Dealership at Alliance
CLIENT: Dunaway Associates
PROJECT NUMBER: G20-2194
START DATE: 10/31/2020 FINISH DATE: 10/31/2020
LOGGED BY: Dalton Hubbard (D&S)
LOCATION: Fort Worth, TX
GPS COORDINATES: N32.993187, W97.296826
GROUND ELEVATION: Approx. 656 feet
DRILL METHOD: Hollow Stem Flight Auger
DRILLED BY: Octavio Herrera (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+	**FAT CLAY (CH);** very stiff; dark brown; trace calcareous nodules		26.5									
	S	4.5+												
	S	4.5+			21.0	62	22	40				10.4	103.5	
	T	5,8												
5	S	4.5+	5.0 ft		18.2									
	T	13,47	651.0 ft **SHALE;** highly to completely weathered; very soft; light brown, brown; calcareous; fissile; trace limestone											
	B													
	T	10,12												
	B													
10	T	14,26												
15	S	4.5+			14.6									
	T	26,13	16.0 ft											
			640.0 ft **SHALE;** slightly to moderately weathered; very soft to soft; light brown, brown, dark gray; calcareous; fissile											
			19.0 ft											
	B		637.0 ft **SHALE;** fresh; medium hard; dark gray, gray; calcareous											
20	T	50=0.5" 50=0.0"												
25	T	50=0.75" 50=0.25"												
			-Limestone seam at 27'											
30	B		30.1 ft											
	T	50=0.5" 50=0.25"	625.9 ft End of boring at 30.1'											
35														

Notes:
-dry during drilling
-dry upon completion

BORING LOG

D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance
CLIENT: Dunaway Associates
PROJECT NUMBER: G20-2194
START DATE: 10/31/2020 FINISH DATE: 10/31/2020
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Fort Worth, TX
GPS COORDINATES: N32.993058, W97.296552
GROUND ELEVATION: Approx. 655 feet
DRILL METHOD: Hollow Stem Flight Auger
DRILLED BY: Octavio Herrera (D&S)

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

Depth (ft)	Material Description	Bottom Depth / Elevation
0 – 5.0	**LEAN CLAY (CL);** very stiff; dark brown; trace calcareous nodules	5.0 ft / 650.0 ft
5.0 – 14.0	**SHALE;** highly to completely weathered; very soft; light brown, brown; fissile; calcareous; trace limestone	14.0 ft / 641.0 ft
14.0 – 23.0	**SHALE;** slightly to moderately weathered; very soft to soft; light brown, dark gray, gray; calcareous fissile -Limestone seam at 23'	23.0 ft / 632.0 ft
23.0 – 30.1	**SHALE;** fresh; medium hard; dark gray, gray; calcareous	30.1 ft / 624.9 ft

Depth (ft)	Sample Type	Hand Pen. (tsf) or SPT or TCP	REC (%) RQD (%)	MC (%)	LL (%)	PL (%)	PI	Passing #200 Sieve (%)	Total Suction (pF)	Clay (%)	Swell (%)	DUW (pcf)	Unconf. Compr. Str (ksf)
0–1	S	4.5+		24.6									
1–2	S	4.5+											
2–3	S	4.5+		17.5	49	21	28				3.9	109.4	
3–4	T	10,12											
4–5	S	4.5+		15.2	49	20	29				6.2	115.9	
5–6	T	11,18											
6–7	B												
7–8	T	21,11											
9–10	B												
10–11	T	12,11											
14–15	S	4.5+		17.9								106.9	10.6
15–16	T	28, 50=3.5"											
19–20	B												
20	T	50=3.0" 50=2.5"											
24–25	B												
25	T	50=0.25" 50=0.25"											
29–30	B												
30	T	50=0.25" 50=0.25"											

End of boring at 30.1'

Notes:
-dry during drilling
-dry upon completion

BORING LOG

D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance	LOCATION: Fort Worth, TX
CLIENT: Dunaway Associates	GPS COORDINATES: N32.993039, W97.297263
PROJECT NUMBER: G20-2194	GROUND ELEVATION: Approx. 656 feet
START DATE: 10/31/2020 FINISH DATE: 10/31/2020	DRILL METHOD: Cont. Push
LOGGED BY: Dalton Hubbard (D&S)	DRILLED BY: Jimmy Venegas (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	1.75		**FAT CLAY (CH);** stiff to very stiff; dark brown; trace to few calcareous nodules		26.1									
	S	4.5+				20.7	55	23	32						
	S	4.5													
	S	4.5+		3.5 ft / 652.5 ft											
5	T	32,48		**SHALE;** highly to completely weathered; very soft to soft; light brown, brown 5.0 ft / 651.0 ft											

End of boring at 5.0'

Notes:
-dry during drilling
-dry upon completion

10

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BORING LOG

D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance
CLIENT: Dunaway Associates
PROJECT NUMBER: G20-2194
START DATE: 10/31/2020 FINISH DATE: 10/31/2020
LOGGED BY: Dalton Hubbard (D&S)

LOCATION: Fort Worth, TX
GPS COORDINATES: N32.993865, W97.295917
GROUND ELEVATION: Approx. 656 feet
DRILL METHOD: Cont. Push
DRILLED BY: Jimmy Venegas (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+		**FAT CLAY (CH);** very stiff; dark brown, brown; few calcareous nodules		19.7	53	24	29						
	S	4.5+													
	S	4.5+				16.1									
5	S	4.5+		5.0 ft / 651.0 ft — **SHALE;** highly to completely weathered; very soft; light brown; calcareous		12.5									
	S	4.5+				12.2	35	17	18						
10	S	4.5+		10.0 ft / 646.0 ft											

End of boring at 10.0'

Notes:
-dry during drilling
-dry upon completion

15

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D&S ENGINEERING LABS

BORING LOG

PROJECT: Toliver RV Dealership at Alliance
LOCATION: Fort Worth, TX
CLIENT: Dunaway Associates
GPS COORDINATES: N32.993240, W97.296158
PROJECT NUMBER: G20-2194
GROUND ELEVATION: Approx. 654 feet
START DATE: 10/31/2020 FINISH DATE: 10/31/2020
DRILL METHOD: Cont. Push
LOGGED BY: Dalton Hubbard (D&S)
DRILLED BY: Jimmy Venegas (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; dark brown, brown; trace calcareous nodules		19.6									
	S	4.5+													
	S	4.5+				19.4	49	22	27						
	S	4.5+		4.0 ft											
5	S	4.5+		**SHALE;** highly to completely weathered; very soft; light brown 650.0 ft											
	S	4.5+				11.9	34	16	18						
	T	18,24		7.0 ft											
				End of boring at 7.0' 647.0 ft											

Notes:
-dry during drilling
-dry upon completion

D&S ENGINEERING LABS

SWELL TEST RESULTS

PROJECT: Toliver RV Dealership at Alliance

CLIENT: Dunaway Associates

PROJECT NUMBER: G20-2194

LOCATION: Fort Worth, TX

Boring Number	Depth feet	Initial Moisture Content, %	Final Moisture Content, %	Applied Pressure, psf	Vertical Swell, %
B1	1-2	24.8	32.1	130	4.0
B2	4-5	13.5	22.1	520	11.0
B3	4-5	20.7	26.6	520	3.9
B4	2-3	21.0	31.8	263	10.4
B5	2-3	17.5	25.0	263	3.9
B5	4-5	15.2	25.3	520	6.2

SOLUBLE SULFATE CONTENT RESULTS

TEX 145-E

PROJECT: Toliver RV Dealership at Alliance

PROJECT NUMBER: G20-2194

CLIENT: Dunaway Associates

LOCATION: Fort Worth, TX

Boring Number:	Depth (feet):	Soil Description	Soluble Sulfate Content (ppm)
P1	1-2	**FAT CLAY (CH);** stiff to very stiff; dark brown	100
P3	0-1	**FAT CLAY (CH);** very stiff; dark brown, brown	100

UNCONFINED COMPRESSION TEST

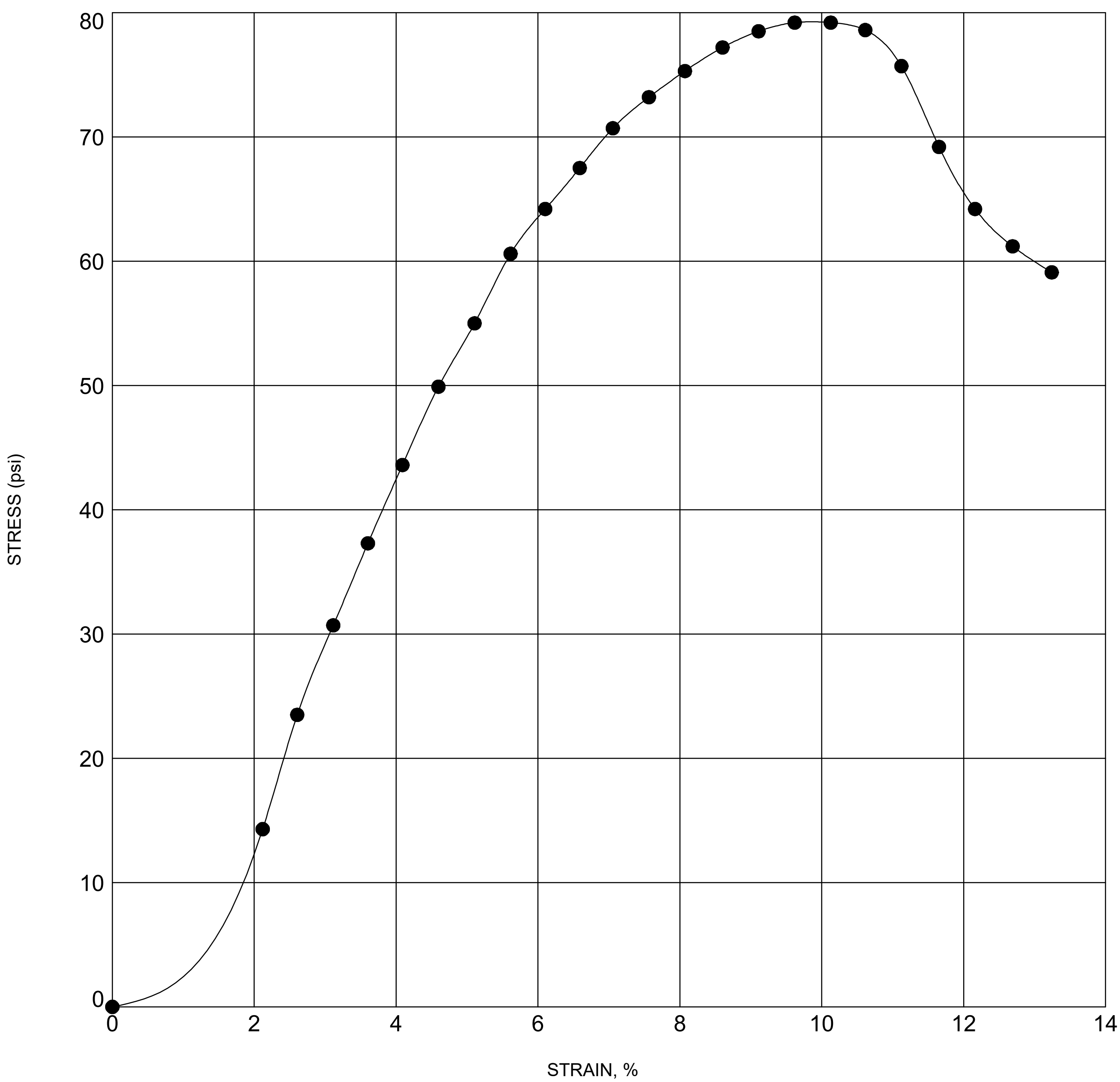
D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance

CLIENT: Dunaway Associates

PROJECT NUMBER: G20-2194

LOCATION: Fort Worth, Tx



	Borehole	Depth	Description	γ_d	MC%
●	**B1**	**14.0**	**SHALE;** slightly to moderately weathered; very soft to soft; dark gray, brown, light brown	**105.5**	**21.7**

UNCONFINED COMPRESSION TEST

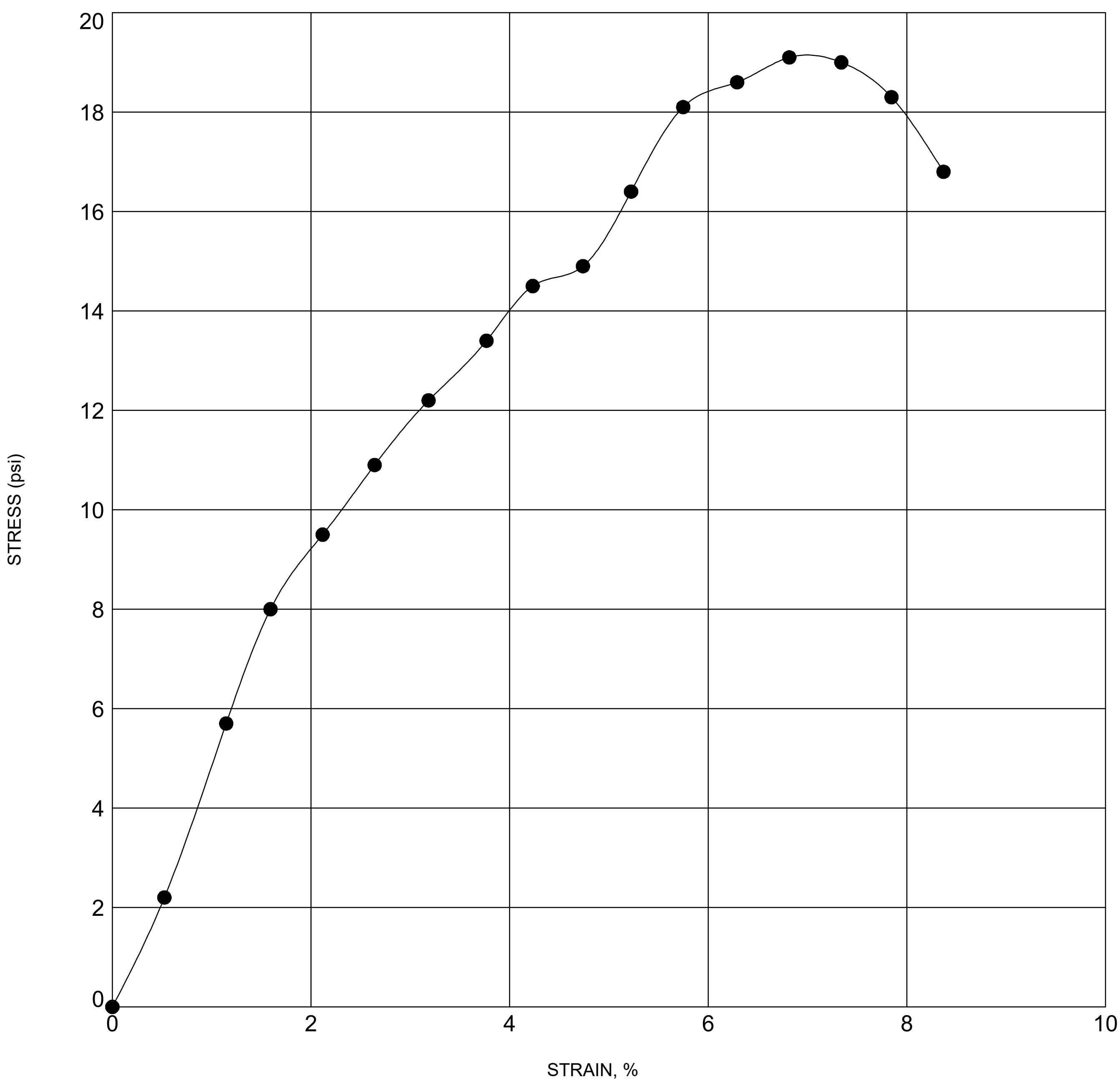
D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance

CLIENT: Dunaway Associates

PROJECT NUMBER: G20-2194

LOCATION: Fort Worth, Tx



	Borehole	Depth	Description	γ_d	MC%
●	**B2**	**9.0**	**SHALE;** highly to completely weathered; very soft; light brown	**100.4**	**22.7**

UNCONFINED COMPRESSION TEST

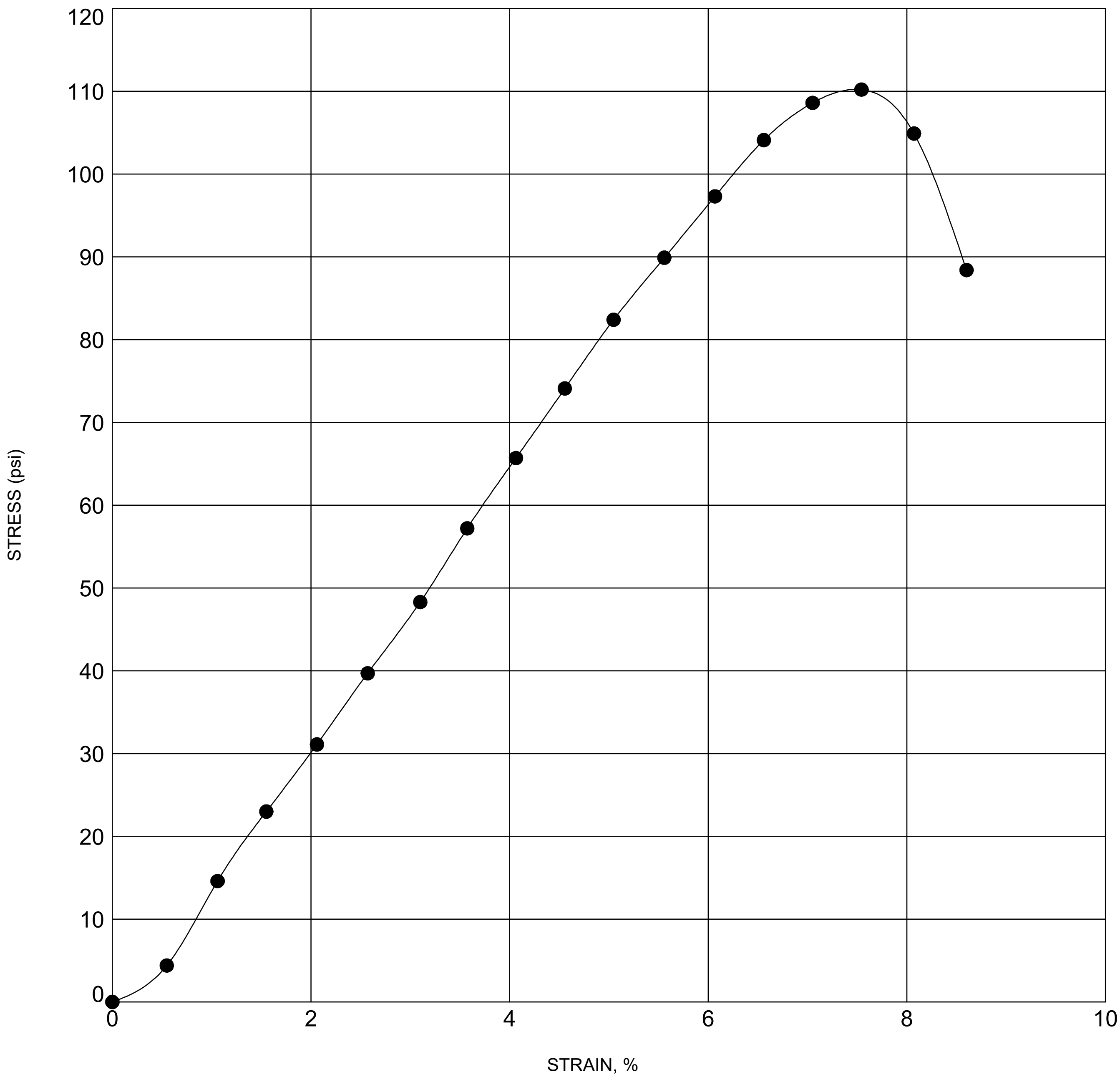
D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance

PROJECT NUMBER: G20-2194

CLIENT: Dunaway Associates

LOCATION: Fort Worth, Tx



Borehole	Depth	Description	γ_d	MC%
● **B2**	**14.0**	**SHALE;** slightly to moderately weathered; very soft to soft; light brown, gray	**114.0**	**15.8**

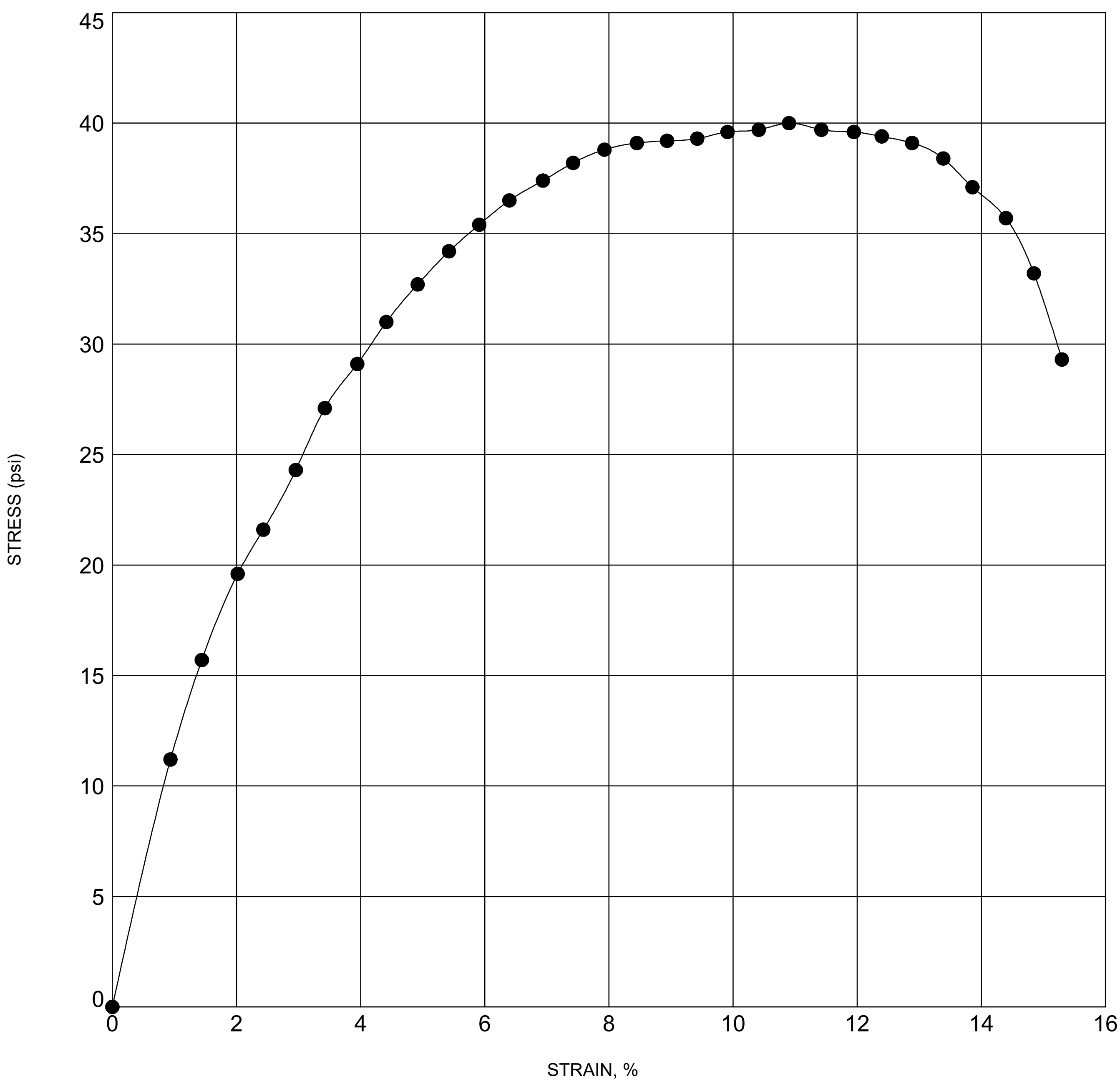
UNCONFINED COMPRESSION TEST

PROJECT: Toliver RV Dealership at Alliance

CLIENT: Dunaway Associates

PROJECT NUMBER: G20-2194

LOCATION: Fort Worth, Tx



Borehole	Depth	Description	γ_d	MC%
● B3	**6.0**	**SHALE;** highly to completely weathered; very soft; light brown	**118.4**	**16.3**

UNCONFINED COMPRESSION TEST

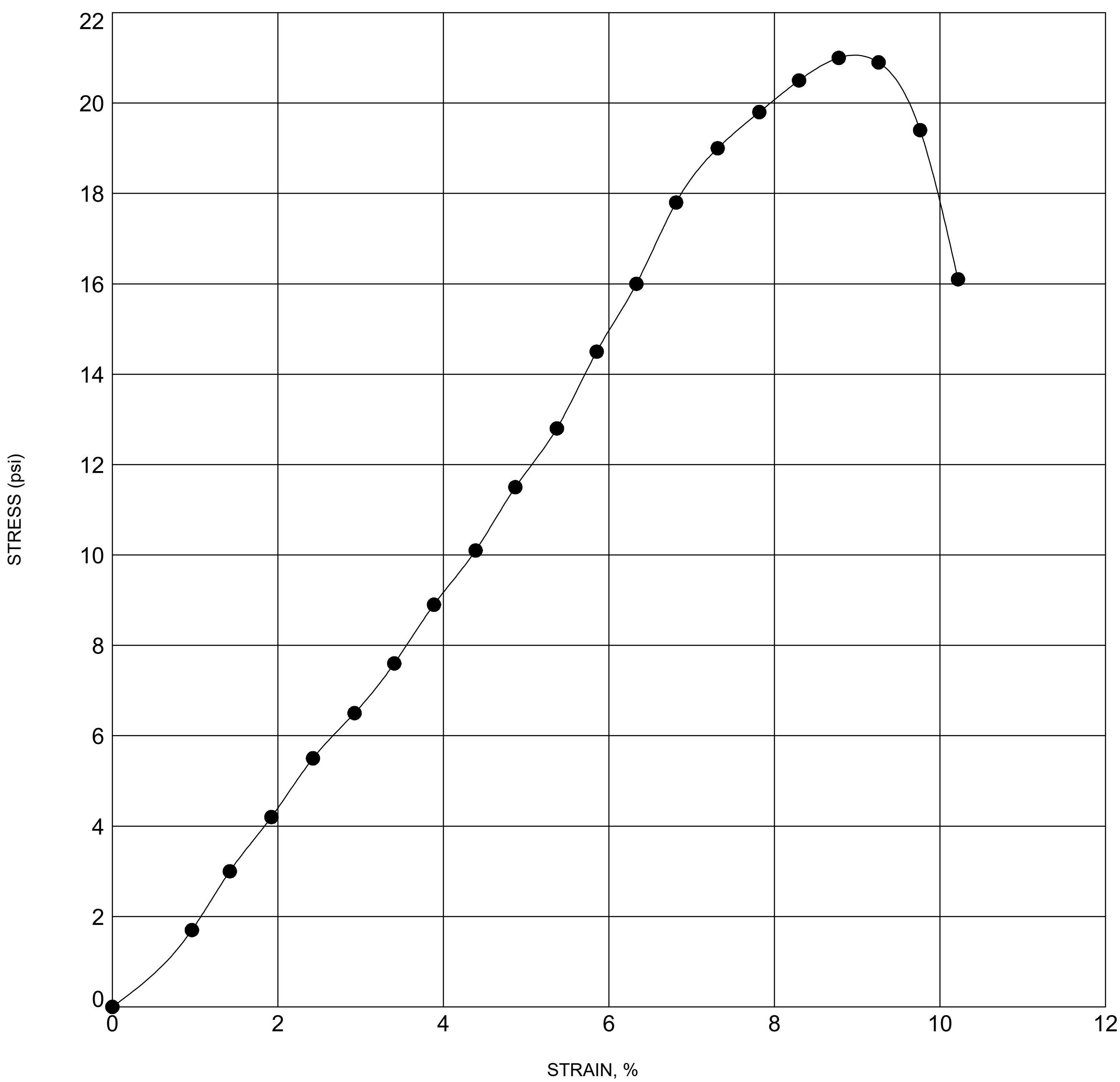
D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance

PROJECT NUMBER: G20-2194

CLIENT: Dunaway Associates

LOCATION: Fort Worth, Tx



	Borehole	Depth	Description	γ_d	MC%
●	**B3**	**14.0**	**SHALE;** slightly to moderately weathered; soft to medium hard; dark gray, light brown	**104.5**	**17.9**

UNCONFINED COMPRESSION TEST

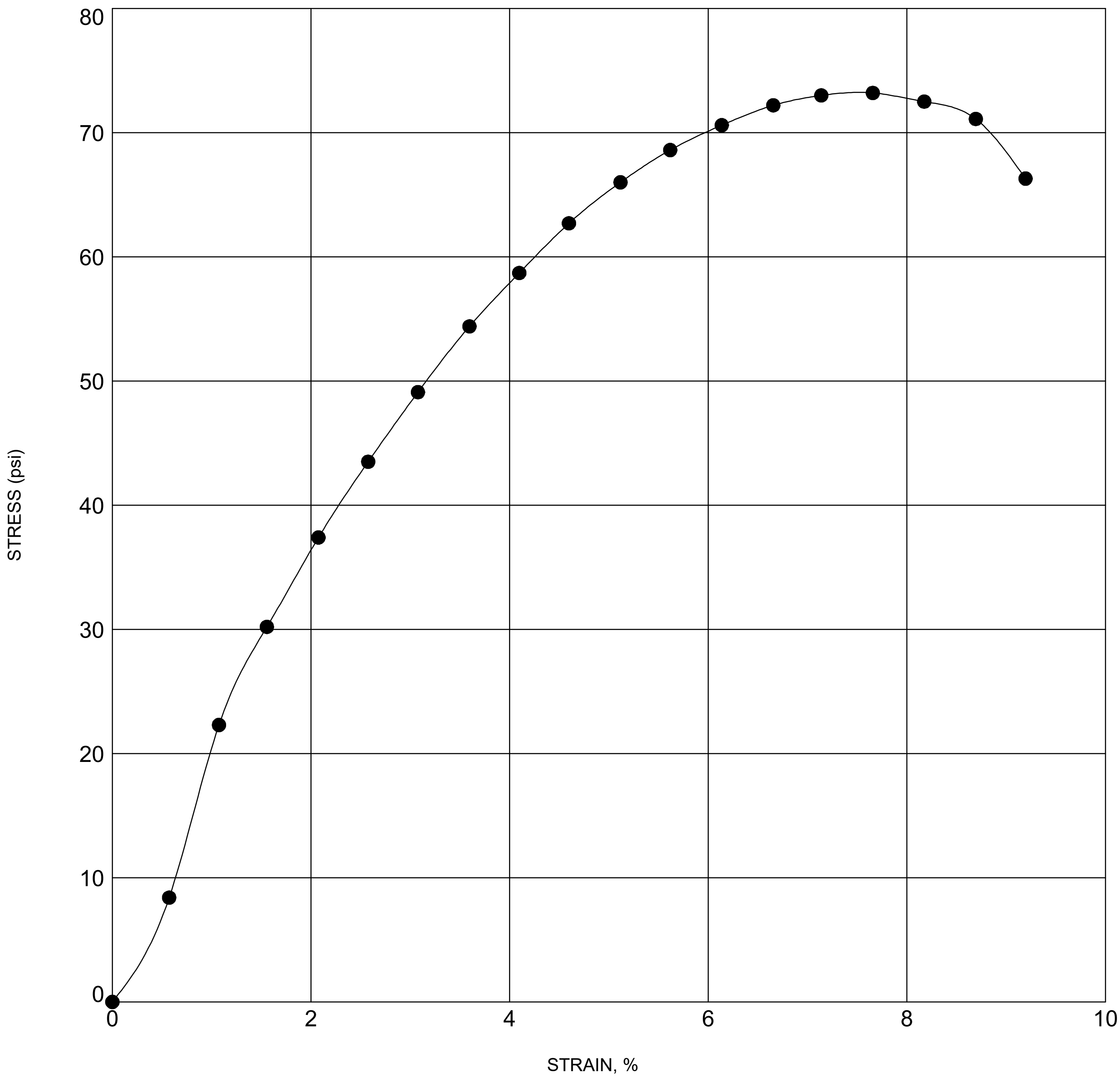
D&S ENGINEERING LABS

PROJECT: Toliver RV Dealership at Alliance

PROJECT NUMBER: G20-2194

CLIENT: Dunaway Associates

LOCATION: Fort Worth, Tx



Borehole	Depth	Description	γ_d	MC%
● B5	14.0	**SHALE;** slightly to moderately weathered; very soft to soft; light brown, dark gray, gray	106.9	17.9

APPENDIX B - GENERAL DESCRIPTION OF PROCEDURES

ANALYTICAL METHODS TO PREDICT MOVEMENT

INDEX PROPERTY AND CLASSIFICATION TESTS

Index property and classification testing is perhaps the most basic, yet fundamental tool available for predicting potential movements of clay soils. Index property 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 index property 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 the relative number 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.

POTENTIAL VERTICAL MOVEMENT

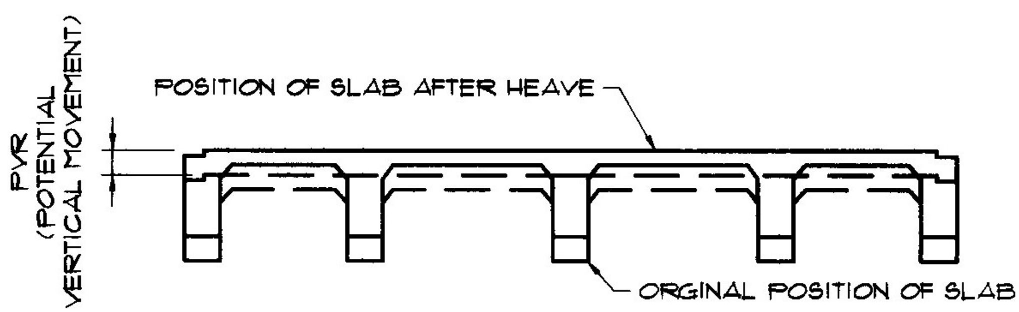
A general index for movement is known as the Potential Vertical Rise (PVR). The actual term PVR refers to the TxDOT Method 124-E mentioned above. For the purpose of this report the term Potential Vertical Movement (PVM) will be used since PVM estimates are derived using TxDOT methods.

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 'liveable'. 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 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.

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 suggests 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 one percent of its depth, particularly when fill depths exceed 10 feet.

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. Similar restraint can occur when pavements are cast directly against rigid bedrock materials. This restriction is called Restraint to Shrinkage (RTS). These RTS cracks do not normally adversely affect the overall performance of foundations or pavements. 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 the 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 with a minimum of one (1) test performed per lift in the building pad area for every 3,00 square feet, with a minimum of three (3) tests per lift, one (1) test per lift per 7,500 square feet in other fill areas, one test per lift in parking areas for every 10,000 square feet, one (1) test lift per 100 linear feet of roadways, drives and sidewalks, and one (1) test lift per 100 linear feet of utility trench backfill. 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. The inspection should be performed prior to and during concrete placement operations. D&S would be pleased to perform these services in support of this project.

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www.dsenglabs.com
Texas Engineering Firm Registration # F-12796
Oklahoma Engineering Firm Certificate of Authorization CA 7181

D&S ENGINEERING LABS