RABA KISTNER

GEOTECHNICAL ENGINEERING STUDY

FOR

PRECINCT 30 – UNITS 2 AND 3
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS

RABA KISTNER
Raba Kistner, Inc.
1913 Post Road, Suite 645
New Braunfels, TX 78130
www.rkci.com

P 830.214.0544
F 830.214.0627
F-3257

Project No. ANA24-019-00
July 26, 2024

Mr. Garrett Mechler
Vice President of Operations
ASA Properties, LLC
387 West Mill Street
New Braunfels, Texas 78130

c/o Mr. Greg Latimer
Project Manager
Pape-Dawson Engineers
1672 Independence Drive, Suite 102
New Braunfels, Texas 78132

RE: Geotechnical Engineering Study
Precinct 30 – Units 2 and 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. PNA24-032-00, dated May 14, 2024. The purpose of this study was to drill borings within the proposed roadways, detention ponds and retaining walls, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the proposed retaining wall, as well as to provide pavement design and construction guidelines.

The following report contains our design recommendations and considerations based on our current understanding of the project information provided to our office. There may be alternatives for value engineering of the foundation and pavement systems, and RKI recommends that a meeting be held with the Owner and design team to evaluate these alternatives.

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.

Ian Perez

TERRANCE IAN PEREZ
101225

Santosh Shrestha, E.I.T.
Graduate Engineer

Ian Perez, P.E.
Vice President

SS/TIP/alg
Attachments
Copies Submitted: Above (1) – Email Only

GEOTECHNICAL ENGINEERING STUDY

For

PRECINCT 30 – UNITS 2 AND 3
VERAMENDI MASTER PLANNED DEVELOPMENT
NEW BRAUNFELS, TEXAS

Prepared for

ASA PROPERTIES, LLC
New Braunfels, Texas

Prepared by

RABA KISTNER, INC.
New Braunfels, Texas

PROJECT NO. ANA24-019-00

July 26, 2024

RABAKISTNER

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ATTACHMENTS

The following figures are attached and complete this report:

Boring Location Map .... Figure 1
Logs of Borings .... Figures 2 through 11
Key to Terms and Symbols .... Figure 12
Results of Soil Analyses .... Figure 13
Moisture-Density Relationship .... Figure 14
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Important Information About Your Geotechnical Engineering Report

INTRODUCTION

RABA KISTNER Inc. (RKI) has completed the authorized subsurface exploration and foundation analysis for the proposed retaining walls as well as the interior roadways within Precinct 30, Units 2 and 3 of the Veramendi Master Planned Development in New Braunfels, Texas as illustrated on Figure 1. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations for retaining walls, as well as for pavement design and construction guidelines.

PROJECT DESCRIPTION

To be considered in this study are the interior roadways within Precinct 30, Units 2 and 3 of the Veramendi Master Planned Development in New Braunfels, Texas. The interior roadways are to be designed in general accordance with the City of San Antonio Pavement design guidance for Local Type A Streets (with and without bus traffic) and Local Type B Streets along with guidance from the City of New Braunfels.

In addition, we understand that there will be 2 detention ponds each with 5 ft berms which will be located on the northeast and east side of Unit 3. We also understand that multiple walls are planned at the site and includes approximately 11 ft cut walls to 20 ft fill walls planned within Unit 3.

Our understanding of the retaining walls at this site is based on the drawings provided to us by Rampart Engineering, titled “Wall Plans and Profile Views”, Sheet No. RW3 through RW11, dated June 2024. Additionally, our understanding of the existing topography and proposed site grading at this site is based on a drawing provided to us by Pape Dawson Engineers, Inc., titled “Overall Grading Plan”, dated May 2024. Based on this drawing, the topographic high and low within Units 2 and 3 are estimated to be 805.0 and 705.0 ft, respectively.

LIMITATIONS

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

The recommendations submitted in this report are based on the data obtained from 10 borings drilled at this site, 2 surficial bulk samples, and our understanding of the project information provided to us. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. This is particularly true with respect to the depth of the surficial soils, the depth to the top of the limestone, and the potential presence of karstic features. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

The scope of our Geotechnical Engineering Study does not include an environmental assessment of the air, soil, rock, or water conditions either on or adjacent to the site. No environmental opinions are presented in this report.

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

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by 10 borings drilled at the locations shown on the Boring Location Map, Figure 1. These locations are approximate, and distances were measured using a recreational grade, hand-held, GPS Locator. The borings were drilled using a truck-mounted drilling rig to depths below the existing ground surface ranging from approximately 10 to 30 ft.

During drilling operations, split-spoon samples with Standard Penetration Testing (SPT) were collected, and where split-spoon samples achieved little to no recovery, supplemental grab samples of auger cuttings were collected. Each sample was visually classified in the laboratory by a member of our geotechnical engineering staff. The geotechnical engineering properties of the strata were evaluated by natural moisture content testing, Atterberg limits determinations, and grain size analyses (percent passing No. 200 sieve).

The results of all laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 11. A key to classification terms and symbols used on the logs is presented on Figure 12. The results of the laboratory and field testing are also tabulated on Figure 13 for ease of reference.

Standard Penetration Test results are noted as “blows per ft” on the boring logs and Figure 13, where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft of penetration into the soil/weak rock (N-value). 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” for 6 in. or less will be noted on the boring logs and on Figure 13.

In addition to the above listed testing and sampling, 2 bulk samples from the surficial soils were collected for use in CBR testing, pH-Lime Series testing, and sulfate content testing. Additional testing on the bulk sample includes Atterberg Limits determination prepared with the percent lime content determined from the pH-Lime Series results (requires a lime content that achieves a pH of 12.4 or higher). The results of the CBR tests are presented on the Moisture-Density Relationship Curve presented on Figure 14. The results of the pH-lime series test are presented on the pH-Lime Series Curve on Figure 15 and the Dry Density vs. CBR is presented graphically on Figure 16. A summary of the bulk sample testing results is presented in the following table:

Material Type	Location	Depth (ft) [(1)]	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Corrected Laboratory CBR	Average Percent Swell (%)	Raw PI	PI with 5% Lime
Dark Brown Clay	100 ft West of P-1	0 – 1.5	83.1	28.1	4.3	2.1	37	11
Dark Brown Clay	P-3	0 – 1.5	87.9	25.6	4.1	1.8	43	7

(1) From the existing ground surface at the time of this study

Dynamic Cone Penetrometer (DCP) tests were also performed at select boring locations near proposed pavement areas from the existing ground surface to approximately 2 ft or practical equipment refusal and are the results are presented on Figure 17.

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

SULFATE TESTING

Sulfate testing was performed on bulk samples collected. The results of the sulfate content tests are 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)
Dark Brown Clay	B-4	0 – 1.5	Less than 100
Dark Brown and Tan Clay	B-6	2.5 – 4	Less than 100
Dark Brown Clay	P-1	0 – 1.5	Less than 100
Dark Brown Clay	P-2	0 – 1.5	Less than 100
Dark Brown Clay	P-3	0 – 1.5	Less than 100
Dark Brown Clay	P-4	0 – 1.5	Less than 100

On the basis of soil sulfate concentration, the soils at this site have a "*Negligible*" potential to cause sulfate induced heave. 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.

GENERAL SITE CONDITIONS

GEOLOGY

A review of the *Geologic Atlas of Texas, San Antonio Sheet*, indicates that this site is naturally underlain with the soils/rock (limestone) of the Edwards Group and potentially the Buda Limestone Formation. The characteristics of each formation are discussed below.

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.

The Buda Limestone can range from a relatively hard, thick, massive limestone to a thin "cap" of extremely weathered, soft, blocky limestone underlain by Del Rio Clay. Del Rio Clay is a tan and gray, bentonitic, highly expansive clay. The formation is typically hard, fine grained and poorly bedded. Bioclastic, commonly glauconitic, pyritiferous with abundant pelecypod bivalve fossils. Weathered layers may be filled with chalky marl.

Key geotechnical engineering concerns for development supported on this formation are the thickness of the Buda Limestone, expansive, soil-related movements resulting from the underlying Del Rio Clay, and groundwater issues. Surface water generally percolates downward through fractures in the Buda Limestone formation and through the weathered zones of the Del Rio clay until relatively unweathered clay is encountered, whereupon groundwater is then diverted laterally. Where the contact between the Del Rio clay and the Buda Limestone intersects the ground surface, typically occurring on hillsides or hilly topographies, transient water seepage will often "daylight" at the ground surface.

SEISMIC CONSIDERATIONS

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

- Site Class Definition: **Class C**. Based on the soil borings conducted for this investigation and our experience in the area, the upper 100 ft of soil may be characterized as very dense soil and soft rock.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% Of Critical Damping): $\mathbf{S_s = 0.051g}$.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% Of Critical Damping): $\mathbf{S_1 = 0.027g}$.
- Values of Site Coefficient: $\mathbf{F_a = 1.3}$
- Values of Site Coefficient: $\mathbf{F_v = 1.5}$
- Where g is the acceleration due to gravity.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec, adjusted: **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 at this site can generally be described as a thin veneer of moderately plastic to highly plastic, dark brown or dark brown and tan clay with plasticity indices ranging from 9 to 54 overlying tan and reddish tan clay with limestone fragments (possibly weathered limestone), if any, that transitions to tan or tan and gray limestone. The soil overburden thickness varies from approximately 0.5 to 3.5 ft. below the existing ground surface. The site is located in an area known to have karst topography (i.e. open and/or clay-filled vugs, voids, and/or vertical/horizontal solution cavities in the bedrock). Hence, there is a potential to encounter these features between the borings drilled at this site. Considerable variation in the top of rock elevation, quality, and quantity of rock excavation should be anticipated.

Each stratum presented on the boring logs has been designated by grouping materials 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 presented 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 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, particularly at the clay/limestone interface, within weathered seams or karst features, and following periods of precipitation. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

FOUNDATION ANALYSIS

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). A PVR value of 1 in. or less was estimated for the stratigraphic conditions encountered in our borings for the subject roadways, the detention pond near B-4, and the subject retaining walls whereas a PVR value of 2-1/4 was estimated for the detention pond near Boring B-6. A surcharge load of 1 psi (concrete slab and sand layer), an active zone of 15 ft or top of the bedrock, and dry moisture conditions were assumed in estimating the above PVR values.

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

MITIGATION OF EXPANSIVE SOIL-RELATED MOVEMENTS

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

FOUNDATION RECOMMENDATIONS FOR RETAINING WALLS

SHALLOW FOUNDATIONS FOR RETAINING WALLS

The proposed retaining walls 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 structures.

Allowable Bearing Capacity

Shallow foundations founded on natural soil or compacted select fill may be proportioned using the design parameters tabulated below. The select fill should be placed in accordance with the *Select Fill* section of this report, respectively.

Shallow Foundation Design Parameters	
Minimum depth below final grade	18 in. (1)
Minimum strip footing width	12 in.
Minimum spread footing/widened beam width	18 in.

(1) If intact bedrock is encountered, the minimum foundation depth should be discussed with the structural engineer.

Shallow Foundation Type	Maximum Allowable Bearing Pressure	
	Select Fill	Bedrock
Strip footings	3,000 psf	4,000 psf
Widened beams or spread footings	3,500 psf	4,500 psf

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

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

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

Uplift Resistance

Resistance to vertical force (uplift) is provided by the weight of the concrete footing plus the weight of the soil directly above the footing. For this site, it is recommended that the ultimate uplift resistance be based on total unit weights for soil and concrete of 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 shallow foundations will be resisted by passive earth pressure acting on one side of the footing and by base friction of the footings in select fill or bedrock. Resistance to sliding for foundations bearing on select fill may be calculated utilizing an ultimate coefficient of friction of 0.35 (alternative select fill), 0.50 (granular select fill), or 0.70 (bedrock) (this value may be applied to the allowable contact pressure, calculated using a factor of safety of 3). An equivalent fluid pressure of 250 pcf (soil/fill) should be utilized to determine the ultimate passive resistance, if required (factor of safety 2 or more should be applied).

RETAINING STRUCTURES

Retaining walls are anticipated to account for grade changes across the site. Global stability analyses have been performed and the results are presented on Figure 18 and are discussed in a subsequent section. The following sections provide general information for evaluating lateral earth pressures, backfill compaction, drainage, and the footings for the walls.

LATERAL EARTH PRESSURES

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

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

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

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

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

DRAINAGE

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

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

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

BACKFILL PLACEMENT AND COMPACTION

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

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed and compacted as recommended in the *Select Fill* section of this report. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the wall backside, we recommend that compaction be accomplished by using thinner fill lifts and using hand-guided compaction equipment capable of achieving the maximum density in a series of 3 to 5 passes.

RETAINING WALL FOOTINGS

The proposed retaining wall may be supported on compacted, select fill or intact limestone at a minimum depth of 1.5 ft below existing grade, or a minimum of 1.5 ft below final grade, whichever is deeper. Footings may be designed using the parameters provided in the section titled *Shallow Foundations.* There is a potential that the retaining walls may partially bear on soil and others on rock. Where the soil/rock transitions occur, there is an increased potential for differential settlement. To reduce the potential for differential settlement at these transitions, we recommend extending the retaining wall foundations down to similar bearing material.

GLOBAL STABILITY

Global stability analysis consists of comparing the sliding and restraining forces along a possible slide plane and calculating the factor of safety. Gravity (i.e. soil weight, water in the slope, and surcharge) provides the driving force while shear strength of the soil provides the restraining force. The accepted standard in local practice is to have an approximate factor of safety of 1.5 or greater for long-term stability of a retaining wall. The acceptable factor of safety selected for design depends on the reliability of available subsurface data, soil strength information, and the consequences of failure.

We understand that the proposed walls will have maximum wall heights of approximately 20 ft. We performed our global stability analysis for the effective stress condition (long-term condition). The computer program SLIDE was used to perform the computations. A surface surcharge of 175 pounds per square foot (psf) was used in the analysis to account for load transferred through beams on grade along the length of the building. Spencer's method of modeling the profile with non-circular sliding surface was considered in the analysis. General Consistency and density of the soil was obtained using the correlation of the field SPT data, and prior experience with similar project sites.

The following table presents the assumed soil properties for drained conditions. Figure 18 of the Attachments also presents the soil properties and wall properties assumed in our analyses.

Soil	Total Unit Weight, pcf	Cohesion, psf	Angle of Internal Friction, degrees
Select Fill	125	50	35
Limestone	155	500	40
Tan Clay	120	50	28
MSE Wall	145	-	-
Stone Gravity Wall	150	-	-
Dark Brown Clay	120	50	22

The performance of global stability analyses involves the selection of a variety of assumptions regarding likely modes of failure, external loads, and construction conditions. Non-circular, or general, failure surfaces were used to perform the final evaluation of global stability for the proposed retaining walls. General failure surfaces differ from circular surfaces in that a specific form of global stability is not assumed before the automated search for the global minimum factor of safety is undertaken. Various global failure modes can result from this type of analysis, including circular, block, wedge, translational, and combination failure surfaces. The computer selects varied starting and ending points for a large number of trial surfaces and chooses an initial failure surface from that initial geometry.

In these analyses, we chose to use a large number of initial trial surfaces and had Slide use an optimization scheme on each assumed failure surface to estimate the local minimum calculated factor of safety. The program then presents the failure surfaces with the minimum calculated factor of safety, which is presented on Figure 18. Presented in the following table, are our calculated factors of safety for the above-mentioned scenarios of proposed retaining wall.

Structure	Exp. Wall Height (ft)	Calculated Global Stability Factor of Safety	Target Factor of Safety
MSE Retaining Wall (Cut Condition)	11.2	1.5	1.5
Stone Gravity Retaining Wall (Fill Condition)	20.0	1.8	1.5

We have reviewed the results of the slide analyses and believe that the calculated failure surfaces are kinematically permissible (i.e. could reasonably occur) and the associated calculated factors of safety are within the range we would expect. On the basis of our analysis, the factor of safety greater than or equals to 1.5 will be achieved if stone gravity wall bearing on the limestone or MSE wall is used for the proposed wall, as depicted in Figure 18. However, we recommend moving the foundation of the building near the retaining wall further away from the wall to maintain wall stability and to reduce the potential for compromising the wall system with the proposed building foundations. **At a minimum, we recommend that structurally sensitive elements be located a distance equal to the height of the wall, away from the wall. Retaining walls should be designed by the wall designer.** Backfill should be select fill as presented in the *Select Fill* section of this report.

FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

Drainage is an important key to the successful performance of any foundation. Good surface drainage should be established prior to and maintained after construction to help prevent water from ponding within or adjacent to the foundation and to facilitate rapid drainage away from the foundation. Failure to provide positive drainage away from the structure can result in localized differential vertical movements in soil supported foundations and floor slabs.

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

SITE PREPARATION

All the areas to support select fill should be stripped of all vegetation, organic topsoil, existing fill, if any, pavements, utilities and associated backfill. It will be critical to plug any utilities and associated utility backfill that extend into the overexcavation to reduce the chances of shallow moisture migration into the select fill pad materials following construction of these improvements.

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

Upon completion of the proofrolling operations and just prior to fill placement or slab construction, the exposed subgrade should be moisture conditioned by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum dry 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.

SELECT FILL

Materials used as select fill preferably should be crushed stone or gravel aggregate. Recommendations for 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 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.
>
> **Low PI Materials** – Low PI materials should consist of CL clays, 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 these 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 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 such as Granular Pit Run or Low PI 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.

Granular Pit Run or Low PI Materials 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.

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.

FILL PLACEMENT AND COMPACTION

Select Fill

Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test, or 98 percent of maximum density as determined by ASTM D698. If fill materials supporting movement sensitive structures are placed that are 8 ft or thicker, we recommend that ASTM D1557 Modified Compaction Test be utilized in lieu of the above compaction methods. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction for imported crushed limestone base. For low PI and granular pit-run materials, 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.

General Fill

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

EXCAVATIONS AND TEMPORARY SLOPES

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

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

The contractor should be aware that excavation depths and inclinations (including adjacent existing slopes) should not exceed those specified in local, state, or federal safety regulations, e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations. Such regulations are strictly enforced and, if not followed, the contractor, or earthwork or utility subcontractors could be subjected to substantial penalties. Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations.

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

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

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

EXCAVATION EQUIPMENT

Please note that limestone bedrock was encountered in our boring at relatively shallow depths below the existing ground surface. Therefore, excavations at this site will require removal of the underlying rock formation. The Edwards limestone is hard to very hard in induration, is massive, and commonly contains chert seams. Consequently, excavations penetrating the rock will encounter hard to very hard materials and may be difficult to remove in narrow trenches or footing excavations. Excavation costs should anticipate hard rock excavation for preliminary planning and construction budget. Our boring 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 earthwork and utility contractors interested in bidding on the work perform their own tests in the form of test pits to determine the quantities of the different materials to be excavated, as well as the preferred excavation methods and equipment for this site.

UTILITIES

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

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

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

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized, and all backfilling procedures should be tested and documented. Trench backfill materials should be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E or Tex-114-E, Compaction Test.
- The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content for non-

cohesive soils and maintained within the range of optimum to 3 percentage points above optimum moisture content for cohesive soils until final compaction.

- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

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.

Swell/Heave Potential

The surficial, dark brown subgrade soils at this site are classified as plastic to highly plastic, and the potential exists for the soils to expand or heave when water is introduced, causing the pavement to become rough or uneven over time. Pavement roughness is generally defined as an expression of irregularities in the pavement surface that adversely affect the ride quality of a vehicle (and thus the user). Roughness is an important pavement characteristic because it affects not only ride quality but also fuel consumption as well as vehicle maintenance costs. Pavement heave can be reduced through various measures but cannot be totally eliminated without full removal of the problematic soil. Measures available for reducing heave include:

- Soil Treatment with Lime or Other Chemicals (using the modified method of treatment)
- Removal and Replacement of Moderate to High PI Soils
- Drains or Barriers to Collect or Inhibit Moisture Infiltration

Soil treatment with lime (or other chemicals) is typically used to reduce the swelling potential of the upper portion of the pavement subgrade containing moderately plastic soils. Lime and water are mixed with the top 6 to 12 inches (or possibly more) of the subgrade and allowed to mellow or cure for a period of time. After mellowing the soil-lime mixture is compacted to form a strong soil matrix that can improve pavement performance and potentially reduce soil heave. However, in highly plastic soils, lime treatment of only the top portion of the expansive subgrade may not provide an acceptable reduction in PVR. For a more substantial reduction in PVR, removal and replacement of the high PI soil may be the only method available to reduce the potential vertical rise of the pavement to an acceptable level. As stated previously, it must be recognized that partial removal of expansive clay soil only reduces the potential (or risk) of the damage swell can cause to a pavement and does not completely eliminate this risk.

In addition, capturing water infiltration via French drains, pavement edge drains, or inhibiting water through the use of vertical moisture barriers would reduce the potential for heave since one important component of the heaving mechanism, water, would be reduced. Geocomposite membranes, like geogrids, are also another tool available that may help reduce the damage that heaving subgrades cause to flexible pavements and may be considered in addition to or as an alternative to other mitigation techniques.

It should be noted that the pavement sections derived in the following sections are structurally adequate for the given traffic levels and existing clay subgrade strength, but do not consider the long-term effects of pavement roughness due to heave, which can only be addressed by the measures discussed in this section.

DESIGN PARAMETERS – ASPHALT PAVEMENTS

The roadways to be considered in this study are the interior roadways in Veramendi Precinct 30, Units 2 and 3. The proposed roadways are to be evaluated in accordance with the City of San Antonio Pavement design guidance for Local Type A Streets (with and without bus traffic) and Local Type B Streets along with guidance from the City of New Braunfels. 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
Local Type A without Bus Traffic	100,000	70	4.2/2.0	0.45	2.02/3.18
Local Type A with Bus Traffic	1,000,000	70	4.2/2.0	0.45	2.58/4.20
Local Type B	2,000,000	90	4.2/2.0	0.45	2.92/5.08

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 either consist of recompacted on-site clays, select fill or rock subgrade (as discussed further in this section). Bulk samples from the surficial soils were collected from the vicinity of Borings P-1 and P-3 for use in CBR testing. The CBR was measured using ASTM D 1883, *Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils* and was determined using the soaked sample methodology. Swell was also measured as part of the CBR procedure. The corrected CBR values and their associated borings are tabulated below:

Material Type	Sample Location	Average Swell (%)	Laboratory CBR	Design CBR
Dark Brown Clay	100 ft West of P-1	2.1	4.3	4.0 (1)
Dark Brown Clay	P-3	1.8	4.1	

(1) Based on CBR test results, DCPs and our experience with similar soil conditions.

These values were 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 results are presented on Figure 14. Based on these CBR results, DCP and our experience with the soils in this area, we have assumed a design CBR value of 4.0 for use in our pavement section analysis for the clay fill subgrade (hereafter referred to as the ‘clay subgrade’). 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 4.0. 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.

A ‘rock subgrade’ condition with a CBR of 10.0 may be utilized for the following conditions:

- If select fill material, in accordance with the *Select Fill* section of this report, is utilized as the subgrade fill from bedrock up to the bottom of the pavement section elevation;
- If native, intact rock is exposed prior to select fill placement (if necessary); or
- If 2 ft or less of surficial on-site clays remain.

For areas that transition between a clay and rock subgrade, we recommend that geogrid be utilized to relieve stress concentrations at the subgrade transitions. The geogrid should be used as a transition for 5 ft or greater on either side of the transition.

STRUCTURAL NUMBER RECOMMENDATIONS

Structural numbers for each street classification and each 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.

PAVEMENT DESIGN PARAMETERS – ASPHALT PAVEMENTS

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:

$M_r = 1{,}500 \times CBR$

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 – Asphalt 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 values on the table presented in the *Design Parameters – Asphalt 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 – Asphalt Pavements* section of this report.

RECOMMENDED PAVEMENT SECTIONS – ASPHALT PAVEMENTS

Appendix 10-A of the *City of San Antonio's Design Guidance Manual* states that 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 upper 5 ft of our borings, the PI of the surficial subgrade clays ranges from 9 to 54. We recommend that pavements on a dark brown clay subgrade at this site include a minimum of 6 in. of treated subgrade. On the basis of our testing, the tan clays encountered at this site were generally low plasticity to non-plastic and will not require subgrade treatment, if exposed prior to flexible base placement or when used as fill to achieve the finished grades. We recommend that the required lime/cement content reduces 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 using dark brown clay, the final 6 in. of fill should be lime/cement treated (see *Treated Subgrade*). If fill grading is not planned and clays remain in-place, then treatment of the stripped clay subgrade should be performed in conjunction with the scarifying, moisture conditioning, and recompaction process described in the *Site Preparation* section of the *Construction Considerations*.

If fill grading is completed utilizing select fill in accordance with the *Select Fill* section of this report, or if native, intact rock is exposed prior to fill placement (if necessary), the treated subgrade may be eliminated from the pavement section and the rock subgrade recommendations should be utilized. Per Appendix 10-A of the *City of San Antonio's Design Guidance Manual*, a rock credit can be given to those pavement sections overlying a rock subgrade. The rock credit is equivalent to a 6 in. structural layer for stabilized subgrade.

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

Clay Subgrade

Local Type A without Bus Traffic; CBR=4.0; Required SN = 2.23	Layer Description	Layer Thickness	Recommended SN Coeff.	SN Extension
Flexible Base Option	Type C or D Surface Course	2.5 in.	0.44	1.10
	Flexible (Granular) Base	9.0 in.	0.14	1.26
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**17.5 in.**		**2.36**
Full Depth Asphalt Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Type B Base Course	6.0 in.	0.38	2.28
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**14.0 in.**		**3.16**
Mechanically Stabilized Layer Option	Type C or D Surface Course	2.5 in.	0.44	1.10
	Mechanically Stabilized Layer	7.0 in.	0.17	1.19
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**15.5 in.**		**2.29**

Local Type A with Bus Traffic; CBR=4.0; Required SN = 3.19	Layer Description	Layer Thickness	Recommended SN Coeff.	SN Extension
Flexible Base Option	Type C or D Surface Course	1.5 in.	0.44	0.66
	Type C Binder Course	2.0 in.	0.44	0.88
	Flexible (Granular) Base	12.0 in.	0.14	1.68
	Treated Subgrade (1)	6.0 in.	0.00	0.00
	Combined Total	**21.5 in.**		**3.22**
Full Depth Asphalt Option	Type C or D Surface Course	2.5 in.	0.44	1.10
	Type B Base Course	6.0 in.	0.38	2.28
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**14.5 in.**		**3.38**
Mechanically Stabilized Layer Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Type C Binder Course	2.0 in.	0.44	0.88
	Mechanically Stabilized Layer	9.0 in.	0.17	1.53
	Treated Subgrade (1)	6.0 in.	0.00	0.00
	Combined Total	**19.0 in.**		**3.29**

Local Type B; CBR=4.0; Required SN = 3.97	Layer Description	Layer Thickness	Recommended SN Coeff.	SN Extension
Flexible Base Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Type C Binder Course	2.5 in.	0.44	1.10
	Flexible (Granular) Base	15.0 in.	0.14	2.10
	Treated Subgrade (1)	6.0 in.	0.00	0.00
	Combined Total	**25.5 in.**		**4.08**
Full Depth Asphalt Option	Type C or D Surface Course	3.0 in.	0.44	1.32
	Type B Base Course	7.0 in.	0.38	2.66
	Treated Subgrade	6.0 in.	0.00	0.00
	Combined Total	**16.0 in.**		**3.98**
Mechanically Stabilized Layer Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Type C Binder Course	2.5 in.	0.44	1.10
	Mechanically Stabilized Layer	12.0 in.	0.17	2.04
	Treated Subgrade (1)	6.0 in.	0.00	0.00
	Combined Total	**22.5 in.**		**4.02**

Rock Subgrade

Local Type A without Bus Traffic; CBR=10.0; Required SN = 2.02(1)	Layer Description	Layer Thickness	Recommended SN Coeff.	SN Extension
Flexible Base Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Flexible (Granular) Base	6.0 in.	0.14	0.84
	Rock Credit	0.0 in.	--	0.48
	Combined Total	**8.0 in.**		**2.20**

(1) The calculated Structural Number (SN) was less than the COSA minimum SN, and the COSA minimum was utilized in design.

Local Type A with Bus Traffic; CBR=10.0; Required SN = 2.58(1)	Layer Description	Layer Thickness	Recommended SN Coeff.	SN Extension
Flexible Base Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Flexible (Granular) Base	9.0 in.	0.14	1.26
	Rock Credit	0.0 in.	--	0.48
	Combined Total	**11.0 in.**		**2.62**
Full Depth Asphalt Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Type B Base Course	6.0 in.	0.38	2.28
	Rock Credit	0.0 in.	--	0.48
	Combined Total	**8.0 in.**		**3.64**

(1) The calculated Structural Number (SN) was less than the COSA minimum SN, and the COSA minimum was utilized in design.

Local Type B; CBR=10.0; Required SN = 2.92(1)	Layer Description	Layer Thickness	Recommended SN Coeff.	SN Extension
Flexible Base Option	Type C or D Surface Course	3.0 in.	0.44	1.32
	Flexible (Granular) Base	8.0 in.	0.14	1.12
	Rock Credit	0.0 in.	--	0.48
	Combined Total	**11.0 in.**		**2.92**
Full Depth Asphalt Option	Type C or D Surface Course	2.0 in.	0.44	0.88
	Type B Base Course	6.0 in.	0.38	2.28
	Rock Credit	0.0 in.	--	0.48
	Combined Total	**8.0 in.**		**3.64**

(1) The calculated Structural Number (SN) was less than the COSA minimum SN, and the COSA minimum was utilized in design.

The flexible base section for Local Type A with Bus and Local Type B (Clay Subgrade) options presented above meet the City of New Braunfels minimum pavement section requirements for one and two family local residential streets.

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 areas of stress concentrations. The 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 – PORTLAND CEMENT CONCRETE PAVEMENTS

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)
Local Type A without Bus Traffic	150,000	70	4.5/2.0	0.35	5.0/6.0
Local Type A with Bus Traffic	1,500,000	70	4.5/2.0	0.35	6.0/8.0
Local Type B	3,000,000	90	4.5/2.0	0.35	7.0/9.0

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."

PAVEMENT DESIGN PARAMETERS – PORTLAND CEMENT CONCRETE PAVEMENTS

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, (M_r) psi
- Effective Modulus of Subbase/Subgrade Reaction, (k-value) psi/in.
- 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 (M_r), psi

The M_r of concrete is a measure of the flexural strength of the concrete as determined by breaking concrete beam test specimens. An 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, psi

An 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 is used for this pavement design.

Effective Modulus of Subbase/Subgrade Reaction(k-value), psi/in.

Concrete slab support is characterized by the modulus of subgrade 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, a k-value of 120 psi/in. 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 – Portland Cement Concrete 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 with load transfer devices (i.e. dowels) at control joints or CRCP.

Drainage Coefficient

The drainage coefficient characterizes the quality of drainage of the subbase layers under the concrete slab. Good draining pavement structures do not give water the chance to saturate the subbase and subgrade; thus, pumping is not as likely to occur. 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 – Portland Cement Concrete 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 – Portland Cement Concrete 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 – Portland Cement Concrete Pavements* section of this report.

RECOMMENDED PAVEMENT SECTIONS – PORTLAND CEMENT CONCRETE PAVEMENTS

The recommended concrete slab thicknesses determined with the inputs discussed above are presented in the table below. We recommend that pavements on a dark brown clay subgrade at this site include a minimum of 6 in. of treated subgrade. We recommend that the required lime content reduces the PI of the subgrade soil to less than 20 and increases the pH of the soil to 12.4 or greater. If the exposed soil has a natural PI of less than 20 or is founded on a "rock subgrade", then the lime treatment may be waived.

Portland Cement Concrete Design - Cross Sections	Layer Description	Layer Thickness
Local Type A without Bus Traffic	Concrete (1) Treated Subgrade (2) **Combined Total**	5.0 in. 6.0 in. **11.0 in.**
Local Type A with Bus Traffic	Concrete (1) Treated Subgrade (2) **Combined Total**	7.0 in. 6.0 in. **13.0 in.**
Local Type B	Concrete (1) HMA Bond Breaker (2) Treated Subgrade (2) **Combined Total**	8.5 in. 1.0 in. 6.0 in. **15.5 in.**

(1) Concrete pavement should consist of continuously reinforced concrete pavement (CRCP), or jointed plain concrete pavement with load transfer devices at control joints.

(2) These layers may be waived for a rock subgrade condition.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SITE PREPARATION

Preparation for the right-of-way (for streets, sidewalks, utilities, etc.) should be performed in accordance with the 2014 TxDOT Standard Specifications, Item 100 – *Preparing Right of Way*. Exposed subgrades should be thoroughly proofrolled in order to locate any weak, compressible zones. A minimum of 5 passes of a fully loaded dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical Engineer or his representative to document subgrade condition and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with a suitable, compacted backfill.

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

Upon completion of fill grading using the on-site clays, the final 6 in. of fill should be lime/cement treated (see *Treatment of Subgrade* section). If fill grading is not planned, then lime treatment of the stripped clay subgrade should be performed in conjunction with the scarifying, moisture conditioning, and recompaction described previously.

ON-SITE CLAY FILL

We recommend that the on-site soils be placed to conform to the 2014 TxDOT Standard Specifications, Item 132 – *Embankment*, Type B, and should be placed in compacted lifts not exceeding 6 in. in thickness and compacted to the requirements of Table 2 in Item 132 based on the maximum density and optimum moisture content as determined by TxDOT, Tex-114-E. The moisture content of the fill should be maintained to be at least equal to the optimum water content, but not exceed 3 percentage points above the optimum water content until permanently covered. Fill materials shall be free of roots and other organic or degradable material. We recommend that the maximum particle size not exceed 3 in. or one half the compacted lift thickness, whichever is smaller. If other import fill materials are utilized, RKI should be notified, as additional CBR testing and thicker pavement sections may be required.

It is imperative that the subgrade modulus utilized in the pavement design process be met or exceeded by the fill material. In the event that the clay fill used is different than the existing subgrade, the recommendations in this report could be invalidated and the design engineer must be consulted to determine if additional CBR testing and thicker pavement sections are required.

TREATMENT OF SUBGRADE

Lime or cement treatment of the subgrade soils, if utilized, should be in accordance with the TxDOT Standard Specifications, Item 260 or Item 275, respectively. A sufficient quantity of hydrated lime should be mixed with the subgrade soils to reduce the soil plasticity index to 20 or less. Based on the results of the pH-Lime Series Curves, we recommend that at least 5 percent hydrated lime/cement treatment by weight be used to increase the pH of the subgrade clays to 12.4 or higher and reduce the PI to 20 or less. This percentage of lime reduced the PI of the tested sample to 6. **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.** Treated subgrade soils should be compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of optimum moisture content to 3 percentage points above the optimum moisture content as determined by Tex-113-E.

We recommend that during site grading operations, additional laboratory testing be performed to determine the concentration of soluble sulfates in the subgrade soils. If present, the sulfate in the soil may react with calcium-based stabilizers such as lime or cement. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure.

GEOGRID REINFORCEMENT

The geogrid reinforcement should be selected and placed in accordance with a Type II TxDOT approved geogrid that conforms to DMS 6240. The geogrid should be placed at the bottom of the flexible (granular) base section in all cases. An alternative to the above geogrid should not be considered without approval from RKI.

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to the 2014 TxDOT Standard Specifications, Item 247 – *Flexible Base*, Type A, Grade 1-2. The base course should be placed in lifts with a maximum compacted thickness of 8 in. (10 inches loose) and compacted to a minimum of 95 percent of the maximum density determined by Tex-113-E 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. **In our opinion, incorporating geogrid into the pavement section will enhance overall pavement performance and reduce the potential for cracking and maintenance in asphalt pavements.** The geogrid reinforcement should conform to TxDOT Type 2 geogrid, or an approved substitute.

CEMENT TREATED BASE COURSE

The cement treated base course should conform to TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 275 or 276. In our experience, cement percentages typically range from 2 to 5 percent, but should be verified with laboratory testing. For estimating purposes, we estimate 4% cement be included. We recommend microcracking be performed approximately 1 - 3 days after placement.

PRIME COAT

A prime coat should be placed on top of the flexible base course (if used) and should be a MC-30, AE-P, EAP&T, or PCE conforming to the 2014 TxDOT Standard Specifications, Item 310 – *Prime Coat* or Item 314 – *Emulsified Asphalt Treatment* as well as Item 300 – *Asphalts, Oils and Emulsions*. Prime coat application rates are typically between 0.1 to 0.3 gal/yd^2 and are generally dependent upon the absorption rate of the granular base and other environmental conditions at the time of placement. The prime coat layer should be placed on the prepared flexible base as soon as possible. This will facilitate plugging the capillary voids in the flexible base surface to reduce migration of moisture and providing a water-resistant surface. The asphalt layer should be placed as soon as possible after the prime coat has been properly set/cured.

TACK COAT

A tack coat should be placed between asphaltic concrete base and/or surface lifts and should be a PG binder with a minimum high-temperature grade of PG 58, SS-1H, CSS-1H, or EAP&T conforming to the 2014 TxDOT Standard Specifications, Item 300 – *Asphalts, Oils and Emulsions*. See additional requirements for tack coats in the appropriate TxDOT Standard Specifications for Asphaltic Concrete Materials.

ASPHALTIC CONCRETE SURFACE AND/OR BINDER[1] COURSES

The asphaltic concrete surface and/or binder courses should conform to the 2014 TxDOT Standard Specifications, Item 341 – *Dense Graded Hot Mix Asphalt or Item 341 Paragraph 2.6.2 Warm Mix Asphalt (WMA)*, Types C or D for the surface and binder, and Type B for the base, if the full depth asphalt section is selected for construction. Recycled asphalt pavement (RAP) should be limited to 20 percent of the total weight of the mix for Types C and D mixes and 30 percent for Type B mixes. Higher percentages of RAP may be permissible depending on the material source. If higher percentages of RAP are desired, contact RKI for consideration. Asphalt cement grades should conform to the table shown below, which conforms to the requirements of Item 341.

	Minimum PG Asphalt Cement Grade		
Street Classifications	**Surface Courses**	**Binder & Level Up Courses**	**Base Courses**
Local Type B Streets	PG 70-22	PG 70-22	PG 64-22
Local Type A Street with Bus Traffic		PG 64-22	
Local Type A Street Without Bus Traffic	PG 64-22		

The asphaltic concrete should be compacted on the roadway to contain from 5 to 9 percent air voids computed using 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.

It is recommended that the hot mix asphalt concrete pavement be placed with a paving machine only and not with a motor grader unless prior approval is granted by the Engineer for special circumstances. The asphalt layer should preferably be placed as soon as possible after the flexible base has been accepted and the prime coat has been placed. This will further protect the flexible base and subgrade from undue moisture fluctuation due to precipitation or sheet flow from rain events.

ASPHALT BOND BREAKER

The hot-mix asphalt bond breaker should be in accordance with the TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 340, Dense-Graded Hot-Mix Asphalt (Small Quantity), Type D, a Performance Graded Binder 76-22 (PG-76-22) and designed with a laboratory density target of 97.5 percent.

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[1] A binder course is defined as the hot mixed asphalt concrete (HMAC) layer placed directly beneath the HMAC surface or wearing course but is not an asphalt treated base layer.

PORTLAND CEMENT CONCRETE

The Portland cement concrete should be in accordance with Class P concrete of the TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 421, Portland Cement Concrete. Requirements include concrete designed to meet a minimum average compressive strength of 3,500 psi at 7-days or a minimum average compressive strength of 4,400 psi at 28-days in accordance with TxDOT standard laboratory test procedure Tex-448-A or Tex-418-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.

CONCRETE PAVEMENT CONSTRUCTION CONTROL

Construction of Portland Cement Concrete Pavements should be controlled by the 2014 TxDOT Standard Specifications, Item 341 – *Concrete Pavement*. The surface of all concrete pavements should be textured or tined. Texturing using carpet dragging or tining should be in accordance with Item 360, Sections 3.4.1 and 3.4.2. Other texturing techniques may be utilized as described in ACI 330.1-03, Section 3, Subparagraph 9.

CONCRETE PAVEMENT TYPE

Jointed Plain Concrete Pavement (which is referred to by TxDOT as Concrete Pavement Contraction Design or CPCD) is suggested for roadways with crosswalks, adjacent parking, or sidewalks and is recommended as the pavement type for this city street.

JOINT SPACING AND DETAILS

Construction joint spacing should not exceed 15 ft in either the longitudinal or transverse direction. The depth of sawcut should be a minimum of 1/4 of the slab depth if utilizing a conventional saw or 1 in. when using an early entry saw (early entry sawing is recommended). The width of the joint will be a function of the sealant chosen to seal the joint. It is recommended that a joint seal be utilized to minimize the introduction of incompressible material into the joint.

It is recommended that dowel bars be used to provide load transfer and reduce differential movement (or faulting) across transverse joints. Dowels should be smooth #9 bars (Grade 60 steel) spaced 12 in. on center with an embedment length of at least 8 in.

Tie bars should be used to tie longitudinal joints within the pavement lanes and at the shoulder. Tie bars should be deformed #4 bars at a minimum (Grade 60 steel) spaced 36 in. on center with a minimum length of 30 in.

Isolation joints are used to separate concrete slabs from other structures or fixed objects within or abutting the paved area to offset the effects of expected differential horizontal and vertical movements. Such structures include, but are not limited to, buildings, light standard foundations, and drop inlets. Isolation joints are also used at "T" intersections to accommodate differential movement along the different axes.

Isolations joints are sometimes referred to as expansion joints. However, they are rarely needed to accommodate concrete expansion, so they are not typically recommended for use as regularly spaced joints.

We recommend a jointing layout plan be established and reviewed by all parties prior to construction. We also recommend avoiding jointing lines which create angles of less than 60 degrees, "T" joints, and interior corners.

Proper curing of the concrete pavement should be initiated immediately after finishing. All control joints should be formed or sawed to a depth of at least 1/4 the thickness of the concrete slab and should extend completely through monolithic curbs (if used). Sawing of control joints should begin as soon as the concrete will not ravel, preferably within 1 to 3 hours using an early entry saw or 4 to 8 hours with a conventional saw. Timing will be dictated by site conditions.

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

SUGGESTED PAVEMENT DETAILS

Suggested details that can be utilized for construction are:

- TxDOT CRCP (1)-20, Continuously Reinforced Concrete Pavement, One Layer Steel Bar Placement, T-7 to 13 inches;
- TxDOT CPCD-14, Concrete Pavement Details, Contraction Design, T-6 to 12 inches; and
- TxDOT JS-14, Concrete Paving Details, Joint Seals.

CONSIDERATIONS FOR TRANSITIONS FROM RIGID TO FLEXIBLE PAVEMENTS

At rigid to flexible pavement transitions, we recommend that special attention be given to designing an appropriate transition from the proposed asphalt flexible pavement to the rigid concrete pavement. This transition detail should be developed to help minimize the amount of movement at the transition and possible faulting or widening the joint. The transition may include constructing a concrete sleeper/approach slab below the flexible pavement section or using full depth asphalt pavement section adjacent to the concrete pavement to a depth equal to the sum of the asphalt and base thicknesses.

GARBAGE DUMPSTERS

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

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

FIRE LANE

Based on available literature, a 75,000-pound fire truck will impart approximately 6.9 ESALs per pass. Therefore, the proposed pavement sections provided herein shall be able to support occasional fire trucks for a design period of 20 years.

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.

Longitudinal Cracking

It should be understood that asphalt pavement sections in highly expansive soil environments, such as those encountered at this site, 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 potholes. 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. All cracks that develop in concrete pavements should be routed and sealed regularly. Joints in concrete pavements should be maintained to reduce the influx of incompressible materials that restrain joint movement and cause spalling and/or cracking. Other typical TxDOT or City of San Antonio/New Braunfels maintenance techniques should be followed as required.

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.

BERM RECOMMENDATIONS

The clays utilized in the berm for the clay liner should meet the following generally accepted specifications. If the core of the berm is required to be relatively impermeable as well, then the clay specifications below may also be used for the berm core.

Clay Specifications (1)		
Property	**Unit**	**Specification**
Permeability	cm/sec	$\leq 1 \times 10^{-7}$
Plasticity Index	-	≥ 15
Liquid Limit	-	≥ 30
% Passing (200 sieve)	%	≥ 30
Required Compaction for Liner	%	95% of TxDOT Tex-114-E or ASTM D 698
Required Compaction for Berm Core	%	90% of TxDOT Tex-114-E or ASTM D 698

(1) From *Design Guidelines for Water Quality Controls, Environmental Criteria Manual* published by City of Austin dated May 15, 2023.

Additionally, the clay soils must be free of organic matter and limestone gravel greater than 3/4 in. Per the *New Braunfels Drainage and Erosion Control Design Manual* Sec. 10.1 (N):

> "Earthen embankments of a height greater than 3 feet used to impound a required detention volume must have a minimum top width of 4 feet. Compaction of all earthen embankments shall have an impermeable core and shall be based on a geotechnical investigation of the site."

Dark brown on-site overburden clays, free of gravel greater than 3/4 in., is anticipated to meet the above criteria; however, **RKI** should be retained to test the soil intended for use in the berm and substantiate its use on site. On-site processing of the dark brown clay may be required to remove any gravel (or limestone fragments) greater than 3/4 in. diameter. If clay soils are imported for the clay liner, they must meet the specifications presented in the table above. We recommend that once the clay soil planned for use as the clay liner has been identified, that additional testing be completed to establish conformity to the specifications. Additional testing should be completed for every 5,000 cubic yards of material placed.

PERMANENT SLOPES

The stability of permanent slopes depends on many factors, including the height and geometry of the slopes, the types of soils contained in the slopes, effects of groundwater, and any surface pressures present. In general, permanent cut and fill slopes, constructed at 1V:3H (1 vertical on 3 horizontal) have been observed to perform satisfactorily. Therefore, it is our opinion that slopes should be constructed at 1V:3H or flatter. Fill slopes should be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration.

Cut slopes can be designed similar to fill slopes. However, the potential for sloughing and/or general slope failure increases with an increase in the steepness and depth of cut, particularly if low strength soil occurs in or near the base of the slope.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

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

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

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

BUDGETING FOR CONSTRUCTION TESTING

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

Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKI looks forward to the opportunity to provide continued support on this project and would welcome the opportunity to meet with the Project Team to develop both the scope and budget for these services.

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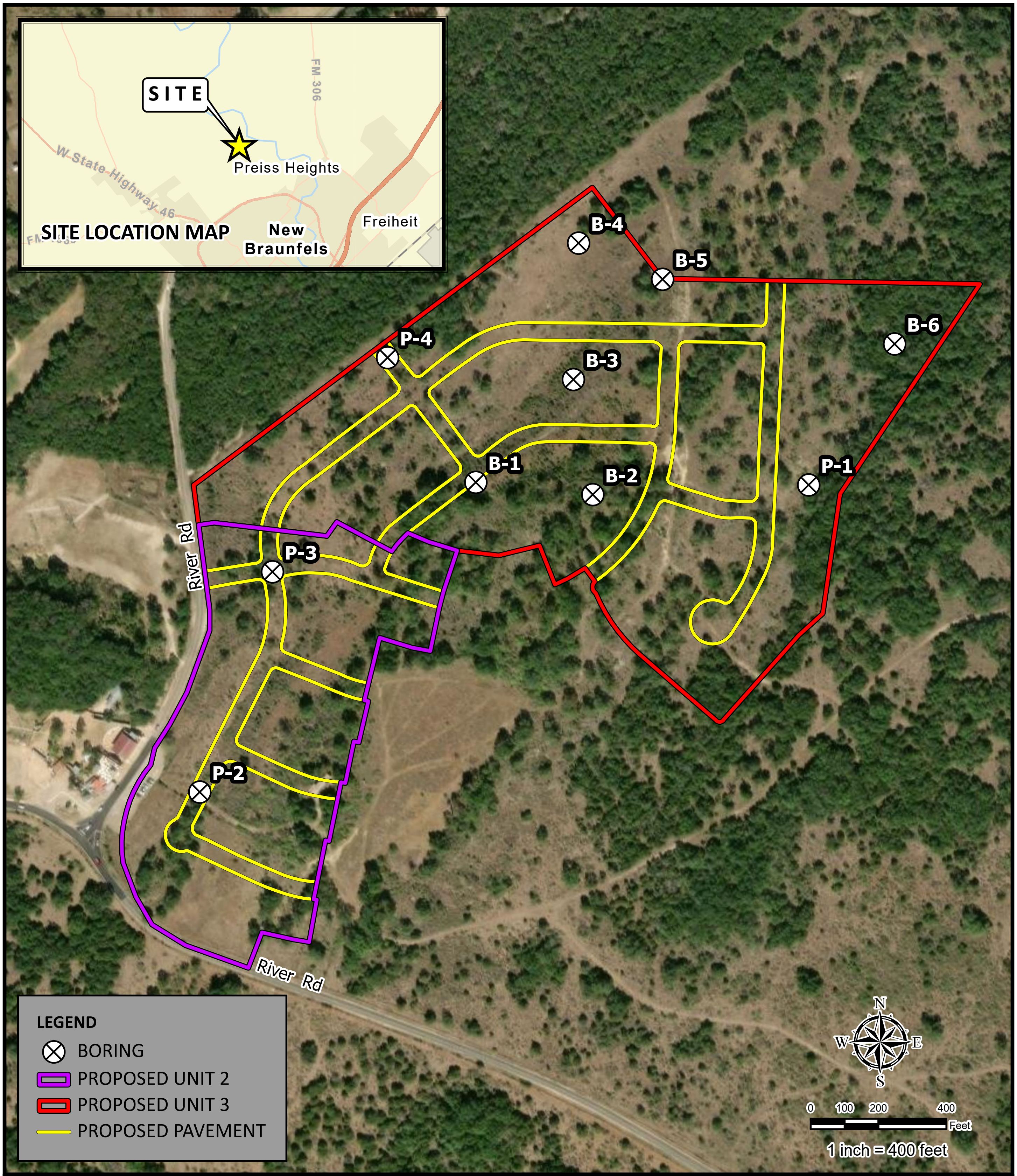
ATTACHMENTS



RABA KISTNER

211 Trade Center, Suite 300
New Braunfels, Texas 78130

(830)214-0544 TEL
(830)214-0627 FAX

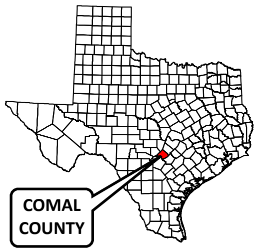
www.rkci.com

TBPE Firm Number 3257

World Imagery: Maxar
World Street Map: City of Austin, City of New Braunfels, Texas Parks & Wildlife, CONANP, Esri, TomTom, Garmin, Foursquare, SafeGraph, METI/NASA, USGS, EPA, NPS, USDA, USFWS

BORING LOCATION MAP

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas



PROJECT No.:	ANA24-019-00
ISSUE DATE:	5/24/2024
DRAWN BY:	BM
CHECKED BY:	SS
REVIEWED BY:	TIP

FIGURE

1

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

LOG OF BORING NO. B-1

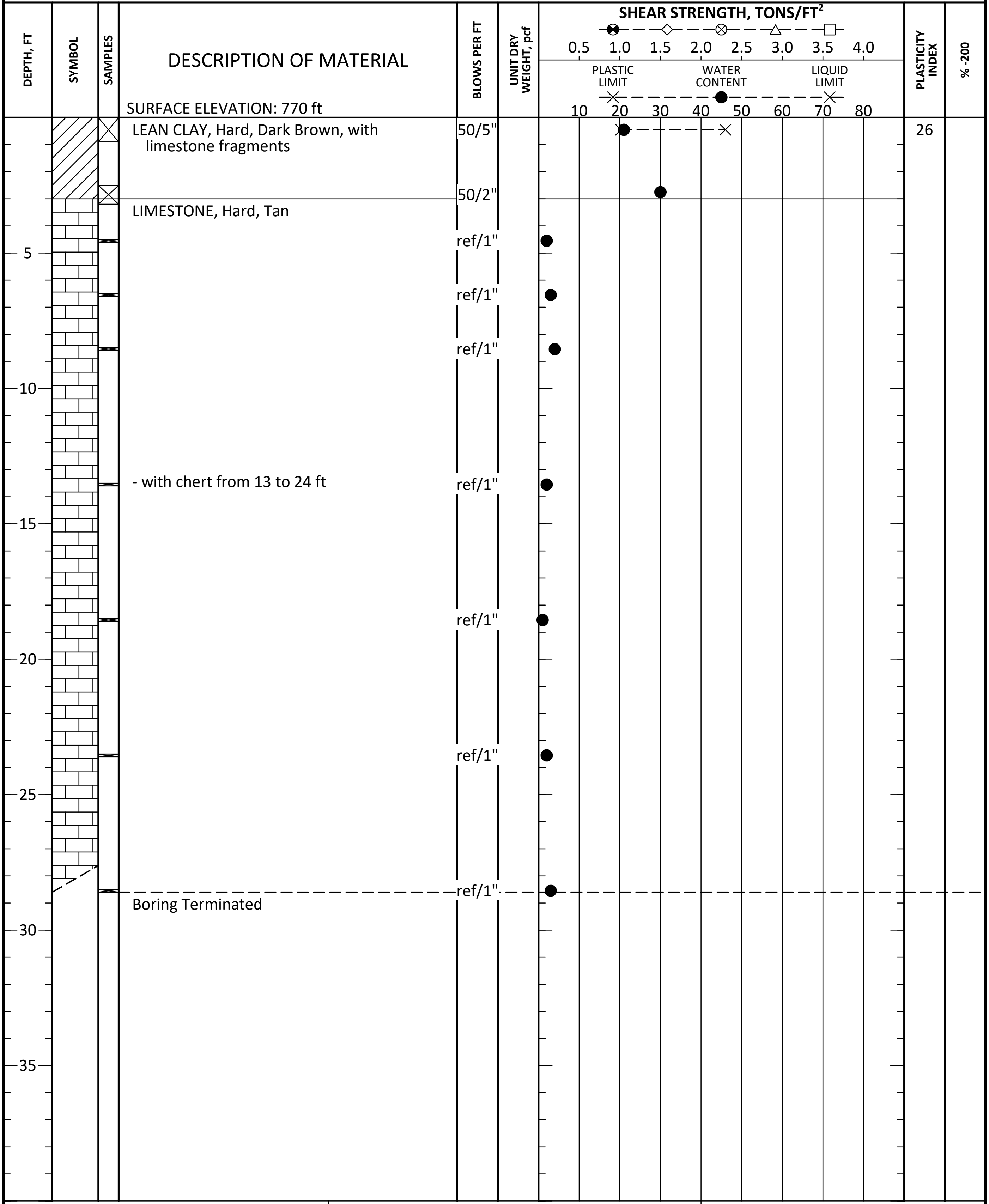
Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.75011; W 98.14177

SURFACE ELEVATION: 770 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
LEAN CLAY, Hard, Dark Brown, with limestone fragments	50/5"		26	
	50/2"			
LIMESTONE, Hard, Tan				
	ref/1"			
	ref/1"			
	ref/1"			
- with chert from 13 to 24 ft	ref/1"			
	ref/1"			
	ref/1"			
	ref/1"			
Boring Terminated				

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

DEPTH DRILLED:	28.6 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ANA24-019-00
DATE DRILLED:	6/24/2024	**DATE MEASURED:**	6/24/2024	**FIGURE:**	2

LOG OF BORING NO. B-2

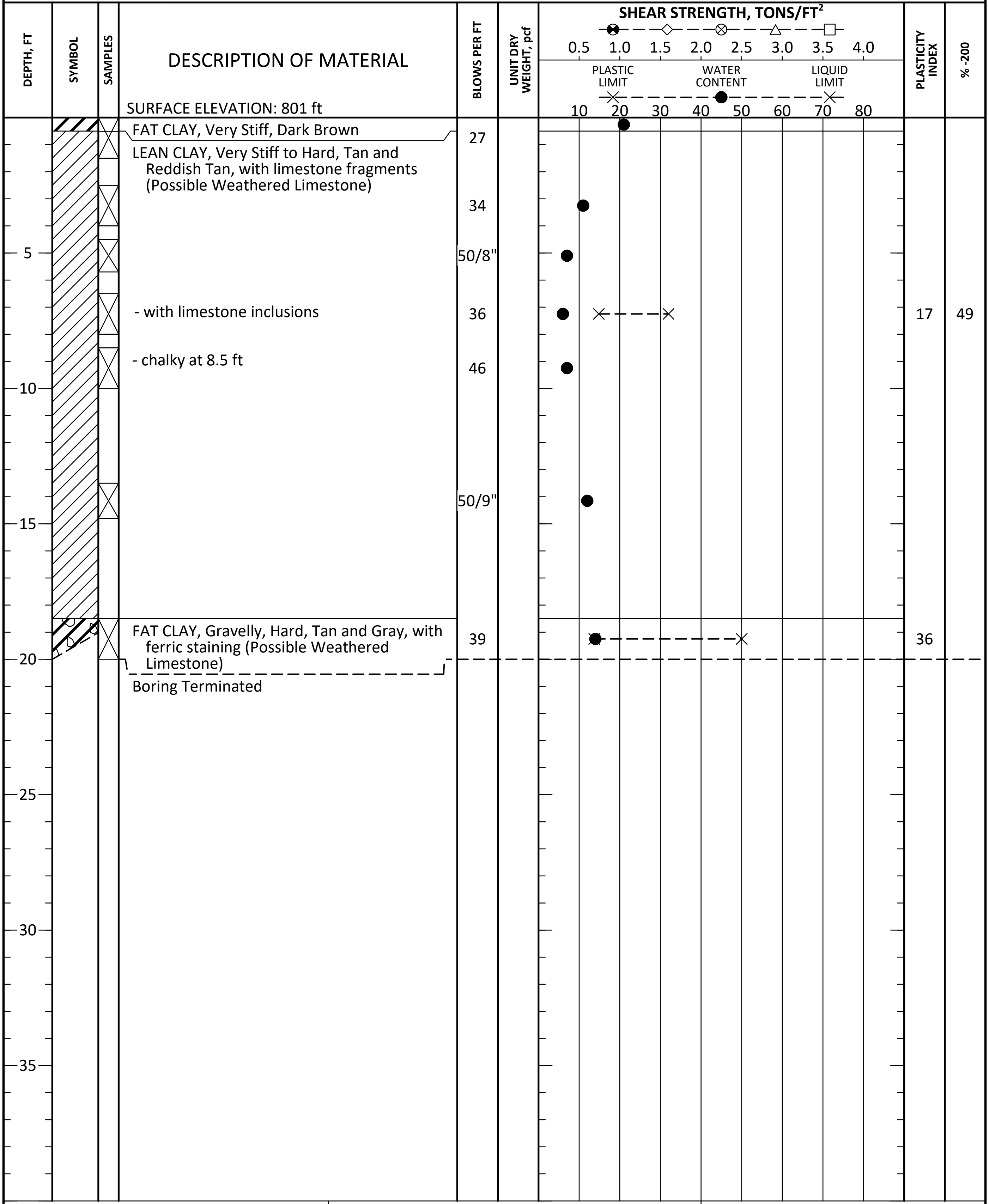
Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.74982; W 98.14060

SURFACE ELEVATION: 801 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX	% -200
FAT CLAY, Very Stiff, Dark Brown	27		
LEAN CLAY, Very Stiff to Hard, Tan and Reddish Tan, with limestone fragments (Possible Weathered Limestone)	34		
	50/8"		
- with limestone inclusions	36	17	49
- chalky at 8.5 ft	46		
	50/9"		
FAT CLAY, Gravelly, Hard, Tan and Gray, with ferric staining (Possible Weathered Limestone)	39	36	
Boring Terminated			

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

DEPTH DRILLED: 20.0 ft
DATE DRILLED: 6/24/2024
DEPTH TO WATER: Dry
DATE MEASURED: 6/24/2024
PROJ. No.: ANA24-019-00
FIGURE: 3

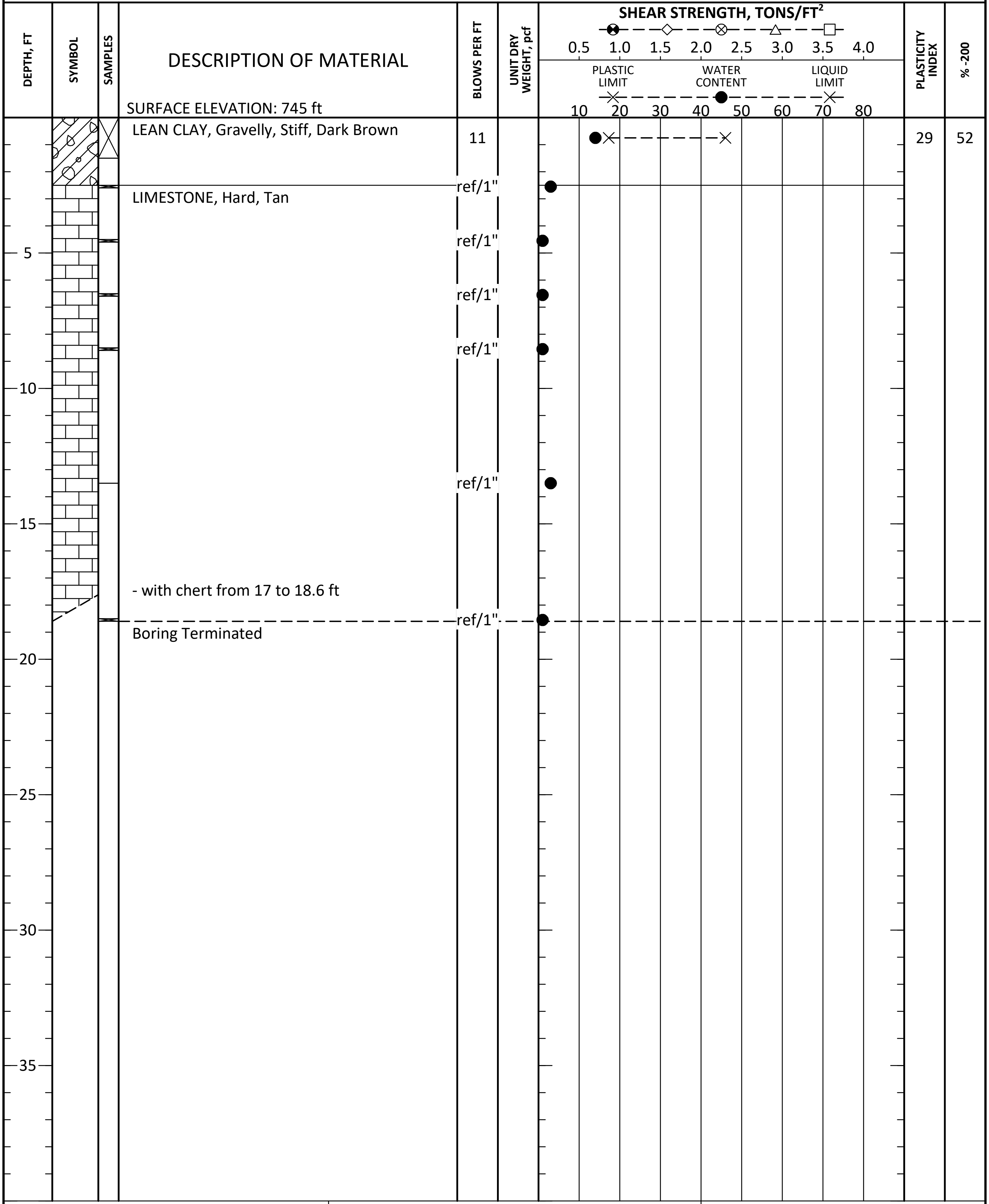
LOG OF BORING NO. B-3

Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.75084; W 98.14092



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX	% -200
LEAN CLAY, Gravelly, Stiff, Dark Brown	11	29	52
LIMESTONE, Hard, Tan	ref/1"		
	ref/1"		
	ref/1"		
	ref/1"		
	ref/1"		
- with chert from 17 to 18.6 ft			
Boring Terminated	ref/1"		

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

DEPTH DRILLED: 18.6 ft
DATE DRILLED: 6/24/2024
DEPTH TO WATER: Dry
DATE MEASURED: 6/24/2024
PROJ. No.: ANA24-019-00
FIGURE: 4

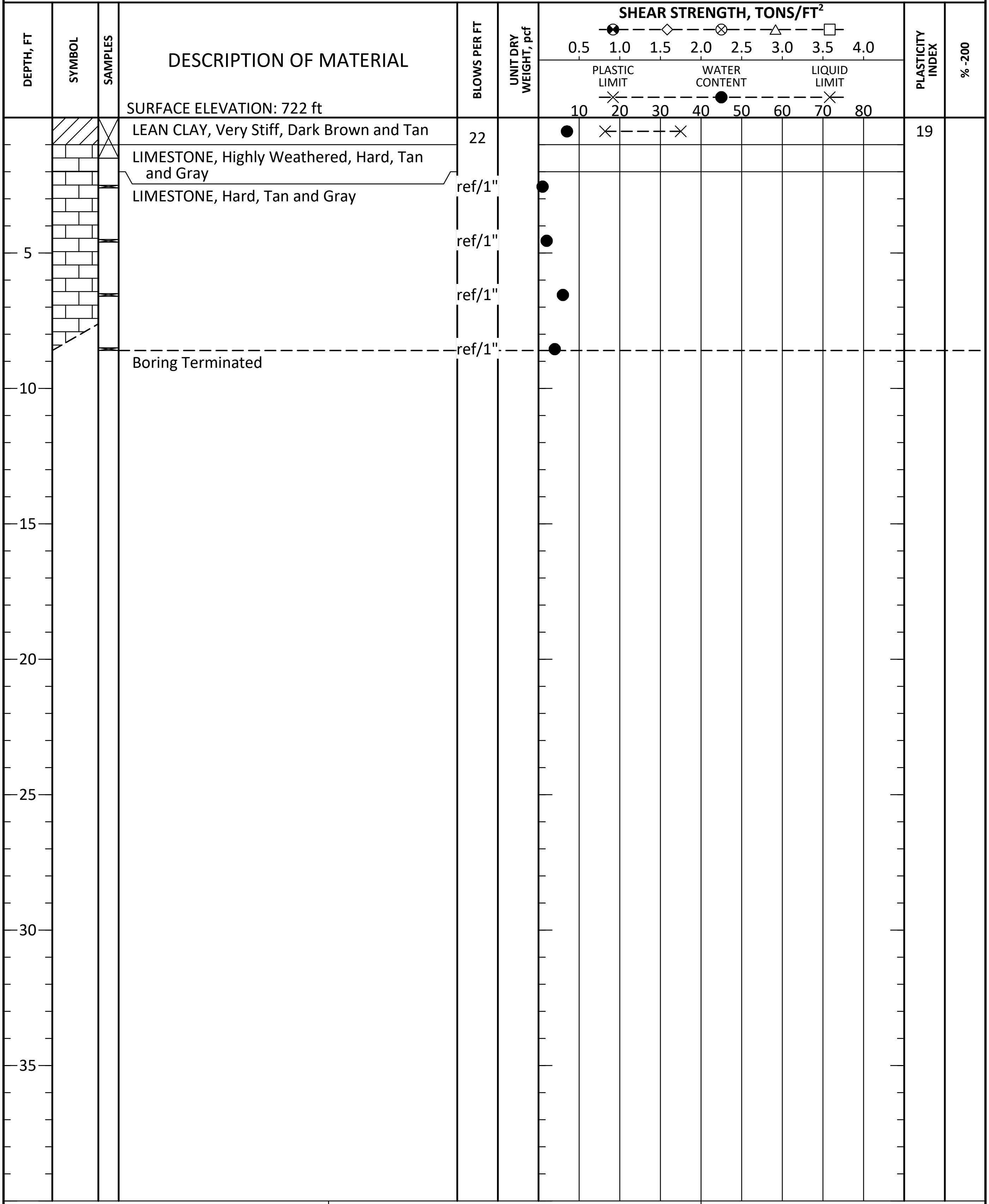
LOG OF BORING NO. B-4

Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.75187; W 98.14071



SURFACE ELEVATION: 722 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX
LEAN CLAY, Very Stiff, Dark Brown and Tan	22	19
LIMESTONE, Highly Weathered, Hard, Tan and Gray		
LIMESTONE, Hard, Tan and Gray	ref/1"	
	ref/1"	
	ref/1"	
	ref/1"	
Boring Terminated		

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

DEPTH DRILLED: 8.6 ft
DATE DRILLED: 6/24/2024
DEPTH TO WATER: Dry
DATE MEASURED: 6/24/2024
PROJ. No.: ANA24-019-00
FIGURE: 5

LOG OF BORING NO. B-5

Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

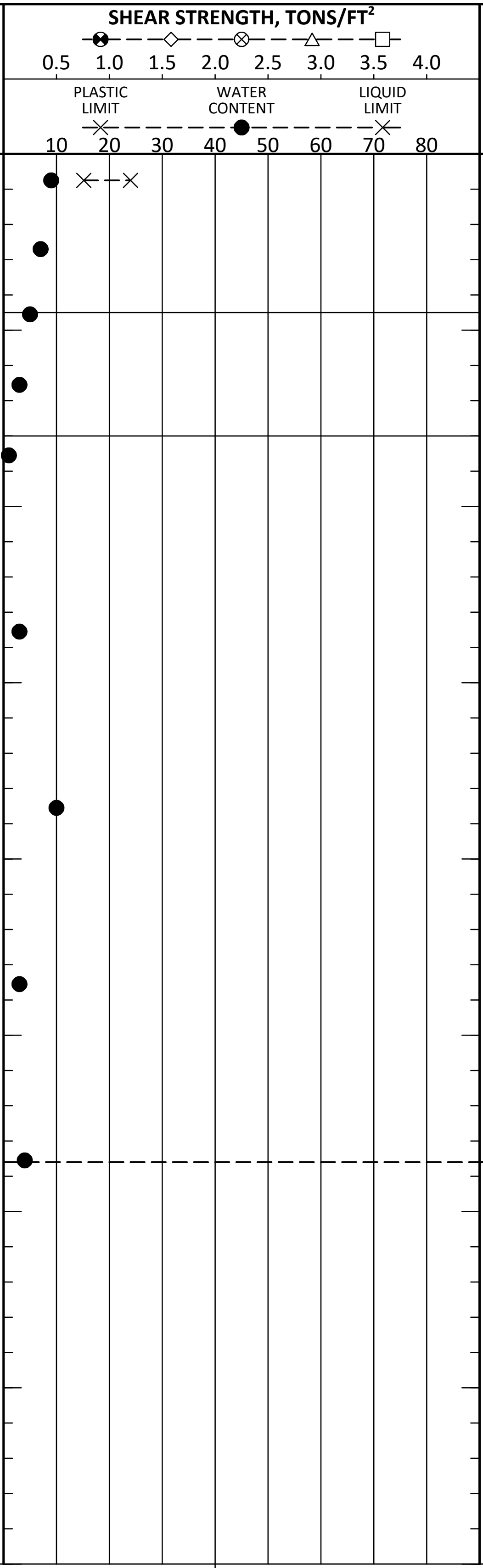
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.75159; W 98.13997

SURFACE ELEVATION: 732 ft

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			LEAN CLAY, Hard, Dark Brown and Tan Clay	50		9	
				ref/5"			
				ref/1"			
5			LIMESTONE, Highly Weathered, Hard, Tan and Gray				
				ref/1"			
			LIMESTONE, Hard, Tan and Gray	ref/1"			
10							
				ref/1"			
15							
			- with clay seams at 18 ft	ref/1"			
20							
				ref/1"			
25							
				ref/1"			
			Boring Terminated				
30							
35							



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

DEPTH DRILLED: 28.6 ft
DATE DRILLED: 5/5/2024

DEPTH TO WATER: Dry
DATE MEASURED: 5/5/2024

PROJ. No.: ANA24-019-00
FIGURE: 6

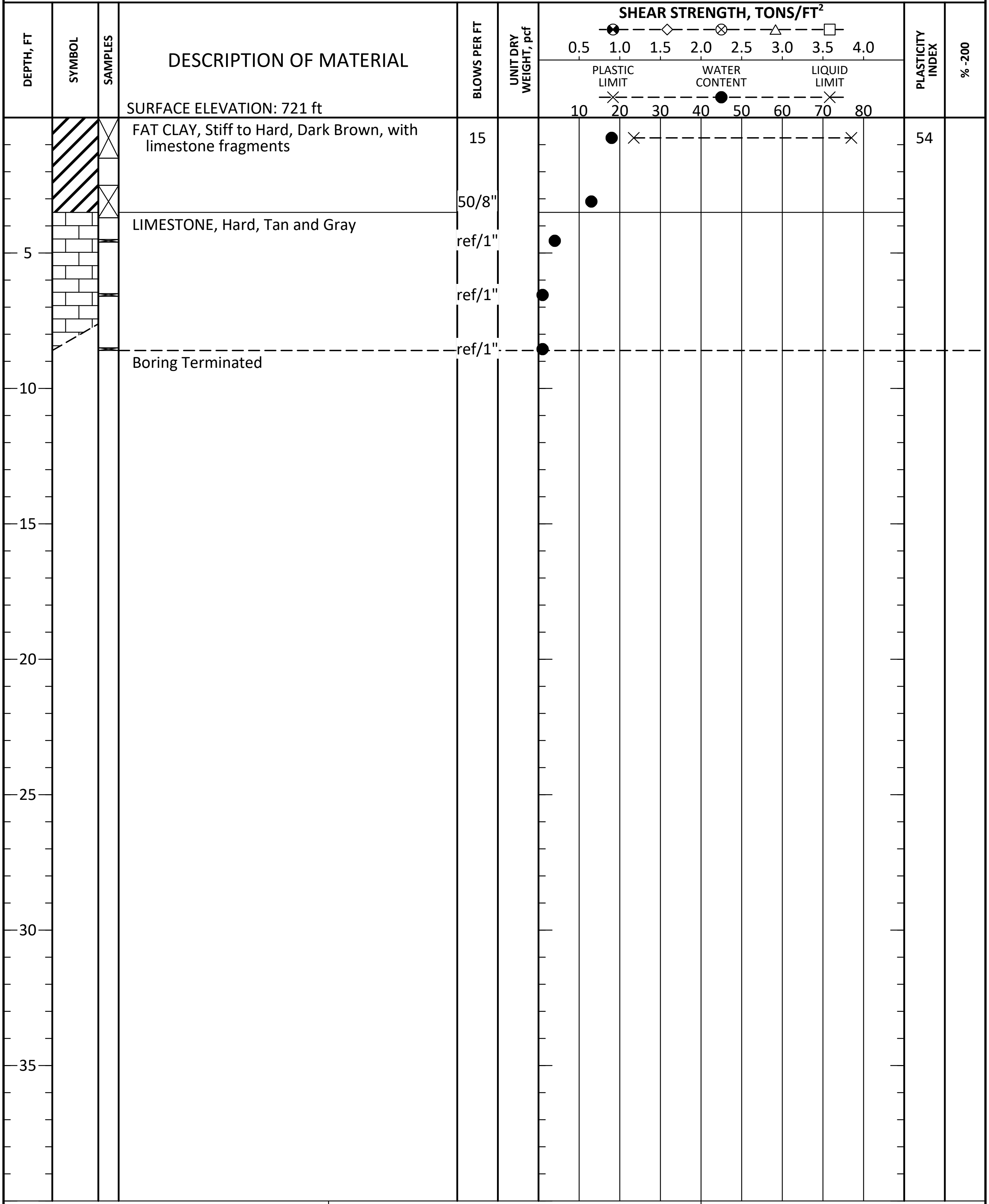
LOG OF BORING NO. B-6

Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.75110; W 98.13791



SURFACE ELEVATION: 721 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
FAT CLAY, Stiff to Hard, Dark Brown, with limestone fragments	15		54	
	50/8"			
LIMESTONE, Hard, Tan and Gray	ref/1"			
	ref/1"			
	ref/1"			
Boring Terminated				

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

DEPTH DRILLED: 8.6 ft
DATE DRILLED: 6/24/2024

DEPTH TO WATER: Dry
DATE MEASURED: 6/24/2024

PROJ. No.: ANA24-019-00
FIGURE: 7

LOG OF BORING NO. P-1

Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

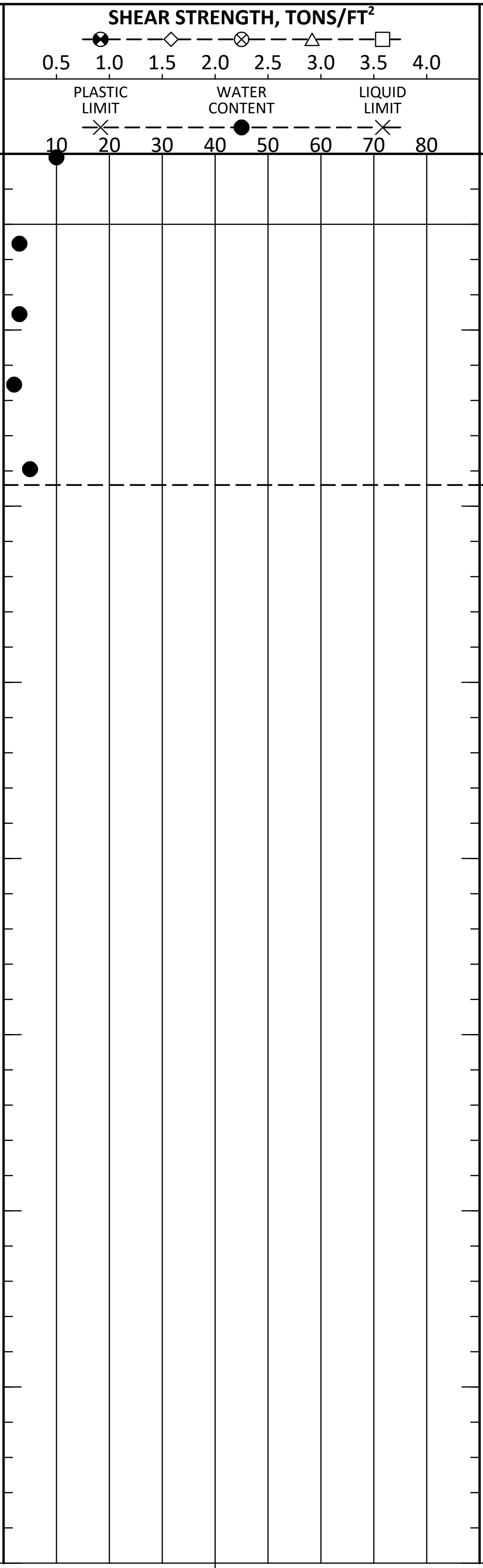
RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.74995; W 98.13880

SURFACE ELEVATION: 739 ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	FAT CLAY, Hard, Dark Brown, with limestone fragments	ref/2"			
2	LIMESTONE, Hard, Tan and Gray	ref/1"			
		ref/1"			
		ref/1"			
8	- highly weathered from 8 to 9.4 ft	50/5"			
9.4	Boring Terminated				



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

DEPTH DRILLED: 9.4 ft
DATE DRILLED: 6/24/2024

DEPTH TO WATER: Dry
DATE MEASURED: 6/24/2024

PROJ. No.: ANA24-019-00
FIGURE: 8

LOG OF BORING NO. P-2

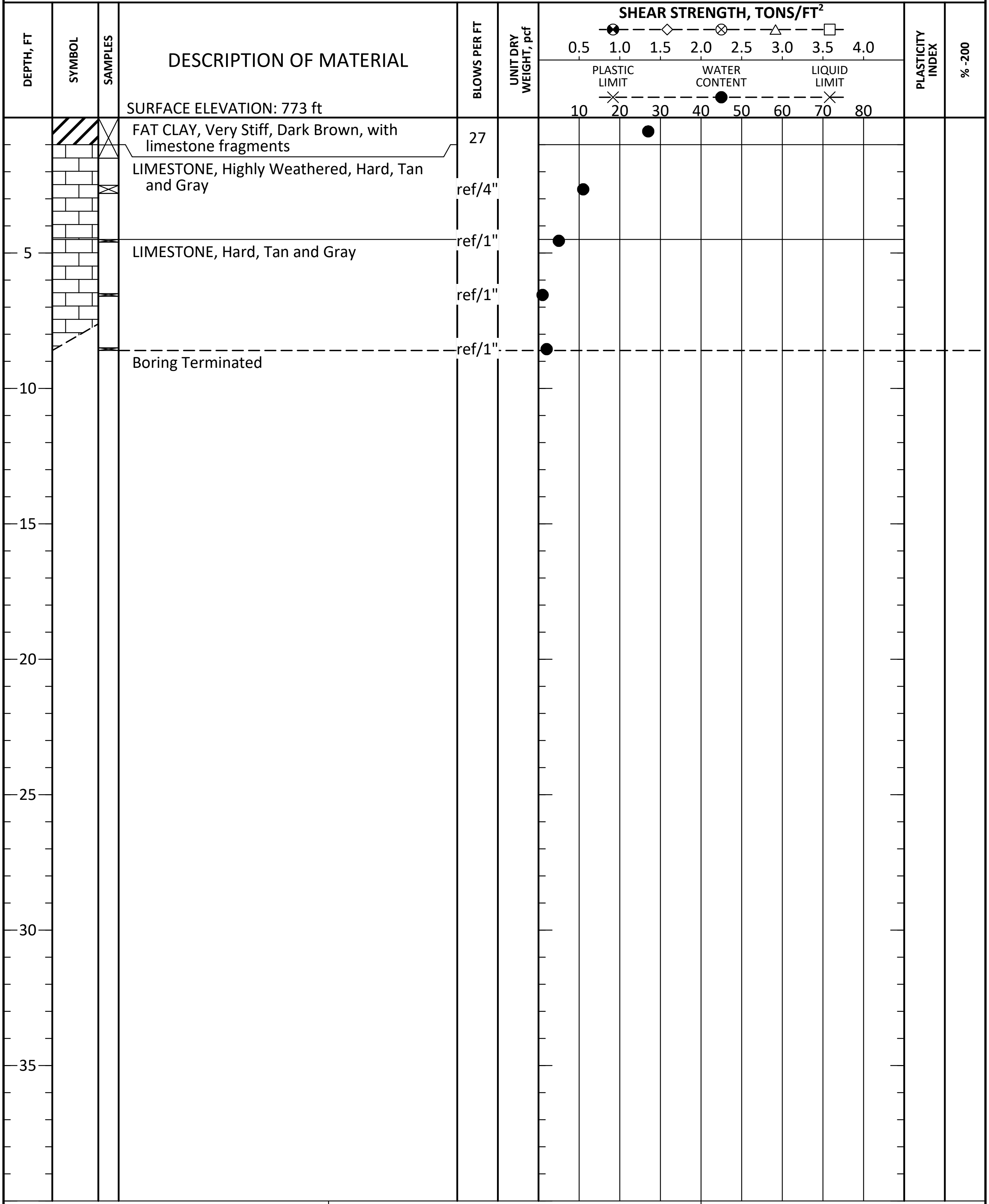
Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.74741; W 98.14426

SURFACE ELEVATION: 773 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT
FAT CLAY, Very Stiff, Dark Brown, with limestone fragments	27
LIMESTONE, Highly Weathered, Hard, Tan and Gray	ref/4"
LIMESTONE, Hard, Tan and Gray	ref/1" ref/1" ref/1"
Boring Terminated	

DEPTH DRILLED:	8.6 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ANA24-019-00
DATE DRILLED:	6/13/2024	**DATE MEASURED:**	6/13/2024	**FIGURE:**	9

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

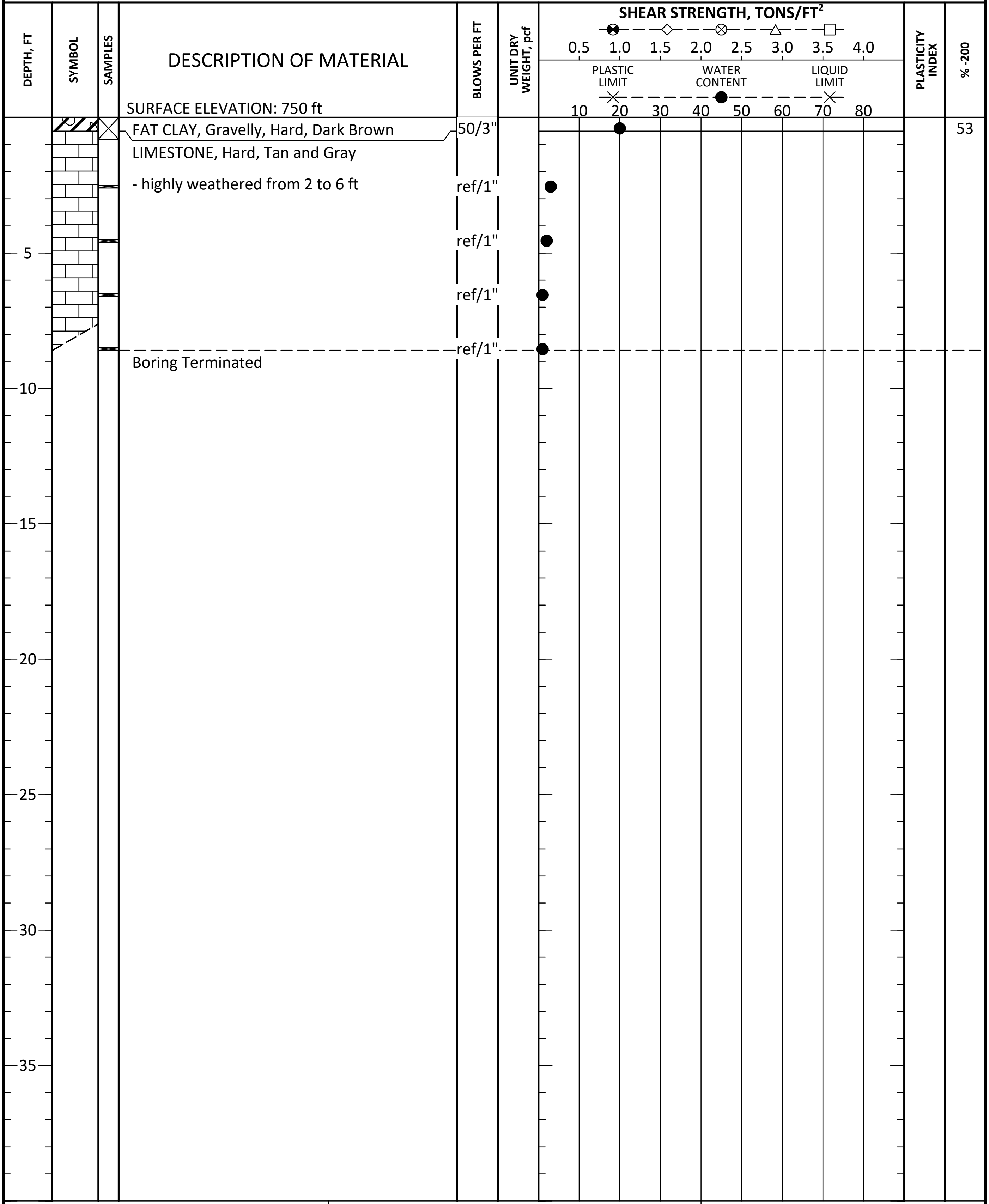
LOG OF BORING NO. P-3

Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.74912; W 98.14354



DESCRIPTION OF MATERIAL	BLOWS PER FT	% -200
FAT CLAY, Gravelly, Hard, Dark Brown	50/3"	53
LIMESTONE, Hard, Tan and Gray		
- highly weathered from 2 to 6 ft	ref/1"	
	ref/1"	
	ref/1"	
Boring Terminated	ref/1"	

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

DEPTH DRILLED: 8.6 ft
DATE DRILLED: 6/24/2024
DEPTH TO WATER: Dry
DATE MEASURED: 6/24/2024
PROJ. No.: ANA24-019-00
FIGURE: 10

LOG OF BORING NO. P-4

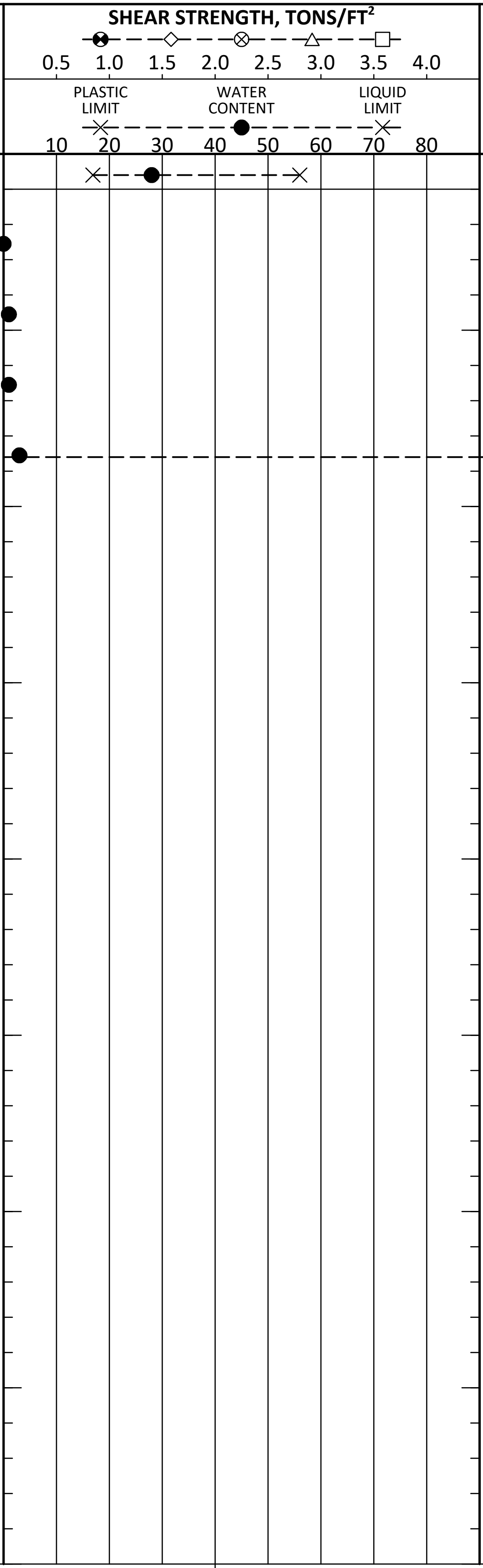
Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.75095; W 98.14247

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			SURFACE ELEVATION: 737 ft				
			FAT CLAY, Hard, Dark Brown	50/8"		39	
			LIMESTONE, Hard, Tan				
				ref/1"			
5				ref/1"			
				ref/1"			
			- highly weathered from 7 to 8 ft				
				ref/1"			
			Boring Terminated				
10							
15							
20							
25							
30							
35							



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

DEPTH DRILLED: 8.6 ft
DATE DRILLED: 6/24/2024

DEPTH TO WATER: Dry
DATE MEASURED: 6/24/2024

PROJ. No.: ANA24-019-00
FIGURE: 11

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

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

ROCK TERMS

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

OTHER

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

WELL CONSTRUCTION AND PLUGGING MATERIALS

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

SAMPLE TYPES

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

STRENGTH TEST TYPES

- ⊗ POCKET PENETROMETER
- ◇ TORVANE
- ⊗ UNCONFINED COMPRESSION
- △ TRIAXIAL COMPRESSION UNCONSOLIDATED-UNDRAINED
- □ TRIAXIAL COMPRESSION CONSOLIDATED-UNDRAINED

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

PROJECT NO. ANA24-019-00

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

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

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

RELATIVE DENSITY

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

COHESIVE STRENGTH

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

PLASTICITY

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

ABBREVIATIONS

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

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

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

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

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

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

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

STANDARD PENETRATION TEST (SPT)

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

FILE NAME: ANA24-019-00 GINT.GPJ 7/17/2024

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-1	0.0 to 0.9	50/5"	21	46	20	26	CL				
	2.5 to 3.2	50/2"									
	2.5 to 3.0		30								
	4.5 to 4.6	ref/1"	2								
	6.5 to 6.6	ref/1"	3								
	8.5 to 8.6	ref/1"	4								
	13.5 to 13.6	ref/1"	2								
	18.5 to 18.6	ref/1"	1								
	23.5 to 23.6	ref/1"	2								
	28.5 to 28.6	ref/1"	3								
B-2	0.0 to 1.5	27									
	0.0 to 0.5		21								
	2.5 to 4.0	34	11								
	4.5 to 5.7	50/8"	7								
	6.5 to 8.0	36	6	32	15	17	CL		49		
	8.5 to 10.0	46	7								
	13.5 to 14.8	50/9"	12								
	18.5 to 20.0	39	14	50	14	36	CH				
B-3	0.0 to 1.5	11	14	46	17	29	CL		52		
	2.5 to 2.6	ref/1"	3								
	4.5 to 4.6	ref/1"	1								
	6.5 to 6.6	ref/1"	1								
	8.5 to 8.6	ref/1"	1								
	13.5 to 13.5	ref/1"	3								
	18.5 to 18.6	ref/1"	1								
B-4	0.0 to 1.5	22									
	0.0 to 1.0		7	35	16	19	CL				
	2.5 to 2.6	ref/1"	1								
	4.5 to 4.6	ref/1"	2								
	6.5 to 6.6	ref/1"	6								
	8.5 to 8.6	ref/1"	4								
B-5	0.0 to 1.5	50	9	24	15	9	CL				
	2.5 to 2.9	ref/5"	7								
	4.5 to 4.6	ref/1"	5								
	6.5 to 6.6	ref/1"	3								
	8.5 to 8.6	ref/1"	1								
	13.5 to 13.6	ref/1"	3								
	18.5 to 18.6	ref/1"	10								
	23.5 to 23.6	ref/1"	3								

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: Precinct 30 - Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

FILE NAME: ANA24-019-00 GINT.GPJ 7/17/2024

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
B-5	28.5 to 28.6	ref/1"	4								
B-6	0.0 to 1.5	15	18	77	23	54	CH				
	2.5 to 3.7	50/8"	13								
	4.5 to 4.6	ref/1"	4								
	6.5 to 6.6	ref/1"	1								
	8.5 to 8.6	ref/1"	1								
P-1	0.0 to 0.2	ref/2"	10								
	2.5 to 2.6	ref/1"	3								
	4.5 to 4.6	ref/1"	3								
	6.5 to 6.6	ref/1"	2								
	8.5 to 9.4	50/5"	5								
P-2	0.0 to 1.5	27									
	0.0 to 1.0		27								
	2.5 to 2.8	ref/4"	11								
	4.5 to 4.6	ref/1"	5								
	6.5 to 6.6	ref/1"	1								
	8.5 to 8.6	ref/1"	2								
P-3	0.0 to 0.8	50/3"	20						53		
	2.5 to 2.6	ref/1"	3								
	4.5 to 4.6	ref/1"	2								
	6.5 to 6.6	ref/1"	1								
	8.5 to 8.6	ref/1"	1								
P-4	0.0 to 1.2	50/8"	28	56	17	39	CH				
	2.5 to 2.6	ref/1"	0								
	4.5 to 4.6	ref/1"	1								
	6.5 to 6.6	ref/1"	1								
	8.5 to 8.6	ref/1"	3								

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

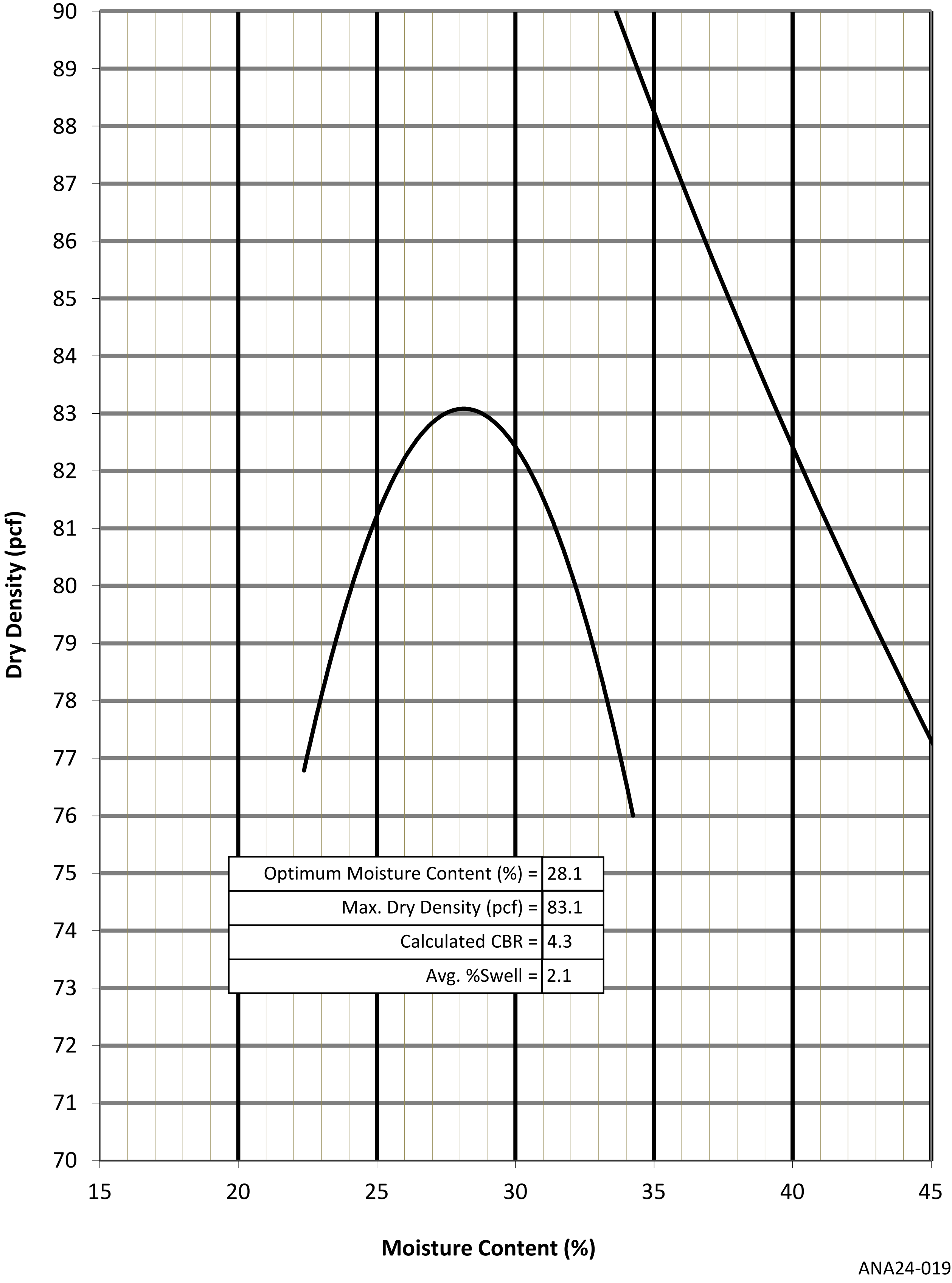
CU = Consolidated Undrained Triaxial

MOISTURE DENSITY RELATIONSHIP CURVE, TEX-114-E, P-1

Precinct 30 - Units 2 and 3

Veramendi Master Planned Development

New Braunfels, Texas



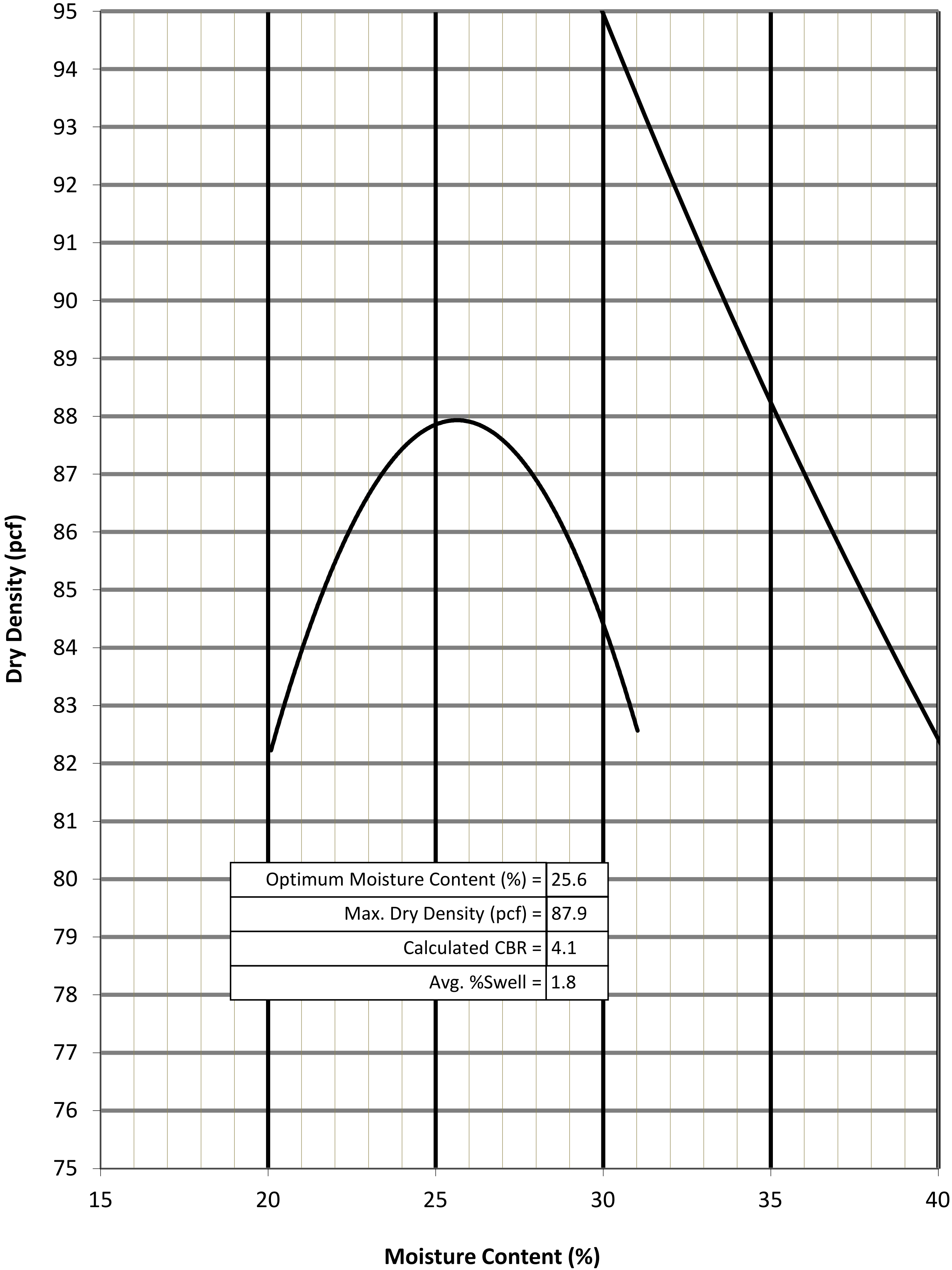
Optimum Moisture Content (%) =	28.1
Max. Dry Density (pcf) =	83.1
Calculated CBR =	4.3
Avg. %Swell =	2.1

MOISTURE DENSITY RELATIONSHIP CURVE, TEX-114-E, P-3

Precinct 30 - Units 2 and 3

Veramendi Master Planned Development

New Braunfels, Texas



Optimum Moisture Content (%) = 25.6

Max. Dry Density (pcf) = 87.9

Calculated CBR = 4.1

Avg. %Swell = 1.8

pH-LIME SERIES CURVE, P-1

Precinct 30 - Units 2 and 3

Veramendi Master Planned Development

New Braunfels, Texas

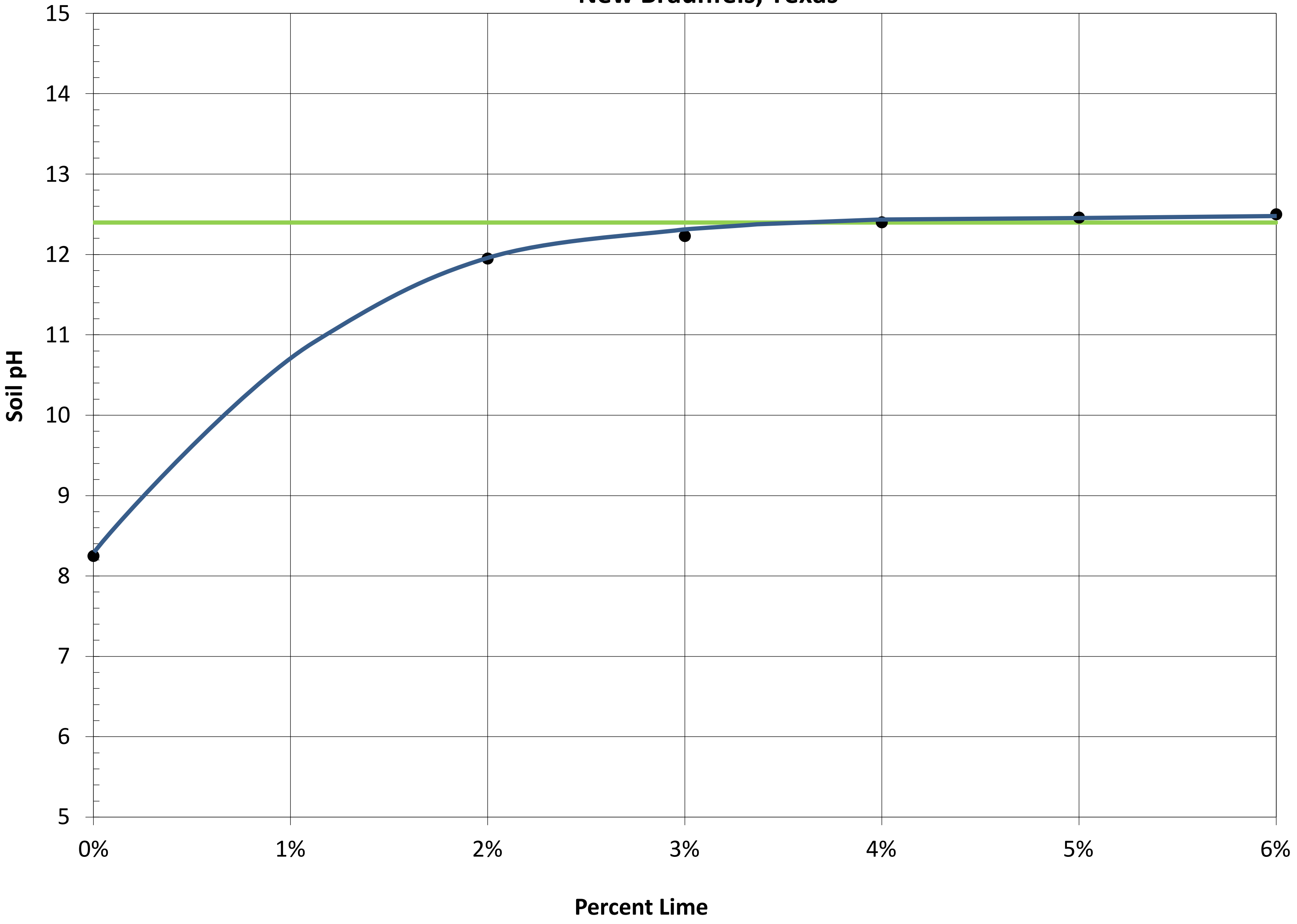


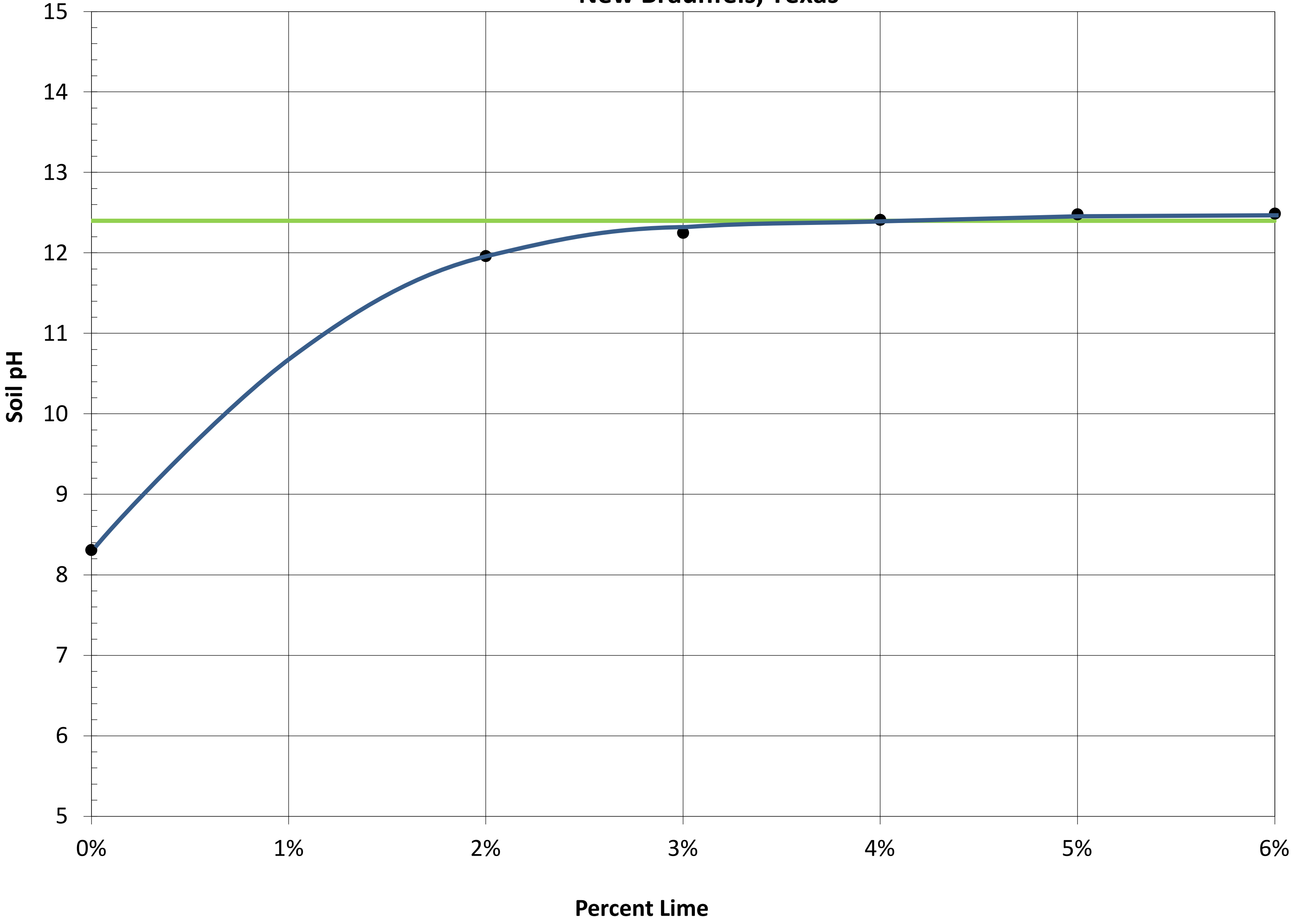
Figure 15a

pH-LIME SERIES CURVE, P-3

Precinct 30 - Units 2 and 3

Veramendi Master Planned Development

New Braunfels, Texas

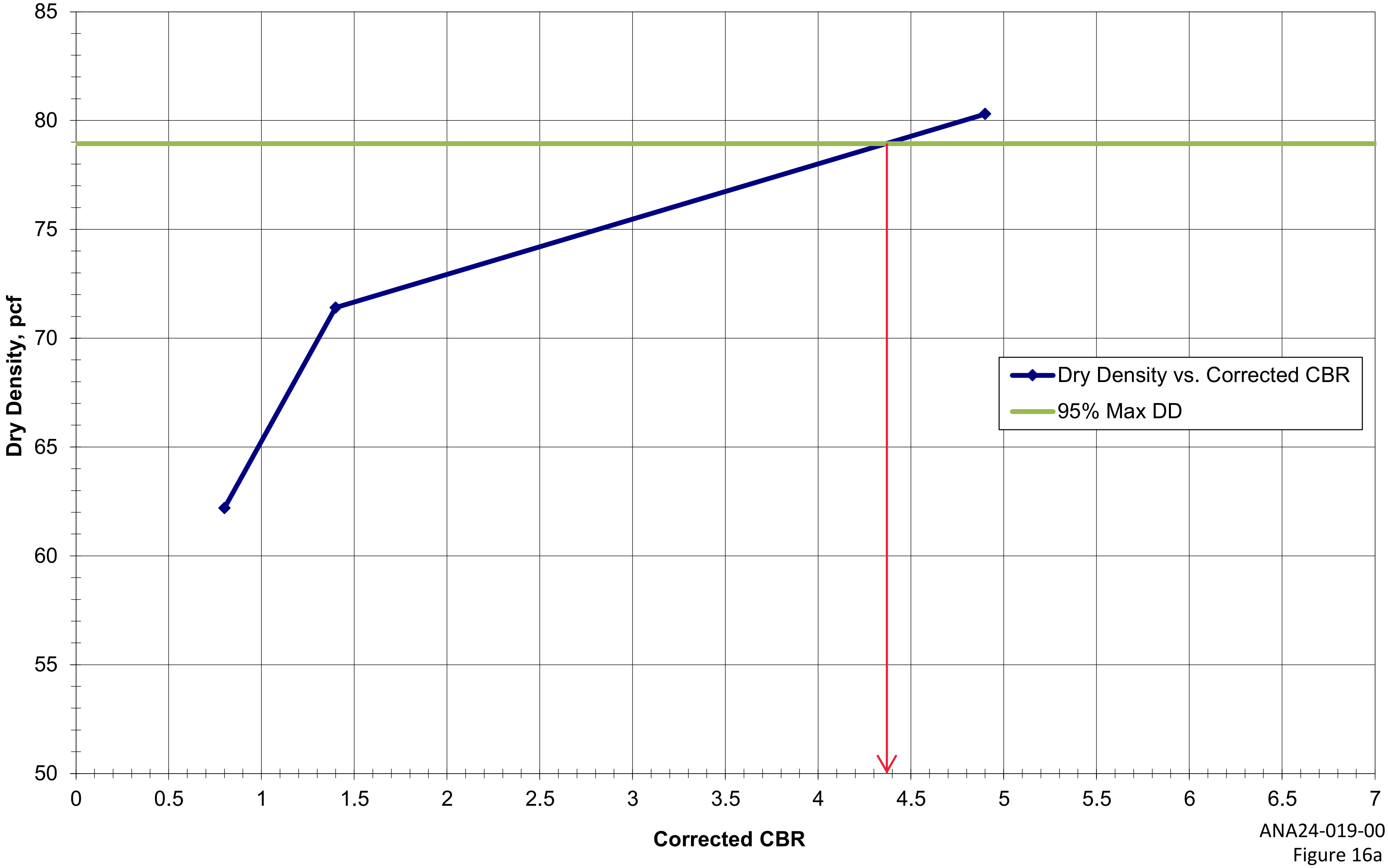


Dry Density vs. Corrected CBR, P-1

Precinct 30 - Units 2 and 3

Veramendi Master Planned Development

New Braunfels, Texas



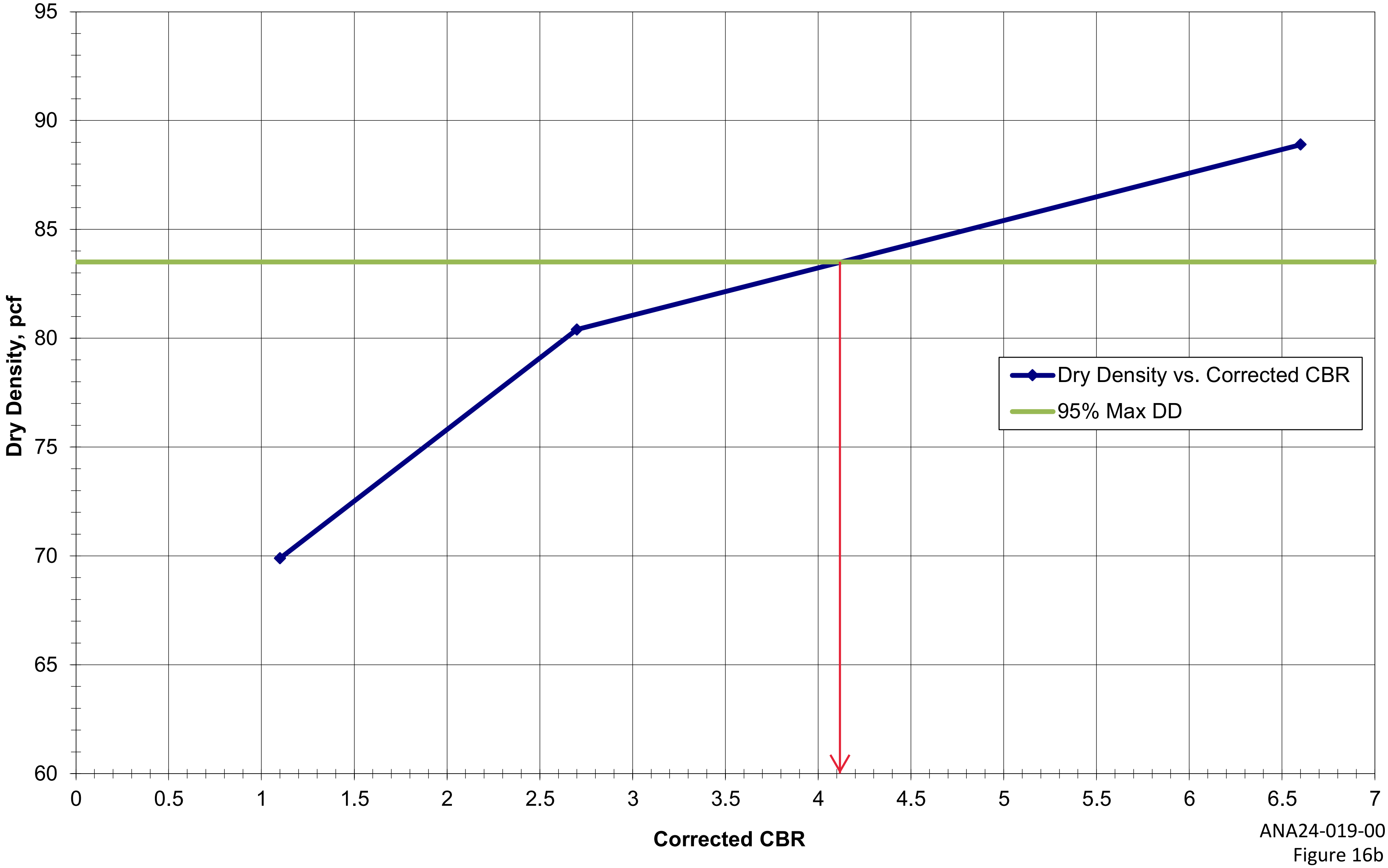
ANA24-019-00
Figure 16a

Dry Density vs. Corrected CBR, P-3

Precinct 30 - Units 2 and 3

Veramendi Master Planned Development

New Braunfels, Texas



ANA24-019-00
Figure 16b

Project Number: ANA24-019-00
Test Date: June 13, 2024

DCP TEST DATA

B-2

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	1	61	2.4	3	4.5	1.13
1	2	36	3.8	11	16.5	2.69
1	2	35	5.2	12	18	2.84
1	2	33	6.5	13	19.5	3.00
1	2	32	7.8	13	19.5	3.00
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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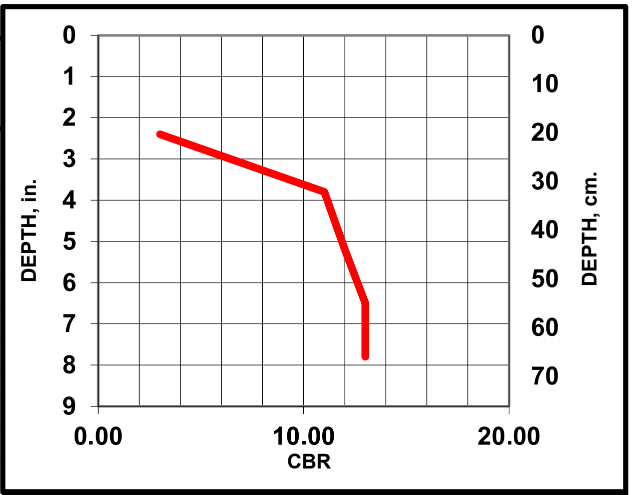
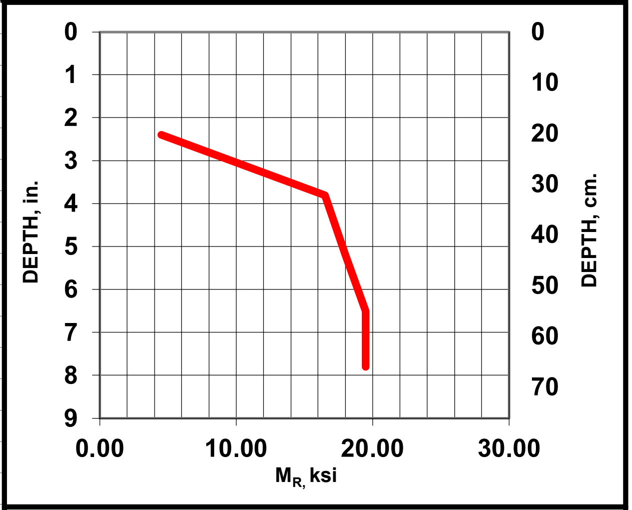
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





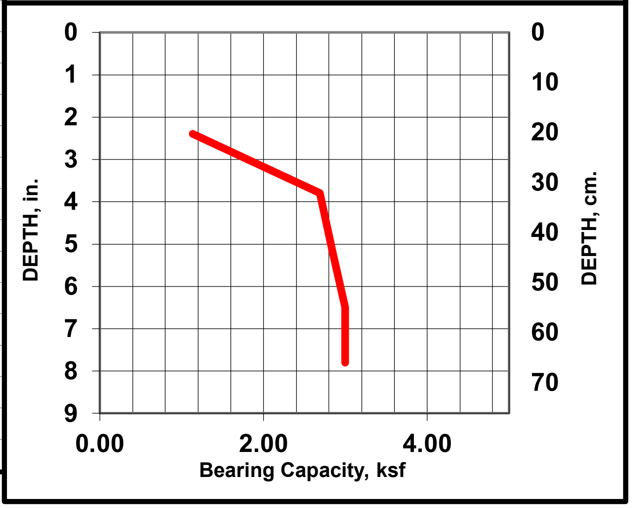


Figure 17a

Project Number: ANA24-019-00
Test Date: June 13, 2024

DCP TEST DATA

B-3

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	50	2	4	6	1.37
1	2	31	3.2	14	21	3.15
1	2	29	4.3	15	22.5	3.30
1	2	30	5.5	14	21	3.15
1	2	30	6.7	14	21	3.15
-	-	-	-	-	-	-
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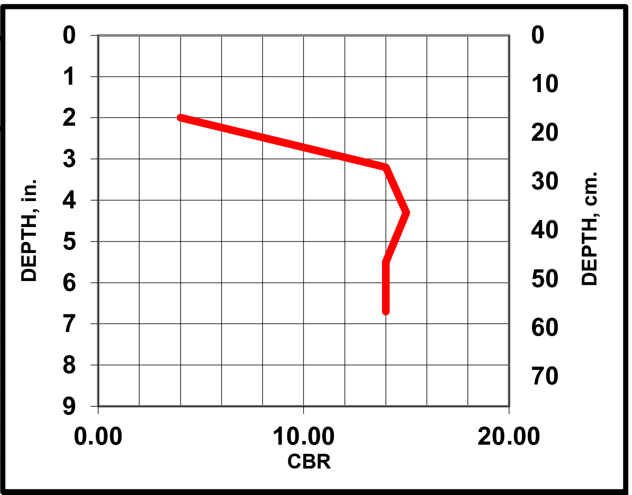
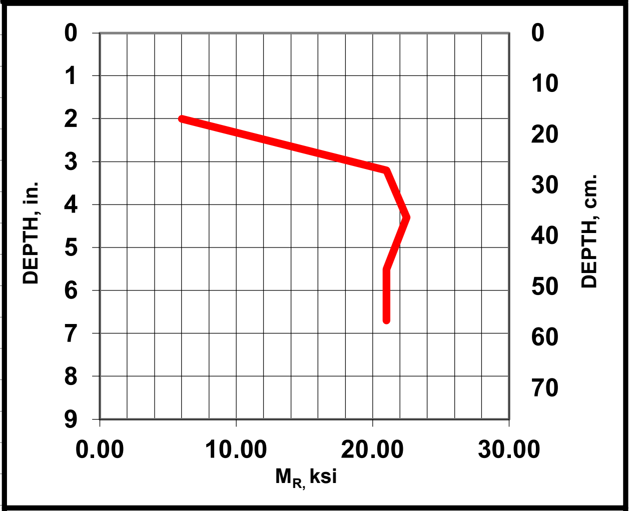
-	-	-	-	-	-	-

NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





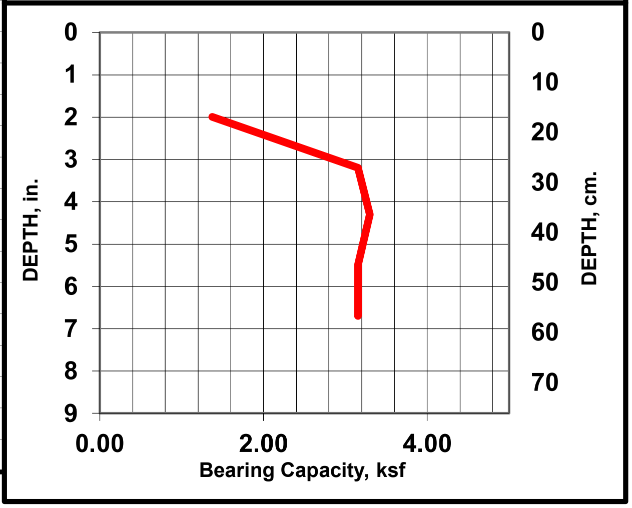


Figure 17b

RABA KISTNER

Project Number: ANA24-019-00

Test Date: June 13, 2024

DCP TEST DATA

B-4

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	86	3.4	2	3	0.87
1	1	58	5.7	3	4.5	1.13
1	1	54	7.8	3	4.5	1.13
1	2	57	10	7	10.5	1.99
1	8	30	11.2	66	99	8.82
1	4	30	12.4	31	46.5	5.34
1	5	35	13.8	33	49.5	5.57
1	5	34	15.1	34	51	5.68
1	6	40	16.7	35	52.5	5.79
1	7	33	18	51	76.5	7.43
1	5	33	19.3	35	52.5	5.79
1	6	54	21.4	25	37.5	4.63
1	6	16	22	97	145.5	11.39
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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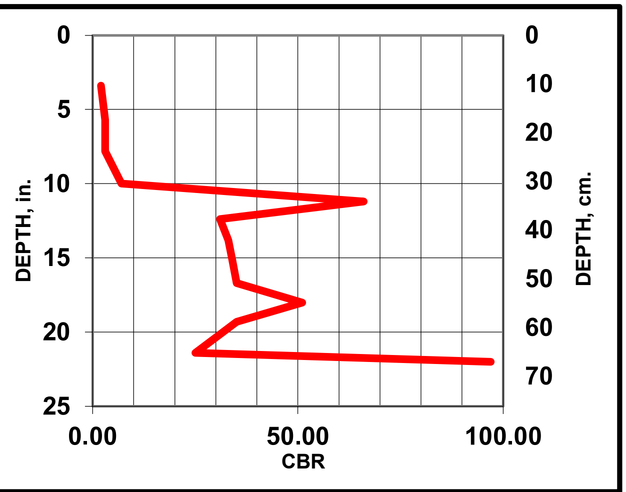
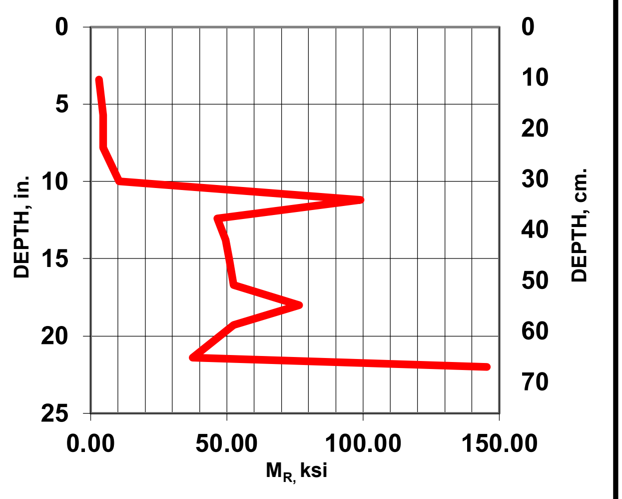
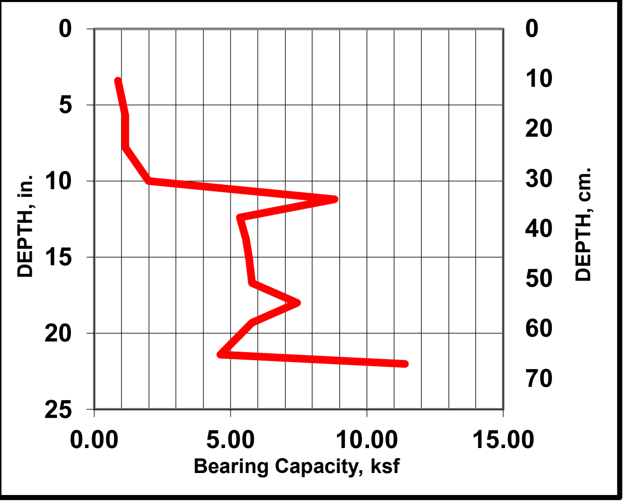
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2







Project Number: ANA24-019-00
Test Date: June 13, 2024

DCP TEST DATA

B-6

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration		CBR	M_R	q_{ult}
		Incre. (mm)	Cumm. (in)	(%)	(ksi)	(ksf)
1	1	96	3.8	2	3	0.87
1	1	39	5.3	5	7.5	1.59
1	1	46	7.1	4	6	1.37
1	1	34	8.5	6	9	1.80
1	2	30	9.6	14	21	3.15
1	2	28	10.7	15	22.5	3.30
1	2	41	12.4	10	15	2.52
1	8	31	13.6	64	96	8.64
1	3	37	15	18	27	3.72
1	3	18	15.7	39	58.5	6.22
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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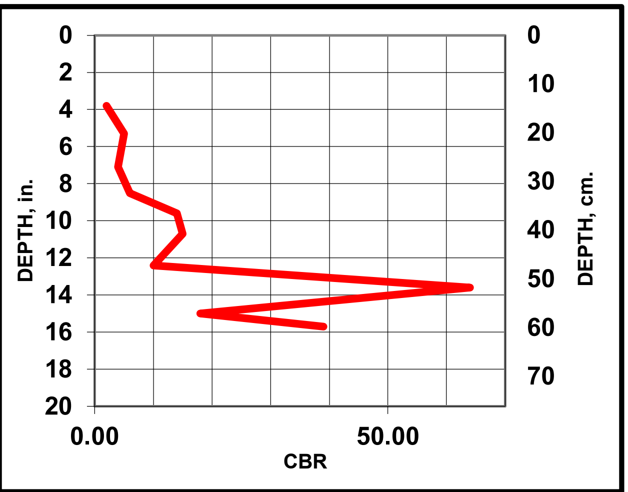
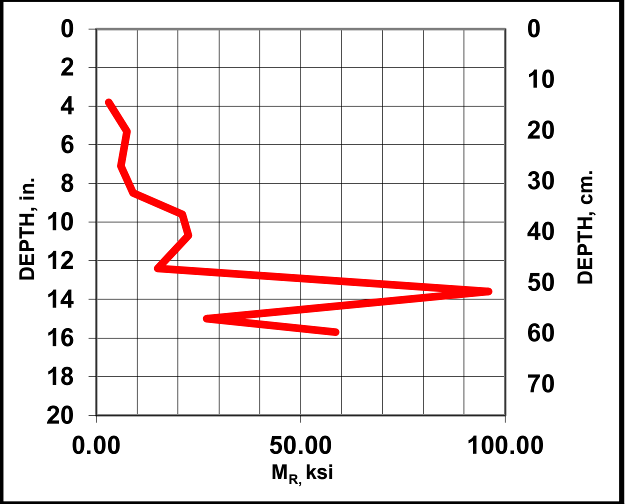
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





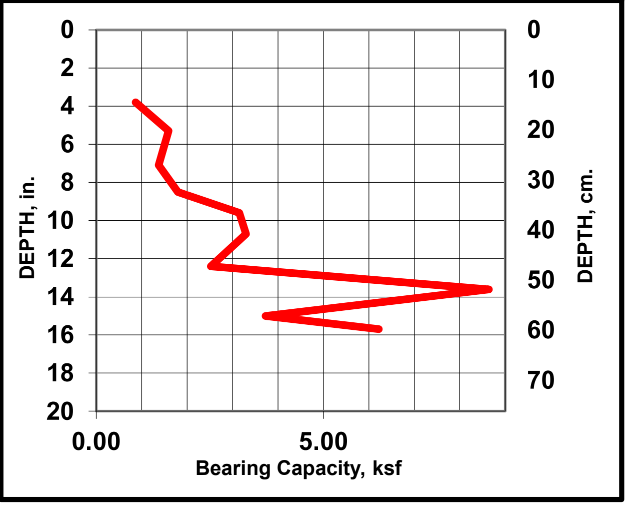


Figure 17d

RABA KISTNER

Project Number: ANA24-019-00
Test Date: June 13, 2024

DCP TEST DATA

P-1

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	4	33	1.3	27	40.5	4.87
1	2	37	2.8	11	16.5	2.69
1	1	37	4.2	5	7.5	1.59
1	1	26	5.2	8	12	2.17
1	1	28	6.3	7	10.5	1.99
1	1	19	7.1	11	16.5	2.69
1	1	4	7.2	62	93	8.46
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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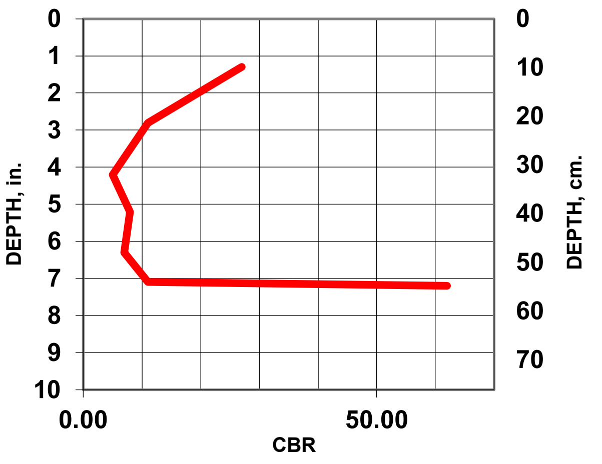
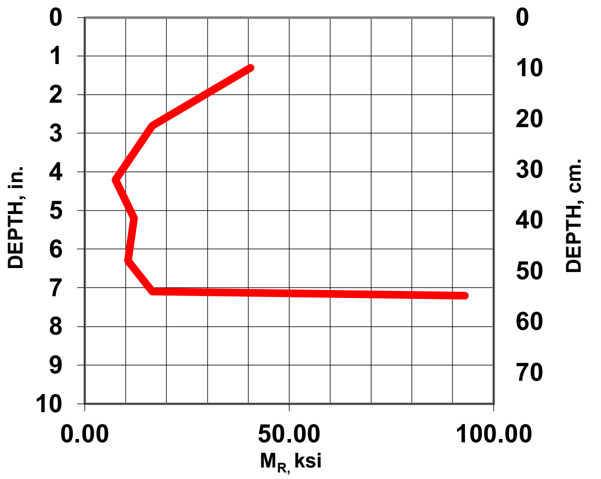
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





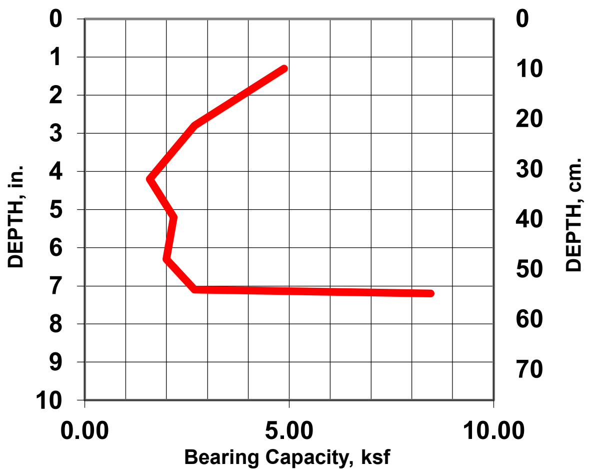


Figure 17e

Project Number: ANA24-019-00
Test Date: June 13, 2024

DCP TEST DATA

P-2

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	105	4.1	2	3	0.87
1	1	65	6.7	3	4.5	1.13
1	1	41	8.3	5	7.5	1.59
1	1	30	9.5	6	9	1.80
1	1	36	10.9	5	7.5	1.59
1	1	30	12.1	6	9	1.80
1	1	31	13.3	6	9	1.80
1	1	25	14.3	8	12	2.17
1	3	32	15.6	21	31.5	4.13
1	4	30	16.7	31	46.5	5.34
1	4	45	18.5	19	28.5	3.86
1	4	45	20.3	19	28.5	3.86
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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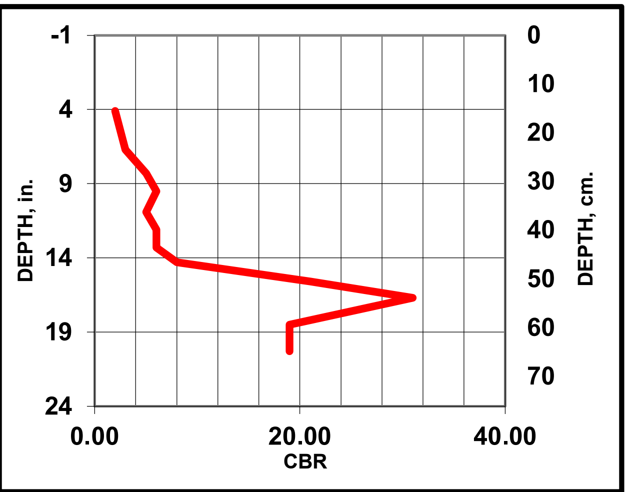
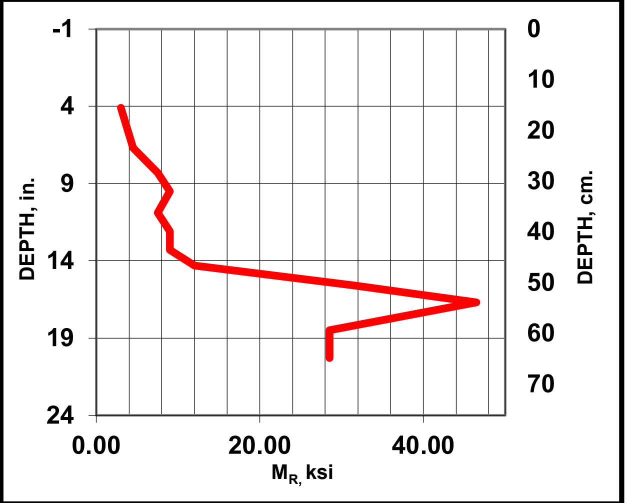
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





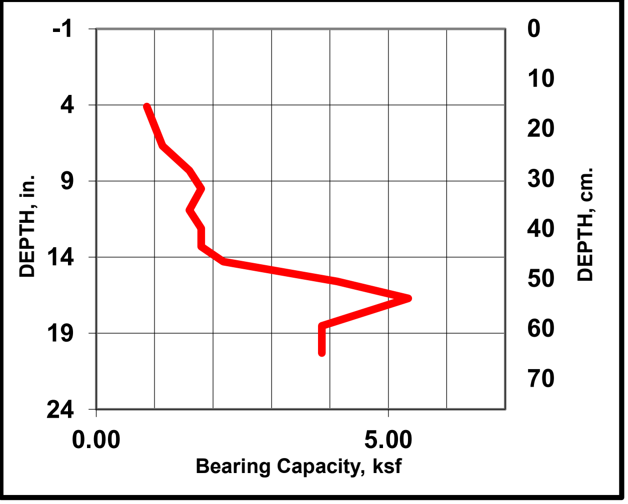


Figure 17f

Project Number: ANA24-019-00
Test Date: June 13, 2024

DCP TEST DATA

P-3

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	65	2.6	3	4.5	1.13
1	1	55	4.7	3	4.5	1.13
1	1	44	6.5	4	6	1.37
1	2	33	7.8	13	19.5	3.00
1	2	33	9.1	13	19.5	3.00
1	2	40	10.6	10	15	2.52
1	2	42	12.3	10	15	2.52
1	2	42	13.9	10	15	2.52
1	4	32	15.2	28	42	4.99
1	3	31	16.4	21	31.5	4.13
1	3	36	17.8	18	27	3.72
1	4	27	18.9	34	51	5.68
1	4	21	19.7	46	69	6.94
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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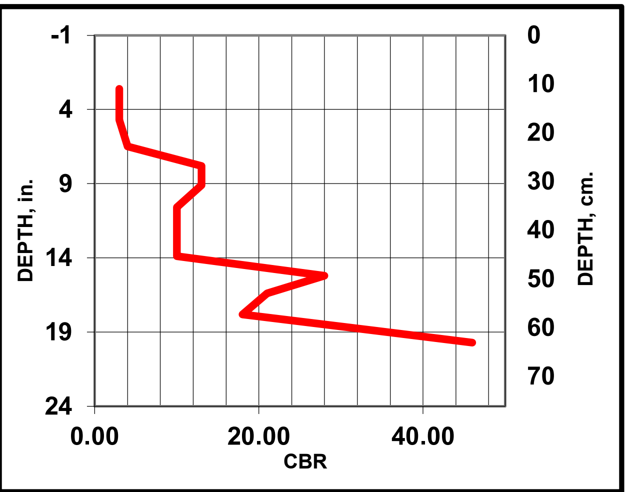
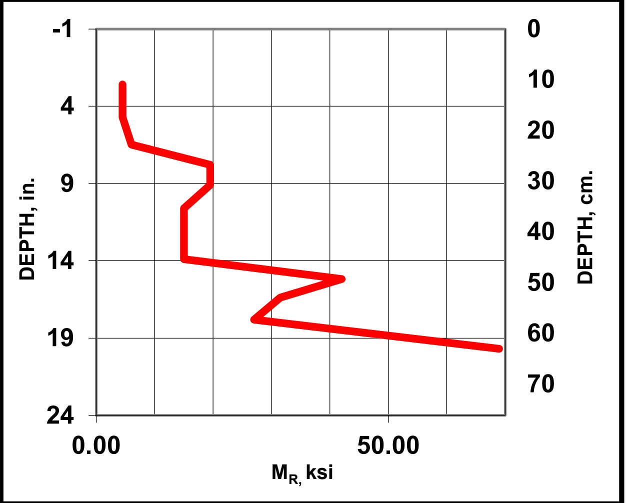
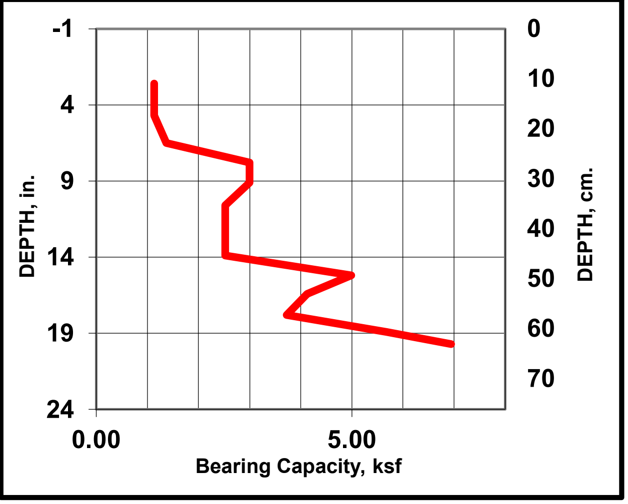
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2







RABA KISTNER

Project Number: ANA24-019-00
Test Date: June 13, 2024

DCP TEST DATA

P-4

Precinct 30 – Units 2 and 3
Veramendi Master Planned Development
New Braunfels, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	3	115	4.5	5	7.5	1.59
1	2	33	5.8	13	19.5	3.00
1	1	7	6.1	33	49.5	5.57
1	1	30	7.3	6	9	1.80
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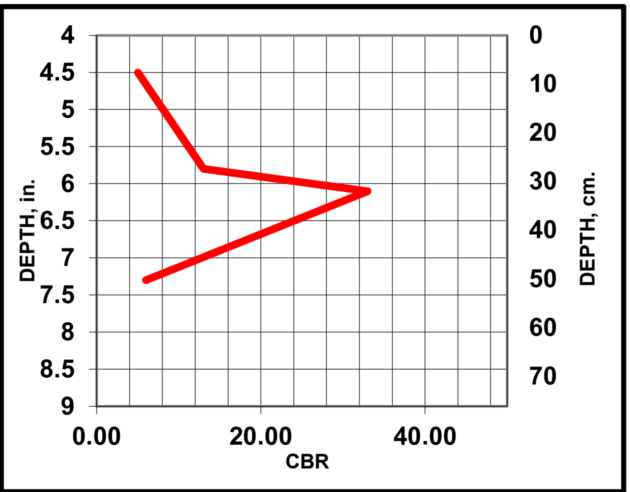
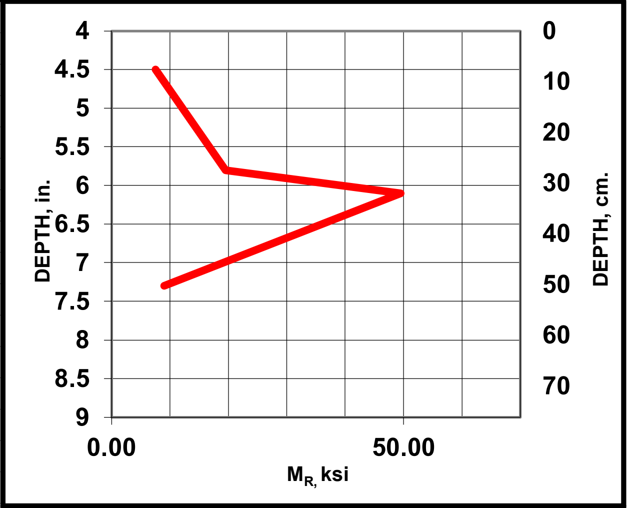
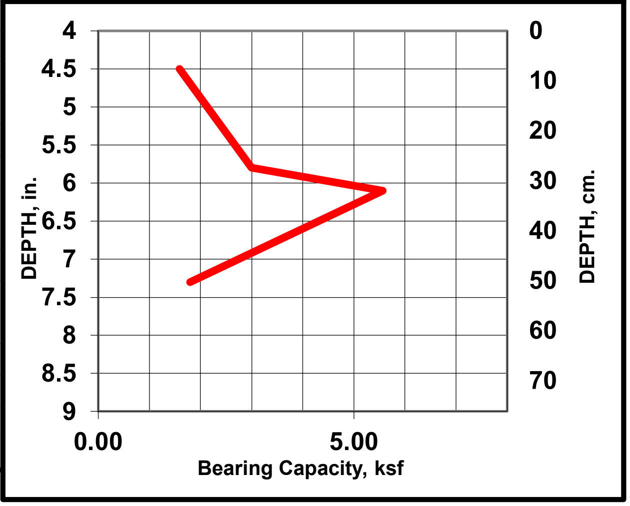
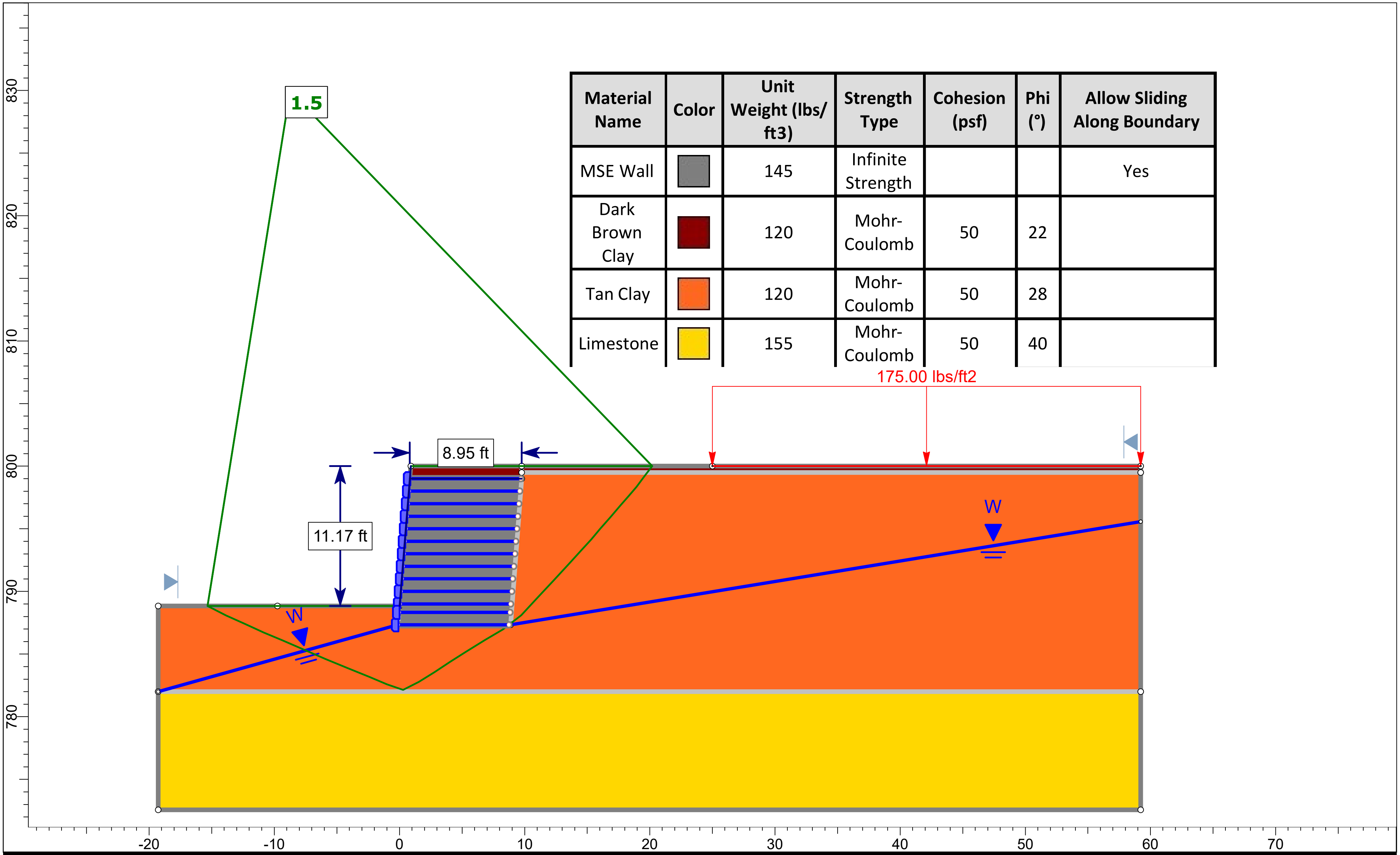
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2









Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary
MSE Wall	⬛	145	Infinite Strength			Yes
Dark Brown Clay	🟥	120	Mohr-Coulomb	50	22	
Tan Clay	🟧	120	Mohr-Coulomb	50	28	
Limestone	🟨	155	Mohr-Coulomb	50	40	

RABA KISTNER

SLIDEINTERPRET 9.034

Project

PRECINCT 30 - UNITS 2 AND 3
Veramendi Master Planned Development
New Braunfels, Texas

Analysis Description

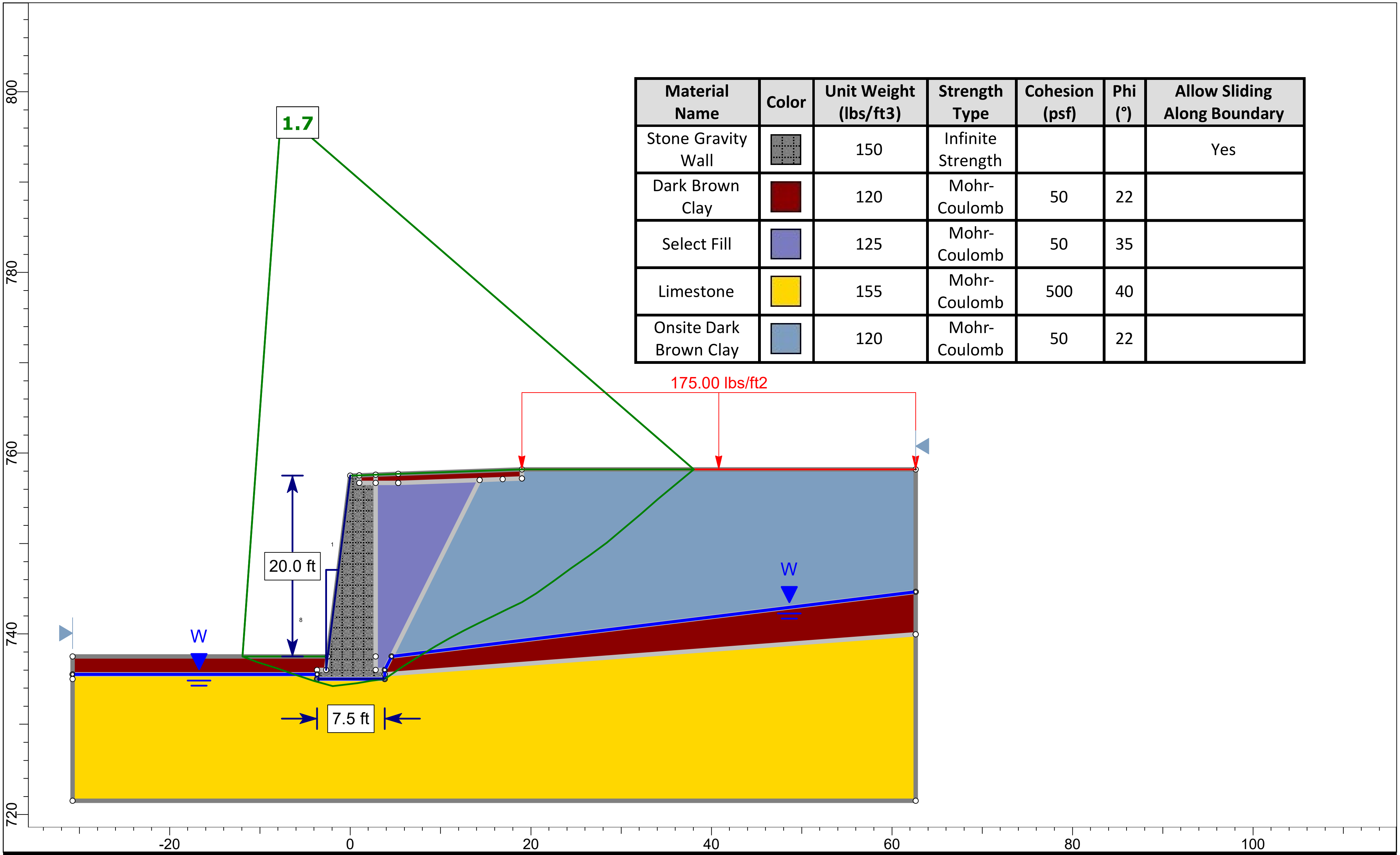
Long Term Condition (Cut Condition)

Date

07/25/2024

Figure

Figure 18a



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (°)	Allow Sliding Along Boundary
Stone Gravity Wall	▨	150	Infinite Strength			Yes
Dark Brown Clay	🟥	120	Mohr-Coulomb	50	22	
Select Fill	🟪	125	Mohr-Coulomb	50	35	
Limestone	🟨	155	Mohr-Coulomb	500	40	
Onsite Dark Brown Clay	🟦	120	Mohr-Coulomb	50	22	

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SLIDEINTERPRET 9.034

Project

PRECINCT 30 - UNITS 2 AND 3
Veramendi Master Planned Development
New Braunfels, Texas

Analysis Description

Long Term Condition (Fill Condition)

Date

07/25/2024

Figure

Figure 18b

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

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

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