

**GEOTECHNICAL ENGINEERING REPORT
COURTSIDE MULTI-FAMILY
JEFFERSON AVENUE
SEGUIN, TEXAS**

Prepared For:

**Rickhaus Design, LLC
1469 South 4th Street
Louisville, Kentucky 40208**

Attn: Mr. Frank Leist

July 2019

PROJECT NO. 19-23544

www.roneengineers.com



July 2, 2019

Mr. Frank Leist
Rickhaus Design, LLC
1469 South 4th Street
Louisville, Kentucky 40208

**Re: Geotechnical Engineering Report
Courtside Multi-Family
Seguin, Texas
Rone Report No. 19-23544**

Dear Mr. Leist:

Rone Engineering Services, Ltd. (Rone) is pleased to submit our Geotechnical Engineering Report for the above referenced project. The geotechnical engineering services performed for this study were carried out in general accordance with Rone Proposal No. P-27095-19, dated April 25, 2019.

This report presents engineering analyses and recommendations for site grading, foundations and pavements with respect to available project characteristics. Results of our field exploration and laboratory testing are shown in detail in the appendix section of the study.

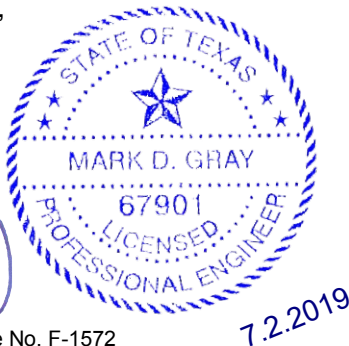
We appreciate the opportunity to be of service to you on this project. We look forward to providing additional Geotechnical Engineering and Construction Materials Testing services as the project progresses through the detailed design and construction phases. Please contact us if you have any questions or if we can be of further assistance.

Respectfully Submitted,

A blue ink signature of Mark D. Gray, consisting of stylized cursive letters.

Mark D. Gray, P.E.
Partner

Texas Engineering Firm License No. F-1572

A blue ink signature of Eric M. Hollabaugh, consisting of stylized cursive letters.

Eric M. Hollabaugh, P.E.
Senior Geotechnical Engineer

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GEOTECHNICAL ENGINEERING REPORT

1 INTRODUCTION

This report presents our geotechnical study for the proposed multi-family residential development, generally located on the north side of Jefferson Avenue, approximately 350 feet east of Ermel Street in Seguin, Texas. This study was performed in general accordance with the Scope of Services presented in our Proposal No. P-27095-19, dated April 25, 2019.

We understand the project consists of developing a multi-family residential complex on an approximately 9.6-acre tract of land. The apartment complex will include eight (8) two- to three-story apartment buildings, a leasing/community building, an in-ground swimming pools and associated paved parking and drive areas. Structural loading information was not available at the time of this report, but loads are expected to be relatively light. The proposed structures are expected to be supported by post-tensioned slab-on-grade foundations designed for potential seasonal vertical movements of up to about 1 or 2 inches. Grading information was not available at the time of this study. For the purpose of this study, we have assumed maximum cuts and fills of up to about 2 feet will be required to achieve final grades within the building pad areas.

2 PURPOSES AND SCOPE OF STUDY

The principal purposes of this study are to evaluate the general subsurface conditions at the project site and to develop geotechnical recommendations for the design and construction of foundations and pavements. To accomplish its intended purposes, the study was conducted in the following phases:

- Borings were drilled and sampled to evaluate the subsurface conditions at the boring locations and to obtain soil and rock samples.
- Laboratory tests were conducted on selected samples recovered from the borings to evaluate the pertinent engineering characteristics of the foundation soils and rock.
- Engineering analyses were performed using field and laboratory data to develop foundation and pavement design recommendations.

3 FIELD OPERATIONS AND LABORATORY TESTING

The borings were located in the field with measurements taken from site landmarks and using an aerial photograph of the site. These locations were not surveyed. The provided locations are accurate only to the extent implied by the technique used in their determination.

Subsurface conditions on site were evaluated by drilling a total of nine (9) borings to a depth of about 20 feet each for the apartment buildings and clubhouse and one (1) boring to a depth of 20 feet for the detention pond. The borings were advanced using a truck-mounted drilling rig in May 2019. The approximate boring locations are shown on Plate A.3, Boring Location Diagram. Sample depth, description of soils, and classification (based on the Unified Soil Classification System) are presented on the Logs of Boring, Plates A.4 through A.13. Keys to terms and symbols used on the logs are shown on Plates A.14 and A.15.

Laboratory tests were performed on selected samples recovered from the borings to confirm visual classification and determine the pertinent engineering properties of the retrieved soils. Classification test results are presented on the Logs of Boring. Swell test results are tabulated and presented in the Appendix section of the report on Plate A.16. Descriptions of the procedures used in the field and laboratory phases of this study are presented in Appendix B.

4 GENERAL SITE CONDITIONS

At the time the field exploration was performed, the site generally consisted of an open tract of land with short grass vegetation cover. Review of topographical information available from Google Earth® indicates the site generally slopes down towards southwest with about 10 feet of relief (approximate elevation 542 feet to 532 feet). A site vicinity map and geology map are attached as Plates A.1 and A.2, respectively. The general location and orientation of the site are shown on the Borings Location Diagram, Plate A.3, in Appendix A of this study.

4.1 Site Geology

Based on the subsurface conditions encountered at the boring locations and the Geologic Atlas of Texas, Seguin Sheet (published by the Bureau of Economic Geology), the site appears to be mapped within fluvial terrace deposits associated with the Guadalupe River (mapped as Qt). Terrace deposits are geologically recent, and developed as flood waters have scoured some formations, then deposited the transported soils downstream as the flood waters receded. Sand, silt, clay, and gravel are present in various proportions, with gravel more predominant in older, higher deposits. Terrace deposits can include point bars, natural levees, and stream channel deposits along valley walls. Terrace deposits become increasingly fine-grained on coastal and Nueces plains. Calcium carbonate-cemented quartz sand, silt, clay, and gravel are intermixed and interbedded. Low terraces of major rivers are typically capped by approximately 5 to 15 feet of clayey sand and silt. Sandy gravel on higher terraces varies somewhat in composition from river to river. Gravel is commonly rounded to angular limestone with chert pebbles and cobbles and some boulders. Please note that the geologic mapping was originally performed using aerial photography. Local variations and anomalies do occur.

4.2 Subsurface Soil Conditions

The various strata and their approximate depths and thickness are shown on the Logs of Boring. The stratification boundaries shown on the Logs of Boring represent the approximate locations of changes in types of soil and rock; in-situ, the transition between material types may be gradual and indistinct.

Subsurface conditions generally consisted of brown fat clay soils (CH) and brown, reddish brown and tan lean clay soils (CL) with calcareous nodules and variable sand content extending from the surface to the termination depth of the borings at 20 feet below existing site grades. Lean clay with sand, or sandy lean clay appeared to be the prevalent soil type across the site; however, the sampled soils varied between the two classifications with little consistency.

The plasticity index of the cohesive samples tested varied from 16 to 49, indicating low to high soil plasticity. A high plasticity index is generally associated with an increased potential for active clayey soils to shrink and swell with changes in moisture content.

The hand penetrometer values varied from 1.5 to more than 4.5 tons per square foot (tsf) in the cohesive soils. The Standard Penetration Test N-values varied between 16 to 37 blows per foot (bpf) in the soils below 15 feet. More detailed stratigraphic information is presented on the Logs of Boring.

4.3 Groundwater

The borings were advanced using continuous flight augers and intermittent sampling observe the potential for water seepage during and after drilling. Free water was not observed in the borings during, or upon completion of drilling; however, soils in the Seguin area are commonly water-bearing, typically present in a gravel layer just above contact with shale on the order of 30 to 40 feet below grade. The scope of work did not include long term observations of groundwater or perched water conditions. In addition, it is difficult to accurately predict the magnitude of subsurface water fluctuations that might occur following periods of inclement weather.

Water can be encountered above any of the less permeable soil or rock at this site, creating a temporary perched water condition, particularly during wet periods of the year. Water levels should be expected to fluctuate throughout the year with variations in precipitation, runoff, irrigation, site topography, utilities and the water levels in nearby surface water features and other factors not evident at the time of the field services.

These observations have been made during the course of the field exploration, as indicated on the Logs of Boring. A groundwater study has not been performed. Long-term observations would be necessary to more accurately evaluate the water levels and fluctuations. If these services are desired, Rone would be pleased to provide water level monitoring as an additional scope of services.

5 ANALYSIS AND RECOMMENDATIONS

Analyses and recommendations presented in this report are based on data collected during the field and laboratory phases of the study, as well as our experience and local knowledge of the general site vicinity. The following paragraphs discuss the findings for the subject site, and options for foundations and subgrade improvement.

5.1 Seismic Site Class

The site class for seismic design is based on several factors that include soil profile (soil or rock), shear wave velocity, density, relative hardness, and strength, with quantified values averaged over a depth of 100 feet. The borings for this project did not extend to a depth of 100 feet; therefore, we assumed the soil and rock conditions below the depth of the borings to be similar to those encountered at the termination depth of the borings. Based on Section 1613.3.2 of the 2015 International Building Code and Table 20.3-1 of ASCE 7-10, we recommend using **Site Class C (dense soil and soft rock)** for seismic design.

5.2 Potential Vertical Rise

Potential Vertical Rise (PVR) calculations were performed in general accordance with the Texas Department of Transportation (TxDOT) Method 124-E. This method is empirical and is based on the Atterberg limits and moisture content of the subsurface soils. Using the TxDOT method within a 12-foot deep active zone in a dry moisture condition and assuming maximum cuts or fills of up to about 2 feet, the estimated PVR ranges from approximately 2 inches to 4 inches.

At the time of our field exploration, the sampled soils at the site ranged from a slightly moist to slightly dry moisture condition. Results of free swell tests are reported on Plate A.16 and range between approximately -0.3 and 2.8 percent. Negative swell results indicate slight consolidation under the applied overburden load.

Based on the estimated PVR using the TxDOT method, we recommend that a PVR of 4 inches be adopted for design. The variability of soil types in conjunction with the comparatively small sample size prevented zoning the site by building. The estimated PVR should be used in developing recommendations and design parameters for all buildings equally.

5.3 Excavation Safety Considerations

Please note that in accordance with Texas State Law, the design and maintenance of excavation safety systems is the sole responsibility of the contractor. Please reference OSHA Standards 29 CFR – 1926 Subpart P, including Appendices A and B, for guidance in the design of such systems.

5.4 Foundation Recommendations

Based on the conditions encountered in our borings and anticipated loading conditions, the proposed apartment buildings may be supported by slab-on-grade foundation systems, provided the estimated soil movements can be tolerated. The following recommendations have been prepared with these considerations in mind.

The proposed apartment buildings may employ ground supported foundations consisting of a post-tensioned slab foundation system, provided some floor movements can be tolerated. A PVR up to approximately 4 inches is possible at this site, and subgrade improvement will be required to reduce the PVR to the desired level of 1 to 2 inches. The foundations should be designed with exterior and interior grade beams adequate to provide sufficient rigidity to the foundation system to sustain the vertical soil movements expected at this site.

A net allowable soil bearing pressure of 1,500 psf may be used for design of all grade beams bearing in moisture conditioned soils. Grade beams should bear at least 18 inches below final grade.

The bottom of the beam trenches should be free of any loose or soft material prior to the placement of the concrete. All grade beams and floor slabs should be adequately reinforced with steel to minimize cracking as normal movements occur in the foundation soils. Moist soil conditions should be maintained within at least 5 feet of the foundation during their service life.

The PTI parameters are calculated based on the method described in the Post-Tensioning Institute (PTI) manual, 3rd edition, for designing slab-on-grade foundation systems. Recommended PTI foundation design parameters for a Thornthwaite Moisture Index (TMI) of -13 is as follows:

Table 1: PTI Foundation Design Parameters

Parameter	Condition	1-inch PVR	2-inch PVR
Edge Moisture Variation Distance, e_m (feet)	Center Lift	7.0	
	Edge Lift	3.0	
Differential Soil Movement, y_m (inches)	Center Lift	0.8	1.2
	Edge Lift	1.0	1.5

The Post Tensioning Institute (PTI) method incorporates numerous design assumptions associated with the derivation of variables needed to estimate the foundation design criteria. The PTI method of predicting differential soil movement is applicable when site moisture conditions are controlled by the climate alone on well-graded building pads (i.e. proper drainage, properly lined landscaped areas, no utility water leaks or other free water sources). As soil moisture increases, the soils may swell. The PTI design method is intended to provide stiffened foundation systems that can perform well under typical natural changes in soil moisture. The differential foundation movements resulting from seasonal soil moisture content changes are typically much lower than movements that occur due to free water sources near or beneath the structure, which are not directly addressed by the PTI design method.

5.5 Subgrade Treatments to Reduce Soil Movement

When considering the various treatment options, it is important to keep in mind that the subsurface conditions which resulted in the calculated PVR values may not be uniformly present within the building footprint, particularly when the subsurface conditions are variable. Some allowance for variable support should be incorporated in the slab design.

Reworking of the existing subgrade is performed to increase the moisture levels of the soils to a level that reduces their ability to absorb additional water that could result in post-construction heave. In order to achieve a design PVR of 1 to 2 inches, subgrade treatment should consist of excavating the subgrade soils to the depth from final pad elevations as indicated in the table below, and replacing them with moisture and density control. The moisture conditioned fill should be brought to within 6 to 12 inches of the final pad elevation, covered with at least 6 mil plastic sheeting, then the final soils may be placed atop the plastic sheeting.

The reworked soils should extend at least 5 feet outside the perimeter of the proposed structures or other perimeter features sensitive to differential movement. Some post-construction drying and settlement of the fill should be expected.

Table 2: Moisture Conditioning Depth, Feet

Target PVR After Treatment, Inches	
2	1
5	8

The subgrade should be excavated to the recommended depth below the final pad elevation. Any deleterious materials or rock fragments greater than 4 inches in diameter encountered within the soils should be removed. The subgrade to receive moisture conditioned clay should be scarified to a depth of 6 inches and compacted to 92 to 96 percent of the material's standard Proctor dry density (ASTM D698) at a moisture content at least 3 percent above optimum. In order to achieve a more uniform soil moisture profile, the moisture treated fill should be placed in maximum 8-inch loose lifts and compacted to a similar density and moisture content. Following fill placement, a plastic membrane at least 6-mil thick should be installed approximately 6 to 12 inches below the final top of pad elevation.

The treated subgrade materials are prone to drying out after the treatment process is complete. The treated subgrade materials should be kept moist prior to slab concrete placement.

Moisture conditioned clay subgrade should be monitored and tested on a full-time basis by Rone Engineers to confirm conditions are as anticipated and to confirm the fill is suitable and placed with the proper moisture content and degree of compaction. Density tests should be performed on each lift of reworked clay.

5.6 Swimming Pool

We understand a swimming pool is planned within the complex. The pool and pool deck area are subject to the same PVR discussed earlier in this report. Subgrade improvement will be required to reduce the PVR to 2 inches or less.

Subgrade improvement below the pool should extend to the depth from the final grades to match the building structures (ie: if the pool is 5 feet deep, moisture treatment should extend 0 to 3 feet below the 5-foot pool depth and to the full 5- or 8-foot depth from final grades under the surrounding pool deck).

The walls of the pool will be subjected to lateral earth pressures due to the materials being retained and drainage conditions. We recommend the backfill consist of free-draining sand or gravel with a drainage system at the bottom of the wall to reduce hydrostatic pressures on the walls. The pool should be designed using the equivalent fluid pressures presented in Table 2.

5.7 Detention/Retention Pond

In general, the purpose of a detention pond is to temporarily store rainfall runoff and release the water at a controlled rate. Depending on the site, project, global stability considerations, environmental regulations and owners' expectations, the detention pond may need to have a low rate of infiltration into the ground.

If permanent water storage is desired (retention pond), a pond lining will be required. The lining may consist of properly placed on-site clay soils, imported clay soils, a synthetic liner, or a combination of these.

A hydraulic conductivity (permeability) study was not included in our scope of services; therefore, the following recommendations should be considered as general guidelines for water retention, and not as an assurance that the pond will consistently hold water. Rone would be pleased to provide these as additional services if the Owner desires.

If water retention is desired, clay liner material should have a liquid limit greater than 60 and a plasticity index greater than 45. The liner material should be placed in maximum 8-inch loose lifts and compacted to between 92 and 96 percent of the maximum dry density as determined by standard Proctor test (ASTM D698) and at a minimum of 4 percent above the soil's optimum moisture content (min +4%). Please note that the liner will be subject to normal shrink/swell behavior unless water is maintained within the pond, or the pond is irrigated sufficiently to prevent the liner from drying and experiencing the formation of shrinkage cracks.

Should water retention be less of a concern, the backfill should be placed in maximum 8-inch loose lifts and compacted to between 95 and 100 percent of the maximum dry density as determined by standard Proctor test (ASTM D698) and within 2 percent of the soil's optimum moisture content (-2% to +2%). The compacted thickness of the liner should be at least 2 feet. The pond slopes should be 4H:1V (horizontal to vertical) or flatter to reduce the risk of sloughing of the pond liner.

6 LATERAL EARTH PRESSURES

The current site plans do not indicate that retaining walls are planned, although retaining walls may be constructed at this site. The following paragraphs provide general guidance for the construction of below grade walls. Global stability analysis (GSA) may be required for walls that are greater than 5 feet in height and/or for walls that are subjected to surcharge loads. Our office should be provided with a copy of grading plans to assess the need for a global stability analysis.

Below grade walls will be subjected to lateral earth pressures from earth backfill. Lateral earth pressures will be influenced by structural design, conditions of the wall restraint, methods of construction and/or compaction, the type of materials being retained, and drainage conditions. Walls that will be restrained from movement and rotation (rigid wall) should be designed for an at-rest earth-pressure condition. The equivalent fluid pressures (triangular distribution) provided may be used for the horizontal backfill in a drained condition. To design for a drained condition, the wall must include an adequate drainage system. The provided equivalent fluid pressures do not include a Factor of Safety and do not provide for hydrostatic or dynamic pressures on the wall.

Lateral Earth Pressures

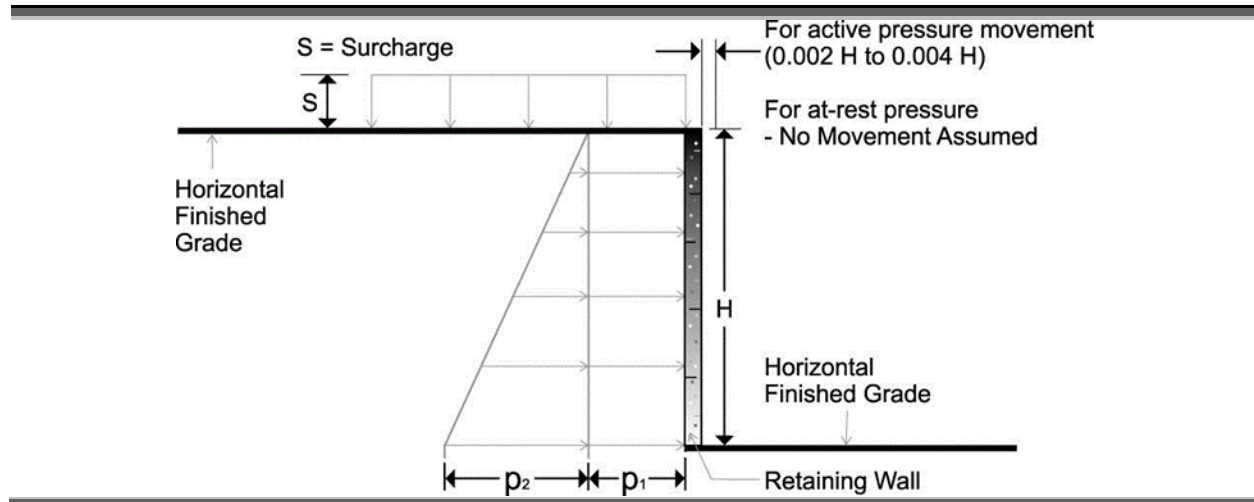


Table 2: Lateral Earth Pressures

Material	Condition	Equivalent Fluid Pressure, pcf	
		Drained	Undrained
Free Draining Granular Soil	At-Rest, $k = 0.45$	55	90
On-Site Clay Soil	At-Rest, $k = 0.79$	--	112

Conditions applicable to Table 2 include:

- Uniform surcharge
- A maximum in-situ total unit weight of 125 pcf
- Properly compacted, horizontal backfill
- No additional loading from heavy equipment
- No loading from nearby pavements, footings, slabs, etc.
- Adequate drainage (ie. no hydrostatic pressures)

The values provided are for a full “wedge” of material behind the wall, where the backfill extends horizontally 1 to 2 feet away from the bottom of the wall and then slopes upward and away from the wall at a slope of 1:1, or flatter.

The location and magnitude of permanent surcharge loads (if present) should be determined. Additional pressures generated by these loads, such as the weight of construction equipment and vehicular loads, must also be considered in the design. Surcharge loads can be factored using the appropriate earth pressure coefficient values provided in table above.

6.1 Wall Drainage

Below grade walls should be expected to collect water due to condensation, surface water infiltration and other means. Positive drainage should be provided behind all below grade walls to reduce the development of hydrostatic pressure and limit saturation of the backfill and foundation soils. Collector pipes should be placed at or slightly below the bottom level of the swimming pool to prevent the collection of water in the drainage material beneath the collector pipes. Pipes should connect to a sump or gravity drainage system to prevent the accumulation of water behind the walls. Gravity lines should include a backflow preventer to block water from being transmitted into the drainage layer in the event of flooding near the gravity outfall.

The drainage material should consist of free-draining, clean, granular fill. This material should be compatible with ASTM C33, sizes 4 through 9. The drainage layer should extend at least 12 inches from the back face of the wall. A geosynthetic wrap should enclose the granular backfill to reduce the infiltration of fines. The top 2 feet of backfill should consist of clay materials with a plasticity index of 25 or more, compacted to at least 95 percent of standard Proctor test (ASTM D698), at a moisture content of at least three percent (+3%) above the optimum moisture content and, extend at least 5 feet beyond the wall excavation limits to reduce surface water infiltration into the underlying fill.

6.2 Wall Backfill

Free-draining backfill soils should be placed in maximum lifts of 1 foot and lightly consolidated by use of a vibrating plate or sled, light hand-held compactors, or other appropriate methods to adequately compact the backfill. If onsite clayey soils are used, these materials should be placed in maximum 6-inch lifts and properly compacted to between 92 and 96 percent of the maximum dry density, as determined by standard Proctor test (ASTM D698), and at a moisture content of at least four percent (+4%) above the optimum moisture content. Heavy compactors and grading equipment should not be allowed to operate within 15 feet of the crest of the wall to avoid developing excessive additional temporary or long-term lateral soil pressures.

7 PAVEMENTS

We understand the portland cement concrete (PCC) is planned for this site. Recommendations for PCC pavement are provided in this report. When designing proposed pavement sections, subgrade conditions must be considered, along with expected traffic use/frequency, pavement type and design period.

7.1 Pavement Design

For this project, traffic loading and frequency conditions were estimated for various conditions as no specific traffic information was provided. The following information was used in our analysis:

- 35,000 annual equivalent single axle load (ESAL) repetitions for residential streets;
- Negligible traffic growth for residential streets;
- Poor to fair drainage; Cd = 1.0;
- A reliability of 85 percent for residential streets;
- A concrete modulus of rupture of 530 psi;
- A 28-day compressive strength of 3,500 psi
- A design life of 20 years;
- Initial serviceability, po, of 4.5, and a terminal serviceability, pt, of 2.0;
- A k-value of 150 pci for lime-treated subgrade.

The pavement thickness determinations were performed in general accordance with the “1993 AASHTO Guide for the Design of Pavement Structures” guidelines. The minimum pavement sections are presented in the table below. These pavement sections are based on estimated traffic volumes. A more precise design can be made with detailed traffic loading information.

Table 3: Concrete Pavement Sections

Roadway	Lime Treated Subgrade Thickness (inches)	Concrete Thickness (inches)
General Site Paving and Parking	6	6
Streets/Fire Lanes / Dumpster Pads	6	7

Note: Please refer to local municipal requirements for pavements. Use the design criteria which will result in the stronger, more durable pavement section.

The concrete minimum 28-day compressive strength should be selected based on the expected traffic. We recommended minimum compressive strengths of 3,600 psi and 4,000 psi at 28 days in residential car/truck traffic areas and fire lanes and dumpster pads respectively. As a minimum, reinforcing steel should consist of #3 bars spaced at a maximum of 18 inches on center in each direction.

Pavement recommendations are based on the estimated loading conditions and commonly accepted design procedures that should provide satisfactory performance for the design life of the pavement. Concrete pavement should have between 4 and 6 percent entrained air. Hand-placed concrete should have a maximum slump of 5 inches. All steel reinforcement, dowel spacing/diameter and pavement joints should conform to applicable city standards.

Saw cutting should be performed in specified locations to control cracking due to shrinkage. Saw cutting should begin as soon as the concrete has obtained enough strength to keep from raveling, but before significant cracks have initiated internally. Saw cut depths generally range from $\frac{1}{4}$ to $\frac{1}{3}$ of the pavement thickness, but should be performed as directed by the civil engineer.

7.2 Pavement Subgrade Preparation

All topsoil, vegetation, and any unsuitable materials should be removed. The pavement subgrade should be proofrolled with a fully loaded (40,000 lbs.) tandem axle dump truck or similar pneumatic-tire equipment to locate areas of loose subgrade. In areas to be cut, the proofroll should be performed after the final grade is established. In areas to be filled, the proofroll should be performed prior to placement of engineered fill and after subgrade construction is complete. Areas of loose or soft subgrade encountered in the proofroll should be removed and replaced with engineered fill, or moisture conditioned (dried or wetted, as needed) and compacted in place.

Lime is commonly used for treating clay soils in this area. It is estimated that at least 7 percent hydrated lime by dry weight (32 pounds per square yard) will be required to treat the existing soils. The actual lime requirement and sulfate levels should be determined after the pavement subgrade has reached final grade. Lime treatment should be performed in accordance with Item 260, current Standard Specifications for Construction of Highways, Streets, and Bridges, Texas Department of Transportation (TxDOT) or applicable standards. It is not necessary to treat 5- or 6-inch pavement subgrade with lime if subjected to light traffic only.

The lime treated subgrade should have a plasticity index between 5 and 15, be compacted between 95 and 100 percent of standard Proctor maximum dry density (ASTM D698) at a moisture content at or above the optimum moisture content (opt +).

It should be understood that lime treating the upper 6 inches of the subgrade soils will not significantly reduce the normal shrinking and swelling of the subgrade which occurs with seasonal moisture fluctuations. Some differential vertical movements of the pavements should be expected. Lime treatment will, however, provide a working platform during construction and create a less erodible subgrade for pavement support. This will reduce the potential for voids to develop beneath the pavement, and decrease the risk of pavement distress and possible failure.

The treated subgrade should extend a minimum of 2 feet outside the curb line. This will improve the edge support of the pavement and lessen the edge effect associated with shrinkage during dry periods. Granular fill should not be used as a leveling course beneath the pavement as these more porous soils allow water inflow between the pavement and subgrade causing heave and strength loss of the subgrade. Utility trenches that lie beneath the pavement must be properly compacted prior to the treatment of the pavement subgrade.

7.3 Pavement Construction and Maintenance Recommendations

It is crucial that the moisture content and compaction be maintained until the concrete is placed. If the treated subgrade is allowed to dry prior to the concrete placement, the risk of shrinkage cracks within the PCC surface is greatly increased.

Proper drainage should be provided both during and after construction. The pavement surface should be contoured such that surface water drains off, away from the pavement and into inlets. Water allowed to pond on or adjacent to pavement surfaces will saturate the subgrade soils leading to premature pavement failure. Additionally, emphasis should be given to areas where the pavements are placed directly adjacent to entries. If the subgrade heaves, the pavement could slope toward the building, causing drainage issues that could impede doors opening and closing and create building access/evacuation issues.

In order to reduce potential differential movement across the pavements resulting from infiltration of surface water, all joints should be adequately sealed. Maintenance should include a regular observation schedule to identify and seal cracks. A flexible joint material should be used to seal cracks as they degrade, which can occur during the design life of pavements.

8 SITE PREPARATION AND FILL PLACEMENT

The following recommendations for site preparation and fill placement may contain elements that do not appear to apply to the presently known conditions at the project site. These items have been included since our experience has been that unforeseen obstacles are encountered on some project sites, and progress can be delayed while written guidance is prepared. While we cannot cover every possible circumstance, we have attempted to address the most frequently occurring issues in this report section.

8.1 General

All grade-supported slabs should be designed to accommodate anticipated vertical movements as presented in section **5.2 Potential Vertical Rise** earlier in this report.

Every attempt should be made to limit the extreme wetting or drying of the subsurface soils because swelling and shrinkage of these soils will result. Standard construction practices of providing good surface water drainage should be used. All grading should provide positive drainage away from paving and should prevent water from collecting near the edge of pavements and structures. Also, ditches or swales should be provided to carry the run-off water both during and after construction. Lawn areas should be watered moderately, without allowing the clay soils to become too dry or too wet. Roof runoff should be collected by gutters and downspouts and should discharge away from the building.

Backfill for utility lines or along the perimeter beams should consist of site-excavated soil. If the backfill is too dense or too dry, it can swell and a mound will form along the trench line. If the backfill is too loose or too wet, it can settle and a depression will form along the trench line. All fill should be placed and compacted as recommended in **Table 4: Fill Placement Criteria** below.

Table 4: Fill Placement Criteria

Item	Description	Plasticity Requirement	Compaction Standard	Density Requirement	Moisture Requirement
On-site soils	General grading	None	ASTM D698	95% to 100% of maximum dry density	Optimum moisture to 3% above optimum moisture
Imported general fill	General grading	Liquid Limit less than 50	ASTM D698	95% to 100% of maximum dry density	Optimum moisture to 3% above optimum moisture
Utility backfill On-site soils	0-10' below grade	None	ASTM D698	95% to 100% of maximum dry density	At least 2% above optimum moisture
	>10' below grade	None	ASTM D698	Minimum 100% of maximum dry density	Minus 2% to plus 2% of optimum moisture
Moisture conditioned on-site soils	Structural fill	None	ASTM D698	92% to 96% of maximum dry density	At least 3% above optimum moisture
Select fill (soils)	Structural fill	$5 \leq PI \leq 15$; $LL \leq 35$	ASTM D698	95% to 100% of maximum dry density	Minus 2% to plus 2% of optimum moisture
Lime Treated subgrade	Pavement support	$5 \leq PI \leq 15$	ASTM D698	95% to 100% of maximum dry density	Minus 2% to plus 2% of optimum moisture
Exterior grade beam backfill	Building pad	On-site clays	ASTM D698	92% to 96% of maximum dry density	At least 4% above optimum moisture
Pavement fill	>10' below grade	On-site clays	ASTM D698	98% to 100% of maximum dry density	Optimum plus
	0-10' below grade	On-site clays	ASTM D698	95% to 100% of maximum dry density	Optimum plus

If granular material is used for embedment in utility trenches, we recommend placing a clay plug as a replacement for the granular embedment. The clay plug should be at least 4 feet in length, centered at the building perimeter and should fill the vertical and horizontal dimensions of the utility trench. The intent is to prevent free moisture from passing through the granular fill and entering the soil beneath the structure.

All excavations should be sloped, shored, or shielded in accordance with OSHA requirements.

8.2 Site Preparation

Preparation of the site for any future construction should include the removal and proper disposal of any obstructions that would hinder construction. These obstructions should include all abandoned structures, foundations, debris, water wells, septic tanks and loose material. It is the intent of these recommendations to provide for the removal and disposal of all obstructions not specifically provided for elsewhere by the plans and specifications.

In general, we recommend that all active utilities that would extend beneath any structure and are not intended to provide service to the structure, be rerouted around the structure footprint. Any abandoned lines should be removed and disposed of properly. All abandoned utilities within the structure footprint that are not removed represent a risk to future building performance; if the lines are abandoned in place, they must be fully grouted and capped so that the pipes do not provide a ready conduit for water.

This study was not performed to evaluate the rippability or excavatability of the subsurface materials at this site, or for use in estimating the number of trucks needed to haul away excavation spoils based on the expected volume of excavated materials. The contractor must use his or her own experience in the area of this site when forming conclusions regarding appropriate means and methods to accomplish the planned construction, specifically including excavation tools, excavation rates, and number of trucks. The selected contractor should have experience in construction and excavation in the observed materials and vicinity of the project site.

All concrete, trees, stumps, brush, abandoned structures, roots, vegetation, rubbish and any other undesirable matter should be removed and disposed of properly. It is the intent of these recommendations to provide a loose surface with no features that would tend to prevent uniform compaction by the equipment to be used.

All areas to be filled should be disced or bladed until uniform and free from large clods. Soils should be brought to the proper moisture content and compacted as indicated in **Table 4: Fill Placement Criteria**.

8.2.1 Select Fill

Select fill should consist of a clean, natural soil meeting the criteria listed in Table 4. The fill should have a moisture content within the specified range, be placed in loose lifts less than 9 inches thick, and compacted as indicated above. Lime treated, on-site soils may also be used as the select fill cap, provided the PI of the material meets the specifications for select fill. The quantity of lime needed to achieve the PI requirement for select fill is not known. The actual percentage of lime should be determined once soils have been stockpiled and sampled.

Recycled concrete or processed rock can also be used as select fill, provided the material meets the criteria listed in Table 4. The material should have a moisture content within the specified range, be placed in loose lifts less than 9 inches thick, and compacted as indicated in Table 4.

The fill material should be placed in level, uniform layers, which, when compacted, should have a moisture content and density conforming to the stipulations called for herein. Each layer should be thoroughly mixed during spreading to provide uniformity of the layer.

8.2.2 Site Grading

Site grading operations should be performed in accordance with the recommendations in this report. The site grading plans and construction should strive to achieve positive drainage around all proposed structures and pavements. Inadequate drainage around structures built on-grade can cause excessive vertical differential movements to occur.

8.2.3 Utility Backfill

If on-site clayey soils are used as backfill, these materials should be placed in maximum 6-inch lifts and properly compacted to between 95 and 100 percent of the maximum dry density, as determined by standard Proctor test (ASTM D698), and at a moisture content of at least two percent (min +2%) above the soil's optimum moisture content.

In instances where utility lines are more than 10 feet deep, the backfill below 10 feet should be compacted to 100 percent of the maximum dry density, as determined by the standard Proctor test (ASTM D698), and at a moisture content of within two percent (-2 to +2%) of the soil's optimum moisture content.

Properly placed and compacted clay fill will typically experience settlement on the order of. On the order of 1 to 2 percent of the fill height. This should be considered when designing utility lines beneath pavements, flatwork or any structure.

8.2.4 Density Tests

Field density tests should be performed by the geotechnical engineer or his representative. Density tests should be taken in each layer of compacted fill below the disturbed surface. If the materials fail to meet the density specified, the course should be reworked as necessary to obtain the specified moisture content and compaction.

The specified moisture content and compaction must be maintained until placement of the overlying lift, or construction of overlying flatwork. Failure to maintain the moisture content and compaction could result in excessive soil movement and can have a detrimental effect on overlying structures such as shallow foundations and floor slabs. The contractor must provide some means of controlling the moisture content and compaction (such as water hoses, water trucks, etc.). Maintaining subgrade moisture and compaction is always critical, but will require extra effort during warm, windy and/or sunny conditions. Density and moisture testing is recommended to provide some indication that adequate earthwork is being provided; however, the quality of the fill is the sole responsibility of the contractor. Satisfactory testing is not a guarantee of the quality of the contractor's earthwork operations.

8.3 Landscaping

We do not recommend the use of landscaping against and around the exterior of the foundations, as landscaped areas can adversely affect subgrade moisture. Landscaped areas can create both saturated and desiccated conditions that cause localized differential movements and the formation of cracks. If used, landscaping should be kept as far away from the foundation as possible and positive drainage must be maintained.

Landscaping elements (such as edging) must not prohibit or slow the drainage of water. When feasible, irrigation lines and heads should not be placed in close proximity to building foundations to prevent the collection of water near the foundation or flatwork, particularly in the event of leaking lines or sprinkler heads.

Trees should not be placed in proximity to the structure or movement sensitive flatwork, as trees are known to cause in localized soil shrinkage due to desiccation of the soil by the root system. This would result in localized differential settlement. The desiccation zone varies by tree size and species, but trees should generally set back at least 1½ times the mature tree height, and in no case should the drip-line of the mature tree extend over or within 15 feet of structures, including the swimming pool.

8.4 Construction Observations

In any geotechnical study, 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, anomalies in the subsurface conditions are quite often revealed during construction. The potential for the presence of varied geologic formations and significantly different support conditions at this site, which could result in changes in our design recommendations, increases the risk of damaging soil movements at this site. It is recommended that Rone be retained to observe earthwork operations and foundation construction, and perform materials evaluation and testing 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.

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 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.

9 STUDY CLOSURE

The analyses, conclusions and recommendations contained in this report are based on site conditions as they existed at the time of the field exploration 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, 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 study to determine the applicability of the conclusions and recommendations, considering the changed conditions and/or time lapse.

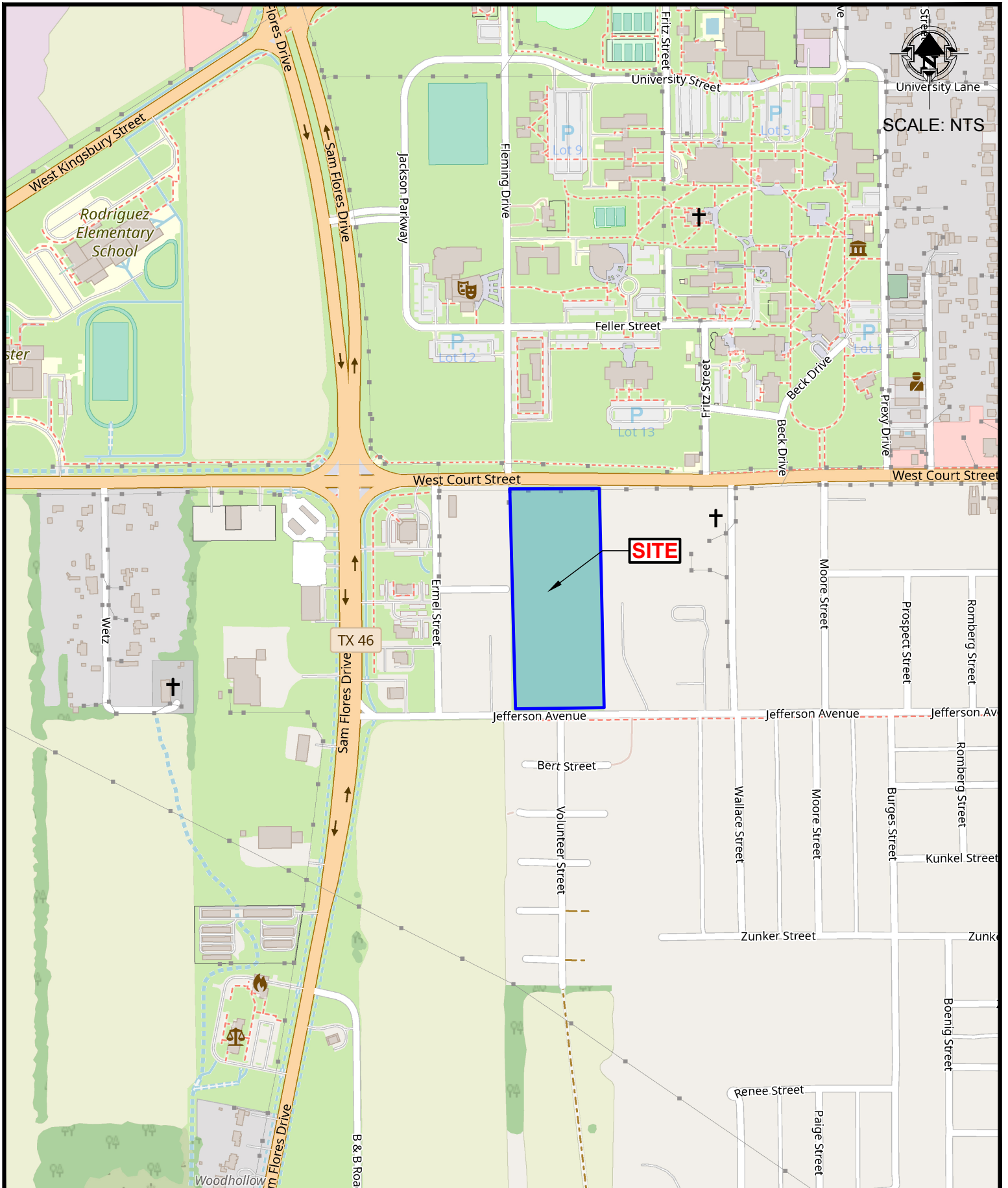
Further, it is urged that Rone be retained to review those portions of the plans and specifications for this particular project that pertain to earthwork and foundations as a means to determine whether the plans and specifications are consistent with the recommendations contained in this study. In addition,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

This study has been prepared for the exclusive use of the client and their designated agents for specific application to design of this project. 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 warranty, expressed or implied, is made or intended.

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APPENDIX A

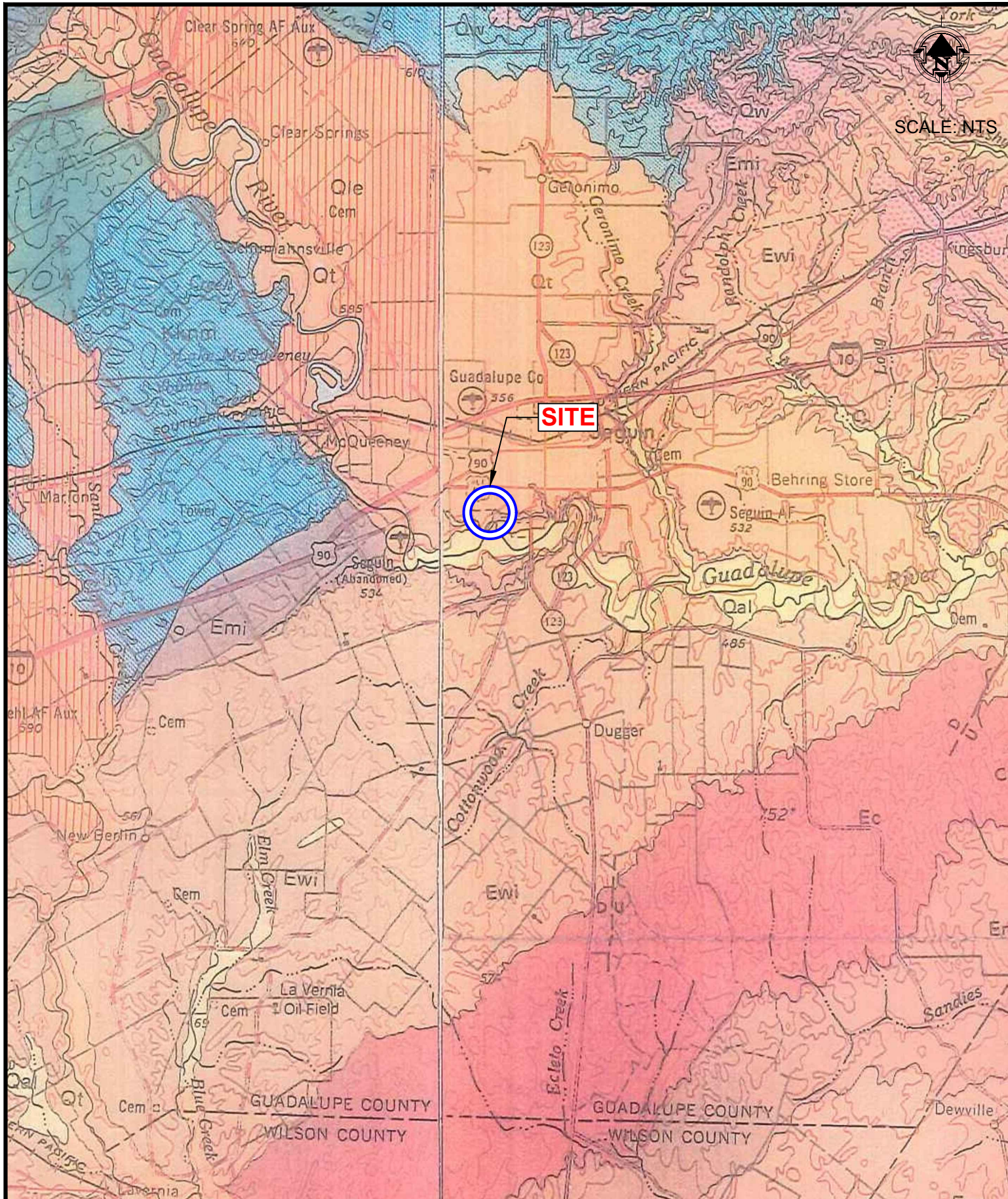


RONE
ENGINEERING

PLATE A.1
VICINITY MAP

COURTSIDE MULTI-FAMILY
JEFFERSON AVE. & WEST COURT STREET
SEGUN, TEXAS

PROJECT NO:	19-23544	
FILE NAME:	1923544.DWG	
DRAWN BY:	CM	DATE: 6-26-2019
REVISED BY:		DATE:
REVISED BY:		DATE:
APPROVED BY:	BT	DATE: 6-26-2019



RONE
ENGINEERING

PLATE A.2
GEOLOGY MAP

COURTSIDE MULTI-FAMILY
JEFFERSON AVE. & WEST COURT STREET
SEGUIN, TEXAS

PROJECT NO:	19-23544	
FILE NAME:	1923544.DWG	
DRAWN BY:	CM	DATE: 6-26-2019
REVISED BY:		DATE:
REVISED BY:		DATE:
APPROVED BY:	BT	DATE: 6-26-2019



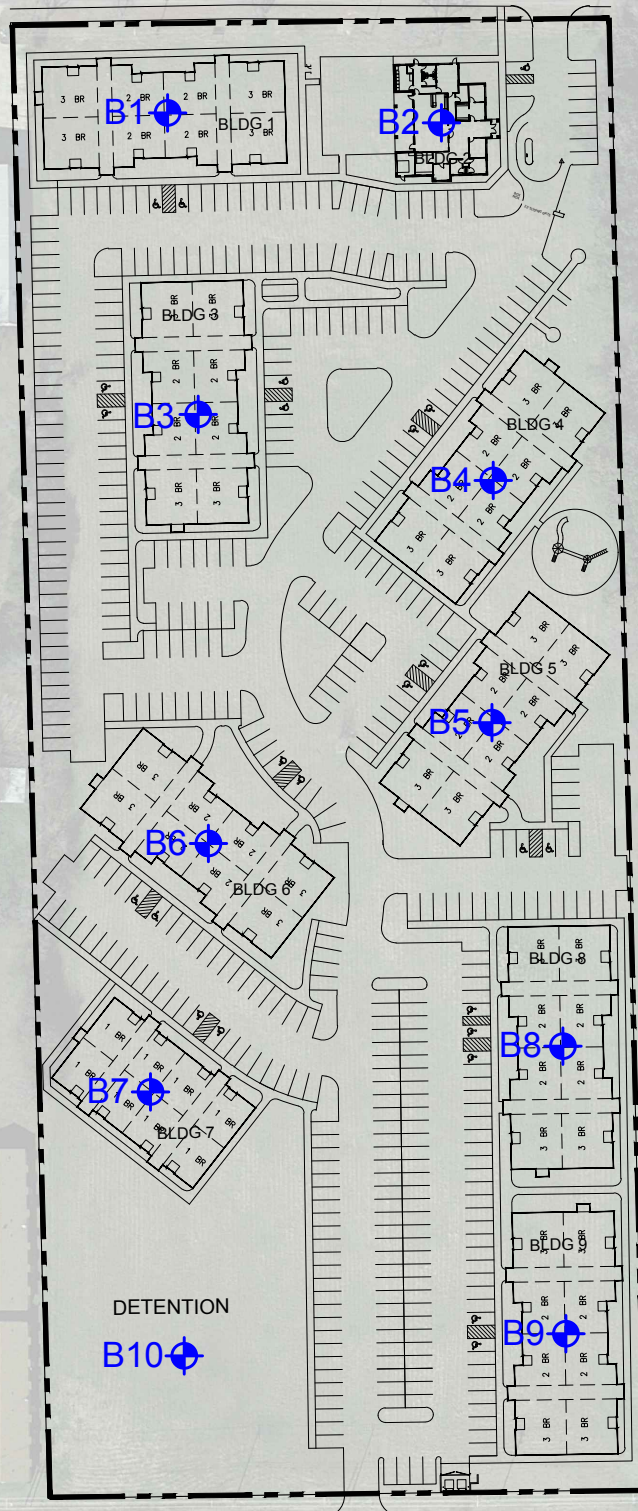
SCALE: NTS

WEST COURT ST. (HWY. 90 ALT.)

MARKGRAF ST.

DETENTION

JEFFERSON AVE.




RONE
ENGINEERING

PLATE A.3
BORING LOCATION DIAGRAM


COURTSIDE MULTI-FAMILY
JEFFERSON AVE. & WEST COURT STREET
SEGUIN, TEXAS

PROJECT NO:	19-23544	
FILE NAME:	1923544.DWG	
DRAWN BY:	CM	DATE: 6-26-2019
REVISED BY:		DATE:
REVISED BY:		DATE:
APPROVED BY:	BT	DATE: 6-26-2019






This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Log B-1		Project No. 19-23544		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas										
Boring Location Building 1														
Latitude 29.56899° N		Water Level Observations (feet)		Date 5-15-19										
Longitude 97.98615° W		While Drilling		Not Observed										
		At Boring Completion		Not Observed										
		End of Day		Not Measured										
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description		Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf
				Approximate Surface Elevation = 534.0 feet										
				LEAN CLAY WITH SAND (CL) - brown, trace calcareous nodules				1.50			35			
								2.50			30			
5								4.50	70	47-19-28	17	0.0	101	
				- reddish/brown, with calcareous nodules, deposit and pieces				4.50			21			
10								4.00			17			
				- tan with calcareous pieces										
15								1.50			20			
								1.00			21			
20			514.0	Boring Terminated at Approximately 20 Feet										
Material boundaries are approximate; in situ, transitions may be gradual.														

Driller: DAS
Drilling Method: Continuous Flight Augers


Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div></div>						
Boring Location		19-23544												
Building 2														
Latitude		Water Level Observations (feet)				Date								
29.56899° N		While Drilling		Not Observed		5-15-19								
Longitude		At Boring Completion		Not Observed										
97.98561° W		End of Day		Not Measured										
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description		Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf
				Approximate Surface Elevation = 534.0 feet										
				FAT CLAY (CH) - brown with sand and calcareous nodules				3.00			30			
				- reddish brown with calcareous nodules and pieces				4.5+	85	70-22-48	22	0.8	103	
5								4.00			20			
								4.50			18			
			526.0	SANDY LEAN CLAY (CL) - tan				4.50			15			
10														
								3.00			14			
15														
								2.50			13			
20			514.0	Boring Terminated at Approximately 20 Feet										
Material boundaries are approximate; in situ, transitions may be gradual.														
Driller: DAS Drilling Method: Continuous Flight Augers														
Plate A.5														


This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div></div>								
Boring Location		19-23544														
Building 3																
Latitude		Water Level Observations (feet)				Date		5-15-19								
29.56844° N		While Drilling		Not Observed												
Longitude		At Boring Completion		Not Observed												
97.98610° W		End of Day		Not Measured												
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description				Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf
				Approximate Surface Elevation = 535.0 feet												
				FAT CLAY (CH) - brown, trace calcareous nodules						1.50			32			
										4.50			25			
5										2.50			30			
				- reddish brown with calcareous deposit and pieces						4.50	68	50-19-31	18	0.1	102	
										4.5+			20			
10			525.0	SANDY LEAN CLAY (CL) - tan												
									5-7-9 N=16				14			
15																
									8-11-12 N=23				18			
20			515.0	Boring Terminated at Approximately 20 Feet												
Material boundaries are approximate; in situ, transitions may be gradual.																

This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Driller: DAS
Drilling Method: Continuous Flight Augers


Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div></div>								
Boring Location		19-23544														
Building 4																
Latitude		Water Level Observations (feet)				Date										
29.56829° N		While Drilling		Not Observed		5-15-19										
Longitude		At Boring Completion		Not Observed												
97.98547° W		End of Day		Not Measured												
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description				Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf
				Approximate Surface Elevation = 533.0 feet								LL-PL-PI				
				LEAN CLAY WITH SAND (CL) - brown, trace calcareous nodules						2.00			30			
										4.5+			23		102	12,210
5										4.5+			17			
				- reddish brown, with calcareous deposits and pieces						4.50			15			
										4.00	83	32-16-16	16	-0.3	105	
10																
				- tan, calcareous, with calcareous pieces						7-12-14 N=26			20			

Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div></div>								
Boring Location		19-23544														
Building 5																
Latitude				Water Level Observations (feet)				Date								
29.56782° N				While Drilling		Not Observed		5-15-19								
Longitude				At Boring Completion		Not Observed										
97.98546° W				End of Day		Not Measured										
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description				Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf
				Approximate Surface Elevation = 533.0 feet												
				LEAN CLAY (CL) - brown, trace calcareous nodules						2.50			29			
										4.50			23			
5										4.50			23			
				- reddish brown, with calcareous pieces						4.5+			17			
										4.00	98	46-20-26	19	-0.1	106	
10																
				- tan, with sand					5-9-14 N=23				15			
									7-12-10 N=22				20			
20			513.0	Boring Terminated at Approximately 20 Feet												
Material boundaries are approximate; in situ, transitions may be gradual.																

This boring log should not be considered valid if separated from the remainder of the geotechnical report.


Driller: DAS
Drilling Method: Continuous Flight Augers

Plate A.8

Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div></div>						
Boring Location		19-23544												
Building 6														
Latitude		Water Level Observations (feet)				Date		5-15-19						
29.56759° N		While Drilling		Not Observed										
Longitude		At Boring Completion		Not Observed										
97.98608° W		End of Day		Not Measured										
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description		Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf
				Approximate Surface Elevation = 532.0 feet						LL-PL-PI				
				FAT CLAY (CH) - dark brown, trace calcareous nodules				1.50			32			
				- medium brown with calcareous nodules and sand				4.5+	86	64-25-39	22	2.8	103	
5								4.5+			22			
								4.50			16			
			524.0	SANDY LEAN CLAY (CL) - tan, with calcareous deposits				4.50			18			
10														
								3.50			18			
15														
								4.5+			18			
20			512.0	Boring Terminated at Approximately 20 Feet										
Material boundaries are approximate; in situ, transitions may be gradual.														

This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Driller: DAS
Drilling Method: Continuous Flight Augers

Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div></div>							
Boring Location		19-23544													
Building 7															
Latitude		Water Level Observations (feet)				Date									
29.56711° N		While Drilling		Not Observed		5-15-19									
Longitude		At Boring Completion		Not Observed											
97.98621° W		End of Day		Not Measured											
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description		Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf	
				Approximate Surface Elevation = 533.0 feet											
				FAT CLAY (CH) - dark brown, trace calcareous nodules				3.00			30				
				- brown, trace calcareous nodules				4.5+			24				
5								4.5+	88	74-25-49	24	0.9	95		
								4.5+			19				
			524.0	SANDY LEAN CLAY (CL) - tan, with calcareous deposits				4.5+			18				
10															
								3.50			20				
15															
				- with silt											
								3.00			16				
20			513.0	Boring Terminated at Approximately 20 Feet											
Material boundaries are approximate; in situ, transitions may be gradual.															
Driller: DAS Drilling Method: Continuous Flight Augers															
Plate A.10															

This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div><div>RONE</div><div>ENGINEERING</div></div>																	
Boring Location		19-23544																							
Building 8																									
Latitude				Water Level Observations (feet)				Date																	
29.56718° N				While Drilling		Not Observed		5-15-19																	
Longitude				At Boring Completion		Not Observed																			
97.98533° W				End of Day		Not Measured																			
Depth, ft.				Stratum Description				Water Level Observations		SPT or TCP		Penetrometer Reading, tsf		Passing No. 200 Sieve %		Atterberg Limits		Moisture Content %		Swell %		Dry Unit Weight, pcf		Unconfined Compression, psf	
Sample Type																									
Elevation, ft.																									
				Approximate Surface Elevation = 532.0 feet																					
				LEAN CLAY WITH SAND (CL) - dark brown, trace calcareous nodules								2.00						35							
												4.5+						23							
5												4.5+						25							
				- reddish brown, with calcareous deposits and pieces								4.50		83		46-14-32		17		0.0		98			
												4.50						15							
10																									
				- light brown and orange-brown, with calcareous nodules and deposits								4.50						13							
												4.50						15							
20				512.0																					
				Boring Terminated at Approximately 20 Feet																					

This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Driller: DAS
Drilling Method: Continuous Flight Augers


Plate A.11

Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div><div>RONE</div><div>ENGINEERING</div></div>								
Boring Location		19-23544														
Building 9																
Latitude				Water Level Observations (feet)				Date								
29.56663° N				While Drilling		Not Observed		5-15-19								
Longitude				At Boring Completion		Not Observed										
97.98533° W				End of Day		Not Measured										
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description				Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf
				Approximate Surface Elevation = 532.0 feet								LL-PL-PI				
				LEAN CLAY WITH SAND (CL) - dark brown, trace calcareous nodules, organics						2.50			33			
										4.5+			23			
5				- brown, with calcareous pieces						4.5+			19			
										4.50			19			
				- reddish/tan, with calcareous pieces and deposits						4.5+	78	40-18-22	16	-0.2	101	
10																
				- tan					6-9-10 N=19				12			

This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Driller: DAS
Drilling Method: Continuous Flight Augers

Plate A.12

Log		Project No.		Courtside Multi-Family Jefferson Avenue and West Court Street Seguin, Texas				<div></div>									
Boring Location		19-23544															
Detention Pond																	
Latitude		Water Level Observations (feet)				Date		5-15-19									
29.56661° N		While Drilling		Not Observed													
		At Boring Completion		Not Observed													
Longitude		End of Day		Not Measured													
97.98617° W																	
Depth, ft.	Symbol	Sample Type	Elevation, ft.	Stratum Description				Water Level Observations	SPT or TCP	Penetrometer Reading, tsf	Passing No. 200 Sieve %	Atterberg Limits	Moisture Content %	Swell %	Dry Unit Weight, pcf	Unconfined Compression, psf	
				Approximate Surface Elevation = 531.0 feet													
				FAT CLAY (CH) - dark brown, trace calcareous nodules, organics						2.00			34				
										4.50			25		97	10,640	
5										4.00	92	73-28-45	26	2.0	94		
				- orange/brownish with calcareous pieces						4.50			25				
				- brown with calcareous nodules						4.00			28				
			517.5	SANDY LEAN CLAY (CL) - orange-brown, with calcareous nodules					12-17-19 N=36				21				
15																	

This boring log should not be considered valid if separated from the remainder of the geotechnical report.

Driller: DAS
Drilling Method: Continuous Flight Augers

Plate A.13



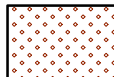
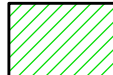

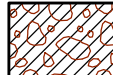
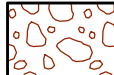





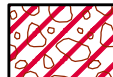
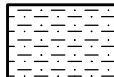

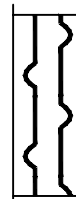

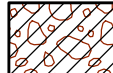
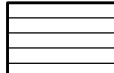
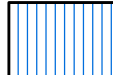
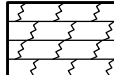

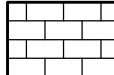

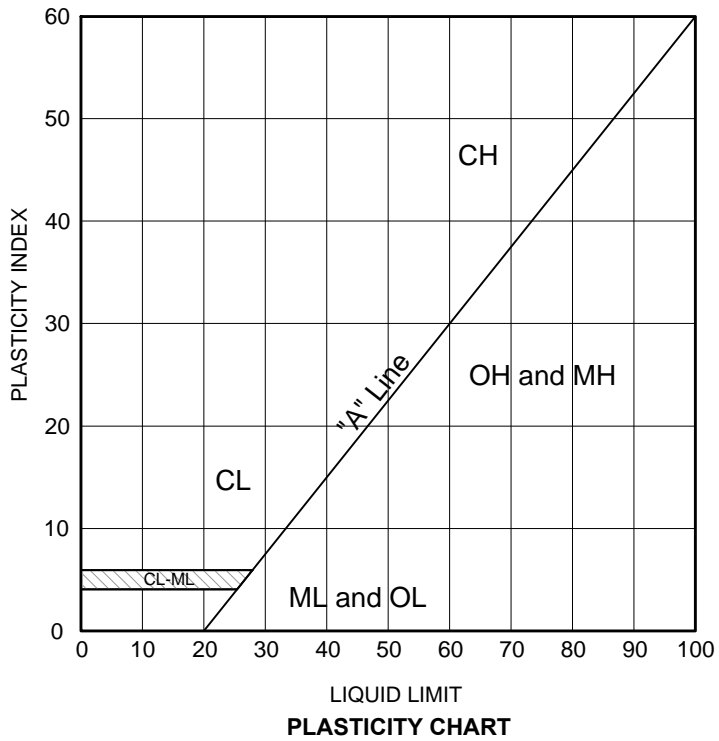
SOIL OR ROCK TYPES		
 Undocumented Fill	 Well-Graded Sand (SW)	
 Lean Clay (CL)	 Clayey Sand (SC)	DRILLING AND SAMPLING METHODS
 Gravelly Lean Clay (CL)	 Well-Graded Gravel (GW)	 Shelby Tube  Split Spoon  Texas Cone Pen
 Fat Clay (CH)	 Marl	
 Gravelly Fat Clay (CH)	 Weathered Shale	 CFA  HSA  Rock Core
 Clayey Gravel (GC)	 Shale	
 Silt (ML)	 Weathered Limestone	
 Poorly-Graded Sand (SP)	 Limestone	
TERMS DESCRIBING CONSISTENCY, CONDITION, AND STRUCTURE OF SOIL		
Fine Grained Soils (More than 50% Passing No. 200 Sieve)		
Consistency	Penetrometer Reading, (tsf)	Unconfined Compression, (psf)
Very Soft	≤ 0.5	≤ 1000
Soft	0.5 to 1.0	1000 to 2000
Firm	1.0 to 2.0	2000 to 4000
Hard	2.0 to 4.0	4000 to 8000
Very Hard	> 4.0	> 8000
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 alternated 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 hard 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.14

Major Divisions			Grp. 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	Not meeting all gradation requirements for SW
			SP	Poorly graded sands, gravelly sands, little or no fines			
		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 the 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			
			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	OH and MH			
		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				

UNIFIED SOIL CLASSIFICATION SYSTEM

PLATE A.15

SWELL TEST RESULTS

Courtside Multi-Family
Jefferson Avenue and West Court Street
Seguin, Texas
Rone Project Number: 19-23544

Boring	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	Initial MC (%)	Final MC (%)	Load (psf)	Swell (%)
B-1	5	47	19	28	18	22	625	0.0
B-2	3	70	22	48	23	28	375	0.8
B-3	7	50	19	30	19	23	875	0.1
B-4	9	32	16	17	17	21	1125	-0.3
B-5	9	46	20	27	19	22	1125	-0.1
B-6	3	64	25	40	22	25	375	2.8
B-7	5	74	25	49	23	30	625	0.9
B-8	7	46	14	32	20	23	875	0.0
B-9	9	40	18	22	19	22	1125	-0.2
B-10	5	73	28	44	27	32	625	2.0

APPENDIX B

FIELD EXPLORATION

Subsurface conditions were defined by 10 sample borings located as shown on the Boring Location Diagram, Plate A.3. The borings were completed at locations staked in the field and were advanced between sample intervals using continuous flight auger drilling procedures. The results of each boring are shown graphically on the Logs of Boring. Sample depth, description, and soil classification based on the Unified Soil Classification System are shown on the Logs of Boring. Keys to the symbols and terms used on the Logs of Boring are presented in the appendix section of the report.

Relatively undisturbed samples of cohesive soils were obtained using nominal 3-inch diameter thick-walled tube samplers at the locations shown on the Logs of Boring. The tube sampler consists of a 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 estimated in the field using a hand penetrometer. In this test, a 1/4-inch diameter piston is pushed into a relatively undisturbed sample at a constant rate to a depth of approximately 1/4 inch. The results of these tests are presented at the respective sample depths on the Logs of Boring. When the capacity of the penetrometer is exceeded, the value is tabulated as 4.5+.

Samples of stiff and/or granular materials were obtained using split-barrel sampling procedures in general accordance with ASTM D1586. In the split-barrel procedure, a disturbed sample is obtained in a standard 2-inch OD split-barrel sampler driven 18 inches into the ground using a 140-pound hammer falling freely 30 inches. The number of blows for the last 12 inches of the standard 18-inch penetration is recorded as the Standard Penetration Test resistance (N-value). The N-values are recorded on the logs of boring at the depth of sampling. The samples were sealed and returned to our laboratory for further examination and testing.

Groundwater observations during and at completion of the borings are shown on the upper right of the logs of boring. Upon completion of the borings, the boreholes were backfilled with auger cuttings to ground level.

LABORATORY TESTING

General

Laboratory tests were performed on selected samples retrieved from the borings to evaluate the engineering characteristics of the subsurface materials, and to provide data for developing engineering design parameters. The subsurface materials recovered during the field exploration were described by an engineering geologist or senior staff member in the field and/or the laboratory, and were later refined based on results of the laboratory tests performed.

Classification Tests

All recovered soil samples were classified and described, in part, using the Unified Soil Classification System (USCS). Visual classification of soils was verified by index property testing, including natural moisture content determinations, Atterberg limits determinations, and gradation tests (percent passing the No. 200 U.S. Standard Sieve). All testing was performed in general accordance with applicable American Society for Testing and Materials (ASTM) procedures as follows:

Atterberg Limits	ASTM D4318
Percentage of Particles Passing the No. 200 Sieve	ASTM D1140
Moisture Content	ASTM D2216
Dry Unit Weight	ASTM D2167
Unconfined Compressive Strength	ASTM D2166
Free Swell Test	ASTM D4546, Method B

Free Swell Test

In the free swell test, a sample is placed in a consolidometer and subjected to the estimated overburden pressure. The sample is then inundated with water and allowed to swell. Moisture contents are determined both before and after completion of the test. Test results are recorded as the percent swell, with initial and final moisture content. Detailed free swell test results are tabulated in Appendix A.16.

Unconfined Compression Strength Test

In the unconfined compression test, a cylindrical specimen is subjected to axial load at a constant rate of strain until failure occurs. Strengths determined by this test are tabulated at their respective sample depths on the log of boring. Results of natural moisture content and dry unit weight determinations are also tabulated at the respective sample depths on the log.

APPENDIX C

Important Information about This Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be, and, in general, if you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying it.* A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may

perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely*. Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old*.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration*. Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists*.



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