GEOTECHNICAL ENGINEERING STUDY

FOR

SPRINTOWN MIXED-USE DEVELOPMENT
1180 THORPE LANE
SAN MARCOS, TEXAS
Project No. AAA16-043-00
June 10, 2016

Mr. G. Nelson Crowe III, Partner
Endeavor
500 West 5th Street, Suite 700
Austin, Texas 78701

RE: Geotechnical Engineering Study
Springtown Mixed-Use Development
1180 Thorpe Lane
San Marcos, Texas

Dear Mr. Crowe:

RABA KISTNER Consultants Inc. (RKCI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKCI Proposal No. PAA16-060-00, Revision No. 1, dated May 12, 2016. The purpose of this study was to drill borings within the proximity of the newly proposed mixed-use building and parking garage structures, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations, as well as to provide pavement design and construction guidelines for the redevelopment of Springtown Shopping Center.

At the time this report was prepared, finished floor elevations for the mixed-use development building and parking garage, building design tolerances, and structural loads were not available. Once finished floor elevations and proposed site grading become available, we recommend that RKCI be retained to evaluate whether our foundation and pavement recommendations remain valid.

There may be alternatives for value engineering of the foundation and pavement systems. RKCI recommends that a meeting be held with the Owner and design team to evaluate if alternatives are available.

We appreciate the opportunity to be of service to you on this project. Should you have any questions about the information presented in this report, or if we may be of additional assistance with value engineering or on the materials testing-quality control program during construction, please call.

Very truly yours,

RABA KISTNER CONSULTANTS, INC.

Yvonne G. Thomas, P.E.
Geotechnical Engineering Manager

Gabriel Ornelas, Jr., P.E., PMP
Vice President

GO/YGT

Attachments
Copies Submitted: Above (1-Electronic; 1-Bound)

6/10/2016
GEOTECHNICAL ENGINEERING STUDY

For

SPRINGTOWN MIXED-USE DEVELOPMENT
1180 THORPE LANE
SAN MARCOS, TEXAS

Prepared for

ENDEAVOR
Austin, Texas

Prepared by

RABA KISTNER CONSULTANTS, INC.
Austin, Texas

PROJECT NO. AAA16-043-00

June 10, 2016
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The following figures are attached and complete this report:

Boring Location Map ........................................................................................................................................ Figure 1
Logs of Borings ........................................................................................................................................... Figures 2 to 9
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Important Information About Your Geotechnical Engineering Report
INTRODUCTION

RABA KISTNER Consultants Inc. (RKCI) has completed the authorized subsurface exploration and foundation analysis for the redevelopment of the existing Springtown Shopping Center to be located at 1180 Thorpe Lane in San Marcos, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations, as well as for pavement design and construction guidelines.

PROJECT DESCRIPTION

The facilities being considered in this study include a 4-story, mixed-use development and a 2-story parking garage to be located off Thorpe Lane, approximately 500 ft east of its intersection with Springtown Way in San Marcos, Texas. The mixed-use development is anticipated to consist of a front section with three floors of wood-frame residential construction atop a concrete podium level, and a back section with four floors of wood-frame residential space. The footprint of the building is anticipated to be a near “figure-eight” shape with residential units surrounding two centralized exterior cores to consist of a swimming pool and other outdoor amenities. One of the exterior sides surrounding the swimming pool area will be left as an open, wooded landscape. Surface parking and ancillary driveways are also planned in addition to the 2-story parking garage.

It is our understanding that at the time of this study, site grading plans and proposed structural loads were not yet available. The recommendations presented in this report were prepared with the assumption that final grade for the building structure will be within ± 1 ft of existing grades.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of central Texas and for the use of Endeavor (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods. The attachments and report text should not be used separately.

The recommendations submitted in this report are based on the data obtained from 8 borings drilled at this site, our understanding of the project information provided to us, and the assumption that site grading will result in only minor changes in the existing topography. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.
The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

If final grade elevations are significantly different from existing grades by more than plus or minus 1 ft, our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

**BORINGS AND LABORATORY TESTS**

Subsurface conditions at the site were evaluated by 8 borings drilled at the locations shown on the Boring Location Map, Figure 1. Boring locations were documented in the field utilizing a hand-held GPS device. The borings were drilled to depths of 15, 40, and 50 ft below the existing ground surface using a truck-mounted drilling rig. During drilling operations, the following samples were collected:

<table>
<thead>
<tr>
<th>Type of Sample</th>
<th>Number Collected</th>
</tr>
</thead>
<tbody>
<tr>
<td>Auger (Grab)</td>
<td>9</td>
</tr>
<tr>
<td>Undisturbed Shelby Tube</td>
<td>62</td>
</tr>
</tbody>
</table>

Each sample was visually classified in the laboratory by a member of our geotechnical engineering staff. The geotechnical engineering properties of the strata were evaluated by the following tests:

<table>
<thead>
<tr>
<th>Type of Test</th>
<th>Number Conducted</th>
</tr>
</thead>
<tbody>
<tr>
<td>Natural Moisture Content</td>
<td>68</td>
</tr>
<tr>
<td>Atterberg Limits</td>
<td>10</td>
</tr>
<tr>
<td>Percent Passing a No. 200 Sieve</td>
<td>2</td>
</tr>
<tr>
<td>Sieve Analysis with Hydrometer</td>
<td>1</td>
</tr>
<tr>
<td>Unconfined Compression (soil)</td>
<td>6</td>
</tr>
<tr>
<td>Sulfate Testing</td>
<td>2</td>
</tr>
</tbody>
</table>

The results of all laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 9. A key to classification terms and symbols used on the logs is presented on Figure 10. The results of the laboratory and field testing are also tabulated on Figure 11 for ease of reference. The results of the sieve analysis have been plotted on the attached Grain Size Curve, Figure 12.

Samples will be retained in our laboratory for 30 days after submittal of this report. Other arrangements may be provided at the request of the Client.
SULFATE TESTING

Sulfate testing was performed on selected samples at each boring location. The purpose of the sulfate testing was to determine the concentration of soluble sulfates in the subgrade soils, in order to investigate the potential for an adverse reaction to lime in sulfate-containing soils. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure.

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth Below Existing Ground Surface (ft)</th>
<th>Type of Soil</th>
<th>Sulfate Content (ppm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-2</td>
<td>1.1 to 2.5</td>
<td>Tan and Gray Clay</td>
<td>106</td>
</tr>
<tr>
<td>B-5</td>
<td>1.3 to 2.5</td>
<td>Tan and Gray Clay</td>
<td>189</td>
</tr>
</tbody>
</table>

*ppm = parts per million

Based on the laboratory test results, the reported values of sulfate content for the near surface soils tested were generally determined to be less than 200 ppm. Reported values of sulfate content above 3,000 ppm are known to cause sulfate induced heaving when the soils are mixed with lime. It should be understood that the identification of sulfates based on discrete soil samples cannot totally identify sulfates in all pavement areas. If the option for lime is considered, a quality assurance program should be implemented to assist in minimizing the risk of sulfate induced heaving.

GENERAL SITE CONDITIONS

SITE DESCRIPTION

The subject site is an existing retail shopping center located near the intersection of Thorpe Lane and Springtown Way in San Marcos, Texas. A portion of the existing Springtown Shopping Center occupies the footprint of the newly proposed mixed-use development and parking garage and the existing structures will have to be demolished to make way for the new development. The majority of the development to the north, as well as some to the south serves as parking area for the existing shopping center. Overall the site is relatively flat with a vertical relief of less than 10 ft across the site.

There are a significant number of buried structures and utilities throughout the area; it is also likely that abandoned foundations, structures, and utilities will be present in the footprint of the existing structure after its demolition. The presence of buried structures (old foundations, pavements, abandoned utilities, etc.) should be anticipated during construction.

GEOLOGY

A review of the Geologic Atlas of Texas, Seguin Sheet, indicates that this site is naturally underlain by alluvium overlying the Pecan Gap Chalk. Alluvium consists of floodplain deposits and can include clays, sands, silts, and gravels. Such deposits are typically highly variable and can therefore result in highly variable conditions over relatively short distances. Key geotechnical engineering concerns for development supported on Alluvium are the expansive nature of the clays, the consistency or relative density of the deposits, and the absence/presence as well as thickness of potentially water-bearing gravels.
The Pecan Gap Chalk weathers to form moderately deep soil and typically consists of clays, marly clays, and marl grading to chalk at depth. Thin seams of bentonite and/or bentonitic clays are also often encountered in this formation. Because such seams are typically thin and random, they are often difficult to locate and identify with standard geotechnical sampling methods and sampling intervals. Key geotechnical engineering concerns for development supported on this formation are expansive, soil-related movement.

SEISMIC CONSIDERATIONS

On the basis of the soil borings conducted for this investigation, the upper 100 feet of soil may be characterized as very dense soil and soft rock and a Class C Site Class Definition (Chapter 20 of ASCE 7) has been assigned to this site.

On the basis of the United States Geological Survey (USGS) website\(^1\) which utilizes the International Building Code (IBC) and U.S. Seismic Design Maps to develop seismic design parameters, the following seismic considerations are associated with this site.

\[
\begin{align*}
S_1 & = 0.068g \\
S_3 & = 0.031g \\
S_{\text{me}} & = 0.082g \\
S_{\text{m1}} & = 0.054g \\
S_{\text{DS}} & = 0.054g \\
S_{\text{01}} & = 0.036g \\
\end{align*}
\]

Based on the parameters listed above as well as Tables 1613.3.5(1) and 1613.3.5(2) of the 2012 IBC, the Seismic Design Category for both short period and 1 second response accelerations is A. As part of the assumptions required to complete the calculations, a Risk Category of “I or II or III” was selected.

STRATIGRAPHY

The subsurface conditions encountered at the boring locations are shown on the boring logs, Figures 2 through 9. These boring logs represent our interpretation of the subsurface conditions based on the field logs, visual examination of field samples by our personnel, and test results of selected field samples. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The lines designating the interfaces between strata on the boring logs represent approximate boundaries. Transitions between strata may be gradual.

Stratum I consists of very stiff, dark brown to grayish-brown, fat clay (CH). These clays are classified as highly plastic based on measured plasticity indices ranging from 45 to 52. Measured moisture contents range from 15 to 31 percent. Shear strength ranges from 1.0 to 1.9 tsf based on pocket penetrometer test data. Based on a single grain size analysis, the percentage of fines (percent passing a No. 200 sieve) was determined to be 87 percent.

Stratum II consists of very stiff to hard, tan and gray, fat clay (CH). This stratum grades to a marly, blocky, tan clay. These clays are classified as plastic to highly plastic based on measured plasticity indices ranging from 31 to 52. Measured moisture contents range from 14 to 23 percent. Shear strength ranges from 1.0 to 2.3 tsf based on pocket penetrometer test data. Unconfined compression tests conducted on samples below 18 ft resulted in shear strength values ranging from 2.5 to 5.5 tsf. A single unconfined compression test on a surficial sample resulted in a shear strength value of 1.5 tsf. Unit dry weight ranges from 101 to 111 pcf. Based on two grain size analyses, the percentage of fines was determined to be 99 percent.

GROUNDWATER

Groundwater was observed in Boring B-8 at an approximate depth of 36 ft below the existing ground surface during drilling operations. A 10 minute water level reading was measured at a depth of about 35 ft. All other borings remained dry during the field exploration phase. However, it is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly following periods of precipitation. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

Based on the findings in our borings and on our experience in this region, we believe that groundwater seepage encountered during shallow depth site earthwork activities and shallow foundation construction may be controlled using temporary earthen berm and conventional sump-and-pump dewatering methods. For excavations to depths greater than about 25 ft, provisions should be made to handle water entering excavations during construction. For deep foundation excavations, this could include the use of temporary casing to reduce groundwater seepage.

FOUNDATION ANALYSIS

EXPANSIVE SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at the site were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). PVR values ranging from 3-1/4 to 4 inches were estimated for the stratigraphic conditions encountered in our borings. A surcharge load of 1 psi (concrete slab and sand layer), an active zone of 15 ft, and dry moisture conditions were assumed in estimating the above PVR values.

The TxDOT method of estimating expansive soil-related movements is based on empirical correlations utilizing the measured plasticity indices and assuming typical seasonal fluctuations in moisture content. If desired, other methods of estimating expansive soil-related movements are available, such as estimations based on swell tests and/or soil-suction analyses. However, the performance of these tests and the detailed analysis of expansive soil-related movements were beyond the scope of the current study. It should also be noted that actual movements can exceed the calculated PVR values due to isolated changes in moisture content or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.
OVEREXCAVATION AND SELECT FILL REPLACEMENT

To reduce expansive soil-related movements in at-grade construction, a portion of the upper highly expansive subgrade clays in the building and garage areas can be removed by overexcavating and backfilling with a suitable select fill material. PVR values have been estimated for overexcavation and select fill replacement to various depths below the existing ground surface and are summarized in the table below. Recommendations for the selection and placement of select backfill materials are addressed in a subsequent section of this report.

<table>
<thead>
<tr>
<th>Depth of Overexcavation and Select Fill Replacement (ft)*</th>
<th>Estimated PVR (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>4</td>
</tr>
<tr>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>4</td>
<td>2</td>
</tr>
<tr>
<td>6</td>
<td>1-1/2</td>
</tr>
<tr>
<td>7</td>
<td>1</td>
</tr>
</tbody>
</table>

*below the ground surface elevation existing at the time of our study.

To maintain the above estimated PVR values, subsequent fill placed in the building and garage areas should consist of select fill material in accordance with the Select Fill section of this report.

DRAINAGE CONSIDERATIONS

When overexcavation and select fill replacement is selected as a method to reduce the potential for expansive soil-related movements at any site, considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structures. Filling an excavation in relatively impervious plastic clays with relatively pervious select fill material creates a “bathtub” beneath the structure, which can result in ponding or trapped water within the fill unless good surface and subsurface drainage is provided.

Water entering the fill surface during construction or entering the fill exposed beyond the building lines after construction may create problems with fill moisture control during compaction and increased access for moisture to the underlying expansive clays both during and after construction.

Several surface and subsurface drainage design features and construction precautions can be used to limit problems associated with fill moisture. These features and precautions may include but are not limited to the following:

- Installing berms or swales on the uphill side of the construction area to divert surface runoff away from the excavation/fill area during construction;
- Sloping of the top of the subgrade with a minimum downward slope of 1.5 percent out to the base of a dewatering trench located beyond the building perimeter;
• Sloping the surface of the fill during construction to promote runoff of rain water to drainage features until the final lift is placed;
• Sloping of a final, well maintained, impervious clay or pavement surface (downward away from the building) over the select fill material and any perimeter drain extending beyond the building lines, with a minimum gradient of 6 in. in 5 ft;
• Constructing final surface drainage patterns to prevent ponding and limit surface water infiltration at and around the building perimeter;
• Locating the water-bearing utilities, roof drainage outlets and irrigation spray heads outside of the select fill and perimeter drain boundaries; and
• Raising the elevation of the ground level floor slab.

Details relative to the extent and implementation of these considerations must be evaluated on a project-specific basis by all members of the project design team. Many variables that influence fill drainage considerations may depend on factors that are not fully developed in the early stages of design. For this reason, drainage of the fill should be given consideration at the earliest possible stages of the project.

FOUNDATION RECOMMENDATIONS

FOUNDATION OPTIONS

The following recommendations are based on the data obtained from our field and laboratory studies, our past experience with geotechnical conditions similar to those at this site, and our engineering design analyses. The following alternatives are available for foundation support:

• Rigid-engineered beam and slab foundation;
• Post-tension beam and slab foundation;
• Drilled-and-underreamed piers; and
• Drilled, straight-shaft piers.

SITE GRADING

Site grading plans can result in changes in almost all aspects of foundation recommendations. We have prepared all foundation recommendations based on the existing ground surface and the stratigraphic conditions encountered at the time of our study. If site grading plans differ from existing grade by more than plus or minus 1 ft, RKCI must be retained to review the site grading plans prior to bidding the project for construction. This will enable RKCI to provide input for any changes in our original recommendations that may be required as a result of site grading operations or other considerations.

MIXED-USE RETAIL/APARTMENT BUILDING FOUNDATION

ENGINEERED BEAM AND SLAB

The mixed-use retail/apartment structure may be founded on a stiffened engineered beam and slab foundation, provided the selected foundation type can be designed to withstand the anticipated soil-
related movements (see Expansive Soil-Related Movements) without impairing either the structural or the operational performance of the structures. We recommend implementing one of the options provided in the Overexcavation and Select Fill Replacement section of this report to reduce expansive soil-related movements to tolerable limits.

**Allowable Bearing Capacity**

Shallow foundations founded on compacted, select fill should be proportioned using the design parameters tabulated below.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum depth below final grade</td>
<td>18 in.</td>
</tr>
<tr>
<td>Minimum beam width</td>
<td>12 in.</td>
</tr>
<tr>
<td>Minimum widened beam width</td>
<td>18 in.</td>
</tr>
<tr>
<td>Maximum allowable bearing pressure for grade beams constructed on native clay subgrade or select fill building pad thicknesses of less than 3 ft</td>
<td>2,200 psf</td>
</tr>
<tr>
<td>Maximum allowable bearing pressure for widened beams constructed on native clay subgrade or select fill building pad thicknesses of less than 3 ft</td>
<td>2,500 psf</td>
</tr>
<tr>
<td>Maximum allowable bearing pressure for grade beams constructed on a minimum of 3 ft of select fill</td>
<td>3,000 psf</td>
</tr>
<tr>
<td>Maximum allowable bearing pressure for widened beams constructed on a minimum of 3 ft of select fill</td>
<td>3,800 psf</td>
</tr>
</tbody>
</table>

The above presented maximum allowable bearing pressures will provide a calculated factor of safety of about 3 with respect to the measured shear strength, provided that fill is selected and placed as recommended in the Select Fill section of this report. For placement of compacted, select fill on the order of 7 ft in thickness in the building pad area, we estimate total settlement to be on the order of about 1 inch, with about half of the settlement occurring during construction. Differential settlement is estimated to be less than 50 percent of the total settlement.

We recommend that a vapor barrier be placed between the supporting soils and the concrete floor slab.

**PTI Design Parameters**

Post Tensioning Institute (PTI) design parameters were estimated for existing stratigraphic conditions using the procedures and criteria discussed in the Post-Tensioning Institute Manual entitled “Design of Post-Tensioned Slabs-on-Ground, Third Edition” dated 2004 with the 2008 supplement.

Differential vertical swell has been estimated for center lift and edge lift conditions for use in designing foundation slabs for the stratigraphy encountered in our borings. These values were determined using a computer program entitled VOLFLO Win 1.5, as recommended by the Post Tensioning Institute. As recommended by PTI, we have evaluated differential swell for both 1) conditions varying from equilibrium and 2) conditions varying between extremes (wet/dry). The values for both of these conditions are presented in the table below. Because soil moisture conditions are likely to vary from wet to dry and vice versa over many cycles during the lifetime of the structure, we recommend that the latter conditions be assumed in design.
In estimating the differential swell values presented below, “percent fine clay” and “depth to constant soil suction” were the variable design parameters. Based on a grain size analysis and hydrometer test performed for this study, a “percent fine clay” value of 53 percent was used in our analyses to characterize the Stratum II clay soils. The depth to constant soil suction was taken to be 15 ft.

**Option 1: Existing Subgrade Conditions Without Overexcavation and Select Fill Replacement (PVR 4 in.)**

<table>
<thead>
<tr>
<th>Differential Swell (in.)</th>
<th>From Equilibrium to Wet</th>
<th>From Equilibrium to Dry</th>
<th>From Dry to Wet</th>
<th>From Wet to Dry</th>
</tr>
</thead>
<tbody>
<tr>
<td>(EL) - Edge Lift Condition</td>
<td>1 (EL)</td>
<td>-1 (CL)</td>
<td>3-1/4 (EL)</td>
<td>-2 (CL)</td>
</tr>
<tr>
<td>(CL) - Center Lift Condition</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Additional design parameters are summarized in the following table:

| Thornthwaite Index, IM | -10 |
| Constant Soil Suction | 3.6 pF |
| Edge Moisture Variation Distance (center lift) | 7.2 ft |
| Edge Moisture Variation Distance (edge lift) | 3.9 ft |

**Option 2: With 4 ft of Overexcavation and Select Fill Replacement (PVR to 2 in.)**

<table>
<thead>
<tr>
<th>Differential Swell (in.)</th>
<th>From Equilibrium to Wet</th>
<th>From Equilibrium to Dry</th>
<th>From Dry to Wet</th>
<th>From Wet to Dry</th>
</tr>
</thead>
<tbody>
<tr>
<td>(EL) - Edge Lift Condition</td>
<td>1/2 (EL)</td>
<td>-1/2 (CL)</td>
<td>1-1/2 (EL)</td>
<td>-1-1/4 (CL)</td>
</tr>
</tbody>
</table>

Additional design parameters are summarized in the following table:

| Thornthwaite Index, IM | -10 |
| Constant Soil Suction | 3.6 pF |
| Edge Moisture Variation Distance (center lift) | 8.3 ft |
| Edge Moisture Variation Distance (edge lift) | 4.3 ft |
PARKING GARAGE STRUCTURE FOUNDATION

OPTION 1: DRILLED-AND-UNDERREADED PIERS

Drilled-and-underreamed piers bearing in the Stratum II tan and gray clays may be considered to support the parking garage. We recommend drilled-and-underreamed piers extend a minimum depth of 30 ft below ground surface elevation existing at the time of our study. This minimum depth excludes the placement of additional fill material required to raise grade. The piers should be designed as end bearing units using a maximum allowable bearing pressure of 15 ksf. This bearing pressure was evaluated using a calculated factor of safety of at least 3 with respect to the design shear strength. Based on the 50 ft maximum depth of exploration, pier depth should not exceed a depth of 45 ft below the ground surface existing at the time of our study.

Pier Shaft Potential Uplift Forces

The pier shafts will be subject to potential uplift forces if the surrounding expansive soils within the active zone are subjected to alternate drying and wetting conditions. The maximum potential uplift force acting on the shaft may be estimated by:

\[ F_u = 105 \times D \]

where:

- \( F_u \) = uplift force in kips; and
- \( D \) = diameter of the shaft in feet.

Allowable Uplift Resistance

Resistance to uplift forces exerted on the drilled piers will be provided by the sustained axial compressive force (dead load) plus the allowable uplift resistance provided by the soil. The resistance provided by the soil depends on the bearing capacity of the soils located above the pier underream (bell) and below the active zone. The allowable uplift resistance for underreamed piers founded at the depth recommended above may be estimated using:

\[ R_u = 12 \times (B^2 - D^2) \]

where:

- \( R_u \) = uplift resistance in kips;
- \( B \) = diameter of the underream in feet; and
- \( D \) = diameter of the shaft in feet.

Due to the blocky nature of the Stratum II clays, we recommend that the bell-to-shaft diameter ratio be a minimum of 2, and not exceed 2.5. Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the sustained compressive load carried by the pier. We recommend that each pier be reinforced to withstand this net force or an amount equal to 1 percent of the cross-sectional area of the shaft, whichever is greater.
OPTION 2: DRILLED, STRAIGHT-SHAFT PIERS

Drilled, straight-shaft piers may also be considered to support the parking garage. Straight-shaft piers should be designed as friction units using an allowable side shear resistance of 0.8 ksf for the portion of pier extending 15 ft below the final surface. This allowable side shear resistance was evaluated using a calculated factor of safety of at least 2.

To proportion the drilled piers for axial compression, the side shear resistance should be neglected along the portion of the shaft located one shaft diameter from the bottom of the pier.

Drilled, straight-shaft piers bearing at a minimum depth of 30 ft below the existing ground surface may also be proportioned using a maximum allowable end bearing pressure of 15 ksf. However, we recommend that a minimum of 70 percent of the applied load be carried in side shear. Based on the 50 ft maximum depth of exploration, pier depths should not exceed a depth of 45 ft below the ground surface existing at the time of our study.

Representatives from RKCI should be present at the time of construction to verify that conditions are similar to those encountered in our borings. For bid purposes, the owner should anticipate that deeper piers will be required in some areas. Consequently, contractors bidding on the job should include unit costs for various depths of additional pier embedment. Unit costs should include those for both greater and lesser depth in both rock and soil.

Pier Shafts

The pier shafts will be subject to potential uplift forces if the surrounding expansive soils within the active zone are subjected to alternate drying and wetting conditions. The maximum potential uplift force acting on the shaft may be estimated by:

\[ F_u = 105\times D \]

where:

- \( F_u \) = uplift force in kips; and
- \( D \) = diameter of the shaft in feet.

Allowable Uplift Resistance

Resistance to uplift forces exerted on the drilled, straight-shaft piers will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the soil. The resistance provided by the soil depends on the shear strength of the soils adjacent to the pier shaft and below the depth of the active zone. The allowable uplift resistance provided by the soils at this site may be estimated using an axial compressive side shear resistance of 0.5 ksf for the portion of the shaft extending below a depth of 15 ft. This value was evaluated using a factor of safety of 2.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the sustained compressive load carried by that pier. We recommend that each pier be reinforced...
to withstand this net force or an amount equal to 1 percent of the cross-sectional area of the shaft, whichever is greater.

**PIER SPACING**

Where possible, we recommend that the piers be spaced at a center to center distance of at least three shaft diameters on-center for straight-shaft piers and three bell-diameters for underreamed piers. Such spacing will not require a reduction in the load carrying capacity of the individual piers.

If design and/or construction restraints require that piers be spaced closer than the recommended three shaft/bell diameters, RKCI must re-evaluate the allowable bearing capacities presented above for the individual piers. Reductions in load carrying capacities may be required depending upon individual loading and spacing conditions.

**GRADE BEAMS**

Grade beams may be ground supported, provided that anticipated movements, as discussed under the *Expansive Soil-Related Movements* section of this report will not impair their performance. We recommend that the grade beams be designed to span from pier to pier.

As an alternative, the grade beams interconnecting the piers may be structurally suspended. A positive void space of at least 12 in., preferably more, should be provided between the soffits of grade beams and the underlying soils.

**FLOOR SLABS**

Two alternatives are available to construct the floor slab system. The Owner may select the alternative best satisfying the required performance criteria.

**Alternative No. 1:** Floor slabs which have high performance criteria or which are movement sensitive in nature, should be structurally suspended because of the anticipated ground movements. A positive void space of at least 12 in., preferably more, should be provided between the slab and the underlying soils (see also *Crawl Space Considerations*). Areas containing critical entry/exit points to the building, such as doorways, should consider using a suspended system to relieve those areas of heave stresses caused by expansive soils.

**Alternative No. 2:** Floor slabs within the superstructure may be ground supported provided the anticipated movements discussed under the *Expansive Soil-Related Movements* section of this report will not impair the performance of the floor, frame, or roof systems.

If differential movements between the slab and the structure are objectionable, soil-supported floor slabs could be dowelled to the perimeter grade beams. Dowelled slabs that are subjected to heaving will typically crack and develop a plastic hinge along a line
which will be approximately 5 to 10 ft inside and parallel to the grade beams. Slabs cast independent of the grade beams, interior columns and partitions should experience minimum cracking, but may create difficulties at critical entry points such as doors and may impact interior partitions that are secured to exterior walls.

We recommend that a vapor barrier be placed between the supporting select fill and the concrete floor slab. Floor slabs bearing on a minimum of 12-inches of compacted Select Fill may be designed using a modulus of subgrade reaction of 225 psi/in.

**AREA FLATWORK**

It should be noted that ground-supported flatwork such as walkways will be subject to the same magnitude of potential soil-related movements as discussed previously (see *Expansive Soil-Related Movement* section). Thus, where these types of elements abut rigid building foundations or isolated/suspended structures, differential movements should be anticipated.

Based on the anticipated ground movements due to swelling of the underlying soils beneath the sidewalk/flatwork it is our opinion that the design team should consider methods of reducing the anticipated movements through methods discussed in the *Overexcavation and Select Fill Replacement* section or to consider structurally suspending critical areas to match the adjacent building performance. It is recommended that in areas of ingress/egress the outside stoop should be structurally suspended.

This is especially true in the movement sensitive areas such as along the approaches to the structure. It is important to note that the transition from structurally suspended flatwork to grade supported flatwork is critical. The transfer slab must be constructed rigid enough to span from the structurally suspended end to the grade supported end. The slab shall be constructed with sufficient slope to accommodate proper drainage, in addition to maintaining drainage in the event that the edge of the flatwork supported on the ground surface heaves upward. Since voids will exist beneath the slab, a drainage system will be necessary to remove water infiltration into the void.

As a minimum, we recommend that flexible joints be provided where such elements abut the main structure to allow for differential movement at these locations where flatwork abuts the exterior perimeter grade beam of the new building care must be taken to provide a smooth, vertical construction joint between the edge of the flatwork and the perimeter grade beam. The flatwork should preferably abut the grade beam at least 4 inches, preferably more, below the bottom of the brick lug (or other exterior veneer material) to avoid damage to the veneer when movements occur in the flatwork. The construction joint should be wide enough to assure that vertical movement in the flatwork will not bind on and subsequently damage the exterior veneer material.

The construction joint should be completed using an appropriate elastomeric expansion joint filler to reduce the amount of water passing through the construction joint. Proper and regular maintenance of the expansion joint will help reduce the water seepage at the flatwork/foundation interface.

A better option, if feasible, is to separate the flatwork away from the building foundation so that movements incurred by the flatwork are totally independent of the building foundation.
FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

Drainage is an important key to the successful performance of any foundation. Good surface drainage should be established prior to and maintained after construction to help prevent water from ponding within or adjacent to the building foundation and to facilitate rapid drainage away from the building foundation. Failure to provide positive drainage away from the structure can result in localized differential vertical movements in soil supported foundations and floor slabs (which can in turn result in cracking in the sheetrock partition walls, and shifting of ceiling tiles, as well as improper operation of windows and doors).

Current ordinances, in compliance with the Americans with Disabilities Act (ADA), may dictate maximum slopes for walks and drives around and into new buildings. These slope requirements can result in drainage problems for buildings supported on expansive soils. We recommend that the maximum permissible slope be provided away from the building on all sides.

Also to help control drainage in the vicinity of the structure, we recommend that roof/gutter downspouts and landscaping irrigation systems not be located adjacent to the building foundation. Where a select fill overbuild is provided outside of the floor slab/foundation footprint, the surface should be sealed with an impermeable layer (pavement or clay cap) to reduce infiltration of both irrigation and surface waters. Careful consideration should also be given to the location of water bearing utilities, as well as to provisions for drainage in the event of leaks in water bearing utilities. All leaks should be immediately repaired.

Other drainage and subsurface drainage issues are discussed in the Expansive Soil-Related Movements section of this report and under Pavement Construction Considerations.

SITE PREPARATION

The footprint of the parking garage and mixed-use development is currently occupied with an existing building structures and parking lots. It is our understanding the building and its foundation will be demolished and completely removed from the site. Wherever possible, foundation remnants and abandoned utilities within the “footprint” of any new structure should be removed. If deep foundation elements cannot be removed then they should be cut off a minimum of 2 ft, preferably more, beneath the bottom of any new floor slab or pavement. We recommend that the design team give consideration to controlling the influx of water for any excavations at the earliest possible stage of design.

In addition, all remaining improvements including utility lines should be completely removed from the site. Beyond the building pad footprint, existing utilities that are not removed should be properly abandoned. This would include grouting abandoned pipes and sealing off granular fill in utility trenches to prevent the migration and seepage of water into the building pad of the new building.
Where foundations (grade beams, continuous footings, spread footings, etc.) are removed, the subgrade should be scarified, moisture conditioned and compacted. The same condition applies to all utilities located in the area of the new structure.

Building areas and all areas to support select fill should be stripped of all vegetation and organic topsoil. Furthermore, as discussed in a previous section of this report, if an engineered beam-and-slab foundation system is chosen for the proposed structure, we recommend that the Overexcavation and Select Fill Replacement option be utilized to reduce expansive soil-related movements.

Exposed subgrades should be thoroughly proofrolled in order to locate weak, compressible zones. A minimum of 5 passes of a fully-loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or their representative to document subgrade condition and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with suitable, compacted on-site clays, free of organics, oversized materials, and degradable or deleterious materials.

Upon completion of the proofrolling operations and just prior to fill placement or slab construction, the exposed subgrade should be moisture conditioned by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density determined from TxDOT, Tex-114-E, Compaction Test. The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum moisture content until permanently covered.

**SELECT FILL**

Materials used as select fill for final site grading preferably should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the gradation requirements of TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Type A, B, or C, Grades 1 through 3, and have a plasticity index between 7 and 20.

Select fill should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction.

If total building pad fill depths are less than 4 ft, select fill composed of GC or SC combination soils, as classified according to the Unified Soil Classification System (USCS), may be considered satisfactory for use as select fill materials at this site. Alternative select fill materials shall have a maximum liquid limit not exceeding 40, a plasticity index between 7 and 20, and a maximum particle size not exceeding 4 in. or one-half the loose lift thickness, whichever is smaller. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with pit-run materials.

If the above listed alternative materials are being considered for bidding purposes, the materials should be submitted to the Geotechnical Engineer for pre-approval at a minimum of 10 working days or more prior
to the bid date. Failure to do so will be the responsibility of the contractor. The contractor will also be responsible for ensuring that the properties of all delivered alternate select fill materials are similar to those of the pre-approved submittal. It should also be noted that when using alternative fill materials, difficulties may be experienced with respect to moisture control during and subsequent to fill placement, as well as with erosion, particularly when exposed to inclement weather. This may result in sloughing of beam trenches and/or pumping of the fill materials.

Soils classified as CH, CL, MH, ML, SM, GM, OH, OL and Pt under the USCS are not considered suitable for use as select fill materials at this site. The native soils at this site are not considered suitable for use as select fill materials.

**SHALLOW FOUNDATION EXCAVATIONS**

Shallow foundation excavations should be observed by the Geotechnical Engineer or their representative prior to placement of reinforcing steel and concrete. This is necessary to observe that the bearing soils at the bottom of the excavations are similar to those encountered in our borings and that excessive loose materials and water are not present in the excavations. If soft pockets of soil are encountered in the foundation excavations, they should be removed and replaced with a compacted non-expansive fill material or lean concrete up to the design foundation bearing elevations.

**DRILLED PIERS**

Each drilled pier excavation must be examined by an RKCI representative who is familiar with the geotechnical aspects of the soil stratigraphy, the structural configuration, foundation design details and assumptions, prior to placing concrete. This is to observe that:

- The shaft and/or bell has been excavated to the specified dimensions at the correct depth established by the previously mentioned criteria;
- An acceptable portion of the shaft penetrates intact limestone versus weathered and/or clay seams;
- The bell is concentric with the pier shaft;
- The shaft has been drilled plumb within specified tolerances along its total length; and
- Excessive cuttings, buildup and soft, compressible materials have been removed from the bottom of the excavation.

Based on visual observations and the results of unconfined compression tests, the Stratum II clays are blocky. This may result in sloughing within the bell at the time of construction. As previously recommended, the bell-to-shaft diameter ratio should not exceed 2.5 to improve constructability of the underreams in these clays.
Reinforcement and Concrete Placement

Reinforcing steel should be checked for size and placement prior to concrete placement. Placement of concrete should be accomplished as soon as possible after excavation to reduce changes in the moisture content or the state of stress of the foundation materials. No foundation element should be left open overnight without concreting.

Temporary Casing

Groundwater seepage was observed in the test borings at the time of our subsurface exploration. Groundwater seepage and/or side sloughing is likely to be encountered at the time of construction, depending on climatic conditions prevalent at the time of construction. Therefore, we recommend that the bid documents require the foundation contractor to specify unit costs for different lengths of casing that may be required.

EXCAVATION SLOPING AND BENCHING

If utility trenches or other excavations extend to or below a depth of 5 ft below construction grade, the contractor or others shall be required to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

EXCAVATION EQUIPMENT

Our boring logs are not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that earth-work and utility contractors interested in bidding on the work perform their own tests in the form of test pits to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

CRAWL SPACE CONSIDERATIONS

If the structurally suspended floor system described as Alternative No. 1 under the Floor Slab section of this report is selected, several special design issues should be considered for the resulting subfloor crawl space. These issues are discussed below.

Ventilation

Observations by members of our firm of open crawl spaces have indicated a need for adequate subfloor ventilation for suspended floor systems. Such ventilation helps promote evaporation of subgrade moisture which may accumulate in spite of special surface and subsurface drainage features. As a minimum, free flowing passive vents may need to be installed along the perimeter beam to provide cross ventilation. If structural configurations will limit the free flow of air through passive vents, forced air, power vents should be installed. All vents should be designed such that they will not allow the drainage of surface water into the crawl space.
Below Slab Utilities

A minimum clearance of 12 in. has been recommended between both the grade beams and floor slab and the underlying finished subgrade should a suspended floor system be employed. Such a minimum clearance is also recommended between the subgrade and any utilities which may be suspended from the underside of the floor. This clearance will allow swell-related subgrade movements without damaging the utilities. It is recommended that the utility clearance not be provided by the addition of narrow trenches running parallel to and immediately below the utilities, unless proper slopes and drainage outlets are provided to prevent ponding of water in the trenches.

Drainage

As discussed throughout this report, positive drainage is a key factor in the long term performance of any foundation. This is not only critical around the perimeter of the structure, but also in any subfloor crawl spaces. In crawl areas, surface drainage should be established that will direct water away from and will prevent water from ponding adjacent to piers. This positive drainage should be maintained both prior to and after construction.

Compaction control of the backfill around the perimeter of the building following the placement of soil retainer blocks is critical to the drainage away from the building following construction. Materials for the backfill around the perimeter of the building should be the on-site clays. These materials should be compacted in uniformly thin lifts (8-inch maximum loose thickness) to at least 90 percent of the maximum dry density as determined by TxDOT Test Method TEX-114-E. These clays should be placed and compacted at optimum to plus 3 percent above optimum moisture content. Compaction by hand operated mechanical tampers will help to avoid damage to the soil retainer blocks. Following backfilling operations the soil retainer blocks should be checked to see that they have not been broken or collapsed during the compaction operations. Any soil retainer blocks that are broken or collapsed should be repaired or replaced.

Carton Forms

When carton forms are used to form subfloor void spaces, the forms often get wet or sometimes absorb water from humid air. This can result in collapse of the forms during the placement of concrete, thus diminishing the design void space. Conversely, if the carton forms are too strong and do not decompose sufficiently with time, they may not collapse as soil heave occurs, resulting in heave damage to the floor slab. Where there is sufficient moisture to cause the appropriate deterioration after construction, there may be a resulting moisture problem in the floor slab as a result of poor ventilation and the accumulation of condensation within the resulting unventilated void space. The lack of ventilation may also result in increased soil movements that will diminish the design void space. For these reasons, we recommend that where possible, consideration be given to methods other than the use of carton forms to form the recommended void space beneath floor slabs. If project specifics require the use of carton forms, then as a minimum, care should be taken to ensure that the carton forms are designed for use in the project location, and that carton forms are properly stored, protected, and installed during construction.
INTERIOR WALLS

It is not uncommon for cracking to occur in interior partition walls that are supported by a “floating” floor slab and structurally tied to either an interior column or an exterior wall supported by deep foundations. This should be taken into account during the design phase of the project if a “floating” slab foundation is used to support the proposed building.

UTILITIES

Utilities which project through slab-on-grade, slab-on-fill, or any other rigid unit should be designed with either some degree of flexibility or with sleeves. Such design features will help reduce the risk of damage to the utility lines as vertical movements occur.

Our experience indicates that significant settlement of backfill can occur in utility trenches, particularly when trenches are deep, when backfill materials are placed in thick lifts with insufficient compaction, and when water can access and infiltrate the trench backfill materials. The potential for water to access the backfill is increased where water can infiltrate flexible base materials due to insufficient penetration of curbs, and at sites where geological features can influence water migration into utility trenches (such as fractures within a rock mass or at contacts between rock and clay formations). It is our belief that another factor which can significantly impact settlement is the migration of fines within the backfill into the open voids in the underlying free-draining bedding material.

To reduce the potential for settlement in utility trenches, we recommend that consideration be given to the following:

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented. Trench backfill materials should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E or Tex-114-E, Compaction Test. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content for non-cohesive soils and maintained within the range of optimum to 3 percentage points above optimum moisture content for cohesive soils until final compaction.
- Curbs should completely penetrate base materials and be installed to a sufficient depth to reduce water infiltration beneath the curbs into the pavement base materials.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

PAVEMENT RECOMMENDATIONS

Recommendations for both flexible and rigid pavements are presented in this report. The Owner and/or design team may select either pavement type depending on the performance criteria established for the project. In general, flexible pavement systems have a lower initial construction cost as compared to
rigid pavements. However, maintenance requirements over the life of the pavement are typically much greater for flexible pavements. This typically requires regularly scheduled observation and repair, as well as overlays and/or other pavement rehabilitation at approximately one-half to two-thirds of the design life. Rigid pavements are generally more "forgiving", and therefore tend to be more durable and require less maintenance after construction.

For either pavement type, drainage conditions will have a significant impact on long term performance, particularly where permeable base materials are utilized in the pavement section. Drainage considerations are discussed in more detail in a subsequent section of this report.

**SUBGRADE CONDITIONS**

We have assumed the subgrade in pavement areas will consist of the Stratum I or Stratum II clays, or recompacted on-site clays, placed and compacted as recommended in the *On-Site Clay Fill* section of this report. Based on our experience with similar subgrade soils, we have assigned California Bearing Ratio (CBR) values of 3.0 for Stratum I dark brown to grayish-brown clays and Stratum II tan clays for use in pavement thickness design analyses.

**DESIGN INFORMATION**

The following recommendations were prepared using the DARWin 3.1 software program which utilizes a procedure based on the 1993 “Guide for the Design of Pavement Structures” by the American Association of State Highway and Transportation Officials (AASHTO). The following recommendations were prepared assuming a 20-yr design life and Equivalent Single Axle Loads (ESAL’s) of 20,000 for light duty pavements and 80,000 for medium duty pavements. This traffic frequency is approximately equivalent to 1 and 3 tractor-trailer trucks per day for a design period of 20 years for light and heavy duty pavements, respectively. The *Project Civil Engineer* should review anticipated traffic loading and frequencies to verify that the assumed traffic loading and frequency is appropriate for the intended use of the facility.

**FLEXIBLE PAVEMENT**

Flexible pavement sections recommended at this site are as listed in the table below:

<table>
<thead>
<tr>
<th>Layer Description</th>
<th>Light Duty (20,000 ESAL's)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Option No. 1 Layer Thickness</td>
<td>Option No. 2 Layer Thickness</td>
</tr>
<tr>
<td>HMAC Surface Course, Type &quot;D&quot;</td>
<td>2.0 in. 9.0 in. 8.0 in. no</td>
</tr>
<tr>
<td>Flexible Base</td>
<td></td>
</tr>
<tr>
<td>Lime Treated Subgrade Geogrid</td>
<td></td>
</tr>
<tr>
<td>Combined Total</td>
<td>19.0 in.</td>
</tr>
<tr>
<td>Layer Description</td>
<td>Option No. 1 Layer Thickness</td>
</tr>
<tr>
<td>---------------------------------------</td>
<td>-----------------------------</td>
</tr>
<tr>
<td>HMAC Surface Course, Type “D”</td>
<td>2.0 in.</td>
</tr>
<tr>
<td>Flexible Base</td>
<td>13.0 in.</td>
</tr>
<tr>
<td>Lime Treated Subgrade</td>
<td>8.0 in.</td>
</tr>
<tr>
<td>Geogrid</td>
<td></td>
</tr>
<tr>
<td>Combined Total</td>
<td>23.0 in.</td>
</tr>
</tbody>
</table>

**Garbage Dumpsters**

Where flexible pavements are constructed at any site, we recommend that reinforced concrete pads be provided in front of and beneath trash receptacles. The dumpster trucks, if any, should be parked on the rigid pavement when the receptacles are lifted.

It is suggested that such pads also be provided in drives where the dumpster trucks make turns with small radii to access the receptacles. The concrete pads at this site should be a minimum of 6 in. thick and reinforced with conventional steel reinforcing bars or welded wire mats.

**RIGID PAVEMENT**

We recommend that rigid pavements be considered in areas of channelized traffic, particularly in areas where truck or bus traffic is planned, and particularly where such traffic will make frequent turns, such as described above for garbage dumpster areas. We recommend that rigid pavement sections at this site consist of the following:

<table>
<thead>
<tr>
<th>Traffic Type</th>
<th>Portland Cement Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Light Duty Traffic</td>
<td>5 in.</td>
</tr>
<tr>
<td>Heavy Duty Traffic</td>
<td>6 in.</td>
</tr>
</tbody>
</table>

We recommend that the concrete pavements be reinforced with bar mats. As a minimum, the bar mats should be No. 3 reinforcing bars spaced 18 in. on center in both directions. The concrete reinforcing should be placed approximately 1/3 the slab thickness below the surface of the slab, but not less than 2 in. The reinforcing should not extend across expansion joints.

Joints in concrete pavements aid in the construction and control the location and magnitude of cracks. Where practical, lay out the construction, expansion, control and sawed joints to form square panels, but not to exceed ACI 302.69 Code recommendations. The ratio of slab length-to-width should not exceed 1.25. Maximum recommended joint spacings are 12 ft longitudinal and 12 ft transverse.
All control joints should be formed or sawed to a depth of at least 1/4 the thickness of the concrete slab. Sawing of control joints should begin as soon as the concrete will not ravel, generally the day after placement. Control joints may be hand formed or formed by using a premolded filler. We recommend that all longitudinal and transverse construction joints be dowelled to promote load transfer. Expansion joints are needed to separate the concrete slab from fixed objects such as drop inlets, light standards and buildings. Expansion joint spacings are not to exceed a maximum of 75 ft and no expansion or construction joints should be located in a swale or drainage collection locations.

If possible, the pavement should develop a minimum slope of 0.015 ft/ft to provide surface drainage. Reinforced concrete pavement should cure a minimum of 3 and 7 days before allowing automobile and truck traffic, respectively.

**PAVEMENT CONSTRUCTION CONSIDERATIONS**

**SUBGRADE PREPARATION**

Areas to support pavements should be stripped of all vegetation and organic topsoil and the exposed subgrade should be proofrolled in accordance with the recommendations in the Site Preparation section under Foundation Construction Considerations.

After completion of the proofrolling operations and just prior to flexible base placement, the exposed subgrade should be moisture conditioned by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density determined from the Texas Department of Transportation Compaction Test (TxDOT, Tex-114-E). The moisture content of the subgrade should be maintained within the range of optimum moisture content to 3 percentage points above optimum until permanently covered.

**DRAINAGE CONSIDERATIONS**

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surf ace and subsurface drainage. Insufficient drainage which allows saturation of the pavement subgrade and/or the supporting granular pavement materials will greatly reduce the performance and service life of the pavement systems.

Surface and subsurface drainage considerations crucial to the performance of pavements at this site include (but are not limited to) the following:

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.

2) Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils. Curbs should completely penetrate base materials and should be installed to sufficient depth to reduce infiltration of water beneath the curbs.
3) Pavement surfaces should be maintained to help minimize surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

**ON-SITE CLAY FILL**

The pavement recommendations presented in this report were prepared assuming that on-site soils will be used for fill grading in proposed pavement areas. If used, we recommend that on-site soils be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 95 percent of the maximum density as determined by TxDOT, Tex-114-E. The moisture content of the fill should be maintained within the range of optimum water content to 3 percentage points above the optimum water content until permanently covered. We recommend that fill materials be free of roots and other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

**LIME TREATMENT OF SUBGRADE**

Lime treatment of the subgrade soils, where utilized, should be in accordance with the TxDOT Standard Specifications, Item 260. A sufficient quantity of hydrated lime should be mixed with the subgrade soils to reduce the soil-lime mixture plasticity index to 15 or less. For estimating purposes, we recommend that 6 percent lime by weight be assumed for treatment. For construction purposes, we recommend that the optimum lime content of the subgrade soils be determined by laboratory testing. Lime-treated subgrade soils should be compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of optimum moisture content to 3 percentage points above the optimum moisture content as determined by Tex-113-E. We recommend that lime treatment extend at least 3 ft beyond the curb.

If lime treatment is considered as a method to improve pavement subgrade conditions, it is also recommended to perform additional laboratory testing to determine the concentration of soluble sulfates in the subgrade soils, in order to investigate the potential for a recently reported adverse reaction to lime in certain sulfate-containing soils. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure.

**GEOGRID REINFORCEMENT**

The geogrid reinforcement should be Type II geogrid meeting TxDOT DMS-6240 specifications or Tensar TX-5.

An approved source of geogrid is The Tensar Corporation, Morrow, GA or their designated representative. The geogrid component shall be integrally formed and produced from a punched sheet of polypropylene which is then oriented in three substantially equilateral directions so that the resulting ribs shall have a high degree of molecular orientation, which continues at least in part through the mass of the integral node. The resulting geogrid structure shall have apertures that are triangular in shape, and shall have ribs with a depth-to-width ratio greater than 1.0.
The geogrid shall have the nominal characteristics shown in the table below, and shall be certified in writing by the manufacturer to be TX-5:

<table>
<thead>
<tr>
<th>Properties</th>
<th>Longitudinal</th>
<th>Diagonal</th>
<th>Transverse</th>
<th>General</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rib pitch, mm (in.)</td>
<td>40 (1.60)</td>
<td>40 (1.60)</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Mid-rib depth, mm (in.)</td>
<td>-</td>
<td>1.3 (0.05)</td>
<td>1.2 (0.05)</td>
<td>-</td>
</tr>
<tr>
<td>Mid-rib width, mm (in.)</td>
<td>-</td>
<td>0.9 (0.04)</td>
<td>1.2 (0.05)</td>
<td>-</td>
</tr>
<tr>
<td>Rib shape</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Rectangular</td>
</tr>
<tr>
<td>Aperture shape</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>Triangular</td>
</tr>
</tbody>
</table>

The geogrid should be placed at the bottom of the flexible (granular) base section in all cases. An alternative to the above geogrid should not be considered without approval from RKCI.

**FLEXIBLE BASE COURSE**

The flexible base course should be crushed limestone conforming to TxDOT Standard Specifications, Item 247, Type A, Grades 1 or 2. Base course should be placed in lifts with a maximum thickness of 8 in. and compacted to a minimum of 100 percent of the maximum density at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E.

**ASPHALTIC CONCRETE SURFACE COURSE**

The asphaltic concrete surface course should conform to TxDOT Standard Specifications, Item 340, Type D. The asphaltic concrete should be compacted to a minimum of 92 percent of the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

**PORTLAND CEMENT CONCRETE**

The Portland cement concrete should be air entrained to result in a 4 percent plus/minus 1 percent air, should have a maximum slump of 5 inches, and should have a minimum 28-day compressive strength of 4,000 psi. A liquid membrane-forming curing compound should be applied as soon as practical after broom finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.
CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, Important Information About Your Geotechnical Engineering Report, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.

Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, RKCI is retained to perform construction observation and testing services during the construction of the project. This is because:

- RKCI has an intimate understanding of the geotechnical engineering report’s findings and recommendations. RKCI understands how the report should be interpreted and can provide such interpretations on site, on the client’s behalf.
- RKCI knows what subsurface conditions are anticipated at the site.
- RKCI is familiar with the goals of the owner and project design professionals, having worked with them in the development of the geotechnical workscope. This enables RKCI to suggest remedial measures (when needed) which help meet the owner’s and the design teams’ requirements.
- RKCI has a vested interest in client satisfaction, and thus assigns qualified personnel whose principal concern is client satisfaction. This concern is exhibited by the manner in which contractors’ work is tested, evaluated and reported, and in selection of alternative approaches when such may become necessary.
- RKCI cannot be held accountable for problems which result due to misinterpretation of our findings or recommendations when we are not on hand to provide the interpretation which is required.

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKCI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKCI looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.
ATTACHMENTS
ASPHALT (4 inches)
BASE (10 inches)
CLAY (CH), Fat, Very Stiff, Dark Brown to Dark Gray
- gravel layer, 10 to 10.5 ft
Boring Terminated

NOTES:
1. Groundwater not encountered during drilling operations.
2. Borehole was backfilled with auger cuttings.
LOG OF BORING NO. B-2
Springtown Mixed-Use Development
1180 Thorpe Lane
San Marcos, Texas

LOCATION: N 29.88641; W 97.92195

DRILLING METHOD: Straight Flight Auger

DESCRIPTION OF MATERIAL

- ASPHALT (3 inches)
- BASE (10 inches)
- CLAY (CH), Fat, Hard, Tan and Gray, with ferrous staining
  - lean, very stiff above 2-1/2 ft
  - blocky below 8 ft

Boring Terminated

NOTES:
1. Groundwater not encountered during drilling operations.
2. Borehole was backfilled with auger cuttings.

DEPHT DRILLED: 15.0 ft
DATE DRILLED: 5/16/2016
DEPTH TO WATER: Dry
DATE MEASURED: 5/16/2016
PROJ. No.: AAA16-043-00
FIGURE: 3
**DESCRIPTION OF MATERIAL**

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **ASPHALT (2 inches)**
- **BASE (5 inches)**
- **CLAY (CH), Fat, Blocky, Hard, Tan and Light Gray, with ferrous staining**

**Boring Terminated**

**NOTES:**
1. Groundwater not encountered during drilling operations.
2. Borehole was backfilled with auger cuttings.

**LOG OF BORING NO. B-3**

**LOCATION:**
N 29.88644; W 97.92120

**DRILLING METHOD:** Straight Flight Auger

<table>
<thead>
<tr>
<th>DEPTH, FT</th>
<th>SYMBOL</th>
<th>SAMPLES</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>15</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td></td>
<td></td>
</tr>
<tr>
<td>25</td>
<td></td>
<td></td>
</tr>
<tr>
<td>30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>35</td>
<td></td>
<td></td>
</tr>
<tr>
<td>40</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DEPTH DRILLED:** 15.0 ft  
**DATE DRILLED:** 5/16/2016

**DEPTH TO WATER:** Dry  
**DATE MEASURED:** 5/16/2016

**PROJ. No.:** AAA16-043-00  
**FIGURE:** 4

**NOTE:** THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT
## Log of Boring No. B-4

### Springtown Mixed-Use Development
1180 Thorpe Lane
San Marcos, Texas

#### DRILLING METHOD:
Straight Flight Auger

#### LOCATION:
N 29.88590; W 97.92131

### Plasticity Index

<table>
<thead>
<tr>
<th>Blows per ft</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>37</td>
</tr>
<tr>
<td>1.0</td>
<td>99</td>
</tr>
<tr>
<td>1.5</td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td>2.5</td>
<td></td>
</tr>
<tr>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td></td>
</tr>
</tbody>
</table>

### Description of Material

- **Asphalt (7 inches)**
- **Base (5 inches)**
- **Clay (CH), Fat, Very Stiff, Tan and Light Gray, with ferrous staining**
  - Hard below 6 ft
  - Blocky below 8 ft

### Boring Terminated

#### Notes:
1. Groundwater not encountered during drilling operations.
2. Borehole was backfilled with auger cuttings.

### Depth Drilled:
15.0 ft

### Date Drilled:
5/16/2016

### Depth to Water:
Dry

### Date Measured:
5/16/2016

### Proj. No.:
AAA16-043-00

### Figure:
5

**Note:** These logs should not be used separately from the project report.
**LOG OF BORING NO. B-5**

**Springtown Mixed-Use Development**
1180 Thorpe Lane
San Marcos, Texas

**DRILLING METHOD:** Straight Flight Auger

**DESCRIPTION OF MATERIAL**

- **ASPHALT (3 inches)**
- **BASE (12 inches)**
- **FILL: CLAY (CH), Fat, Firm, Tan, Gray, and Dark Brown, with small voids**
- **CLAY (CH), Fat, Hard, Tan and Light Gray, with ferrous staining**
  - very stiff above 4 ft
  - blocky below 4 ft

**BORING TERMINATED**

**NOTES:**
1. Groundwater not encountered during drilling operations.
2. Borehole was backfilled with auger cuttings.

**LOCATION:** N 29°85589; W 97°92178

**SHEAR STRENGTH, TONS/FT²**

- **PLASTICITY INDEX**
- **DRILLING METHOD:**
- **LOCATION:**

**BLOWS PER FT**

- **UNIT DRY WEIGHT,pcf**
- **SHEAR STRENGTH, TONS/FT²**

**DEPTH, FT**

10 20 30 40 50 60 70 80

**SYMBOL SAMPLES**

**PROJ. No.:** AAA16-043-00

**FIGURE:** 6

**DEPTH DRILLED:** 15.0 ft

**DATE DRILLED:** 5/16/2016

**DEPTH TO WATER:** Dry

**DATE MEASURED:** 5/16/2016

**NOTE:** THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT.
### LOG OF BORING NO. B-6
Springtown Mixed-Use Development
1180 Thorpe Lane
San Marcos, Texas

**LOCATION:** N 29.88599; W 97.92251

**NOTES:**
1. Groundwater not encountered during drilling operations.
2. Borehole was backfilled with auger cuttings.

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>SAMPLES</th>
<th>DESCRIPTION OF MATERIAL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td></td>
<td>ASPHALT (2 inches)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>BASE (6 inches)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>CLAY (CH), Fat, Very Stiff, Grayish-Brown</td>
</tr>
<tr>
<td></td>
<td></td>
<td>- blocky below 12 ft</td>
</tr>
</tbody>
</table>

**Boring Terminated**

**DEPTH DRILLED:** 15.0 ft  **DATE DRILLED:** 5/16/2016  **DEPTH TO WATER:** Dry  **DATE MEASURED:** 5/16/2016  **PROJ. No.:** AAA16-043-00  **FIGURE:** 7
LOG OF BORING NO. B-7
Springtown Mixed-Use Development
1180 Thorpe Lane
San Marcos, Texas

LOCATION: N 29.88563; W 97.92288

DRILLING METHOD: Straight Flight Auger

DESCRIPTION OF MATERIAL:

- ASPHALT (4 inches)
- BASE (7 inches)
- FILL: CLAY, Fat, Very Stiff, Tan and Dark Brown, with gravel
- CLAY (CH), Fat, Very Stiff, Dark Brown to Brown
- CLAY (CH), Fat, Hard, Tan and Gray, with calcareous nodules
- blocky, tan below 17 ft

Boring Terminated

DEPT, FT | SYMBOL | SAMPLES |
---------|--------|--------|
40.0     |        |        |

NOTE: THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT

DEPT, FT | BLOWS PER FT | UNIT DRY WEIGHT, pcf | PLASTICITY INDEX |
---------|--------------|---------------------|----------------|
0.0      | 102          | 29.88563; 97.92288  | 52             |
2.0      | 111          | 34.0                | 34             |
5.0      | 107          | 30.0                | 34             |
10.0     | 108          | 29.8               | 50             |

SHEAR STRENGTH, TONS/FT²

DEPTH DRILLED: 40.0 ft
DATE DRILLED: 5/18/2016

DEPTH TO WATER: Dry
DATE MEASURED: 5/18/2016

PROJ. No.: AAA16-043-00
FIGURE: 8a
**NOTES:**
1. Groundwater not encountered during drilling operations.
2. Borehole was backfilled with auger cuttings.

---

**LOG OF BORING NO. B-7**

Springtown Mixed-Use Development  
1180 Thorpe Lane  
San Marcos, Texas

**LOCATION:** N 29.88563; W 97.92288

---

**DRILLING METHOD:** Straight Flight Auger  

<table>
<thead>
<tr>
<th>DEPTH, FT</th>
<th>SYMBOL</th>
<th>SAMPLES</th>
<th>DESCRIPTION OF MATERIAL</th>
<th>BLOWS PER FT</th>
<th>UNIT DRY WEIGHT, pcf</th>
<th>PLASTICITY INDEX % -200</th>
<th>SHEAR STRENGTH, TONS/FT²</th>
</tr>
</thead>
<tbody>
<tr>
<td>45</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>55</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>60</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>65</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>70</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>75</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>80</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**DEPTH DRILLED:** 40.0 ft  
**DATE DRILLED:** 5/18/2016  
**DEPTH TO WATER:** Dry  
**DATE MEASURED:** 5/18/2016  
**PROJ. No.:** AAA16-043-00  
**FIGURE:** 8b

**NOTE:** THESE LOGS SHOULD NOT BE USED SEPARATELY FROM THE PROJECT REPORT.
Log of Boring No. B-8
Springtown Mixed-Use Development
1180 Thorpe Lane
San Marcos, Texas

Drilling Method: Straight Flight Auger

Location: N 29.88557; W 97.92232

Driller's Note: Groundwater encountered at 36.1 ft

Depth Drilled: 50.0 ft
Date Drilled: 5/18/2016

Depth to Water: 36.1 ft
Date Measured: 5/18/2016

Proj. No.: A1A6-043-00
Figure: 9a

Description of Material:
- Asphalt (2 inches)
- Base (8 inches)
  CLAY (CH), Fat, Very Stiff, Dark Brown
  - Blocky, tan below 17 ft
- CLAY (CH), Fat, Light Gray-Brown, with calcareous nodules and gravel
- CLAY (CH), Fat, Tan and Gray, with calcareous nodules and ferrous staining

Plasticity Index:

<table>
<thead>
<tr>
<th>Plastic Limit</th>
<th>Liquidity Limit</th>
<th>Water Content</th>
<th>Blow per FT</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.5</td>
<td>1.0</td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>4.0</td>
<td></td>
<td>5.0</td>
<td>6.0</td>
</tr>
</tbody>
</table>

Unit Dry Weight, pcf:

N 29.88557; W 97.92232

Shear Strength, Tons/ft²:

0.5 1.0 1.5 2.0

Note: These logs should not be used separately from the project report.
**LOG OF BORING NO. B-8**

Springtown Mixed-Use Development  
1180 Thorpe Lane  
San Marcos, Texas

**DRILLING METHOD:** Straight Flight Auger

<table>
<thead>
<tr>
<th>SYMBOL</th>
<th>SAMPLES</th>
<th>DESCRIPTION OF MATERIAL</th>
<th>DEPTH, FT</th>
<th>BLOWS PER FT</th>
<th>UNIT DRY WEIGHT, pcf</th>
<th>PLASTIC LIMIT</th>
<th>LIQUID LIMIT</th>
<th>WATER CONTENT</th>
<th>PLASTICITY INDEX</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>CLAY (CH), Fat, Tan and Gray, with calcareous nodules and ferrous staining (continued)</td>
<td>45</td>
<td>108</td>
<td>29.88557; W 97.92232</td>
<td>0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0</td>
<td>5.00</td>
<td>52</td>
<td></td>
</tr>
</tbody>
</table>

Boring Terminated

**NOTES:**
1. After 10 minutes, groundwater measured at 34.7 feet.
2. Borehole was backfilled with auger cuttings.
KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

- CLAREOUS
- CALICE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY

ROCK TERMS

- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

OTHER

- CHALK
- CLAYSTONE
- CLAY-SHALE
- CONGLOMERATE
- DOLOMITE
- IGNEOUS
- LIMESTONE
- MARL
- METAMORPHIC
- SANDSTONE
- SHALE
- SILTSTONE

WELL CONSTRUCTION AND PLUGGING MATERIALS

- BLANK PIPE
- SCREEN
- BENTONITE
- CEMENT GROUT
- CUTTINGS
- CONCRETE/CEMENT
- GRAVEL
- VOLCLAY

SAMPLE TYPES

- AIR ROTARY
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC DAMAGED
- ROTOSONIC INTACT

STRENGTH TEST TYPES

- POCKET PENETROMETER
- TORVANE
- UNCONFINED COMPRESSION
- TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

NOTE: VALUES SYMBOLIZED ON BORING LOGS REPRESENT SHEAR STRENGTHS UNLESS OTHERWISE NOTED

PROJECT NO. AAA16-043-00

REVISED 04/2012

FIGURE 10a
KEY TO TERMS AND SYMBOLS (CONT’D)

TERMINOLOGY
Terms used in this report to describe soils with regard to their consistency or conditions are in general accordance with the discussion presented in Article 45 of SOILS MECHANICS IN ENGINEERING PRACTICE, Terzaghi and Peck, John Wiley & Sons, Inc., 1967, using the most reliable information available from the field and laboratory investigations. Terms used for describing soils according to their texture or grain size distribution are in accordance with the UNIFIED SOIL CLASSIFICATION SYSTEM, as described in American Society for Testing and Materials D2487-06 and D2488-00, Volume 04.08, Soil and Rock; Dimension Stone; Geosynthetics; 2005.

The depths shown on the boring logs are not exact, and have been estimated to the nearest half-foot. Depth measurements may be presented in a manner that implies greater precision in depth measurement, i.e. 6.71 meters. The reader should understand and interpret this information only within the stated half-foot tolerance on depth measurements.

<table>
<thead>
<tr>
<th>Penetration Resistance</th>
<th>Relative Density</th>
<th>Cohesive Strength</th>
<th>Plasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>Blows per ft</td>
<td>0 - 2</td>
<td>Very Soft</td>
<td>0 - 0.125</td>
</tr>
<tr>
<td></td>
<td>2 - 4</td>
<td>Soft</td>
<td>0.125 - 0.25</td>
</tr>
<tr>
<td></td>
<td>4 - 8</td>
<td>Firm</td>
<td>0.25 - 0.5</td>
</tr>
<tr>
<td></td>
<td>8 - 15</td>
<td>Stiff</td>
<td>0.5 - 1.0</td>
</tr>
<tr>
<td></td>
<td>15 - 30</td>
<td>Very Stiff</td>
<td>1.0 - 2.0</td>
</tr>
<tr>
<td></td>
<td>&gt; 30</td>
<td>Hard</td>
<td>&gt; 2.0</td>
</tr>
</tbody>
</table>

ABBREVIATIONS

B = Benzene  Qam, Qas, Qal = Quaternary Alluvium  Kef = Eagle Ford Shale
T = Toluene  Qat = Low Terrace Deposits  Kbu = Buda Limestone
E = Ethylbenzene  Qbc = Beaumont Formation  Kdr = Del Rio Clay
X = Total Xylenes  Qt = Fluvial Terrace Deposits  Kft = Fort Terrett Member
BTEX = Total BTEX  Qao = Seymour Formation  Kgt = Georgetown Formation
TPH = Total Petroleum Hydrocarbons  Qle = Leona Formation  Kep = Person Formation
ND = Not Detected  Q-Tu = Uvalde Gravel  Kek = Kainer Formation
NA = Not Analyzed  Ewi = Wilcox Formation  Kes = Escondido Formation
NR = Not Recorded/No Recovery  Emi = Midway Group  Kew = Walnut Formation
OVA = Organic Vapor Analyzer  Mc = Catahoula Formation  Kgr = Glen Rose Formation
ppm = Parts Per Million  EI = Laredo Formation  Kgru = Upper Glen Rose Formation
Kknm = Navarro Group and Marlbrook Marl  Kgrl = Lower Glen Rose Formation
Kpg = Pecan Gap Chalk  Kh = Hensell Sand
Kau = Austin Chalk

FIGURE 10b  REVISED 04/2012
KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

- **Slickensided**: Having planes of weakness that appear slick and glossy.
- **Fissured**: Containing shrinkage or relief cracks, often filled with fine sand or silt; usually more or less vertical.
- **Pocket**: Inclusion of material of different texture that is smaller than the diameter of the sample.
- **Parting**: Inclusion less than 1/8 inch thick extending through the sample.
- **Seam**: Inclusion 1/8 inch to 3 inches thick extending through the sample.
- **Layer**: Inclusion greater than 3 inches thick extending through the sample.
- **Laminated**: Soil sample composed of alternating partings or seams of different soil type.
- **Interlayered**: Soil sample composed of alternating layers of different soil type.
- **Intermixed**: Soil sample composed of pockets of different soil type and layered or laminated structure is not evident.
- **Calcareous**: Having appreciable quantities of carbonate.
- **Carbonate**: Having more than 50% carbonate content.

SAMPLING METHODS

**RELATIVELY UNDISTURBED SAMPLING**

Cohesive soil samples are to be collected using three-inch thin-walled tubes in general accordance with the Standard Practice for Thin-Walled Tube Sampling of Soils (ASTM D1587) and granular soil samples are to be collected using two-inch split-barrel samplers in general accordance with the Standard Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586). Cohesive soil samples may be extruded on-site when appropriate handling and storage techniques maintain sample integrity and moisture content.

**STANDARD PENETRATION TEST (SPT)**

A 2-in.-OD, 1-3/8-in.-ID split spoon sampler is driven 1.5 ft into undisturbed soil with a 140-pound hammer free falling 30 in. After the sampler is seated 6 in. into undisturbed soil, the number of blows required to drive the sampler the last 12 in. is the Standard Penetration Resistance or "N" value, which is recorded as blows per foot as described below.

**SPLIT-BARREL SAMPLER DRIVING RECORD**

<table>
<thead>
<tr>
<th>Blows Per Foot</th>
<th>Description</th>
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<tbody>
<tr>
<td>25</td>
<td>25 blows drove sampler 12 inches, after initial 6 inches of seating.</td>
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<tr>
<td>50/7&quot;</td>
<td>50 blows drove sampler 7 inches, after initial 6 inches of seating.</td>
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<tr>
<td>Ref/3&quot;</td>
<td>50 blows drove sampler 3 inches during initial 6-inch seating interval</td>
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</table>

**NOTE**: To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.
## RESULTS OF SOIL SAMPLE ANALYSES

**PROJECT NAME:** Springtown Mixed-Use Development  
1180 Thorpe Lane  
San Marcos, Texas

**FILE NAME:** AAA16-043-00.GPJ  
6/10/2016

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<th>Boring No.</th>
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<th>Water Content (%)</th>
<th>Liquid Limit</th>
<th>Plastic Limit</th>
<th>Plasticity Index</th>
<th>USCS</th>
<th>Dry Unit Weight (pcf)</th>
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**Notes:**  
PP = Pocket Penetrometer  
TV = Torvane  
UC = Unconfined Compression  
FV = Field Vane  
UU = Unconsolidated Undrained Triaxial  
CU = Consolidated Undrained Triaxial

**FIGURE 11a**

**PROJECT NO. AAA16-043-00**
# RESULTS OF SOIL SAMPLE ANALYSES

**PROJECT NAME:** Springtown Mixed-Use Development  
1180 Thorpe Lane  
San Marcos, Texas

**FILE NAME:** AAA16-043-00.GPJ  
6/10/2016

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PP = Pocket Penetrometer  
TV = Torvane  
UC = Unconfined Compression  
FV = Field Vane  
UU = Unconsolidated Undrained Triaxial  
CU = Consolidated Undrained Triaxial  

**PROJECT NO.** AAA16-043-00

**FIGURE 11b**
**Specimen Identification**

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<th>Specimen Identification</th>
<th>Classification</th>
<th>LL</th>
<th>PL</th>
<th>PI</th>
<th>Cc</th>
<th>Cu</th>
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<tr>
<td>B-4</td>
<td>FAT CLAY(CH)</td>
<td>53</td>
<td>16</td>
<td>37</td>
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**Specimen Identification**

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<tr>
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<th>$G_s$</th>
<th>2 mm</th>
<th>0.425 mm</th>
<th>0.075 mm</th>
<th>0.02 mm</th>
<th>0.002 mm</th>
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<td>99.7</td>
<td>99.5</td>
<td>99.1</td>
<td>96.7</td>
<td>53.0</td>
</tr>
</tbody>
</table>

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FIGURE 12
Geotechnical 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 civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared solely for the client. No one except you 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 the Full Report
Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors
Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client’s goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:
• not prepared for you;
• not prepared for your project;
• not prepared for the specific site explored; or
• completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:
• 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. Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.

Subsurface Conditions Can Change
A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. Do not rely on a geotechnical-engineering report whose adequacy may have been affected by: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. Contact the geotechnical engineer before applying this report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions
Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report’s Recommendations Are Not Final
Do not overrely on the confirmation-dependent recommendations included in your report. Confirmation-dependent recommendations are not final, because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. The geotechnical engineer who developed your report cannot assume responsibility or liability for the report’s confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations’ applicability.

A Geotechnical-Engineering Report Is Subject to Misinterpretation
Other design-team members’ misinterpretation of geotechnical-engineering reports has resulted in costly
problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team’s plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer’s Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should never be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, but recognize that separating logs from the report can elevate risk.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, but preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report’s accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. Be sure constructors have sufficient time to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an environmental study differ significantly from those used to perform a geotechnical 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 environmental problems have led to numerous project failures. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. Do not rely on an environmental report prepared for someone else.

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the express purpose of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold-prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical-engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; none of the services performed in connection with the geotechnical engineer’s study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of the Geoprosfessional 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. Confer with your GBC-Member geotechnical engineer for more information.
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Houston, TX
McAllen, TX
Mexico
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