# GEOTECHNICAL ENGINEERING STUDY

FOR

# LIFT STATION AND WASTEWATER TREATMENT FACILITY
# NB WEST
# NEW BRAUNFELS, TEXAS

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Project No. ANA24-027-00
October 17, 2024

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**RE: Geotechnical Engineering Study**
**Lift Station and Wastewater Treatment Facility**
**NB West**
**New Braunfels, Texas**

Dear Mr. Mullins:

RABA KISTNER, Inc. (**RKI**) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with **RKI** Proposal No. PNA24-044-00 dated June 25, 2024. The purpose of this study was to drill borings in the vicinity of the proposed lift station and wastewater treatment facility, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations.

The following report contains our design recommendations and considerations based on our current understanding of information provided to us at the time of this study. There may be alternatives for value engineering of the foundation. **RKI** 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, INC.**

Santosh Shrestha, E.I.T.
Graduate Engineer

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SS/TIP/mmd
Attachments
Copies Submitted: Above (1-electronic)

**GEOTECHNICAL ENGINEERING STUDY**

For

**LIFT STATION AND WASTEWATER TREATMENT FACILITY**
**NB WEST**
**NEW BRAUNFELS, TEXAS**

Prepared for

**CHESMAR HOMES**
San Antonio, Texas

Prepared by

**RABA KISTNER, INC.**
New Braunfels, Texas

**PROJECT NO. ANA24-027-00**

October 17, 2024

RABA KISTNER

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## ATTACHMENTS

The following figures are attached and complete this report:

Boring Location Map ........ Figure 1
Logs of Borings ........ Figures 2 through 6
Key to Terms and Symbols ........ Figure 7
Results of Soil Sample Analyses ........ Figure 8
Important Information About Your Geotechnical Engineering Report

## INTRODUCTION

RABA KISTNER, Inc. (**RKI**) has completed the authorized subsurface exploration and foundation analysis for the proposed lift station and wastewater treatment facility (WWTF) at the NB West development in New Braunfels, Texas, as illustrated in Figure 1. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations.

## PROJECT DESCRIPTION

To be considered in this study is a new wastewater treatment facility (WWTF) and separate lift station at the NB West development in New Braunfels, Texas. The site is located on the westernmost extents of the proposed development just north of the northernmost extent of Oak Creek Drive. There is not a site plan depicting the proposed structures currently available for review; however, on the basis of information provided by the designer, JA Wastewater, LLC via email, we understand that the WWTF will include the following structures:

- Influent lift station about 20 ft deep, consisting of fiberglass reinforced polymer (FRP) or concrete;
- Wastewater plant with slab on grade foundation;
- Controls/operations/electrical building with a slab on grade foundation; and
- Effluent lift station about 15 ft deep, consisting of fiberglass reinforced polymer (FRP) or concrete.

In addition to these structures, another lift station is planned east of the WWTF and is currently planned near the proposed thoroughfare road, south of the currently proposed location of Phase 8 of the development. It is our understanding that the lift station will have a maximum depth of 20 ft and will be constructed of either FRP or concrete.

## 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 Chesmar Homes (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 5 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. This is particularly true of this site with respect to the depth of the upper surficial clays and the potential presence of solution cavities and/or voids that may not have been encountered in our test borings. 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 site grading results in elevations that vary significantly from the existing grades (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 5 borings drilled at the locations shown on the Boring Location Map, Figure 1. These locations are approximate, and distances were measured using a hand-held, recreational-grade GPS locator. The borings were drilled to depths ranging from approximately 30 to 40 ft below the existing ground surface using a truck-mounted drilling rig.

During the drilling operations, split-spoon samples with Standard Penetration Tests (SPT) were collected at the depths annotated on our boring logs. 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 moisture content and Atterberg Limits tests.

The laboratory test results are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 6. A key to classification terms and symbols used on the logs is presented on Figure 7. The results of the laboratory and field testing are also tabulated on Figure 8 for ease of reference.

Standard Penetration Test results (N-values) are noted as “blows per ft” on the boring logs and on Figure 8. The N-value is the number of blows required to drive a split-spoon sampler 1 ft into soil/weak rock with a falling, 140-lb hammer following 6 inches of seating blows. Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal (“ref”) will be noted on the boring logs and on Figure 8.

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.

## GENERAL SITE CONDITIONS

### GEOLOGY

A review of the *Geologic Atlas of Texas, San Antonio Sheet*, indicates that this site is naturally underlain with the soils/rock (limestone) of the Edwards Group. Edwards limestone is generally considered hard in induration and typically contains harder zones/seams of chert and dolomite. Edwards limestone also typically contains karstic features in the form of open and/or clay-filled vugs, voids, and/or solution cavities that form as a result of solution movement through fractures in the rock mass.

Key geotechnical engineering considerations for development supported on this formation will be the depth to rock, the expansive nature of the overlying clays, the condition of the rock, and the presence/absence of karstic features.

## SEISMIC CONSIDERATIONS

The following information has been summarized for seismic considerations associated with this site per ASCE 7-16 edition.

- Site Class Definition: **Class C**. Based on the soil borings conducted for this investigation and our experience in the area, the upper 100 ft of soil may be characterized as very dense soil and soft rock.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% Of Critical Damping): $\mathbf{S_s = 0.050g}$.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% Of Critical Damping): $\mathbf{S_1 = 0.027g}$.
- Values of Site Coefficient: $\mathbf{F_a = 1.3}$
- Values of Site Coefficient: $\mathbf{F_v = 1.5}$
- Where g is the acceleration due to gravity.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec, adjusted: $\mathbf{S_{ms} = 0.066g}$
- 1 sec, adjusted: $\mathbf{S_{m1} = 0.041g}$

The Design Spectral Response Acceleration Parameters (SA) are as follows:

- 0.2 sec SA: $\mathbf{S_{DS} = 0.044g}$
- 1 sec SA: $\mathbf{S_{D1} = 0.027g}$

## STRATIGRAPHY

The natural subsurface stratigraphy can generally be described as a thin veneer of highly plastic dark brown clay with limestone fragments overlying tan and gray limestone. In Borings B-3 and B-4, marl overlays the tan and gray limestone. The limestone was encountered at approximate depths ranging from 2.5 ft to 13 ft below the ground surface existing at the time of our study and extends to at least the boring termination depths in all of the borings drilled for this study.

The boring logs should be consulted for more specific stratigraphic information. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by **RKI** in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

The boring logs and related information depict subsurface conditions only at the specific locations and times where sampling was conducted. The passage of time may result in changes in conditions, interpreted to exist, at or between the locations where sampling was conducted.

## GROUNDWATER

Groundwater was not observed in the borings either during or immediately upon completion of the drilling operations. The 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.

# FOUNDATION ANALYSIS

## KARSTIC FEATURES

The site is located in an area known to have karst topography (i.e. open and/or clay-filled vugs, voids, and/or solution cavities in the bedrock). The potential presence of karst features in the vicinity of the site introduces some element of risk and uncertainty for design, construction and performance of the proposed structures. Depending on the final site grading plan, foundation depth and the top of bedrock, boulders, pinnacles, ledge rock (stringers), or clayed filled solution features may be encountered near or at the required bearing stratum. Considerable variation in the bearing elevation and quantity of rock excavation should be anticipated. Appropriate contingency fees should be allocated for removal of weathered limestone and extending foundations through karstic features.

## 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 value of 1 in. or less 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 or to the depth of bedrock, 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 (such as due to leaks, landscape watering....) or if water seeps into the soils to greater depths than the assumed active zone depth due to deep trenching or excavations.

## MITIGATION OF EXPANSIVE SOIL-RELATED MOVEMENTS

Because the estimated PVR values are on the order of the generally accepted 1 in. or less, no mitigation is required to reduce the PVR. Fill utilized to achieve the final grade elevations should be selected and placed in accordance with the *Select Fill* section of this report in order to maintain the estimated existing PVR values.

# FOUNDATION RECOMMENDATIONS

## 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 changes, **RKI** must be retained to review the site grading plans prior to bidding the project for construction. This will enable **RKI** to provide input for any changes in our original recommendations that may be required as a result of site grading operations or other considerations.

## WET WELL STRUCTURE

Based on the information provided to us, the proposed wet well structures/lift station will extend approximately 15 to 20 ft below the grade existing at the time of our study. The excavation method for the construction of the wet well was not known at the time of this report. If open cut excavation techniques are utilized, the maximum side slopes shall be in accordance to *Excavations and Temporary Slopes* section of this report.

### Allowable Bearing Capacity

Foundations for the wet well bearing in hard limestone at an approximate depth of 15 to 20 feet below the existing ground surface should be designed for a maximum allowable bearing pressure of 18 ksf. The above presented maximum allowable bearing pressure will provide a factor of safety of about 3 with respect to the measured shear strength.

## RIGID-ENGINEERED BEAM AND SLAB FOUNDATION

Proposed wastewater plant, controls/operations/electrical buildings or any other ancillary structures, if any, may be founded on a shallow 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.

### Differential Settlement in Transition Zone

**To reduce the potential for differential settlement at soil/fill and rock transitions, the more positive approach for foundation support would be to extend all footings to rock/marl.** Alternatively, the footings may bear on a combination of soil/fill and rock if differential movements can be tolerated. With footings on mixed bearing conditions, the client must recognize and accept a greater than normal risk of differential settlement as hinges may occur at unpredictable locations due to the irregular occurrence of shallow bedrock. Special provisions that should be considered for footings bearing on mixed bearing materials (natural soil/ fill and rock) to reduce the effects of differential settlement include the following:

- Frequent jointing of exterior walls;
- Selection of flexible building veneer materials; and
- Overexcavation of footing subgrades to top of rock and backfilling with compacted crushed rock.

**Allowable Bearing Capacity**

| Shallow Foundation Design Parameters | |
|---|---|
| Minimum depth below final grade | 18 in. [1] |
| Minimum beam or strip footing width | 12 in. |
| Minimum widened beam or spread footing width | 18 in. |

(1) If intact bedrock is encountered, minimum foundation depth should be discussed with the structural engineer, but may be reduced to 12 in.

| Shallow Foundation Type | Maximum Allowable Bearing Pressure |
|---|---|
| Grade Beams or strip footings | 3,000 psf |
| Widened beams or spread footings | 3,500 psf |
| Foundations on intact or weathered limestone | 4,500 psf [1] |

(1) Mixed bearing conditions (i.e. bearing on soil/fill and bedrock) should be avoided to reduce potential for differential settlement.

We do not recommend that the grade beams for an individual structure be founded partially in bedrock and partially in natural soils or compacted fill as this condition may result in greater differential movements. **If mixed bearing conditions are encountered, we recommended that all grade beams either be extended down into the bedrock, or if constructed on a select fill building pad, that a minimum of 1 ft of select fill be placed and compacted beneath the grade beams.**

The above presented maximum allowable bearing pressures will provide a factor of safety of about 3, provided that fill is placed as discussed herein and the subgrade is prepared in accordance with the recommendations outlined in the *Site Preparation* section of this report.

Depending on the structural loads and if higher bearing pressures are requested/desired, rock-bearing shallow foundations proportioned for greater than 4,500 psf bearing pressures may require additional probe borings with pilot holes at actual foundation locations. Alternatively, the requirement for pilot hole or probe holes may be waived if the bearing pressure provided herein is used.

Rock bearing foundations should bear on relatively competent rock, which may underlie a few feet of weathered rock. The foundations should be excavated through the weathered rock to expose competent rock. Excavation into the limestone will require hard rock excavation techniques. The bottom of the excavation should generally be level; however, it is permissible to excavate vertical steps if required to expose sound bedrock. Loose rock should be removed from all foundation excavations. Overexcavation may be backfilled with lean concrete or flowable fill.

The foundation subgrade 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 materials at the bottom of the excavations are similar to those encountered in our borings, that excessive loose materials, mixed bearing conditions, and water are not present in the excavations. If soft soils are encountered in the foundation excavations, they should be removed and replaced with compacted engineered fill material, flowable fill, or lean concrete up to the design foundation bearing elevations.

### Uplift Resistance

Resistance to vertical force (uplift) is provided by the weight of the concrete footing plus the weight of the soil directly above the footing. For this site, it is recommended that the ultimate uplift resistance be based on total unit weights for soil and concrete of 120 pcf and 150 pcf, respectively. The calculated ultimate uplift resistance should be reduced by a factor of safety of 1.2 to calculate the allowable uplift resistance.

### Lateral Resistance

Horizontal loads acting on shallow foundations will be resisted by passive earth pressure acting on one side of the footing and by base adhesion for footings in soil or limestone. Resistance to sliding for foundations bearing on natural/compacted soil or limestone should be calculated utilizing an ultimate coefficient of friction of 0.30 or 0.70, respectively. The ultimate resistance for these foundations should be limited to 1,050 psf (soil) or 3,150 psf (rock). An equivalent fluid pressure of 240 pcf (soil) or 350 pcf (rock) should be utilized to determine the ultimate passive resistance, if required.

## AREA FLATWORK

It should be noted that ground-supported flatworks such as walkways, courtyards, etc. 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 structure foundations, differential movements should be anticipated. 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 the potential for differential movement is objectionable, it may be beneficial to consider methods of reducing anticipated movements such as transitioning the select fill building pad to beneath critical sections of flatwork.

For flatwork supported by 6 inches of compacted crushed rock, a subgrade modulus (k-value) of 150 pci may be utilized for slabs constructed for this project. The subgrade modulus may be increased to 250 pci if the floor slabs and flatwork are underlain by 2 feet or more of compacted aggregate select fill.

## DRILLED PIERS

Deep foundations (i.e. drilled, straight-shaft piers) bearing in competent limestone may be considered to support the structure. Consequently, pier capacity may be equal to the summation of the following:

- The end area of the pier multiplied by the allowable end-bearing pressure; and
- The wall area of the pier socket below a depth of 5 ft into the underlying bedrock surface area multiplied by the allowable side shear resistance.

For shafts excavated in limestone with potential karstic features, pilot holes can be drilled at the bottom of the pier excavations to evaluate the presence of voids near the bottom of the shaft. Pilot holes are performed at the time of drilled pier construction and consist of 2-inch diameter holes drilled from the bottom of excavated shafts to a depth equal to two pier diameters below the bottom of each pier.

An allowable end-bearing pressure of 40 ksf may be utilized for piers where pilot holes are performed. This bearing pressure was calculated using a factor of safety of 3. If pilot holes are not performed, then we recommend piers be designed using side friction only. We recommend that drilled, straight-shaft piers extend a minimum of 5 ft into native, intact limestone. An allowable side shear resistance of 3.5 ksf may be utilized for the portion of the shaft extending to a minimum depth of 5 ft or 2 pier shaft diameters, whichever results in the lower elevation, into the native, intact limestone layer. This is based on a factor of safety of 2 with respect to the design shear strength. These values may be increased by 1/3 for transient load conditions.

Side shear should be neglected in fill material, clay layers, voids, and/or clay filled voids. Based on the maximum depth of exploration, piers should be sized such that the pier bottom does not extend deeper than 35 ft without prior review and approval from **RKI** and/or observations of the pier/pilot holes confirm the presence of native, intact limestone the full depth of the pier and below.

Final shaft depths will be based on interpretation of conditions in the field at the time of construction. If clay seams/and or voids are encountered within the limestone formation during drilled shaft excavations, the shafts must be extended by that length to develop the required side shear resistance.

Representatives from **RKI** must be present at the time of construction to verify that conditions are similar to those encountered in our borings and that sufficient penetration into the limestone is achieved. 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.

Due to the presence of limestone high-powered, high-torque drilling equipment should be anticipated for drilled pier construction at this site (see also Excavation Equipment).

Excavations for grade beams may be performed vertically. In addition, since the grade beams will be excavated in limestone or select fill, carton forms are not required and may bear on the exposed bedrock or select fill.

**<u>Pier Shaft Potential Uplift Forces</u>**

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:

$$F_u = 15*D$$

where:

$F_u$ = uplift force in kips; and
D = diameter of the shaft in ft.

**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 bedrock. The allowable uplift resistance provided by the bedrock at this site may be estimated using 2 ksf for that portion of the shaft penetrating the limestone, respectively, and neglecting the upper 5 ft into the native, intact limestone layer.

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.

**Pier Spacing**

Where possible, we recommend that the piers be spaced at a center-to-center distance of at least three shaft diameters for straight-shaft 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 pier diameters, RKI 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, spacing conditions and settlement tolerances.

**Lateral Resistance**

Resistance to lateral loads and the expected pier behavior under the applied loading conditions will depend not only on subsurface conditions, but also on loading conditions, the pier size, and the engineering properties of the pier. As this information is not yet available, analysis of pier behavior is not possible at this time. Once preliminary pier sizes, concrete strength, and reinforcement are known, piers should be analyzed to determine the resulting lateral deflection, maximum bending moment, and ultimate bending moment. This type of analysis is typically performed utilizing a computer analysis program and usually requires a trial-and-error procedure to appropriately size the piers and meet project tolerances.

To assist the design engineer in this procedure, we are providing the following soil parameters for use in analysis. These parameters are in accordance with the input requirements of one of the more commonly used computer programs for laterally loaded piles, the LPile program. If a different program is used for analysis, different parameters and limitations may be required than what were assumed in selecting the parameters given below. Thus, if a program other than LPile is used, **RKI** must be notified of the analysis method, so that we can review and revise our recommendations if required.

| Assumed Behavior for Analysis | Material | c (psf) | $k_s$ (pci) | $\varepsilon_{50}$ | γ (pcf) | γ'(pcf) | $q_u$ (psi) |
|---|---|---|---|---|---|---|---|
| Soft Clay (Matlock) | Soil Overburden | 500 | 30 | 0.020 | 115 | 53 | ---- |
| Strong Rock (Vuggy Limestone) | Limestone | ---- | ---- | ---- | 140 | 78 | 1,000[(1)] |

(1) Based on our experience with the Edwards Limestone formation.

Where:

c = undrained cohesion
$k_s$ = p-y modulus
$\varepsilon_{50}$ = strain factor
γ = total unit weight
γ' = effective unit weight
$q_u$ = unconfined compressive strength

The values presented above for subgrade modulus and the strain at 50% are based on recommended values for the LPile program for the strength of materials encountered in our borings and are not necessarily based on laboratory test results.

The parameters presented in the above table do **not** include factors of safety nor have they been factored. It should be noted that where piers are spaced closer than three shaft diameters center to center, a modification factor should be applied to the p-y curves to account for a group effect. We recommend the following p-Multipliers for the corresponding center to center pier spacing to determine factored lateral loads. The reduction factors presented below are applicable for lateral resistance but is not intended for use to reduce the allowable bearing or side shear resistance values presented for axial capacity. If piers are utilized that impede the 3-shaft diameter spacing, reduction values should be evaluated on a case-by-case basis when the pier geometries, spacing, and loading are available.

| Spacing (in shaft diameters) | p-Multiplier |
|---|---|
| 3 | 1.0 |
| 2 | 0.75 |
| 1 | 0.50 |

## RETAINING STRUCTURES

Retaining walls and foundation stem walls are anticipated to accommodate potential grade changes; however, the locations, heights, and other important information are not available at this time. The following sections provide general information for evaluating lateral earth pressures, backfill compaction, drainage, and the footings for the walls. Discussion on vertical rock cuts is also provided herein.

## LATERAL EARTH PRESSURES

Equivalent fluid density values for computation of lateral soil pressures acting on walls were evaluated for various types of backfill materials that may be placed behind the walls. These values, as well as corresponding lateral earth pressure coefficients and estimated unit weights, are presented in the following table.

| Back Fill Type | Estimated Total Unit Weight (pcf) | Active Condition | | At Rest Condition | |
|---|---|---|---|---|---|
| | | Earth Pressure Coefficient, $k_a$ | Equivalent Fluid Density (pcf) | Earth Pressure Coefficient, $k_o$ | Equivalent Fluid Density (pcf) |
| Washed Gravel | 135 | 0.29 | 40 | 0.45 | 60 |
| Crushed Limestone | 145 | 0.24 | 35 | 0.38 | 55 |
| Clean Sand | 120 | 0.33 | 40 | 0.50 | 60 |
| Pit Run Clayey Gravels or Sands | 135 | 0.32 | 45 | 0.48 | 65 |
| Inorganic Clays of Low to Medium Plasticity (Liquid Limit less than 40 percent) | 120 | 0.40 | 50 | 0.55 | 65 |
| Clays | 120 | 0.59 | 70 | 0.74 | 90 |

The values tabulated above under “Active Conditions” pertain to flexible retaining walls free to tilt outward as a result of lateral earth pressures. For rigid, non-yielding walls (i.e. foundation stem walls) the values under “At-Rest Conditions” should be used. For the above values to be valid for washed gravel, crushed limestone, clean sand, or pit clayey gravels/sands backfill, the backfill should be placed in a wedge extending upward and away from the edge of the wall footing at a 45-degree angle or flatter. If the materials are to be placed with a steeper wedge, the values for low to medium plasticity soil, given above, should be used.

The values presented above assume the surface of the backfill materials to be level. Sloping the surface of the backfill materials will increase the surcharge load acting on the structures. The above values also do not include the effect of surcharge loads such as construction equipment, vehicular loads, or future storage near the structures. Nor do the values account for possible hydrostatic pressures resulting from groundwater seepage entering and ponding within the retained backfill materials. As discussed later, the walls should be provided with a drain system to allow for the dissipation of water. Surcharge loads and groundwater pressures should be considered in designing any structures subjected to lateral pressures.

The onsite surficial dark brown clays exhibit significant shrink/swell characteristics. The use of clay soils as backfill against the proposed retaining structures is **not** recommended. These soils generally provide higher design active earthen pressures, as indicated above, but may also exert additional active pressures associated with swelling. Controlling the moisture and density of these materials during placement will help reduce the likelihood and magnitude of future active pressures due to swelling, but this is no guarantee.

## BACKFILL COMPACTION

Placement and compaction of backfill behind the walls will be critical, particularly at locations where backfill will support adjacent near-grade foundations, floor slabs, and/or flatwork. **If the backfill is not properly compacted in these areas, the adjacent foundations floor slabs, or flatwork can be subject to settlement**.

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test, or 98 percent of maximum density as determined by ASTM D698. **To reduce the potential settlement, fills greater than 8 ft should be compacted to at least 95 percent of maximum density as determined by ASTM D 1557, Modified Compaction Test and should be crushed limestone conforming to the 2024 TxDOT Standard Specifications, Item 247 – Flexible Base, Type A, Grade 1-2**. 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. Note that free-draining gravel materials are not typically tested for density and moisture content, but rather monitored by observation. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the walls, we recommend that compaction be accomplished using hand-guided compaction equipment capable of achieving the maximum density in a series of 3 to 5 passes. Thinner lifts may be required to achieve the required level of compaction.

## DRAINAGE

The use of drainage systems is a positive design step toward reducing the possibility of hydrostatic pressure acting against the retaining structures. Drainage may be provided by the use of a drain trench and pipe. The drainpipe should consist of a slotted, heavy duty, corrugated polyethylene pipe and should be installed and bedded according to the manufacturer's recommendations. The drain trench should be filled with gravel (meeting the requirements of ASTM D 448 coarse concrete aggregate Size No. 57 or 67) and extend from the base of the structure to within 2 ft of the top of the structure. The bottom of the drain trench will provide an envelope of gravel around the pipe with minimum dimensions consistent with the pipe manufacturer's recommendations. The gravel should be wrapped with a suitable geotextile fabric (such as Mirafi 140N or equivalent) to help minimize the intrusion of fine-grained soil particles into the drain system. The pipe should be sloped and equipped with clean-out access fittings consistent with state-of-the-practice plumbing procedures.

As an alternative to a full-height gravel drain trench behind the proposed retaining structures, consideration may be given to utilizing a manufactured geosynthetic material for wall drainage. A number of products are available to control hydrostatic pressures acting on earth retaining structures, including Amerdrain (manufactured by American Wick Drain Corp.), Miradrain (manufactured by Mirafi, Inc.), Enkadrain (manufactured by American Enka Company), and Geotech Insulated Drainage Panel (manufactured by Geotech Systems Corp.). The geosynthetics are placed directly against the retaining structures and are hydraulically connected to the gravel envelope located at the base of the structures.

Weepholes may be provided along the length of the proposed retaining structures, if desired, in addition to one of the two alternative drainage measures presented above. Based on our experience, weepholes, as the only drainage measure, often become clogged with time and do not provide the required level of drainage from behind retaining structures. We recommend that **RKI** review the final retaining structure drainage design before construction.

## VERTICAL ROCK CUTS

The project site is underlain by the Edward Limestone formation. The Edward Limestone is karstic and may have undergone variable degrees of weathering (i.e. fractures, voids, clay filled voids, caves, weathered material, or other solution features). Where competent limestone bedrock is exposed, cuts into this material may be performed vertically. However, it is not uncommon to encounter karstic features in the limestone bedrock. Exposed limestone bedrock that contains these features can exhibit a characteristic mode of slope failure known as raveling. This failure mechanism involves raveling of the rock/other material along fractures, bedding planes, seams, and other pre-existing planes of weakness, resulting in the separation of blocks, weathered material or soil. Cobble- to boulder-sized blocks will eventually become dislodged as the result of this process and fall from the cut wall. The raveling process can be exacerbated by the presence of existing dissolution or karstic features in the rock, and by discharge of perched groundwater, if any, through the face of the rock cut.

Owing to increased moisture conditions typically associated with fractures, tree roots and other vegetation tend to exploit these weaknesses in the rock outcrop and serve to enhance the rate of erosion. As tree roots, etc. proliferate through fractures, fractures are enlarged owing to both mechanical and chemical erosional processes. Raveling failures can be expected to occur more frequently when these conditions occur.

In most instances, near-vertical rock slopes or cuts can be unprotected and unsupported provided that an adequate catchment area or buffer area is provided at the toe to prevent rockfall from affecting adjacent improvements. A flat catchment area should be at least 0.5 times the height in width. In areas where adequate catchment cannot be provided due to right-of-way or other geometrical constraints, the slope should be protected from raveling and differential erosion or laid back at a 1 Vertical to 1 Horizontal slope, or flatter. In addition to these protective measures, seepage, and surface water control to prevent stormwater from flowing over and down the face of the cut are essential in minimizing raveling and erosion.

For fixed-head walls that may be formed against the exposed competent limestone bedrock, we recommend that the following lateral pressure be used:

$$p_h = 45h + 0.3q \text{ (for fixed-head walls)}$$

Where:

$p_h$ = lateral pressure at any depth h, psf
h = depth below adjacent grade, feet
q = surcharge loads, psf

The above equation does not account for hydrostatic pressures. The walls should be designed to withstand the hydrostatic pressures and/or designed with a drainage system.

## 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 foundation and to facilitate rapid drainage away from the foundation. Failure to provide positive drainage away from the structure can result in localized differential vertical movements in soil supported foundations and floor slabs.

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

Where a select fill overbuild is provided outside of the floor slab/foundation footprint, if any, 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.

### SITE PREPARATION

All the areas to support select fill/slab should be stripped of all vegetation, organic topsoil, existing fill, if any, pavements, utilities and associated backfill.

Exposed subgrades should be thoroughly proofrolled in order to locate weak, compressible zones. A fully-loaded tandem wheeled 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 engineered fill, free of organics, oversized materials, and degradable or deleterious materials.

In areas where clay will remain in place or where clays remain after stripping, the exposed subgrade should be moisture conditioned. This should be done after completion of the proofrolling operations and just prior to fill placement and/or slab/foundation construction. Moisture conditioning is done 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 or ASTM D698, 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. Moisture conditioning of the subgrade may be waived where native, intact limestone is exposed.

### ONSITE SOIL

The use of onsite expansive soils may be a considered for general fill (outside of the building footprint) if the potential vertical movements in excess of those discussed previously will not adversely impact either the structural or operational tolerances for the proposed improvements for which this material is being considered.

## ON-SITE ROCK FILL

If excavations extend to significant depths into the limestone formation, consideration can be given to utilizing the excavated limestone for select fill. However, processing of the excavated material will be required to reduce the maximum particle size to 4 in. Furthermore, special care will be required during excavation activities to separate organics and any plastic clay seams encountered. In addition, the processed material must meet the specifications given above for alternative select fill materials. If on-site materials cannot be processed to meet the required criteria, imported select fill materials should be utilized.

## SELECT FILL

Materials used as select fill preferably should be imported crushed limestone base materials consisting of crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2024 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Type A or B, Grades 1-2 or 3.

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

### Select Fill Placement and Compaction

Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test, or 98 percent of maximum density as determined by ASTM D698. If fill materials supporting movement sensitive structures are placed that are 8 ft or thicker, we recommend that ASTM D1557 Modified Compaction Test be utilized in lieu of the above compaction methods. 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 for imported crushed limestone base.

### General Fill Placement and Compaction

The remaining fill may be compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-114-E, Compaction Test, or ASTM D698. The moisture content of the fill should be maintained within the range of optimum to plus 3 percentage points above the optimum moisture content until final compaction.

## 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 boring 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 **RKI** 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 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 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.

If clay seams and/or voids are encountered within the limestone formation during drilled shaft excavations, the shafts must be extended to develop the required side shear resistance. 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.

### 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.

## EXCAVATIONS AND TEMPORARY SLOPES

Depending on the planned improvement depth(s), temporary slopes or retention systems may be required. In areas where back slopes are feasible and have heights less than 20 ft, excavation slopes should be consistent with safety regulations. Worker safety and classification of soil type is the responsibility of the contractor. The surficial soils encountered during the borings are anticipated to consist of relatively hard fine-grained soils. Hence, temporary slopes should be classified as OSHA Type A soil. Excavations into intact/competent bedrock may be performed vertically. If weathered bedrock is encountered and depending on the degree of weathering, this material may be considered as Type A material. Additional recommendations are provided in the *Vertical Rock Cuts* section of this report.

For Type A material, the temporary slopes may be constructed at 3/4V:1H. Excavations extending deeper than 20 ft must be evaluated by a professional engineer.

The contractor should be aware that excavation depths and inclinations (including adjacent existing slopes) should not exceed those specified in local, state, or federal safety regulations, e.g., OSHA Health and Safety

Standards for Excavations, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the contractor, or earthwork or utility subcontractors could be subjected to substantial penalties. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations.

Temporary slopes left open may undergo sloughing and result in an unstable situation. The contractor should evaluate stability and failure consequences before open cut slopes are made. Minor sloughing of open face slopes may occur. If the slope is expected to remain open for an extended time, an impermeable membrane covering the slopes could be considered as a means to reduce the potential for slope degradation and instability.

It is important to note that soils encountered in the construction excavations may vary across the site and that even if the OSHA criteria are used, there is a potential for slope failure. If different subsurface conditions are encountered at the time of construction, **RKI** should be contacted to evaluate the conditions encountered.

An excavated temporary slope may not be feasible at all locations, and a temporary retention system may be required. While many different types and configurations of retention systems can be used, the more common include trench boxes or braced systems. The design of the system should be performed by the contractor that performs the work. The design should account for the possibility of overexcavating unsuitable or disturbed subgrades. The contractor should also be responsible for monitoring the performance of the retention system. OSHA regulations should be followed with respect to bracing requirements. Worker safety and classification of soil type is the responsibility of the contractor.

## EXCAVATION EQUIPMENT

**Please note that limestone bedrock was encountered in our boring at relatively shallow depths below the existing ground surface. Therefore, excavations at this site will require removal of the underlying rock formation. The Edwards limestone is hard to very hard in induration, is massive, and commonly contains chert seams. Consequently, excavations penetrating the rock will encounter hard to very hard materials and may be difficult to remove in narrow trenches or footing excavations**. **Excavation costs should anticipate hard rock excavation for preliminary planning and construction budget.** Our boring log is not intended for use in determining construction means and methods and may therefore be misleading if used for that purpose. We recommend that earthwork 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.

## UTILITIES

Utilities which project through any 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.
- 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.

## 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, **RKI** is retained to perform construction observation and testing services during the construction of the project. This is because:

- **RKI** has an intimate understanding of the geotechnical engineering report's findings and recommendations. **RKI** understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- **RKI** knows what subsurface conditions are anticipated at the site.
- **RKI** 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 **RKI** to suggest remedial measures (when needed) which help meet the owner's and the design teams' requirements.

- **RKI** 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.
- **RKI** 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.

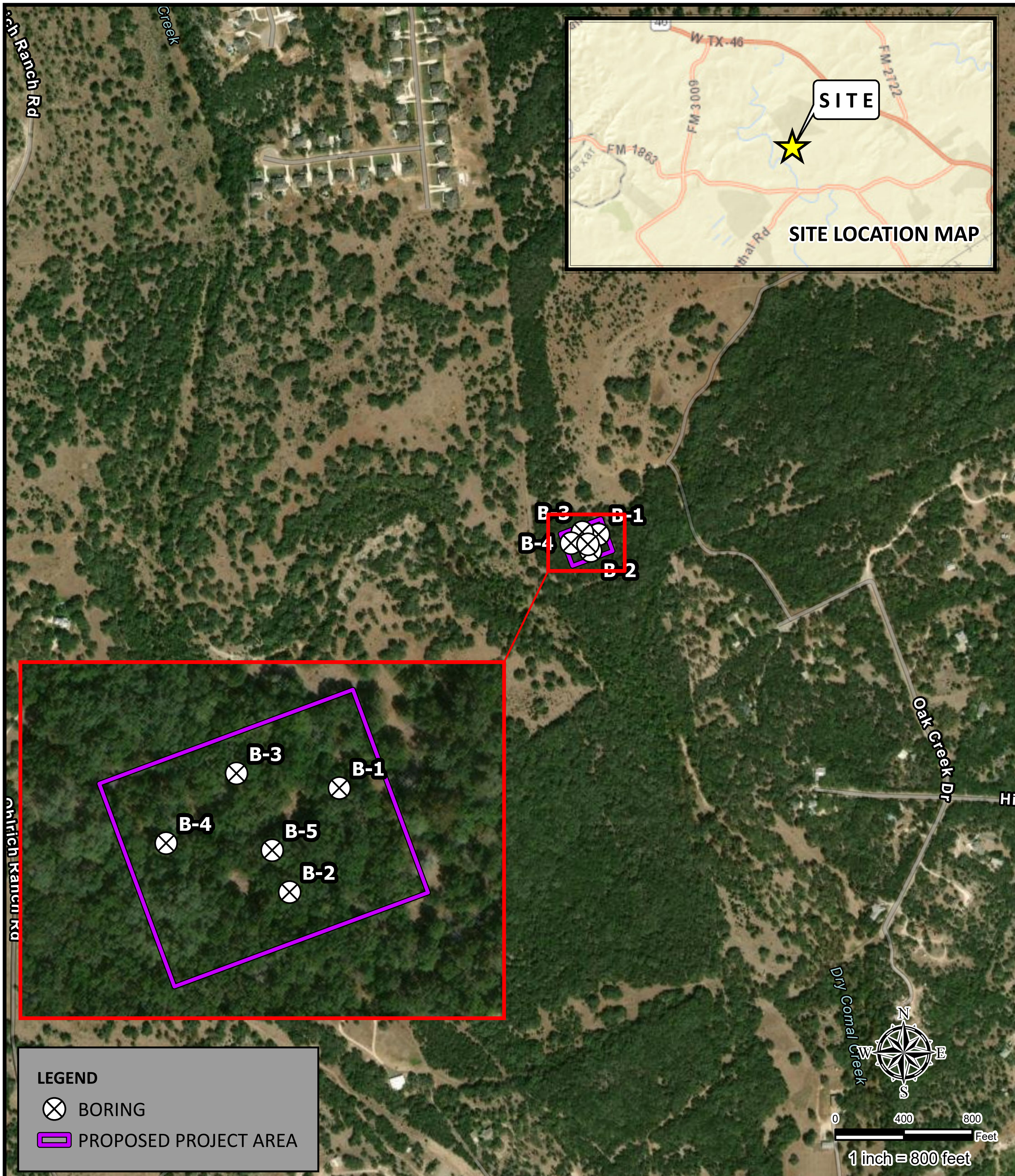
## BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that **RKI** 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. **RKI** 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 the scope and budget for these services.

* * * * * * * * * * * * * * * * * *

# ATTACHMENTS

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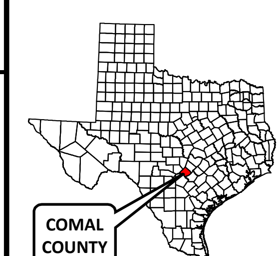
TBPE Firm Number 3257

Hybrid Reference Layer: Esri Community Maps Contributors, City of New Braunfels, BCAD, Comal County, Texas Parks & Wildlife, © OpenStreetMap, Microsoft, CONANP, Esri, TomTom, Garmin, Foursquare, SafeGraph, GeoTechnologies, Inc, METI/NASA, USGS, EPA, NPS, US Census Bureau, USDA, USFWS
World Street Map: Esri, HERE, Garmin, NGA, USGS, NPS

## BORING LOCATION MAP

Lift Station and WWTF
NB West
New Braunfels, Texas




| PROJECT No.: | ANA24-027-00 |
|---|---|
| ISSUE DATE: | 8/8/2024 |
| DRAWN BY: | BM |
| CHECKED BY: | SS |
| REVIEWED BY: | TIP |

**FIGURE**

**1**

NOTE: This Drawing is Provided for Illustration Only, May Not be to Scale and is Not Suitable for Design or Construction Purposes

# LOG OF BORING NO. B-1

Lift Station and Waste Water Treatment Facility
NB West
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

**DRILLING METHOD:** Straight Flight Auger & Air Rotary **LOCATION:** N 29.72318; W 98.25983

| DEPTH, FT | DESCRIPTION OF MATERIAL | BLOWS PER FT | UNIT DRY WEIGHT, pcf | PLASTICITY INDEX | % -200 |
|---|---|---|---|---|---|
| 0–2 | FAT CLAY, Hard, Dark Brown, with limestone fragments | 35 | | 36 | |
| 2 | LIMESTONE, Hard, Tan and Gray | ref/1" | | | |
| 4 | | ref/1" | | | |
| 6 | | ref/2" | | | |
| 8.5 | | ref/1" | | | |
| 13.5 | | ref/1" | | | |
| 16–18 | - highly weathered from 16 to 18 ft with clay seams | | | | |
| 18.5 | | ref/2" | | | |
| 23.5 | | ref/1" | | | |
| 28.5 | | ref/1" | | | |
| 33.5 | | ref/2" | | | |
| 38.5 | | ref/1" | | | |
| 38.6 | Boring Terminated | | | | |

SHEAR STRENGTH, TONS/FT$^2$: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

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

| | | | | | |
|---|---|---|---|---|---|
| **DEPTH DRILLED:** | 38.6 ft | **DEPTH TO WATER:** | Dry | **PROJ. No.:** | ANA24-027-00 |
| **DATE DRILLED:** | 9/6/2024 | **DATE MEASURED:** | 9/6/2024 | **FIGURE:** | 2 |

# LOG OF BORING NO. B-2

Lift Station and Waste Water Treatment Facility
NB West
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

**DRILLING METHOD:** Air Rotary

**LOCATION:** N 29.72288; W 98.26000

SHEAR STRENGTH, TONS/FT$^2$: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

| DEPTH, FT | DESCRIPTION OF MATERIAL | BLOWS PER FT | UNIT DRY WEIGHT, pcf | PLASTICITY INDEX | % -200 |
|---|---|---|---|---|---|
| 0–2.5 | FAT CLAY, Hard, Dark Brown, with limestone fragments | 48 | | 32 | |
| 2.5–38.6 | LIMESTONE, Hard, Tan and Gray, with weathered seams | ref/3" | | | |
| | | ref/1" | | 10 | |
| | | ref/1" | | | |
| | | ref/1" | | | |
| | | ref/1" | | | |
| | | ref/1" | | | |
| | | ref/2" | | | |
| | | ref/1" | | | |
| | | ref/2" | | | |
| 38.6 | | ref/1" | | | |
| | Boring Terminated | | | | |

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

**DEPTH DRILLED:** 38.6 ft
**DATE DRILLED:** 9/6/2024
**DEPTH TO WATER:** Dry
**DATE MEASURED:** 9/6/2024
**PROJ. No.:** ANA24-027-00
**FIGURE:** 3

# LOG OF BORING NO. B-3

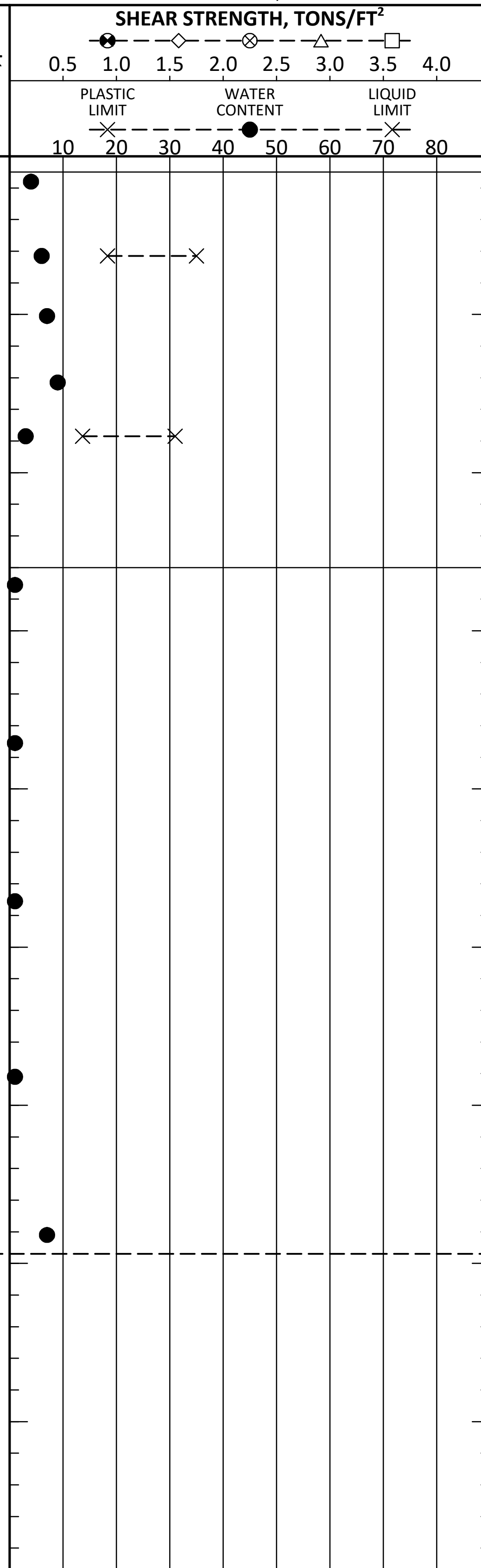
Lift Station and Waste Water Treatment Facility
NB West
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

**DRILLING METHOD:** Air Rotary

**LOCATION:** N 29.72312; W 98.26010

| DEPTH, FT | DESCRIPTION OF MATERIAL | BLOWS PER FT | UNIT DRY WEIGHT, pcf | PLASTICITY INDEX | % -200 |
|---|---|---|---|---|---|
| 0 | FAT CLAY, Hard, Dark Brown, with limestone fragments | 50/8" | | | |
| | MARL, Hard, Tan, with limestone fragments | | | | |
| | | 50/10" | | 17 | |
| 5 | | 50/7" | | | |
| | | 50/10" | | | |
| | | 50/2" | | 17 | |
| 10 | | | | | |
| | LIMESTONE, Hard, Tan and Gray, with weathered seams | ref/2" | | | |
| 15 | | | | | |
| | | ref/2" | | | |
| 20 | | | | | |
| | | ref/2" | | | |
| 25 | | | | | |
| | - with reddish brown clay from 27 to 35 | | | | |
| | | 50/8" | | | |
| 30 | | | | | |
| | | 50/8" | | | |
| 35 | Boring Terminated | | | | |
| 40 | | | | | |




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

| | | | | | |
|---|---|---|---|---|---|
| **DEPTH DRILLED:** | 34.7 ft | **DEPTH TO WATER:** | Dry | **PROJ. No.:** | ANA24-027-00 |
| **DATE DRILLED:** | 9/10/2024 | **DATE MEASURED:** | 9/10/2024 | **FIGURE:** | 4 |

# LOG OF BORING NO. B-4

Lift Station and Waste Water Treatment Facility
NB West
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

**DRILLING METHOD:** Air Rotary

**LOCATION:** N 29.72299; W 98.26036

SHEAR STRENGTH, TONS/FT$^2$: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10, 20, 30, 40, 50, 60, 70, 80

| DEPTH, FT | DESCRIPTION OF MATERIAL | BLOWS PER FT | UNIT DRY WEIGHT, pcf | PLASTICITY INDEX | % -200 |
|---|---|---|---|---|---|
| 0 | FAT CLAY, Hard, Dark Brown, with limestone fragments | 30 | | 51 | |
| | | 50/9" | | | |
| 4 | MARL, Hard, Tan, with clay seams limestone fragments | 50/2" | | | |
| | | ref/4" | | | |
| | | ref/2" | | 6 | |
| 13 | LIMESTONE, Hard, Tan and Gray, with weathered seams | ref/1" | | | |
| | | ref/2" | | | |
| | | ref/1" | | | |
| | | ref/2" | | | |
| | | ref/2" | | | |
| | | ref/2" | | | |
| 38.7 | Boring Terminated | | | | |

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

**DEPTH DRILLED:** 38.7 ft
**DATE DRILLED:** 9/10/2024
**DEPTH TO WATER:** Dry
**DATE MEASURED:** 9/10/2024
**PROJ. No.:** ANA24-027-00
**FIGURE:** 5

# LOG OF BORING NO. B-5

Lift Station and Waste Water Treatment Facility
NB West
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

**DRILLING METHOD:** Air Rotary

**LOCATION:** N 29.72296; W 98.26009

SHEAR STRENGTH, TONS/FT$^2$: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

PLASTIC LIMIT — WATER CONTENT — LIQUID LIMIT: 10 20 30 40 50 60 70 80

| DEPTH, FT | SYMBOL | SAMPLES | DESCRIPTION OF MATERIAL | BLOWS PER FT | UNIT DRY WEIGHT, pcf | PLASTICITY INDEX | % -200 |
|---|---|---|---|---|---|---|---|
| | | | FAT CLAY, Hard, Dark Brown, with limestone fragments | 39 | | 39 | |
| | | | | ref/4" | | | |
| | | | LIMESTONE, Hard, Tan and Gray, with weathered seams | | | | |
| 5 | | | | ref/2" | | | |
| | | | | ref/1" | | | |
| | | | | ref/1" | | | |
| 10 | | | | | | | |
| | | | | ref/2" | | | |
| 15 | | | | | | | |
| | | | | ref/1" | | | |
| 20 | | | | | | | |
| | | | | ref/1" | | | |
| 25 | | | | | | | |
| | | | - with reddish brown clay seams from 27 to 30 | | | | |
| | | | | ref/4" | | | |
| 30 | | | | | | | |
| | | | | ref/2" | | | |
| 35 | | | | | | | |
| | | | | ref/1" | | | |
| | | | Boring Terminated | | | | |
| 40 | | | | | | | |

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

**DEPTH DRILLED:** 38.6 ft
**DATE DRILLED:** 9/6/2024
**DEPTH TO WATER:** Dry
**DATE MEASURED:** 9/6/2024
**PROJ. No.:** ANA24-027-00
**FIGURE:** 6

# KEY TO TERMS AND SYMBOLS

## MATERIAL TYPES

### SOIL TERMS

- CALCAREOUS
- CALICHE
- CLAY
- CLAYEY
- GRAVEL
- GRAVELLY
- PEAT
- SAND
- SANDY
- SILT
- SILTY
- FILL

### ROCK TERMS

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

### OTHER

- ASPHALT
- BASE
- CONCRETE/CEMENT
- BRICKS / PAVERS
- WASTE
- NO INFORMATION

## WELL CONSTRUCTION AND PLUGGING MATERIALS

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

## SAMPLE TYPES

- AIR ROTARY
- GRAB SAMPLE
- CORE
- GEOPROBE SAMPLER
- ROTOSONIC -DAMAGED
- MUD ROTARY
- NO RECOVERY
- NX CORE
- PITCHER
- ROTOSONIC -INTACT
- SHELBY TUBE
- SPLIT BARREL
- SPLIT SPOON
- TEXAS CONE PENETROMETER
- DISTURBED

## 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. ANA24-027-00

# 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.

### RELATIVE DENSITY

| Penetration Resistance Blows per ft | Relative Density |
|---|---|
| 0 - 4 | Very Loose |
| 4 - 10 | Loose |
| 10 - 30 | Medium Dense |
| 30 - 50 | Dense |
| > 50 | Very Dense |

### COHESIVE STRENGTH

| Resistance Blows per ft | Consistency | Cohesion TSF |
|---|---|---|
| 0 - 2 | Very Soft | 0 - 0.125 |
| 2 - 4 | Soft | 0.125 - 0.25 |
| 4 - 8 | Firm | 0.25 - 0.5 |
| 8 - 15 | Stiff | 0.5 - 1.0 |
| 15 - 30 | Very Stiff | 1.0 - 2.0 |
| > 30 | Hard | > 2.0 |

### PLASTICITY

| Plasticity Index | Degree of Plasticity |
|---|---|
| 0 - 5 | None |
| 5 - 10 | Low |
| 10 - 20 | Moderate |
| 20 - 40 | Plastic |
| > 40 | Highly Plastic |

## ABBREVIATIONS

B = Benzene
T = Toluene
E = Ethylbenzene
X = Total Xylenes
BTEX = Total BTEX
TPH = Total Petroleum Hydrocarbons
ND = Not Detected
NA = Not Analyzed
NR = Not Recorded/No Recovery
OVA = Organic Vapor Analyzer
ppm = Parts Per Million

Qam, Qas, Qal = Quaternary Alluvium
Qat = Low Terrace Deposits
Qbc = Beaumont Formation
Qt = Fluviatile Terrace Deposits
Qao = Seymour Formation
Qle = Leona Formation
Q-Tu = Uvalde Gravel
Ewi = Wilcox Formation
Emi = Midway Group
Mc = Catahoula Formation
El = Laredo Formation
Kknm = Navarro Group and Marlbrook Marl
Kpg = Pecan Gap Chalk
Kau = Austin Chalk

Kef = Eagle Ford Shale
Kbu = Buda Limestone
Kdr = Del Rio Clay
Kft = Fort Terrett Member
Kgt = Georgetown Formation
Kep = Person Formation
Kek = Kainer Formation
Kes = Escondido Formation
Kew = Walnut Formation
Kgr = Glen Rose Formation
Kgru = Upper Glen Rose Formation
Kgrl = Lower Glen Rose Formation
Kh = Hensell Sand

# 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

| Blows Per Foot | Description |
|---|---|
| 25 | 25 blows drove sampler 12 inches, after initial 6 inches of seating. |
| 50/7" | 50 blows drove sampler 7 inches, after initial 6 inches of seating. |
| Ref/3" | 50 blows drove sampler 3 inches during initial 6-inch seating interval. |

<u>NOTE:</u> To avoid damage to sampling tools, driving is limited to 50 blows during or after seating interval.

# RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Lift Station and Waste Water Treatment Facility
NB West
New Braunfels, Texas

FILE NAME: ANA24-027-00 GINT.GPJ 10/14/2024

| Boring No. | Sample Depth (ft) | Blows per ft | Water Content (%) | Liquid Limit | Plastic Limit | Plasticity Index | USCS | Dry Unit Weight (pcf) | % -200 Sieve | Shear Strength (tsf) | Strength Test |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B-1 | 0.0 to 1.5 | 35 | 9 | 51 | 15 | 36 | CH | | | | |
| | 2.5 to 2.6 | ref/1" | 2 | | | | | | | | |
| | 4.5 to 4.6 | ref/1" | 1 | | | | | | | | |
| | 6.5 to 6.7 | ref/2" | 1 | | | | | | | | |
| | 8.5 to 8.6 | ref/1" | 0 | | | | | | | | |
| | 13.5 to 13.6 | ref/1" | 0 | | | | | | | | |
| | 18.5 to 18.7 | ref/2" | 1 | | | | | | | | |
| | 23.5 to 23.6 | ref/1" | 1 | | | | | | | | |
| | 28.5 to 28.6 | ref/1" | 1 | | | | | | | | |
| | 33.5 to 33.7 | ref/2" | 1 | | | | | | | | |
| | 38.5 to 38.6 | ref/1" | 2 | | | | | | | | |
| B-2 | 0.0 to 1.5 | 48 | 12 | 48 | 16 | 32 | CH | | | | |
| | 2.5 to 2.8 | ref/3" | 1 | | | | | | | | |
| | 4.5 to 4.6 | ref/1" | 1 | 23 | 13 | 10 | CL | | | | |
| | 6.5 to 6.6 | ref/1" | 1 | | | | | | | | |
| | 8.5 to 8.6 | ref/1" | 1 | | | | | | | | |
| | 13.5 to 13.6 | ref/1" | 1 | | | | | | | | |
| | 18.5 to 18.6 | ref/1" | 2 | | | | | | | | |
| | 23.5 to 23.7 | ref/2" | 1 | | | | | | | | |
| | 28.5 to 28.6 | ref/1" | 0 | | | | | | | | |
| | 33.5 to 33.7 | ref/2" | 1 | | | | | | | | |
| | 38.5 to 38.6 | ref/1" | 1 | | | | | | | | |
| B-3 | 0.0 to 1.1 | 50/8" | | | | | | | | | |
| | 0.5 to 1.1 | | 4 | | | | | | | | |
| | 2.5 to 3.8 | 50/10" | 6 | 35 | 18 | 17 | CL | | | | |
| | 4.5 to 5.6 | 50/7" | 7 | | | | | | | | |
| | 6.5 to 7.8 | 50/10" | 9 | | | | | | | | |
| | 8.5 to 9.2 | 50/2" | 3 | 31 | 14 | 17 | CL | | | | |
| | 13.5 to 13.6 | ref/2" | 1 | | | | | | | | |
| | 18.5 to 18.6 | ref/2" | 1 | | | | | | | | |
| | 23.5 to 23.6 | ref/2" | 1 | | | | | | | | |
| | 28.5 to 29.7 | 50/8" | 1 | | | | | | | | |
| | 33.5 to 34.7 | 50/8" | 7 | | | | | | | | |
| B-4 | 0.0 to 1.5 | 30 | 11 | 72 | 21 | 51 | CL | | | | |
| | 2.5 to 3.7 | 50/9" | 13 | | | | | | | | |
| | 4.5 to 5.2 | 50/2" | 4 | | | | | | | | |
| | 6.5 to 6.8 | ref/4" | 3 | | | | | | | | |
| | 8.5 to 8.7 | ref/2" | 2 | 21 | 15 | 6 | CL | | | | |
| | 13.5 to 13.6 | ref/1" | 2 | | | | | | | | |

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

CU = Consolidated Undrained Triaxial

# RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Lift Station and Waste Water Treatment Facility
NB West
New Braunfels, Texas

FILE NAME: ANA24-027-00 GINT.GPJ 10/14/2024

| Boring No. | Sample Depth (ft) | Blows per ft | Water Content (%) | Liquid Limit | Plastic Limit | Plasticity Index | USCS | Dry Unit Weight (pcf) | % -200 Sieve | Shear Strength (tsf) | Strength Test |
|---|---|---|---|---|---|---|---|---|---|---|---|
| B-4 | 18.5 to 18.7 | ref/2" | 1 | | | | | | | | |
| | 23.5 to 23.6 | ref/1" | 0 | | | | | | | | |
| | 28.5 to 28.7 | ref/2" | 0 | | | | | | | | |
| | 33.5 to 33.7 | ref/2" | 0 | | | | | | | | |
| | 38.5 to 38.7 | ref/2" | 1 | | | | | | | | |
| B-5 | 0.0 to 1.5 | 39 | 8 | 56 | 17 | 39 | CH | | | | |
| | 2.5 to 2.8 | ref/4" | 17 | | | | | | | | |
| | 4.5 to 4.7 | ref/2" | 1 | | | | | | | | |
| | 6.5 to 6.6 | ref/1" | 0 | | | | | | | | |
| | 8.5 to 8.6 | ref/1" | 0 | | | | | | | | |
| | 13.5 to 13.7 | ref/2" | 0 | | | | | | | | |
| | 18.5 to 18.6 | ref/1" | 1 | | | | | | | | |
| | 23.5 to 23.6 | ref/1" | 1 | | | | | | | | |
| | 28.5 to 28.8 | ref/4" | 2 | | | | | | | | |
| | 33.5 to 33.7 | ref/2" | 1 | | | | | | | | |
| | 38.5 to 38.6 | ref/1" | 0 | | | | | | | | |

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

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# Important Information about This

# Geotechnical-Engineering Report

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

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

## 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 Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL
*of the Geoprofessional Business Association*

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