GEOTECHNICAL ENGINEERING STUDY
PROPOSED RESIDENCE AND RETAINING WALL
3600 LANDS END STREET
FORT WORTH, TEXAS

Presented To:

Mr. Wally Burge

September 2021

PROJECT NO. 2875-21-01

CMJ Engineering

September 28, 2021
Report No. 2875-21-01

Mr. Wally Burge
4800 Springwillow Road
Fort Worth, Texas 76109

GEOTECHNICAL ENGINEERING STUDY
PROPOSED RESIDENCE AND
RETAINING WALL
3600 LANDS END STREET
FORT WORTH, TEXAS

Dear Mr. Burge:

Submitted here are the results of a geotechnical engineering study for the referenced project. This study was performed in general accordance with CMJ Proposal 21-8111 dated May 18, 2021. Formal authorization to initiate the geotechnical services was provided by Mr. Wally Burge on May 27, 2021.

Engineering analyses and recommendations are contained in the text section of the report. Results of our field and laboratory services are included in the appendix of the report. We would appreciate the opportunity to be considered for providing the materials engineering and geotechnical observation services during the construction phase of this project.

We appreciate the opportunity to be of service to Mr. Wally Burge and his consultants. Please contact us if you have any questions or if we may be of further service at this time.

Respectfully submitted,
CMJ ENGINEERING, INC.
TEXAS FIRM REGISTRATION NO. F-9177

STATE OF TEXAS
JAMES P. SAPPINGTON, IV
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LICENSED
PROFESSIONAL ENGINEER
9.28.21

James P. Sappington IV, P.E.
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1.0 INTRODUCTION

1.1 Project Description

The site is located at 3600 Lands End Street in Fort Worth, Texas. The project, as currently planned, will consist of a new single-family residence with an approximate footprint of 4,000 square feet or less. The previously existing residence structure has been demolished. In addition, the existing railroad tie retaining wall along the western side of the property atop the existing slope will be replaced and relocated to the west, with an estimated length on the order of 170 feet and heights ranging from less than one foot at the north end to near 10 to 12 feet along the southern portion. A tiered wall system is anticipated. Additional fill will be required to establish final grade behind the planned wall locations. An existing elevated concrete driveway slab structure will remain. This report and recommendations presented herein should be reviewed by CMJ Engineering once structure locations and planned Finished Floor Elevation (s) are established. Plate A.1, Plan of Borings depicts the site area with the approximate locations of exploration borings.

1.2 Purpose and Scope

The purpose of this geotechnical engineering study has been to determine the general subsurface conditions, evaluate the engineering characteristics of the subsurface materials encountered, analyze slope conditions along the southeast boundary, and provide recommendations and geotechnical design parameters for the proposed residential structures.

To accomplish its intended purposes, the study has been conducted in the following phases: (1) drilling sample borings to determine the general subsurface conditions and to obtain samples for testing; (2) performing laboratory tests on appropriate samples to determine pertinent engineering properties of the subsurface materials; and (3) performing engineering analyses, using the field and laboratory data, to develop geotechnical recommendations for the proposed construction.

The design is currently in progress and the locations and/or elevations of the structures could change. Once the final design is near completion (80-percent to 90-percent stage), it is recommended that CMJ Engineering, Inc. be retained to review those portions of the design documents pertaining to the geotechnical recommendations, as a means to determine that our recommendations have been interpreted as intended.

1.3 Report Format

The text of the report is contained in Sections 1 through 11. All plates and large tables are contained in Appendix A. The alpha-numeric plate and table numbers identify the appendix in which they appear. Small tables of less than one page in length may appear in the body of the text and are numbered according to the section in which they occur.

Units used in the report are based on the English system and may include tons per square foot (tsf), kips (1 kip = 1,000 pounds), kips per square foot (ksf), pounds per square foot (psf), pounds per cubic foot (pcf), and pounds per square inch (psi).

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

Subsurface materials at the project site were explored by four (4) borings drilled to depths of 35 to 40 feet in the area of the proposed residence and tiered wall system. The borings were drilled using continuous flight augers at the approximate locations shown on the Plan of Borings, Plate A.1. The boring logs are included on Plates A.4 through A.7 and keys to classifications and symbols used on the logs are provided on Plates A.2 and A.3. Elevations shown on the boring logs are approximate as interpreted from the topographic survey and "Contour Sheet" as provided by the client dated June 2021.

Undisturbed samples of cohesive soils were obtained with nominal 3-inch diameter thin-walled (Shelby) tube samplers at the locations shown on the logs of borings. The Shelby tube sampler consists of a thin-walled steel tube with a sharp cutting edge connected to a head equipped with a ball valve threaded for rod connection. The tube is pushed into the soil by the hydraulic pulldown of the drilling rig. The soil specimens were extruded from the tube in the field, logged, tested for consistency with a hand penetrometer, sealed, and packaged to limit loss of moisture.

The consistency of cohesive soil samples was evaluated in the field using a calibrated hand penetrometer. In this test a 0.25-inch diameter piston is pushed into the relatively undisturbed sample at a constant rate to a depth of 0.25 inch. The results of these tests, in tsf, are tabulated at respective sample depths on the logs. When the capacity of the penetrometer is exceeded, the value is tabulated as 4.5+.

To evaluate the relative density and consistency of the harder formations, a modified version of the Texas Cone Penetration test was performed at selected locations. Texas Department of Transportation (TXDOT) Test Method Tex-132-E specifies driving a 3-inch diameter cone with a 170-pound hammer freely falling 24 inches. This results in 340 foot-pounds of energy for each blow. This method was modified by utilizing a 140-pound hammer freely falling 30 inches. This results in 350 foot-pounds of energy for each hammer blow. In relatively soft materials, the penetrometer cone is driven 1 foot and the number of blows required for each 6-inch penetration is tabulated at respected test depths, as blows per 6 inches on the log. In hard materials (rock or rock-like), the penetrometer cone is driven with the resulting penetrations, in inches, recorded for the first and second 50 blows, a total of 100 blows. The penetration for the total 100 blows is recorded at the respective testing depths on the boring logs.

2.2 Laboratory Testing

Laboratory soil tests were performed on selected representative samples recovered from the borings. In addition to the classification tests (liquid limits, plastic limits, and particle size analyses), moisture content, unconfined compressive strength, and unit weight tests were performed. Results of the laboratory classification tests, moisture content, unconfined compressive strength, and unit weight tests conducted for this project are included on the boring logs. Results of particle size analyses are presented on Plate A.8.

Free swell testing was conducted to establish the general swell characteristics of onsite soils. The test was performed in guiding calculations for potential expansive soil movements at the site. Free swell test results are provided on Plate A.9.

Direct shear tests were performed within the overburden soils to check initial and select residual shear strength. The shear tests were performed in order to obtain strength parameters of the soils in their existing state. The results of the direct shear tests are presented on Plates A.10 through A.13.

The above laboratory tests were performed in general accordance with applicable ASTM procedures, or generally accepted practice.

3.0 SUBSURFACE CONDITIONS

3.1 Site Geology

According to the Dallas Sheet of the Geologic Atlas of Texas, the project site is geologically located in the Duck Creek Formation overlying the Kiamichi Formation of the Lower Cretaceous age. The Duck Creek Formation typically consists of limestone in the lower portion. The Kiamichi Formation is approximately 25 feet thick and is composed of highly active clays. Outcrops of the Kiamichi normally form narrow bands on hillsides. Heavy water seepage can occur along the contact zone between the Duck Creek and Kiamichi Formation during periods of heavy rainfall.

3.2 Soil and Rock Conditions

Specific types and depths of subsurface strata encountered at the boring locations are shown on the boring logs in Appendix A. The generalized subsurface stratigraphy encountered in the borings are discussed below. Note that depths on the borings refer to the depth from the existing grade or ground surface present at the time of the investigation, and the boundaries between the various soil types are approximate.

Fills are present at the surface in all borings consisting of dark brown, brown, and gray silty clays containing limestone fragments, gravel, calcareous nodules, and occasional asphalt fragments, iron stains, shale fragments, and gypsum. Limestone boulders are noted within the fill below 4 feet in Boring B-1. The fill soils encountered in the borings had tested Liquid Limits (LL) of 26 to 44 with Plasticity Indices (PI) of 11 to 25 and are classified as CL by the USCS. The surficial clayey fill soils were generally hard (soil basis) in consistency with pocket penetrometer readings of over 4.5 tsf. Tested unit dry weight values vary from 101 to 119 pcf and tested unconfined compressive strength values vary from 1,020 to 21,090 psf. Lower unconfined compressive strength values reflect the presence of limestone fragments and gravel within the existing fills, indicating higher in-situ strength than the tested value.

Tan limestone was next encountered in Borings B-1 through B-4 at depths of 1 to 7 feet below existing grade. The tan limestone occurs fractured within the upper 5 to 7 feet and contains clay seams throughout. The tan limestone is considered moderately hard to very hard (sedimentary rock basis), with Texas Cone Penetrometer (THD) test values of 1 to 2½ inches of penetration for 100 hammer blows. Gray shale seams were noted below 26 feet within the tan limestone in Boring B-2. Gray limestone was next encountered at a depth of 20 feet in Boring B-4 only. The gray limestone is considered hard, with a THD test value of 1¼ inches of penetration for 100 hammer blows.

Gray shale was next encountered in Borings B-1 through B-4 at depths of 27 to 29 feet below existing grade. The top elevation of this stratum is assumed to be the contact of the Duck Creek and Kiamichi geologic formations, near Elevation 642 to 645. The gray shale contains limestone seams and is considered moderately hard to hard (sedimentary rock basis), with Texas Cone Penetrometer (THD) test values of 1¼ to 2½ inches of penetration for 100 hammer blows. The borings were terminated within the gray shale at depths of 35 to 40 feet.

The Atterberg Limits tests indicate the various clays encountered at this site range from generally slightly active to moderately active with respect to moisture induced volume changes. Active clays can experience volume changes (expansion or contraction) with fluctuations in their moisture content.

3.3 Ground-Water Observations

The borings were drilled using continuous flight augers in order to observe ground-water seepage during drilling. Ground-water seepage was not encountered during drilling and all borings were dry at completion of drilling operations.

Fluctuations of the ground-water level can occur due to seasonal variations in the amount of rainfall; site topography and runoff; hydraulic conductivity of soil strata; and other factors not evident at the time the borings were performed. The possibility of ground-water level fluctuations should be considered when developing the design and construction plans for the project. The possibility exists that perched water may occur atop clays, limestone, or via more permeable strata, particularly after periods of heavy or extended rainfall. In addition, heavy water seepage can occur and are known to exist along the contact zone between the Duck Creek and Kiamichi Formation during periods of heavy rainfall, which typically occurs along hillsides in this geologic setting.

4.0 EXISTING FILLS

Existing fills were encountered to depths of 1 to 7 feet in the borings. Samples of the fills were reasonably dense and free of significant voids. However, in the absence of documented density control, the possibility of undercompacted zones or voids exists. Complete removal and replacement of all the fill is the only method eliminating the risk of unusual settlement where historical documentation of proper fill placement cannot be obtained. Alternative methods are presented below in lieu of removing and replacing all existing fill. These alternatives are presented for the owner's consideration, as the use of these methods will not eliminate the risk of unacceptable movements.

5.0 RESIDENTIAL FOUNDATION RECOMMENDATIONS

5.1 General Foundation Considerations

Two independent design criteria must be satisfied in the selection of the type of foundation to support the proposed residential structure. First, the ultimate bearing capacity, reduced by a sufficient factor of safety, must not be exceeded by the bearing pressure transferred to the foundation soils. Second, due to consolidation or expansion of the underlying soils during the operating life of the structures, total and differential vertical movements must be within tolerable limits. The foundation alternatives for the proposed structures are discussed below.

The moisture induced volume changes associated with the slightly to moderately active clays present at this site indicate that shallow or near surface footings could be subject to differential movements of a potentially detrimental magnitude. The most positive foundation system for the proposed structures would be situated below the zone of most significant seasonal moisture variations. A deep foundation system transferring column loads to a suitable bearing stratum is considered the most positive foundation system. Straight drilled, reinforced concrete shafts penetrating the tan limestone offer a positive foundation system and are recommended. A suspended floor with all underground utilities isolated from expansive soil movement concerns is considered the most positive floor system and may be utilized for the residence.

Consideration can also be given to the use of a monolithic slab-on-grade. Supporting the slab on straight drilled shafts is recommended due to the possible indeterminant settlement of the existing fills. If this option is desired with the slab-on-grade foundation, it is recommended that the pier steel be vertical and sheathed to allow freedom of movement of the slab. The key to success of slab-on-grade construction is proper design/construction, and providing the most optimum conditions for reduced slab movements. Providing excellent drainage away from the structures, preventing ponding of water aside the slabs, preventing excess drying of soils, and using onsite soil backfill to prevent water intrusion into utility line backfill will enhance slab performance. Recommendations these foundation systems are presented below.

5.2 Potential Vertical Movements

The fill soils encountered at the site are variable and range from slightly to moderately expansive, in addition to the variable depth and presence of rock. Analyses indicate that the potential vertical movements of onsite soils due to their expansive characteristics are on the order of less than 1 inch to on the order of 1½ inches. The greatest movements will occur where the greater thicknesses of dryer, more highly plastic clays are present. The actual amount of movement will depend greatly on the moisture content of the soils prior to construction. In other words, where a ground-supported floor slab is placed upon moist soils, the future expansive soil movement of these soils will be limited since these soils exist in a pre-swelled state, and additional moisture will not cause significant additional heaving of the soils. Conversely, when onsite soils are extremely dry, moisture will cause significant swelling of these soils.

5.3 Drilled Piers

5.3.1 Design Parameters – Straight Drilled Shafts

Recommendations and parameters for the design of cast-in-place straight-shaft drilled piers are outlined below. Specific recommendations for the construction and installation of the drilled piers are included in the following section and shall be followed during construction.

Bearing Stratum	Tan LIMESTONE, with clay seams
Depth of Bearing Stratum:	Approximately 8 to 13 feet below existing grades
Required Penetration/Depth:	All piers should penetrate into the bearing stratum a minimum of 3 feet, or to a minimum depth of 16 feet below finished grade, whichever is deeper.
Allowable End Bearing Capacity:	16,000 psf
Allowable Skin Friction:	Applicable below a minimum penetration of 3 feet into tan limestone and below any temporary casing; 2,500 psf for compressive loads and 1,900 psf for tensile loads.

Drilled shafts should extend through any fractured zones and clay seams/layers and bear only in the intact tan limestone. Due to the presence and possible caving conditions of the overlying fills, temporary casing may be required for proper shaft installation. The above values contain a safety factor of three (3). A minimum pier diameter of 18 inches is recommended.

In order to develop full load carrying capacity in skin friction, adjacent shafts should have a minimum center-to-center spacing of 3 times the diameter of the larger shaft. Closer spacing may require some reductions in skin friction and/or changes in installation sequences. Closely spaced shafts should be examined on a case-by-case basis. As a general guide, the design skin friction will vary linearly from the full value at a spacing of 3 diameters to 50 percent of the design value at 1 diameter.

Settlements for properly installed and constructed straight shafts in the tan limestone will be primarily elastic and are estimated to be less than 1 inch.

5.3.2 Soil Induced Uplift Loads

The drilled shafts could experience tensile loads as a result of post construction heave in the site soils. The magnitude of these loads varies with the shaft diameter, soil parameters, and particularly the in-situ moisture levels at the time of construction. In order to aid in the structural design of the reinforcement, the reinforcement quantity should be adequate to resist tensile forces based on soil adhesion equal to 1,450 psf acting over the upper 7 feet of the pier shaft. This load must be resisted by the dead load on the shaft, continuous vertical reinforcing steel in the shaft, and a shaft adhesion developed within the bearing strata as previously discussed. In order to aid in the structural design of the reinforcement, minimum reinforcing should be equal to 0.5 percent of the shaft area.

5.3.3 Drilled Shaft Construction Considerations

Drilled pier construction should be monitored by a representative of the geotechnical engineer to observe, among other things, the following items:

- Identification of bearing material
- Adequate penetration of the shaft excavation into the bearing layer
- The base and sides of the shaft excavation are clean of loose cuttings
- If seepage is encountered, whether it is of sufficient amount to require the use of temporary steel casing. If casing is needed it is important that the field representative observe that a high head of plastic concrete is maintained within the casing at all times during their extraction to prevent the inflow of water.

Caving soils may be encountered during installation of select straight shafts and may require the use of temporary casing for installation of select straight shafts. The casing should be seated in the bearing stratum with all water and most loose material removed prior to beginning the design penetration. Care must then be taken that a sufficient head of plastic concrete is maintained within the casing during extraction.

Shaft excavations should be maintained in the dry. Precautions should be taken during the placement of reinforcing steel and concrete to prevent loose, excavated soil from falling into the excavation. Concrete should be placed as soon as practical after completion of the drilling, cleaning, and observation. Excavation for a drilled pier should be filled with concrete before the end of the workday, or sooner if required to prevent deterioration of the bearing material. Prolonged exposure or inundation of the bearing surface with water will result in changes in strength and compressibility characteristics. If delays occur, the drilled pier excavation should be deepened as necessary and cleaned, in order to provide a fresh bearing surface.

The concrete should have a slump of 6 inches plus or minus 1 inch. The concrete should be placed in a manner to prevent the concrete from striking the reinforcing cage or the sides of the excavation. Concrete should be tremied to the bottom of the excavation to control the maximum free fall of the plastic concrete to less than 10 feet, or funneled between reinforcing steel to prevent concrete segregation.

It should be anticipated that very hard limestone layers and limestone boulders within the existing fill will be encountered during straight drilled shaft installation. A drilling rig of sufficient size and weight will be necessary for drilling and/or coring through the very dense layers to reach the desired bearing stratum, maintain plumbness requirements, and achieve the required penetration.

In addition to the above guidelines, the specifications from the Association of Drilled Shaft Contractors Inc. "Standards and Specifications for the Foundation Drilling Industry" as Revised 1999 or other recognized specifications for proper installation of drilled shaft foundation systems should be followed.

5.3.4 Grade Beams (for Structurally Suspended Floor System)

All grade beams should be supported by the drilled shafts. Grade beams used in conjunction with the drilled piers should be tied into the tops of the piers and should have a minimum 6-inch void space to prevent contact with the swelling clay soils. This void will serve to minimize distress resulting from swell pressures generated by the clays.

Grade beams may be cast on cardboard carton forms or formed above grade. If cardboard carton forms are used, care should be taken to not crush the carton forms, or allow the carton forms to become wet prior to or during concrete placement operations. A soil retainer should be provided to help prevent in-filling of this void.

Backfill against the exterior face of grade beams or panels should be properly compacted on-site clays. Compaction should be a minimum of 93 percent of ASTM D 698, at a minimum of 2 percentage points above the optimum moisture content determined by that test. This clay fill is intended to reduce surface water infiltration beneath the structure.

5.3.5 Structurally Suspended Floor Slab

The most positive method of preventing slab distress due to swelling soils and differential soil movement is to structurally suspend the interior slab. Support of the structural floor is provided by the drilled piers. Due to the expansion potential of the site clays, it is recommended that the suspended floor slab be constructed on carton forms with a minimum 6-inch void space or crawl space.

Care should be taken to assure that the void boxes are not allowed to become wet or crushed prior to or during concrete placement and finishing operations. Corrugated steel, placed on the top of the carton forms, could be used to reduce the risk of crushing of the carton forms during concrete placement and finishing operations. As a quality control measure during construction, "actual" concrete quantities placed should be checked against "anticipated" quantities. Significant concrete "overage" would be an early indication of a collapsed void.

Provision should be made to provide drainage of the crawl space below the slab, in the event water becomes trapped or seeps into this area. Drain inlets which are tied into the storm sewer or a sump and pump system may be necessary. Also, because of capillary moisture buildup, proper ventilation should be provided in the crawl space below the slab. Ventilation of the void below the floors should be provided if high humidity can cause problems with floor tile adhesives.

Vehicle or pedestrian ramps leading up to the building should be structurally connected to the building grade beams to avoid abrupt differential movement between the building slab and the ramps. Transitioning details will be required at the points where ramps connect with paving and slab on grade elements. In addition, ramp slabs should be constructed so that slopes sufficient for effective drainage of surface water are still provided after potential differential movements.

5.4 Stiffened, Monolithic Slab-on-Grade Option

5.4.1 Design Parameters

A stiffened, monolithically placed slab-on-grade foundation, either rebar or post-tensioned, used at this site must be designed with exterior and interior grade beams to provide sufficient rigidity to tolerate the differential soil movements. These differential movements typically will occur between the periphery and interior of the slab-on-grade system. Foundation movements are anticipated to occur primarily due to post construction heave of the underlying soils but also can occur due to shrinkage of the clays around the perimeter of the slab. With the presence of the existing fills and possible indeterminate magnitude of settlement, pier support is recommended. Steel dowels used to connect the slab to the piers should be sheathed and vertical to allow upward slab movement. It is recommended that all fill soils be properly placed and compacted in accordance with this report section and Section 8.0, Earthwork prior to foundation installation.

Slab-on-grade construction only should be considered if slab movement can be tolerated. The owner must fully understand that if the floor slab is placed on-grade, some movement and resultant cracking within the floor and interior wall partitions may occur. This upward slab movement and cracking usually is difficult and costly to repair, and may require continued maintenance expense.

Site grading will affect potential movements. For example, fills using higher plasticity clays will increase potential movements. Once the Finished Floor Elevation is established, this office should be contacted to review and evaluate the effects of site grading on the potential moisture induced movements. Non-expansive select fill with a Liquid Limit less than 35 and a Plasticity Index (PI) between 5 and 16 may be utilized as fill beneath the building slab without affecting the design parameters below. The select fill should be compacted in maximum 9-inch loose lifts at minus 2 to plus 3 percentage points of the soil's optimum moisture content at a minimum of 95 percent of Standard Proctor density (ASTM D 698).

The foundation should be designed by a structural engineer familiar with stiffened slabs-on-grade subject to differential movement. Design parameters are presented below for PVR and differential swell using the Post-Tensioning Institute's (PTI) slab-on-grade design method, 3rd Edition.

Slab Design Parameters

Design PVR:	1½ inches
Edge Moisture Variation	
Approximate Center Lift:	9.0 feet
Approximate Edge Lift:	4.6 feet
Differential Swell	
Approximate Center Lift:	1.2 inches
Approximate Edge Lift:	1.7 inches

Beams may be designed based on an allowable soil bearing pressure of 2,000 pounds per square foot or less within the shallow soils or tan limestone. The beams should extend at least 12 inches into undisturbed soil or compacted and tested fill. The beam depth is given in regard to bearing capacity and is not intended to be a structural recommendation.

It should be recognized that a post tensioned or conventionally reinforced slab-on-grade foundation system placed at this site will be subject to differential movements as indicated above. If slab stiffness is not sufficient to resist the ground movements, these movements can cause cracking of interior sheet rock walls and exterior brick walls. Poor drainage, water leaks, free water sources, long-term percolation in recessed planter areas and/or trees can result in greater differential movements. For example, should leaks develop in underground water or sewer lines or the grades around the structures are changed and cause ponding of water, unacceptable slab movements could develop.

A properly engineered and constructed vapor barrier should be provided beneath slabs-on-grade which will be carpeted or receive moisture sensitive coverings or adhesives.

6.0 EXPANSIVE SOIL CONSIDERATIONS

6.1 Potential Vertical Movements

The soils encountered at this site can shrink and swell as the soil moisture content fluctuates during seasonal wet and dry cycles. Additionally, the site environment is impacted by grading and drainage, landscaping, ground-water conditions, paving and many other factors which affect the structure during and after construction. Therefore, the amount of soil movement is difficult to determine due to the many unpredictable variables involved.

The estimated soil movements are based on the subsurface conditions revealed by the borings and on seasonal moisture fluctuations. Soil movements, significantly larger than estimated, could occur due to inadequate site grading, poor drainage, ponding of rainfall, and/or leaking pipelines.

6.2 Site Drainage

An important feature of the project is to provide positive drainage away from the proposed buildings. If water is permitted to stand next to or below the structures, excessive soil movements (heave) can occur. This could result in differential floor slab or foundation movement.

A well-designed site drainage plan is of utmost importance and surface drainage should be provided during construction and maintained throughout the life of the structures. Consideration should be given to the design and location of gutter downspouts, planting areas, or other features which would produce moisture concentration adjacent to or beneath the structure or paving. Joints next to the structure should be sealed with a flexible joint sealer to prevent infiltration of surface water. Proper maintenance should include periodic inspection for open joints and cracks and resealing as necessary.

Rainwater collected by the gutter system should be transported by pipe to a storm drain or to a paved area. If downspouts discharge next to the structure onto flatwork or paved areas, the area should be watertight in order to eliminate infiltration next to the building.

6.3 Additional Design Considerations

The following information has been assimilated after examination of numerous projects constructed in active soils throughout the area. It is presented here for your convenience. If these features are incorporated in the overall design of the project, the performance of the structures should be improved.

- Every attempt should be made to limit the extreme wetting or drying of the subsurface soils since swelling and shrinkage will result, therefore causing extreme suction conditions. Standard construction practices of providing good surface water drainage should be used. A positive slope of the ground away from the foundation should be provided to carry off the run-off water both during and after construction.

- Provide irrigation systems away from the edge of the house such that any leakage of said systems will not cause undue localized moisture gain below the slab.

- Always provide positive drainage away from all sides of the foundation to prevent zones of ponded water adjacent to the slab.

- Rainfall is recommended to be collected by gutters and downspouts and transmitted well away from the structure to prevent water from entering the building subgrade adjacent to the slab.

- Homeowners should be educated in providing a uniform moisture condition adjacent to the edge of their foundation. This involves sprinkling/watering their foundation during the dry, hot summers and providing good drainage of excess water away from the foundation during the wet periods of the year.

- Sidewalks should not be structurally connected to the building. They should be sloped away from the building so that water will drain away from the structure.

- Backfill for utility lines or along the perimeter beams should consist of on-site material so that they will be stable. If the backfill is too dense or too dry, swelling may form a mound along the ditch line. If the backfill is too loose or too wet, settlement may form a sink along the ditch line. Either case is undesirable since several inches of movement is possible and floor cracks are likely to result. The soils should be processed using the previously discussed compaction criteria.

- All utility lines should be properly compacted and it is recommended that a clay plug be established at the edge of the building line leading out a horizontal distance of 5 feet. The purpose of this clay is to prevent excess water in a utility ditch backfill from entering the foundation and causing non-uniform movements.

- A floor slab placed at or below existing grade should be provided with a moisture barrier to prevent wet spots from penetrating through the pervious concrete slab.

- Trees and deep rooted shrubs should not be used as landscaping around the structure perimeter as the root systems can lead to desiccation of the subgrade soils. Any existing trees or trees to be planted should be at a minimum distance from the building such that the building will not fall within the drip line of the mature plants (usually one to one-and-one-half times the mature height of the tree)

- Leave outs around the perimeter of the slab foundation should be backfilled in loose lifts not exceeding 6 to 8 inches, moistened as required, and compacted and tested to ensure a tight fill of onsite soils exist around the building perimeter.

- Provide a uniform moisture condition within a 5-foot zone around the house perimeter to help maintain a more uniform soil moisture content and resulting reduced differential soil heaving movement below the slab

- Provide swimming pools or ancillary structures well away from the house edge and follow similar procedures of excellent drainage, proper backfilling of utilities, and regular maintenance to further prevent unusual or excess moisture gains or loss adjacent to the slab foundation.

- Utility line details and fixtures must consider the potential for differential movement beneath any piping. In conjunction with a structural slab all underground utility lines should be isolated from expansive clays. A similar 6-inch void is recommended between the utility bottom and underlying clay soils. This prevents the utility lines from uplifting into the suspended slab.

7.0 SLOPE STABILITY ANALYSES

7.1 General Comments

The results of Borings B-1 through B-4 indicate that the contact elevation between Duck Creek and Kiamichi geologic formations at this site is near approximately Elevation 642 to 645. This agrees with visual observations of the limestone outcropping along the slope at the subject property. In general, the clays and shaly clays of the Kiamichi formation which are known locally to be associated with slope instability problems occur at an elevation approximately 13 to 16 feet below the toe of the subject slope, and below hard to very hard tan limestone rock.

The following text identifies the engineering analyses conducted for slope stability. In slope stability analyses, a factor of safety of 1.5 corresponds to generally accepted engineering practices in the Dallas-Fort Worth metroplex area for the long-term stability of a slope. Readers also will note that a safety factor of 1.0 implies impending failure. Furthermore, a safety factor of 1.5 implies the resisting forces to failure are 1.5 times the forces tending to cause failure.

No visual evidence of on-going slope instability problems along this slope were noted by the writer at the time of this investigation. The slope angle of this particular hillside increases with elevation, from the on the order of 5H:1V near the toe along the existing drainage swale and on the order of 1.5H: 1V nearer the crest. A system of tiered retaining walls is planned in association with the new residence. A primary purpose of this investigation is the effect of overall global slope stability as a result of the tiered wall installation and construction of the planned residence.

7.2 Slope Stability Analysis Computer Solutions

CMJ Engineering, Inc. selected GEOSTASE to perform the slope stability analyses for this project. GEOSTASE software is a recent update from GSTABL7, an off shoot based on the original PCSTABL6-1986 developed at Purdue University. It is a two-dimensional, limit equilibrium slope stability program developed and enhanced by Garry H. Gregory, P.E. and Harold W. VanAller, P.E. CMJ Engineering, Inc. utilized GEOSTASE, Version 4.30.24.

This slope stability analysis utilizes Modified Bishop, Simplified Janbu, or the Spencer Method of Slices for analysis. Circular, random, and sliding block search routines are available for analysis.

Analysis also allows the utilization of anisotropic soil strength parameters which aid in modeling tension cracks of bedding planes as well as different soil strength in different directions. The system overall allows analyses of hundreds of search options and potential failure surfaces and results in a print out showing the geometry, soil parameter summary, and listing of the ten most critical failure surfaces analyzed, focusing and highlighting the most critical surface with the lowest safety factor.

7.3 Input Parameters

Slope stability analyses were performed to simulate circular-type failure planes. One general slope cross section was considered appropriate to simulate the slope and planned tiered retaining wall system, based on the referenced provided topographic and preliminary site plan information, near cross section B-B as depicted on the referenced "Contour Sheet."

The selected geometry to simulate slope conditions has design parameters as identified on Plates B.1 and B.2 in the top of the graph. Five soil zones consist of the following:

- Soil 1 – Upper Silty Clay Fill Soil - hard cohesive soil conditions containing limestone fragments and gravel, exhibiting relatively low to moderate strength with a cohesion of 300 psf, and selected to a friction angle of 24 degrees.
- Soils 2 and 3 – Tan limestone zones, exhibiting low to moderate rock strength reflecting the presence of fractured, clay seams and clay layers in the upper zone.
- Soil 4 – Gray shale, possessing increased strength parameters over Soils 2 and 3 (Kiamichi shale).
- Soil 5 – Wall backfill soils, with an assumed cohesion of 200 psf, and selected to a friction angle of 30 degrees.

Numerous analyses were conducted by CMJ Engineering, Inc. to identify the worst-case methodology to use in analysis as well as the appropriate soil parameters, which affect the slope stability. Slope stability analyses were checked using both circular-type and wedge-type failure conditions. The assumed soil properties utilized for analysis are denoted in the table in the upper left on Plates B.1 and B.2. The soil type color below each profile line is denoted with a color legend that corresponds to the table in the upper left. The table lists the assumed unit weight and strength properties for each soil type.

7.4 Analyses

7.4.1 Proposed Slope Geometry

Based on the referenced provided preliminary site plan and "Contour Sheet", primarily a two-tiered wall system is planned west of the residence in order to establish final pad grade and Finished Floor Elevation. The tops of the primary two-tiered wall tier correspond to Elevations 668 and 671 at the selected cross section. Finished Floor for the residence is planned near Elevation 672, and is conservatively modelled in the analysis as 300 psf, denoted as DL1 and DL2 on Plates B.1 and B.2. The following describes the failure planes demonstrated on the slope stability analyses contained in Plates B.1 and B.2:

> Plate B.1 – Simulates the proposed slope and tiered wall geometry west of the residence with a 2-tiered wall system, soil conditions as identified on Plate B.1; analyses utilized circular arc type failure with Spencer's Method; results in a factor of safety of 3.4.
>
> Plate B.2 – Simulates the proposed slope and tiered wall geometry west of the residence with a 2-tiered wall system, soil conditions as identified on Plate B.1; analyses utilized wedge type failure with Spencer's Method; results in a factor of safety of 3.3.

Readers should understand that a factor of safety of 1.0 implies impending failure and a factor of safety of less than 1.0 implies the slope would fail based on the input parameters. Common practice in the geotechnical industry requires that a long-term safety factor on the order of 1.5 or greater be established for the safety of a slope. This essentially means that the resisting forces to sliding will be 50 percent greater that the driving forces for long-term conditions. As seen from the analysis on Plates B.1 and B.2, the global slope stability of the existing slope to include the assumed surcharge load is satisfactory and in excess of required standard of 1.5 based on the available data. No external slope stabilizing remediation or structures are necessary for global stability at this time based on the presently available data. These analyses are not intended to analyze the occurrence of localized failures within the slope or minor slope creep which can periodically occur.

7.5 Additional Slope Related Comments

With the proposed wall system, minor erosion of surface soils and related maintenance should be anticipated. If weather patterns of considerably long periods of extended rainfall occur, the surface soils will tend to moisten, soften to a certain degree, and be more susceptible to surface sloughing as their relative strength decreases, particularly if surface vegetation is not established. Surface run-off from heavy rainfall may also cause surface erosion of soils and/or surface rills with associated progressive erosion along the slope if left unvegetated or if and where concentrated surface flows develop. The proper design of surface drainage with appropriate conveying of storm-water to suitable outlets or conveyance structures should be provided, and proper re-vegetation of the slope should be established.

The analyses generally considered global-type failure planes. Near surface skin slides also can occur in isolated locations and are considered minor and relate more closely to erosion and surface soil creep. Proper erosion control measures should alleviate potential surface slides or creep. All earthwork related to this effort should be performed in accordance with Section 9, Earthwork.

It is imperative that trained field personnel be vigilant to observing anomalies that may occur in the subsurface conditions during the excavation and placement of the proposed retaining walls. Any anomalous conditions should be brought to the attention of this office and the engineers on this project for evaluation and potential remediation, as necessary. Onsite materials are known to vary in soil type and consistency. Isolated zones of loosely packed or soft materials can exist and their potential should be carefully observed by trained construction personnel.

8.0 RETAINING WALLS

8.1 General Retaining Wall Considerations

Five geotechnical design criteria must be satisfied in the selection of the type and configuration of the retaining walls. These criteria are; the wall must have an acceptable factor of safety with respect to (1) overturning failure, (2) a sliding (translation) failure, (3) a bearing capacity failure, and (4) a global (deep-seated) slope failure. In addition, (5) the deformation of the wall caused by deflection from earth pressure, and from settlement or heave of the foundation soils or backfill soils, must be within tolerable limits during the functional life of the structures.

8.2 Retaining Wall Foundations

8.2.1 Design Criteria

A deep foundation system transferring loads to a suitable bearing stratum is considered the most positive foundation system. However, if differential movements are acceptable, the retaining wall foundations can be supported on continuous footings. Based on the provided preliminary topographic information, the lower wall tier is anticipated to be situated in the natural fractured tan limestone, while the upper tier would be founded within the existing silty clay fills containing limestone fragments and gravel. A deep foundation system must be used if the retaining walls are sensitive to movements. Recommendations for a deep foundation system should follow the recommendations previously presented in Section 4.3.

Footings for the lower tier wall situated a minimum of 2 feet below finished grade situated within the fractured tan limestone may be proportioned using a maximum allowable bearing pressure of 4,000 psf. For the upper tier wall, foundations may be designed for an allowable bearing pressure of 1,200 psf.

Soils existing in a soft to firm state should be evaluated on a case-by-case basis. Close inspection of soils strength should be conducted by a geotechnical engineer to allow designation and removal of very soft soils not meeting the bearing capacity stated above. It should be noted that retaining wall foundations are typically subjected to non-uniform pressure across the foundation, and possibly negative pressure (separation of foundation from soil) under a portion of the foundation, due to the overturning moment induced by the lateral earth pressures. The allowable foundation pressures given above are for the maximum pressure induced by the foundation loads, and not the average pressure under the foundation base.

The horizontal bases of the footings will develop resistance to sliding by means of a combination of friction and adhesion (for cohesive foundation materials). Given the nature of the foundation materials, an adhesion of 500 psf may be used for earth formed footings. An ultimate friction factor of 0.35 may be used to calculate sliding resistance of the footings bearing on site soils. Only long-term dead loads should be considered in calculating the available friction on the foundation base.

The vertical earth-formed sides of keyways (only below the lowest wall tier) will resist lateral forces by developing passive earth pressures. A passive lateral earth pressure coefficient of 3.0 should be used for passive resistance calculations where passive resistance is developed against a vertical earth-formed side of a keyway, based on a soil unit weight of 120 pcf, per foot of footing height. Passive earth pressures on the toe of wall foundations, keys or similar structural members should be considered for counteracting lateral forces only if the member is placed in direct contact with the fractured tan limestone in a "neat cut" excavation. If the foundation is constructed by using forms, lean concrete may be placed between the footing and the undisturbed wall of the adjacent excavation (after removal of the forms) in order to provide the direct contact required to consider passive pressure for counteracting lateral movement. The lean concrete should have a minimum 28-day compressive strength of 1,500 psi.

The base of all excavated footings should be inspected by a geotechnical engineer or geotechnician under his or her supervision to assure that the bottom is firm, level and free of loose soil material and/or debris. In the areas of existing fills, the exposed subgrade in the footing excavation should be proofrolled as follows: the subgrade should be proof rolled using a heavy pneumatic tired or small width drum roller making several passes over the subgrade. Any soft or spongy areas should be overexcavated to firm materials and backfilled following the recommendations provided in report Section 9, Earthwork. The proof rolling operations should also be observed by the project geotechnical engineer or his/her representative.

Foundations for the retaining walls designed in accordance with these recommendations will have a minimum factor of safety of 3 with respect to a bearing capacity failure, and should experience a total settlement of 1 inch or less and a differential settlement of ½ inch or less, after construction.

8.2.2 Foundation Construction

Mat type or spread foundation construction should be monitored by a representative of the geotechnical engineer to observe, among other things, the following items:

- Identification of bearing material
- Adequate penetration of the foundation excavation into the bearing layer
- The base and sides of the excavation are clean of loose cuttings
- When seepage is encountered, whether it is sufficient amount to require the use of excavation dewatering methods

Precautions should be taken during the placement of reinforcing steel and concrete to prevent loose, excavated soil from falling into the excavation. Concrete should be placed as soon as practical after completion of the excavating, cleaning, reinforcing steel placement and observation. Excavation for a shallow foundation should be filled with concrete before the end of the workday, or sooner if required, to prevent deterioration of the bearing material. Prolonged exposure or inundation of the bearing surface with water will result in changes in strength and compressibility characteristics. If delays occur, the excavation should be deepened as necessary and cleaned, in order to provide a fresh bearing surface. If more than 24 hours of exposure of the bearing surface is anticipated in the excavations, a "mud slab" should be used to protect the bearing surfaces. If a mud slab is used, the foundation excavations should initially be over-excavated by approximately 4 inches and a lean concrete mud slab of approximately 4 inches in thickness should be placed in the bottom of the excavations immediately following exposure of the bearing surface by excavation. The mud slab will protect the bearing surface, maintain more uniform moisture in the subgrade, facilitate dewatering of excavations if required, and provide a working surface for the placement of formwork and reinforcing steel.

The concrete should be placed in a manner that will prevent the concrete from striking the reinforcing steel or the sides of the excavation in a manner that would cause segregation of the concrete.

8.3 Lateral Earth Pressure

8.3.1 Equivalent Fluid Pressures

Lateral earth pressures on retaining walls will depend on a variety of factors, including the type of soils behind the wall, the condition of the soils, and the drainage conditions behind the wall. Recommended lateral earth pressures expressed as equivalent fluid pressures, per foot of wall height, are presented in Table 8.3.1-for a wall with a level backfill behind the top of the wall. The equivalent fluid pressure for an undrained condition should be used if a drainage system is not present to remove water trapped in the backfill and behind the wall. Pressures are provided for at-rest and active earth pressure conditions. In order to allow for an active condition, the top of the wall(s) must deflect on the order of 0.4 percent.

TABLE 8.3.1-1
Equivalent Fluid Pressures – Level Backfill

Backfill Material	At-Rest Equivalent Fluid Pressure (pcf)		Active Equivalent Fluid Pressure (pcf)	
	Drained	Undrained	Drained	Undrained
On-site clay or similar clay fill material	100	110	85	100
Select fill, flowable fill, or on-site soils meeting select material specifications	65	90	50	85
Free draining granular backfill material	50	90	35	80

For the select fill or free-draining granular backfill, these values assume that a "full" wedge of the material is present behind the wall. The wedge is defined where the wall backfill limits extend outward at least 2 feet from the base of the wall and then upward on a 1H:2V slope. For narrower backfill widths of granular or select fill soils, the equivalent fluid pressures for the on-site soils should be used.

8.4 Wall Backfill Material Requirements

Granular Wall Backfill: All free-draining granular wall backfill material should be a crushed stone, sand/gravel mixture, or sand/crushed stone mixture. The material should have less than 3 percent passing the No. 200 sieve and less than 30 percent passing the No. 40 sieve. The minus No. 40 sieve material should be non-plastic. Granular wall backfill should not be water jetted during installation.

Select Fill Behind Walls: All select backfill material behind walls should consist of clayey sand and/or sandy clay material with a plasticity index of 16 or less, with a liquid limit not exceeding 35. The select fill should be placed in maximum 8-inch lifts and compacted to between 95 and 100 percent of Standard Proctor density (ASTM D 698) within a moisture range of plus to minus 3 percentage points of the optimum moisture. Compaction within five feet of the walls should be accomplished using hand compaction equipment and should be compacted between 90 and 95 percent of the Standard Proctor Density.

Flowable Backfill: Item 401, Texas Department of Transportation Standard Specifications for Construction and Maintenance of Highways, Streets, and Bridges, 2014 Edition.

On-Site Soil Backfill: For wall backfill areas with site-excavated materials or similar imported materials, all oversized fragments larger than four inches in maximum dimension should be removed from the backfill materials before placement. The backfill should be free of all organic and deleterious materials and should be placed in maximum 8-inch compacted lifts at a minimum of 95 percent of Standard Proctor density (ASTM D 698) within a moisture range of plus to minus 3 percentage points of optimum moisture. Compaction within five feet of the walls should be accomplished using hand compaction equipment and should be between 90 and 95 percent of the Standard Proctor Density.

8.4.1 Wall Backfill Settlement

Settlement of the wall backfill should be anticipated. Piping and conduits through the fill should be designed for potential soil loading due to fill settlement. Slabs, sidewalks, and pavements over fill material also may settle. Backfill compacted to the density recommended above is anticipated to settle on the order of 0.2 to 0.5 percent of the fill thickness.

8.5 Below-Grade Drainage Requirements

The design recommendations presented above assume hydrostatic pressure will not develop behind the retaining walls. In order to achieve the drained condition for lateral earth pressure for low-permeability walls (concrete, masonry, etc.), a vertical drainage blanket or geocomposite drainage member must be installed adjacent to the wall on the backfill side. In conjunction with the retaining wall, a collection pipe situated at or below the base of the wall or weep holes near the base of the retaining wall is recommended. The drainage blanket must be connected to an outlet drain at the base of the wall, weep holes, or to a sump/pump system. Drains should be properly filtered to minimize the potential for erosion through these drains, and /or the plugging of drain lines. Design or specific recommendations for drainage members is beyond the scope for this study. These services can be provided as an additional service upon request. In order to achieve the drained condition, the entire backfill material must be free draining. It is recommended the backfill-wall geometry be such that the backfill will not become saturated from rainfall, ground water, adjacent water courses, or other sources.

9.0 EARTHWORK

9.1 Site Preparation

The building areas should be stripped of vegetation, roots, old construction debris, and other organic material. It is estimated that the depth of stripping will be on the order of 6 inches. The actual stripping depth should be based on field observations with particular attention given to old drainage areas, uneven topography, and excessively wet soils. The stripped areas should be observed to determine if additional excavation is required to remove weak or otherwise objectionable materials that would adversely affect the fill placement or other construction activities.

The subgrade should be firm and able to support the construction equipment without displacement. Soft or yielding subgrade should be corrected and made stable before construction proceeds. The subgrade should be proof rolled to detect soft spots, which if exist, should be excavated to provide a firm and otherwise suitable subgrade. Proof rolling should be performed using a heavy pneumatic tired roller, loaded dump truck, or similar piece of equipment. The proof rolling operations should be observed by the project geotechnical engineer or his/her representative. Prior to fill placement, the subgrade should be scarified to a minimum depth of 8 inches, its moisture content adjusted, and recompacted to the moisture and density recommended for fill.

9.2 Placement and Compaction

Fill material should be placed in loose lifts not exceeding 8 inches in uncompacted thickness. The uncompacted lift thickness should be reduced to 4 inches for structure backfill zones requiring hand-operated power compactors or small self-propelled compactors. The fill material should be uniform with respect to material type and moisture content. Clods and chunks of material should be broken down and the fill material mixed by disking, blading, or plowing, as necessary, so that a material of uniform moisture and density is obtained for each lift. Water required for sprinkling to bring the fill material to the proper moisture content should be applied evenly through each layer.

The on-site soils are suitable for use in general site grading. Imported general fill material should be clean soil with a Liquid Limit less than 35 and no rock greater than 4 inches in maximum dimension. Excavated rock materials may be utilized provided 50 percent of the material passes the No. 4 sieve and no rock fragments are larger than 4 inches in any dimension. Significant processing efforts of excavated oversize surficial limestone should be anticipated in order to utilize as fill. The fill materials should be free of vegetation and debris.

The fill material should be compacted to a density ranging from 95 to 100 percent of maximum dry density as determined by ASTM D 698, Standard Proctor. In conjunction with the compacting operation, the fill material should be brought to the proper moisture content. The moisture content for general earth fill should range from 2 percentage points below optimum to 5 percentage points above optimum (-2 to +5). These ranges of moisture contents are given as maximum recommended ranges. For some soils and under some conditions, the contractor may have to maintain a more narrow range of moisture content (within the recommended range) in order to consistently achieve the recommended density.

Field density tests should be taken as each lift of fill material is placed. As a guide, one field density test per lift for each 5,000 square feet of compacted area is recommended. For small areas or critical areas the frequency of testing may need to be increased to one test per 2,500 square feet. A minimum of 2 tests per lift should be required. The earthwork operations should be observed and tested on a continuing basis by an experienced geotechnician working in conjunction with the project geotechnical engineer.

Each lift should be compacted, tested, and approved before another lift is added. The purpose of the field density tests is to provide some indication that uniform and adequate compaction is being obtained. The actual quality of the fill, as compacted, should be the responsibility of the contractor and satisfactory results from the tests should not be considered as a guarantee of the quality of the contractor's filling operations.

If fill is to be placed on existing slopes that are steeper than five horizontal to one vertical, then the fill materials should be benched into the existing slopes in such a manner as to provide a good contact between the two materials and allow relatively horizontal lift placement.

9.3 Trench Backfill

Trench backfill for pipelines or other utilities should be properly placed and compacted. Overly dense or dry backfill can swell and create a mound along the completed trench line. Loose or wet backfill can settle and form a depression along the completed trench line. Distress to overlying structures, pavements, etc. is likely if heaving or settlement occurs. On-site soil fill material is recommended for trench backfill. Care should be taken not to use free draining granular material, to prevent the backfilled trench from becoming a french drain and piping surface or subsurface water beneath structures, pipelines, or pavements. If a higher class bedding material is required for the pipelines, a lean concrete bedding will limit water intrusion into the trench and will not require compaction after placement. The soil backfill should be placed in approximately 4- to 6-inch loose lifts. The density and moisture content should be as recommended for fill in Section 9.2, Placement and Compaction, of this report. A minimum of one field density test should be taken per lift for each 150 linear feet of trench, with a minimum of 2 tests per lift.

9.4 Excavation

Based on the exploration borings, major excavations will encounter intact limestone units in select areas. These limestones are generally moderately hard to very hard and will require heavy duty specialized equipment for excavation. In addition, overexcavation should be anticipated within the limestones. Overexcavation may result from large blocks or chunks breaking along weathered seams or jointed seams beyond the planned excavation.

The side slopes of excavations through the overburden soils should be made in such a manner to provide for their stability during construction. Existing structures, pipelines or other facilities, which are constructed prior to or during the currently proposed construction and which require excavation, should be protected from loss of end bearing or lateral support.

Seasonal water seeps can occur where the tan limestones are approached or exposed by cuts. Subsoil drains may be required in some areas to intercept this seepage. This can be evaluated after grading has been performed.

Temporary construction slopes and/or permanent embankment slopes should be protected from surface runoff water. Site grading should be designed to allow drainage at planned areas where erosion protection is provided, instead of allowing surface water to flow down unprotected slopes.

Trench safety recommendations are beyond the scope of this report. The contractor must comply with all applicable safety regulations concerning trench safety and excavations including, but not limited to, OSHA regulations.

9.5 Acceptance of Imported Fill

Any soil imported from off-site sources should be tested for compliance with the recommendations for the particular application and approved by the project geotechnical engineer prior to the materials being used. The owner should also require the contractor to obtain a written, notarized certification from the landowner of each proposed off-site soil borrow source stating that to the best of the landowner's knowledge and belief there has never been contamination of the borrow source site with hazardous or toxic materials. The certification should be furnished to the owner prior to proceeding to furnish soils to the site. Soil materials derived from the excavation of underground petroleum storage tanks should not be used as fill on this project.

9.6 Soil Corrosion Potential

Specific testing for soil corrosion potential was not included in the scope of this study. However, based upon past experience on other projects in the vicinity, the soils at this site may be corrosive. Standard construction practices for protecting metal pipe and similar facilities in contact with these soils should be used.

9.7 Erosion and Sediment Control

All disturbed areas should be protected from erosion and sedimentation during construction, and all permanent slopes and other areas subject to erosion or sedimentation should be provided with permanent erosion and sediment control facilities. All applicable ordinances and codes regarding erosion and sediment control should be followed.

10.0 CONSTRUCTION OBSERVATIONS

In any geotechnical investigation, the design recommendations are based on a limited amount of information about the subsurface conditions. In the analysis, the geotechnical engineer must assume the subsurface conditions are similar to the conditions encountered in the borings. However, quite often during construction anomalies in the subsurface conditions are revealed. Should such anomalies be discovered, it is recommended Mr. Wally Burge immediately notify CMJ Engineering, Inc. before proceeding further with construction to allow CMJ Engineering, Inc. to reconsider its recommendations as necessary. It is also recommended that Mr. Wally Burge retain CMJ Engineering, Inc. to observe earthwork and foundation installation and perform materials evaluation during the construction phase of the project. This enables the geotechnical engineer to stay abreast of the project and to be readily available to evaluate unanticipated conditions, to conduct additional tests if required and, when necessary, to recommend alternative solutions to unanticipated conditions. Until these construction phase services are performed by the project geotechnical engineer, the recommendations contained in this report on such items as final foundation bearing elevations, proper soil moisture condition, and other such subsurface related recommendations shall only be considered as preliminary, and not final, recommendations.

It is proposed that construction phase observation and materials testing commence by the project geotechnical engineer at the outset of the project. Experience has shown that the most suitable method for procuring these services is for the owner or the owner's design engineers to contract directly with the project geotechnical engineer. This results in a clear, direct line of communication between the owner and the owner's design engineers and the geotechnical engineer.

11.0 REPORT CLOSURE

The borings for this study were staked by CMJ Engineering, Inc. using hand-held GPS equipment. The actual boreholes were placed as close as practical to the staked locations by CMJ Engineering, Inc. The locations and elevations of the borings should be considered accurate only to the degree implied by the methods used in their determination. The boring logs shown in this report contain information related to the types of soil encountered at specific locations and times and show lines delineating the interface between these materials. The logs also contain our field representative's interpretation of conditions that are believed to exist in those depth intervals between the actual samples taken. Therefore, these boring logs contain both factual and interpretive information. Laboratory soil classification tests were also performed on samples from selected depths in the borings. The results of these tests, along with visual-manual procedures were used to generally classify each stratum. Therefore, it should be understood that the classification data on the logs of borings represent visual estimates of classifications for those portions of each stratum on which the full range of laboratory soil classification tests were not performed. It is not implied that these logs are representative of subsurface conditions at other locations and times.

With regard to ground water conditions, this report presents data on ground water levels as they were observed during the course of the field work. In particular, water level readings have been made in the borings at the times and under conditions stated in the text of the report and on the boring logs. It should be noted that fluctuations in the level of the ground-water table can occur with passage of time due to variations in rainfall, temperature and other factors. Also, this report does not include quantitative information on rates of flow of ground water into excavations, on pumping capacities necessary to dewater the excavations, or on methods of dewatering excavations. Unanticipated soil conditions at a construction site are commonly encountered and cannot be fully predicted by mere soil samples, test borings or test pits. Such unexpected conditions frequently require that additional expenditures be made by the owner to attain a properly designed and constructed project. Therefore, provision for some contingency fund is recommended to accommodate such potential extra cost.

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of our field investigation and further on the assumption that the exploratory borings are representative of the subsurface conditions throughout the site; that is, the subsurface conditions everywhere are not significantly different from those disclosed by the borings at the time they were completed. If, during construction, different subsurface conditions from those encountered in our borings are observed, or appear to be present in excavations, we must be advised promptly so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between submission of this report and the start of the work at the site (more than twelve months is considered a substantial lapse of time; however, depending on the circumstances, less than six months may be considered a substantial lapse of time), if conditions have changed due either to natural causes or to construction operations at or adjacent to the site, or if structure locations, structural loads or finish grades are changed, we urge that we be promptly informed and retained to review our report to determine the applicability of the conclusions and recommendations, considering the changed conditions and/or time lapse. In this regard, if (a) construction at the site does not commence within twelve months of the date of this report and (b) CMJ Engineering, Inc. is not present at the site when construction commences to confirm that conditions have not changed since the date of this report, the information in this report cannot be relied upon or used for any purpose.

Further, it is urged that CMJ Engineering, Inc. be retained to review those portions of the plans and specifications for this particular project that pertain to earthwork and foundations as a means to determine whether the plans and specifications are consistent with the recommendations contained in this report. In addition, we are available to observe construction, particularly the compaction of structural fill, or backfill and the construction of foundations as recommended in the report, and such other field observations as might be necessary.

The scope of our services did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, ground water or air, on or below or around the site.

This report has been prepared for use in developing an overall design concept. Paragraphs, statements, test results, boring logs, diagrams, etc. should not be taken out of context, nor utilized without a knowledge and awareness of their intent within the overall concept of this report. The reproduction of this report, or any part thereof, supplied to persons other than the owner, should indicate that this study was made for design purposes only and that verification of the subsurface conditions for purposes of determining difficulty of excavation, trafficability, etc. are responsibilities of the contractor.

This report has been prepared for the exclusive use of Mr. Wally Burge and his consultants for specific application to design of this project only, and not for additions or modifications to the project. The only warranty made by us in connection with the services provided is that we have used that degree of care and skill ordinarily exercised under similar conditions by reputable members of our profession practicing in the same or similar locality. No other warranty, expressed or implied, is made or intended.

* * * *

Appendix A

PLATE A.1

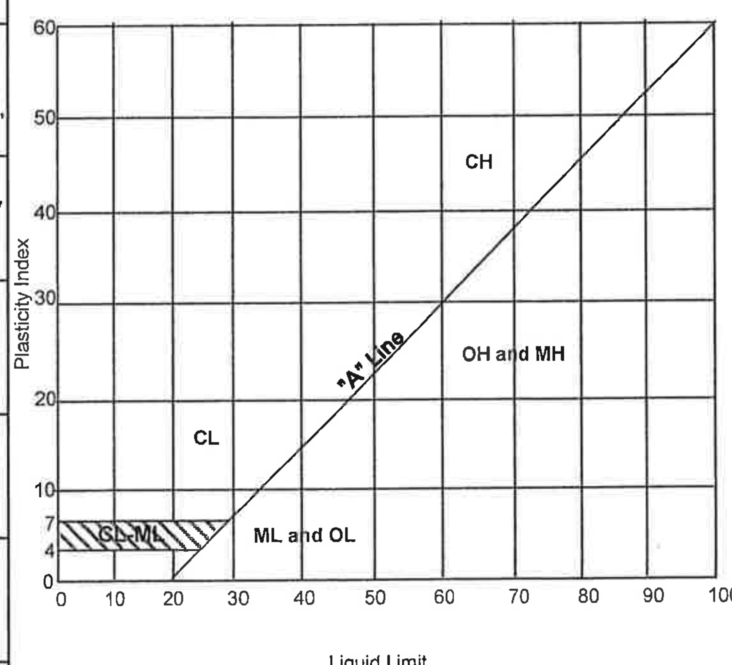
PLAN OF BORINGS

BURGE RESIDENCE
3600 LANDS END
FORT WORTH, TEXAS

CMJ ENGINEERING, INC.

CMJ PROJECT NO. 2875-21-01

Major Divisions			Sym.	Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (more than half of the material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent.........GW, GP, SW, SP More than 12 percent.........GM, GC, SM, SC 5 to 12 percent.........*Borderline* cases requiring dual symbols	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4: $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures		Liquid and Plastic limits below "A" line or P.I. greater than 4	Liquid and plastic limits plotting in hatched zone between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures		Liquid and Plastic limits above "A" line with P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 6: $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ between 1 and 3	
			SP	Poorly graded sands; gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures		Liquid and Plastic limits below "A" line or P.I. less than 4	Liquid and plastic limits plotting between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures		Liquid and Plastic limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half of material is smaller than No. 200 sieve)	Silts and clays (Liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity	Plasticity Chart		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, and lean clays			
			OL	Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH	Inorganic clays of high plasticity, fat clays			
			OH	Organic clays of medium to high plasticity, organic silts			
Highly Organic soils			Pt	Peat and other highly organic soils			

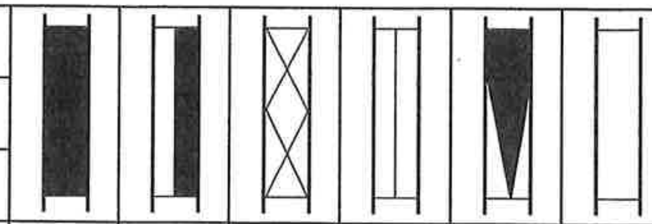


Plasticity Chart

UNIFIED SOIL CLASSIFICATION SYSTEM **PLATE A.2**

SOIL OR ROCK TYPES

GRAVEL	LEAN CLAY	LIMESTONE
SAND	SANDY	SHALE
SILT	SILTY	SANDSTONE
CLAYEY	HIGHLY PLASTIC CLAY	CONGLOMERATE



Shelby Tube	Auger	Split Spoon	Rock Core	Cone Pen	No Recovery

TERMS DESCRIBING CONSISTENCY, CONDITION, AND STRUCTURE OF SOIL

Fine Grained Soils (More than 50% Passing No. 200 Sieve)

Descriptive Item	Penetrometer Reading, (tsf)
Soft	0.0 to 1.0
Firm	1.0 to 1.5
Stiff	1.5 to 3.0
Very Stiff	3.0 to 4.5
Hard	4.5+

Coarse Grained Soils (More than 50% Retained on No. 200 Sieve)

Penetration Resistance (blows/foot)	Descriptive Item	Relative Density
0 to 4	Very Loose	0 to 20%
4 to 10	Loose	20 to 40%
10 to 30	Medium Dense	40 to 70%
30 to 50	Dense	70 to 90%
Over 50	Very Dense	90 to 100%

Soil Structure

Calcareous	Contains appreciable deposits of calcium carbonate; generally nodular
Slickensided	Having inclined planes of weakness that are slick and glossy in appearance
Laminated	Composed of thin layers of varying color or texture
Fissured	Containing cracks, sometimes filled with fine sand or silt
Interbedded	Composed of alternate layers of different soil types, usually in approximately equal proportions

TERMS DESCRIBING PHYSICAL PROPERTIES OF ROCK

Hardness and Degree of Cementation

Very Soft or Plastic	Can be remolded in hand; corresponds in consistency up to very stiff in soils
Soft	Can be scratched with fingernail
Moderately Hard	Can be scratched easily with knife; cannot be scratched with fingernail
Hard	Difficult to scratch with knife
Very Hard	Cannot be scratched with knife
Poorly Cemented or Friable	Easily crumbled
Cemented	Bound together by chemically precipitated material; Quartz, calcite, dolomite, siderite, and iron oxide are common cementing materials.

Degree of Weathering

Unweathered	Rock in its natural state before being exposed to atmospheric agents
Slightly Weathered	Noted predominantly by color change with no disintegrated zones
Weathered	Complete color change with zones of slightly decomposed rock
Extremely Weathered	Complete color change with consistency, texture, and general appearance approaching soil

KEY TO CLASSIFICATION AND SYMBOLS **PLATE A.3**

Project No.	Boring No.	Project
2875-21-01	B-1	Proposed Residence and Retaining Wall 3600 Lands End Street - Fort Worth, Texas

Location	Water Observations
See Plate A.1	Dry during drilling; dry at completion

Completion Depth	Completion Date	Surface Elevation	Type
35.0'	9-7-21	672.0	CME-55, w/ CFA

Depth, Ft.	Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
0		**SILTY CLAY**, brown and gray, w/ limestone fragments and calcareous nodules, hard (FILL) -w/ gypsum, 1' to 2'			4.5+	63	32	15	17	10		
					4.5+					8	119	2470
										8		
5		-w/ limestone boulders below 4'			100/4.5"					5		
	665.0	**LIMESTONE**, tan, fractured, w/ clay seams, moderately hard to hard										
10					100/2"					6		
	659.0	**LIMESTONE**, tan, w/ clay seams, hard to very hard										
15					100/1.625"					8		
20					100/1"							
25					100/1.25"							
	645.0	**SHALE**, gray, w/ limestone seams, moderately hard to hard										
30					100/2"							
35	637.0				100/2.25"							

LOG OF BORING 2875-21-01.GPJ CMJ.GDT 9/27/21

Project No.	Boring No.	Project
2875-21-01	B-2	Proposed Residence and Retaining Wall 3600 Lands End Street - Fort Worth, Texas

Location	Water Observations
See Plate A.1	Dry during drilling; dry at completion

Completion Depth	Completion Date	Surface Elevation	Type
35.0'	9-7-21	672.0	CME-55, w/ CFA

Stratum Description

Depth, Ft.	Elev.	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
0	671.0	**SILTY CLAY**, dark brown, w/ gravel, limestone fragments, and asphalt fragments (FILL)				45	26	15	11	8		
1		**LIMESTONE**, tan, fractured, w/ clay seams, hard								4		
4					100/1.5"					5		
8	664.0	**LIMESTONE**, tan, w/ clay seams, hard to very hard										
9					100/1"					9		
14					100/1.125"					8		
19					100/1.375"							
24					100/1.5"							
26		-w/ gray shale seams and layers below 26'										
28	644.0	**SHALE**, gray, w/ limestone seams, hard										
29					100/1.625"							
32		-4-inch thick limestone seam at 32'										
34	637.0	-2-inch thick limestone seam at 34'			100/1.5"							

LOG OF BORING 2875-21-01.GPJ CMJ.GDT 9/27/21

Project No.	Boring No.	Project
2875-21-01	B-3	Proposed Residence and Retaining Wall 3600 Lands End Street - Fort Worth, Texas

Location	Water Observations
See Plate A.1	Dry during drilling; dry at completion

Completion Depth	Completion Date	Surface Elevation	Type
40.0'	9-7-21	670.0	CME-55, w/ CFA

Elevation	Stratum Description	REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
	SILTY CLAY, brown, w/ limestone fragments, calcareous nodules, and occasional iron stains, hard (FILL) -w/ dark brown, 1' to 2' -w/ dark brown below 3'					27	14	13	6		
				4.5+					7		
				4.5+					8	101	1890
				4.5+		35	16	19	9	103	
665.0				4.5+					13	116	21090
	LIMESTONE, tan, fractured, w/ clay seams, moderately hard								4		
660.0				100/2.5"					5		
	LIMESTONE, tan, w/ clay seams, hard			100/1.375"					7		
				100/1.5"							
642.0				100/1.375"							
	SHALE, gray, w/ limestone seams, hard -6-inch thick limestone seam at 32' -2-inch thick limestone seam at 32' -2-inch thick limestone seam at 38'			100/1.375"							
				100/1.75"							
630.0				100/1.25"							

Project No.	Boring No.	Project
2875-21-01	B-4	Proposed Residence and Retaining Wall 3600 Lands End Street - Fort Worth, Texas

Location	Water Observations
See Plate A.1	Dry during drilling; dry at completion

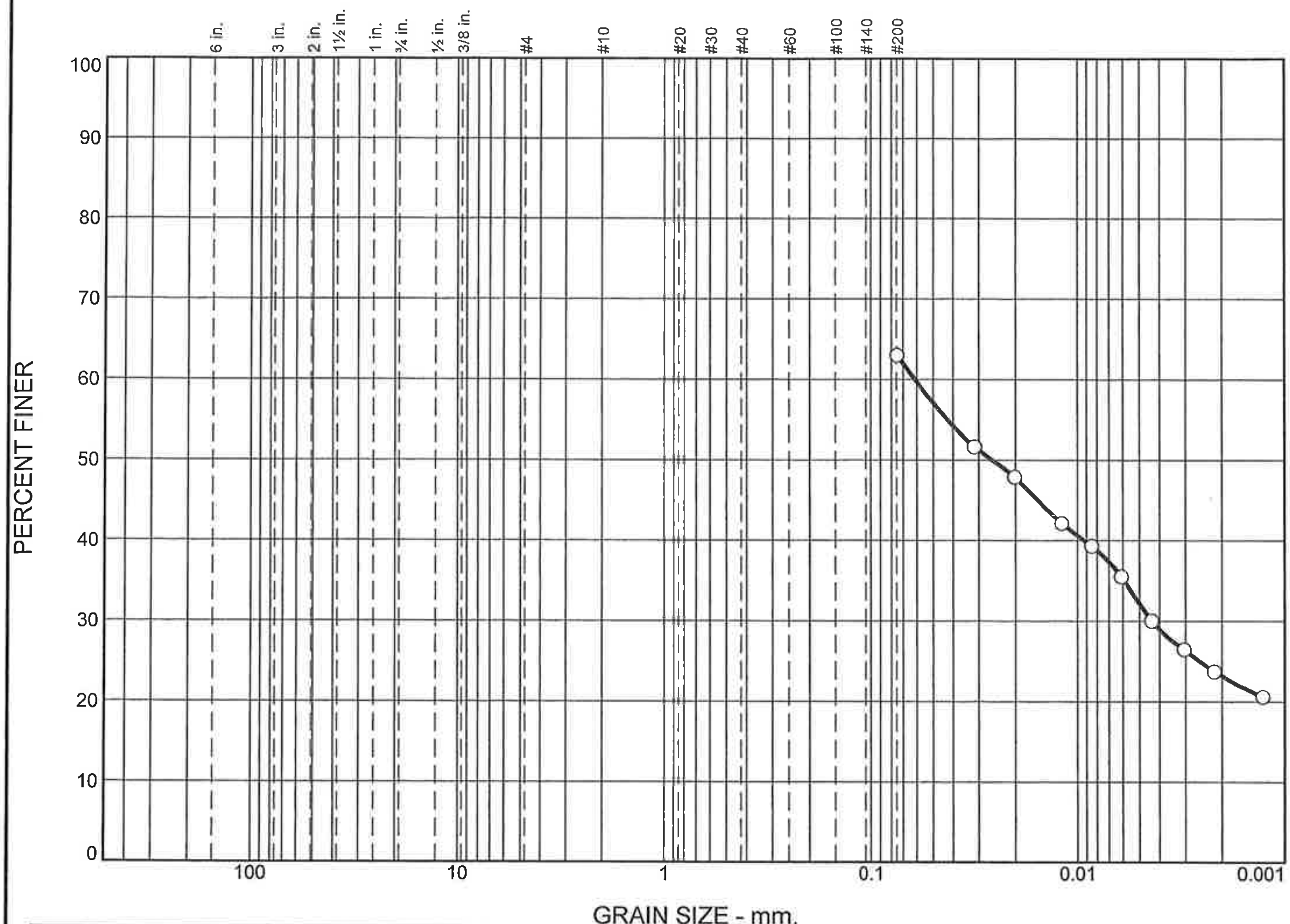
Completion Depth	Completion Date	Surface Elevation	Type
40.0'	9-7-21	671.0	CME-55, w/ CFA

Stratum Description

Elevation	Stratum Description
	SILTY CLAY, brown and dark brown, w/ limestone fragments, gravel, and calcareous nodules, hard (FILL) -w/ shale fragments, 1' to 2'
664.0	**LIMESTONE**, tan, fractured, w/ clay seams, hard
658.0	**LIMESTONE**, tan, w/ clay seams, hard
651.0	**LIMESTONE**, gray, hard
642.0	**SHALE**, gray, w/ limestone seams, moderately hard to hard -6-inch thick limestone seam at 32' -4-inch thick limestone seam at 37'
631.0	

REC %	RQD %	Blows/Ft. or Pen Reading, T.S.F.	Passing No 200 Sieve, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index	Moisture Content, %	Unit Dry Wt. Lbs./Cu. Ft.	Unconfined Compression Pounds/Sq. Ft.
							6		
		4.5+					9	105	1020
		4.5+	62	38	15	23	10		
		4.5+		44	19	25	15	102	
		4.5+					12	105	
							5		
		100/1.375"					5		
		100/1.25"					9		
		100/1.625"					8		
		100/1.25"							
		100/2.25"							
		100/2.5"							
		100/2"							

Particle Size Distribution Report



	% +3"	% Gravel		% Sand			% Fines	
		Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
○							39.8	23.2

	LL	PL	D_{85}	D_{60}	D_{50}	D_{30}	D_{15}	D_{10}	C_c	C_u
○	32	15		0.0612	0.0259	0.0044				

	Material Description	USCS	AASHTO
○			

Project No. 2875-21-01 **Client:** Mr. Wally Burge
Project: Prop. Res & Retain.Wall-3600 Lands End St.-FW, TX

○ **Depth:** 0-1 **Sample Number:** B-1

Remarks:

CMJ ENGINEERING, INC.

Fort Worth, Texas

PLATE A.8

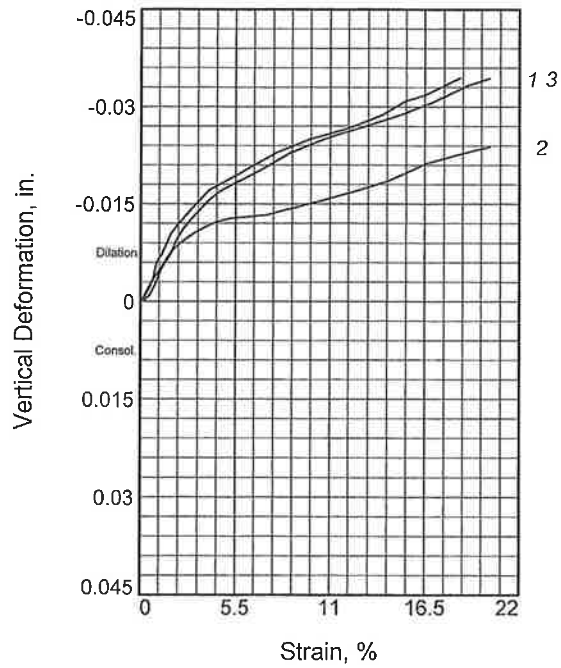
FREE SWELL TEST RESULTS

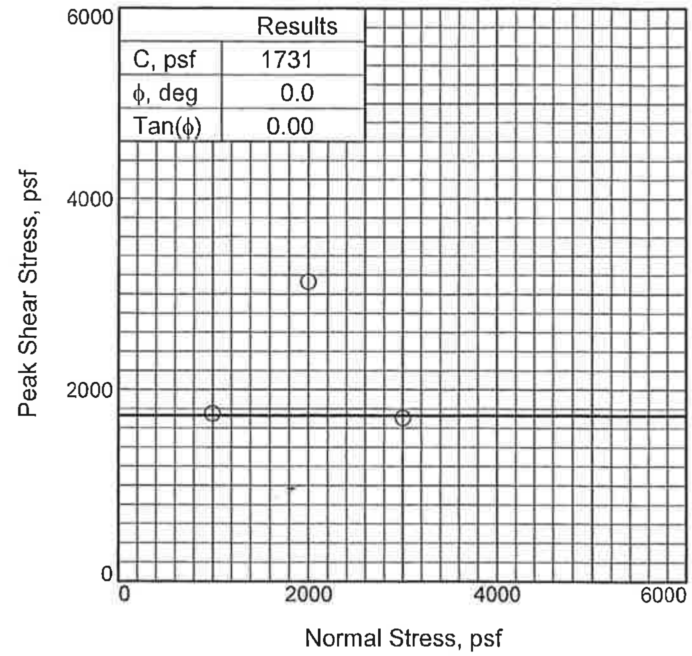
Project: Proposed Residence and Retaining Wall
3600 Lands End Street - Fort Worth, Texas

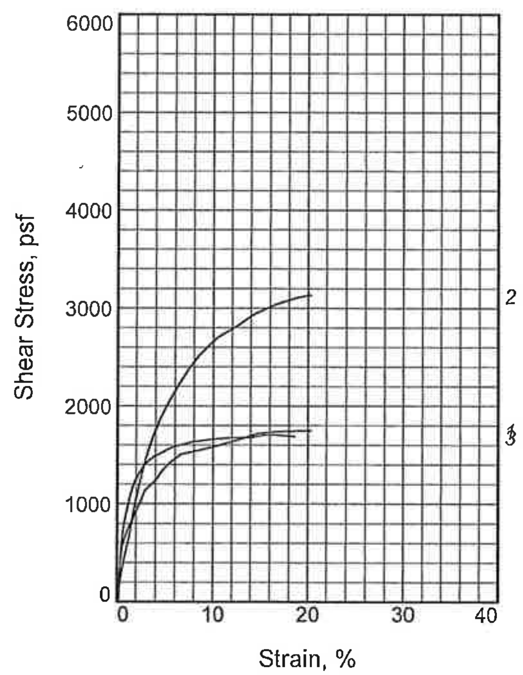
Project No.: 2875-21-01

Boring No.	Depth Interval (ft.)	Sample Description	Liquid Limit LL	Plastic Limit PL	Plasticity Index PI	Moisture Content % Initial	Moisture Content % Final	Percent Swell (%)
B-4	4 – 5	Silty Clay (fill)	44	19	25	12.3	19.6	0.0

1. Free swell tests performed at approximate overburden pressure.
2. Atterberg Limits performed on adjacent recovered sample.







Sample No.		1	2	3
Initial	Water Content, %	8.6	8.6	8.6
	Dry Density, pcf	102.6	100.3	103.7
	Saturation, %	37.2	35.1	38.2
	Void Ratio	0.6130	0.6498	0.5958
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
At Test	Water Content, %	21.5	22.0	20.2
	Dry Density, pcf	102.6	100.3	103.7
	Saturation, %	93.0	89.7	90.1
	Void Ratio	0.6130	0.6498	0.5958
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
Normal Stress, psf		1000	2000	3000
Peak Shear Stress, psf		1750	3131	1706
Strain, %		20.2	20.2	15.4
Residual Stress, psf				
Strain, %				
Strain rate, in./min.		0.000	0.000	0.000

LL= 35 **PL=** 19 **PI=** 16

Assumed Specific Gravity= 2.65

Remarks:

Client: Mr. Wally Burge

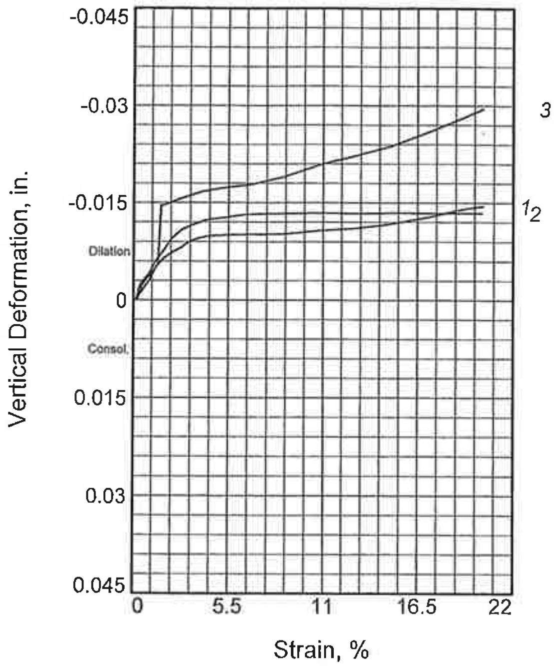
Project: Prop. Res & Retain.Wall-3600 Lands End St.-FW, TX

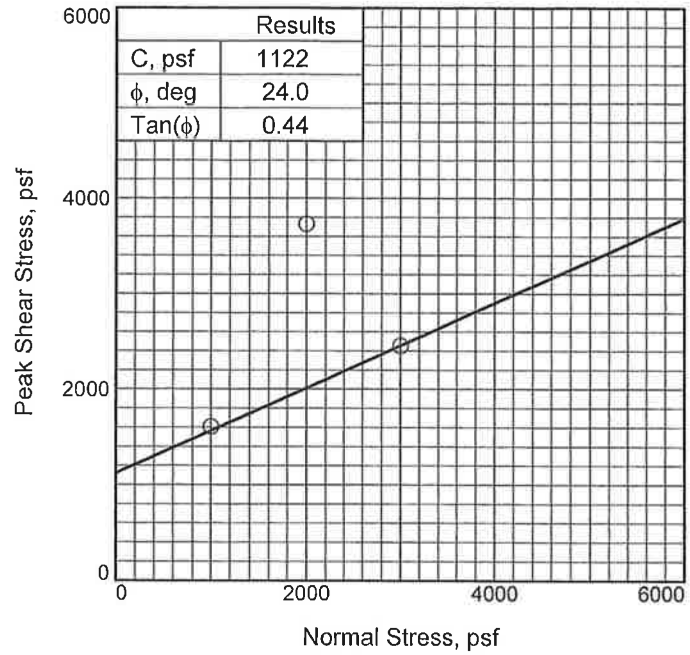
Sample Number: B-3 **Depth:** 3-4

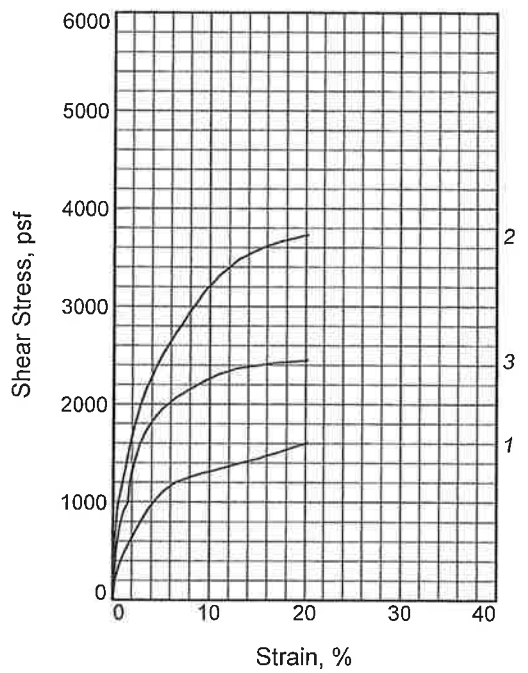
Proj. No.: 2875-21-01

DIRECT SHEAR TEST REPORT
CMJ ENGINEERING, INC.
Fort Worth, Texas

PLATE A.10







	Sample No.	1	2	3
Initial	Water Content, %	8.6	8.6	8.6
	Dry Density, pcf	102.6	100.3	103.7
	Saturation, %	37.2	35.1	38.2
	Void Ratio	0.6130	0.6498	0.5958
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
At Test	Water Content, %	21.5	22.0	20.2
	Dry Density, pcf	102.6	100.3	103.7
	Saturation, %	93.0	89.7	90.1
	Void Ratio	0.6130	0.6498	0.5958
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
	Normal Stress, psf	1000	2000	3000
	Peak Shear Stress, psf	1607	3730	2453
	Strain, %	20.2	20.1	20.2
	Residual Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.025	0.025	0.025

LL= 35 **PL=** 19 **PI=** 16
Assumed Specific Gravity= 2.65
Remarks:

Client: Mr. Wally Burge

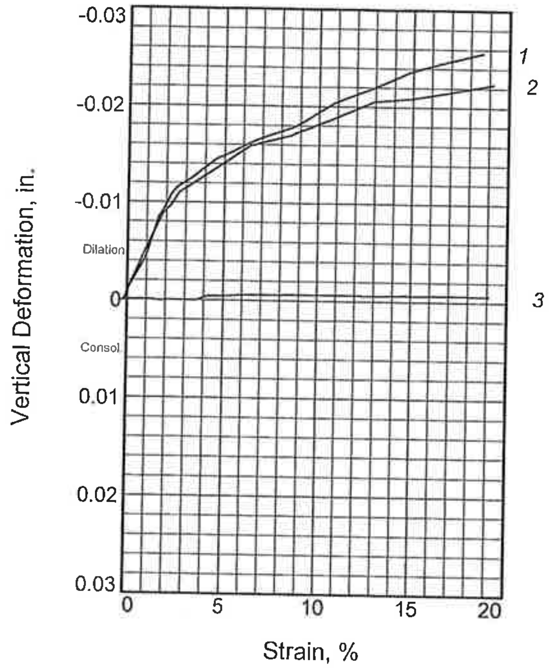
Project: Prop. Res & Retain.Wall-3600 Lands End St.-FW, TX

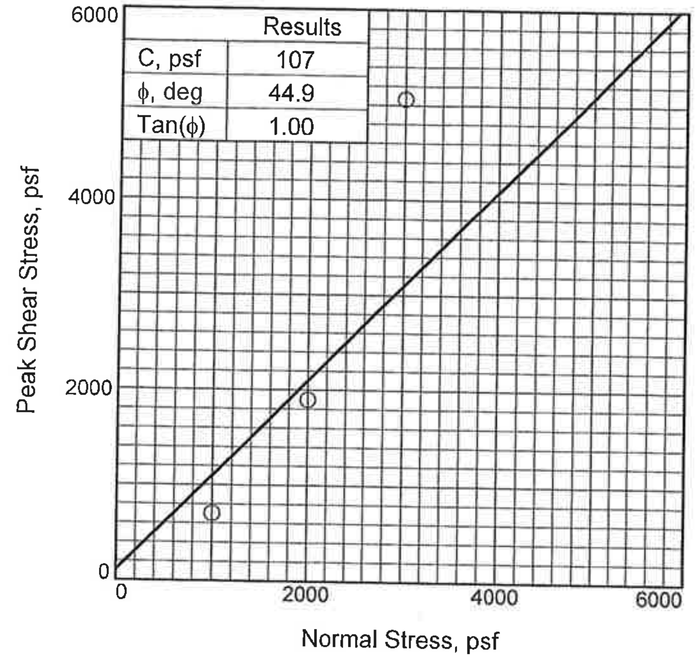
Sample Number: B-3 **Depth:** 3-4 RES

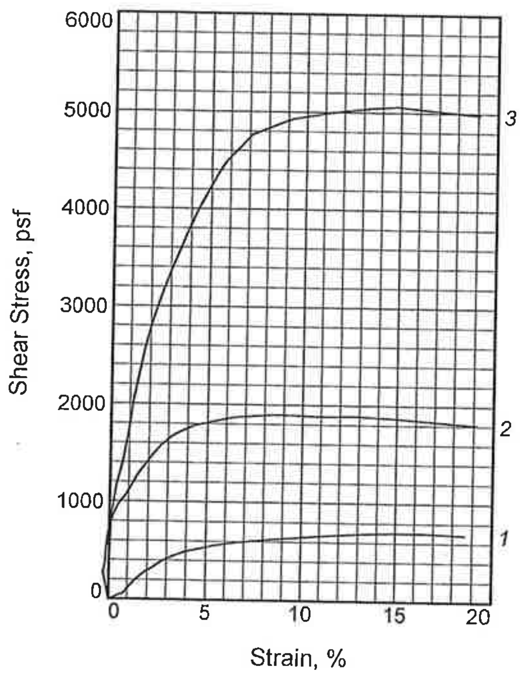
Proj. No.: 2875-21-01

DIRECT SHEAR TEST REPORT
CMJ ENGINEERING, INC.
Fort Worth, Texas

PLATE A.11







	Sample No.	1	2	3
Initial	Water Content, %	14.7	14.7	14.7
	Dry Density, pcf	94.0	103.7	101.6
	Saturation, %	51.1	65.3	61.9
	Void Ratio	0.7595	0.5948	0.6278
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
At Test	Water Content, %	27.0	20.4	23.0
	Dry Density, pcf	94.0	103.7	101.6
	Saturation, %	94.3	91.1	97.0
	Void Ratio	0.7595	0.5948	0.6278
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
Normal Stress, psf		1000	2000	3000
Peak Shear Stress, psf		701	1898	5063
Strain, %		14.8	8.8	14.8
Residual Stress, psf				
Strain, %				
Strain rate, in./min.		0.005	0.005	0.005

LL= 44 **PL=** 25 **PI=** 19
Assumed Specific Gravity= 2.65
Remarks:

Client: Mr. Wally Burge

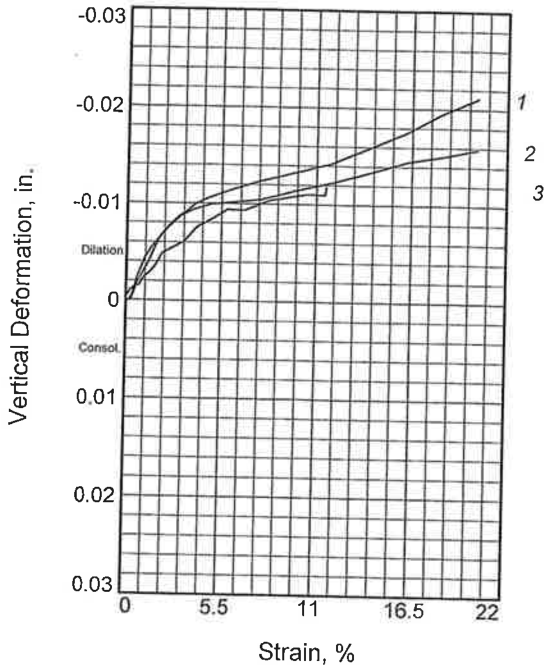
Project: Prop. Res & Retain.Wall-3600 Lands End St.-FW, TX

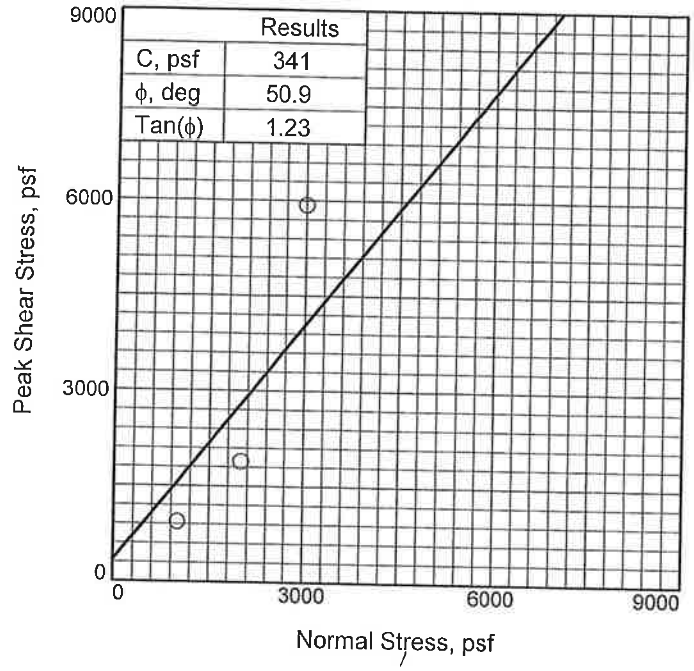
Sample Number: B-4 **Depth:** 3-4

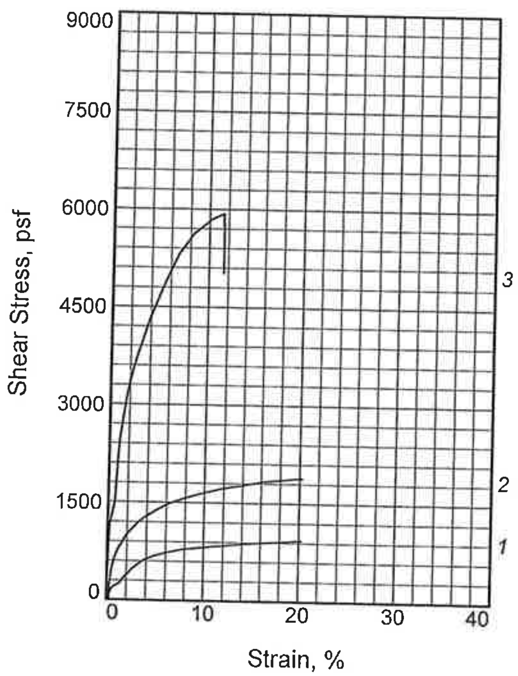
Proj. No.: 2875-21-01

DIRECT SHEAR TEST REPORT
CMJ ENGINEERING, INC.
Fort Worth, Texas

PLATE A.12







	Sample No.	1	2	3
Initial	Water Content, %	14.7	14.7	14.7
	Dry Density, pcf	94.0	103.7	101.6
	Saturation, %	51.1	65.3	61.9
	Void Ratio	0.7595	0.5948	0.6278
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
At Test	Water Content, %	27.0	20.4	23.0
	Dry Density, pcf	94.0	103.7	101.6
	Saturation, %	94.3	91.1	97.0
	Void Ratio	0.7595	0.5948	0.6278
	Diameter, in.	2.47	2.47	2.47
	Height, in.	0.96	0.96	0.96
	Normal Stress, psf	1000	2000	3000
	Peak Shear Stress, psf	936	1889	5931
	Strain, %	20.1	20.0	11.5
	Residual Stress, psf			
	Strain, %			
	Strain rate, in./min.	0.025	0.025	0.025

LL= 44 **PL=** 25 **PI=** 19
Assumed Specific Gravity= 2.65
Remarks:

Client: Mr. Wally Burge

Project: Prop. Res & Retain.Wall-3600 Lands End St.-FW, TX

Sample Number: B-4 **Depth:** 3-4 RES

Proj. No.: 2875-21-01

DIRECT SHEAR TEST REPORT
CMJ ENGINEERING, INC.
Fort Worth, Texas

PLATE A.13

Appendix B

Burge Residence
3600 Lands End - Fort Worth

CMJ Engineering, Inc.

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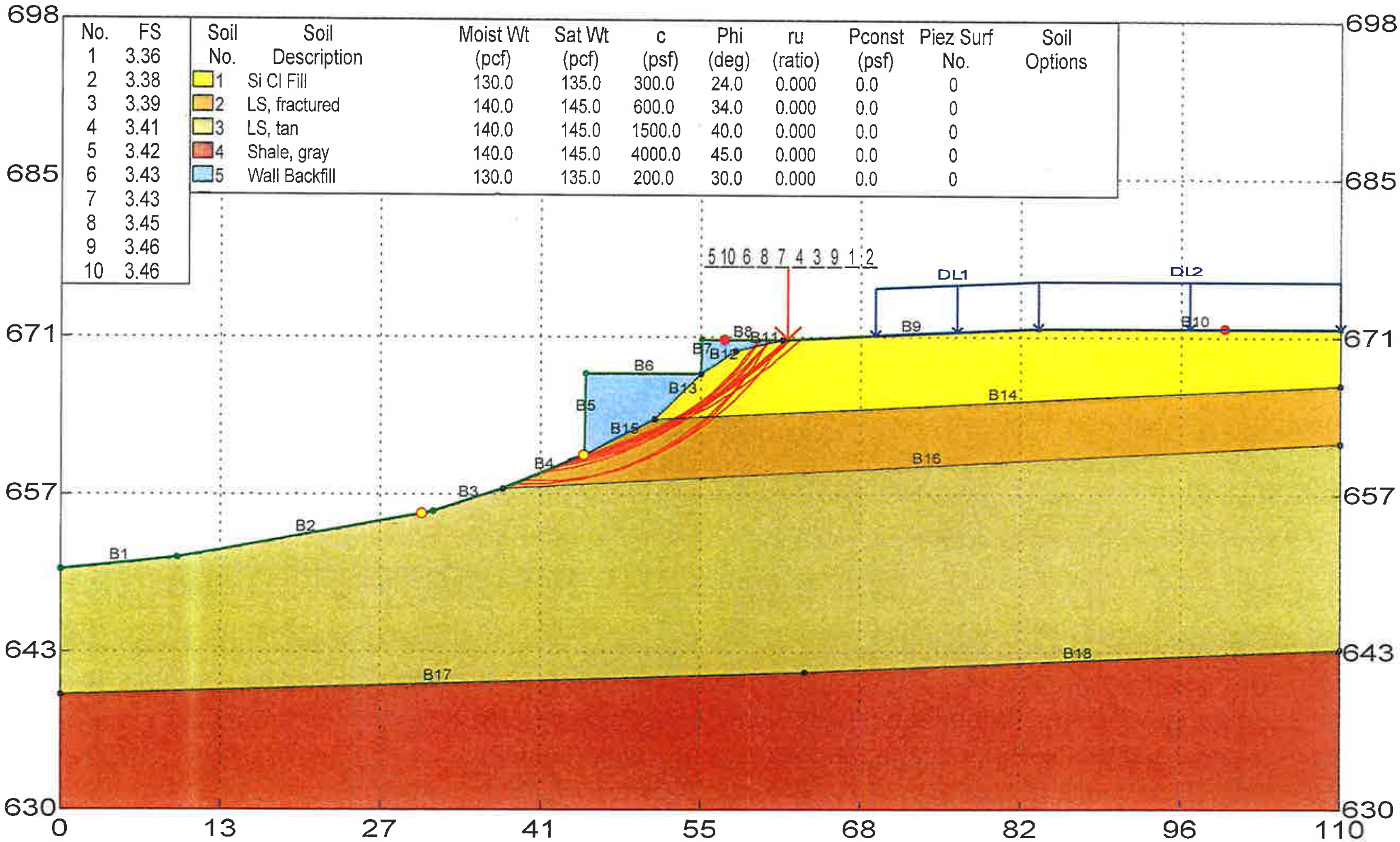

No.	FS
1	3.36
2	3.38
3	3.39
4	3.41
5	3.42
6	3.43
7	3.43
8	3.45
9	3.46
10	3.46

Soil No.	Soil Description	Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Si Cl Fill	130.0	135.0	300.0	24.0	0.000	0.0	0	
2	LS, fractured	140.0	145.0	600.0	34.0	0.000	0.0	0	
3	LS, tan	140.0	145.0	1500.0	40.0	0.000	0.0	0	
4	Shale, gray	140.0	145.0	4000.0	45.0	0.000	0.0	0	
5	Wall Backfill	130.0	135.0	200.0	30.0	0.000	0.0	0	

GEOSTASE FS = 3.36

Spencer Method

GEOSTASE Slope Stability Analysis

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

PLATE B.1

Burge Residence
3600 Lands End - Fort Worth

CMJ Engineering, Inc.

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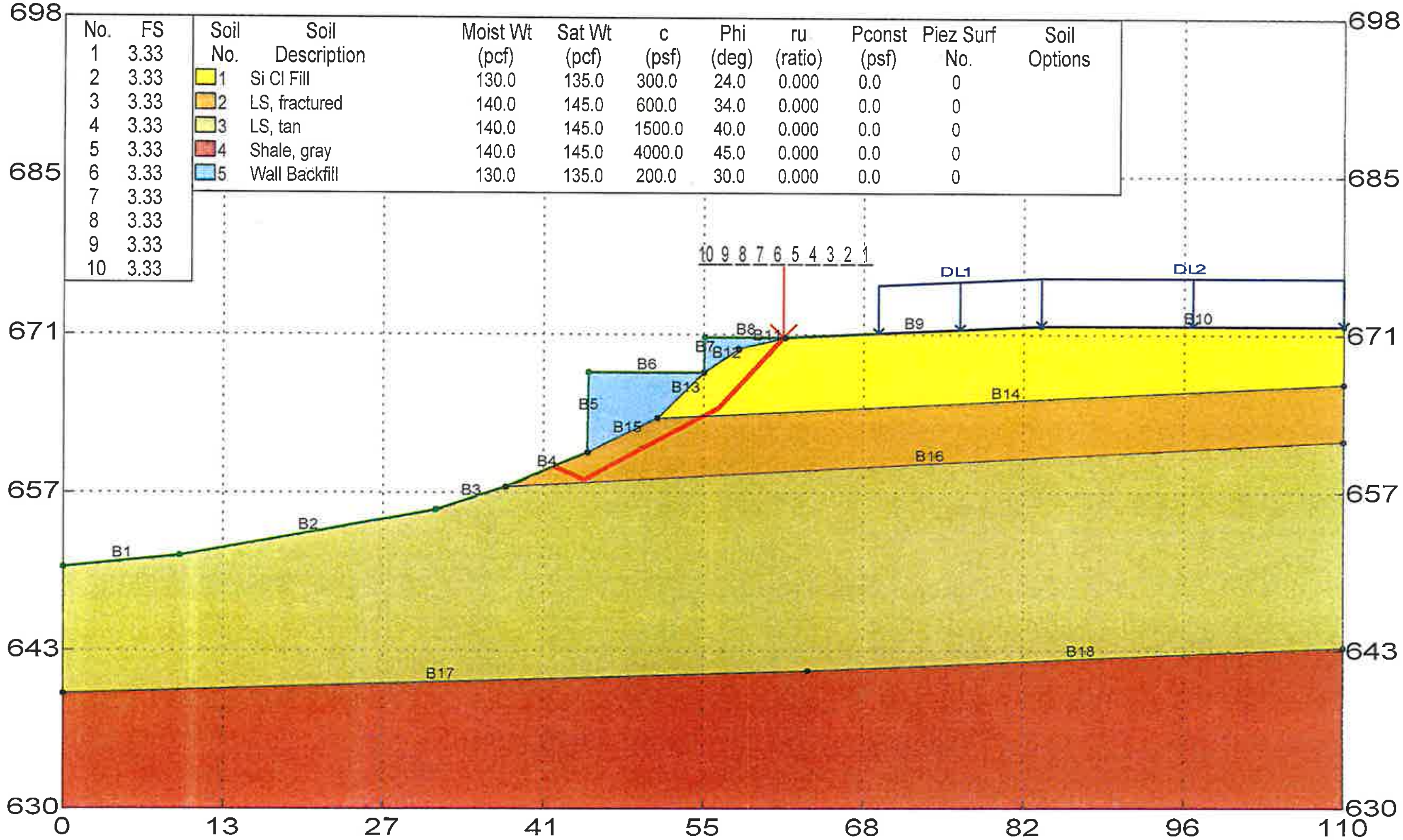

No.	FS
1	3.33
2	3.33
3	3.33
4	3.33
5	3.33
6	3.33
7	3.33
8	3.33
9	3.33
10	3.33

Soil No.	Soil Description	Moist Wt (pcf)	Sat Wt (pcf)	c (psf)	Phi (deg)	ru (ratio)	Pconst (psf)	Piez Surf No.	Soil Options
1	Si Cl Fill	130.0	135.0	300.0	24.0	0.000	0.0	0	
2	LS, fractured	140.0	145.0	600.0	34.0	0.000	0.0	0	
3	LS, tan	140.0	145.0	1500.0	40.0	0.000	0.0	0	
4	Shale, gray	140.0	145.0	4000.0	45.0	0.000	0.0	0	
5	Wall Backfill	130.0	135.0	200.0	30.0	0.000	0.0	0	

GEOSTASE FS = 3.33

Spencer Method

GEOSTASE Slope Stability Analysis

GEOSTASE® by GREGORY GEOTECHNICAL SOFTWARE

PLATE B.2