RABA KISTNER

GEOTECHNICAL ENGINEERING STUDY

FOR

VIDA AMENITY CENTER
SAN ANTONIO, TEXAS

RABA KISTNER

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Project No. ASA22-064-00
September 30, 2022

Mr. Matthew Cox, AIA, NCARB
Marmon Mok Architecture
1020 NE Loop 410, Suite 201
San Antonio, Texas 78209

RE: Geotechnical Engineering Study
VIDA Amenity Center
San Antonio, Texas

Dear Mr. Cox:

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. PSA22-031-00 Revised July 14, 2022. The purpose of this study was to drill 8 borings, to perform laboratory testing to classify and characterize subsurface conditions, and to prepare an engineering report presenting foundation design and construction recommendations for the proposed new amenity center, 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 information provided to us at the time of this study. There may be alternatives for value engineering of the foundation and pavement systems. RKI recommends that a meeting be held with the Owner and design team to evaluate if alternatives are available.

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

Very truly yours,

RABA KISTNER, INC.

Ryan M. DeWees
Graduate Engineer

Isaac Molina, P.E.
Project Manager

STATE OF TEXAS
ISAAC MOLINA
124165
LICENSED
PROFESSIONAL ENGINEER

IM/RMD/kv

Attachments

Copies Submitted: Above (1-electronic)

GEOTECHNICAL ENGINEERING STUDY

For

VIDA AMENITY CENTER
SAN ANTONIO, TEXAS

Prepared for

MARMON MOK ARCHITECTURE
San Antonio, Texas

Prepared by

RABA KISTNER, INC.
San Antonio, Texas

PROJECT NO. ASA22-064-00

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RABAKISTNER

ATTACHMENTS

The following figures are attached and complete this report:

INTRODUCTION

RABA KISTNER, Inc. (RKI) has completed the authorized subsurface exploration for the proposed community amenity center to be located southwest of the intersection of Mitra Branch and University Way in San Antonio, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for foundation design and construction considerations, as well as for pavement design and construction guidelines.

PROJECT DESCRIPTION

To be considered in this study is a community amenity center facility to be located southwest of the intersection of Mitra Branch and University Way in San Antonio, Texas. We understand that the facility will include an amenity center building, pool, play scape, food truck area, picnic area, playing field (natural grass), and a community garden. Based on the project information provided to us by the Client, we understand that relatively light loads are anticipated to be carried by the foundation systems. Also included will be parking area and driveway pavements.

LIMITATIONS

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

The recommendations submitted in this report are based on the data obtained from 8 borings drilled at this site, our understanding of the project information provided to us, and the site grading plan labeled “VIDA SAN ANTONIO PHASE 1B” provided to us by the CLIENT. 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 differ from site grading plan by more than plus or minus 1 ft, our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by 8 borings drilled using a truck-mounted drilling rig at the locations shown on the Boring Location Map, Figure 1. Boring locations were documented in the field utilizing a hand-held GPS device. The borings were drilled to approximate depths ranging from 5 to 25 ft below the existing ground surface. During drilling operations split-spoon (with standard penetration tests) samples 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, sulfates, pH-lime series, Atterberg Limit (plasticity) tests, as well as grain size analyses (with hydrometer), and moisture density relationship tests.

The results of all laboratory tests are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 9. A key to classification terms and symbols used on the logs is presented on Figure 10. The results of the laboratory and field testing are also tabulated on Figure 11 for ease of reference.

Standard Penetration Test results (N-values) are noted as “blows per ft” on the boring logs and on Figure 11. 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 11.

Grain size analyses were completed using sieve and hydrometer testing, and the results of these are presented in Figure 12. Proctors were performed on the fill stockpile located on site, from the north and south sides, the results of the moisture density curves are presented in Figure 13. A pH-Lime series test was also conducted on the same samples used for the Proctor tests and Boring B-3, and the results are presented in Figure 14.

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

GENERAL SITE CONDITIONS

GEOLOGY

A review of the *Geologic Atlas of Texas, San Antonio Sheet*, indicates that this site is naturally underlain with soils of the Leona Formation. The Leona Formation is associated with terrace deposits of the Nueces and Leona Rivers and typically consists of clays/silts grading down into coarse gravel and cobbles. The Leona Formation can be highly variable and can therefore result in highly variable conditions over relatively short distances. Key geotechnical engineering concerns for development supported on the Leona Formation are the expansive nature of the clays, the consistency and/or relative density of the deposits, and the absence/presence as well as thickness of potentially water-bearing gravels.

SEISMIC CONSIDERATIONS

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

On the basis of the Structural Engineers Association of California/Office of Statewide Health Planning and Development (SEAOC/OSHPD) website[1] which utilizes the 2015 International Building Code (IBC) and U.S. Seismic Design Maps to develop seismic design parameters, the following seismic considerations are associated with this site.

F_a = 1.3	S_s = 0.051g	S_{ms} = 0.067g	S_{DS} = 0.045g
F_v = 1.5	S_1 = 0.020g	S_{m1} = 0.030g	S_{D1} = 0.020g

Based on the parameters listed above as well as Tables 1613.2.5(1) and 1613.2.5(2) of the 2018 IBC, the Seismic Design Category for both short period and 1 second response accelerations is **A**. As part of the assumptions required to complete the calculations, a Risk Category of "I, II, or III" was selected.

STRATIGRAPHY

In our borings, we encountered a layer of fill material extending from the surface to depths of 3 to 10 ft below the surface at the time of our study. We did not encounter fill material at the Boring B-1 location. The natural subsurface stratigraphy can generally be described as layers of low plasticity tan, sandy or silty lean clays. These clays range extend to boring termination.

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

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

GROUNDWATER

No groundwater was observed in the borings during our drilling operations. It is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly following periods of

[1] https://seismicmaps.org

precipitation and with granular stratums. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

SULFATE TESTING

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

Sulfate Ion Test Results	
Boring Number and Depth of Sample	**Sulfate Ion Concentration (ppm)**
P-2, 0-1.5 ft.	280
NS Stockpile, 0-1 ft.	580
SS Stockpile, 0-1 ft.	540

On the basis of soil sulfate concentration data shown on the table, the soils have a "*Negligible*" potential for attacking concrete or to cause sulfate induced heave. Reported sulfate concentrations above 3,000 ppm are known to cause sulfate induced heaving when the soils are mixed with lime. It should be understood that the identification of sulfates based on discrete soil samples cannot totally identify sulfates in all areas. If the option for lime is considered, a quality assurance program should be implemented to assist in reducing the risk of sulfate induced heaving.

FOUNDATION ANALYSIS

EXPANSIVE SOIL-RELATED MOVEMENTS

The anticipated ground movements due to swelling of the underlying soils at the site were estimated for slab-on-grade construction using the empirical procedure, Texas Department of Transportation (TxDOT) Tex-124-E, Method for Determining the Potential Vertical Rise (PVR). A PVR values of less than 1 in. to 1-1/4 in. were estimated for the stratigraphic conditions encountered in our borings. A surcharge load of 1 psi (concrete slab and sand layer), an active zone extending to 15 ft, and dry moisture conditions were assumed in estimating the above PVR values. We also assumed that the existing fill material will be utilized to raise grades below the select fill building pad.

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

Overexcavation and Select Fill Replacement

EXISTING FILL

There is a large stockpile of fill material on site. As previously noted in the *Stratigraphy* section of this report, variable depths of fill were encountered in our borings at approximate depths ranging from 3 to 10 ft below the existing ground surface. Documentation regarding the fill placement was not available at the time of this study. Based on the results of our borings, the existing fill should be considered as uncontrolled and potentially compressible. **For any movement sensitive structures, we recommend that the existing fill be overexcavated and recompacted or replaced**.

It is our understanding that this material will be used in the proposed improvement areas to grades. This material sampled at two locations as shown in Figure 1. The following table summarizes the results of the Proctors, pH lime series, and classification laboratory tests.

Soil Properties from Stockpile Samples						
Sample	PI	Optimal Moisture Content (%)	Maximum Dry Density (lb/ft^3)	Grain Size Analysis		
				% Gravel	% Sand	% Clay/Silt
NS	25	14.1	109.7	17.7	44.4	35.0
SS	25	14.8	108.6	26.7	39.9	33.4

The proposed finished floor elevation (FFE) for the building, sidewalks and decking is 621 ft., msl. Based on the site plan, the lowest natural elevation for these areas is approximately 615.5 ft., msl. We recommend overexacavating the entire building footprint to the elevation of 615 ft., msl, and then use fill material to build the pad upto the needed height to achieve the FFE by placing in controlled lifts. In order to achieve a PVR of 1 in. or less we recommend the onsite stockpile material be placed in compacted lifts to the elevation of 619 ft., msl. The top of the building pad should include approximately 2 ft. of compacted select fill material.

The overexcavation should extend at least 3 ft beyond the structure footprint if no additional overexcavation is required for soil-related movements. The onsite stockpile fill and select fill materials should be placed as recommended in the *Select Fill* section of this report.

FOUNDATION RECOMMENDATIONS

Site features that will influence the geotechnical approach to the proposed project include:

- Depth of existing fill material;
- Variable soil properties of existing fill material; 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).

SITE GRADING

Site grading plans can result in changes in almost all aspects of foundation recommendations. We were supplied with a site grading plan by the CLIENT, as mentioned in the *Limitations* section of this report. The proposed FFE are as follows: Building, decking, and sidewalk at entry 621-0 ft-msl, Pool and pool deck 619-0 ft-msl. If site grading plans are changed in the future by more than plus or minus 1 ft, RKI must be retained to review the updated 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. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations. A review of the report is not required if the only additional fill materials placed at the site are select fill materials placed within the interior of grade beams for slab support provided that the select fill materials are placed as specified in the *Select Fill* section of this report.

FOUNDATION OPTIONS

Based on results of our field and laboratory testing programs, analyses, and our past experience in the vicinity of this site, it is our opinion that shallow foundations are available to support the proposed structure at this site.

The owner and design team may select foundation systems depending on the performance criteria established for the structures. Cost analyses have not been conducted for any foundation system and are beyond the scope of this study. Additional alternatives for support of the structure may exist and RKI recommends that a meeting be held with the Owner and design team to evaluate if there are additional alternatives.

SHALLOW FOUNDATIONS

The proposed amenity center may be founded on a shallow foundation system provided the selected foundation type can be designed to withstand the anticipated soil-related movements (see *Expansive Soil-Related Movements*) without impairing either the structural or the operational performance of the structures.

Allowable Bearing Capacity

Shallow Foundation Design Parameters	
Minimum depth below final grade	18 in.
Minimum beam or strip footing width	12 in.
Minimum widened beam or spread footing width	18 in.
Maximum allowable bearing pressure for foundations on compacted alternative select fill	2,400 psf*
Maximum allowable bearing pressure for foundations on compacted crushed limestone select fill	3,000 psf

*See *Select Fill* section for alternative select fill options

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

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

Uplift Resistance

Resistance to vertical force (uplift) is provided by the weight of the concrete footing plus the weight of the soil directly above the footing. For this site, it is recommended that the ultimate uplift resistance be based on total unit weights for soil and concrete of 125 pcf and 150 pcf, respectively. The calculated ultimate uplift resistance should be reduced by a factor of safety of 1.2 to calculate the allowable uplift resistance.

Lateral Resistance

Horizontal loads acting on shallow foundations will be resisted by passive earth pressure acting on one side of the footing and by base adhesion for footings in soil or bedrock. Resistance to sliding for foundations bearing on natural/compacted soil may be calculated utilizing an ultimate coefficient of friction of 0.30. The ultimate lateral resistance for these foundations should be limited to 900 psf. An equivalent fluid pressure of 250 pcf may be utilized to determine the ultimate passive resistance, if required.

B.R.A.B. Criteria

Engineered beam and slab foundations are sometimes designed using criteria developed by the Building Research Advisory Board (B.R.A.B.). It should be noted that if the highest plasticity index (PI) value encountered in the subsurface profile occurs in the uppermost subsurface layer, B.R.A.B. criteria requires that this PI value be selected as the design PI. The B.R.A.B. design plasticity index, soil support index (C), Climatic Rating (C_w), and estimated unconfined compressive strength (q_u) presented in the following table may be utilized for the proposed structures.

B.R.A.B. Criteria	
Climatic Rating, C_w	16
Estimated Unconfined Compressive Strength, q_u	2,500 psf
B.R.A.B. Design Plasticity Index	31
Soil Support Index (C)	0.89

The design criteria will change if a select fill building pad is constructed for the proposed foundation. If site grading operations alter the thickness of the on-site soil then the criteria for the amenity center should be re-evaluated for the appropriate slab design parameters. If existing soils are removed and replaced with select fill to achieve a PVR of 1 inch or less, as proposed in the *Overexcavation and Select Fill Replacement* section of this report, a design plasticity index (PI) of 20 and a soil support index (C) value of 0.94 may be used.

PTI Design Parameters

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

Differential vertical swell has been estimated for center lift and edge lift conditions for use in designing foundation slabs for the stratigraphy encountered in our borings. These values were determined using a computer program entitled VOLFLO Win 1.5, as recommended by the Post Tensioning Institute. As recommended by PTI, we have evaluated differential swell for both 1) conditions varying from equilibrium and 2) conditions varying between extremes (wet/dry). The values for both of these conditions are presented in the following table. Because soil moisture conditions are likely to vary from wet to dry and dry to wet over many cycles during the lifetime of the structure, we recommend that the conditions varying between the extremes be assumed for design. We have also evaluated the various conditions both without (Design condition A) and with (Design condition B) a 30 in. vertical moisture barrier (exterior grade beam) for amenity center and pool areas.

Design Condition	Edge Moisture Variation Distance (CL)[1], ft	Edge Moisture Variation Distance (EL)[1], ft	Differential Swell (in.) From Equilibrium to Wet	From Equilibrium to Dry	From Dry to Wet	From Wet to Dry
A	5.9	3.1	1/2	1/2	1-1/2	1
B	5.9	3.1	1/4	1/4	1/2	1/2

[1] - (EL) Edge Lift Condition, (CL) Center Lift Condition

AREA FLATWORK

We recommend that flexible joints be provided where flatwork elements abut the structures to allow for differential movement at these locations. Where flatwork abuts the exterior perimeter grade beam of the building, care must be taken to provide a smooth, vertical construction joint between the edge of the flatwork and the perimeter grade beam. The construction joint should be wide enough to assure that vertical movement in the flatwork will not bind on and subsequently damage the exterior veneer material.

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

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

PERMANENT SLOPES

The stability of permanent slopes depends on many factors, including the height and geometry of the slopes, the types of soils contained in the slopes, effects of groundwater, and any surface pressures present. In general, permanent cut and fill slopes less than 20 ft in height, constructed at 1V:3H (1 vertical on 3 horizontal) have been observed to perform satisfactorily. Therefore, it is our opinion that slopes should be constructed at 1V:3H or flatter. Fill slopes should be constructed by extending the compacted fill beyond the planned profile of the slope and then trimming the slope to the desired configuration. Cut slopes can be designed similar to fill slopes. However, the potential for sloughing and/or general slope failure increases with an increase in the steepness and depth of cut, particularly if low strength soil occurs in or near the base of the slope.

RETAINING STRUCTURES

LATERAL EARTH PRESSURES

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

Backfill 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
Lean Clays	115	0.36	40	0.53	60

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 the values under "At-Rest Conditions" 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 use of fat 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 retaining walls will be critical, particularly at locations where backfill will support adjacent near-grade foundations and/or flatwork. If the backfill is not properly compacted in these areas, the adjacent foundations/flatwork can be subject to settlement.

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

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 drain pipe 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 reduce 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.

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.

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. Careful consideration should also be given to the location of water bearing utilities, as well as to provisions for drainage in the event of leaks in water bearing utilities. All leaks should be immediately repaired.

SITE PREPARATION

The building area should be stripped of all vegetation and organic topsoil as well as existing pavement materials. Tree roots greater than 1 inch in diameter should be grubbed and removed. Any voids resulting from removal of limestone boulders or tree roots should be backfilled with a suitable, compacted fill material, free of organics, degradable material, and particles exceeding 4 inches in size.

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

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

In areas of exposed competent and intact limestone rock subgrade, the subgrade shall be proofrolled in order to locate and densify any weak compressible zones. Scarification and moisture conditioning will not be required on competent and intact limestone rock.

SELECT FILL

Recommendations for imported granular select fill materials are provided below.

> **Imported Crushed Limestone Base** – Imported crushed limestone base materials should be crushed stone or gravel aggregate. We recommend that materials specified for use as select fill meet the TxDOT 2014 Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 247, Flexible Base, Type A or B, Grades 1-2 or 3.
>
> **Recycled Materials** – Recycled materials (i.e. concrete) are a viable alternative to crushed limestone to be used as fill, provided the recycled material is determined to be environmentally acceptable. We recommend that the recycled concrete material meet the requirements of TxDOT Item 247, Paragraph 2.13.2.1. prior to hauling to the site.
>
> Recycled material may be used as fill if deleterious materials can be separated (i.e. rebar, soil, wood, metal, plastic, piping, conduit, etc). Oversized rubble should be processed to a well-graded material similar to the *Imported Crushed Limestone Base* with a maximum particle size of 4 inches. Rubble larger than 4 inches in any dimension should be discarded or processed to the maximum dimension. Care should be taken when placing the fill that the larger pieces are not concentrated in a manner such that voids develop between nested pieces; a sufficient quantity of fines should be provided to reduce this risk.

Recommendations for alternative select fill materials are provided below.

Treated Onsite Materials – Lime treatment of the onsite soils may be considered in reducing the soil plasticity index (TxDOT Item 260 for Lime). A sufficient quantity of product should be mixed with the subgrade soils to reduce the soil-product mixture plasticity index to approximately 20 or less. We estimate that approximately 3 percent lime (based on pH-Lime series test results on Figure 14).

Based onsulfate content test results, the sulfate content of representative soil subgrade samples was neglibible. However, we recommend that during site grading operations that additional laboratory testing be performed to determine the appropriate treatment dosage rate and concentration of soluble sulfates in the subgrade and imported soils.

Granular Pit Run Materials – Granular pit run materials should consist of GC, SC & combination soils (clayey gravels), as classified according to the Unified Soil Classification System (USCS). Alternative select fill materials shall have a maximum liquid limit not exceeding 40, a plasticity index between 7 and 20, and a maximum particle size not exceeding 4 inch. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with pit-run materials.

Low PI Materials – Low PI materials should consist of CL clays, as classified according to the Unified Soil Classification System (USCS). Alternative select fill materials shall have a maximum liquid limit not exceeding 40, a plasticity index between 7 and 20, and a maximum particle size not exceeding 4 inch. In addition, if these materials are utilized, grain size analyses and Atterberg Limits must be performed during placement at a rate of one test each per 5,000 cubic yards of material due to the high degree of variability associated with these materials.

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

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

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

GENERAL FILL

Fills placed outside of select fill areas may consist of general fill material. General fill materials should have maximum particle sizes of 4 inches and placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 95 percent of maximum density as determined by TxDOT, Tex-114-E, Compaction Test. The moisture content of the fill should be maintained within the range of optimum water content to plus 3 percentage points above optimum.

CLAY CAP

Where a select fill overbuild is provided outside the building footprint or along the perimeter of critical and non-critical pavement/flatwork areas where the fill would be exposed, the surface should be sealed with a 2 ft minimum impermeable clay layer to reduce infiltration of both irrigation and surface water runoff. Materials used as clay cap material should consist of fat clay (CH) soils as classified according to the Unified Soil Classification System (USCS).

We recommend that clay soils be placed in loose lifts not exceeding 8 inches in thickness and compacted to at least 90 percent of the maximum density as determined by TxDOT, Tex-114-E. The moisture content of the fill should be maintained within the range of optimum water content to plus 3 percentage points above optimum. We recommend that fill materials be free of roots or other organic or degradable material.

SHALLOW FOUNDATION EXCAVATIONS

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

EXCAVATION SLOPING AND BENCHING

Excavations should not undermine adjacent utilities, foundations, walkways, streets, or other hardscapes unless shoring or underpinned support is provided. If utility trenches or other excavations extend to or below a depth of 5 ft below construction grade, the contractor or others shall be required to develop a trench safety plan to protect personnel entering the trench or trench vicinity. The collection of specific geotechnical data and the development of such a plan, which could include designs for sloping and benching or various types of temporary shoring, are beyond the scope of the current study. Any such designs and safety plans shall be developed in accordance with current OSHA guidelines and other applicable industry standards.

EXCAVATION EQUIPMENT

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

UTILITIES

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

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

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

PAVEMENT RECOMMENDATIONS

Recommendations for flexible and rigid pavements are presented in this report. The Owner and/or design team should carefully select the pavement option depending on the traffic types and performance criteria established for each of the pavements being considered for these improvements. 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. Rigid pavements are preferable for heavy duty pavement sections and lime treated subgrade is recommended for all pavements at this site due to the expansive soils conditions. Porous pavements should only be considered for the walking track pavements with no wheeled traffic.

For all pavement types, 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 native lean clay or recompacted on-site soils, placed and compacted as recommended in the *On-Site Fill* section of this report. Based on our experience with similar subgrade soils, we have assigned a CBR value of 4 for use in pavement thickness design analyses.

DESIGN INFORMATION

The following recommendations were prepared using the procedure based on the 1993 “Guide for the Design of Pavement Structures” by the American Association of State Highway and Transportation Officials (AASHTO). **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.**

Pavement Design Parameters		
Pavement Design Parameters	Flexible Pavement	Rigid Pavement
Performance Period	20 yrs	20 yrs
Design Traffic, 18-kip ESALs Light Duty Heavy Duty	 30,000(1) 90,000(2)	 150,000(3) 300,000(4)
California Bearing Ratio (CBR)	4.0(5)	
Initial Serviceability Index	4.2	4.5
Terminal Serviceability Index	2.0	
Overall Standard Deviation	0.45	0.35
Reliability	70	
Modulus of Subgrade reaction (k-value)	-	125 pci
28-day Concrete Modulus of Rupture	-	600 psi
28-day Concrete Elastic Modulus	-	4,000,000 psi
Load Transfer Coefficient	-	3.2
Drainage Coefficient	-	1.02
Roadbed Soil Resilient Modulus	4,500 psi	-

(1)Passenger Vehicle Parking Areas
(2)Passenger Vehicle Drives and Entrances - up to 3 Single Trailer Trucks per day
(3)Passenger Vehicle Parking Areas - up to 5 Single Trailer Trucks per day
(4) Passenger Vehicle Drives and Entrances - up to 10 Single Trailer Trucks per day
(5)The assigned CBR is based on the DCP results and our experience with similar soils.

FLEXIBLE PAVEMENT

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

	Layer Description	Layer Thickness
Light Duty Traffic	HMAC Surface Course, Type C or D Flexible Base **Combined Total**	2.0 in. 7.0 in. **9.0 in.**
Heavy Duty Traffic	HMAC Surface Course, Type C or D Flexible Base **Combined Total**	3.0 in. 7.0 in. **10.0 in.**

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. Lime or cement treatment of the subgrade soils may also be considered, however, the gravel within the existing fill material at this site may cause difficulty when mixing.

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.

CONSIDERATIONS FOR TRANSITIONS FROM RIGID TO FLEXIBLE PAVEMENTS

At rigid to flexible pavement transitions, if any, 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.

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. We recommend that rigid pavement sections at this site consist of the following:

Traffic Type	Portland Cement Concrete
Light Duty Traffic	5 in.
Heavy Duty Traffic	5.5 in.

Rigid Pavement Consideration

Either Jointed Plain Concrete Pavement (JPCP) or reinforced concrete pavements may 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.

If reinforced concrete is utilized it is recommended that the concrete pavements be reinforced with welded wire mats or bar mats. As a minimum, the welded wire mats should be 6 x 6 in., W4.0 x W4.0, or the bar mats should be No. 3 reinforcing bars spaced 18 in. on center in both directions. The concrete reinforcing should be placed approximately 1/3 the slab thickness below the surface of the slab, but not less than 2 in. The reinforcing should not extend across expansion joints.

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.

GARBAGE DUMPSTERS

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

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

FIRE LANE

Based on available literature, a 75,000 pound fire truck will impart approximately 6.9 ESALs per pass. Therefore, the proposed pavement medium or heavy duty sections provided herein will be able to support occasional fire truck traffic.

PAVEMENT CONSTRUCTION CONSIDERATIONS

SUBGRADE PREPARATION

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

DRAINAGE CONSIDERATIONS

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

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

1) Any known natural or man-made subsurface seepage at the site which may occur at sufficiently shallow depths as to influence moisture contents within the subgrade should be intercepted by drainage ditches or below grade French drains.
2) Final site grading should eliminate isolated depressions adjacent to curbs which may allow surface water to pond and infiltrate into the underlying soils.
3) Pavement surfaces should be maintained to help reduce surface ponding and to provide rapid sealing of any developing cracks. These measures will help reduce infiltration of surface water downward through the pavement section.

ON-SITE FILL

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

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to TxDOT Standard Specifications, Item 247, Type A, Grade 1-2. Base course should be placed in lifts with a maximum thickness of 8 in. and compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E.

ASPHALTIC CONCRETE SURFACE COURSE

The asphaltic concrete surface course should conform to TxDOT Standard Specifications, Item 340, Type 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 broom finishing the concrete surface. The curing compound will help reduce the loss of water from the concrete. The reduction in the rapid loss in water will help reduce shrinkage cracking of the concrete.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

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

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

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

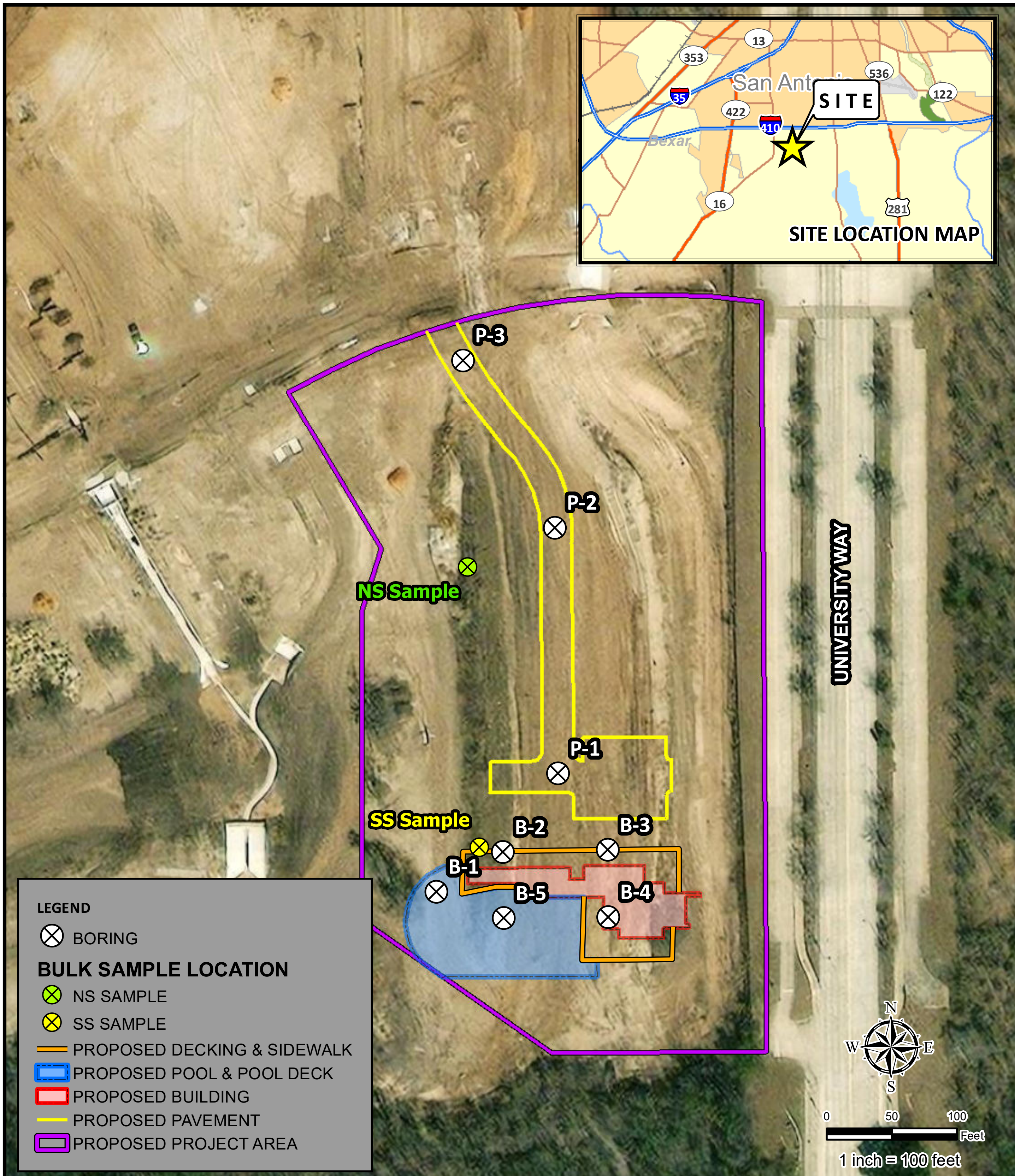
BUDGETING FOR CONSTRUCTION TESTING

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

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

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

ATTACHMENTS



RABA KISTNER

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TBPE Firm Number 3257

SOURCE: Aerial imagery from Google Earth Pro - 2021

BORING LOCATION MAP

VIDA PHASE 1B
SAN ANTONIO, TEXAS

BEXAR COUNTY

PROJECT No.:	ASA22-064-00
ISSUE DATE:	09/27/2022
DRAWN BY:	BM
CHECKED BY:	RD
REVIEWED BY:	IM

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

VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31394; W 98.52575

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

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

SURFACE ELEVATION: 614* ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	LEAN CLAY, Sandy, Very Stiff, Light Tan to Reddish Tan, with calcareous deposits	23			
		20			
5		15		10	
	LEAN CLAY, Sandy, Very Stiff, Light Tan	17			
10		28			
	LEAN CLAY, Hard, Tan				
		33			
15	Boring Terminated				

NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

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

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA22-064-00
DATE DRILLED:	7/29/2022	**DATE MEASURED:**	7/29/2022	**FIGURE:**	2

LOG OF BORING NO. B-2

VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31410; W 98.52558

SURFACE ELEVATION: 615* ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0–3	FILL: GRAVEL, Clayey, Hard, Reddish Brown	30		25	
3–6	LEAN CLAY, Sandy, Stiff to Very Stiff, Light Tan to Reddish Tan, with calcareous deposits	13			
		29			
6–25	SILT, Very Stiff to Hard, Tan, with sand	27		6	
		15			
		50/1"			
		31		2	
		40			
25	Boring Terminated				

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

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

NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

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

DEPTH DRILLED:	25.0 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA22-064-00
DATE DRILLED:	7/29/2022	**DATE MEASURED:**	7/29/2022	**FIGURE:**	3

LOG OF BORING NO. B-3

VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31413; W 98.52531

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (SURFACE ELEVATION: 620* ft)	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT² (0.5 – 4.0) / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT (10 – 80)	PLASTICITY INDEX	% -200
			FILL: GRAVEL, Clayey, Firm to Hard, Reddish Brown	21				
				11				
5				38			22	
				50/11"				
				50/10"				
10			LEAN CLAY, Silty, Very Stiff to Hard, Light Tan, with calcareous deposits					
15				27				
				50/4"			7	
20			SILT, Very Stiff to Hard, Tan, with sand					
				50/3"				
25			Boring Terminated					
30								
35								

NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

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

DEPTH DRILLED:	24.3 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA22-064-00
DATE DRILLED:	7/29/2022	**DATE MEASURED:**	7/29/2022	**FIGURE:**	4

LOG OF BORING NO. B-4

VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31394; W 98.52537

SHEAR STRENGTH, TONS/FT²: 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 3.5, 4.0

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

SURFACE ELEVATION: 617* ft

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0	FILL: GRAVEL, Clayey, Hard to Very Stiff, Reddish Brown	36			
		28			
4	LEAN CLAY, Sandy, Very Stiff, Light Tan	15			
		24		10	
8	LEAN CLAY, Calcareous, Very Stiff, Light Tan	17			
14	SAND, Clayey, Medium Dense to Dense, Tan	22			
		50/11"		11	43
19.9	Boring Terminated				

NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

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

DEPTH DRILLED: 19.9 ft	**DEPTH TO WATER:** Dry	**PROJ. No.:** ASA22-064-00	
DATE DRILLED: 7/29/2022	**DATE MEASURED:** 7/29/2022	**FIGURE:** 5	

LOG OF BORING NO. B-5

VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31398; W 98.52560

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

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

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			SURFACE ELEVATION: 616* ft				
0			FILL: GRAVEL, Clayey, Hard to Stiff, Reddish Brown	36		31	
				18			
5				9		20	
			LEAN CLAY, Sandy, Very Stiff to Hard, Tan	22			
10				17			
				48			
15			Boring Terminated				

NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

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

DEPTH DRILLED: 15.0 ft	**DEPTH TO WATER:** Dry	**PROJ. No.:** ASA22-064-00	
DATE DRILLED: 7/29/2022	**DATE MEASURED:** 7/29/2022	**FIGURE:** 6	

LOG OF BORING NO. P-1

VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31427; W 98.52536

SHEAR STRENGTH, TONS/FT²: 0.5 1.0 1.5 2.0 2.5 3.0 3.5 4.0

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

SURFACE ELEVATION: 623* ft

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0–5			FILL: GRAVEL, Clayey, Very Stiff to Stiff, Reddish Brown	18		12	
				13			
5			Boring Terminated				

NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

DEPTH DRILLED: 5.0 ft	**DEPTH TO WATER:** Dry	**PROJ. No.:** ASA22-064-00	
DATE DRILLED: 7/29/2022	**DATE MEASURED:** 7/29/2022	**FIGURE:** 7	

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

LOG OF BORING NO. P-2

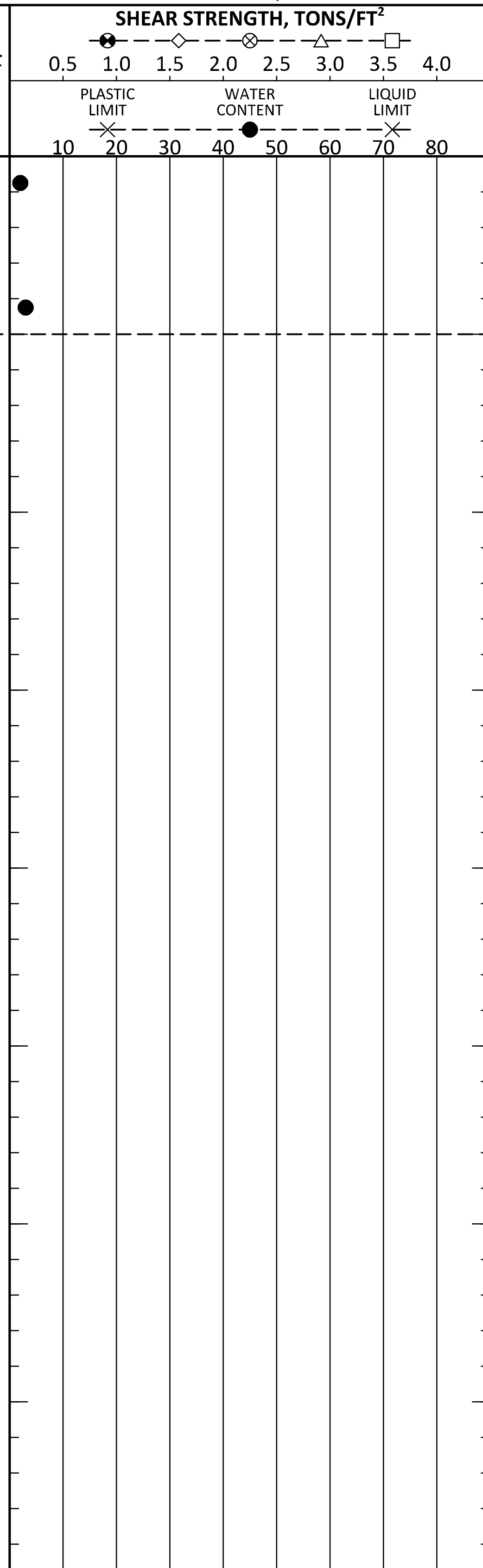
VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31481; W 98.52543

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL SURFACE ELEVATION: 623* ft	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT² / PLASTIC LIMIT, WATER CONTENT, LIQUID LIMIT	PLASTICITY INDEX	% -200
			FILL: GRAVEL, Clayey, Stiff, Reddish Brown	15				
				12				
5			Boring Terminated					



NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

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

DEPTH DRILLED: 5.0 ft
DATE DRILLED: 7/29/2022
DEPTH TO WATER: Dry
DATE MEASURED: 7/29/2022
PROJ. No.: ASA22-064-00
FIGURE: 8

LOG OF BORING NO. P-3

VIDA Amenity Center
San Antonio, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.31520; W 98.52573

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL (SURFACE ELEVATION: 619* ft)	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT² (0.5–4.0) / PLASTIC LIMIT – WATER CONTENT – LIQUID LIMIT (10–80)	PLASTICITY INDEX	% -200
			FILL: GRAVEL, Clayey, Very Stiff, Reddish Brown	29				
				23				
5			Boring Terminated					

NOTES:
*Approximate Elevation from "KFW Survey Amenity Center Site & PD Topo Map 2022-08-17 MM_ Markups"

Depth scale: 5, 10, 15, 20, 25, 30, 35

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

DEPTH DRILLED:	5.0 ft	**DEPTH TO WATER:**	Dry	**PROJ. No.:**	ASA22-064-00
DATE DRILLED:	7/29/2022	**DATE MEASURED:**	7/29/2022	**FIGURE:**	9

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

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

ROCK TERMS

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

OTHER

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

WELL CONSTRUCTION AND PLUGGING MATERIALS

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

SAMPLE TYPES

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

STRENGTH TEST TYPES

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

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

PROJECT NO. ASA22-064-00

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

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

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

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

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

STANDARD PENETRATION TEST (SPT)

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: VIDA Amenity Center
San Antonio, Texas

FILE NAME: ASA22-064-00 VIDA AMENITY CENTER.GPJ 9/28/2022

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
NS	0.0								35		
SS	0.0 to 5.0	NA							33		
B-1	0.0 to 1.5	23	8								
	2.5 to 4.0	20	12								
	4.5 to 6.0	15	11	26	16	10					
	6.5 to 8.0	17	12								
	8.5 to 10.0	28	8								
	13.5 to 15.0	33	8								
B-2	0.0 to 1.5	30	13	43	18	25					
	2.5 to 4.0	13	7								
	4.5 to 6.0	29	6								
	6.5 to 8.0	27	6	29	23	6					
	8.5 to 10.0	15	5								
	13.5 to 14.1	50/1"	6								
	18.5 to 20.0	31	20	27	25	2					
	23.5 to 25.0	40	24								
B-3	0.0 to 1.5	21	4								
	2.5 to 4.0	11	6								
	4.5 to 6.0	38	10	41	19	22					
	6.5 to 7.9	50/11"	7								
	8.5 to 9.8	50/10"	4								
	13.5 to 15.0	27	7								
	18.5 to 19.3	50/4"	7	26	19	7					
	23.5 to 24.3	50/3"	13								
B-4	0.0 to 1.5	36	4								
	2.5 to 4.0	28	11								
	4.5 to 6.0	15	10								
	6.5 to 8.0	24	8	28	18	10					
	8.5 to 10.0	17	6								
	13.5 to 15.0	22	9								
	18.5 to 19.9	50/11"		28	17	11	SC		43		
B-5	0.0 to 1.5	36	5	48	17	31					
	2.5 to 4.0	18	7								
	4.5 to 6.0	9	11	35	15	20					
	6.5 to 8.0	22	9								
	8.5 to 10.0	17	8								
	13.5 to 15.0	48	8								
P-1	0.0 to 1.5	18	6	29	17	12					
	3.5 to 5.0	13	5								

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

CU = Consolidated Undrained Triaxial

PROJECT NO. ASA22-064-00

RABA KISTNER

RESULTS OF SOIL SAMPLE ANALYSES

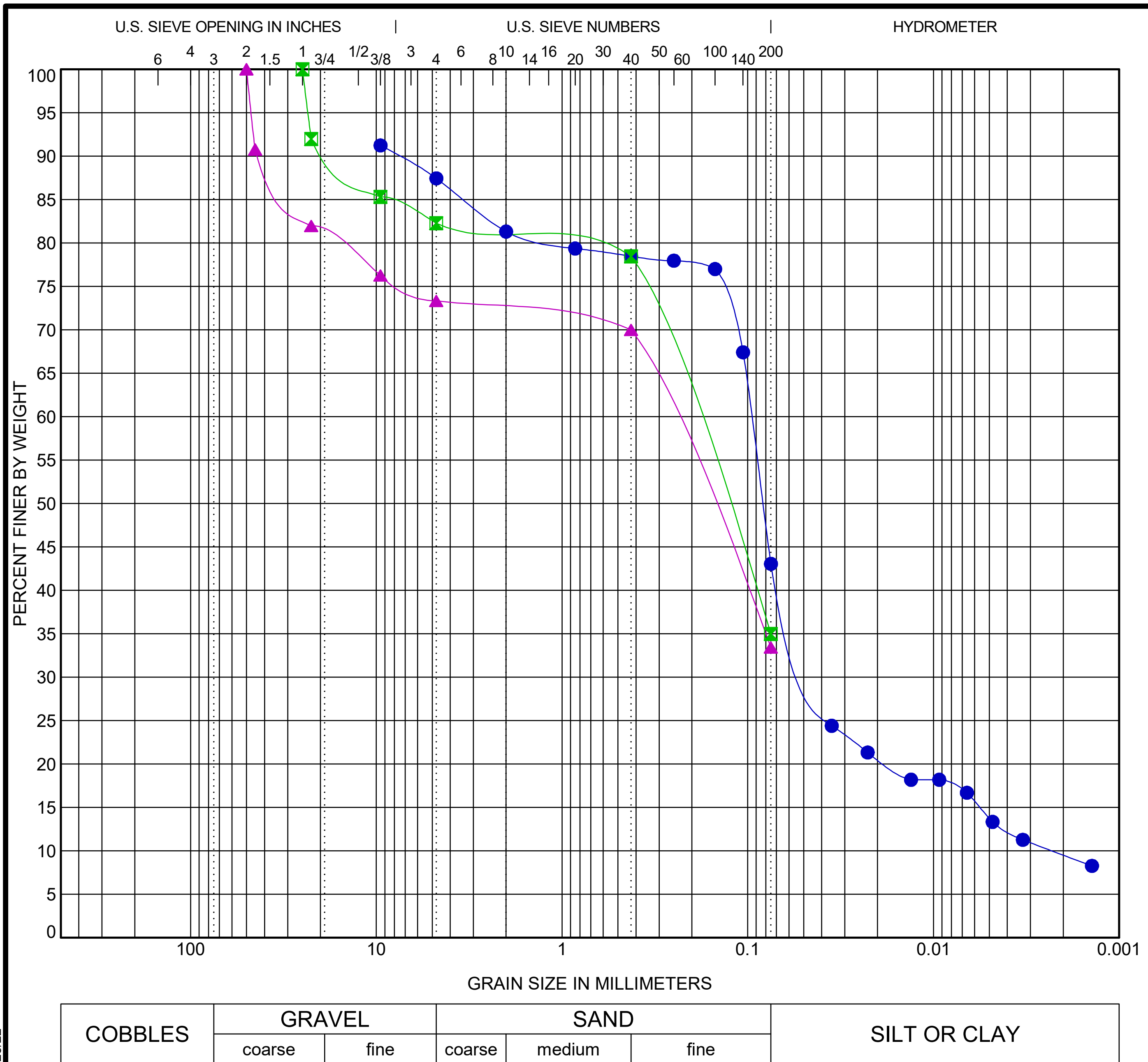
PROJECT NAME: VIDA Amenity Center
San Antonio, Texas

FILE NAME: ASA22-064-00 VIDA AMENITY CENTER.GPJ 9/28/2022

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
P-2	0.0 to 1.5	15	2								
	3.5 to 5.0	12	3								
P-3	0.0 to 1.5	29	7								
	3.5 to 5.0	23	6								

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

CU = Consolidated Undrained Triaxial



	Specimen Identification		0.02 mm	0.005 mm	0.002 mm	0.001 mm	LL	PL	PI	Cc	Cu
●	**B-4**	**18.5**	**20.6**	**13.8**	**9.5**		**28**	**17**	**11**	**8.94**	**41.54**
⊠	**NS**	**0.0**									
▲	**SS**	**0.0**									
★											
⊙											

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-4**	**18.5**	**9.5**	**0.095**	**0.044**	**0.002**	**3.8**	**44.4**	**29.3**	**13.8**
⊠	**NS**	**0.0**	**25**	**0.204**			**17.7**	**47.3**	**35.0**	
▲	**SS**	**0.0**	**50**	**0.265**			**26.7**	**39.9**	**33.4**	
★										
⊙										

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TBPE Firm Registration No. F-3257

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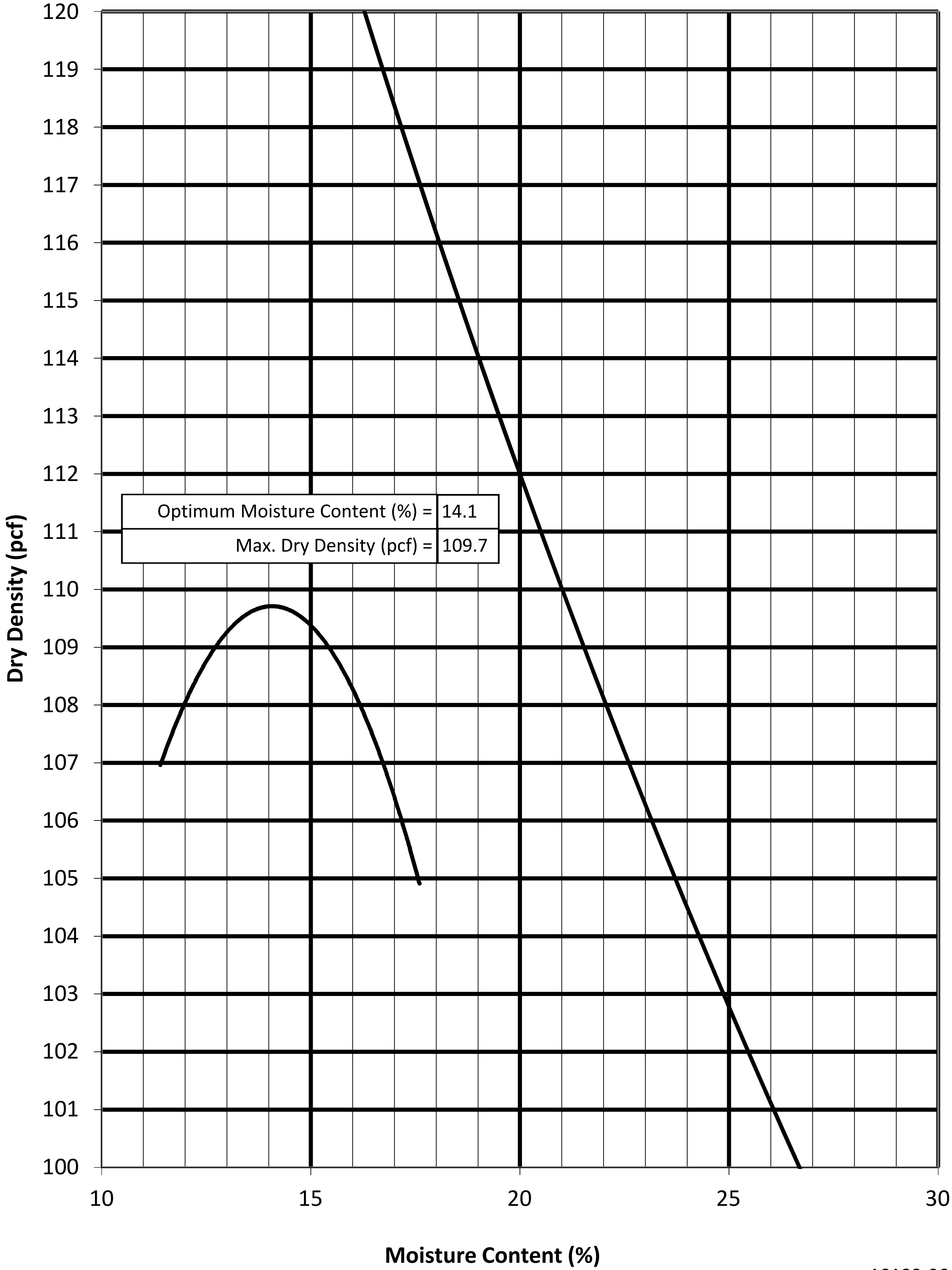
GRAIN SIZE DISTRIBUTION

VIDA Amenity Center
San Antonio, Texas

R-K GRAIN SIZE FOR AASHTO ASA22-064-00 VIDA AMENITY CENTER.GPJ RKCI.GDT 9/28/22

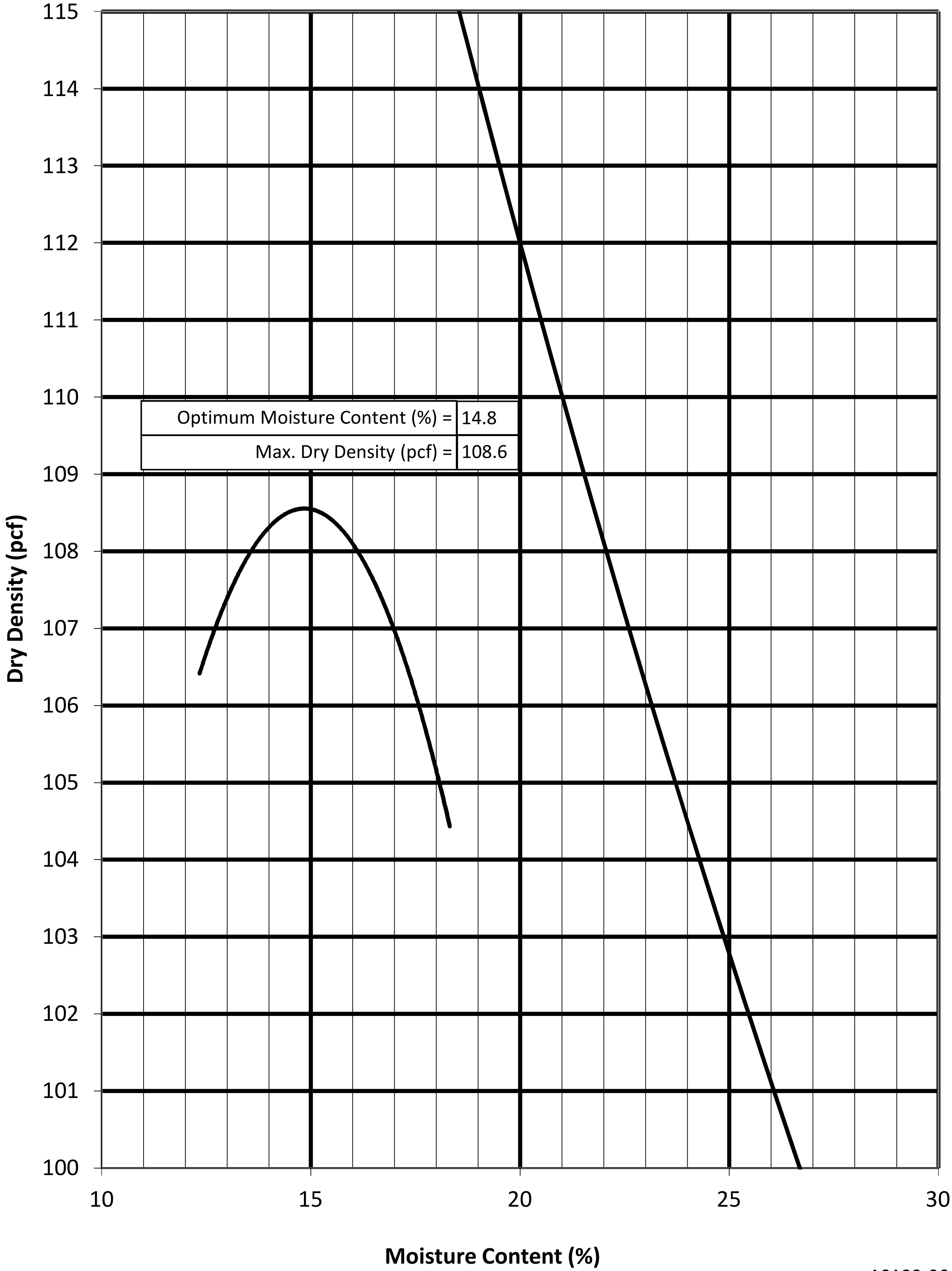
MOISTURE DENSITY RELATIONSHIP CURVE (ASTM D698)

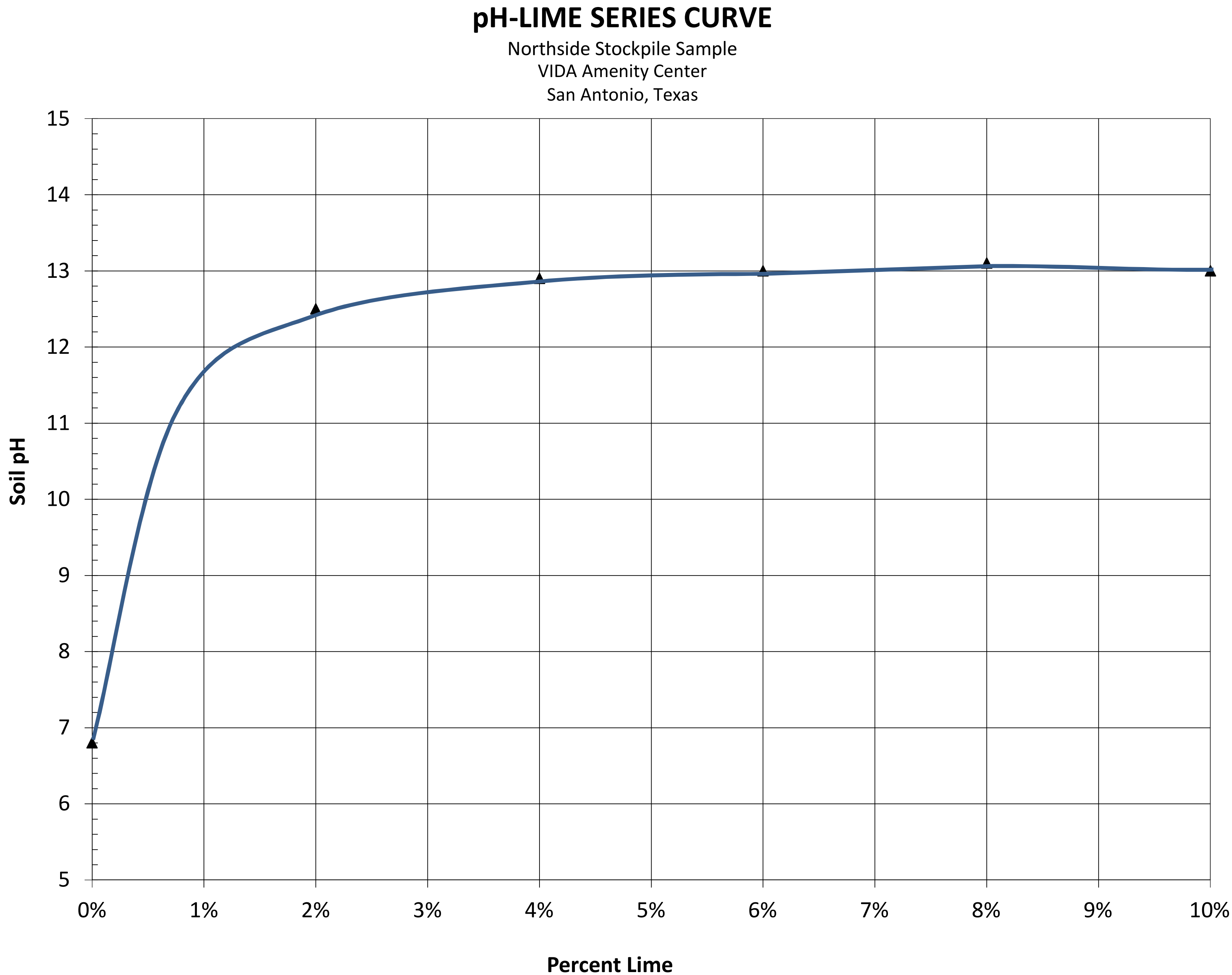
Northside Stockpile Sample
VIDA Amenity Center
San Antonio, Texas



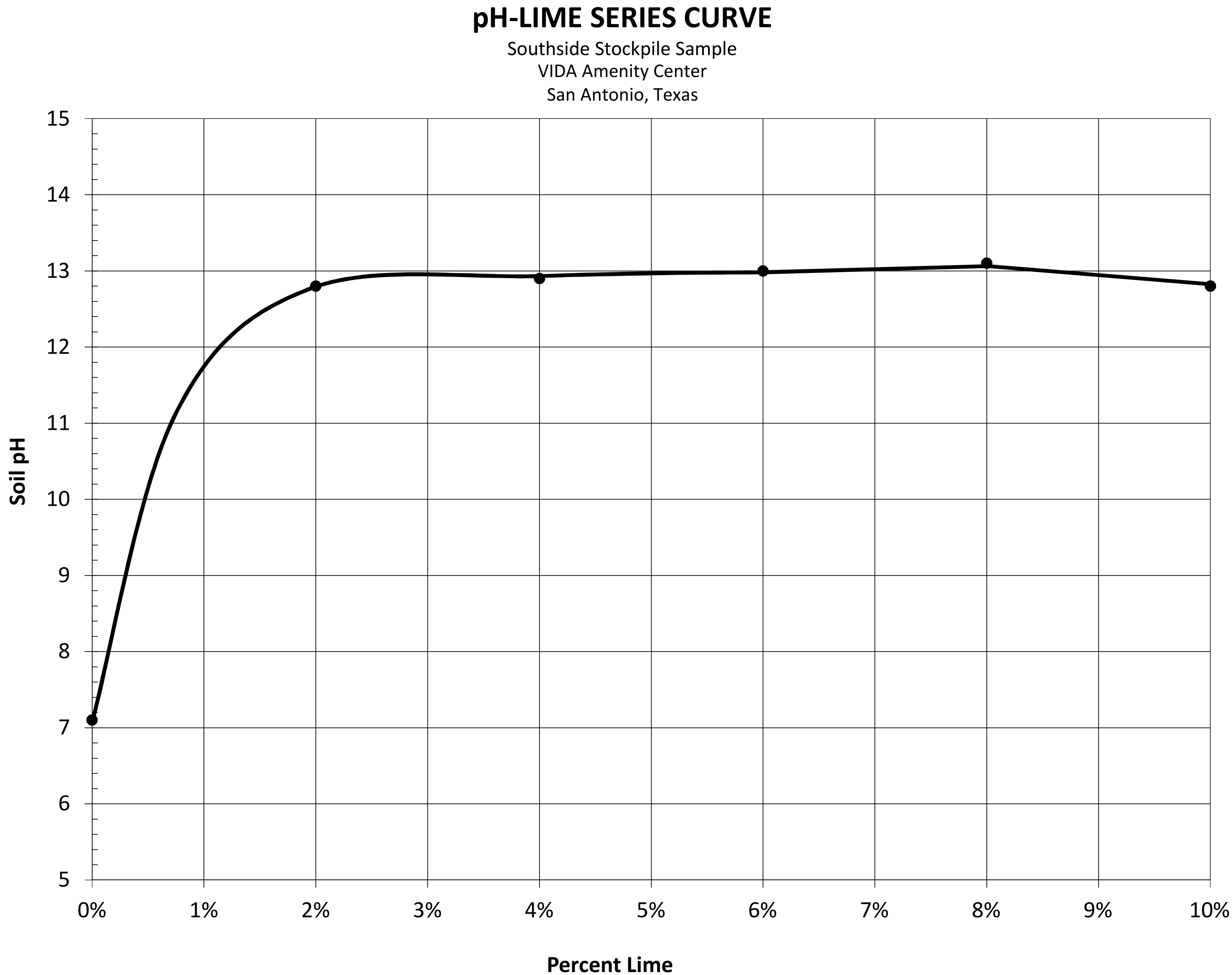
MOISTURE DENSITY RELATIONSHIP CURVE (ASTM D698)

Southside Stockpile Sample
VIDA Amenity Center
San Antonio, Texas





pH-LIME SERIES CURVE
Northside Stockpile Sample
VIDA Amenity Center
San Antonio, Texas
Soil pH
15
14
13
12
11
10
9
8
7
6
5
0%
1%
2%
3%
4%
5%
6%
7%
8%
9%
10%
Percent Lime


pH-LIME SERIES CURVE
Southside Stockpile Sample
VIDA Amenity Center
San Antonio, Texas
Soil pH
15
14
13
12
11
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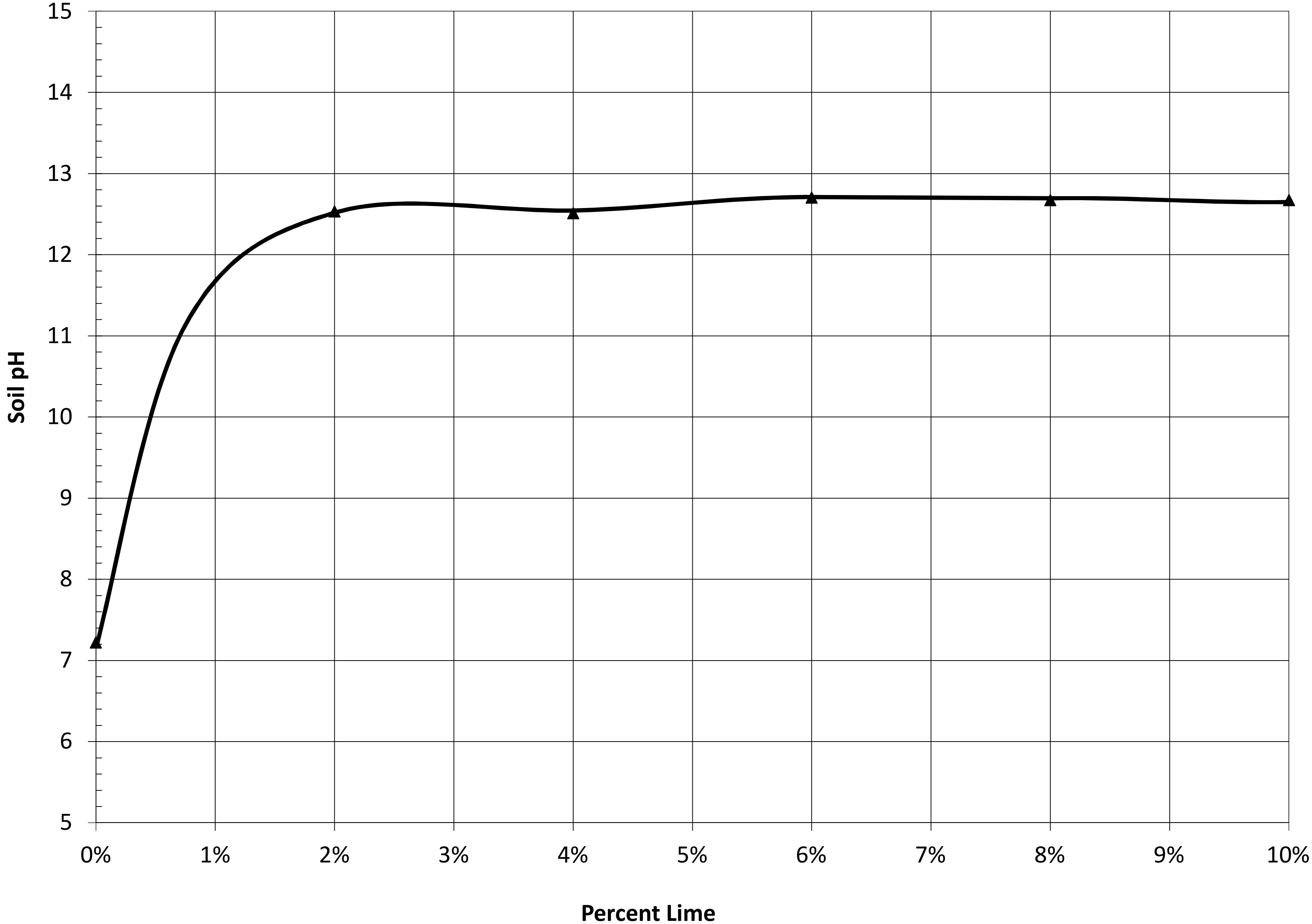
8
7
6
5
0%
1%
2%
3%
4%
5%
6%
7%
8%
9%
10%
Percent Lime

pH-LIME SERIES CURVE

B-3 Soil Sample, 0 to 1.5 ft.
VIDA Amenity Center
San Antonio, Texas



Project Number: ANA22-064-00
Test Date: July 29, 2022

DCP TEST DATA

P-1

VIDA Amenity Center
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	3	20	0.8	35	52.5	5.79
1	5	15	1.4	85	127.5	10.44
1	10	5	1.6	635	952.5	39.67
1	10	1	1.6	3849	5773.5	131.26
-	-	-	-	-	-	-
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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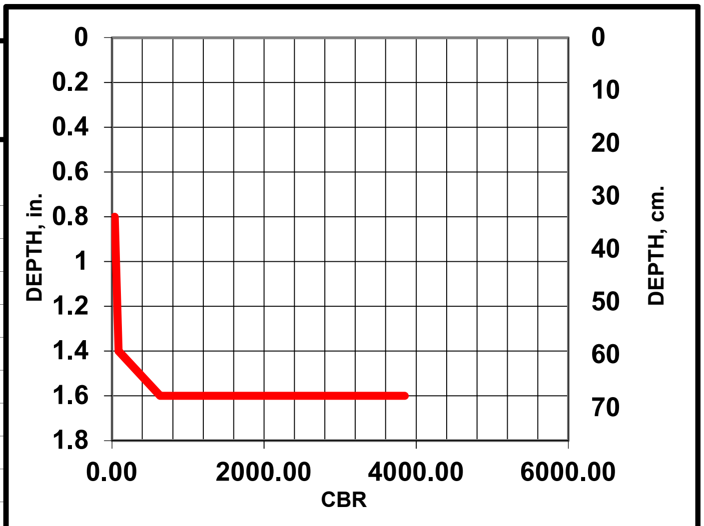
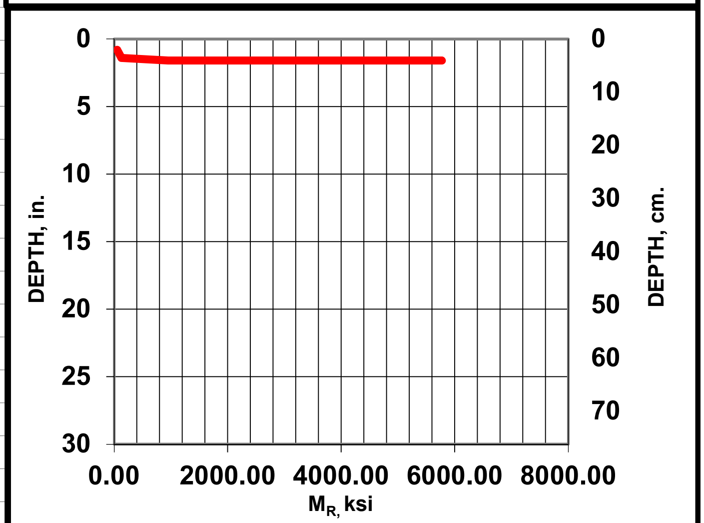
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





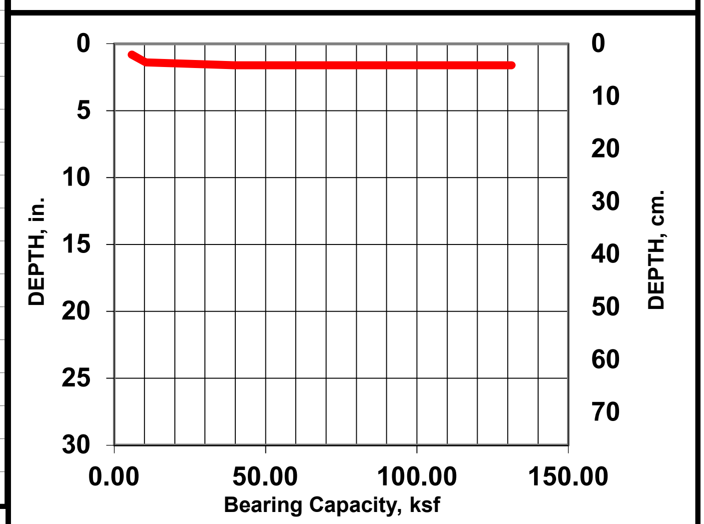


Figure 15a

Project Number: ANA22-064-00
Test Date: July 29, 2022

DCP TEST DATA

P-2

VIDA Amenity Center
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	1	30	1.2	6	9	1.80
1	2	20	2	22	33	4.25
1	3	20	2.8	35	52.5	5.79
1	3	20	3.5	35	52.5	5.79
1	4	20	4.3	48	72	7.14
1	7	20	5.1	90	135	10.84
1	5	20	5.9	62	93	8.46
1	2	20	6.7	22	33	4.25
1	2	20	7.5	22	33	4.25
1	2	20	8.3	22	33	4.25
1	2	20	9.1	22	33	4.25
1	3	20	9.8	35	52.5	5.79
1	3	25	10.8	27	40.5	4.87
1	2	15	11.4	31	46.5	5.34
1	2	20	12.2	22	33	4.25
1	3	20	13	35	52.5	5.79
1	2	20	13.8	22	33	4.25
1	2	15	14.4	31	46.5	5.34
1	2	20	15.2	22	33	4.25
1	2	15	15.7	31	46.5	5.34
1	2	20	16.5	22	33	4.25
1	3	20	17.3	35	52.5	5.79
1	3	20	18.1	35	52.5	5.79
1	3	20	18.9	35	52.5	5.79
1	3	20	19.7	35	52.5	5.79
-	-	-	-	-	-	-
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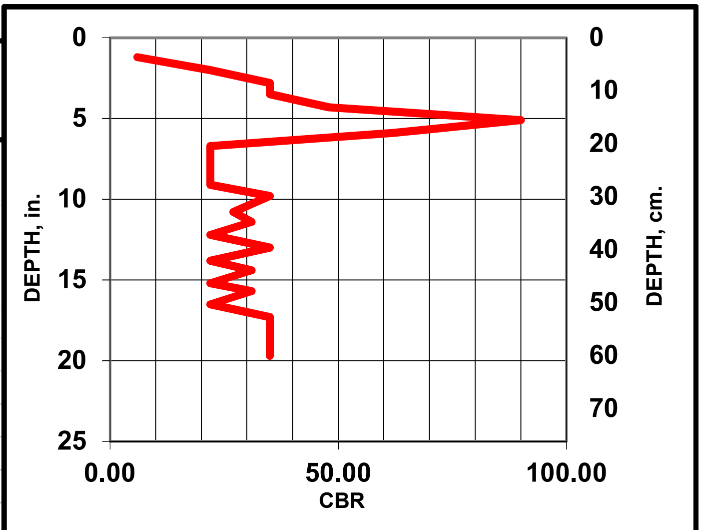
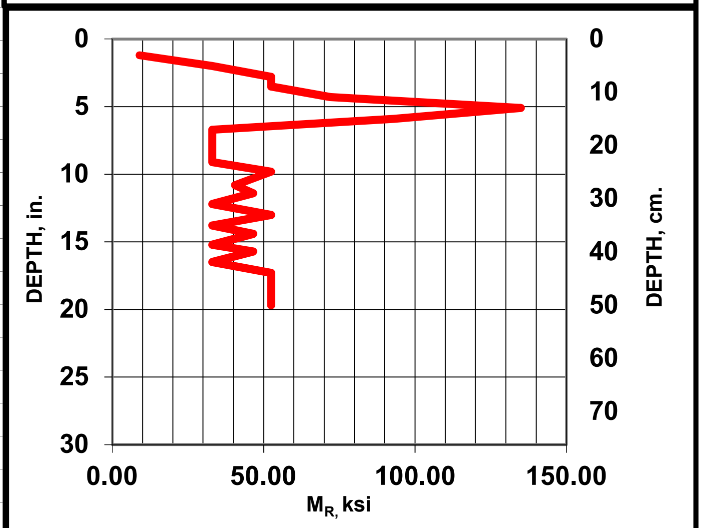
NOTES: Hammer 17.6 lbs = 1 Hammer 10.1 lbs = 2





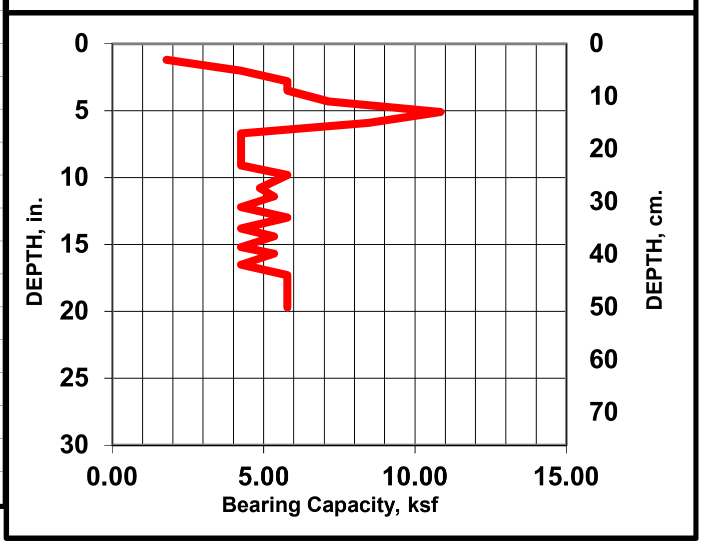


Figure 15b

Project Number: ANA22-064-00
Test Date: July 29, 2022

DCP TEST DATA

P-3

VIDA Amenity Center
San Antonio, Texas

Type of Ham.	No. of Blows	Penetration Incre. (mm)	Penetration Cumm. (in)	CBR (%)	M_R (ksi)	q_{ult} (ksf)
1	3	20	0.8	35	52.5	5.79
1	2	30	2	14	21	3.15
1	2	25	3	17	25.5	3.59
1	2	20	3.7	22	33	4.25
1	2	30	4.9	14	21	3.15
1	3	20	5.7	35	52.5	5.79
1	2	35	7.1	12	18	2.84
1	2	20	7.9	22	33	4.25
1	3	35	9.3	19	28.5	3.86
1	5	15	9.8	85	127.5	10.44
1	6	25	10.8	59	88.5	8.19
1	4	20	11.6	48	72	7.14
1	5	25	12.6	48	72	7.14
1	2	20	13.4	22	33	4.25
1	3	25	14.4	27	40.5	4.87
1	3	25	15.4	27	40.5	4.87
1	3	20	16.1	35	52.5	5.79
1	4	20	16.9	48	72	7.14
1	5	20	17.7	62	93	8.46
1	4	20	18.5	48	72	7.14
1	3	20	19.3	35	52.5	5.79
1	3	20	20.1	35	52.5	5.79
-	-	-	-	-	-	-
-	-	-	-	-	-	-
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-	-	-	-	-	-	-
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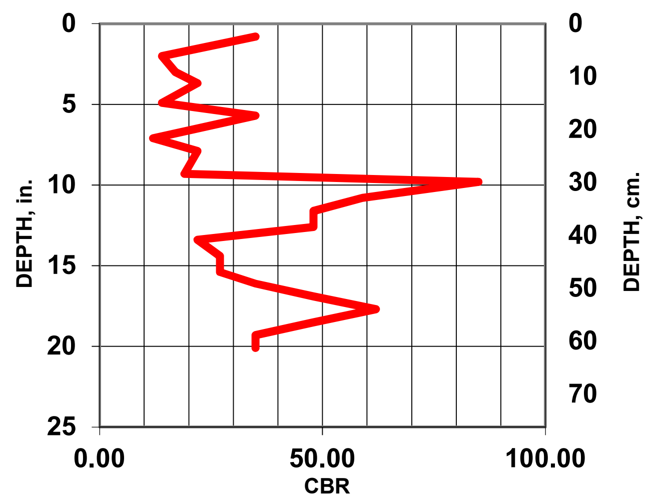
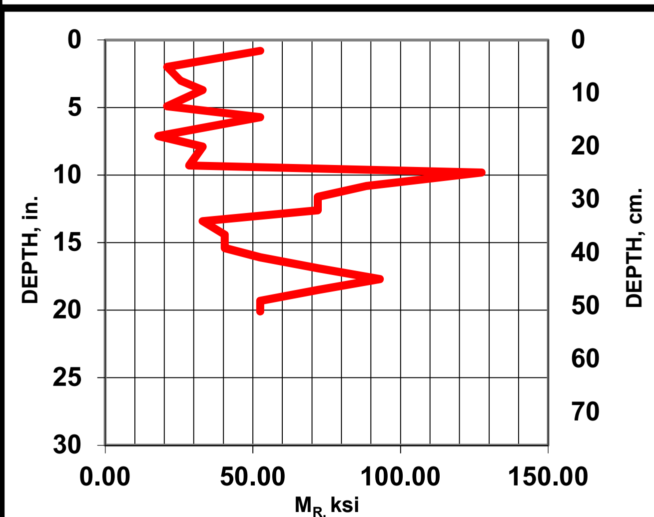
-	-	-	-	-	-	-
-	-	-	-	-	-	-

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





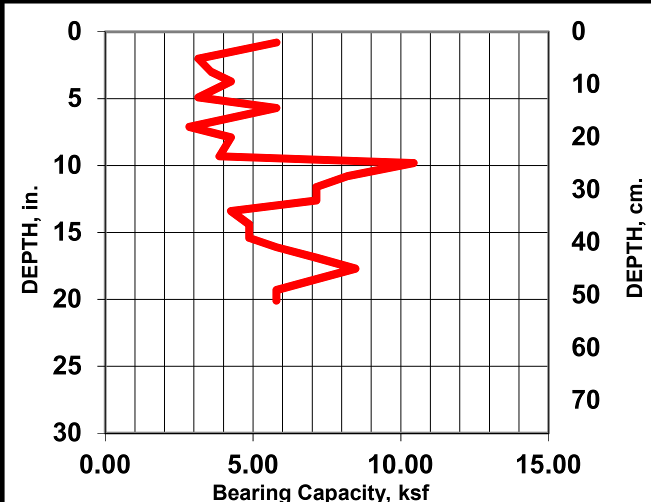


Figure 15c

Important Information about This
Geotechnical-Engineering Report

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

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

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply this report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are Not Final

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

A Geotechnical-Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

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

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

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

GBC GEOTECHNICAL BUSINESS COUNCIL of the Geoprofessional Business Association

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