RABA KISTNER

GEOTECHNICAL ENGINEERING STUDY

FOR

WORD PARKWAY – PHASE 3
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS

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Project No. ANA22-022-00
March 1, 2024

Mr. Garrett Mechler, P.E.
ASA Properties
2168 Oak Run Parkway, Suite 101
New Braunfels, Texas 78131

RE: Geotechnical Engineering Study
Word Parkway – Phase 3
Veramendi Master Planned Development
New Braunfels, Texas

Dear Mr. Mechler:

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. PNA22-034-00, dated March 31, 2022. The purpose of this study was to drill borings within the proposed pavement and bridge improvement areas, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design recommendations for the proposed bridge.

The following report contains our design recommendations and considerations based on our current understanding of the project information provided to us. 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.

Ryan DeWees

Ian Perez

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Ryan DeWees
Graduate Engineer

Ian Perez, P.E.
Vice President

RD/TIP/mmd

Attachments

GEOTECHNICAL ENGINEERING STUDY

For

WORD PARKWAY – PHASE 3
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS

Prepared for

ASA PROPERTIES
New Braunfels, Texas

Prepared by

RABA KISTNER, INC.
New Braunfels, Texas

PROJECT NO. ANA22-022-00

March 1, 2024

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INTRODUCTION

RABA KISTNER, Inc. (RKI) has completed the authorized subsurface exploration for the proposed new bridge and pavements for the proposed Word Parkway extension located in New Braunfels, 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

To be considered in this study is the proposed approximately 2,000 LF extension of Word Parkway (Phase 3) to its future connection with Hill Country Road (Phase 3) in the Veramendi Master Planned Development in New Braunfels, Texas. Originally, the project scope included a 600 LF, cast-in-place bridge. We understand that the bridge was moved to a new location, reduced to 100 LF, and is proposed to be a precast structure. A bridge design plan set for the 100 LF bridge, titled “WORD PARKWAY PHASE 3 BRIDGE” dated November 9, 2023 was provided via email by Mr. Gregory Latimer, P.E. of Pape-Dawson Engineers on December 18, 2023. The proposed roadways are planned to be designed utilizing guidance from the City of San Antonio’s (CoSA) Pavement Design Guidance Manual and the City of New Braunfels. We anticipate that the roadway will be designed in accordance with a CoSA Arterial street classification.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of South Texas and for the use of ASA Properties (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 recommendations submitted in this report are based on the data obtained from 7 borings drilled at this site, our understanding of the project information provided to us, and the bridge documents mentioned in the *Project Description* section of this report. 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.

BORINGS AND LABORATORY TESTING

As previously discussed, subsurface conditions at the site were evaluated by 7 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 approximate depths ranging from 5 to 40 ft below the existing ground surface using a truck-mounted drilling rig. During drilling operations Texas Cone Penetrometer (TCP) testing, split-spoon samples with Standard Penetration Tests (SPT), and auger grab samples were collected.

Each sample was visually classified by a member of our geotechnical engineering staff. The geotechnical engineering properties of the strata encountered in our borings were evaluated by natural moisture content, sieve analysis (percent passing a No. 200 sieve), unconfined compression testing, and Atterberg limits tests.

The laboratory test results are presented in graphical or numerical form on the boring logs illustrated in Figures 2 through 8. A key to classification terms and symbols used on the logs is presented in Figure 9. The results of the laboratory and field testing are also tabulated in Figure 10 for ease of reference.

Texas Cone Penetrometer (TCP) test results are noted as “blows per foot” on the boring logs (divided into 6 in. increments) where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft of penetration into soil/weak rock. Where hard or dense materials were encountered, each increment was terminated at 50 blows even if 6 in. of penetration had not been achieved in that increment.

SPT results (N-values) are noted as “blows per ft” on the boring logs and on Figure 10. 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 10.

In addition to the above listed testing and sampling, a bulk sample of the predominant subgrade soil from along the project alignment was also collected for use in California Bearing Ratio (CBR) testing, pH-Lime series testing, and sulfate content testing. The results of the CBR testing can be found on the Moisture Density Relationship Curve on Figure 11. The graph for Dry Density vs. Corrected CBR is presented in Figure 12. The pH-Lime Series Curve can be found in Figure 13.

A summary of the bulk sample testing results are presented in the following table:

Material Type, Location and Depth	Max Dry Density and Optimum Moisture	Corrected Laboratory CBR	Average Percent Swell (%)	Raw Plasticity Index (PI)	PI with 4 Percent Lime
Reddish Dark Brown Clay (Boring P-5, 0 - 1 ft)	95.2 pcf and 22.2%	3.7	1.3	37	11

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

SULFATE TESTING

Sulfate testing was performed on the bulk sample collected from an area near P-5. The results of the sulfate content testing is presented in the table below.

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. Sulfates can also affect the durability of concrete when encountered in high concentrations.

Soil Type	Boring Number	Approximate Depth Below Existing Ground Surface (ft)	Sulfate Content (ppm)
Reddish Dark Brown Clay	P-5	0 - 1	<100

Based on the laboratory test results, the reported sulfate concentration values were generally determined to be negligible. Reported sulfate concentrations above 3,000 ppm are known to cause sulfate induced heaving when the soils are mixed with lime. If the option for lime is considered, a quality assurance program should be implemented to assist in reducing the risk of sulfate induced heaving.

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): **S_s = 0.051g**.

- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% Of Critical Damping): **S_1 = 0.027g**.
- Values of Site Coefficient: **F_a = 1.3**
- Values of Site Coefficient: **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: **S_{ms} = 0.066g**
- 1 sec, adjusted: **S_{m1} = 0.041g**

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

- 0.2 sec SA: **S_{DS} = 0.044g**
- 1 sec SA: **S_{D1} = 0.027g**

STRATIGRAPHY

The natural subsurface stratigraphy can generally be described as a thin veneer of dark brown to reddish brown clay underlain by limestone which was encountered within the upper 1 to 3 ft of our borings and which extends to at least the boring termination depths. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. 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. However, it is possible for groundwater to exist beneath this site at shallow depths on a transient basis following periods of precipitation or at the clay and limestone interface. 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.

BRIDGE FOUNDATION RECOMMENDATIONS

Pier recommendations for the originally proposed 600 LF bridge are based on information obtained from Borings P-1 through P-4 (Figure 1A) using guidance from the Texas Department of Transportation. Due to the distance of Borings P-1 through P-4 to the proposed new bridge location additional field and laboratory testing were completed to support recommendations for the new bridge type and location. Borings P-6 and P-7 (Figure 1B) were drilled in the vicinity of the relocated bridge footprint to substantiate the condition of the bearing soils along the new alignment.

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 to support the structures:

- Drilled, straight-shaft piers; and/or
- Shallow Foundations.

The owner may select either one of these foundation systems depending on the performance criteria established for the structures. Cost analyses have not been conducted for any foundation system and are beyond the scope of this study.

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 on the order of 1 in. or less were estimated for the stratigraphic conditions encountered in our borings. A surcharge load of 1 psi, 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 (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.

Overexcavation and Select Fill Replacement

To maintain expansive soil-related movements in at-grade construction to on the order of 1 in. or less, we recommend that all of the dark brown clays be removed from within proposed foundation areas. Recommendations for the selection and placement of select backfill materials are addressed in a subsequent section of this report.

If the proposed site grading allows, we recommend that the bridge footings bear directly on bedrock. We estimate a PVR on the order of 1 in. or less if the bridge footings bear directly on the bedrock.

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.

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 perimeter of the proposed foundations. The dewatering trench should be dilated by gravity or routed to a sump pump;
- 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 (minimum 2 ft) or pavement surface (downward away from the foundations) over the select fill material, with a minimum gradient of 6 in. in 5 ft; and
- Constructing final surface drainage patterns to prevent ponding and limit surface water infiltration at and around the foundation perimeter.

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.

600 AND 100 LF BRIDGE DEEP FOUNDATIONS

Axial Capacity

For the bridge abutments, we have computed allowable downward vertical capacities for 24, 30, 36, and 42 in. diameter drilled, straight-shaft piers. Straight-shaft piers may be designed as end-bearing and friction units using the capacities presented graphically on the “Drilled Pier Axial Capacity Curves” in Figure 14. Portions of the piers that are embedded in fill materials should also be neglected in addition to the 10 ft portrayed in the curves.

The pier capacity curves were developed using the TxDOT Geotechnical Manual dated July 2020. The capacity curves are based on the recorded TCP blow count data. The indicated capacities on these figures are for dead plus live loads. Dead loads should not exceed two-thirds of the computed capacities.

Groundwater seepage may 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 to reduce the potential for groundwater to enter the pier excavation and to prevent sloughing of the pier sidewalls.

We recommend that the slump of the pier concrete be no less than 6 in. and no more than one inch of water in the bottom of the pier excavation before concrete is placed.

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:

$$F_u = 10*D$$

where:

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

Uplift Resistance

Resistance to uplift forces exerted on the drilled, straight-shaft piers (as presented in the following section) will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the underlying subsurface material. Hence, the remaining uplift resistance will be provided by the underlying material below the active zone (15 ft below the existing surface or to the top of bedrock (i.e. marl or limestone)). The resistance provided by the underlying material depends on the shear strength of the material adjacent to the pier shaft and below the depth of the active zone. The allowable uplift resistance provided by the materials at this site may be estimated using the “Drilled Pier Uplift Capacity Curves” presented graphically in Figure 15. Friction resistance was neglected to the elevations presented on the uplift capacity curves.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the uplift resistive force and 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	Depth (ft)	c (psf)	k_s (pci)	ε_{50}	γ (pcf)	γ' (pcf)	q_u (psi)
Soft Clay (Matlock)	0 to 5	500	30	0.020	120	58	-
Strong Rock (Vuggy Limestone)	5 to 40	-	-	-	-	83	1,000

Where:

c = undrained cohesion
k_s = p-y modulus
ε_{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.

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

100 LF BRIDGE SHALLOW FOUNDATION RECOMMENDATIONS

It is our understanding that a shallow foundation is being considered for the proposed 100 LF bridge. The proposed bridge may be founded on shallow foundations, 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 structure.

Net Allowable Bearing Capacity

Shallow foundations bearing on limestone may be proportioned using the design parameters presented in the following table:

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

Shallow Foundation Type	Maximum Allowable Bearing Pressure on Limestone	
	Without pilot holes	With pilot holes
Bridge footings	7,500 psf	9,000 psf

We recommend that the surficial expansive dark brown clay be removed and the bridge footings bear on bedrock (i.e. intact or weathered limestone). We do not recommend that the bridge footings be founded partially in bedrock and partially in natural soils or compacted fill as this condition may result in greater differential movements. If not practical to extend to bedrock, foundations bearing on a minimum of 1 ft of crushed limestone select fill placed and compacted beneath the bridge footings may be considered.

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 *Overexcavation and Select Fill Replacement* and *Site Preparation* sections of this report.

If the higher bearing capacity presented in the table above is needed, we recommend probe borings with rock coring or pilot holes at actual foundation locations. We recommend extending the pilot holes to a depth of at least 2 times the width of the footing and a frequency of at least 1 hole per every 25 sf of footing area. The purpose of the pilot holes is to explore the condition of the limestone below the individual footings and to explore the presence or absence of karst or voids immediately below the footprint of the individual footings.

The foundation subgrade should be observed by the Geotechnical Engineer 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. In addition, probe or pilot holes may be required to assess presence of voids or clay layers below the shallow foundations. If voids or karstic features are encountered any necessary environmental protocols should be followed, and the foundation design changes should be re-evaluated on a case-by-case basis.

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 125 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 spread footings will be resisted by passive earth pressure acting on one side of the footing and by base adhesion for footings in cohesive soils. Resistance to sliding for foundations bearing on bedrock (i.e intact or weathered limestone) can be calculated utilizing an ultimate coefficient of friction of 0.70. The resistance for these foundations should be limited to 4,900 psf. An equivalent fluid pressure of 250 pcf can be utilized to determine the ultimate passive resistance, if required.

AREA FLATWORK

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

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 flatwork are underlain by at least 2 feet of compacted aggregate select fill or bedrock (i.e. weathered or intact limestone).

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 grade supported features.

Where a select fill overbuild is provided outside of the bridge foundation, if any, the surface should be sealed with an impermeable layer (i.e. clay cap, geomembrane or pavement) 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 areas to support select fill should be stripped of all vegetation, root mass, organic topsoil, pavement section, utilities, structures, and associated backfill. Exposed subgrades should be thoroughly proofrolled in order to locate weak, compressible zones. 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 select fill.

Upon completion of the proofrolling operations and just prior to fill placement or foundation construction, the exposed clay subgrades 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 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.

ONSITE SOIL

The use of onsite expansive soils may be a considered for general fill (outside of the bridge foundation area), 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.

SELECT FILL

Recommendations for imported granular select fill materials are provided below.

> **Imported Crushed Limestone Base** – Imported crushed limestone base materials should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Type A, B, or C, Grades 1-2 or 3.

Recommendations for alternative select fill materials are provided below.

> **Granular Pit Run Materials** – Granular pit run materials should consist of GC, SC & combination soils (clayey gravels), as classified according to the Unified Soil Classification System (USCS). 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 inch. 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 materials or alternative select fills are being considered for bidding purposes, the materials should be submitted to the Geotechnical Engineer for evaluation at a minimum of 10 working days or more prior *to* fill placement. 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 such as Granular Pit Run, 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.

Granular Pit Run will be very susceptible to small changes in moisture content and to disturbance from foot traffic during the placement of steel reinforcement in beam trenches, particularly in periods of inclement weather. Disturbance from such foot traffic and from the accumulation of excess water can result in losses in bearing capacity and increased settlement. If inclement weather is anticipated at the time construction, consideration should be given to protecting the bottom of foundation excavations by placing a thin mud mat (layer of flowable fill or lean concrete) at the bottom of trenches immediately following excavation. This will reduce disturbance from foot traffic and will impede the infiltration of surface water. The side slopes of beam trench excavations may also need to be flattened to reduce sloughing in cohesionless soils. All necessary precautions should be implemented to protect open excavations from the accumulation of surface water runoff and rain.

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. 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.

General Fill Placement and Compaction

General fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to a minimum of 95 percent of the maximum density determined from the TxDOT, Tex-114-E 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.

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.

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, 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 a compacted non-expansive fill material or lean concrete up to the design foundation bearing elevations.

DRILLED PIERS

Each drilled pier excavation must be observed by a Raba Kistner 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;
- 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.

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. As a minimum, the concrete should be allowed at least 24 hours or more to cure prior to constructing adjacent foundation elements.

Temporary Casing

While not anticipated, groundwater seepage and/or side sloughing may 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 EQUIPMENT

Please note that hard bedrock (i.e. marl or limestone) was encountered in our borings, therefore, pier excavations at this site will require removal of the underlying rock formation. Consequently, high torque, high download drilling equipment may be required to successfully construct the pier excavations. Additionally, difficulties may be experienced in narrow trenches or footing excavations where penetration into the underlying rock formation occurs. Excavation costs should anticipate hard rock excavation for preliminary planning and construction budget. 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.

UTILITIES

Utilities which project through shallow foundations, if any, 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. 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.
- 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.

CITY OF NEW BRAUNFELS PAVEMENT DESIGN

The City of New Braunfels has adopted minimum pavement sections for streets classified as a Residential Local Street or a Residential Collector. If the street classifications change to one of these classifications the following minimum pavement sections should be used. Furthermore, RKI should be retained to evaluate our recommendations.

City of New Braunfels Street Classification	Layer Description	Layer Thickness
One and Two Family Residential Local Parking Both Sides	HMA Type D Surface Course Flexible (Granular) Base Treated Subgrade **Combined Total**	2.0 in. 12.0 in. 6.0 in. **20.0 in.**
Residential Collector Parking Both Sides	HMA Type D Surface Course Flexible (Granular) Base Treated Subgrade **Combined Total**	3.0 in. 15.0 in. 6.0 in. **24.0 in.**

RECOMMENDED PAVEMENT DESIGN

Design Parameters – Flexible Pavements

The proposed roadway pavement sections were evaluated using guidance from the City of San Antonio's Design Guidance Manual. Typical traffic loading scenarios were utilized based on the anticipated use at this site. We have included Arterial street classification recommendations for consideration by the design team. Based on information provided by the City of San Antonio, we understand that the following design parameters are required for use in the design of flexible pavements for these types of streets.

Street Classification	Equivalent 18-kip Single Axle Load Applications (ESALs)	Reliability	Serviceability Initial/Terminal	Standard Deviation	Structural Number Minimum/Maximum
Arterial	3,000,000	95	4.2/2.5	0.45	3.80/5.76

The required structural number is related to the CBR value of the pavement subgrade and the amount of traffic that the pavement will carry over its service life. The CBR provides an estimate of the relative strength of the subgrade and consequently indicates the ability of the pavement section to carry load. This site specific CBR value is utilized in conjunction with the above specified parameters to determine the required Structural Number (SN) for use in the design of the pavement section.

To determine the required design SN value, we utilized a method based on the 1993 edition of the AASHTO "Guide for the Design of Pavement Structures." The "required by design" SN values are presented in the tables of the pavement sections as well as the values subsequently determined in the design of the pavement sections for this site.

Subgrade Strength Characterization

We have assumed the pavement subgrade will consist of either recompacted on-site clays or native limestone. The CBR value for the on-site clays (referred to as 'clay subgrade' hereafter) was determined using ASTM D 1883, *Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils* with the soaked sample methodology. If native limestone or select fill (including rock millings) are utilized as the subgrade (referred to as 'rock subgrade' hereafter), then a Design CBR of 10.0 may be used. Corrected CBR and percent swell for the collected bulk sample from the site are tabulated below.

Sample Location	Average Swell (%)	Corrected Laboratory CBR	Design CBR
P-5	1.3	3.7	3.5

This value was determined using 3-points compacted at varying efforts to determine the corrected CBR value at 95 percent of the maximum dry density as determined by TxDOT, Tex-114-E. The moisture-density relationship is presented in Figure 11. If clay soils are imported for the purpose of constructing the roadbed then imported materials must be selected that have a CBR value of at least the Design CBR value presented in the table above. The Design CBR value was assigned based on the laboratory CBR results and our experience with similar soils. If lower quality clay fill materials are utilized, the pavement sections will have to be increased based on the quality (tested CBR value) of the clays imported.

Structural Number Recommendations

Structural number for the street classification and subgrade condition were calculated using the parameters provided in the table presented in the previous section. The resulting Structural Numbers are presented in the pavement section tables.

The following input variables are utilized to design flexible base pavements (commonly referred to as Asphaltic Cement Concrete or Asphalt pavements) when using the procedures detailed in the *1993 AASHTO Guide for Design of Pavement Structures*:

- Performance Period, years
- Roadbed Soil Resilient Modulus, psi
- Serviceability Indices
- Overall Standard Deviation
- Reliability, %
- Design Traffic, 18-kip ESALs

Performance Period, years

The pavement structure was designed for a 20-year performance period, which is typical for most flexible pavements.

Roadbed Soil Resilient Modulus, psi

The Resilient Modulus (M_r) is the material property used to characterize the support characteristics of the roadbed soils in flexible pavement design. It is a measure of the soil's deformation response to cyclic applications of loads much smaller than a failure load.

To determine the resilient modulus (M_r) of the subgrade, we utilized the correlation equation shown below:

M_r = 1,500 x CBR

Serviceability Indices

Initial serviceability is a measure of the pavement's smoothness or rideability immediately after construction. Terminal serviceability is the minimum tolerable serviceability of a pavement. When the serviceability of a pavement reaches its terminal value, rehabilitation is required. See the recommended Initial and Terminal Serviceability Indices on the table presented in the *Design Parameters – Flexible Pavements* section of this report.

Overall Standard Deviation

Overall standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given traffic. Higher values represent more variability; thus, the pavement thickness increases with higher overall standard deviations. A value of 0.45 was utilized for the flexible pavement designs presented herein.

Reliability, %

The reliability value represents a "safety factor," with higher reliabilities representing pavement structures with less chance of failure. The AASHTO Guide recommends values ranging from 50 to 99.9%, depending on the functional classification and the location (urban vs. rural) of the roadway. See the recommended Reliability value on the table presented in the *Design Parameters – Flexible Pavements* section of this report.

Design Traffic, 18-kip ESALs

The 18-kip ESALs were determined from the traffic data specified in the Unified Development Code for the City of San Antonio. See the recommended values on the table presented in the *Design Parameters – Flexible Pavements* section of this report.

Recommended Flexible Pavement Sections

Appendix 10-A of the *City of San Antonio's Design Guidance Manual* states that clay subgrade soils with a PI greater than 20 must be treated with lime or other proven methods of treatment to reduce the PI of the soil to 20 or less. Based on the results of our Atterberg Limits testing performed on the bulk samples and in the surficial clays of our borings, the PI of the clay subgrade ranges from 31 to 46. We recommend that pavements on a clay subgrade at this site include a minimum of 6 in. of lime-treated subgrade. We recommend that the required lime content reduce the PI of the subgrade soil to less than 20 and increases the pH of the soil to 12.4 or greater.

If on-site clay fill is utilized for fill grading, it should be placed and compacted as discussed in the *On-Site Clay Fill* section of this report. For areas that require fill and where pavement sections will utilize the clay subgrade recommendations, the final 6 in. of fill should be lime treated (see *Treatment of Subgrade*). If fill grading is not planned and clays remain in-place, then lime treatment of the stripped clay subgrade should be performed in conjunction with the scarifying, moisture conditioning, and recompaction process described in the *Subgrade Preparation* section of the *Pavement Construction Considerations*.

For this site, the following options for pavement sections are available. Additional options are also available and can be provided upon request.

Arterial; Clay Subgrade CBR = 4.0; Required SN = 4.96	**Layer Description**	**Layer Thickness** (1)	**Recommended SN Coefficient**	**SN Extension**
Flexible Base Option	Type C or D Surface Course	5.0 in.	0.44	2.20
	Flexible (Granular) Base	20.0 in.	0.14	2.80
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**31.0 in.**		**5.00**
Full Depth Asphalt Option	Type C or D Surface Course	4.0 in.	0.44	1.76
	Type B Base Course	9.0 in.	0.38	3.42
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**19.0 in.**		**5.18**
Mechanically Stabilized Layer Option	Type C or D Surface Course	5.0 in.	0.44	2.20
	Mechanically Stabilized Layer	17.0 in.	0.17	2.89
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**28.0 in.**		**5.09**

(1) Alternative layer thicknesses are available and can be provided upon request.

Arterial; Rock Subgrade CBR = 10.0; Required SN = 3.80	**Layer Description**	**Layer Thickness** (1)	**Recommended SN Coefficient**	**SN Extension**
Flexible Base Option	Type C or D Surface Course	4.0 in.	0.44	1.76
	Flexible (Granular) Base	15.0 in.	0.14	2.10
	Combined Total	**19.0 in.**		**3.86**
Full Depth Asphalt Option	Type C or D Surface Course	3.0 in.	0.44	1.32
	HMA Type B Base Course	7.0 in.	0.38	2.66
	Combined Total	**10.0 in.**		**3.98**
Mechanically Stabilized Layer Option	Type C or D Surface Course	3.0 in.	0.44	1.32
	Mechanically Stabilized Layer	15.0 in.	0.17	2.55
	Combined Total	**18.0 in.**		**3.87**

(1) Alternative layer thicknesses are available and can be provided upon request.

The full depth asphalt option results in a more rigid pavement section and should be carefully considered by the design team before including along the alignments. More rigid pavement sections have a higher likelihood of tensile cracking due to the potential for expansive soils heaving and creating isolated stress concentrations. The lime treated subgrade layer will assist in reducing the potential for expansive soil related movements, but will not eliminate the potential, as discussed previously.

A Mechanically Stabilized Layer (MSL) is a composite layer consisting of flexible (granular) base and a geogrid product. Geogrid provides lateral restraint to the flexible base by confining aggregate particles within the plane of the geogrid, thereby creating a reinforced, or mechanically stabilized layer.

Design Parameters – Rigid Pavements

The proposed roadways are to be evaluated in accordance with the City of San Antonio's Design Guidance Manual. We understand that the streets will be classified as Arterial. Based on information provided by the City of San Antonio, we understand that the following design parameters are required for use in the design of rigid pavements for the aforementioned street classifications.

Street Classification	Equivalent 18-kip Single Axle Load Applications (ESALs)	Reliability (%)	Serviceability (Initial/Terminal)	Standard Deviation	Rigid Pavement Slab Thickness (Minimum/Maximum)
Arterial	4,500,000	95	4.5/2.5	0.35	9.0/13.0

To calculate the required design rigid pavement thickness, we utilized a method based on the 1993 edition of the AASHTO "Guide for the Design of Pavement Structures."

The following input variables are utilized to design rigid pavements (commonly referred to as Portland Cement Concrete or PCC pavements) when using the procedures detailed in the *1993 AASHTO Guide for Design of Pavement Structures*:

- Performance Period
- 28-day Concrete Modulus of Rupture, psi
- 28-day Concrete Elastic Modulus, psi
- Effective Modulus of Subbase/Subgrade Reaction, pci
- Serviceability Indices
- Load Transfer Coefficient
- Drainage Coefficient
- Overall Standard Deviation
- Reliability, %
- Design Traffic, 18-kip ESALs

Performance Period

The pavement structure was designed for a 30-year performance period which is typical for most rigid pavements.

28-day Concrete Modulus of Rupture, Mr

The M_r of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. A M_r of approximately 600 psi at 28 days was used in the analysis and is typical of local concrete production.

28-day Concrete Elastic Modulus

Elastic modulus of concrete is an indication of concrete stiffness and varies depending on the coarse aggregate type used in the concrete. A modulus of 4,000,000 psi was used for this pavement design.

Effective Modulus of Subbase/Subgrade Reaction: k-value

Concrete slab support is characterized by the modulus of subgrade/subbase reaction, otherwise known as the k-value, with units typically shown as psi/in. A subbase layer is typically recommended for higher traffic volume roadways or in areas where additional concrete slab support is warranted. Based on the use of subgrade, k-values of 120 psi/in. (clay subgrade) and 200 psi/in. (rock subgrade), was used in the rigid pavement design procedure.

Serviceability Indices

Initial serviceability is a measure of the pavement's smoothness or rideability immediately after construction. Terminal serviceability is the minimum tolerable serviceability of a pavement. When the serviceability of a pavement reaches its terminal value, rehabilitation is required. See the recommended Initial and Terminal Serviceability Indices on the table presented in the *Design Parameters – Rigid Pavements* section of this report.

Load Transfer Coefficient

The load transfer coefficient is used to incorporate the effect of dowels, reinforcing steel, tied shoulders, and tied curb and gutter on reducing the stress in the concrete slab due to traffic loading and therefore causing a reduction in the required concrete slab thickness.

The load transfer coefficient used in this pavement design is 3.2 for pavements designed without load transfer devices or CRCP.

Drainage Coefficient

The drainage coefficient characterizes the quality of drainage of the subbase layers under the concrete slab. Pavement structures with good drainage do not give water the chance to saturate the subbase and subgrade; thus, subgrade pumping is not as likely to occur. For the project region, a drainage coefficient of 1.01 is utilized for rigid pavement design.

Overall Standard Deviation

Overall standard deviation accounts for both chance variation in the traffic prediction and normal variation in pavement performance prediction for a given traffic. Higher values represent more variability; thus, the pavement thickness increases with higher overall standard deviations. See the recommended Overall Standard Deviation on the table presented in the *Design Parameters – Rigid Pavements* section of this report.

Reliability, %

The reliability value represents a "safety factor," with higher reliabilities representing pavement structures with less chance of failure. The AASHTO Guide recommends values ranging from 50 to 99.9%, depending on the functional classification and the location (urban vs. rural) of the roadway. See the recommended Reliability on the table presented in the *Design Parameters – Rigid Pavements* section of this report.

Design Traffic 18-kip ESAL

The 18-kip ESALs were determined from the street classifications as discussed previously in the *Design Parameters – Rigid Pavements* section of this report.

Recommended Rigid Pavement Sections

Appendix 10-A of the *City of San Antonio's Design Guidance Manual* states that clay subgrade soils with a PI greater than 20 must be treated with lime or other proven methods of treatment to reduce the PI of the soil to less than 20. Based on the results of our Atterberg Limits testing performed on the bulk samples and in the surficial clays of our borings, the PI of the clay subgrade ranges from 31 to 46. We recommend that pavements on a clay subgrade at this site include a minimum of 6 in. of lime-treated subgrade. We recommend that the required lime content reduce the PI of the subgrade soil to 20 or less and increases the pH of the soil to 12.4 or greater. The recommended concrete slab thicknesses determined with the inputs discussed above are presented in the table below. Additional options are also available and can be provided upon request.

Arterial Street Classification	Layer Description	Layer Thickness
Clay Subgrade	Concrete Pavement (1) Treated Subgrade **Combined Total**	10.0 in. 6.0 in. **16.0 in.**
Rock Subgrade	Concrete Pavement (1) **Combined Total**	9.5 in. **9.5 in.**

1) Concrete pavement should consist of jointed plain concrete pavement with load transfer devices.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

The subgrade should be prepared in accordance with the recommendations in the *Site Preparation* section under *Foundation Construction Considerations* of this report.

DRAINAGE CONSIDERATIONS

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface 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:

- 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.
- Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils.
- Pavement surfaces should be maintained to help reduce 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.

PAVEMENT SELECT FILL

If utilized beneath pavement sections, select fill preferably should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the 2014 TxDOT Standard Specifications, Item 247 – *Flexible Base*, Type A, Grade 2.

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.

Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 100 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 select fill is placed over moisture conditioned clays, the first lift of select fill may be placed at 95 percent of the maximum density as determined by TxDOT, Tex 113-E, Compaction Test.

If excavations extend to significant depths into the limestone formation, consideration can be given to utilizing the excavated limestone as an “alternative 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. Alternative select fill materials shall have a maximum liquid limit not exceeding 40 and a plasticity index between 7 and 20. 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 on-site materials cannot be processed to meet the required criteria, imported select fill materials should be utilized.

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.

ON-SITE CLAY FILL

We recommend that on-site clays be placed in loose lifts not exceeding 8 in. 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.

TREATMENT OF SUBGRADE

Treatment of the natural subgrade soils should be in accordance with the TxDOT Standard Specifications, Item 260 (Lime treatment) or Item 275 (cement treatment). A sufficient quantity of product should be mixed with the subgrade soils to reduce the soil mixture plasticity index to 20 or less and increase the pH to 12.4 or greater. Based on the results of the pH-Lime Series Curves and lime treated Atterberg limit tests, we recommend that at least 4 percent hydrated lime or 4 percent cement treatment by weight be used to increase the pH of the subgrade clays to 12.4 or higher and decrease the plasticity index to 20 or below. **For construction purposes, we recommend that the optimum lime or cement content of the subgrade soils be determined by laboratory testing with representative samples of the subgrade materials being used for this project.** If lime or cement treatment is utilized, we also recommend that soluble sulfate content testing be completed to determine the susceptibility of the soils to sulfate-induced heave in lime-treated soils.

Lime or cement-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, Compaction Test.

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to TxDOT Standard Specifications, Item 247, Type A, Grade 1-2. 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. Warm mix asphalt may also be used for the surface courses and should conform to paragraph 2.6.2 of Item 340. 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 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.

MISCELLANEOUS PAVEMENT RELATED CONSIDERATIONS

Drainage Considerations

As with any soil-supported structure, the satisfactory performance of a pavement system is contingent on the provision of adequate surface 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:

- 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.
- 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 be installed to a sufficient depth to reduce infiltration of water beneath the curbs and into the pavement base materials.
- 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.

Utilities

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.
- 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.

Alternatively, consideration may be given to utilizing a low strength flowable fill in utility trenches located within the roadway alignments.

Longitudinal Cracking

It should be understood that asphalt pavement sections in expansive soil environments can develop longitudinal cracking along unprotected pavement edges. In the semi-arid climate of south central Texas this condition typically occurs along the unprotected edges of pavements where moisture fluctuation is allowed to occur over the lifetime of the pavements.

Pavements that do not have a protective barrier to reduce moisture fluctuation of the highly expansive clay subgrade between the exposed pavement edge and that beneath the pavement section tend to develop longitudinal cracks 1 to 4 ft from the edge of the pavement. Once these cracks develop, further degradation and weakening of the underlying granular base may occur due to water seepage through the cracks. The occurrence of these cracks can be more prevalent in the absence of lateral restraint and steep embankments. This problem can best be addressed by providing either a horizontal or vertical moisture barrier at the unprotected pavement edge.

A horizontal barrier is commonly in the form of a paved shoulder extending 8 feet or greater beyond the edge of the pavement. Other methods of shoulder treatment, such as using geofabrics beyond the edge of the roadway, are sometimes used in an effort to help reduce longitudinal cracking. Although this alternative does not eliminate the longitudinal cracking phenomenon, the location of the cracking is transferred to the shoulder rather than within the traffic lane.

Vertical barriers installed along the unprotected edges of roadway pavements are also effective in preventing non-uniform drying and shrinkage of the subgrade clays. These barriers are typically in the form of a vertical moisture barrier/membrane extending 6 feet or greater below the top of the subgrade at the pavement edge. Both types of barriers must be sealed at the edge of the pavement to prevent a crack that would facilitate the drying of the subgrade clays.

At a minimum, we recommend that the curbs are constructed such that the depth of the curb extends through the entire depth of the granular base material and into the subgrade to act as a protective barrier against the infiltration of water into the granular base.

In most cases, a longitudinal crack does not immediately compromise the structural integrity of the pavement system. However, if left unattended, infiltration of surface water runoff into the crack will result in isolated saturation of the underlying base. This will result in pumping of the flexible base, which could lead to rutting, cracking, and pot-holes. For this reason, we recommend that the owner of the facility immediately seal the cracks and develop a periodic sealing program.

Pavement Maintenance

Regular pavement maintenance is critical in maintaining pavement performance over a period of several years. All cracks that develop in asphalt pavements should be regularly sealed. Areas of moderate to severe fatigue cracking (also known as alligator cracking) should be sawcut and removed. The underlying base should be checked for contamination or loss of support and any insufficiencies fixed or removed and the entire area patched. Other typical maintenance techniques should be followed as required.

Curb and Gutter

It is good practice to construct curbs such that the depth of the curb extends through the entire depth of the granular base material to act as a protective barrier against the infiltration of water into the granular base. Pavements that do not have this protective barrier to moisture tend to develop longitudinal cracks 1 to 2 ft from the edge of the pavement. Once these cracks develop, further degradation and weakening of the underlying granular base may occur due to water seepage through the cracks.

Construction Traffic

Construction traffic on prepared subgrade, granular base or asphalt treated base (black base) should be restricted as much as possible until the protective asphalt surface pavement is applied. Significant damage to the underlying layers resulting in weakening may occur if heavily loaded vehicles are allowed to use these areas.

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.

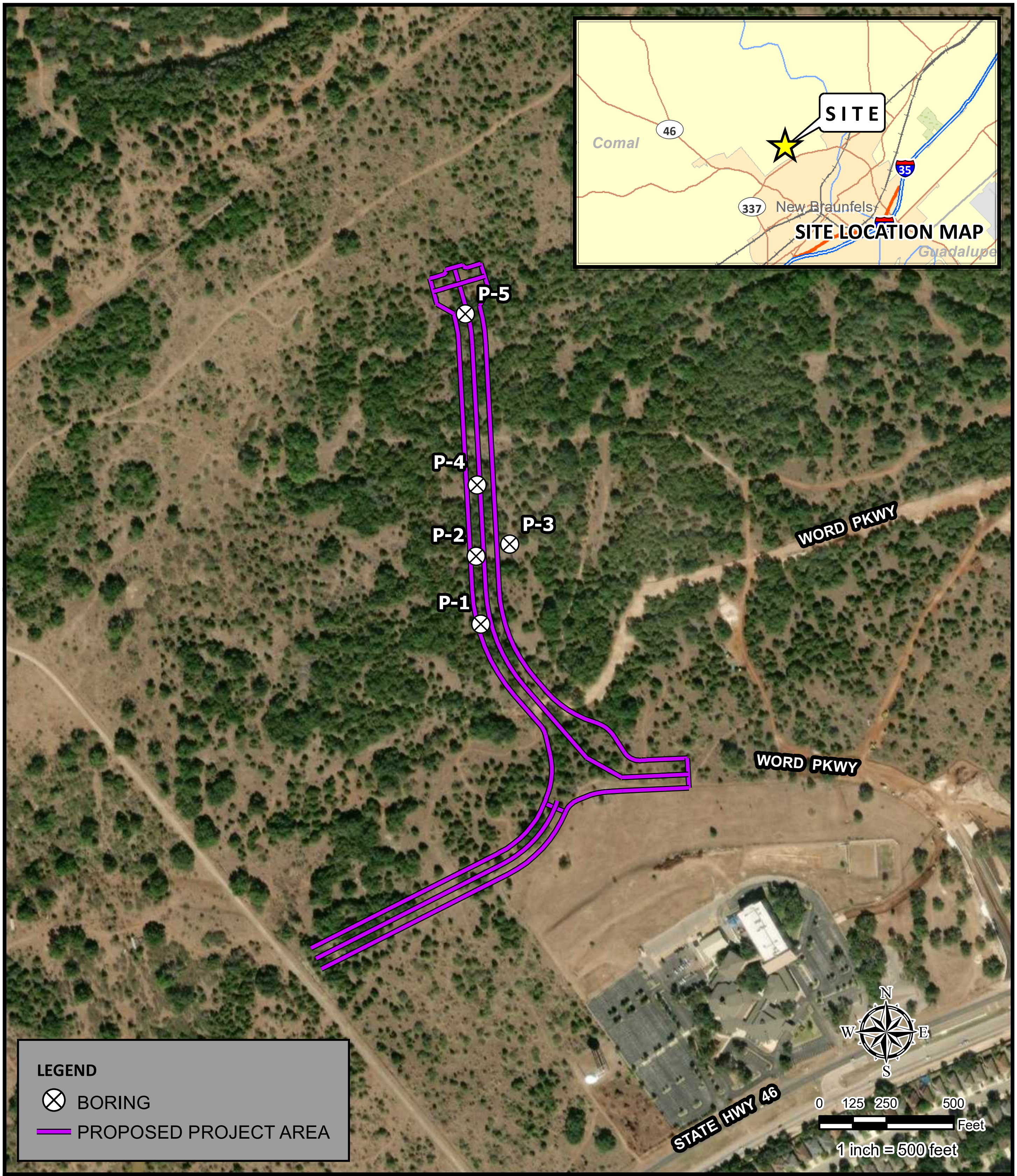
BUDGETING FOR CONSTRUCTION TESTING

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 a scope and budget for these services.

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

ATTACHMENTS



Source: Esri, Maxar, Earthstar Geographics, and the GIS User Community

RABA KISTNER

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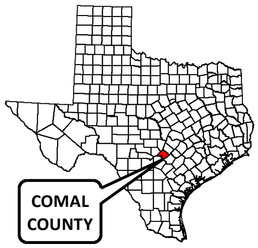
TBPE Firm Number 3257

BORING LOCATION MAP

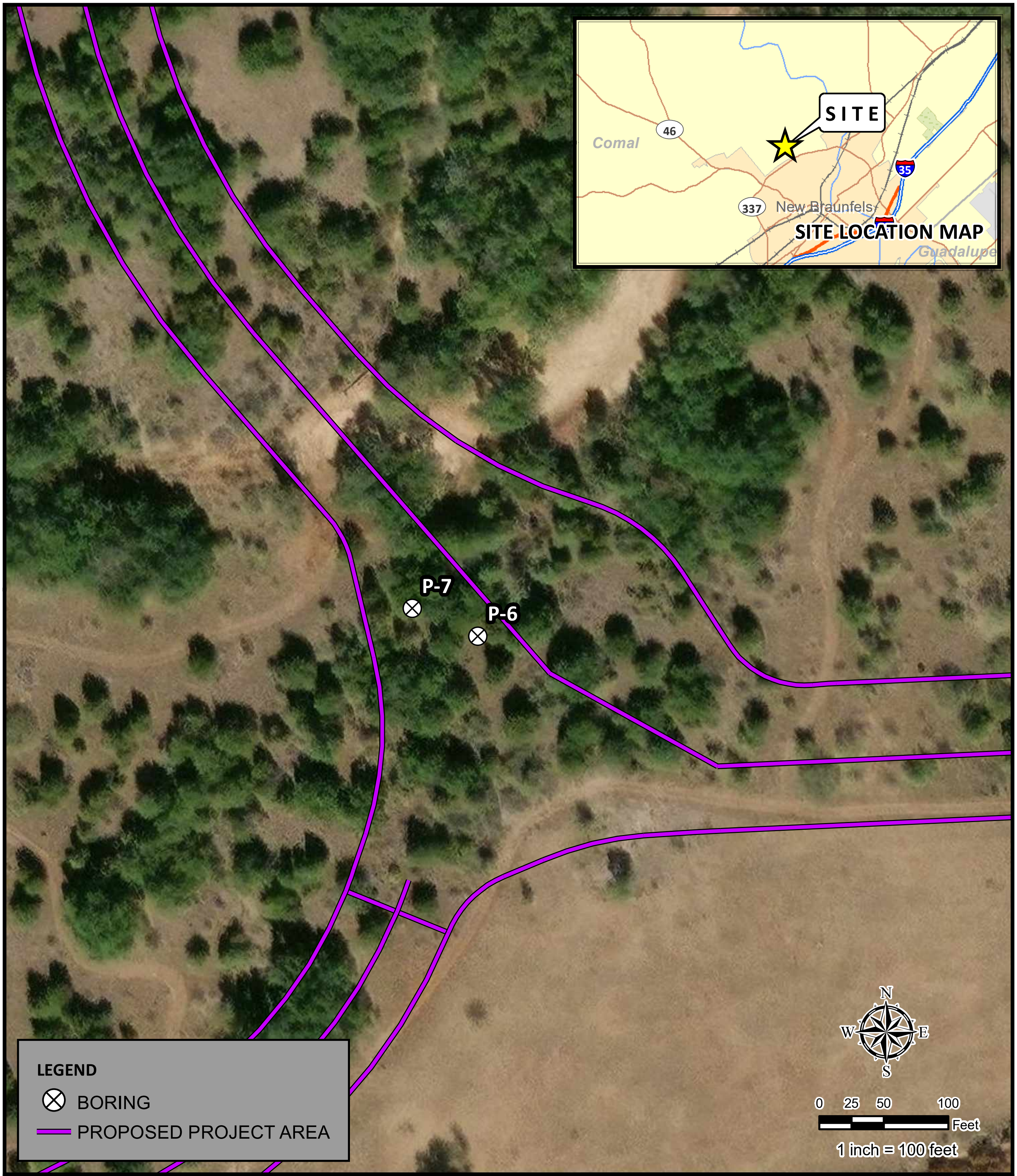
WORD PARKWAY – PHASE 3
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS



PROJECT No.:	ANA22-022-00
ISSUE DATE:	2/27/2024
DRAWN BY:	BM
CHECKED BY:	RD
REVIEWED BY:	TIP

FIGURE

1A



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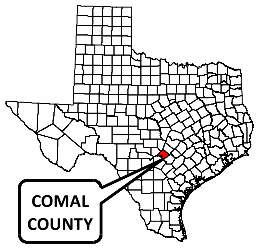
BORING LOCATION MAP

WORD PARKWAY – PHASE 3
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS



PROJECT No.:	ANA22-022-00
ISSUE DATE:	2/27/2024
DRAWN BY:	BM
CHECKED BY:	RD
REVIEWED BY:	TIP

FIGURE

1B

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. P-1

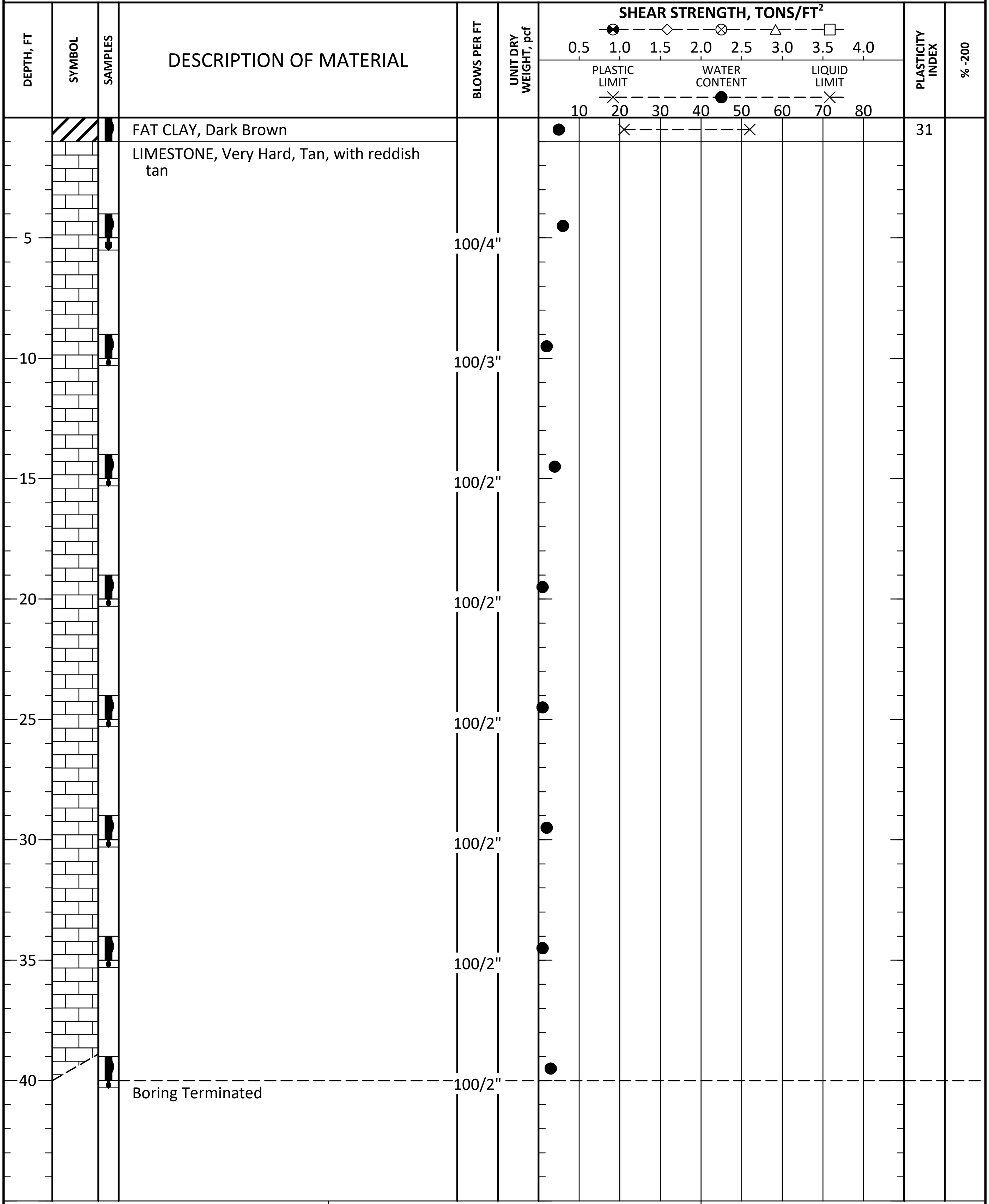
Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.73068; W 98.14483

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	FAT CLAY, Dark Brown			31	
	LIMESTONE, Very Hard, Tan, with reddish tan				
5		100/4"			
10		100/3"			
15		100/2"			
20		100/2"			
25		100/2"			
30		100/2"			
35		100/2"			
40	Boring Terminated	100/2"			



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

DEPTH DRILLED: 40.0 ft
DATE DRILLED: 7/18/2022

DEPTH TO WATER: Dry
DATE MEASURED: 7/18/2022

PROJ. No.: ANA22-022-00
FIGURE: 2

LOG OF BORING NO. P-2

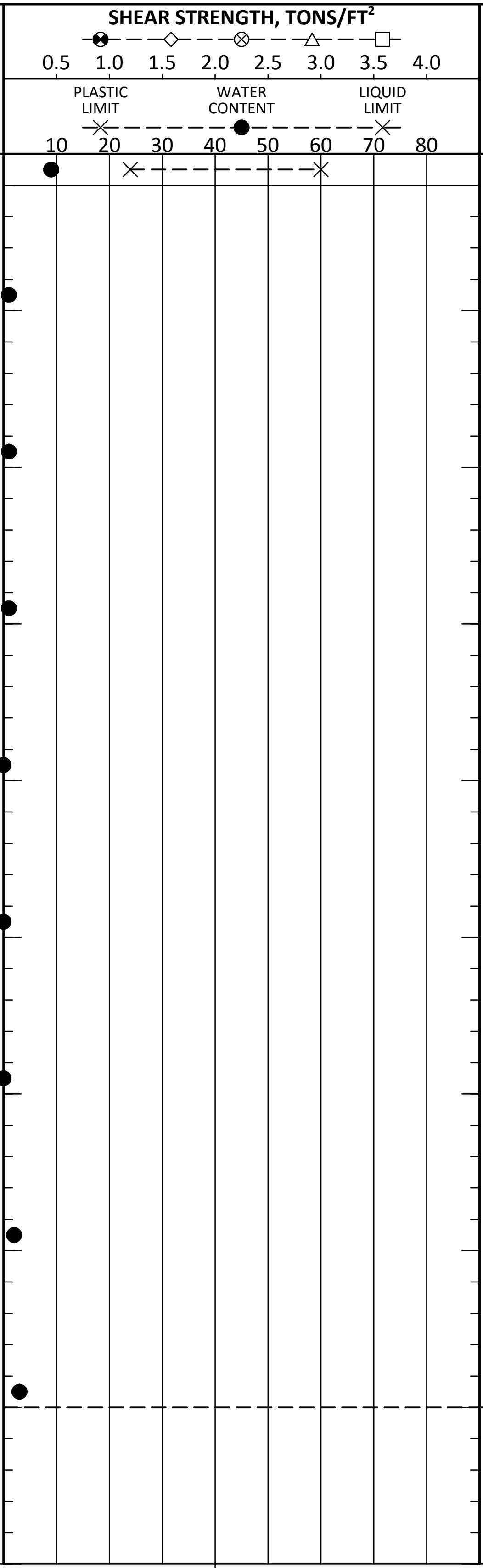
Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.73137; W 98.14487

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			FAT CLAY, Dark Brown			36	
			LIMESTONE, Very Hard,Tan, with reddish tan				
5				100/4"			
10				100/2"			
15				100/2"			
20				100/3"			5
25				100/2"			
30				100/2"			
35				100/2"			
40				100/2"			
			Boring Terminated				



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

DEPTH DRILLED: 40.0 ft
DATE DRILLED: 7/18/2022
DEPTH TO WATER: Dry
DATE MEASURED: 7/18/2022
PROJ. No.: ANA22-022-00
FIGURE: 3

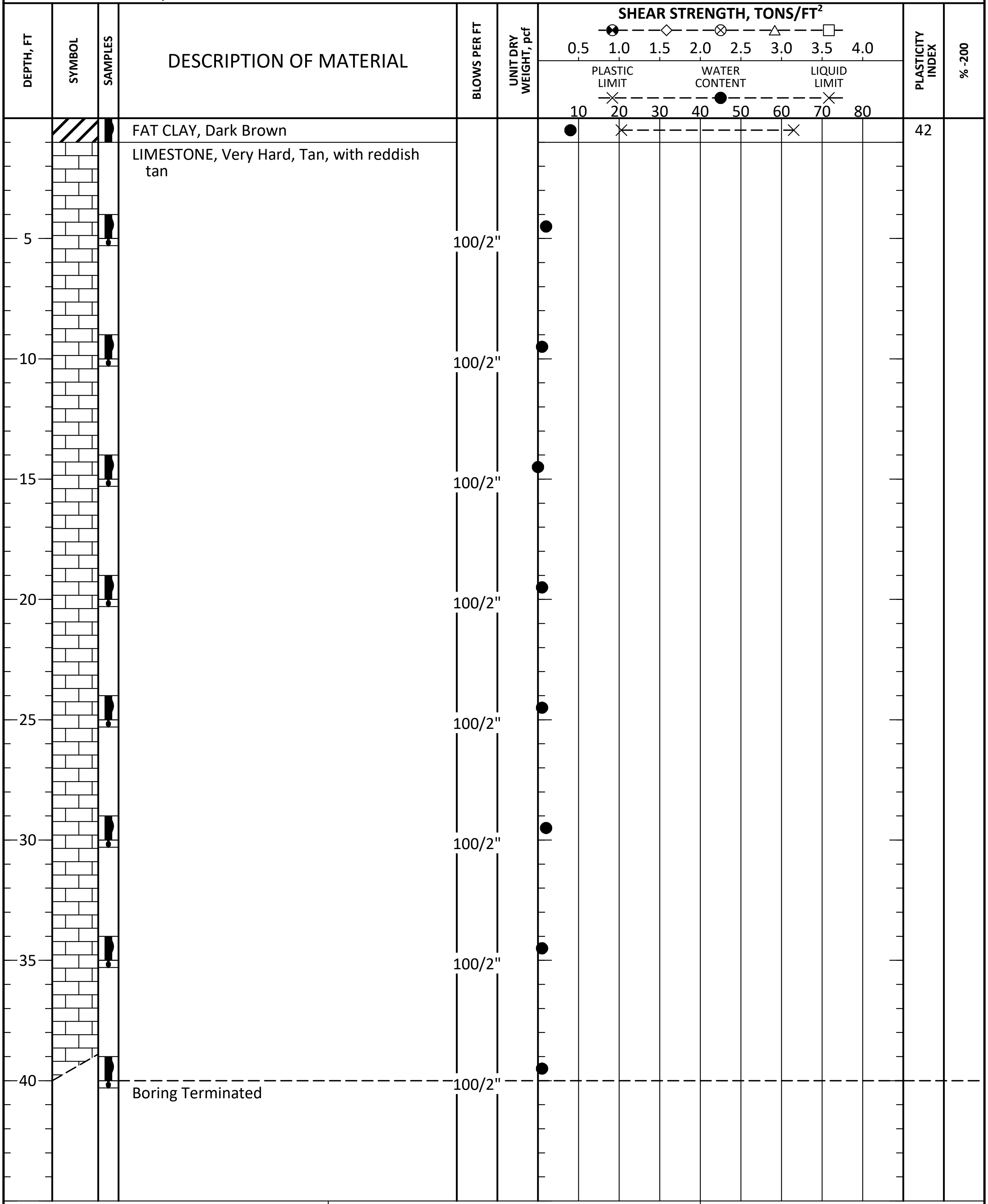
LOG OF BORING NO. P-3

Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.73149; W 98.14448



FAT CLAY, Dark Brown

LIMESTONE, Very Hard, Tan, with reddish tan

Boring Terminated

DEPTH, FT	BLOWS PER FT	PLASTICITY INDEX
0		42
5	100/2"	
10	100/2"	
15	100/2"	
20	100/2"	
25	100/2"	
30	100/2"	
35	100/2"	
40	100/2"	

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

DEPTH DRILLED: 40.0 ft
DATE DRILLED: 7/19/2022

DEPTH TO WATER: Dry
DATE MEASURED: 7/19/2022

PROJ. No.: ANA22-022-00
FIGURE: 4

LOG OF BORING NO. P-4

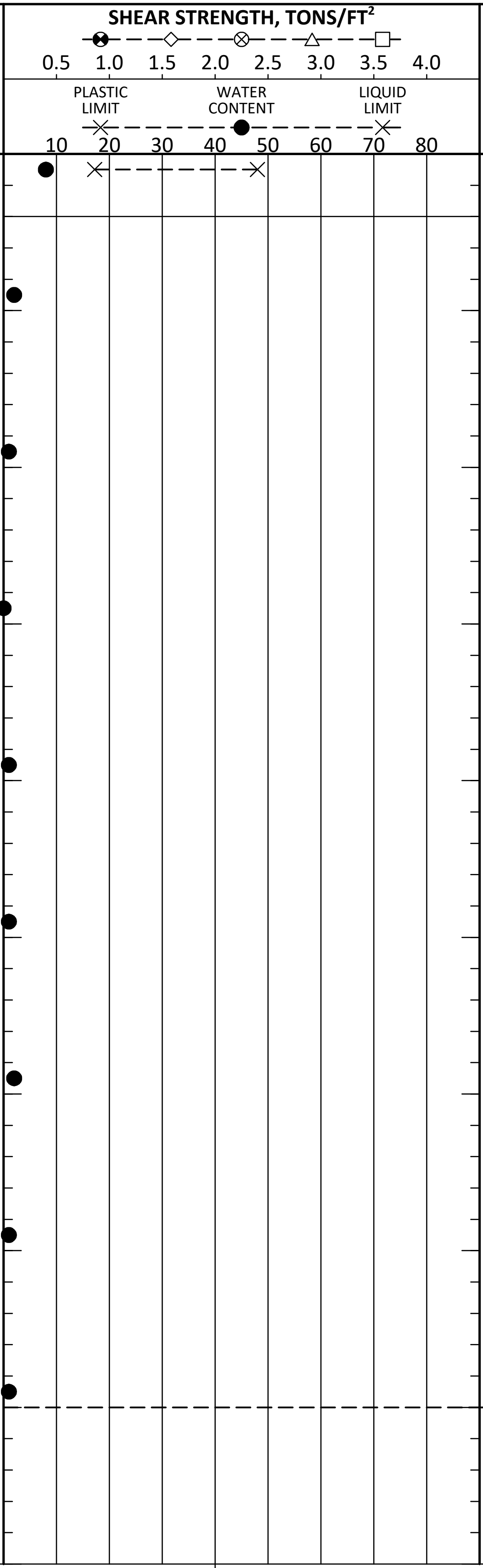
Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.73210; W 98.14485

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	LEAN CLAY, Dark Brown			31	
2	LIMESTONE, Very Hard, Tan, with reddish tan				
5		100/2"			
8	-with clay seam 8 ft to 9 ft				
10		100/2"			
15		100/2"			
20		100/2"			
25		100/2"			
30		100/2"			
35		100/2"			
40	Boring Terminated	100/2"			



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

DEPTH DRILLED: 40.0 ft
DATE DRILLED: 7/19/2022
DEPTH TO WATER: Dry
DATE MEASURED: 7/19/2022
PROJ. No.: ANA22-022-00
FIGURE: 5

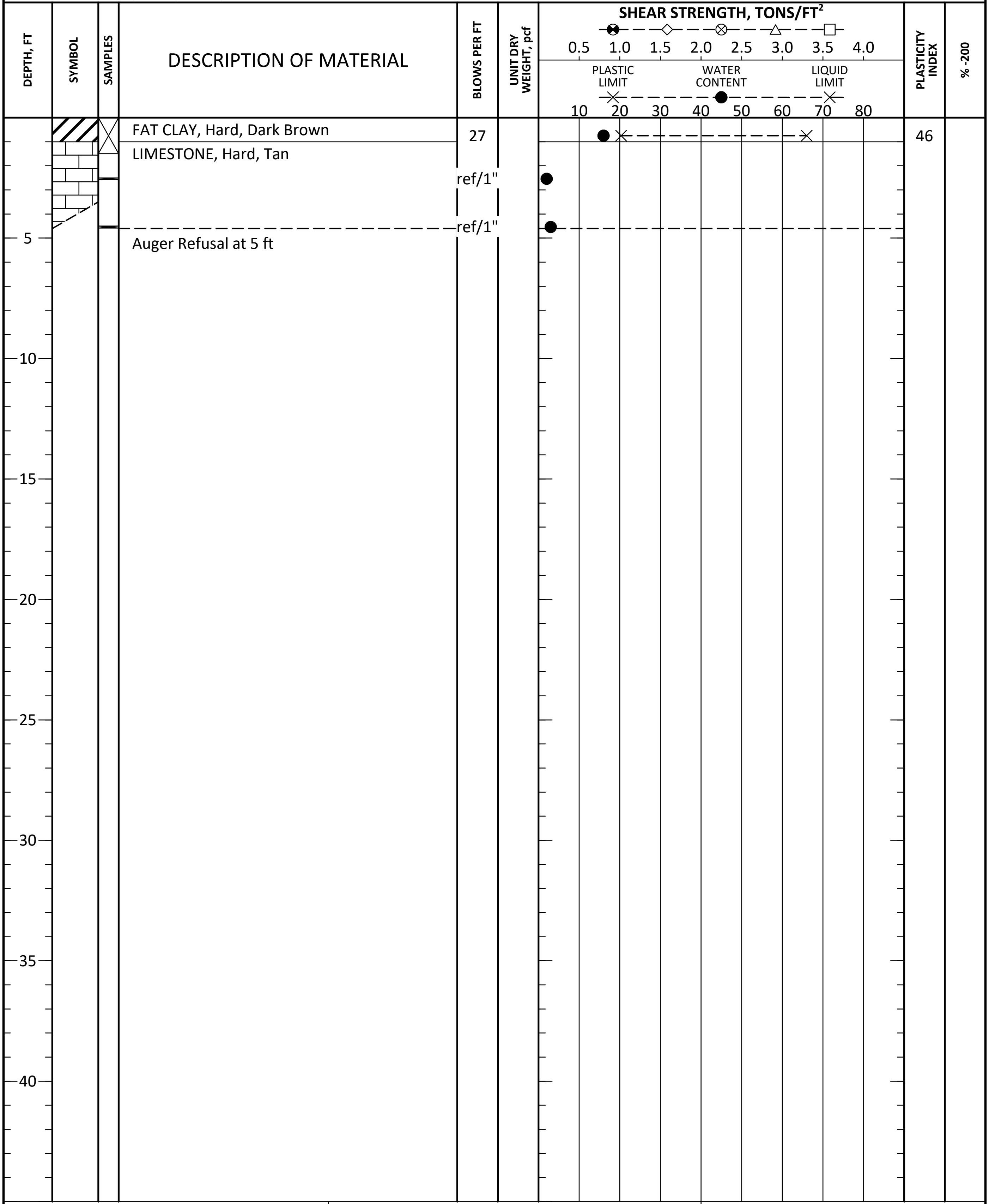
LOG OF BORING NO. P-5

Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.73385; W 98.14498



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX
FAT CLAY, Hard, Dark Brown	27	46
LIMESTONE, Hard, Tan	ref/1"	
	ref/1"	
Auger Refusal at 5 ft		

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

DEPTH DRILLED: 5.0 ft
DATE DRILLED: 7/19/2022
DEPTH TO WATER: Dry
DATE MEASURED: 7/19/2022
PROJ. No.: ANA22-022-00
FIGURE: 6

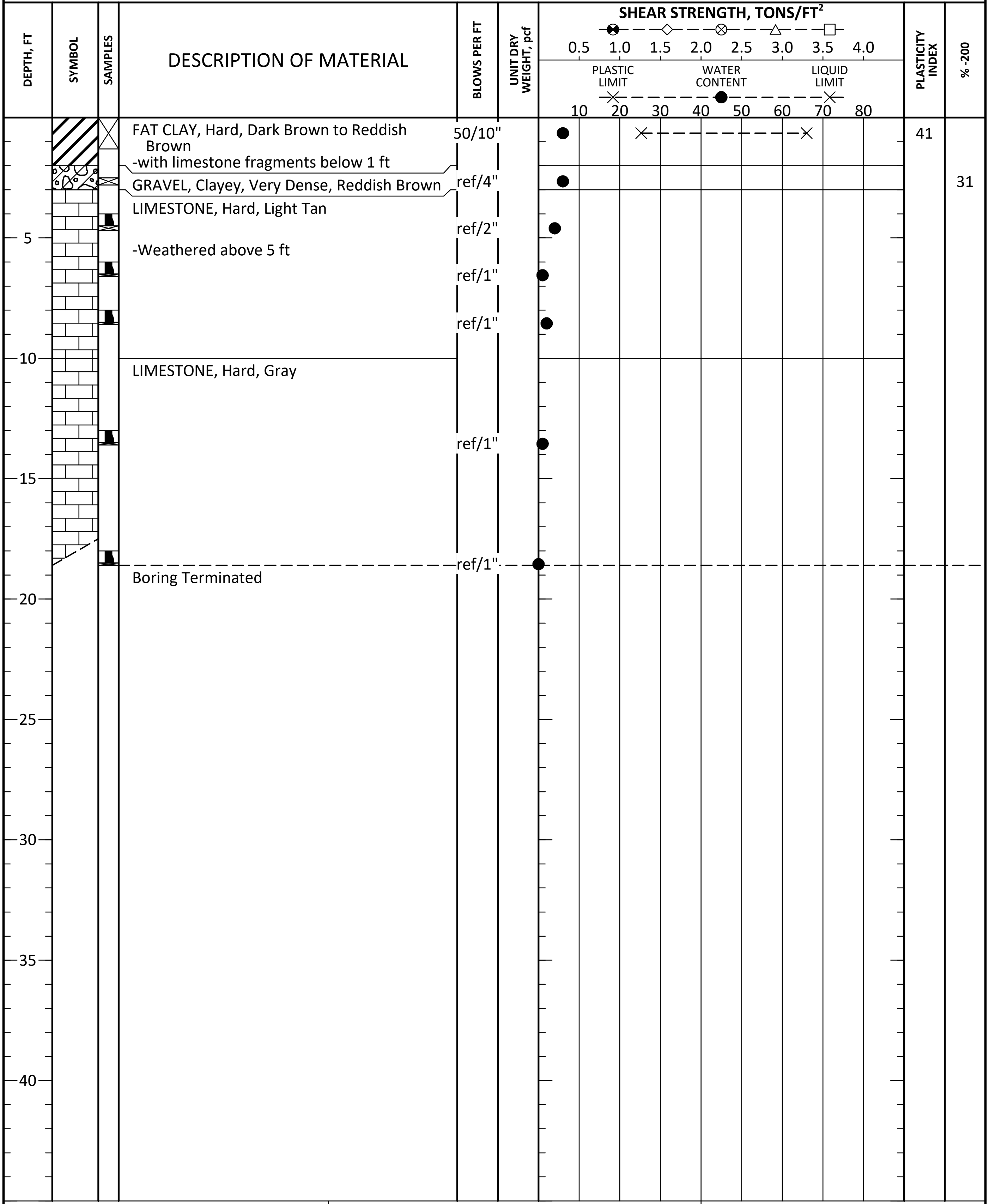
LOG OF BORING NO. P-6

Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.72938; W 98.14377



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX	% -200
FAT CLAY, Hard, Dark Brown to Reddish Brown -with limestone fragments below 1 ft	50/10"	41	
GRAVEL, Clayey, Very Dense, Reddish Brown	ref/4"		31
LIMESTONE, Hard, Light Tan -Weathered above 5 ft	ref/2" ref/1" ref/1"		
LIMESTONE, Hard, Gray	ref/1" ref/1"		
Boring Terminated			

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

DEPTH DRILLED: 18.6 ft
DATE DRILLED: 1/10/2024
DEPTH TO WATER: Dry
DATE MEASURED: 1/10/2024
PROJ. No.: ANA22-022-00
FIGURE: 7

LOG OF BORING NO. P-7

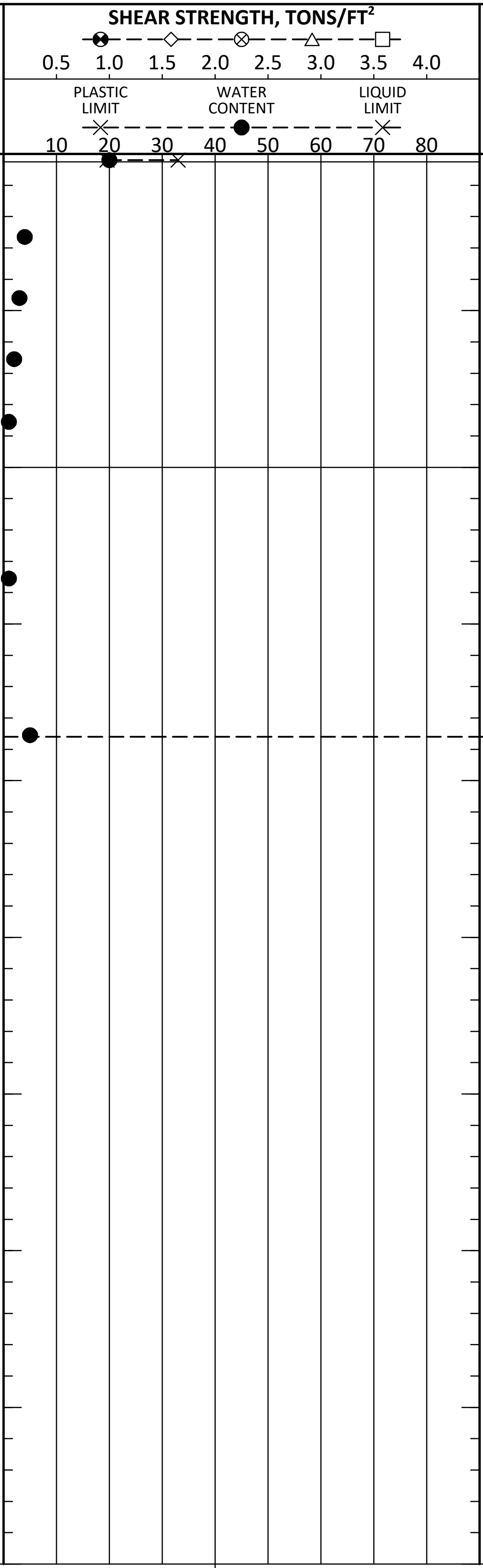
Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.72944; W 98.14393

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
FAT CLAY, Hard, Dark Brown, with gravel	ref/5"		13	
LIMESTONE, Hard, Light Tan				
	ref/1"			
-Weathered from 3 to 10 ft				
	ref/1"			
	ref/1"			
	ref/1"			
LIMESTONE, Hard, Gray				
	ref/1"			
	ref/1"			
Boring Terminated				



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

DEPTH DRILLED: 18.6 ft
DATE DRILLED: 1/10/2024

DEPTH TO WATER: Dry
DATE MEASURED: 1/10/2024

PROJ. No.: ANA22-022-00
FIGURE: 8

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. ANA22-022-00

FIGURE 9a

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.

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: Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

FILE NAME: ANA22-022-00 VERAMENDI WORD PKWY PHASE 3 - GINT.GPJ 2/26/2024

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
P-1	0.0 to 1.0		5	52	21	31					
	4.0 to 5.0		6								
	5.0 to 5.5	100/4"									
	9.0 to 10.0		2								
	10.0 to 10.3	100/3"									
	14.0 to 15.0		4								
	15.0 to 15.3	100/2"									
	19.0 to 20.0		1								
	20.0 to 20.3	100/2"									
	24.0 to 25.0		1								
	25.0 to 25.3	100/2"									
	29.0 to 30.0		2								
	30.0 to 30.3	100/2"									
	34.0 to 35.0		1								
	35.0 to 35.3	100/2"									
	39.0 to 40.0		3								
	40.0 to 40.3	100/2"									
P-2	0.0 to 1.0		9	60	24	36					
	4.0 to 5.0		1								
	5.0 to 5.5	100/4"									
	9.0 to 10.0		1								
	10.0 to 10.3	100/2"									
	14.0 to 15.0		1								
	15.0 to 15.3	100/2"									
	19.0 to 20.0		0						5		
	20.0 to 20.3	100/3"									
	24.0 to 25.0		0								
	25.0 to 25.3	100/2"									
	29.0 to 30.0		0								
	30.0 to 30.3	100/2"									
	34.0 to 35.0		2								
	35.0 to 35.3	100/2"									
	39.0 to 40.0		3								
	40.0 to 40.3	100/2"									
P-3	0.0 to 1.0		8	63	21	42					
	4.0 to 5.0		2								
	5.0 to 5.3	100/2"									
	9.0 to 10.0		1								
	10.0 to 10.3	100/2"									

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: Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

FILE NAME: ANA22-022-00 VERAMENDI WORD PKWY PHASE 3 - GINT.GPJ 2/26/2024

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
P-3	14.0 to 15.0		0								
	15.0 to 15.3	100/2"									
	19.0 to 20.0		1								
	20.0 to 20.3	100/2"									
	24.0 to 25.0		1								
	25.0 to 25.3	100/2"									
	29.0 to 30.0		2								
	30.0 to 30.3	100/2"									
	34.0 to 35.0		1								
	35.0 to 35.3	100/2"									
	39.0 to 40.0		1								
	40.0 to 40.3	100/2"									
P-4	0.0 to 1.0		8	48	17	31					
	4.0 to 5.0		2								
	5.0 to 5.3	100/2"									
	9.0 to 10.0		1								
	10.0 to 10.3	100/2"									
	14.0 to 15.0		0								
	15.0 to 15.3	100/2"									
	19.0 to 20.0		1								
	20.0 to 20.3	100/2"									
	24.0 to 25.0		1								
	25.0 to 25.3	100/2"									
	29.0 to 30.0		2								
	30.0 to 30.3	100/2"									
	34.0 to 35.0		1								
	35.0 to 35.3	100/2"									
	39.0 to 40.0		1								
	40.0 to 40.3	100/2"									
P-5	0.0 to 1.5	27	16	66	20	46					
	2.5 to 2.6	ref/1"	2								
	4.5 to 4.6	ref/1"	3								
P-6	0.0 to 1.3	50/10"	6	66	25	41					
	2.5 to 2.8	ref/4"	6						31		
	4.0 to 4.5										
	4.5 to 4.7	ref/2"	4								
	6.0 to 6.5										
	6.5 to 6.6	ref/1"	1								
	8.0 to 8.5										

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: Veramendi Word Parkway Phase 3- Pavements
New Braunfels, Texas

FILE NAME: ANA22-022-00 VERAMENDI WORD PKWY PHASE 3 - GINT.GPJ 2/26/2024

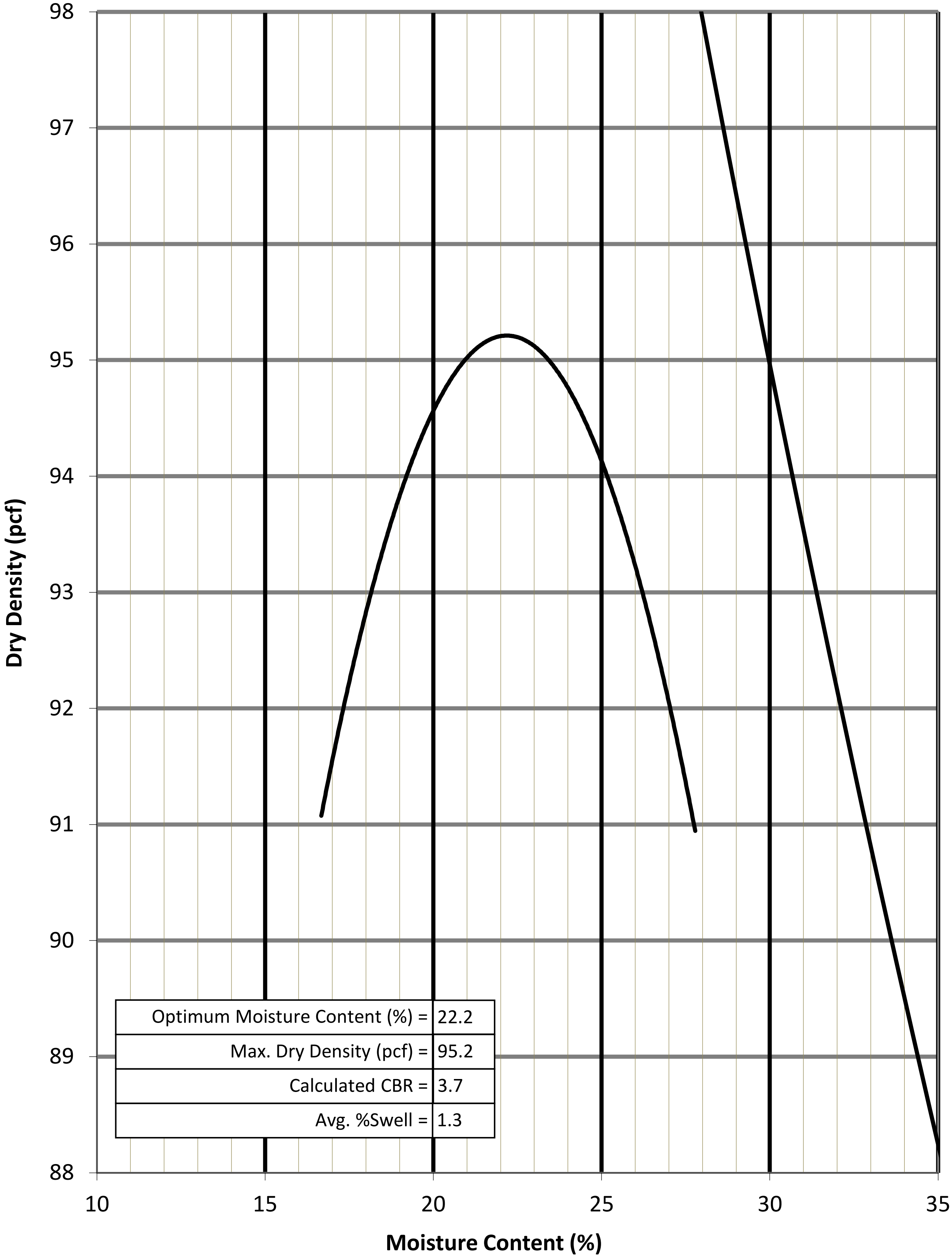
Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
P-6	8.5 to 8.6	ref/1"	2								
	13.0 to 13.5										
	13.5 to 13.6	ref/1"	1								
	18.0 to 18.5										
	18.5 to 18.6	ref/1"	0								
P-7	0.0 to 0.4	ref/5"	20	33	20	13					
	2.0 to 2.5										
	2.5 to 2.8	ref/1"	4								
	4.0 to 4.5										
	4.5 to 4.7	ref/1"	3								
	6.0 to 6.5										
	6.5 to 6.6	ref/1"	2								
	8.0 to 8.5										
	8.5 to 8.6	ref/1"	1								
	13.0 to 13.5										
	13.5 to 13.6	ref/1"	1								
	18.0 to 18.5										
	18.5 to 18.6	ref/1"	5								

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

CU = Consolidated Undrained Triaxial

MOISTURE DENSITY RELATIONSHIP CURVE - TEX-114-E

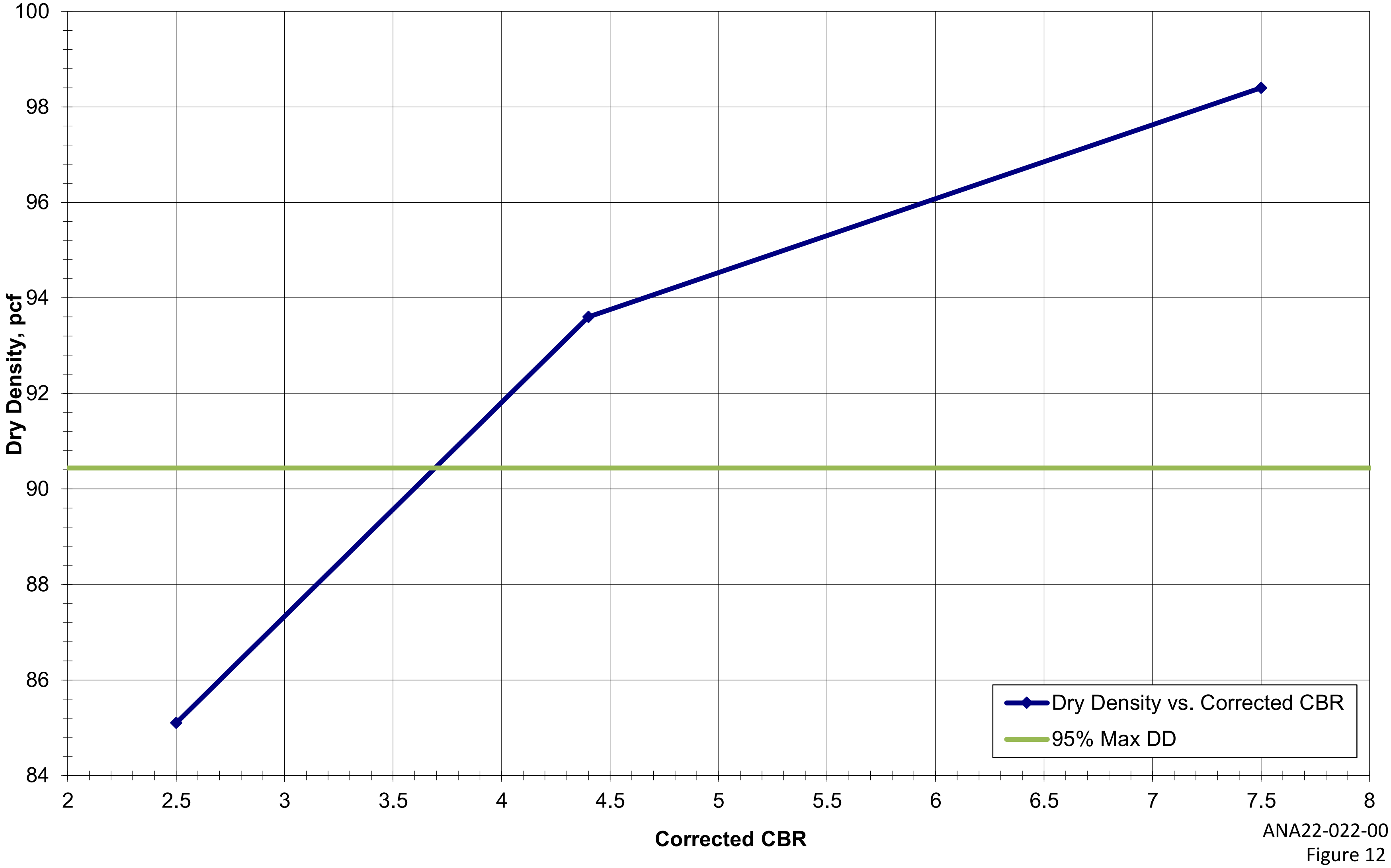
Word Parkway - Phase 3, Veramendi Master Planned Development, New Braunfels, Texas



Optimum Moisture Content (%) =	22.2
Max. Dry Density (pcf) =	95.2
Calculated CBR =	3.7
Avg. %Swell =	1.3

Dry Density vs. Corrected CBR

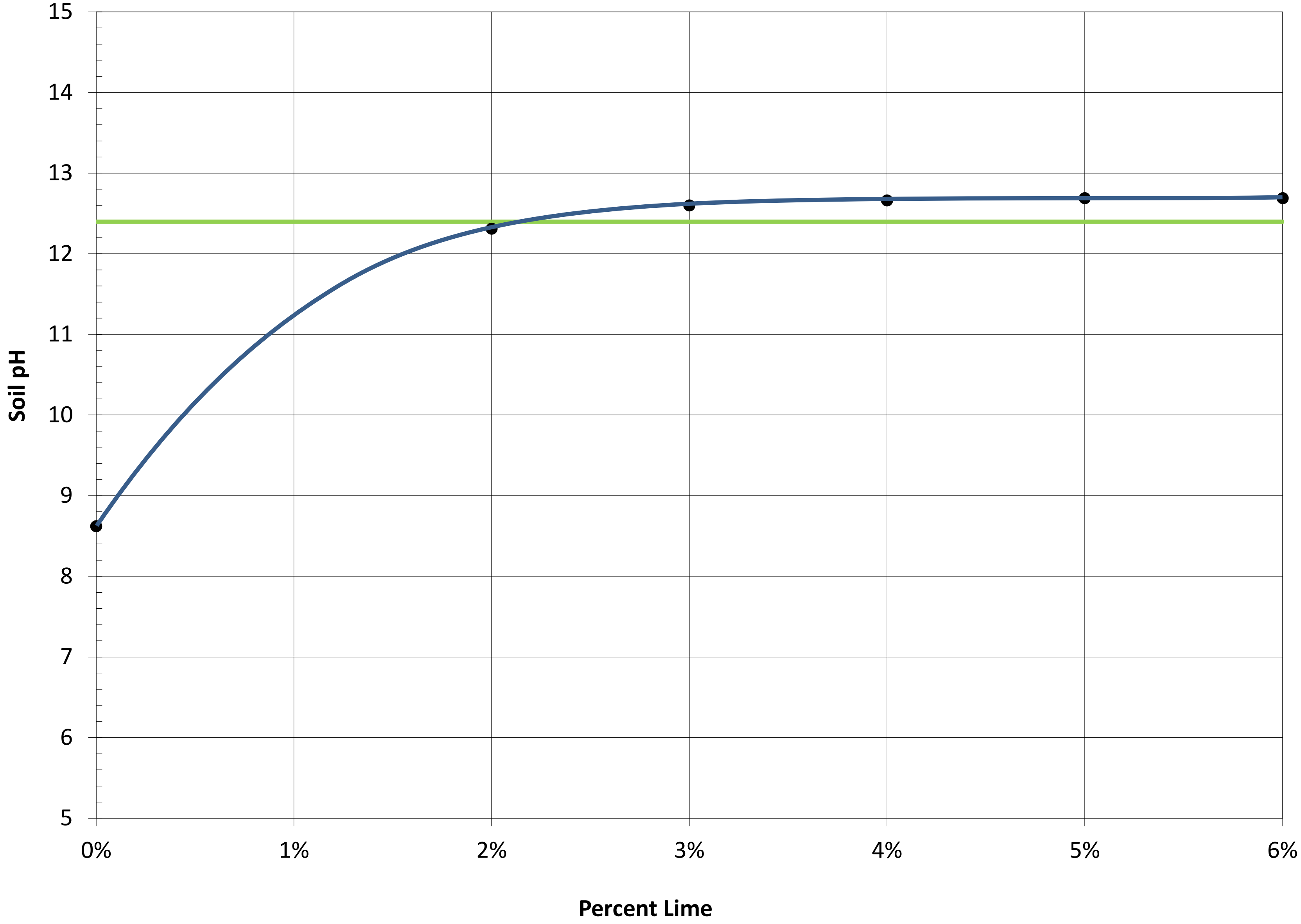
Word Parkway - Phase 3, Veramendi Master Planned Development, New Braunfels, Texas



ANA22-022-00
Figure 12

pH-LIME SERIES CURVE

Word Parkway - Phase 3, Veramendi Master Planned Development, New Braunfels, Texas

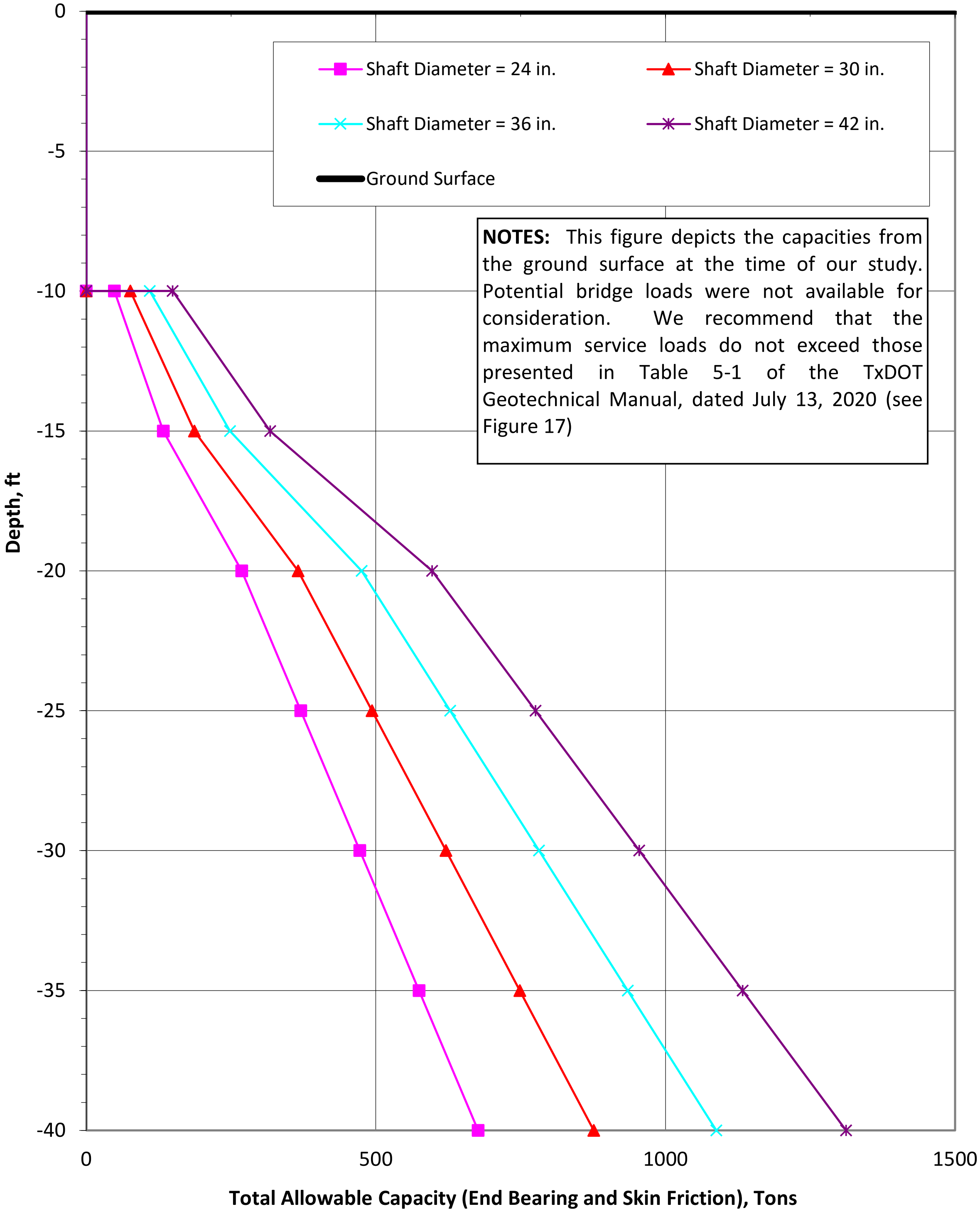


DRILLED PIER AXIAL CAPACITY CURVE

Straight Shaft Piers

Veramendi - Word Parkway
Comal County, Texas

Composite Boring Profile



NOTES: This figure depicts the capacities from the ground surface at the time of our study. Potential bridge loads were not available for consideration. We recommend that the maximum service loads do not exceed those presented in Table 5-1 of the TxDOT Geotechnical Manual, dated July 13, 2020 (see Figure 17)

Figure 14

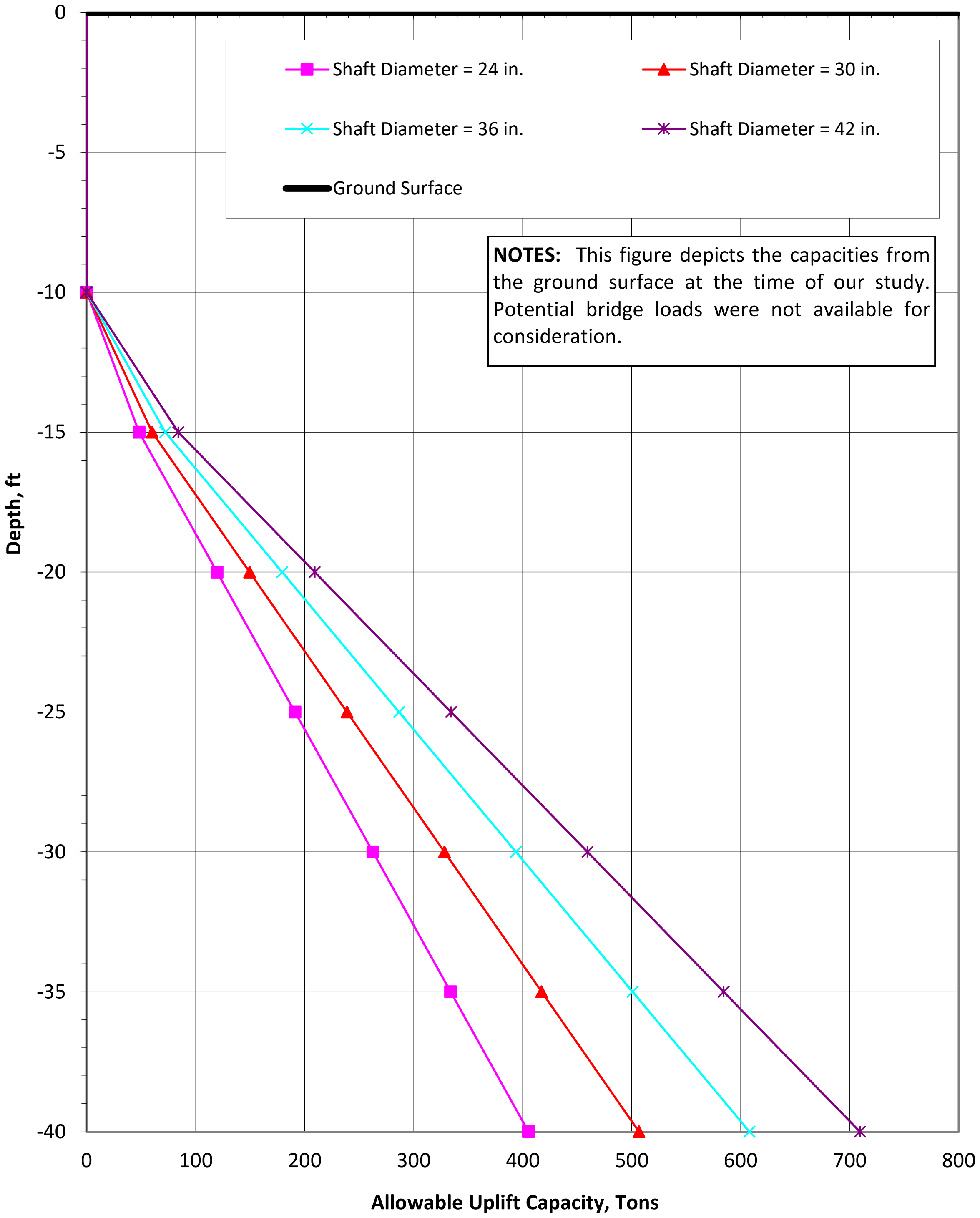
DRILLED PIER UPLIFT CAPACITY CURVE

Straight Shaft Piers

Veramendi - Word Parkway

Comal County, Texas

Composite Boring Profile



NOTES: This figure depicts the capacities from the ground surface at the time of our study. Potential bridge loads were not available for consideration.

Figure 16
TxDOT Geotechnical Manual Table 5-1

Table 5-1: Maximum Allowable Drilled Shaft Service (1)

Size (in.)	Load (tons)
24	175
30	275
36	400
42	525
48	700
54	900
60	1,100
66	1,350
72	1,600
84	2,175
96	2,850
108	3,625
120	4,475

(1) *TxDOT Geotechnical Manual*, Chapter 5, Section 3, Dated July 13, 2020
http://onlinemanuals.txdot.gov/TxDOTOnlineManuals/TxDOTManuals/geo/manual_notice.html

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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