RABA KISTNER

GEOTECHNICAL ENGINEERING STUDY

FOR

LA CANTERA TOWN CENTER – PHASE I
SAN ANTONIO, TEXAS

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Project No. ASA24-027-00
June 7, 2024

Mr. Preston Hart
Development Partner
Phoenix Property Company
5625 Village Glen Drive, Suite 200
Dallas, Texas 75206

RE: Geotechnical Engineering Study
La Cantera Town Center – Phase I
San Antonio, Texas

Dear Mr. Hart:

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. PSA24-019-00, dated February 9, 2024. The purpose of this study was to drill borings within the proposed development, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations as well as to provide pavement design and construction guidelines.

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

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

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Santosh Shrestha, E.I.T.
Graduate Engineer

Ian Perez, P.E.
Vice President

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Attachments

Copies Submitted: Electronic (1)

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For

LA CANTERA TOWN CENTER – PHASE I
SAN ANTONIO, TEXAS

Prepared for

PHEONIX PROPERTY COMPANY
Dallas, Texas

Prepared by

RABA KISTNER, INC.
San Antonio, Texas

PROJECT NO. ASA24-027-00

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RABAKISTNER

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ATTACHMENTS

The following figures are attached and complete this report:

INTRODUCTION

RABA KISTNER, Inc. (**RKI**) has completed the authorized subsurface exploration for the proposed multifamily development to be located on Via La Cantera, west of Old Fredericksburg Road in San Antonio, 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, as well as for pavement design and construction guidelines.

PROJECT DESCRIPTION

To be considered in this study is a new 4-acre multifamily development to be located on Via La Cantera, west of Old Fredericksburg Road in San Antonio, Texas. The property is located on the north side of Via La Cantera with an existing gas line as the eastern limit of the property and future “Street A” will be the western boundary (not considered in this study). The development is anticipated to be comprised of several buildings which will include approximately 350 apartment units, restaurant/retail space, and a parking garage. The southern half of the development is anticipated to be podium style while the northern half will wrap the parking garage. Skywalks will interconnect the buildings and a majority of the parking will be in the garage and the podium portion of the building with limited surface parking. The development is anticipated to occupy a majority of the 4-acre tract of land. The proposed structure is anticipated to create relatively moderate loads to be carried by the foundation system.

We understand that the vertical relief across the site is approximately 45 ft, sloping downwards towards the south. We also understand that the bottom levels of the development will step up several times to accommodate the grade change and to reduce the anticipated cut/fill required to accommodate the development.

Our understanding of the proposed facility layout and grading is based on a drawing provided to us via email by Mr. Bryson Marshall with Pheonix Property Company on February 9, 2024. The exhibit presents the existing topography, the proposed grading, and planned Finished Floor Elevations (FFEs) for different levels of the multifamily building. On the basis of the site plan provided to us, the topographic high and low within the project site are approximately 1,053 and 1,008 ft, respectively. The topographic high and low within the footprint of the proposed building are approximately 1,050 and 1,015 ft, respectively with proposed FFEs ranging from 1,012 to 1,040 ft.

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 Phoenix Property Company (Client) and its representatives for design purposes. This report may not contain sufficient information for the purpose 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, our understanding of the project information provided to us, and our previous experience in the vicinity of this site. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

This report may not reflect the actual variations of the subsurface conditions across the site. The nature and extent of variations across the site may not become evident until construction commences. The construction process itself may also alter subsurface conditions. If variations appear evident at the time of construction, it may be necessary to reevaluate our recommendations after performing on-site observations and tests to establish the engineering impact of the variations.

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

If final grade elevations are significantly different from those previously stated, 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 hand-held, recreational-grade GPS locator. Boring elevations, as annotated on the boring logs, were estimated from the provided site plan. The borings were drilled to depths ranging from approximately 20 to 40 ft below the existing ground surface using a truck-mounted drilling rig. During drilling operations split-spoon samples (with Standard Penetration Testing, SPT) 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 the moisture content, percent passing a No. 200 sieve, and Atterberg Limits.

The laboratory test results 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 log is presented on Figure 12. The results of the laboratory and field testing are also tabulated on Figure 13 for ease of reference. The results from the DCP field testing are presented in Figure 14.

Standard Penetration Test results (N-values) are noted as “blows per ft” on the boring logs and on Figure 13. The N-value is the number of blows required to drive a split-spoon sampler 1 ft into soil/weak rock with a falling, 140-lb hammer following 6 inches of seating blows. Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal (“ref”) will be noted on the boring logs and on Figure 13.

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

GENERAL SITE CONDITIONS

SITE GRADING

A topography map was provided, along with anticipated Finished Floor Elevations (FFE). Site grading plans can result in changes in almost all aspects of foundation recommendations. A summary of the boring elevations and FFE for structures are summarized in the following table:

Multifamily	Boring No.	Approximate Existing Elevation (ft, msl)	Proposed FFE Elevation (ft, msl)
Level 1	B-8	1019.0	1012.0
	B-9	1019.0	
	B-10	1013.0	
Level 2	B-5	1030.0	1019.0
	B-6	1028.0	
	B-7	1023.0	
Level 3	B-4	1037.0	1029.0
Level 4	B-1	1048.0	1040.0
	B-2	1042.0	
	B-3	1033.0	

The approximate existing ground surface elevations at the boring locations were estimated from the site grading plan provided to us. The boring locations should be surveyed if greater accuracy is required for the purposes of this study. If anticipated grades change by more than plus or minus 1 ft, **RKI** must be retained to review the site grading plans prior to bidding the project for construction. This will enable **RKI** to provide input for any changes in our original recommendations that may be required as a result of site grading operations or other considerations.

GEOLOGY

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

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

SEISMIC COEFFICIENTS

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

- Site Class Definition: **Class C**. Based on the soil borings conducted for this investigation and our experience in the area, the upper 100 ft of soil may be characterized as very dense soil and soft rock.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 0.2-Second Spectral Response Acceleration (5% Of Critical Damping): **S_s = 0.049g**.
- Risk-Targeted Maximum Considered Earthquake Ground Motion Response Accelerations for the Conterminous United States of 1-Second Spectral Response Acceleration (5% Of Critical Damping): **S_1 = 0.023g**.
- Values of Site Coefficient: **F_a = 1.3**
- Values of Site Coefficient: **F_v = 1.5**
- Where g is the acceleration due to gravity.

The Maximum Considered Earthquake Spectral Response Accelerations are as follows:

- 0.2 sec, adjusted: **S_{ms} = 0.064g**
- 1 sec, adjusted: **S_{m1} = 0.035g**

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

- 0.2 sec SA: **S_{DS} = 0.043g**
- 1 sec SA: **S_{D1} = 0.023g**

STRATIGRAPHY

The natural subsurface stratigraphy can generally be described as plastic to highly plastic tan to dark brown clay with limestone fragments overlying the tan and gray limestone. The limestone/bedrock was encountered at approximate depths ranging from 0.5 ft to 13 ft below the ground surface existing at the time of our study (elevation ranging from 1,009.0 to 1,047.5 ft) and extends to at least the boring termination depths. A void approximately 1 ft thick was encountered in Boring B-1 at an approximate depth of 26 ft below the ground surface existing at the time of our study (approximate elevation of 1,022.0 ft). Intermittent reddish brown clay seams were encountered within limestone strata which is common in this formation.

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

The boring logs and related information depict subsurface conditions only at the specific location and time 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. All borings remained dry during the field exploration phase. However, it is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly following periods of precipitation, within weathered seams, or at the soil/bedrock interface. Fluctuations in groundwater levels occur due to variations in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

FOUNDATION ANALYSIS

KARSTIC FEATURES

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

EXPANSIVE SOIL-RELATED MOVEMENTS

The depth of potentially expansive overburden soils ranges from about 0.5 to 13 ft below the existing ground surface in our borings (elevations ranging from 1,009.0 to 1,047.5 ft). The soil overburden is underlain by weathered rock and competent limestone bedrock. The overburden soils contribute solely to the Potential Vertical Rise (PVR) values estimated for this site. 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 PVR. **PVR values of 1 in. or less were estimated considering the cut/fill required to achieve the various proposed FFEs.** A surcharge load of 1 psi (concrete slab and sand layer), a soil active zone to the top of limestone, and dry moisture conditions were assumed in estimating the above PVR value.

To reduce the risk for potential soil-related movements (particularly if the building is surrounded by irrigated landscaped areas), consideration should be given to **completely removing the potentially expansive soils to the top of the bedrock, if any**. With this consideration, we recommend that all of the overburden soils be completely removed from within and 3 ft. around the proposed building areas and the overexcavation backfilled with compacted select fill in accordance with the *Select Fil* section of this report.

The overexcavated soils may be reused on site, and beyond the building pad, provided that the potential vertical movements in excess of those discussed previously will not adversely impact either the structural or operational tolerances for the proposed improvements for which this material is being considered.

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 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.

Drainage Considerations

At this site, considerations of surface and subsurface drainage may be crucial to construction and adequate foundation performance of the soil-supported structures. Water entering the fill surface during construction or entering the fill exposed beyond the building lines after construction may create problems with fill moisture control during compaction both during and after construction.

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

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

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

DESIGN RECOMMENDATIONS

Foundation recommendations are based on the conditions existing at the time of our study. Site features that will influence the geotechnical approach to the proposed project include:

- Varying depth to bedrock;
- The depth and quality of bedrock (i.e. limestone) and the potential for karstic features;
- Proposed loads on planned foundations;
- Depending on site grading, the potential for variable subgrades for shallow foundation systems; and
- The amount of cut or fill required to achieve the proposed FFE.

Please note that the foundation capacities presented herein are based on the Allowable Stress Design methodology. In general, the allowable values given herein for foundations can be increased by 33 percent for seismic, wind or other transitory loads (2018 IBC, Section 1806.1).

FOUNDATION OPTIONS

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

Drilled, straight-shaft piers are recommended for the multifamily building. Small, ancillary structures associated with this project may be supported by shallow foundation systems. Depending on the site grading plan, variable bearing subgrades may be encountered and should be carefully considered in design. Cost analyses have not been conducted for any foundation system and are beyond the scope of this study.

Due to the presence of hard materials, high-powered, high-torque drilling equipment will be required to construct piers at this site (See also Section titled *Excavation Equipment*). Only contractors with the equipment and experience in drilling in these hard materials should be considered for the construction of piers on this project.

SHALLOW FOUNDATIONS

Small ancillary structures may be founded on shallow foundations or a stiffened engineered beam and slab foundation, provided the selected foundation type can be designed to withstand the anticipated soil-related movements (see section *Expansive Soil-Related Movements*) without impairing either the structural or the operational performance of the structures.

Differential Settlement in Transition Zone

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

- Frequent jointing of exterior walls;
- Strip footings designed as grade beams to span unsupported lengths up to 10 to 15 feet;
- Selection of flexible building veneer materials; and
- Overexcavation of footing subgrades to top of rock and backfilling with compacted crushed rock.

Allowable Bearing Capacity

Shallow foundations founded on compacted select fill or limestone should be proportioned using the design parameters in the following table.

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

1) If intact limestone is encountered, minimum beam depths should be discussed with the structural engineer, but may be reduced to 12 in.

Footings	Maximum Allowable Bearing Pressure
Grade beam or strip footing (Natural material or compacted select fill)	3,000 psf
Widened beams or spread footing (Natural material or compacted select fill)	3,500 psf
Shallow foundations on intact bedrock (without pilot or probe holes)	4,000 psf (1)

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

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

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

Depending on the structural loads and if higher bearing pressures are requested/desired, rock-bearing shallow foundations proportioned for greater than 5,000 psf bearing pressures may require additional probe borings with pilot holes at actual foundation locations. Alternatively, the requirement for pilot hole or probe holes may be waived if the bearing pressure provided herein is used.

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

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

Uplift Resistance

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

Lateral Resistance

Horizontal loads acting on shallow foundations will be resisted by passive earth pressure acting on one side of the footing and by base adhesion for footings in soil or limestone. Resistance to sliding for foundations bearing on natural/compacted soil or limestone should be calculated utilizing an ultimate coefficient of friction of 0.30 or 0.70, respectively. The ultimate resistance for these foundations should be limited to 1,050 psf (soil) or 2,800 psf (rock). An equivalent fluid pressure of 240 pcf (soil) or 350 pcf (rock) should be utilized to determine the ultimate passive resistance, if required.

FLOOR SLABS

Floor slabs within the superstructure may be ground supported provided the anticipated movements discussed under the *Expansive Soil-Related Movements* section of this report will not impair the performance of the floor, frame, or roof systems.

If differential movements between the slab and the structure are objectionable, soil-supported floor slabs could be dowelled to the perimeter grade beams. Dowelled slabs that are subjected to heaving will typically crack and develop a plastic hinge along a line which will be approximately 5 to 10 ft. inside and parallel to the grade beams. Slabs cast independent of the grade beams, interior columns and partitions should experience minimum cracking, but may create difficulties at critical entry points such as doors and may impact interior partitions that are secured to exterior walls. We recommend that a vapor barrier comprised of polyethylene or polyvinyl chloride (PVC) sheeting be placed between the supporting select fill and the concrete floor slab.

AREA FLATWORK

It should be noted that ground-supported flatwork such as pavements, sidewalks, courtyards, and buried piping, if any, will be subject to the same magnitude of potential soil-related movements as discussed previously (see Expansive Soil-Related Movement section). Thus, where these types of elements abut rigid foundations, differential movements should be anticipated. As a minimum, we recommend that flexible joints or connections be provided where such elements abut the main structure to allow for differential movement at these locations.

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

DRILLED PIERS

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

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

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

An allowable end-bearing pressure of 30 ksf may be utilized for piers where pilot holes are performed. This bearing pressure was calculated using a factor of safety of 3. If pilot holes are not performed, then we recommend piers be designed using side friction only. We recommend that drilled, straight-shaft piers extend a minimum of 5 ft into native, intact limestone. An allowable side shear resistance of 3.5 ksf may be utilized for the portion of the shaft extending to a minimum depth of 5 ft or 2 pier shaft diameters, whichever results in the lower elevation, into the native, intact limestone layer. This is based on a factor of safety of 2 with respect to the design shear strength. These values may be increased by 1/3 for transient load conditions.

Side shear should be neglected in fill material, clay layers, voids, and/or clay filled voids. Based on the data obtained as a part of this study, we recommend that the tip elevation for the drilled piers be at or above elevation 979 ft, msl unless prior approval is received from **RKI** and/or observations of the pier/pilot holes confirm the presence of native, intact limestone the full depth of the pier and below.

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

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

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

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

Pier Shaft Potential Uplift Forces

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

$F_u = 10*D$ (Based on cut/fill required to achieve the proposed FFEs)

where:

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

Allowable Uplift Resistance

Resistance to uplift forces exerted on the drilled, straight-shaft piers will be provided by the sustained compressive axial force (dead load) plus the allowable uplift resistance provided by the bedrock. The allowable uplift resistance provided by the bedrock at this site may be estimated using 2 ksf for that portion of the shaft penetrating the limestone, respectively, and neglecting the upper 5 ft of the shaft.

Reinforcing steel will be required in each pier shaft to withstand a net force equal to the uplift force minus the sustained compressive load carried by that pier. We recommend that each pier be reinforced to withstand this net force.

Pier Spacing

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

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

Lateral Resistance

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

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

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

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

Where:

C = undrained cohesion
k_s = p-y modulus
ε_{50} = strain factor
γ = total unit weight
γ’ = effective unit weight
q_u = unconfined compressive strength

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

The parameters presented in the above table do **not** include factors of safety nor have they been factored. It should be noted that where piers are spaced closer than three shaft diameters center to center, a modification factor should be applied to the p-y curves to account for a group effect. We recommend the following p-Multipliers for the corresponding center to center pier spacing to determine factored lateral loads. **Allowable end bearing and side shear resistance reduction factors should not utilize the reduction**

factors presented above and will need to be evaluated on a case-by-case basis once pier geometries, spacing, and loads are available for our review.

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

RETAINING STRUCTURES

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

LATERAL EARTH PRESSURES

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

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

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

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

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

BACKFILL COMPACTION

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

We expect the total settlement in the order of 1 percent of the total fill thickness, provided the fill is selected and placed in accordance with the *Select Fill* section of this report. We anticipate that half of this settlement will occur during the construction. Differential settlement can reasonably be estimated to be on the order of 1/2 of the total settlement.

To reduce potential settlement of adjacent foundations/flatwork, the backfill materials should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test, or 98 percent of maximum density as determined by ASTM D698. **To reduce the potential settlement, fills greater than 8 ft should be compacted to at least 95 percent of maximum density as determined by ASTM D 1557, Modified Compaction Test and should be crushed limestone conforming to the 2014 TxDOT Standard Specifications, Item 247 – Flexible Base, Type A, Grade 1-2**. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction. Note that free-draining gravel materials are not typically tested for density and moisture content, but rather monitored by observation. Each lift or layer of the backfill should be tested during the backfilling operations to document the degree of compaction. Within at least a 5-ft zone of the walls, we recommend that compaction be accomplished using hand-guided compaction equipment capable of achieving the maximum density in a series of 3 to 5 passes. Thinner lifts may be required to achieve the required level of compaction.

DRAINAGE

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

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

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

VERTICAL ROCK CUTS

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

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

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

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

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

Where:

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

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

FOUNDATION CONSTRUCTION CONSIDERATIONS

SITE DRAINAGE

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

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

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

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

SITE PREPARATION

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

Upon completion of the proofrolling operations and just prior to fill placement or slab construction, the exposed subgrade should be moisture conditioned by scarifying to a minimum depth of 6 in. and recompacting to a minimum of 95 percent of the maximum density determined from TxDOT, Tex-114-E 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. This consideration may be waived if the subgrade consists of limestone bedrock.

ON-SITE SOIL

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

ON-SITE ROCK FILL

If excavations extend into the Edwards Limestone formation, consideration can be given to utilizing the excavated material 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 below for select fill materials. If on-site materials cannot be processed to meet the required criteria, imported select fill materials should be utilized.

SELECT FILL

Materials used as select fill preferably should be crushed stone or gravel aggregate. At a minimum, we recommend that onsite/imported, crushed limestone base be utilized in the upper 2 ft or the full depth of the grade beams, whichever results in the lower elevation. Recommendations for Granular Select Fill materials are provided below:

> **On-site/Imported Crushed Limestone Base** – Onsite/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 B, Grades 1-2 or 3.

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

Select Fill Placement and Compaction

Select fill should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-113-E, Compaction Test, or 98 percent of maximum density as determined by ASTM D698. Fills greater than 8 ft should be compacted to at least 95 percent of maximum density as determined by ASTM D 1557, Modified Compaction Test. 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 Placement and Compaction

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

SHALLOW FOUNDATION EXCAVATIONS

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

DRILLED PIERS

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

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

If clay seams and/or voids are encountered within the limestone formation during drilled shaft excavations, the shafts must be extended to develop the required side shear resistance. For bid purposes, the owner should anticipate that deeper piers will be required in some areas. Consequently, contractors bidding on the job should include unit costs for various depths of additional pier embedment. Unit costs should include those for both greater and lesser depth in both rock and soil.

Reinforcement and Concrete Placement

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

EXCAVATIONS AND TEMPORARY SLOPES

Depending on the planned improvement depths, 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 overburden soils encountered during excavations for the proposed project are anticipated to consist of relatively stiff fine-grained soils. Hence, temporary slopes should be classified as OSHA Type B 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.

OSHA guidelines for temporary slopes performed in Type B materials should be constructed at 1 Vertical (V): 1 Horizontal (H) or flatter. 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 borings 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 slab-on-grade, slab-on-fill, or any other rigid unit should be designed with either some degree of flexibility or with sleeves. Such design features will help reduce the risk of damage to the utility lines as vertical movements occur.

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

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

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized, and all backfilling procedures should be tested and documented.
- Curbs should completely penetrate base materials and be installed to a sufficient depth to reduce water infiltration beneath the curbs into the pavement base materials.

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

PAVEMENT RECOMMENDATIONS

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

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

SUBGRADE CONDITIONS

We have assumed the subgrade in pavement areas will consist of recompacted on-site clays; hereinafter referred to as the “Clay Subgrade,” placed and compacted as recommended in the On-Site Clay Fill section of this report; or native intact limestone; hereinafter referred to as the “Rock Subgrade.” Based on our experience with similar subgrade soils and rock, we have assigned California Bearing Ratio (CBR) values of 3.0 and 10.0 for the Clay Subgrade and Rock Subgrade, respectively.

DESIGN INFORMATION

The following recommendations were prepared using the 1993 “Guide for the Design of Pavement Structures” by the American Association of State Highway and Transportation Officials (AASHTO). The following recommendations were prepared assuming a 20-yr design life and Equivalent Single Axle Loads (ESAL’s) listed in the table below. Our traffic estimates were based on our prior experience with similar projects. The following Equivalent Single Axle Loads (ESALs) were estimated for light and medium traffic types. **The Owner and/or the Project Civil Engineer should review anticipated traffic loading and frequencies to verify that the assumed traffic loading and frequency is appropriate for the intended use of the facility.**

Traffic Type	Use	Flexible 20-yr ESALs	Rigid 20-yr ESALs
Light Duty	Passenger Vehicle parking Areas	25,000	45,000
Medium Duty	Passenger Vehicle Drives and Entrances or traffic equivalent to 5 single trailer trucks per day	89,400	160,000

Flexible Pavements	
Pavement Design Parameters	**Input Variables**
Reliability (%)	70
Initial Serviceability	4.2
Terminal Serviceability	2.0
Overall Standard Deviation	0.45
Roadbed Soil Resilient Modulus	4,500 psi(1)

1) Determined from CBR correlation.

Rigid Pavements	
Pavement Design Parameters	**Input Variables**
Reliability (%)	70
Initial Serviceability	4.5
Terminal Serviceability	2.0
Modulus of Subgrade reaction (k-value)	75 pci
Overall Standard Deviation	0.35
28-day Concrete Modulus of Rupture	550 psi
28-day Concrete Elastic Modulus	4,000,000 psi
Load Transfer Coefficient	3.7
Drainage Coefficient	1.01

FLEXIBLE PAVEMENT

The minimum flexible pavement sections recommended for this site are as listed in the table below:

Traffic Type	Layer Description (1)	Layer Thickness
Light Duty	Surface Course, Type "D" Flexible Base Treated Subgrade **Combined Total**	2.0 in. 8.0 in. 6.0 in. **16.0 in.**
Medium Duty	Surface Course, Type "D" Flexible Base Treated Subgrade **Combined Total**	3.0 in. 9.0 in. 6.0 in. **18.0 in.**

1) The flexible base layer may be reduced to 6 inches and treated subgrade may be eliminated if subgrade consisting of bedrock is encountered.

Flexible Pavement Consideration

Based on our experience, the reported flexible pavement sections often perform adequately; however, maintenance or an overlay is generally needed sooner than would be required for a thicker design section. Consideration could be given to adding additional asphalt (i.e. an additional 1 in.) or incorporating a geogrid below the flexible base. 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.

Another option to help reduce the potential for cracking and maintenance in asphalt pavements is including reinforcing fibers, such as Forta-Fi®, into the Hot Mix Asphalt (HMA). These are options and are not required. The geogrid reinforcement should conform to TxDOT Type 2 geogrid, or an approved substitute. If geogrid or reinforcing fibers are used in the provided options, we do not recommend reducing the report sections without further discussion with the design team. **Geogrid may also be considered at the rock/clay transition zones to reduce hinging caused by differential movement between the soils and rock subgrade. If considered, the geogrid should extend approximately 5 ft into the transitioned material.**

RIGID PAVEMENT

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

Traffic Type	Portland Cement Concrete
Light Duty	5 in.
Medium Duty	6 in.

Rigid Pavement Consideration

We recommend Jointed Plain Concrete Pavement (JPCP) be utilized for the rigid pavement sections. JPCP typically does not require distributed steel, micro- or macro-fibers, or any other "reinforcing" material. The following recommendations are based on ACI 330R-08 "Guide for the Design and Construction of Concrete parking Lots."

Typical joint types in JPCP include control (contraction) joints, isolation joints (sometimes called expansion joints), and construction joints. The recommended joint spacing is 30 times the thickness of the slab up to a maximum of 15 ft. The length of a slab or panel should not be more than 25% greater than its width. For pavements with a thickness of 7 in. or greater (if any), dowels may be required along all control joints. Tie bars may be required at the first longitudinal joint from the pavement edge to keep the outside edge from separating from the pavement.

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.

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 section should be able to support occasional fire trucks for a design period of 20 years. Flexible pavement sections for the medium-duty traffic recommended on the *"Flexible Pavement"* and *"Rigid Pavement"* sections of the report may be used.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

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

ON-SITE SOILS

As discussed previously, the pavement recommendations presented in this report were prepared assuming that on-site soils will be used for fill grading in proposed pavement areas. If used, we recommend that on-site soils be placed in loose lifts not exceeding 8 in. in thickness and 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 water content to 3 percentage points above the optimum water content until permanently covered. We recommend that fill materials be free of roots and other organic or degradable material. We also recommend that the maximum particle size not exceed 4 in. or one half the lift thickness, whichever is smaller.

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 completely penetrate base materials and should be installed to sufficient depth to reduce infiltration of water beneath the curbs**.
- 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.

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 or cement should be mixed with the subgrade soils to reduce the soil plasticity index to 20 or less. For estimating purpose, we recommend that at least 4 percent hydrated lime or 4 percent cement treatment by weight be used to increase the pH of the subgrade clays to 12.4 or higher. **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.

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to TxDOT 2014 Standard Specifications, Item 247, Type A, Grade 1-2. Base course should be placed in loose lifts not exceeding 8 in. in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT Tex-113-E Compaction Test, or 98 percent of maximum density as determined by ASTM D698. The moisture content of the fill should be maintained within the range of 2 percentage points below to 2 percentage points above the optimum moisture content until final compaction.

ASPHALTIC CONCRETE SURFACE COURSE

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

PORTLAND CEMENT CONCRETE

The Portland cement concrete should have a minimum 28-day compressive strength of 4,000 psi. A liquid membrane-forming curing compound should be applied as soon as practical after finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

MISCELLANEOUS PAVEMENT RELATED CONSIDERATIONS

Longitudinal Cracking

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

Pavements that do not have a protective barrier to reduce moisture fluctuation of the 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 embankments. This problem can best be addressed by providing either a horizontal or vertical moisture barrier at the unprotected pavement edge.

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 the owner of the facility immediately seal the cracks and develop a periodic sealing program.

Curb and Gutter

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

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 maintenance techniques should be followed as required.

Construction Traffic

Construction traffic on prepared subgrade or granular 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 prior to the complete construction of the pavement section. Heavy traffic loads should not be allowed on light duty traffic areas either before or after completion of the pavement section.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

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

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

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

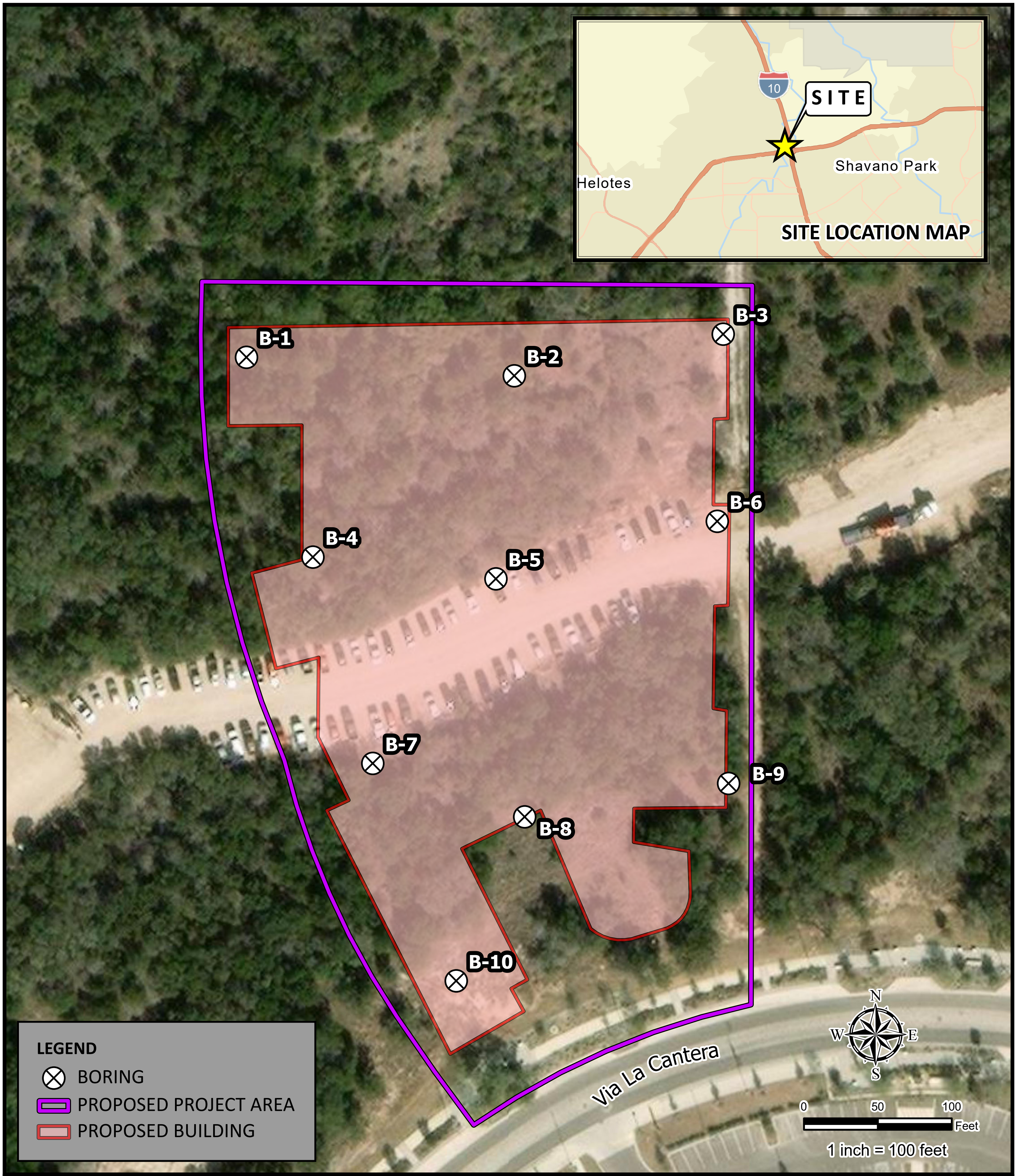
BUDGETING FOR CONSTRUCTION TESTING

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

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

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

ATTACHMENTS



RABA KISTNER

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San Antonio, Texas 78249

(210)699-9090 TEL
(210)699-6426 FAX
www.rkci.com

TBPE Firm Number 3257

SOURCE: Aerial imagery from Google Earth Pro - 2021

BORING LOCATION MAP

LA CANTERA TOWN CENTER – PHASE I
SAN ANTONIO, TEXAS

PROJECT No.: ASA24-027-00

ISSUE DATE:	3/24/2024
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

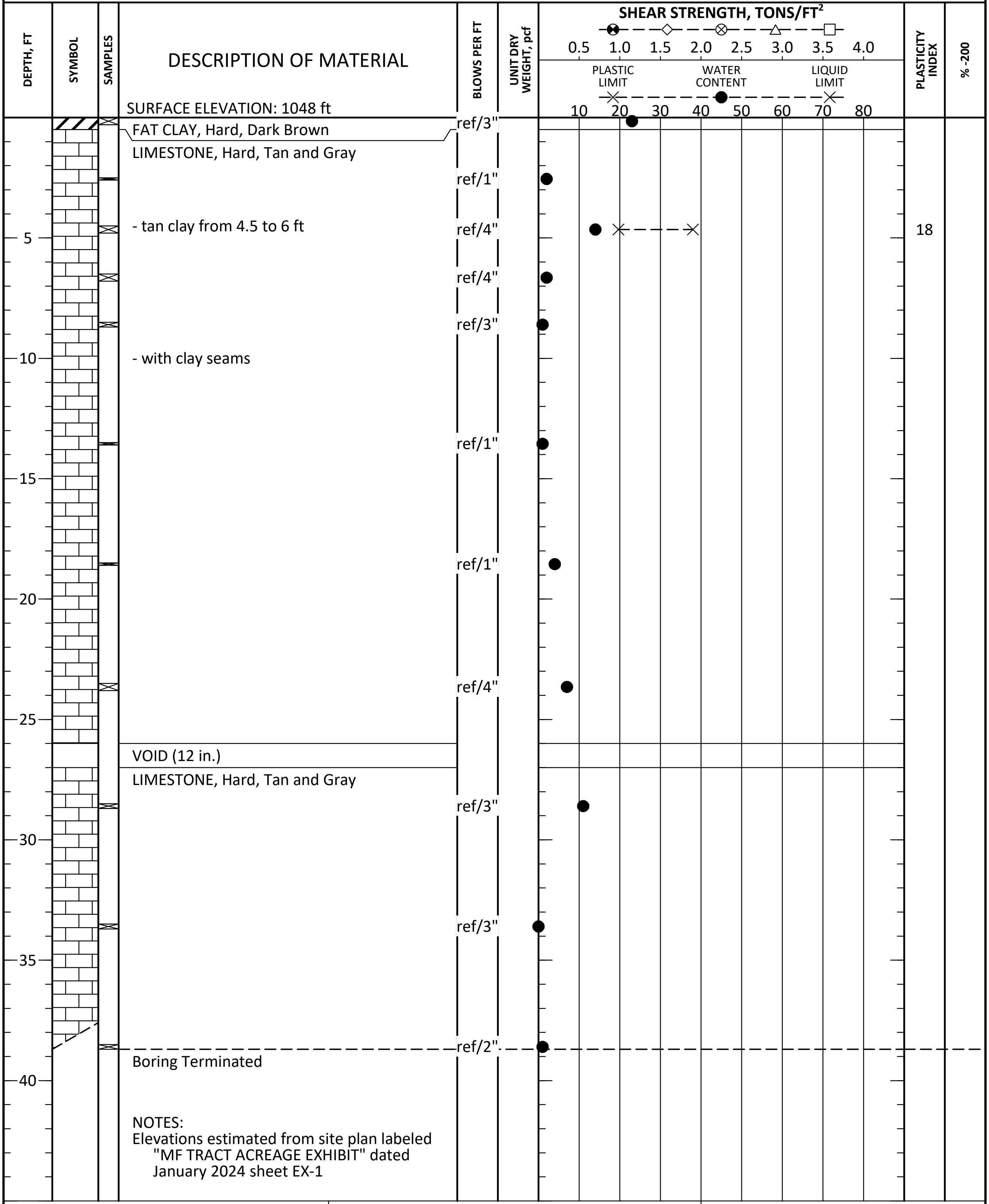
LOG OF BORING NO. B-1

La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59653; W 98.60409



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX
FAT CLAY, Hard, Dark Brown	ref/3"	
LIMESTONE, Hard, Tan and Gray	ref/1"	
- tan clay from 4.5 to 6 ft	ref/4"	18
	ref/4"	
	ref/3"	
- with clay seams		
	ref/1"	
	ref/1"	
	ref/4"	
VOID (12 in.)		
LIMESTONE, Hard, Tan and Gray	ref/3"	
	ref/3"	
Boring Terminated	ref/2"	

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

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

DEPTH DRILLED:	38.7 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA24-027-00
DATE DRILLED:	5/14/2024	**DATE MEASURED:**	5/14/2024	**FIGURE:**	2

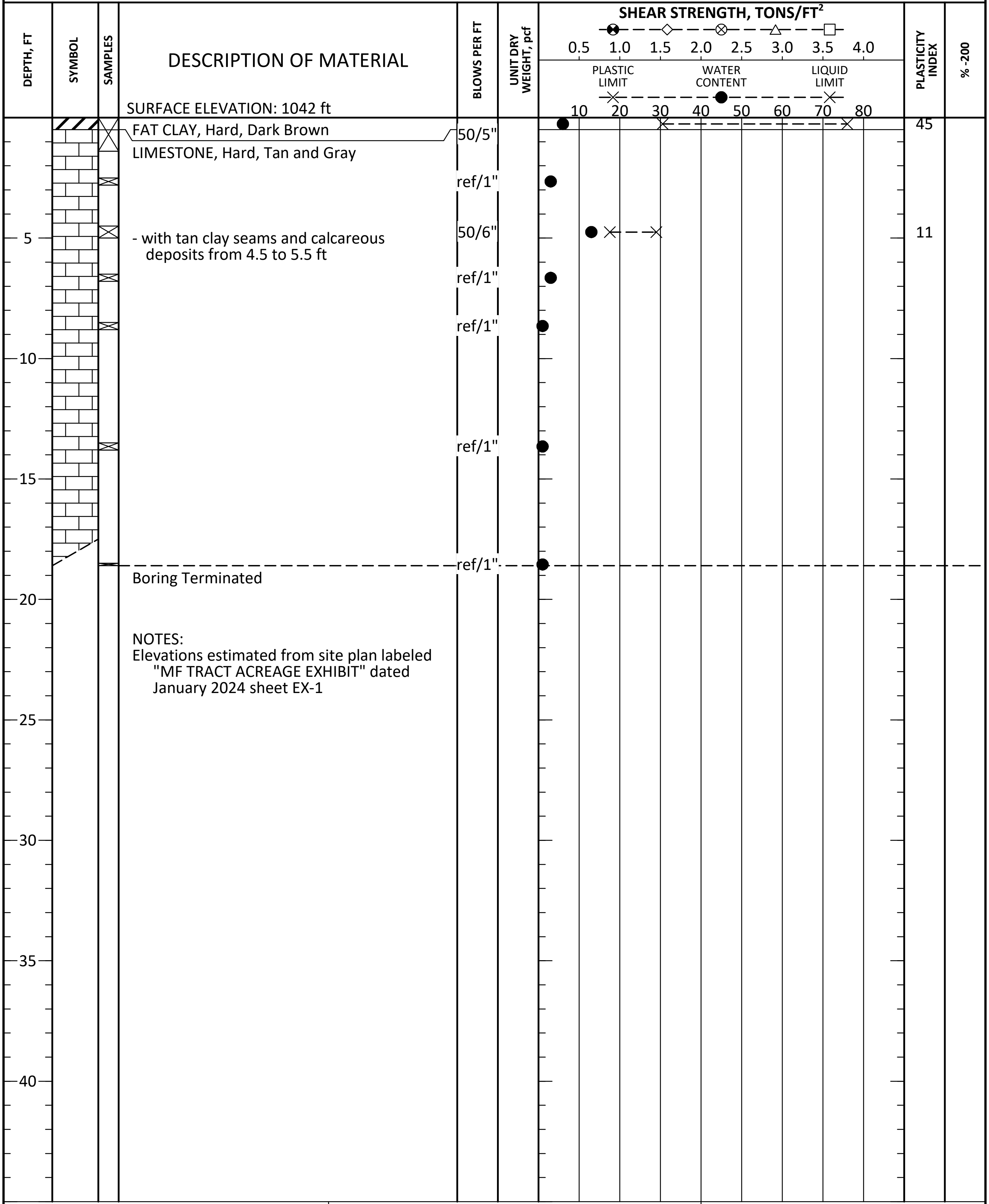
LOG OF BORING NO. B-2

La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59650; W 98.60352



FAT CLAY, Hard, Dark Brown

LIMESTONE, Hard, Tan and Gray

- with tan clay seams and calcareous deposits from 4.5 to 5.5 ft

Boring Terminated

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

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

DEPTH DRILLED: 18.6 ft
DATE DRILLED: 5/13/2024
DEPTH TO WATER: Dry
DATE MEASURED: 5/13/2024
PROJ. No.: ASA24-027-00
FIGURE: 3

LOG OF BORING NO. B-3

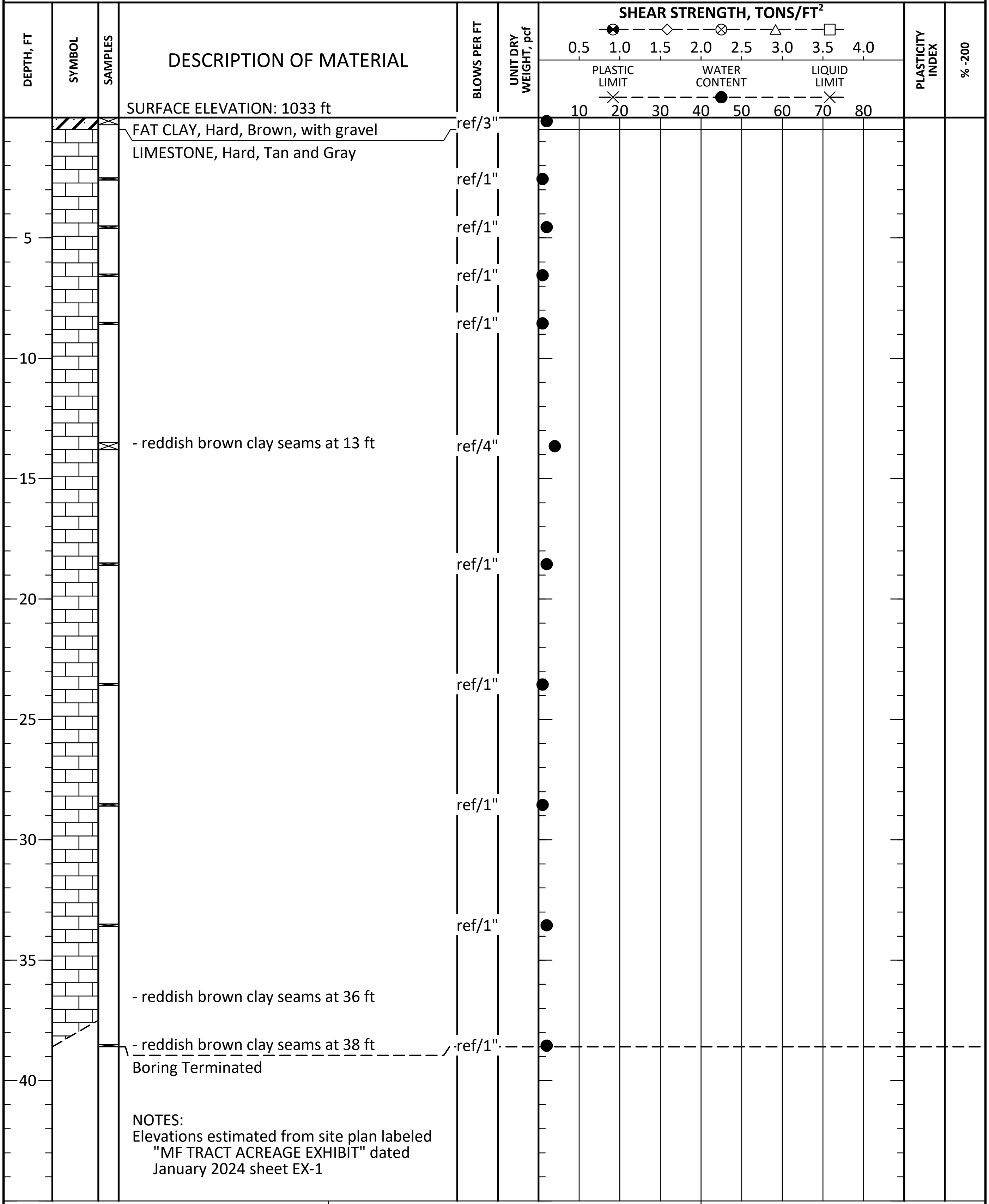
La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59657; W 98.60308

SURFACE ELEVATION: 1033 ft



FAT CLAY, Hard, Brown, with gravel

LIMESTONE, Hard, Tan and Gray

- reddish brown clay seams at 13 ft

- reddish brown clay seams at 36 ft

- reddish brown clay seams at 38 ft

Boring Terminated

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

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

DEPTH DRILLED: 38.6 ft
DATE DRILLED: 5/9/2024
DEPTH TO WATER: Dry
DATE MEASURED: 5/9/2024
PROJ. No.: ASA24-027-00
FIGURE: 4

LOG OF BORING NO. B-4

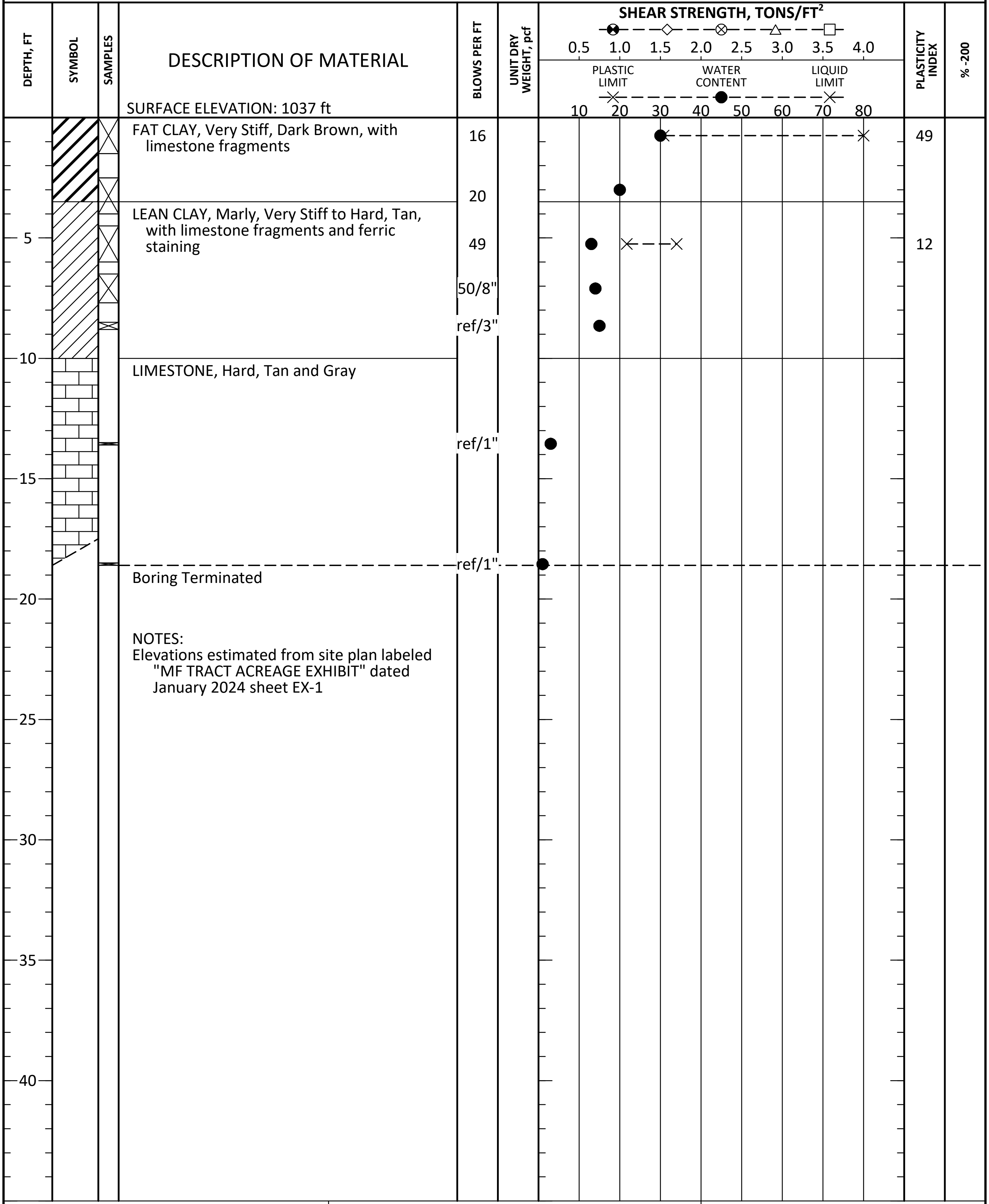
La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59616; W 98.60395

SURFACE ELEVATION: 1037 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX
FAT CLAY, Very Stiff, Dark Brown, with limestone fragments	16 20	49
LEAN CLAY, Marly, Very Stiff to Hard, Tan, with limestone fragments and ferric staining	49 50/8" ref/3"	12
LIMESTONE, Hard, Tan and Gray	ref/1" ref/1"	

Boring Terminated

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

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

DEPTH DRILLED: 18.6 ft
DATE DRILLED: 5/16/2024
DEPTH TO WATER: Dry
DATE MEASURED: 5/16/2024
PROJ. No.: ASA24-027-00
FIGURE: 5

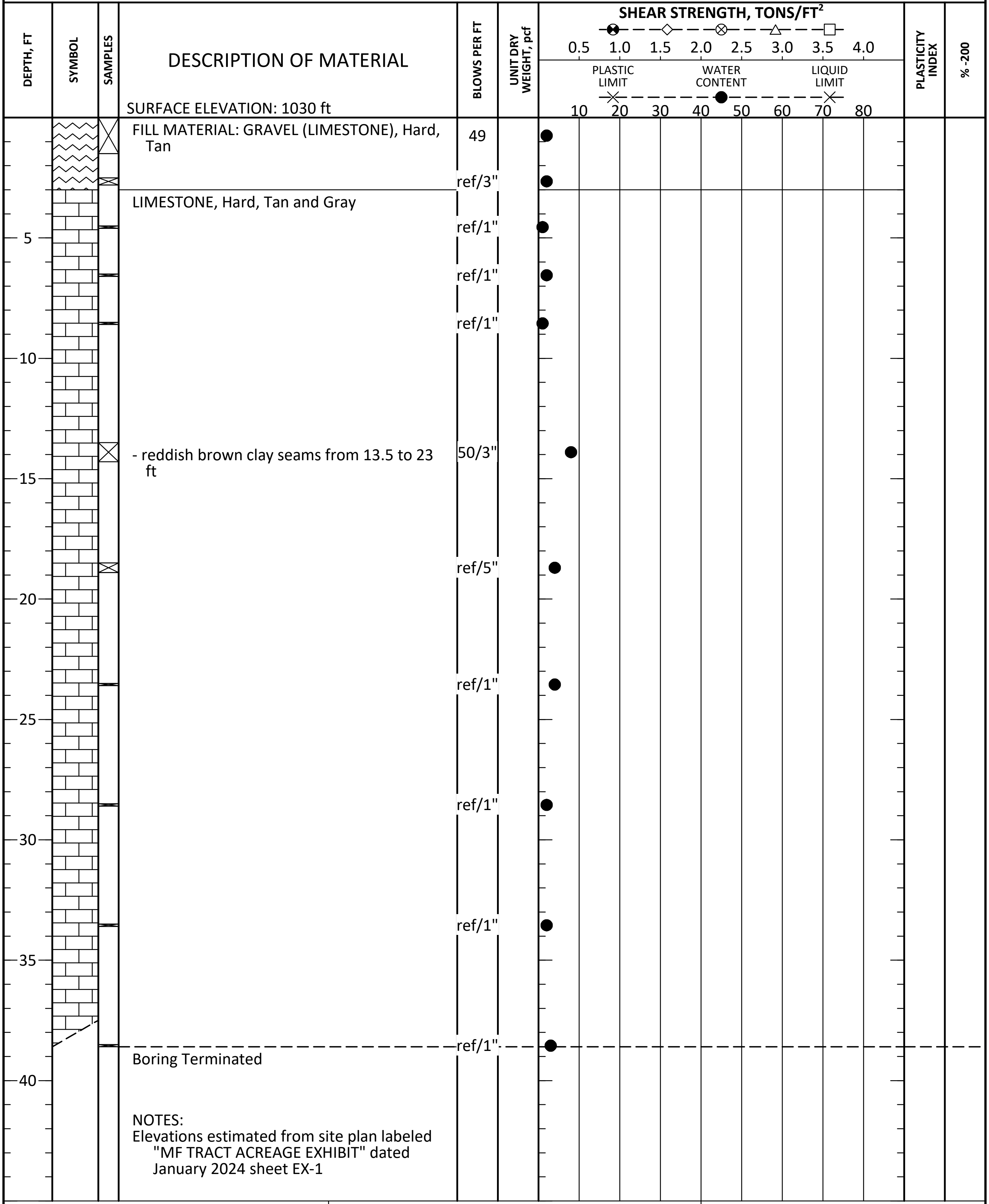
LOG OF BORING NO. B-5

La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.59612; W 98.60356



SURFACE ELEVATION: 1030 ft

FILL MATERIAL: GRAVEL (LIMESTONE), Hard, Tan

LIMESTONE, Hard, Tan and Gray

- reddish brown clay seams from 13.5 to 23 ft

Boring Terminated

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

DEPTH DRILLED: 38.6 ft
DATE DRILLED: 5/13/2024
DEPTH TO WATER: Dry
DATE MEASURED: 5/13/2024
PROJ. No.: ASA24-027-00
FIGURE: 6

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

LOG OF BORING NO. B-6

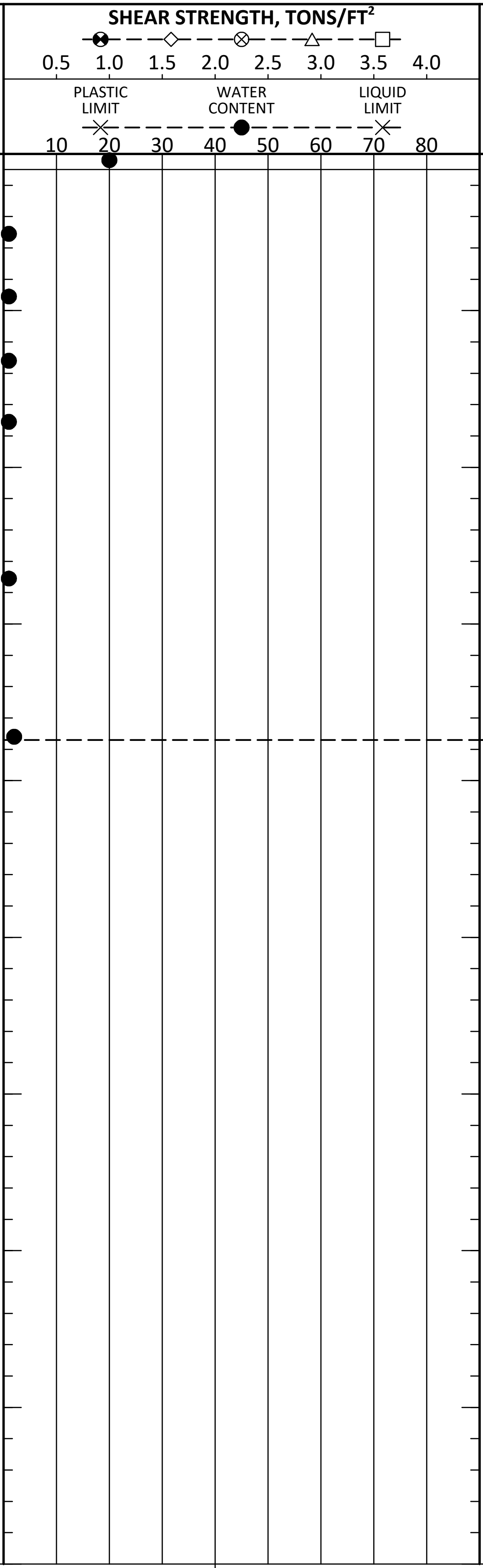
La Cantera Town Center - Phase I
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger and Air Rotary

LOCATION: N 29.59622; W 98.60309

DESCRIPTION OF MATERIAL	BLOWS PER FT
SURFACE ELEVATION: 1028 ft	
LEAN CLAY, Sandy, Hard, Brown	ref/5"
LIMESTONE, Hard, Tan and Gray	
	ref/1"
	ref/1"
	ref/2"
	ref/1"
	ref/1"
	ref/2"
Boring Terminated	



NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

DEPTH DRILLED: 18.7 ft
DATE DRILLED: 5/6/2024
DEPTH TO WATER: Dry
DATE MEASURED: 5/6/2024
PROJ. No.: ASA24-027-00
FIGURE: 7

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

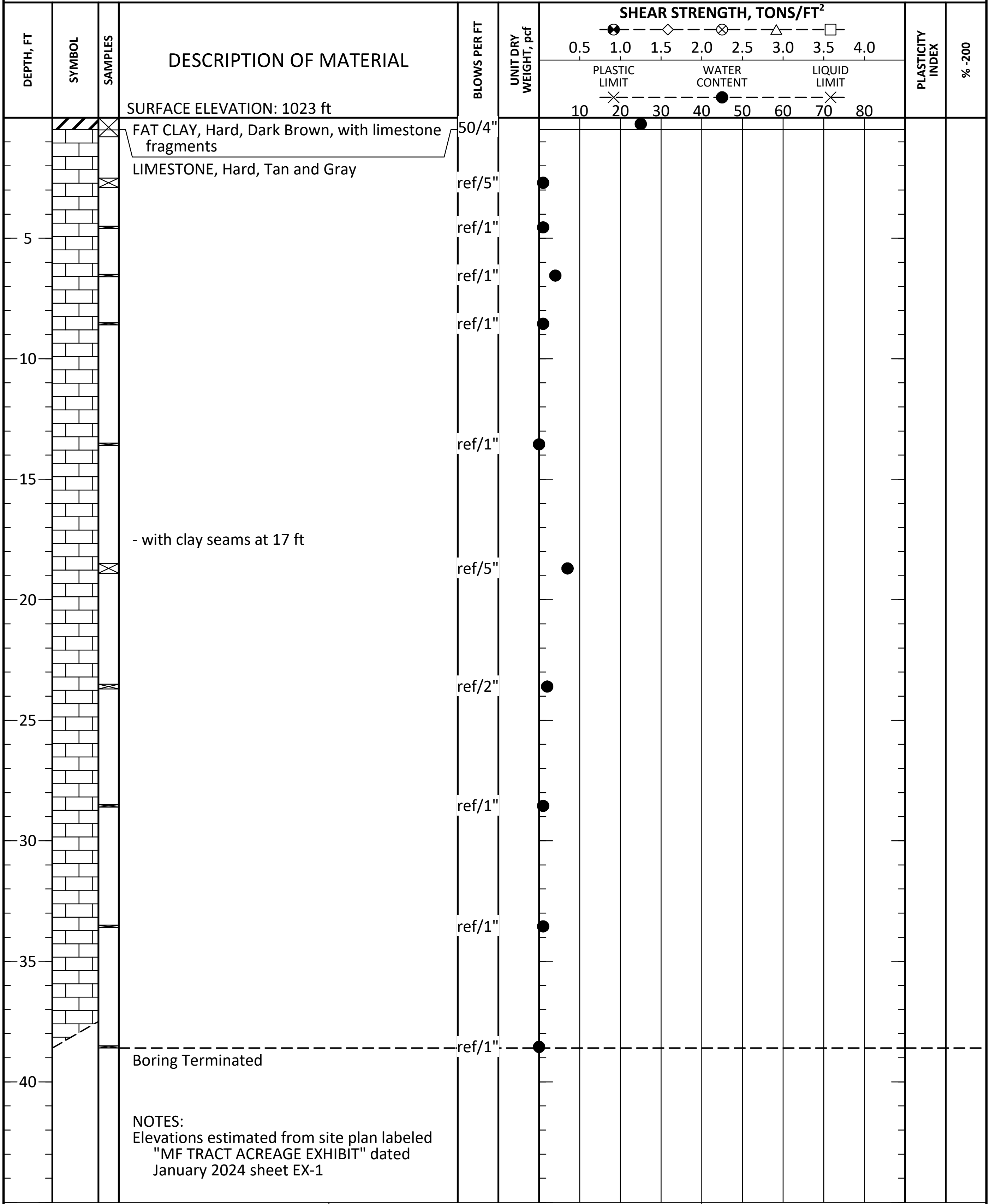
LOG OF BORING NO. B-7

La Cantera Town Center - Phase I
San Antonio, Texas

TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59578; W 98.60382



SURFACE ELEVATION: 1023 ft

FAT CLAY, Hard, Dark Brown, with limestone fragments

LIMESTONE, Hard, Tan and Gray

- with clay seams at 17 ft

Boring Terminated

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

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

DEPTH DRILLED: 38.6 ft
DATE DRILLED: 5/14/2024
DEPTH TO WATER: Dry
DATE MEASURED: 5/14/2024
PROJ. No.: ASA24-027-00
FIGURE: 8

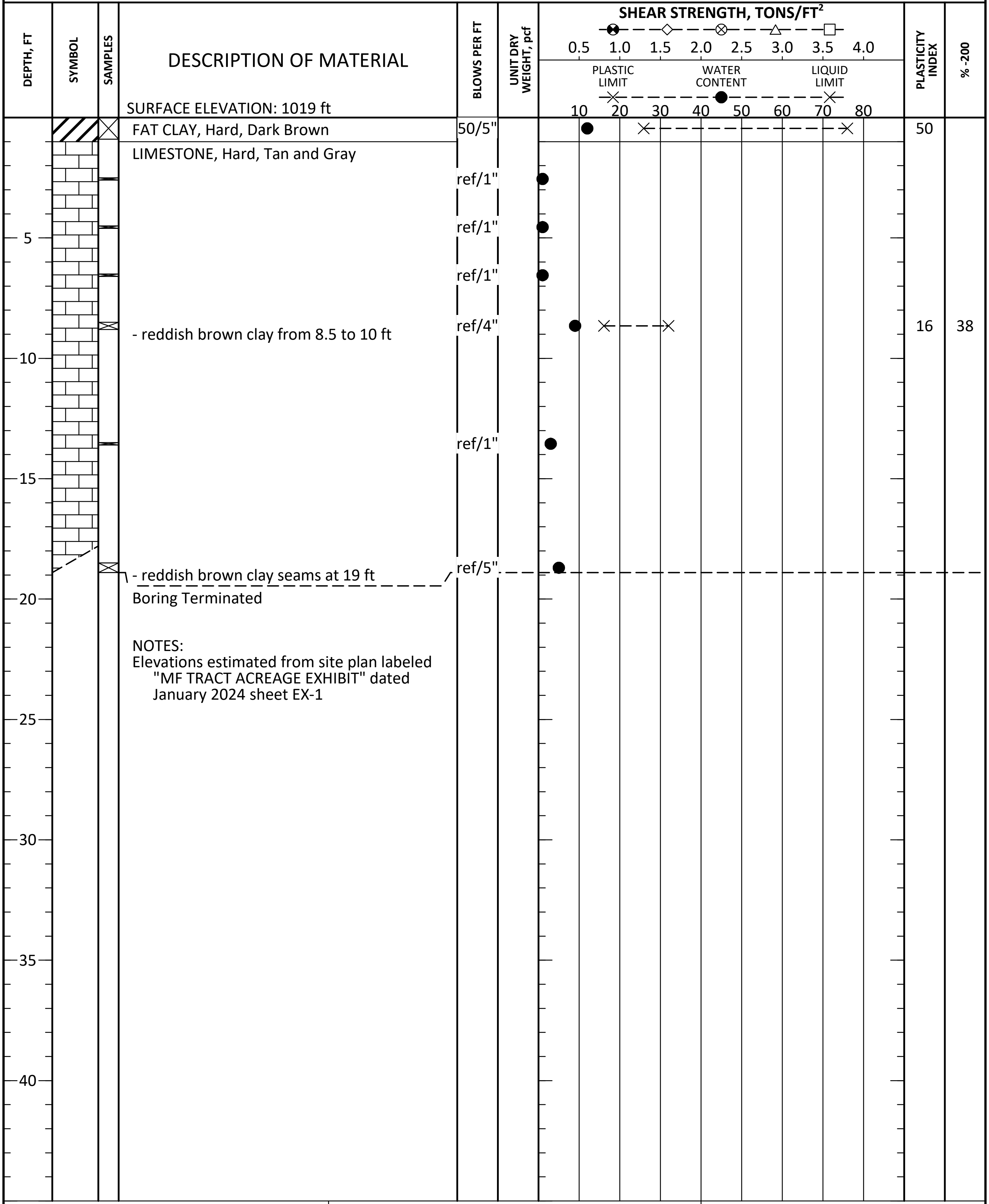
LOG OF BORING NO. B-8

La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59562; W 98.60350



SURFACE ELEVATION: 1019 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX	% -200
FAT CLAY, Hard, Dark Brown	50/5"	50	
LIMESTONE, Hard, Tan and Gray	ref/1"		
	ref/1"		
	ref/1"		
- reddish brown clay from 8.5 to 10 ft	ref/4"	16	38
	ref/1"		
- reddish brown clay seams at 19 ft	ref/5"		

Boring Terminated

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

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

DEPTH DRILLED: 18.9 ft
DATE DRILLED: 5/9/2024
DEPTH TO WATER: Dry
DATE MEASURED: 5/9/2024
PROJ. No.: ASA24-027-00
FIGURE: 9

LOG OF BORING NO. B-9

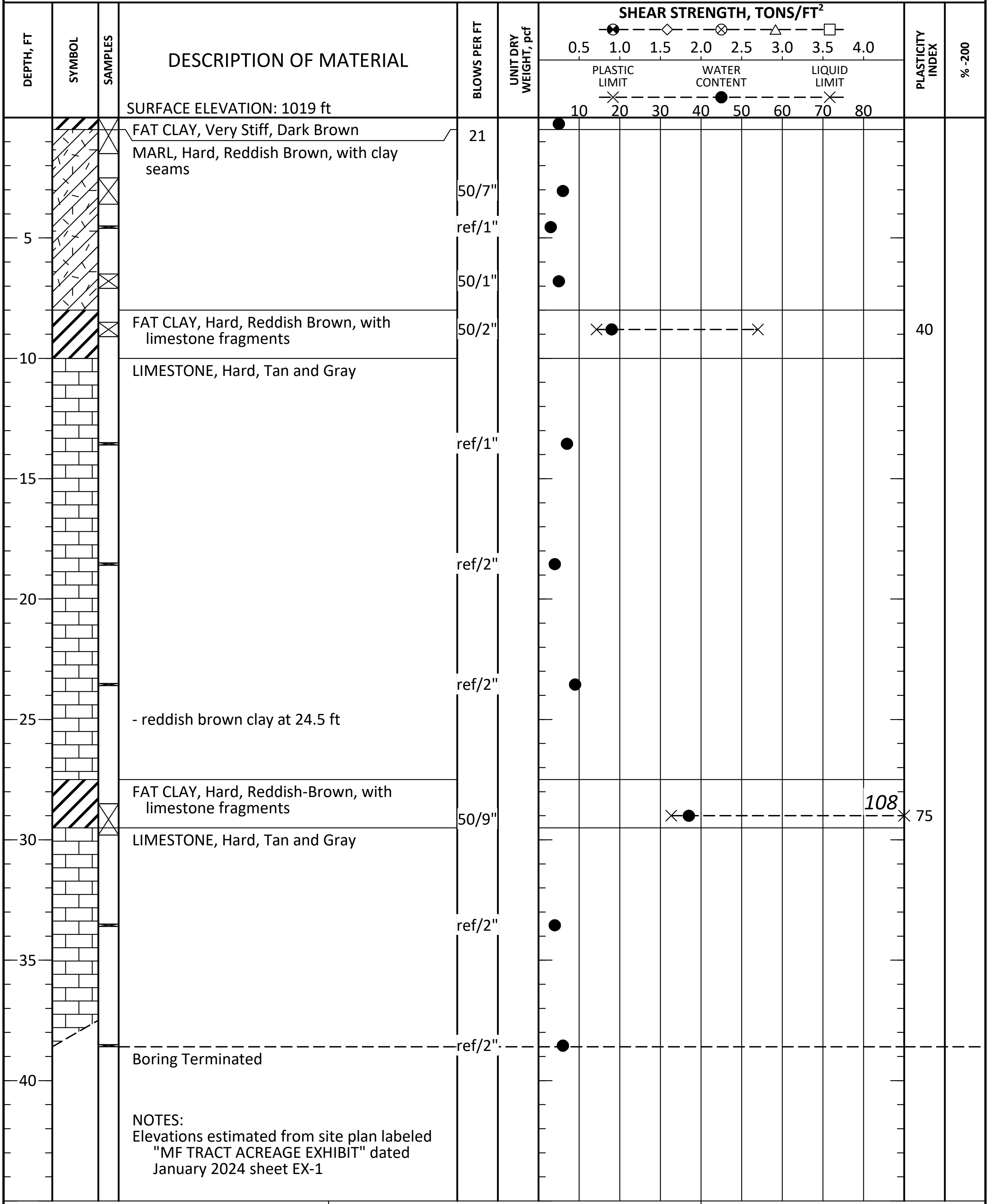
La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59572; W 98.60304

SURFACE ELEVATION: 1019 ft



DESCRIPTION OF MATERIAL	BLOWS PER FT	PLASTICITY INDEX
FAT CLAY, Very Stiff, Dark Brown	21	
MARL, Hard, Reddish Brown, with clay seams	50/7"	
	ref/1"	
	50/1"	
FAT CLAY, Hard, Reddish Brown, with limestone fragments	50/2"	40
LIMESTONE, Hard, Tan and Gray	ref/1"	
	ref/2"	
	ref/2"	
- reddish brown clay at 24.5 ft		
FAT CLAY, Hard, Reddish-Brown, with limestone fragments	50/9"	75
LIMESTONE, Hard, Tan and Gray	ref/2"	
	ref/2"	
Boring Terminated		

NOTES:
Elevations estimated from site plan labeled "MF TRACT ACREAGE EXHIBIT" dated January 2024 sheet EX-1

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

DEPTH DRILLED:	38.6 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA24-027-00
DATE DRILLED:	5/9/2024	**DATE MEASURED:**	5/9/2024	**FIGURE:**	10

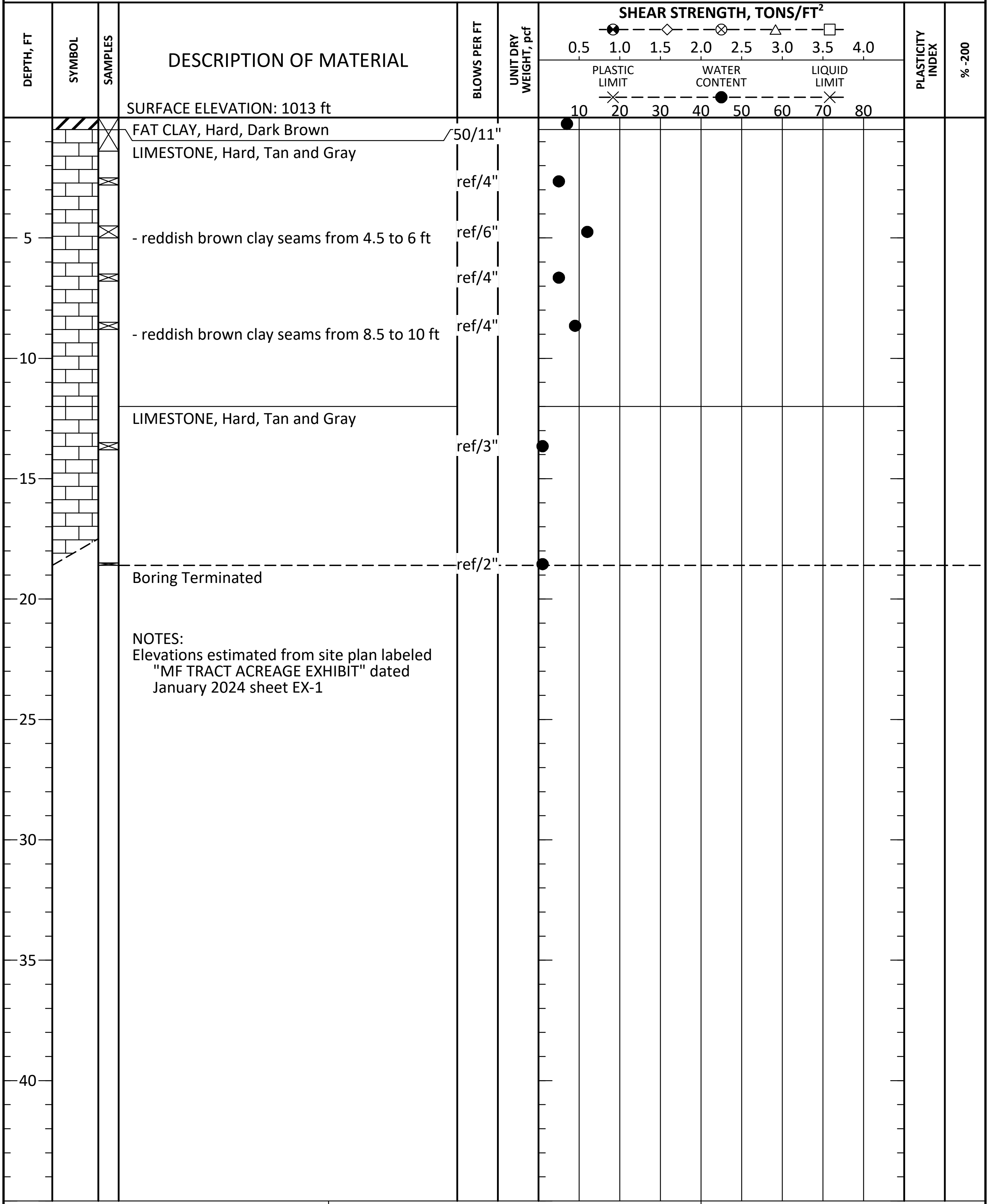
LOG OF BORING NO. B-10

La Cantera Town Center - Phase I
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Air Rotary

LOCATION: N 29.59537; W 98.60365



DESCRIPTION OF MATERIAL

SURFACE ELEVATION: 1013 ft

DESCRIPTION OF MATERIAL	BLOWS PER FT
FAT CLAY, Hard, Dark Brown	50/11"
LIMESTONE, Hard, Tan and Gray	ref/4"
- reddish brown clay seams from 4.5 to 6 ft	ref/6"
	ref/4"
- reddish brown clay seams from 8.5 to 10 ft	ref/4"
LIMESTONE, Hard, Tan and Gray	ref/3"
	ref/2"
Boring Terminated	

NOTES:
Elevations estimated from site plan labeled
"MF TRACT ACREAGE EXHIBIT" dated
January 2024 sheet EX-1

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

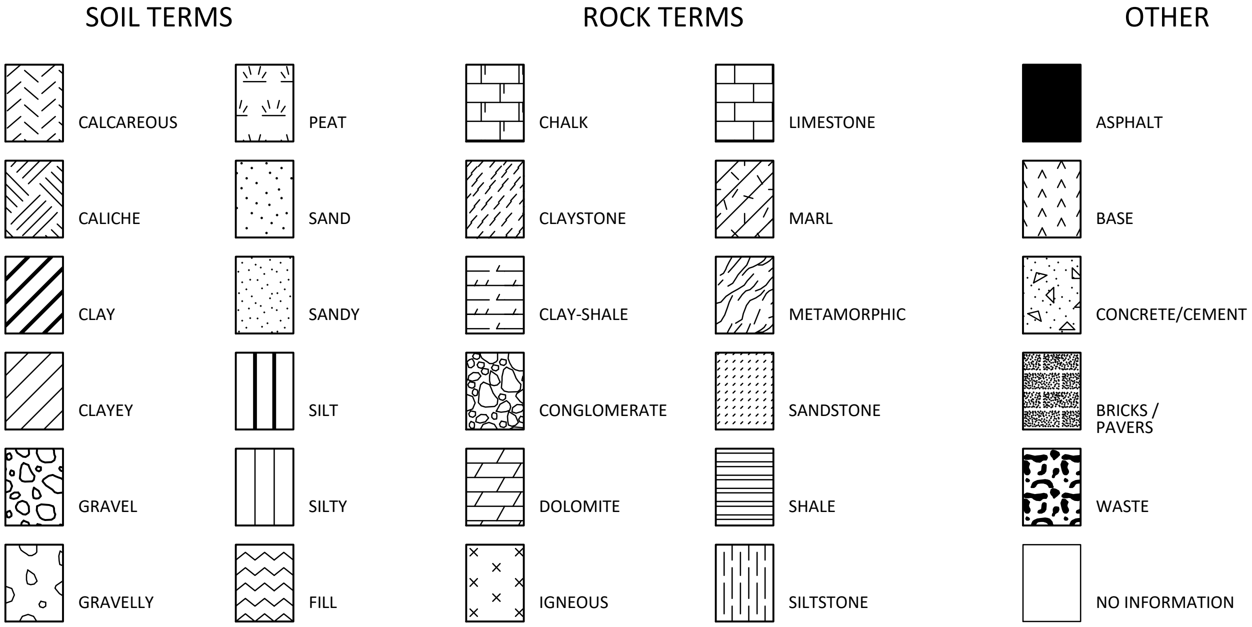
DEPTH DRILLED: 18.6 ft
DATE DRILLED: 5/9/2024

DEPTH TO WATER: Dry
DATE MEASURED: 5/9/2024

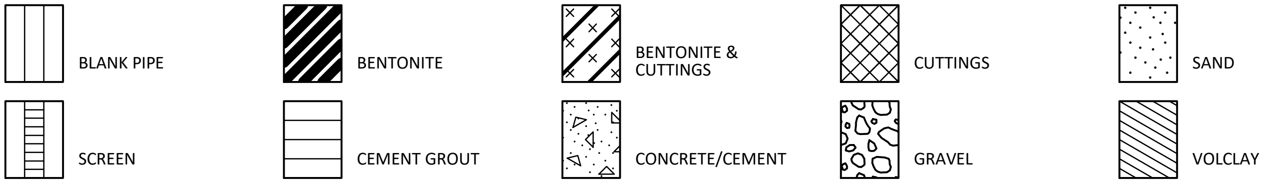
PROJ. No.: ASA24-027-00
FIGURE: 11

KEY TO TERMS AND SYMBOLS

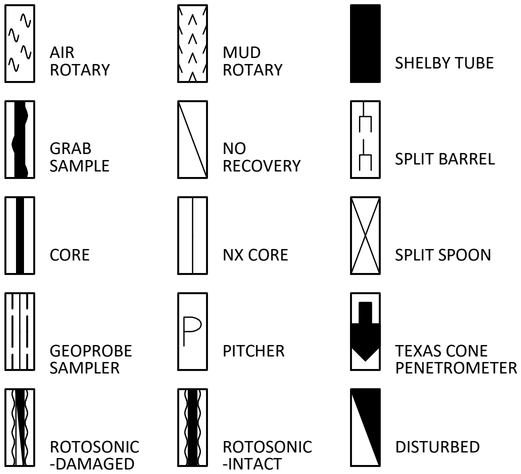
MATERIAL TYPES



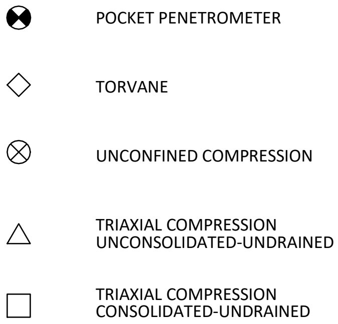
WELL CONSTRUCTION AND PLUGGING MATERIALS



SAMPLE TYPES



STRENGTH TEST TYPES



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

PROJECT NO. ASA24-027-00

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

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

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

RELATIVE DENSITY

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

COHESIVE STRENGTH

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

PLASTICITY

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

ABBREVIATIONS

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

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

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

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

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

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

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

STANDARD PENETRATION TEST (SPT)

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

PROJECT NO. ASA24-027-00

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: La Cantera Town Center - Phase I
San Antonio, Texas

FILE NAME: ASA24-027-00.GPJ 6/6/2024

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 0.3	ref/3"	23								
	2.5 to 2.6	ref/1"	2								
	4.5 to 4.8	ref/4"	14	38	20	18	CL				
	6.5 to 6.8	ref/4"	2								
	8.5 to 8.7	ref/3"	1								
	13.5 to 13.6	ref/1"	1								
	18.5 to 18.6	ref/1"	4								
	23.5 to 23.8	ref/4"	7								
	28.5 to 28.7	ref/3"	11								
	33.5 to 33.7	ref/3"	0								
	38.5 to 38.7	ref/2"	1								
B-2	0.0 to 1.4	50/5"									
	0.0 to 0.5		6	76	31	45	CH				
	2.5 to 2.8	ref/1"	3								
	4.5 to 5.0	50/6"	13	29	18	11	CL				
	6.5 to 6.8	ref/1"	3								
	8.5 to 8.8	ref/1"	1								
	13.5 to 13.8	ref/1"	1								
	18.5 to 18.6	ref/1"	1								
B-3	0.0 to 0.3	ref/3"	2								
	2.5 to 2.6	ref/1"	1								
	4.5 to 4.6	ref/1"	2								
	6.5 to 6.6	ref/1"	1								
	8.5 to 8.6	ref/1"	1								
	13.5 to 13.8	ref/4"	4								
	18.5 to 18.6	ref/1"	2								
	23.5 to 23.6	ref/1"	1								
	28.5 to 28.6	ref/1"	1								
	33.5 to 33.6	ref/1"	2								
	38.5 to 38.6	ref/1"	2								
B-4	0.0 to 1.5	16	30	80	31	49	CH				
	2.5 to 4.0	20									
	2.5 to 3.5		20								
	4.5 to 6.0	49	13	34	22	12	CL				
	6.5 to 7.7	50/8"	14								
	8.5 to 8.8	ref/3"	15								
	13.5 to 13.6	ref/1"	3								
	18.5 to 18.6	ref/1"	1								
B-5	0.0 to 1.5	49	2								

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

CU = Consolidated Undrained Triaxial

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: La Cantera Town Center - Phase I
San Antonio, Texas

FILE NAME: ASA24-027-00.GPJ 6/6/2024

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	2.5 to 2.8	ref/3"	2								
	4.5 to 4.6	ref/1"	1								
	6.5 to 6.6	ref/1"	2								
	8.5 to 8.6	ref/1"	1								
	13.5 to 14.3	50/3"	8								
	18.5 to 18.9	ref/5"	4								
	23.5 to 23.6	ref/1"	4								
	28.5 to 28.6	ref/1"	2								
	33.5 to 33.6	ref/1"	2								
	38.5 to 38.6	ref/1"	3								
B-6	0.0 to 0.4	ref/5"	20								
	2.5 to 2.6	ref/1"	1								
	4.5 to 4.6	ref/1"	1								
	6.5 to 6.7	ref/2"	1								
	8.5 to 8.6	ref/1"	1								
	13.5 to 13.6	ref/1"	1								
	18.5 to 18.7	ref/2"	2								
B-7	0.0 to 0.8	50/4"									
	0.0 to 0.5		25								
	2.5 to 2.9	ref/5"	1								
	4.5 to 4.6	ref/1"	1								
	6.5 to 6.6	ref/1"	4								
	8.5 to 8.6	ref/1"	1								
	13.5 to 13.6	ref/1"	0								
	18.5 to 18.9	ref/5"	7								
	23.5 to 23.7	ref/2"	2								
	28.5 to 28.6	ref/1"	1								
	33.5 to 33.6	ref/1"	1								
	38.5 to 38.6	ref/1"	0								
B-8	0.0 to 0.9	50/5"	12	76	26	50	CH				
	2.5 to 2.6	ref/1"	1								
	4.5 to 4.6	ref/1"	1								
	6.5 to 6.6	ref/1"	1								
	8.5 to 8.8	ref/4"	9	32	16	16	SC		38		
	13.5 to 13.6	ref/1"	3								
	18.5 to 18.9	ref/5"	5								
B-9	0.0 to 1.5	21									
	0.0 to 0.5		5								
	2.5 to 3.6	50/7"	6								

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: La Cantera Town Center - Phase I
San Antonio, Texas

FILE NAME: ASA24-027-00.GPJ 6/6/2024

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-9	4.5 to 4.6	ref/1"	3								
	6.5 to 7.1	50/1"	5								
	8.5 to 9.1	50/2"	18	54	14	40	CH				
	13.5 to 13.6	ref/1"	7								
	18.5 to 18.6	ref/2"	4								
	23.5 to 23.6	ref/2"	9								
	28.5 to 29.8	50/9"									
	28.5 to 29.5		37	108	33	75	CH				
	33.5 to 33.6	ref/2"	4								
	38.5 to 38.6	ref/2"	6								
B-10	0.0 to 1.4	50/11"									
	0.0 to 0.5		7								
	2.5 to 2.8	ref/4"	5								
	4.5 to 5.0	ref/6"	12								
	6.5 to 6.8	ref/4"	5								
	8.5 to 8.8	ref/4"	9								
	13.5 to 13.8	ref/3"	1								
	18.5 to 18.6	ref/2"	1								

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

CU = Consolidated Undrained Triaxial

Project Number: ASA24-027-00
Test Date: May 14, 2024

DCP TEST DATA

B-2

La Cantera Town Center - Phase I
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	58	2.3	3	4.5	1.13
1	1	21	3.1	10	15	2.52
1	1	16	3.7	13	19.5	3.00
1	1	50	5.7	4	6	1.37
1	1	30	6.9	6	9	1.80
1	2	30	8.1	14	21	3.15
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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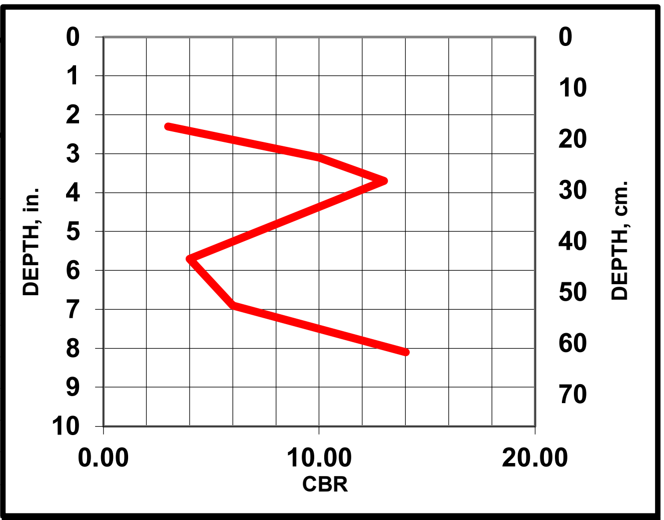
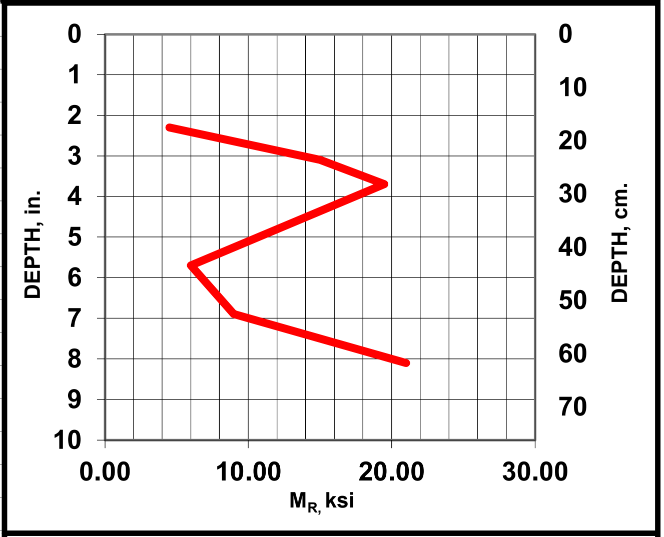
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





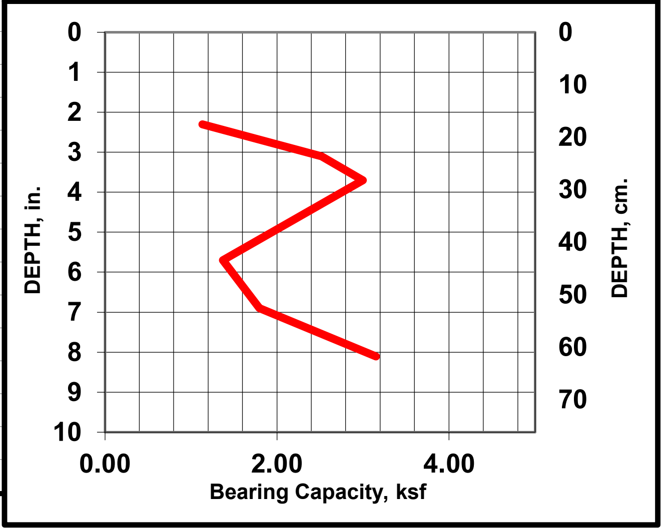


Figure 14a

Project Number: ASA24-027-00
Test Date: May 14, 2024

DCP TEST DATA

B-4

La Cantera Town Center - Phase I
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	41	1.6	5	7.5	1.59
1	1	23	2.5	9	13.5	2.35
1	1	20	3.3	10	15	2.52
1	1	19	4.1	11	16.5	2.69
1	1	17	4.7	12	18	2.84
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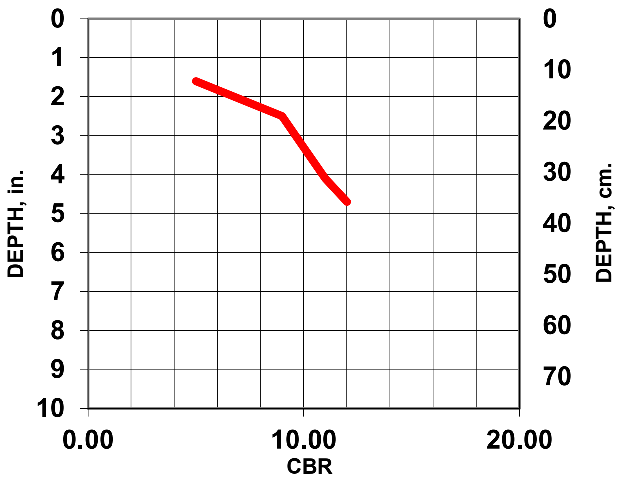
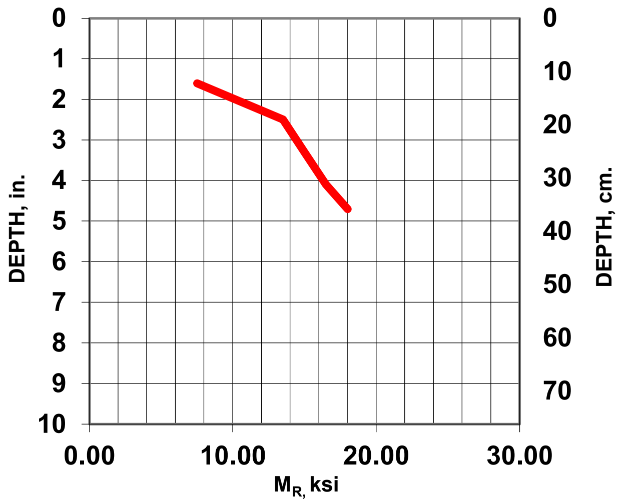
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





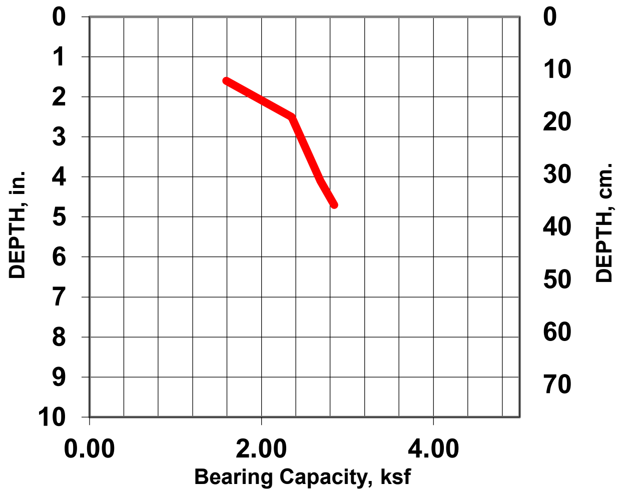


Figure 14b

Project Number: ASA24-027-00
Test Date: May 14, 2024

RABA KISTNER

DCP TEST DATA

B-8

La Cantera Town Center - Phase I
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration		CBR	M_R	q_{ult}
		Incre. (mm)	Cumm. (in)	(%)	(ksi)	(ksf)
1	1	30	1.2	6	9	1.80
1	1	22	2	9	13.5	2.35
1	1	16	2.7	13	19.5	3.00
1	3	20	3.5	35	52.5	5.79
1	2	6	3.7	85	127.5	10.44
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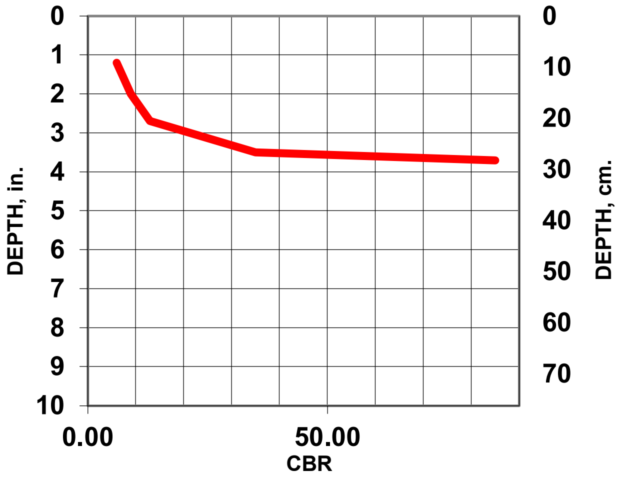
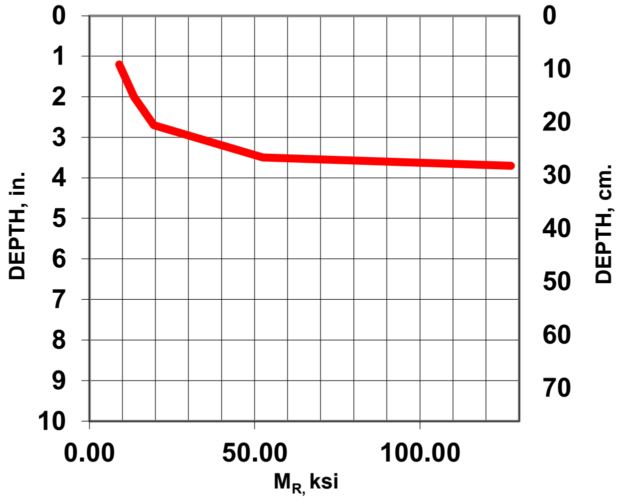
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





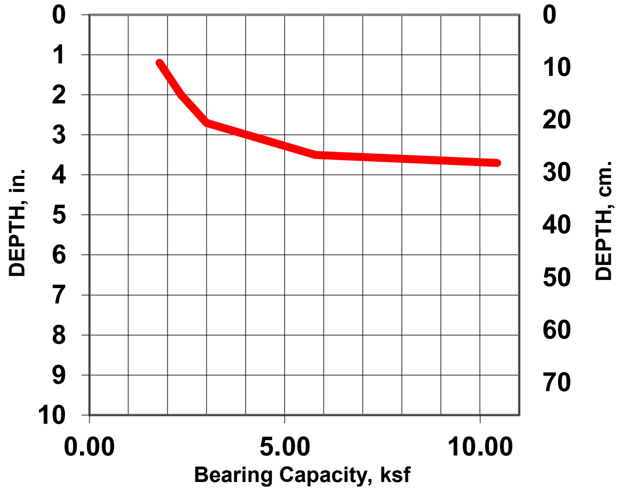


Figure 14c

Project Number: ASA24-027-00
Test Date: May 14, 2024

RABA KISTNER

DCP TEST DATA

B-10

La Cantera Town Center - Phase I
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration		CBR (%)	M_R (ksi)	q_{ult} (ksf)
		Incre. (mm)	Cumm. (in)			
1	1	25	1	8	12	2.17
1	2	66	3.6	6	9	1.80
1	1	29	4.7	7	10.5	1.99
1	1	29	5.9	7	10.5	1.99
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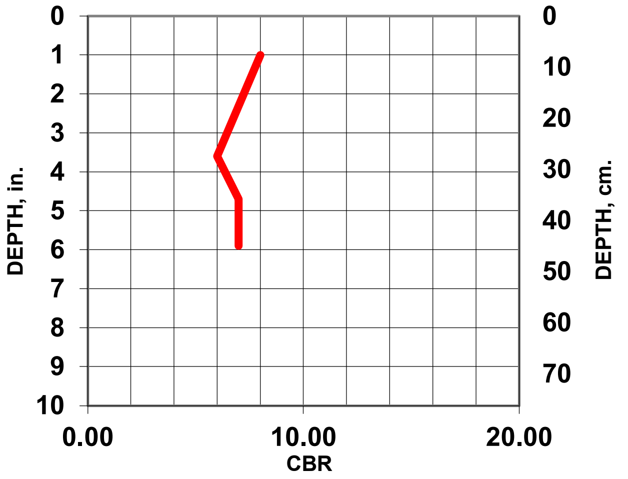
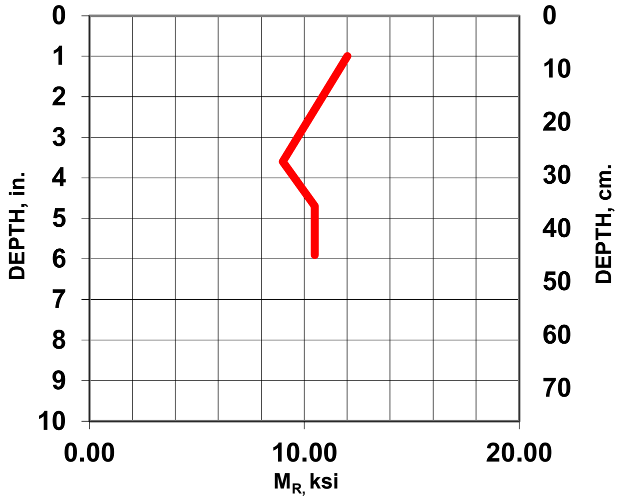
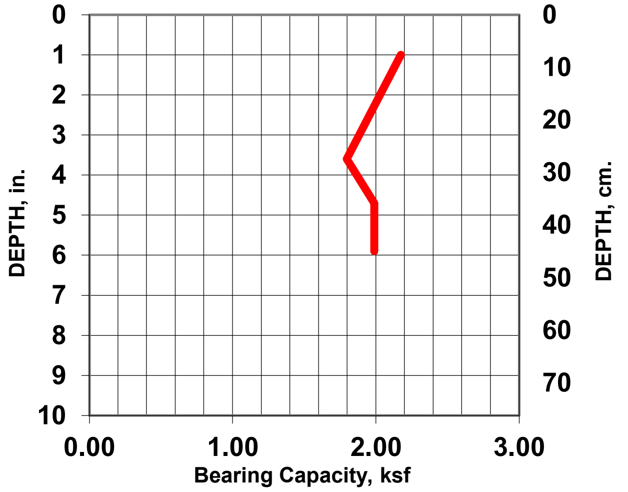
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2







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.

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