Palo Alto College – Veteran's Center

Geotechnical Engineering Report

May 30, 2024 | Terracon Project No. 90235329

Prepared for:

Alamo College District
2222 N Alamo St
San Antonio, TX 78215

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May 30, 2024

Alamo College District
2222 N Alamo St
San Antonio, TX 78215

Attn: Ms. Ana Fasone
P: 210-445-5667
E: afasone@alamo.edu

Re: Geotechnical Engineering Report
Palo Alto College – Veteran's Center
1400 W Villaret Blvd.
San Antonio, Texas
Terracon Project No. 90235329

Dear Ms. Fasone:

We have completed the scope of Geotechnical Engineering services for the above referenced project in general accordance with Terracon Proposal No. P90235329 dated December 7, 2023. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning the design and construction of foundations and pavements for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,

Terracon
(Firm Registration No. F3272)

STATE OF TEXAS
ARINDAM BARKATAKI
104544
LICENSED
PROFESSIONAL ENGINEER

Ali Hashemi
Project Manager

Arin Barkataki, P.E.
Principal

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Introduction

This report presents the results of our subsurface exploration and Geotechnical Engineering services performed for the proposed veteran's center to be located at 1400 W Villaret Blvd. in San Antonio, Texas. The purpose of these services was to provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Seismic site classification per IBC
- Site preparation and earthwork
- Demolition considerations
- Dewatering considerations
- Foundation design and construction
- Floor slab design and construction
- Pavement design and construction
- Detention pond considerations

The Geotechnical Engineering Scope of Services for this project included drilling, laboratory testing, engineering analysis, and preparation of this report. The field program for this project included the advancement of two (2) test borings, each drilled to a depth of 50 feet below existing site grades.

Drawings showing the site and boring locations are shown on the **Site Location** and **Exploration Plan**, respectively. The results of the laboratory testing performed on soil samples obtained from the site during our field exploration are included on the boring logs and separate graphs in the **Exploration Results** section.

Project Description

Our initial understanding of the project was provided in our proposal and was discussed during project planning. A period of collaboration has transpired since the project was initiated, and our final understanding of the project conditions is as follows:

Item	Description
Information Provided	We have been provided the pertinent project details provided by Mr. Joseph H. Huizar, PE with Intelligent Engineering Services, LLP dated November 20, 2023, and via an email by Ms. Ana Fasone dated November 29, 2023. The request included project information and geotechnical requirement, and a borehole location plan.
Project Description	A new veteran's center with a footprint of about 7,000 sqft is planned to the south of the natatorium.
Finished Floor Elevation	Not provided
Maximum Loads	Columns loads: 100 to 350 kips
Pavements	Both asphalt and concrete are considered
Building Code	IBC 2021

Terracon should be notified if any of the above information is inconsistent with the planned construction, especially the grading limits, as modifications to our recommendations may be necessary.

Site Conditions

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Parcel Information	The project is located at 1400 W Villaret Blvd. in San Antonio, Texas. Latitude/Longitude 29.321774° N, 98.549319° W See **Site Location**
Existing Improvements	Existing buildings and tennis courts
Current Ground Cover	Bare soil, concrete
Existing Topography	A topographic survey map is not provided to Terracon currently. Based on Google Earth, average elevation across the building footprint is about EL. 627 feet.

Geotechnical Characterization

Site Geology

The San Antonio Sheet (1983) of the Geologic Atlas of Texas, published by the Bureau of Economic Geology of the University of Texas at Austin, has mapped the Wilcox formation at the project location. Locally, the Wilcox is mostly a mudstone with varying amounts of sandstone and lignite.

Subsurface Conditions

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration, laboratory data, geologic setting and our understanding of the project. This characterization, termed GeoModel, forms the basis of our geotechnical calculations and evaluation of the site. Conditions observed at each exploration point are indicated on the individual logs. The individual logs can be found in the **Exploration Results** and the GeoModel can be found in the **Figures section** of this report.

As part of our analyses, we identified the following model layers within the subsurface profile. For a more detailed view of the model layer depths at each boring location, refer to the GeoModel.

Model Layer	Layer Name	General Description
1	Fat Clay (CH)	Dark brown; Stiff to Very Stiff
2	Clayey Gravel (GC)	Light brown, Tan; Medium Dense to Very Dense
3	Lean Clay (CL)	Tan; Stiff to Hard

The individual logs can be found in the **Exploration Results** section of this report. It should be emphasized the stratification boundaries on the boring logs represent the approximate location of changes in native soil types; in situ, the transition between materials may be gradual.

Groundwater Conditions

Groundwater generally appears as either a permanent or temporary water source. Permanent groundwater is generally present year-round, which may or may not be influenced by seasonal and climatic changes. Temporary groundwater water is also referred to as a "perched" water source, which generally develops because of seasonal and climatic conditions.

The borings were advanced to the required depths using dry drilling techniques to evaluate groundwater conditions at the time of our field program. The boreholes were observed for the presence of groundwater during and after completion of drilling. Free water was not observed in any of the borings.

Seasonal variations such as amount of rainfall and runoff, climatic conditions and other factors generally result in fluctuations of the groundwater level over time. Groundwater seepage is possible at this site, particularly in the form of seepage traveling along granular material such as the Clayey Gravel zones. A relatively long period may be necessary for a groundwater level to develop and stabilize in a borehole. Therefore, groundwater levels during construction or at other times in the life of the structure may be higher or lower than the levels indicated on the boring logs. The foundation contractor should check the groundwater conditions just before foundation excavation activities. Long term observations in piezometers sealed from the influence of surface water are often required to define groundwater levels in materials of this type. Groundwater conditions should be evaluated immediately prior to construction.

Seismic Site Class

Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with 1613.2.2 in the 2021 IBC and Table 20.3-1 in the 2016 ASCE-7. Based on the soil properties observed at the site and as described on the exploration logs and results, our professional opinion is that a **Seismic Site Classification of D** be considered for the project. Subsurface explorations at this site were extended to a maximum depth of 50 feet. The site properties below the from boring termination depth down to 50 feet were estimated based on our experience and knowledge of geologic conditions of the general area.

Geotechnical Overview

Expansion Potential

Expansive soils are present at this site. The near surface, high plasticity fat clay could become unstable with typical earthwork and construction traffic, especially after precipitation events. Effective drainage should be completed early in the construction sequence and maintained after construction to avoid potential issues. If possible, the grading should be performed during the warmer and drier times of the year. If grading is performed during the winter months, an increased risk for possible undercutting and replacement of unstable subgrade will persist. Additional general site preparation recommendations, including subgrade improvement and fill placement, are provided in the **Earthwork** section.

The potential heave values were computed using the Potential Vertical Rise (PVR) method developed by TxDOT (TEX 124-E). This method is based on the plasticity of the soil, the soil's initial moisture condition, and overburden pressure. The method is approximate and based on relatively modest increases in moisture content. We estimate that the subgrade soils at this site exhibit a Potential Vertical Rise (PVR) of about 3 inches in their present condition. However, if source of excessive moisture such as inadequate drainage, ponded water and moisture infiltration beneath the foundation or pavement after construction is available, the actual heave values may far exceed the computed PVR values.

We understand numerous foundation movements have been reported in several of the campus buildings over the years. The site is underlain by highly expansive soils. Most of the foundation movement issues have been attributed to surface and subsurface drainage problems at the site. Some surface and subsurface drainage control measures have been implemented with varying degrees of success. **Therefore, we strongly recommend the owner consider the suspension of the building on drilled piers above grade.**

The foundation being considered must satisfy two independent engineering criteria with respect to the subsurface conditions encountered at this site. One criterion is the foundation system must be designed with an appropriate factor of safety to reduce the possibility of a bearing capacity failure of the soils underlying the foundation when subjected to axial and lateral load conditions. The other criterion is that the movement of the foundation system due to compression (consolidation or shrinkage) or expansion (swell) of the underlying soils must be within tolerable limits.

The report also provides recommendations to help mitigate the effects of soil shrinkage and expansion in pavement and grade-supported flatwork areas. However, even if these procedures are followed, some movement and (at least minor) cracking in the pavement and flatwork should be anticipated.

Sulfate Considerations

Sulfate tests were performed on selected samples collected from the borings to check for possible adverse reactions with concrete. Sulfate results are tabulated below:

Boring No.	Approximate Depth, feet	Sulfate Content, mg/Kg
B-1	2.5-4	1300

The sulfate concentration value is below the threshold level for adverse reactions based on TxDOT (>3,000mg/Kg), the National Lime Association (>3,000mg/Kg) and AASHTO (>5,000mg/Kg). Therefore, potential of adverse sulfate induced distress due to the addition of lime during lime stabilization is not a concern.

Water Soluble Sulfate Content in Soil (mg/kg)	Severity of Potential Exposure
> 10,000	Class 3

Water Soluble Sulfate Content in Soil (mg/kg)	Severity of Potential Exposure
1,500 – 10,000	Class 2
150 – 1,500	Class 1
0 – 150	Class 0

Based on the test results, the severity of potential exposure of concrete to sulfate attack typically falls under Class 1.

Pressuremeter Testing

Pressuremeter testing (PMT) of the substrata was performed at various depths at north of the boring location B-2. The PMT is a 'downhole' boring test that is performed on the natural undisturbed subsoils and is a means to evaluate the in-situ strength of the substrata without obtaining a physical sample. Unlike the sampling techniques available during exploratory drilling, which generally results in some sample disturbance, the PMT technology allows testing of the substrata without sampling and disturbance. The undrained strength values obtained from the pressuremeter testing are generally significantly higher than those obtained from laboratory tests.

The PMT consists of placing a cylindrical probe in the boring and expanding the probe to pressurize the soil on the sides of the borehole. An in-situ stress-strain curve is developed from the relationship between the pressure applied to the probe and the relative increase in the size of the probe as it expands the borehole radially. The test yields a PMT modulus which characterizes the lateral stress-strain behavior of the soil in the material's pseudo-elastic range. In cases where the applied stress reaches the material's elastic behavior limit, a "creep" pressure can also be discerned. The PMT modulus and the creep limit aid in estimation of the load-deflection behavior, including the lateral and vertical bearing capacities of the soil being evaluated. The allowable bearing pressures and side friction values presented in this report for drilled shaft foundation design were derived from the PMT data, in addition to the conventional boring information and laboratory test data.

Demolition Considerations

The existing tennis court is to be demolished and replaced with a new veteran's center structure. As a result, abandoned (or to be abandoned) underground utilities will be present within the footprint area of the planned structures. Utilities and associated backfill and granular bedding material can provide avenues for subsurface water to enter under the structure subgrade. We recommend that all abandoned utility lines be completely removed from the proposed structure areas. Abandoned pipes which remain underground should be grouted.

Any below-grade foundation or structures removal associated with demolition will likely create large subsurface voids. It is very important that all subsurface voids formed from the removal of the foundation system be backfill completely with moisture conditioned, compacted, engineered fill as described **Earthwork** section of this report. It is our experience that improperly backfilled excavations can cause significant settlement under and around the proposed structures.

As an alternative to compacted soil backfill, a flowable fill material may be considered. Flowable fill, or slurry, when properly designed provides a competent subgrade and can still be readily excavated if the utilities require repair or maintenance. In addition, flowable fill does not need to be placed in lifts, compacted, or tested.

Earthwork

Earthwork is anticipated to include clearing and grubbing, excavations, and engineered fill placement. The following sections provide recommendations for use in the preparation of specifications for the work. Recommendations include critical quality criteria, as necessary, to render the site in the state considered in our geotechnical engineering evaluation for foundations, and pavements.

General Site Preparation

Prior to construction, the work area should be cleared of loose topsoil and any unsuitable materials. After stripping and grubbing, the exposed subgrade should be proof-rolled where possible to aid in locating loose or soft areas. Proof-rolling can be performed with a fully loaded dump truck or comparable pneumatic tired vehicle. Soils that are observed to rut or deflect excessively (typically greater than 1-inch) under the moving load should be overexcavated to provide a firm, uniform bearing layer. The proof rolling and overexcavation activities should be witnessed by a representative of the Geotechnical Engineer and should be performed during a period of dry weather. Subgrade stabilization may also be performed as described below if the exposed subgrade exhibits yielding or pumping under construction traffic.

- Removal and replacement with select fill.
- Chemical treatment of the soil to dry and increase the stability of the subgrade.
- Drying by natural means if the schedule allows.

Pad Preparation

We recommend the floor slabs be constructed as structurally suspended floor system. **Therefore, remedial earthwork measures in pad area will not be required other than general site grading.**

Flatwork

As previously stated, we estimate that the subgrade soils at this site exhibit a swell potential (movement) of about 3 inches in their present condition.

Flatwork which abuts the building: Movement sensitive flatwork i.e., flatwork next to the building should be suspended. If the movement sensitive flatwork is not suspended, then movement and cracking should be expected that may result in uneven flatwork which in turn may cause trip hazard or reverse surface flow or doors which drag on the flatwork when opened. Consider including the door stoops into the slab to prevent interference with door operations. We recommend hinge slab be provided in the areas transitioning from movement sensitive flatwork to pavements. Dowel length and quantities should be determined by the Structural Engineer so that it performs as intended.

Flatwork which does not abut the building: Any other flatwork away from the building may be grade supported provided the owner understands that, without any subgrade modifications, pavement movement up to 3 inches should be expected. Otherwise, the pad beneath flatwork should also be suspended or prepared as recommended below:

- Depth of excavation, thickness of moisture conditioned subgrade, thickness of Select Fill and the corresponding PVR is listed in the table below.

Excavation Depth (ft)	Thickness of Select Fill (ft)	Resulting PVR (inches)
3	3	2
7	7	1

- After completing stripping operations as discussed in the **General Site Preparation** section, excavate to depths as furnished in table above in the pad area.

- After excavating to the depth specified above, the exposed subgrade in the pad should be proof rolled with a fully loaded dump truck or comparable pneumatic tired vehicle to evidence any weak yielding zones. A Terracon Geotechnical Engineer or their representative should be present to observe proof rolling operations. If any weak yielding zones are present, they should be over excavated, both vertically and horizontally, to expose competent soil.

- After proof-rolling and the replacement of weak yielding zones, the exposed subgrade should be scarified, moisture conditioned between 0 and +4 percentage points of the optimum moisture content and compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D 698.

- Place Imported Cohesive Select Fill soil in loose lifts of about 8 inches to the thicknesses listed on the table above. Each lift should be moisture conditioned between -2 and +3 percentage points of the optimum moisture content, and then compacted to at least 98 percent of the maximum dry density determined in accordance with ASTM D 698.

- If grades are to be raised further, then Imported Cohesive Select Fill should be used to achieve Finished Pad Elevation.

Clay Cap- If not covered with concrete flatwork or pavements, the upper 2 feet (clay "cap") of the 5-foot (horizontal) overbuild should consist of a cohesive clay with a plasticity index greater than 25. The purpose of the clay cap is to reduce the potential for water to infiltrate the building pad causing the subgrade soils to swell. The clay "cap" material should have at least 70 percent by weight passing the No. 200 Sieve and no more that 15 percent by weight retained in the No. 4 Sieve. The clay "cap" may be replaced with concrete flatwork or pavement extending to the edge of the foundation. Properly compacted, this clay layer should help to reduce migration of moisture into the select fill below.

Building Perimeter Protection – **It should be noted ingress of moisture into the building pad or prolonged dry spells of the subgrade will lead to swell - shrink of the clay within and beneath the building pad, which may manifest in the form of cracks in the floor slabs and walls.** Plants in landscape areas adjacent to the building should be carefully selected to avoid species which will produce lateral migrating root structures which would tend to deplete the moisture from the prepared building pad. Drip irrigation system should be in landscape beds to maintain a relatively constant soil moisture. The irrigation system will need to be diligently maintained throughout the life of the structure. If these measures are properly implemented, they should provide adequate perimeter moisture protection. A plastic pond liner material which wraps up the side of the adjacent tilt-wall panel, buried about 3 feet below grade and sloped at least 2% away from the building up to 10 feet from the building perimeter or to the edge of pavement should also help prevent moisture ingress into the building pad.

Landscaping - We understand that landscaping is vital to the aesthetics of any project and is generally typical for this type of project. The owner and design team should be made aware that placing large bushes and trees adjacent to the structure may contribute to future distress to the foundation system. Vegetation placed in landscape beds adjacent to the structure should be limited to plants and shrubs that will not exceed a mature height of about three (3) to four (4) feet. Large bushes and trees that will generally exceed these heights should be planted at a reasonable distance away from the structure so their canopy or "drip line" does not extend over the structure when the tree reaches maturity. Watering of vegetation should be performed in a timely and controlled manner and prolonged watering should be avoided.

Fill Material Types

Earthen materials used for select and general fill should meet the following material property requirements.

Soil Type [1]	USCS Classification	Acceptable Parameters
Select fill	CL LL ≤40 and 7<PI≤20 ▪ % passing #200 sieve ≥65% ▪ Maximum particle size 1½"	All locations and elevations.
On-Site Soil (General Fill)	CL, CH, GC	Onsite soils should not be used as Select Fill.

Fill Placement and Compaction Requirements

Select fill and general fill should meet the following compaction requirements.

Item	Requirements
Fill Lift Thickness	All fill should be placed in thin, loose lifts of about 8 inches, with compacted thickness not exceeding 6 inches.
Compaction of On-Site Soil	95 percent of the material's Standard Proctor maximum dry density (ASTM D 698).
Compaction of Select Fill Soil	98 percent of the material's Standard Proctor maximum dry density (ASTM D 698).
Moisture Content of On-Site Soil	The materials should be moisture conditioned between 0 and +4 percentage points of the optimum moisture content

Item	Requirements
Moisture Content of Select Fill	The materials should be moisture conditioned between -2 and +3 percentage points of the optimum moisture content

Utility Trench Backfill

Any soft or unsuitable materials encountered at the bottom of utility trench excavations should be removed and replaced with structural fill or bedding material in accordance with public works specifications for the utility be supported. This recommendation is particularly applicable to utility work requiring grade control and/or in areas where subsequent grade raising could cause settlement in the subgrade supporting the utility. Trench excavation should not be conducted below a downward 1:1 projection from existing foundations without engineering review of shoring requirements and geotechnical observation during construction.

On-site materials are considered suitable for backfill of utility and pipe trenches from 1 foot above the top of the pipe to the final ground surface, provided the material is free of organic matter and deleterious substances.

Trench backfill should be mechanically placed and compacted as discussed earlier in this report. Compaction of initial lifts should be accomplished with hand-operated tampers or other lightweight compactors. Where trenches are placed beneath slabs or footings, the backfill should satisfy the gradation and expansion index requirements of engineered fill discussed in this report. Flooding or jetting for placement and compaction of backfill is not recommended. Utility trench backfill compaction should be 95 percent of Standard Proctor for paved and structure areas and 90 percent of Standard Proctor for unpaved and non-structure areas.

Utility trenches are a common source of water infiltration and migration. Utility trenches penetrating beneath the building should be effectively sealed to restrict water intrusion and flow through the trenches, which could migrate below the building. The trench should provide an effective trench plug that extends at least 5 feet from the face of the building exterior. The plug material should consist of cementitious flowable fill or low permeability clay. The trench plug material should be placed to surround the utility line. If used, the clay trench plug material should be placed and compacted to comply with the water content and compaction recommendations for structural fill stated previously in this report. Utilities transitioning from grade supported areas to the building portion which is suspended should be carefully designed to accommodates differential movements. Flexible connections can be considered at the interface between soil supported and suspended areas.

Grading and Drainage

Effective drainage should be provided during construction and maintained throughout the life of the new improvements. After pad construction, we recommend verifying final grades to document that effective drainage has been achieved. Grades around the structure should also be periodically inspected and adjusted as necessary, as part of the structure's maintenance program.

Proper site drainage should be maintained during the entire construction phase so that ponding of surface runoff does not occur and cause construction delays and/or inhibit site access, particularly in cut areas. During construction, it is possible that the surficial soils may become excessively wet as a result of inclement weather conditions. When the moisture content of these gravelly soils elevates above what is considered to be the optimum range of moisture for compaction operations, they can become difficult to handle and compact. If such conditions create a hindrance to compaction operations or site access, cement may be mixed with these soils to improve their workability. The additive can be mixed as per 2014 TxDOT Item 260 (lime). The purpose of the additive is to dry out the subgrade and improve site access. The strict requirements for curing and actual quantity of additive can be at the discretion of the contractor. The subgrade, however, should be compacted to at least 95 percent of the maximum dry density as per ASTM D 698 at moisture contents ranging from -2 to +3 percentage points of the optimum moisture content.

Flatwork and pavements will be subjected to post-construction movement. Maximum grades that are feasible should be used for paving and flatwork to prevent water from ponding. Allowances in final grades should also consider post-construction movement of flatwork, particularly if such movements are deemed critical. Where paving or flatwork abuts the structure, joints should be effectively sealed and maintained to prevent surface water infiltration. In areas where sidewalks or paving do not immediately adjoin the structure, we recommend that protective slopes be provided with a grade of at least five percent for at least 10 feet from perimeter walls (except in areas where ADA ramps are required; these should comply with state and local regulations). Backfill against grade beams, exterior walls, and in utility and sprinkler line trenches should be well compacted and free of construction debris to reduce the possibility of moisture infiltration.

All downspouts should be extended to discharge storm water a minimum of five feet away from the building perimeter. Downspouts should be discharged onto splash blocks and to an area that is properly graded to direct water away from the perimeter of the buildings. Routine maintenance of the downspouts and gutters should be performed to keep them free of debris and working properly.

Sprinkler mains and spray heads should preferably be located at least 5 feet away from the structures such that they cannot become a potential source of water directly adjacent to the structures. Watering of vegetation should be performed in a timely and controlled manner and prolonged watering should be avoided. Landscaped irrigation adjacent to the foundation units should be minimized or eliminated. Special care should be taken such that underground utilities do not develop leaks with time.

Earthwork Construction Considerations

It is anticipated that excavations for the proposed construction can be accomplished with conventional earthmoving equipment. Upon completion of filling and grading, care should be taken to maintain the subgrade water content prior to construction of foundation elements and pavements. Construction traffic over the completed subgrades should be avoided. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to construction.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, "Excavations" and its appendices, and in accordance with any applicable local, and/or state regulations.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

Deep Foundations

Drilled Shaft Design Parameters

Structural column loads for the proposed structure may be supported on underreamed drilled piers bearing at a depth no shallower than 40 feet below the crawl space and 45 feet below existing grade, whichever is deeper. This depth was chosen as a result of the uplift forces anticipated from the on-site soils encountered during our subsurface exploration and to bear the piers below the zone of seasonal moisture variation. **To enhance the uplift resistance capacity of piers, we recommend the piers to be belled**.

Depth (feet)	Allowable End Bearing (psf)	Allowable Skin Friction (psf)
0 to 10	---	---
10 to 20	---	600
20 to 30	---	1,000
30 to 40	---	1,500
40 to 50	30,000	2,000

1/ The allowable skin friction values include a factor of safety of 2.
2/ The allowable bearing pressure includes a factor of safety of 2.
3/ Upper 10 feet of the soil are considered not to contribute to side friction.

The bearing pressures presented above assume that the bearing surface will be free and clean of soft or moist material and loose debris. The allowable end bearing and skin friction values are based on center-to-center spacing of at least three bell diameters. A closer spacing may be considered but it may reduce the axial capacity of the foundation depending on the spacing pattern of the foundations. If a clearance of three bell diameters cannot be maintained in every case, the above bearing capacities should be reduced by 20 percent. Drilled piers installed at a center-to-center clearance of two bell diameters or less are not recommended.

In addition to the axial compressive loads on the piers, these piers will also be subjected to axial tension loads due to the expansive soil conditions and possibly due to other induced structural loading conditions. To compute the axial tension force due to the swelling soils, the following equation may be used.

$$Q_u = 80 \bullet d$$

Where: Q_u = Uplift force due to expansive soil conditions in kips (k)
d = Diameter of pier shaft in feet (ft)

This calculated force may be used to compute the longitudinal reinforcing steel required in the pier to resist the uplift force induced by the swelling clays. However, the cross-sectional area of the reinforcing steel should not be less than one (1) percent of the gross cross-sectional area of the drilled pier shaft. The reinforcing steel should extend from the top to the bottom of the shaft to resist this potential uplift force.

The ultimate uplift resistance of the underreamed pier may be evaluated using the following equation:

$$Q_{ar} = 6(D^2 - d^2) + W_p + P_{DL}$$

Where: Q_{ar} = Ultimate uplift resistance of underreamed pier in kips (k)

D	=	Diameter of underream in feet (ft)
d	=	Diameter of pier shaft in feet (ft)
W_p	=	Weight of the drilled pier in kips (k)
P_{DL}	=	Dead Load acting on the drilled pier in kips (k)

The structural engineer may want to factor the dead load value based on their degree of certainty.

We should note that the diameter of the underream should be large enough to overcome the uplift forces induced on the pier without causing a localized soil failure to the soils immediately overlying the underream. We recommend the ratio of underream diameter to shaft diameter be larger than 2:1 but in no case should this ratio exceed 3:1 so that the likelihood of problems developing during construction is reduced.

Total settlements, based on the indicated bearing pressures, should be approximately ¾ inch or less for properly designed and constructed drilled piers. Differential settlement may also occur between adjacent piers. The amount of differential settlement could approach 50 to 75 percent of the total pier settlement. For properly designed and constructed piers, differential settlement between adjacent piers is estimated to be less than ½ of an inch. Settlement response of drilled piers is impacted more by the quality of construction than by soil-structure interaction.

Improper pier installation could result in differential settlements significantly greater than we have estimated. In addition, larger magnitudes of settlement should be expected if the soil is subjected to bearing pressures higher than the allowable values presented in this report.

Lateral Loading

Criteria for the LPILE program are contained in the table below. The parameters provided in the table can be used for lateral analysis of drilled shafts.

Layer	LPILE Soil Types	Depth to Bottom of Layer (feet)	Effective Unit Weight (pcf)	Undrained Shear Strength (psf)	Soil Strain Factor (ε_{50})
1	Stiff Clay without Free Water	5	120	1,000	LPILE Default
2	Stiff Clay without Free Water	20	125	2,500	LPILE Default
3	Stiff Clay without Free Water	30	125	3,000	LPILE Default
4	Stiff Clay without Free Water	50	125	4,000	LPILE Default

Drilled Shaft Construction Considerations

The pier excavations should be augered and constructed in a continuous manner. Steel and concrete should be placed in the pier excavation immediately following drilling and evaluation for proper bearing stratum, embedment, and cleanliness. In no circumstances should the pier excavation remain open overnight.

During the time of our drilling operations, subsurface water was not encountered in the borings. Subsurface water levels are influenced by seasonal and climatic conditions which result in fluctuations in subsurface water elevations. Gravels encountered in the borings is prone to sloughing. Therefore, the contractor should be prepared to use temporary casing should water be encountered and/or sloughing of the excavation sidewalls occur. Prior to any excavation, the contractor should verify the groundwater levels. The contractor should consider performing a "test" pier excavation to determine the constructability of a drilled pier with the dry auger process. The casing and slurry methods are discussed below.

Casing Method - Casing should provide stability of the excavation walls and should reduce water influx; however, casing may not completely eliminate subsurface water influx potential. In order for the casing to be effective, a "water tight" seal must be achieved between the casing and surrounding soils. The drilling subcontractor should determine casing depths and casing procedures. Water that accumulates in excess of 3 inches in the bottom of the pier excavation should be pumped out prior to steel and concrete placement. If the water is not pumped out, a closed-end tremie should be used to place the concrete completely to the bottom of the pier excavation in a controlled manner to effectively displace the water during concrete placement. If water is not a factor, concrete may be placed with a short tremie so the concrete is directed to the bottom of the pier excavation. The concrete should not be allowed to ricochet off the walls of the pier excavation nor off the reinforcing steel. If this operation is not successful or to the satisfaction of the foundation contractor, the pier excavation should be flooded with fresh water to offset the differential water pressure caused by the unbalanced water levels inside and outside of the casing. The concrete should be tremied completely to the bottom of the excavation with a closed-end tremie.

Removal of casing should be performed with extreme care and under proper supervision to reduce mixing of the surrounding soil and water with the fresh concrete. Rapid withdrawal of casing or the auger may develop suction that could cause the soil to intrude into the excavation. An insufficient head of concrete in the casing during its withdrawal could also allow the soils to intrude into the wet concrete. Both of these conditions may induce "necking", a section of reduced diameter, in the pier.

Slurry Method - As an alternate to the use of casing to install the pier foundations, water or a weighted drilling fluid may be considered. Slurry displacement drilling can only prevent sloughing and water influx but cannot control sloughing once it has occurred. Therefore, slurry displacement drilling techniques must begin at the ground surface, not after sloughing materials are encountered.

Typical drilling fluids include those which contain polymers or bentonite. If a polymer is used with "hard" mixing water, a water softening agent may be required to achieve intimate mixing and the appropriate viscosity. The polymer manufacturer should be consulted concerning proper use of the polymer. If bentonite slurry is used, the bentonite should be mixed with water several hours before placing in the pier excavation. Prior mixing gives the bentonite sufficient time to hydrate properly. The drilling fluid should only be of sufficient viscosity to control sloughing of the excavation walls and subsurface water flow into the excavation. Care should be exercised while extracting the auger so that suction does not develop and cause disturbance or create "necking" in the excavation walls as described above. Casing should not be employed in conjunction with the slurry drilling technique due to possible trapping of loose soils and slurry between the concrete and natural soil.

The use of weighted drilling fluid when installing drilled pier foundations requires extra effort to ensure an adequate bearing surface is obtained. A clean-out bucket should be used just prior to pier completion in order to remove any cuttings and loose soils which may have accumulated in the bottom of the excavation. Steel and concrete should be placed in the excavation immediately after pier completion. A closed-end tremie should be used to place the concrete completely to the bottom of the excavation in a controlled manner to effectively displace the slurry during concrete placement. The concrete should be placed completely to the bottom of the excavation with a closed-end tremie in the pier excavation if more than six (6) inches of water is ponded on the bearing surface or the water should be pumped from the excavation. A short tremie may be used if the excavation has less than 6 inches of ponded water. The fluid concrete should not be allowed to strike the pier reinforcement, temporary casing (if required) or excavation sidewalls during concrete placement

All aspects of concrete design and placement should comply with the American Concrete Institute (ACI) 318 Code Building Code Requirements for Structural Concrete, ACI 336.1 Standard Specification for the Construction of Drilled Piers, and ACI 336.3R entitled Suggested Design and Construction Procedures for Pier Foundations.

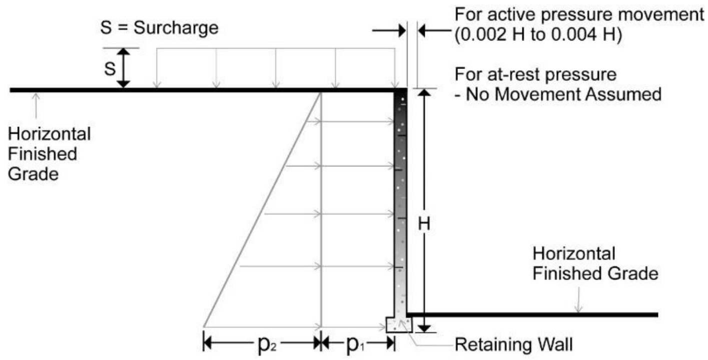
Foundation Construction Monitoring

The performance of the foundation system for the proposed structure will be highly dependent upon the quality of construction. Thus, we recommend that fill pad compaction and foundation installation be monitored full time by an experienced Terracon soil technician under the direction of our Geotechnical Engineer. During foundation installation, the base should be monitored to evaluate the condition of the subgrade. We would be pleased to develop a plan for compaction and foundation installation monitoring to be incorporated in the overall quality control program.

Lateral Earth Pressure

Retaining walls may be constructed at the site due to the grading. The recommendations given in the following paragraphs are applicable to the design of rigid retaining walls subject to slight rotation, such as cantilever, or gravity type concrete walls. Note that the parameters are not applicable to the design of mechanically stabilized earth (MSE) or modular block wall.

Walls with unbalanced backfill levels on opposite sides should be designed for earth pressures at least equal to those indicated in the following table. Earth pressures will be influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition assumes no wall movement. The recommended design lateral earth pressures do not include a factor of safety.. Presented below are earth pressure coefficients that may be used to design the wall.



Undrained Condition (With Hydrostatic Pressure)					
Earth Pressure Conditions	**Backfill Type**	**Coefficient of Earth Pressure**	**Equivalent Fluid Density (pcf)**	**Lateral Pressure due to Surcharge (psf)**	**Earth Pressure (psf)**
Active (K_a)	Granular Select Fill[1]	0.33	85	0.33S	85H
	Lean Clay	0.53	93	0.53S	93H
	Free Draining Granular Fill[2]	0.22	77	0.22S	77H
At-Rest (K_o)	Granular Select Fill	0.50	96	0.50S	96H
	Lean Clay	0.70	102	0.70S	102H
	Free Draining Granular Fill	0.36	87	0.36S	87H
Passive (K_p)	Granular Select Fill	3.0	265	---	---
	Lean Clay	1.9	170	---	----
	Free Draining Granular Fill	4.60	373	---	---

1/ Granular Select Fill should conform to the gradation requirements of 2014 TxDOT Item 247, Type A, Grade 1-2 material.
2/ Free Draining Granular Fill should conform to the gradation requirements of ASTM C33, Grade 57 coarse aggregate material.

Drained Condition (Without Hydrostatic Pressure)					
Earth Pressure Conditions	**Backfill Type**	**Coefficient of Earth Pressure**	**Equivalent Fluid Density (pcf)**	**Lateral Pressure due to Surcharge (psf)**	**Earth Pressure (psf)**
Active (K_a)	Granular Select Fill[1]	0.33	43	0.33S	43H
	Lean Clay	0.53	63	0.53S	63H
	Free Draining Granular Fill[2]	0.22	28	0.22S	28H
At-Rest (K_o)	Granular Select Fill	0.5	65	0.5S	65H
	Lean Clay	0.70	83	0.70S	83H
	Free Draining Granular Fill	0.36	46	0.36S	46H
Passive (K_p)	Granular Select Fill	3.0	390	---	---
	Lean Clay	1.9	227	---	---
	Free Draining Granular Fill	4.6	600	---	---

1/ Granular Select Fill should conform to the gradation requirements of 2014 TxDOT Item 247, Type A, Grade 1-2 material.
2/ Free Draining Granular Fill should conform to the gradation requirements of ASTM C33, Grade 57 coarse aggregate material.

<u>Applicable conditions to the above include:</u>

For active earth pressure, wall must rotate about base, with top lateral movements of about 0.002 H to 0.004 H, where H is wall height.

- For passive earth pressure to develop, wall must move horizontally to mobilize resistance.
- Uniform surcharge, where S is surcharge pressure.
- In-situ soil backfill weight a maximum of 120 pcf.
- Horizontal backfill, compacted to 95 percent of standard Proctor maximum dry density.
- Loading from heavy compaction equipment not included.

- No hydrostatic pressures acting on wall.
- No dynamic loading.
- No safety factor included in lateral earth pressures.
- Ignore passive pressure in upper 2 feet.

Backfill placed against structures should consist of granular soils or low plasticity cohesive soils. For the granular values to be valid, the granular backfill must extend out from the base of the wall at an angle of at least 30, 45, and 60 degrees from vertical for the active, at rest, and passive cases, respectively. If it is not possible to construct a wedge of granular backfill, then a minimum of 12 inches of clean gravel should be placed behind the wall for drainage purposes and the onsite soil values presented in the table should be used. To control hydrostatic pressure behind the wall we recommend that a drain be installed at the foundation wall with a collection pipe leading to a reliable discharge. Heavy equipment should not operate within a distance closer than the exposed height of retaining walls to prevent lateral pressures more than those provided.

To calculate the resistance to sliding, a value of 0.35 should be used as the ultimate coefficient of friction between the footing and the underlying soil or select fill. The "active" earth pressures need to be resisted by the passive earth pressures on the face of the retaining wall base or a key (if applicable), and by the friction that will develop along the base of the wall. Passive fluid density values furnished in the table above should be used to calculate passive earth pressure. It should be noted that values furnished in the table are ultimate values. A factor of safety of 2 should be used to determine allowable values. Allowable bearing pressure along the base of the wall should be 2,000 psf. This bearing pressure includes a factor of safety against a bearing capacity failure of at least 3. Allowable bearing pressures are also based on the bearing surface being comprised of compacted soil that is free and clean of all debris and loose material.

A drainage system is recommended regardless of the backfill used. Weep holes along the front of, and a drain system located behind, the wall will provide an outlet for water which may collect in the wall backfill. The wall backfill should drain much more effectively if a granular material is used behind the wall. The free-draining backfill should be protected from clogging by surrounding finer-grained soils through use of a geotextile filter fabric. The filter fabric should prevent the finer-grained materials from infiltration into the interstitial space between the individual grains of the free-draining backfill.

It is critical that surface water infiltration be reduced behind the wall. The upper 1 to 2 feet of backfill should be a clay soil having a Plasticity Index in the range of 25 to 40; or, the backfill material should be covered with pavement. This clay soil cap or pavement coupled with sloping the ground surface away from the wall will help to reduce infiltration of surface water into the backfill. The clay soil should be at least 12 inches in thickness and compacted to at least 95 percent of the maximum dry density as evaluated by the ASTM D 698 test method. The clay should be moisture conditioned between -2 and +3 percentage points of the optimum moisture content.

Suspended Floor Slabs

We understand structurally suspended floor slabs with a crawl spaces will be considered. **Note that we do not recommend use of carton forms to establish the void beneath the slab.**

For a structurally suspended floor slab system at this site, Terracon recommends a void space of at least 12 inches beneath the floor slabs, structural beams, and subfloor plumbing systems. In many cases, the thickness of the void space is several feet to facilitate maintenance activities in the crawl space. Subfloor plumbing pipes should be installed using an approved suspended system and should have a similar void space between the pipe and the subgrade.

Drainage beneath the structure must be designed to remove and/or reduce the possibility of water accumulation in these areas. The subgrade below the floor slab should be sloped to appropriate drainage outlets. Surface and subsurface drainage of water away from the building will enhance the performance of the foundation.

In addition, proper ventilation should be provided to reduce the possibility that a high humidity environment could develop in the void space areas. Measures should be taken to maintain voids beneath the perimeter beams. Carton forms (minimum 8 inches high) with appropriate fill retainers maybe used for this purpose.

If the site has been prepared in accordance with the requirements noted in **Earthwork**, the following design parameters are applicable.

Drain System

We recommend that a permanent drain system be designed along the perimeter of the structure. The drain should be installed to at least 10 feet below existing grade and to at least 3 feet below the lowest point of the crawl space, whichever is deeper. The drain system should be designed to gravity flow toward common sump areas for continuous collection and removal of water. The perimeter collector drains should preferably consist of clean, well graded fine aggregate for concrete. Compatible perforated collector pipes with a minimum diameter of 4 inches should be provided at/near the bottom of the aggregate. Alternatively, a manufactured drainage mat may be considered. If a mat is used, the manufacturer of the geotextile drainage mat should be consulted in regard to applicability, selection, and placement of the drainage mat. In addition, a representative of the drainage mat manufacturer should be present during initial and/or critical phases of the installation such that proper installation techniques are utilized. Consideration should be given to the use of utility trenches outside of the building area as interceptor drains. For instance, these excavations can be backfilled near full depth with granular material and sloped to drain into storm sewers or similar outflow areas. To separate granular backfill materials within drainage system and trench backfill materials, geotextiles should be placed or wrapped around the drainage gravel. As a result, some additional "protection" from the negative effects of subsurface water flow may be gained with little extra cost.

Pavements

Both flexible and rigid pavement systems may be considered for the project. Based on our knowledge with similar projects, we anticipate that traffic loads will be produced primarily by automobile traffic, delivery trucks, trash removal trucks and occasional fire trucks.

Subgrade Preparation

Prior to construction, any vegetation, loose topsoil and any otherwise unsuitable materials should be removed from the new pavement areas. After stripping, the subgrade should be proof-rolled where possible to aid in locating loose or soft areas. Proof-rolling can be performed with a 20-ton roller or fully loaded dump truck. Wet, soft, low-density or dry material should either be removed, or moisture conditioned and recompacted to the moisture contents and densities described in section **Fill Placement and Compaction Requirements** prior to placing fill.

Due to the presence of the expansive clay soils at the site, movement of the pavements should be expected. As previously mentioned, the Potential Vertical Rise (PVR) is about 3 inches for the current conditions at the site. Therefore, pavement movement up to 3 inches should be expected unless subgrade in the pavement areas is prepared as recommended in **Flatwork** section.

Pavement Design Recommendations

For this project Light and Heavy pavement section alternatives have been provided. Light is for areas expected to receive only car traffic. Heavy assumes areas with heavy traffic, such as trash pickup areas, delivery areas and main access drive areas.

Design of Asphaltic Concrete (HMAC) pavements are based on the procedures outlined in the 1993 Guideline for Design of Pavement Structures by the American Association of State Highway and Transportation Officials (AASHTO-1993). Design of Portland Cement

Concrete (PCC) pavements are based upon American Concrete Institute (ACI) 330R; Guide for Design and Construction of Concrete Parking Lots.

Asphalt Pavement		
Layer	**Light Duty (inch)**	**Heavy Duty (inch)**
Hot Mix Asphaltic Concrete	2.0	3.0
Granular Base Material	6.0	10.0
Lime Treated Subgrade	8.0	8.0

Concrete Pavement		
Layer	**Light Duty (inch)**	**Heavy Duty (inch)**
Reinforced Concrete	5.0	6.0
Lime treated Subgrade	8.0	8.0

We recommend that primary driveways and dumpster pads (if any) be constructed of heavy-duty reinforced concrete pavement. The concrete pad areas for the dumpster areas should be designed so that the vehicle wheels of the collection truck are supported on the concrete while the dumpster is being lifted to support the large wheel loading imposed during waste collection. Dumpster areas that are not designed in this manner often experience localized failures due to large wheel loading imposed during waste collection. Reinforced concrete pavements typically result in better performance and less maintenance than flexible pavement systems in these areas.

Where practical, we recommend early-entry cutting of crack-control joints in PCC pavements. Cutting of the concrete in its "green" state typically reduces the potential for micro-cracking of the pavements prior to the crack control joints being formed, compared to cutting the joints after the concrete has fully set. Micro-cracking of pavements may lead to crack formation in locations other than the sawed joints, and/or reduction of fatigue life of the pavement.

Pavement Section Materials

Presented below are selection and preparation guidelines for various materials that may be used to construct the pavement sections. Submittals should be made for each pavement material. The submittals should be reviewed by the Geotechnical Engineer and appropriate members of the design team and should provide test information necessary to verify full compliance with the recommended or specified material properties.

- **Hot Mix Asphaltic Concrete Surface Course** - The asphaltic surface material should meet the specification requirements of 2014 TxDOT Standard Specification Item 341 or SS3076/3077, Type D.

- **Concrete** - Concrete should have a minimum 28-day design compressive strength of 4,000 psi.

- **Granular Base Material** - Base material may be composed of crushed limestone base meeting all of the requirements of 2014 TxDOT Item 247, Type A, Grade 1-2; including triaxial strength. The material should be compacted to at least 95 percent of the maximum dry density as determined in accordance with ASTM D 1557 at moisture contents ranging from -2 and +3 percentage points of the optimum moisture content.

- **Lime Treated Subgrade -**The subgrade may be treated with hydrated lime in accordance with TxDOT Item 260 in order to improve its strength and improve its load carrying capacity. We recommend 6 percent hydrated lime will be required. This is equivalent to about 40 pounds of hydrated lime per square yard for a 8-inch treatment depth. The optimum lime content should result in a soil-lime mixture with a pH of at least 12.4 and should reduce the Plasticity Index to 20 or less.

  The lime should initially be blended with a mixing device such as a Pulvermixer, sufficient water added, and be allowed to cure for at least 48 hours. After curing, the lime-soil should be remixed to meet the in-place gradation requirements of Item 260 and compacted to at least 95 percent of the maximum dry density. The target lime content and optimum moisture content should be determined in accordance with ASTM D968 Standard Proctor.

Details regarding subgrade preparation, fill materials, placement and compaction are presented in **Earthwork** section under subsections **Fill Materials Requirement** and **Compaction Requirements.**

Pavement Joints and Reinforcement

The following is recommended for all concrete pavement sections in this report. Refer to ACI 330 "Guide for Design and Construction of Concrete Parking Lots" and "TxDOT Standard Specifications" for additional information.

Item	Description
Reinforcing Steel	No 3 reinforcing steel bars at 18 inches on-center-each-way, Grade 60. It is imperative that the distributed steel be positioned accurately in the pavement cross section, namely 2 inches from the top of the pavement.
Contraction Joint Spacing	12.5 feet each way for pavement thickness of 5 to 5.5 inches.

Item	Description
	15 feet each way for pavement thickness of 6 inches or greater. Saw cut control joints should be cut within 6 to 12 hours of concrete placement.
Contraction Joint Depth	At least ¼ of pavement thickness.
Contraction Joint Width	One-fourth inch or as required by joint sealant manufacturer.
Construction Joint Spacing	To attempt to limit the quantity of joints in the pavement, consideration can be given to installing construction joints at contraction joint locations, where it is applicable.
Construction Joint Depth/Width	Full depth of pavement thickness. Construct sealant reservoir along one edge of the joint. Width of reservoir to be ¼ inch or as required by joint sealant manufacturer. Depth of reservoir to be at least ¼ of pavement thickness.
Isolation Joint Spacing	As required to isolate pavement from structures, etc.
Isolation Joint Depth	Full depth of pavement thickness.
Isolation Joint Width	½ to 1 inch or as required by the joint sealant manufacturer.
Expansion Joint	In this locale, drying shrinkage of concrete typically significantly exceeds anticipated expansion due to thermal affects. As a result, the need for expansion joints is eliminated provided all joints (including saw cuts) are sealed. Construction of an unnecessary joint may be also become a maintenance problem. All joints should be sealed. If all joints, including saw cuts, are not sealed then expansion joints should be installed.

All construction joints have dowels. Dowel information varies with pavement thickness as presented as follows:

Layer	5 inches	6 inches
Dowels	⅝ inch diameter	¾ inch diameter
Dowel Spacing	12 inches on center	12 inches on center
Dowel Length	12 inches long	14 inches long
Dowel Embedment	5 inches	6 inches

Pavement Maintenance and Drainage

It is of paramount importance to maintain proper drainage, maintain subgrade moisture levels and provide routine maintenance on the pavement to help long-term pavement performance. The following recommendations should be implemented:

- The subgrade and the pavement surface should be designed to promote proper surface drainage, preferably at a minimum grade of 2 percent.
- Install joint sealant and seal cracks immediately.
- Extend curbs into the treated subgrade for a depth of at least 4 inches to help reduce moisture migration into the subgrade soils beneath the pavement section.
- Place compacted, low permeability clayey backfill against the exterior side of the curb and gutter.
- Slope subgrade in landscape islands to low points should drain to an appropriate outlet.
- Edge drains are recommended along pavement/ landscape borders.
- Strip (wick) drains installed behind the curbs will also help protect the pavements from water which ponds behind the curbs.

Note that even with the subgrade preparation and pavement maintenance measures, minor pavement distress should be anticipated.

Detention Pond

We understand that a detention pond is being considered for the proposed development. We anticipate that the detention pond may be constructed of earthen materials. Generally, the base of the earthen ponds is critical when considering permeability and limiting detained waters from infiltrating through the detention pond's base. It is not uncommon for it to be acceptable for some water seepage to escape through the sidewalls of the berms of the pond. Typically, the design of the detention pond will attempt to limit the amount of detained water that may escape. Polyvinyl Chloride (PVC) sheeting with a protective soil cover can also be considered for the pond's base. The top 2 to 6 feet of existing soils at the site is primarily comprised of Fat Clay (CH) over Clayey Gravel (GC) soils underlain by Lean Clay (CL). We do not recommend the use of the on-site Lean Clay (CL) or Clayey Gravel (GC) soils to build the berm. Instead, a more cohesive material, such as Fat Clay (CH), should be used for this purpose.

All clayey soil material to be used as fill for the berm construction should have a Plasticity Index (PI) of at least 30 percent and a Liquid Limit (LL) greater than 50 percent. This type of material may be suitable to construct the pond's base and berms provided it is clean of organic material and gravel and is properly moisture conditioned and compacted.

The berms of the ponds should be stable. Generally, for clayey soils, the berms should be constructed with slopes not exceeding 3 horizontal to 1 vertical (3:1). Considerations should be given to vegetating the berms and base of the ponds to limit erosion. The clayey materials selected to construct the ponds should be placed in loose lifts not exceeding 8 inches and then be compacted lifts of about 6 inches in thickness. The materials should be moisture conditioned to between optimum to +4 percentage points of the optimum moisture content and compacted to at least 95 percent of ASTM D 698. Care should be taken to limit dry clods and provide a relatively homogenous mixture of clay material for the pond's base.

General Comments

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

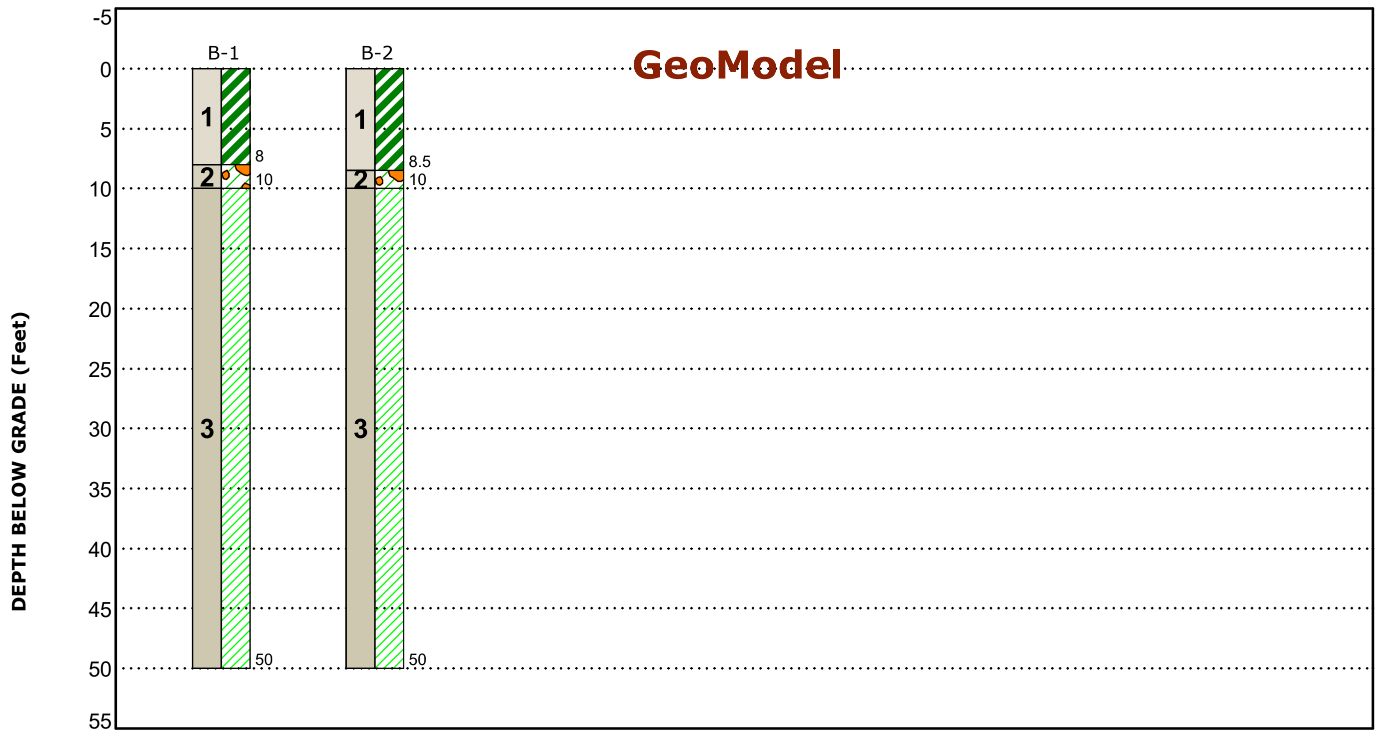
Our services and any correspondence are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly effect excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety and cost estimating including excavation support and dewatering requirements/design are the responsibility of others. Construction and site development have the potential to affect adjacent properties. Such impacts can include damages due to vibration, modification of groundwater/surface water flow during construction, foundation movement due to undermining or subsidence from excavation, as well as noise or air quality concerns. Evaluation of these items on nearby properties are commonly associated with contractor means and methods and are not addressed in this report. The owner and contractor should consider a preconstruction/precondition survey of surrounding development. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

Figures

Contents:

6000 Northwest Pkwy Ste 100
San Antonio, TX



This is not a cross section. This is intended to display the Geotechnical Model only. See individual logs for more detailed conditions.

Model Layer	Layer Name	General Description
1	Fat Clay (CH)	Dark brown; medium stiff to very stiff
2	Clayey Gravel (GC)	Light brown, tan; medium dense to very dense
3	Lean Clay (CL)	Tan; stiff to hard

Legend

Fat Clay
Clayey Gravel
Lean Clay

NOTES:
Layering shown on this figure has been developed by the geotechnical engineer for purposes of modeling the subsurface conditions as required for the subsequent geotechnical engineering for this project.
Numbers adjacent to soil column indicate depth below ground surface.

Attachments

Exploration and Testing Procedures

Field Exploration

Number of Borings	Approximate Boring Depth (feet)	Location
(B-1 and B-2)	50	New Veteran's Center

Boring Layout and Elevations: Terracon personnel provided the boring layout using handheld GPS equipment (estimated horizontal accuracy of about ±10 feet) and referencing existing site features. If elevations and a more precise boring layout are desired, we recommend borings be surveyed.

Subsurface Exploration Procedures: We advanced the borings with a truck-mounted, drill rig using continuous flight augers (solid stem). Five samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. In the thin-walled tube sampling procedure, a thin-walled, seamless steel tube with a sharp cutting edge was pushed hydraulically into the soil to obtain a relatively undisturbed sample. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the last 12 inches of a normal 18-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. A 3-inch O.D. split-barrel sampling spoon with 2.5-inch I.D. ring lined sampler was used for soil sampling. For safety purposes, all borings were backfilled with auger cuttings after the groundwater observations were completed.

We also observed the boreholes while drilling and at the completion of drilling for the presence of groundwater. The groundwater levels are shown on the attached boring logs.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials observed during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests. The laboratory testing program included the following types of tests:

- Water content
- Atterberg limits
- Percent Passing No. 200 sieve
- Sulfate Tests

The laboratory testing program often included examination of soil samples by an engineer. Based on the results of our field and laboratory programs, we described and classified the soil samples in accordance with the Unified Soil Classification System.

Site Location and Exploration Plans

Contents:

Note: All attachments are one page unless noted above.

Site Location

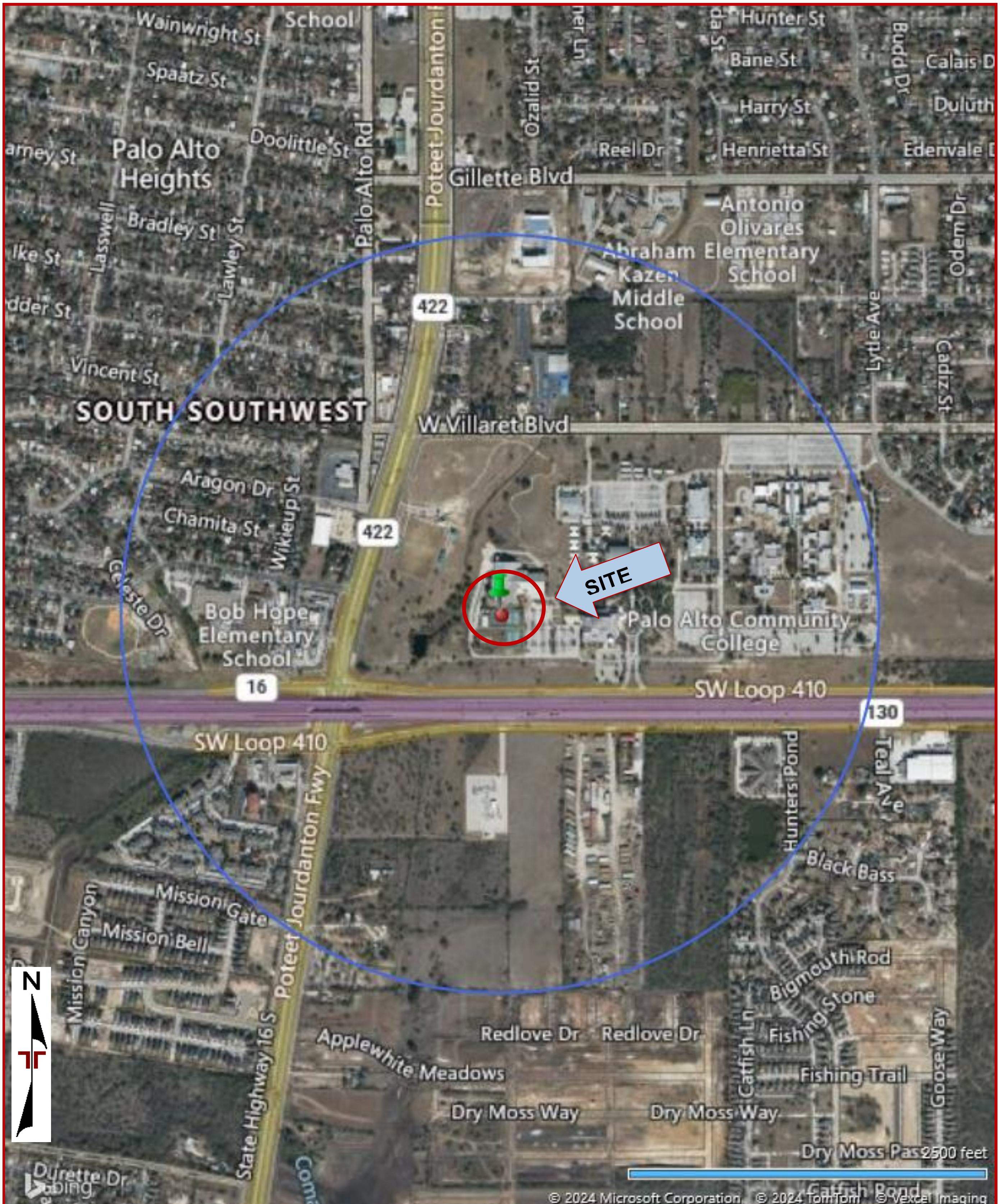


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration Plan

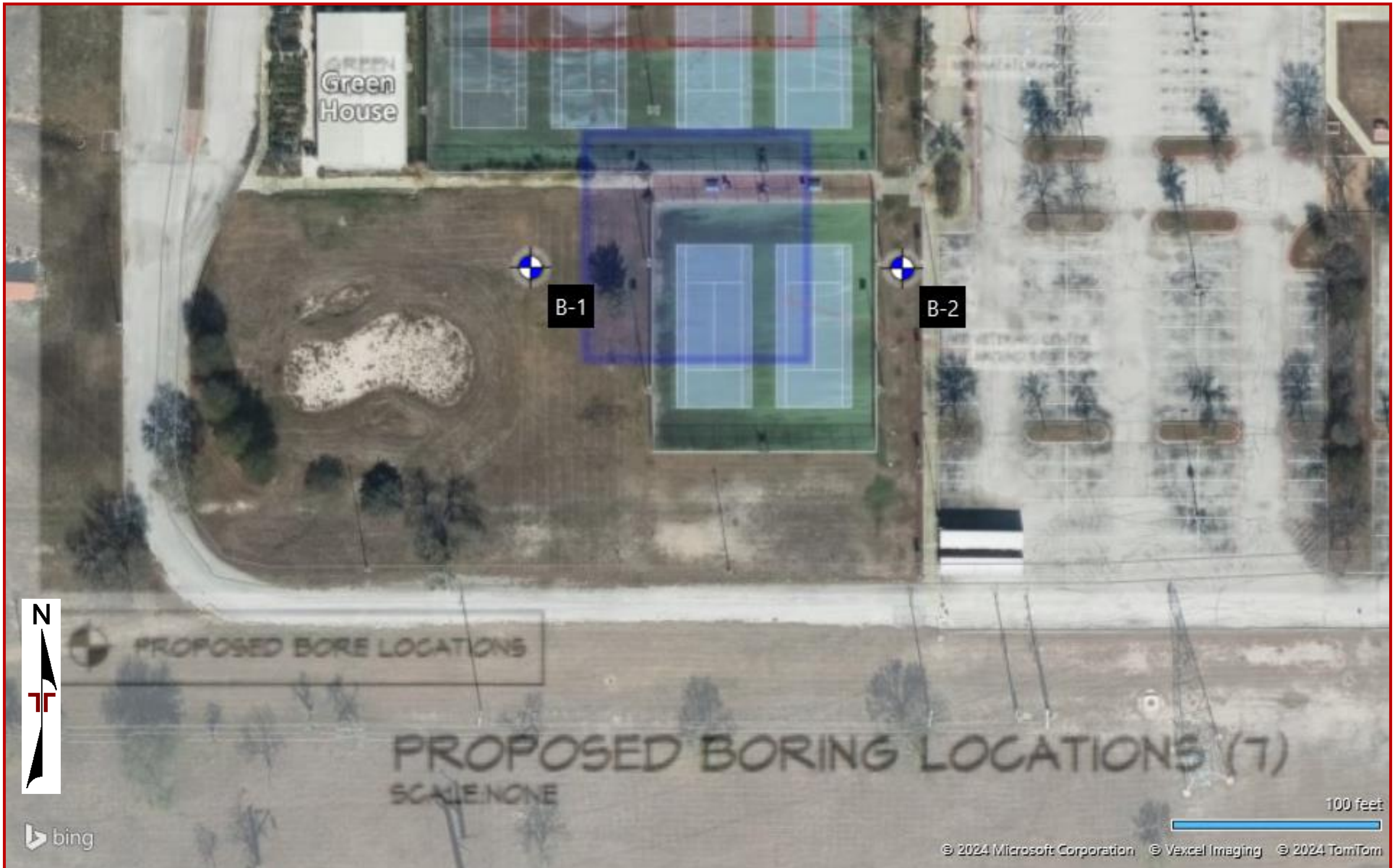


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

Exploration and Laboratory Results

Contents:

Note: All attachments are one page unless noted above.

Terracon
6000 Northwest Pkwy Ste 100
San Antonio, TX

Boring Log No. B-1

Location: See Exploration Plan

Latitude: 29.3215° Longitude: -98.5495°

Model Layer	Graphic Log	Depth (Ft.) / Description	Depth (Ft.)	Water Level Observations	Sample Type	Field Test Results	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
1		**FAT CLAY (CH)**, dark brown, medium stiff to very stiff				5-4-2 N=6	20.2		
						3-3-7 N=10	23.0		
			5			3.5 (HP)	21.8	57-15-42	82
		8.0				3.5 (HP)	23.2	64-20-44	
2		**CLAYEY GRAVEL (GC)**, light brown to tan, medium dense				10-9-13 N=22	14.9		37
		10.0	10						
3		**LEAN CLAY (CL)**, tan, stiff to hard							
		highly calcareous white clay between 13.5 to 15 feet				3-7-7 N=14	30.1		
			15						
						6-8-12 N=20	28.1		
			20						
		gravelly between 23.5 to 25 feet				50/3"	10.9		43
			25						
						11-13-18 N=31	24.7		
			30						
						12-14-18 N=32	25.4		
			35						
						10-16-20 N=36	25.1		
			40						
						17-23-24 N=47	25.0		
			45						
		50.0				10-15-20 N=35	25.1		
		Boring Terminated at 50 Feet	50						

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).

See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
No free water observed during augering

Drill Rig
CME 75

Hammer Type
Automatic

Driller
Burge

Notes

Advancement Method
Flight Auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Johnny

Boring Started
02-20-2024

Boring Completed
02-20-2024

Terracon
6000 Northwest Pkwy Ste 100
San Antonio, TX

Boring Log No. B-2

Location: See Exploration Plan

Latitude: 29.3215° Longitude: -98.5489°

Model Layer	Depth (Ft.)	Description	Field Test Results	Water Content (%)	Atterberg Limits LL-PL-PI	Percent Fines
1	0.0–8.5	**FAT CLAY (CH)**, dark brown, stiff to very stiff	3-3-5 N=8	14.8		
			6-7-8 N=15	15.3	53-29-24	
			6-6-8 N=14	17.3		
	8.5		3-5-22 N=27	20.4	57-19-38	77
2	8.5–10.0	**CLAYEY GRAVEL (GC)**, light brown to tan, very dense	38-37-50/5"	10.2		
3	10.0–50.0	**LEAN CLAY (CL)**, tan, very stiff to hard	11-14-17 N=31	20.9	39-15-24	
			7-10-15 N=25	27.0		
			10-12-16 N=28	26.1		99
			10-16-20 N=36	25.4		
			16-18-22 N=40	25.6		
			12-15-20 N=35	25.4		100
			13-17-21 N=38	26.3		
	50.0		19-21-24 N=45	27.9		
		Boring Terminated at 50 Feet				

See Exploration and Testing Procedures for a description of field and laboratory procedures used and additional data (If any).
See Supporting Information for explanation of symbols and abbreviations.

Water Level Observations
No free water observed during augering

Drill Rig
CME 75

Hammer Type
Automatic

Driller
Burge

Notes

Advancement Method
Flight Auger

Abandonment Method
Boring backfilled with auger cuttings upon completion.

Logged by
Johnny

Boring Started
02-20-2024

Boring Completed
02-20-2024

ALAMO ANALYTICAL LABORATORIES, LTD. **Date:** *05-Mar-24*

Analytical Results Report

Client:	Terracon Consultants, Inc.	**Collection Date:**	2/26/2024
Lab Order:	2403004	**Matrix:**	SOLID
Project ID:		**Lab ID:**	2403004-01A
Project Name:	90235329		
Client Sample ID:	B - 1 2.5 - 4		

Analyses	Result	Report Limit	Units	Dilution	Date Analyzed
TEX-620-J			**TX620J**		Analyst: **YK**
Sulfate	1300	125	mg/Kg	5	05-Mar-24

Approved by: Reddy Gosala, Laboratory Direct

Report of Laboratory Analysis

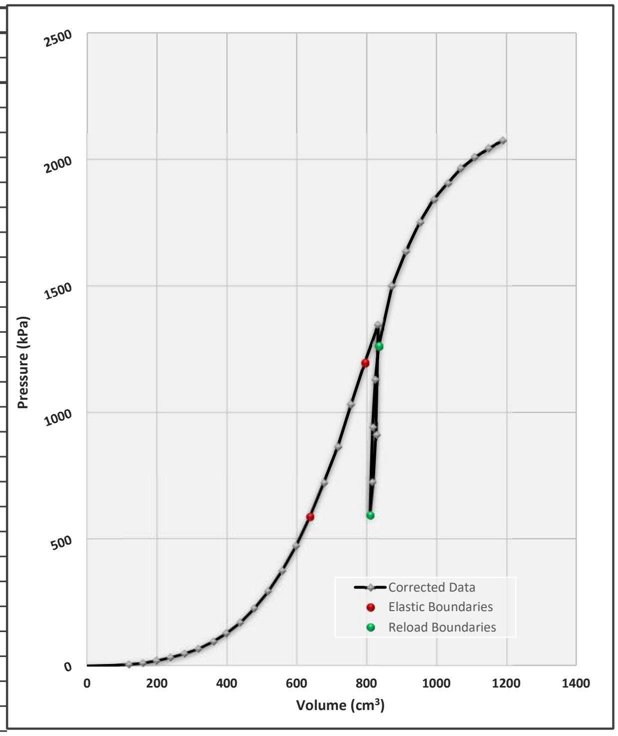
MAPES IN-SITU

PRESSUREMETER TEST

Project:	90235317	**Boring ID:**	B-1	**Test Depth (ft):**	11.5
City, State:	San Antonio, TX	**Mapes In-Situ No:**	P2024011	**Client:**	Terracon

Test date:	2/29/24	Probe body SN:	001A 1900011	Pressure Calibration ID:	P 19 (1)
Pressuremeter SN:	001A17002	Probe diameter (mm):	73.8	Volume Calibration ID:	V 19 (1)
Pressuremeter model:	TEXAM^{e}	Calibration tube I.D. (mm):	76.2	Calibration coefficient, c (cm^3/kPa):	0.006643
Test zone drilling method:	Shelby Tube Sampling	Calibration tube O.D. (mm):	101.6	Reload calibration coefficient, c_r (cm^3/kPa):	0.005778
Poisson's Ratio of soil/rock:	0.33	Tubing length (m):	30	Contact volume, V_a (cm^3):	398
Method for estimating P_L:	1/V vs. P			Initial volume of probe, V_0 (cm^3):	2068

Raw Test Data		Pressure Calibration		Volume Calibration		Corrected Data		
Volume	Pressure	Volume	Pressure	Volume	Pressure	Volume	$\Delta R/R_o$	Pressure
cm^3	kPa	cm^3	kPa	cm^3	kPa	cm^3	%	kPa
0.0	-25	0.0	4	0.0	8	0.0	0.00	-3
36.8	-20	73.9	11	196.4	509	36.9	0.89	-2
76.7	-15	155.6	15	206.6	1032	76.8	1.84	0
117.7	-9	237.4	18	215.2	1578	117.7	2.81	4
157.6	-2	316.9	20	220.4	2005	157.6	3.74	9
198.8	9	398.8	22	225.3	2465	198.8	4.69	18
239.2	23	477.2	23	230.4	3037	239.0	5.62	31
278.4	39	557.1	24	238.1	4050	278.1	6.51	46
319.0	60	640.4	25	245.5	5013	318.6	7.42	66
360.2	89	722.8	26	252.5	5990	359.6	8.34	94
399.6	123	803.0	27	260.0	7003	398.7	9.21	127
439.7	166	884.0	28	266.6	8007	438.6	10.09	169
480.1	222	964.1	29	272.9	9026	478.6	10.97	225
520.7	291	1044.9	30	278.8	9963	518.7	11.84	293
559.8	372	1125.3	31			557.3	12.67	374
601.2	471	1205.7	33			598.0	13.54	472
640.9	585	1285.9	34			637.0	14.37	586
681.8	720	1365.9	34			677.1	15.21	720
722.6	863	1447.1	36			716.8	16.04	863
762.1	1031					755.3	16.84	1030
802.1	1195					794.1	17.64	1194
842.0	1348					833.0	18.44	1346
832.7	913					826.6	18.31	911
823.1	725					818.3	18.13	724
813.3	593					809.3	17.95	592
822.8	942					816.5	18.10	941
832.3	1131					824.8	18.27	1130
842.4	1262					834.0	18.46	1260
882.6	1501					872.6	19.24	1499
923.0	1638					912.1	20.04	1635
963.0	1754					951.4	20.83	1751
1003.3	1844					991.1	21.62	1840
1043.5	1909					1030.8	22.41	1905
1083.7	1969					1070.6	23.19	1964
1123.9	2013					1110.5	23.97	2008
1163.9	2050					1150.3	24.74	2044
1203.4	2082					1189.6	25.50	2075
				Reload Cycle				
				231.8	2446			
				246.8	4997			



Interpreted Test Results

Deformation Modulus, E_P	28,658	kPa	4,156	psi
Reload Modulus, E_R	204,409	kPa	29,647	psi
At-Rest Horizontal Pressure, P_{OH}	50	kPa	7	psi
Yield Pressure, P_F	1,194	kPa	173	psi
Limit Pressure, P_L	2,626	kPa	381	psi
Net Limit Pressure, P_L*	2,576	kPa	374	psi
E_P / P_L	10.9			
P_L / P_F	2.2			

Remarks

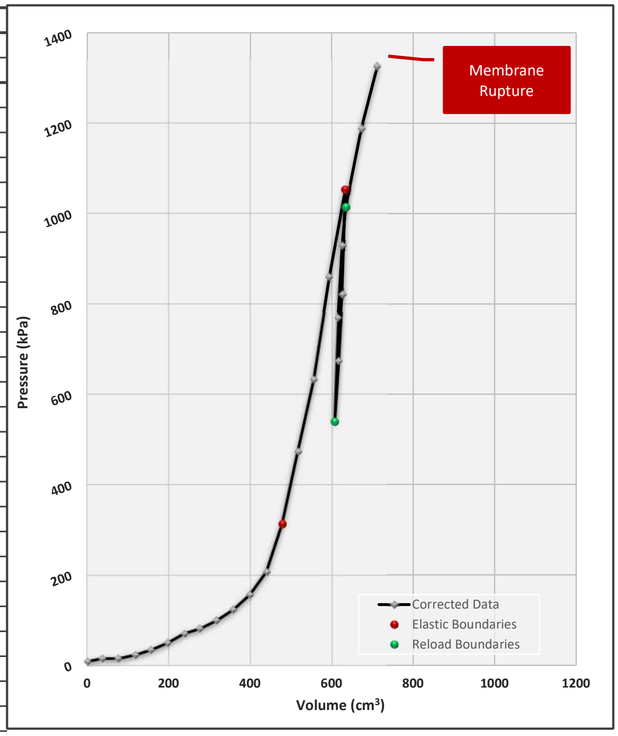
PRESSUREMETER TEST

MAPES IN-SITU

Project:	90235317	Boring ID:	B-1	Test Depth (ft):	32.5
City, State:	San Antonio, TX	Mapes In-Situ No:	P2024011	Client:	Terracon

Test date:	2/29/24	Probe body SN:	001A 1900011	Pressure Calibration ID:	P 19 (1)
Pressuremeter SN:	001A17002	Probe diameter (mm):	73.8	Volume Calibration ID:	V 19 (1)
Pressuremeter model:	TEXAM[e]	Calibration tube I.D. (mm):	76.2	Calibration coefficient, c (cm^3/kPa):	0.006643
Test zone drilling method:	Shelby Tube Sampling	Calibration tube O.D. (mm):	101.6	Reload calibration coefficient, c_r (cm^3/kPa):	0.005778
Poisson's Ratio of soil/rock:	0.33	Tubing length (m):	30	Contact volume, V_a (cm^3):	399
Method for estimating P_L:	1/V vs. P			Initial volume of probe, V_0 (cm^3):	2068

Raw Test Data		Pressure Calibration		Volume Calibration		Corrected Data		
Volume	Pressure	Volume	Pressure	Volume	Pressure	Volume	$\Delta R/R_o$	Pressure
cm^3	kPa	cm^3	kPa	cm^3	kPa	cm^3	%	kPa
0.0	-76	0.0	4	0.0	8	0.0	0.00	9
36.1	-66	73.9	11	196.4	509	36.5	0.88	15
76.6	-62	155.6	15	206.6	1032	77.0	1.84	16
117.6	-52	237.4	18	215.2	1578	118.0	2.81	23
157.2	-39	316.9	20	220.4	2005	157.5	3.74	35
197.8	-22	398.8	22	225.3	2465	197.9	4.67	50
237.4	0	477.2	23	230.4	3037	237.4	5.58	71
277.9	12	557.1	24	238.1	4050	277.8	6.50	82
318.3	31	640.4	25	245.5	5013	318.1	7.41	100
358.8	56	722.8	26	252.5	5990	358.4	8.32	124
399.4	90	803.0	27	260.0	7003	398.8	9.22	157
439.8	141	884.0	28	266.6	8007	438.8	10.10	207
479.0	247	964.1	29	272.9	9026	477.3	10.94	313
518.4	408	1044.9	30	278.8	9963	515.7	11.77	473
558.8	568	1125.3	31			555.1	12.62	633
599.5	796	1205.7	33			594.2	13.46	860
640.0	989	1285.9	34			633.4	14.29	1053
630.0	757	1365.9	34			625.0	14.11	821
620.5	610	1447.1	36			616.4	13.93	674
609.7	475					606.6	13.72	539
620.3	705					615.6	13.91	769
631.2	865					625.5	14.12	929
640.4	950					634.1	14.30	1014
680.3	1125					672.9	15.12	1188
720.1	1263					711.7	15.93	1326
				Reload Cycle				
				231.8	2446			
				246.8	4997			



Interpreted Test Results

Deformation Modulus, E_P	33,089	kPa	4,799	psi
Reload Modulus, E_R	121,914	kPa	17,682	psi
At-Rest Horizontal Pressure, P_{OH}	80	kPa	12	psi
Yield Pressure, P_F	1,053	kPa	153	psi
Limit Pressure, P_L	n.a.	kPa	n.a.	psi
Net Limit Pressure, P_L*	n.a.	kPa	n.a.	psi
E_P / P_L	n.a.			
P_L / P_F	n.a.			

Remarks

Membrane ruptured prior to fully defining the plastic phase of the test. Accordingly, Limit Pressure and Net Limit Pressure values could not be interpreted.

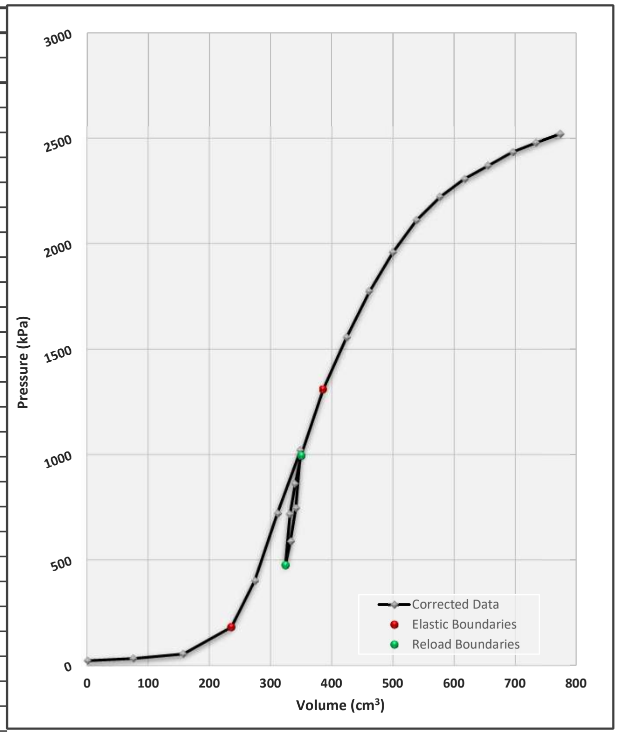
MAPES IN-SITU

PRESSUREMETER TEST

Project:	90235317	Boring ID:	B-1	Test Depth (ft):	42.5
City, State:	San Antonio, TX	Mapes In-Situ No:	P2024011	Client:	Terracon

Test date:	2/29/24	Probe body SN:	2000020	Pressure Calibration ID:	P 20a (1)
Pressuremeter SN:	001A17002	Probe diameter (mm):	70	Volume Calibration ID:	V 20a (1)
Pressuremeter model:	TEXAM^{e}	Calibration tube I.D. (mm):	76.2	Calibration coefficient, c (cm^3/kPa):	0.008717
Test zone drilling method:	Shelby Tube Sampling	Calibration tube O.D. (mm):	101.6	Reload calibration coefficient, c_r (cm^3/kPa):	0.010091
Poisson's Ratio of soil/rock:	0.33	Tubing length (m):	50	Contact volume, V_a (cm^3):	236
Method for estimating P_L:	1/V vs. P			Initial volume of probe, V_0 (cm^3):	1682

Raw Test Data		Pressure Calibration		Volume Calibration		Corrected Data		
Volume	Pressure	Volume	Pressure	Volume	Pressure	Volume	$\Delta R/R_\circ$	Pressure
cm^3	kPa	cm^3	kPa	cm^3	kPa	cm^3	%	kPa
0.0	-82	0.0	12	0.0	12	0.0	0.00	23
76.1	-64	75.5	20	359.1	509	76.6	2.25	32
157.4	-36	155.9	26	384.1	1016	157.7	4.58	54
237.1	93	236.3	28	403.7	1509	236.3	6.79	181
276.9	314	317.2	31	418.2	2012	274.2	7.84	401
316.8	635	397.6	33	428.9	2535	311.3	8.86	721
357.2	936	478.0	35	436.5	3026	349.0	9.89	1021
347.3	662	558.8	37	448.9	4013	341.5	9.68	747
336.9	504	640.7	38	460.3	5068	332.5	9.44	589
327.8	390	722.1	39	469.5	6091	324.4	9.22	475
337.2	632	802.3	41	478.4	7099	331.7	9.42	717
347.2	776	882.7	41	487.5	8064	340.4	9.65	861
357.6	910	964.0	43	496.3	9039	349.6	9.90	995
396.9	1226	1043.9	43	504.2	10071	386.2	10.89	1310
437.0	1473	1124.2	45			424.2	11.90	1556
476.8	1693	1205.7	45			462.0	12.90	1775
516.5	1879	1285.2	48			500.1	13.90	1960
556.3	2032	1365.8	48			538.6	14.90	2112
596.1	2143	1447.6	49			577.5	15.90	2222
635.9	2228					616.5	16.90	2307
675.8	2292					655.8	17.89	2370
715.4	2356					694.9	18.87	2434
755.1	2402					734.2	19.85	2479
795.0	2445					773.6	20.83	2521
				Reload Cycle				
				434.4	2504			
				460.3	5051			



Interpreted Test Results

Deformation Modulus, E_P	39,886	kPa	5,785	psi
Reload Modulus, E_R	114,345	kPa	16,584	psi
At-Rest Horizontal Pressure, P_{OH}	75	kPa	11	psi
Yield Pressure, P_F	1,310	kPa	190	psi
Limit Pressure, P_L	3,013	kPa	437	psi
Net Limit Pressure, P_L*	2,938	kPa	426	psi
E_P / P_L	13.2			
P_L / P_F	2.3			

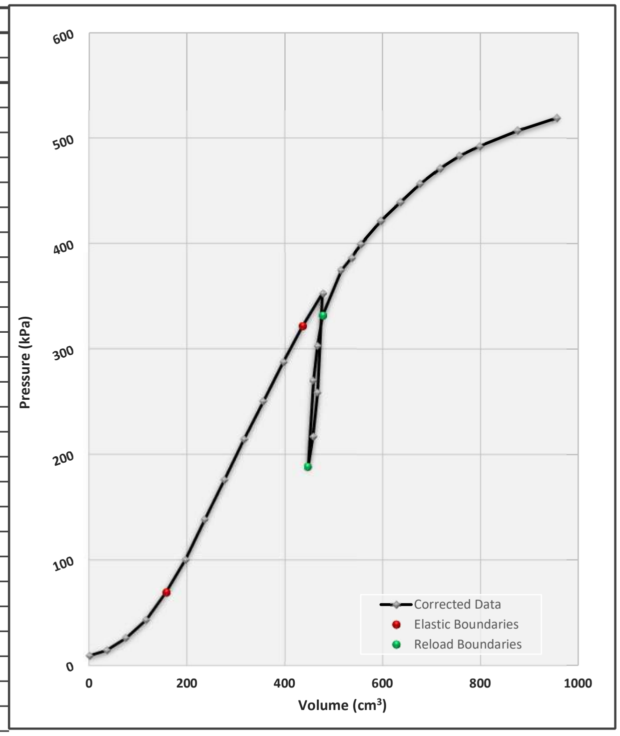
Remarks

PRESSUREMETER TEST

Project:	90235317	Boring ID:	B-3	Test Depth (ft):	17.0
City, State:	San Antonio, TX	Mapes In-Situ No:	P2024011	Client:	Terracon

Test date:	2/29/24	Probe body SN:	2000020	Pressure Calibration ID:	P 20a (1)
Pressuremeter SN:	001A17002	Probe diameter (mm):	70	Volume Calibration ID:	V 20a (1)
Pressuremeter model:	TEXAM[e]	Calibration tube I.D. (mm):	76.2	Calibration coefficient, c (cm^3/kPa):	0.008717
Test zone drilling method:	Shelby Tube Sampling	Calibration tube O.D. (mm):	101.6	Reload calibration coefficient, c_r (cm^3/kPa):	0.010091
Poisson's Ratio of soil/rock:	0.33	Tubing length (m):	50	Contact volume, V_a (cm^3):	76
Method for estimating P_L:	1/V vs. P			Initial volume of probe, V_0 (cm^3):	1682

Raw Test Data		Pressure Calibration		Volume Calibration		Corrected Data		
Volume	Pressure	Volume	Pressure	Volume	Pressure	Volume	$\Delta R/R_\circ$	Pressure
cm^3	kPa	cm^3	kPa	cm^3	kPa	cm^3	%	kPa
0.0	-19	0.0	12	0.0	12	0.0	0.00	9
36.0	-10	75.5	20	359.1	509	36.1	1.07	14
76.4	6	155.9	26	384.1	1016	76.4	2.24	26
117.2	26	236.3	28	403.7	1509	117.0	3.42	43
157.4	55	317.2	31	418.2	2012	**156.9**	**4.56**	**69**
197.6	87	397.6	33	428.9	2535	196.8	5.69	100
237.5	126	478.0	35	436.5	3026	236.4	6.80	138
277.9	165	558.8	37	448.9	4013	276.5	7.91	176
318.1	205	640.7	38	**460.3**	**5068**	316.3	9.00	214
358.1	242	722.1	39	469.5	6091	356.0	10.08	250
398.7	280	802.3	41	478.4	7099	396.2	11.16	287
438.4	315	882.7	41	487.5	8064	**435.6**	**12.20**	**321**
479.0	347	964.0	43	496.3	9039	475.9	13.27	352
468.8	253	1043.9	43	**504.2**	**10071**	466.6	13.02	259
458.6	211	1124.2	45			456.7	12.76	217
448.6	182	1205.7	45			447.0	12.51	188
460.1	264	1285.2	48			457.8	12.79	270
468.5	297	1365.8	48			465.9	13.00	303
479.3	326	1447.6	49			476.4	13.28	331
518.3	370					515.1	14.29	374
538.4	382					535.1	14.81	386
559.0	396					555.5	15.34	399
600.0	419					596.3	16.38	422
640.3	437					636.5	17.41	439
681.2	455					677.2	18.43	457
721.8	470					717.7	19.44	471
761.6	483					757.4	20.43	483
801.6	493					797.3	21.41	492
881.4	508					877.0	23.34	507
961.3	522					956.8	25.25	519
				Reload Cycle				
				434.4	2504			
				460.3	5051			



Interpreted Test Results

Deformation Modulus, E_P	4,759	kPa	690	psi
Reload Modulus, E_R	28,182	kPa	4,088	psi
At-Rest Horizontal Pressure, P_{OH}	65	kPa	9	psi
Yield Pressure, P_F	321	kPa	47	psi
Limit Pressure, P_L	584	kPa	85	psi
Net Limit Pressure, P_L*	519	kPa	75	psi
E_P / P_L	8.1			
P_L / P_F	1.8			

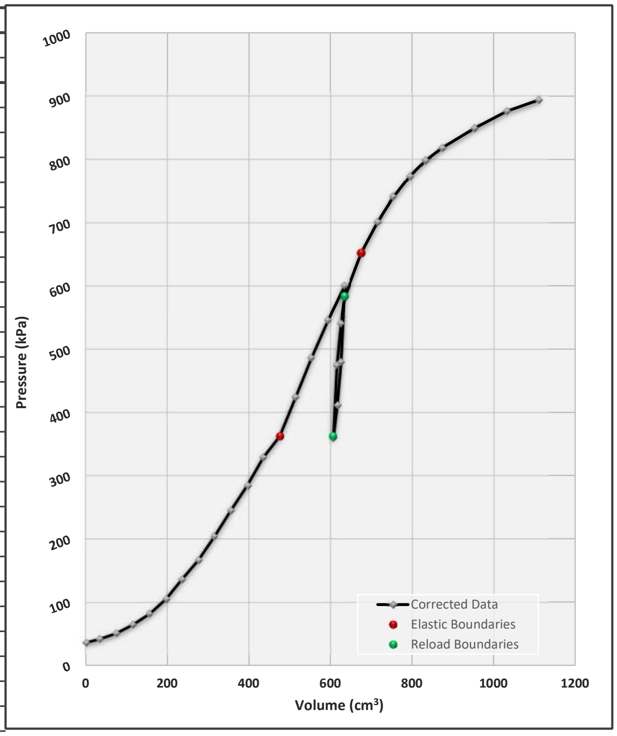
Remarks

PRESSUREMETER TEST

Project:	90235317	Boring ID:	B-3	Test Depth (ft):	27.0
City, State:	San Antonio, TX	Mapes In-Situ No:	P2024011	Client:	Terracon

Test date:	2/29/24	Probe body SN:	2000020	Pressure Calibration ID:	P 20a (1)
Pressuremeter SN:	001A17002	Probe diameter (mm):	70	Volume Calibration ID:	V 20a (1)
Pressuremeter model:	TEXAM[e]	Calibration tube I.D. (mm):	76.2	Calibration coefficient, c (cm^3/kPa):	0.008717
Test zone drilling method:	Shelby Tube Sampling	Calibration tube O.D. (mm):	101.6	Reload calibration coefficient, c_r (cm^3/kPa):	0.010091
Poisson's Ratio of soil/rock:	0.33	Tubing length (m):	50	Contact volume, V_a (cm^3):	160
Method for estimating P_L:	1/V vs. P			Initial volume of probe, V_0 (cm^3):	1682

Raw Test Data		Pressure Calibration		Volume Calibration		Corrected Data		
Volume	Pressure	Volume	Pressure	Volume	Pressure	Volume	$\Delta R/R_o$	Pressure
cm^3	kPa	cm^3	kPa	cm^3	kPa	cm^3	%	kPa
0.0	-22	0.0	12	0.0	12	0.0	0.00	36
35.7	-12	75.5	20	359.1	509	35.8	1.06	42
75.8	1	155.9	26	384.1	1016	75.8	2.23	51
116.8	18	236.3	28	403.7	1509	116.7	3.41	65
158.3	38	317.2	31	418.2	2012	157.9	4.59	82
197.6	62	397.6	33	428.9	2535	197.1	5.70	105
237.2	94	478.0	35	436.5	3026	236.4	6.80	136
277.4	126	558.8	37	448.9	4013	276.3	7.90	167
317.5	165	640.7	38	**460.3**	**5068**	316.1	8.99	204
357.8	207	722.1	39	469.5	6091	356.0	10.07	245
398.1	247	802.3	41	478.4	7099	396.0	11.15	284
438.0	293	882.7	41	487.5	8064	435.5	12.20	329
478.4	327	964.0	43	496.3	9039	**475.6**	**13.26**	**362**
518.1	390	1043.9	43	**504.2**	**10071**	514.7	14.28	424
557.9	453	1124.2	45			553.9	15.30	486
599.0	513	1205.7	45			594.5	16.34	546
640.1	568	1285.2	48			635.2	17.37	600
630.1	447	1365.8	48			626.2	17.14	479
620.0	379	1447.6	49			616.7	16.90	411
609.7	327					606.8	16.65	360
619.8	442					615.9	16.88	474
630.2	508					625.8	17.13	540
640.1	551					635.3	17.38	583
680.5	620					**675.1**	**18.38**	**652**
721.8	670					715.9	19.40	701
761.1	711					754.9	20.37	741
801.0	744					794.5	21.34	773
840.6	769					833.9	22.30	798
880.4	789					873.5	23.26	818
960.5	822					953.3	25.17	849
1040.2	849					1032.8	27.04	876
1120.1	869					1112.5	28.90	894
				Reload Cycle				
				434.4	**2504**			
				460.3	**5051**			



Interpreted Test Results

Deformation Modulus, E_P	8,712	kPa	1,264	psi
Reload Modulus, E_R	48,796	kPa	7,077	psi
At-Rest Horizontal Pressure, P_{OH}	85	kPa	12	psi
Yield Pressure, P_F	652	kPa	95	psi
Limit Pressure, P_L	1,015	kPa	147	psi
Net Limit Pressure, P_L*	930	kPa	135	psi
E_P / P_L	8.6			
P_L / P_F	1.6			

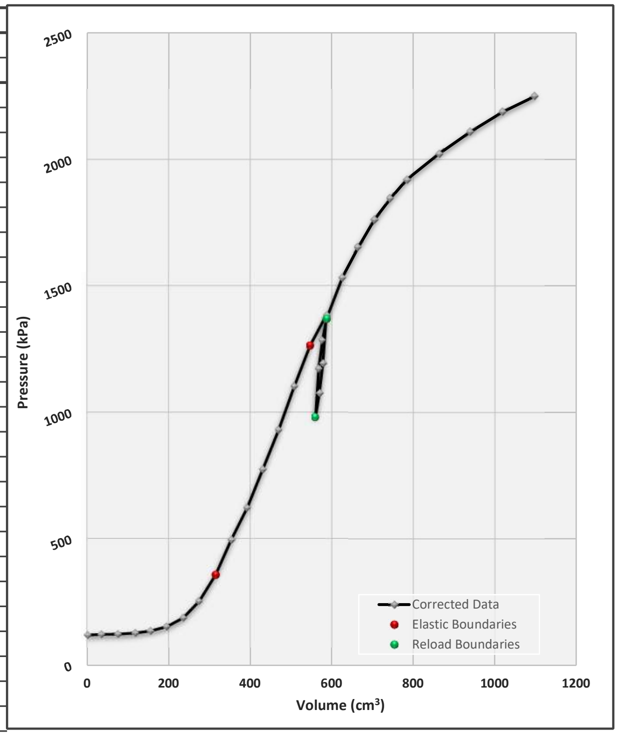
Remarks

PRESSUREMETER TEST

Project:	90235317	Boring ID:	B-3	Test Depth (ft):	47.0
City, State:	San Antonio, TX	Mapes In-Situ No:	P2024011	Client:	Terracon

Test date:	2/29/24	Probe body SN:	2000020	Pressure Calibration ID:	P 20a (1)
Pressuremeter SN:	001A17002	Probe diameter (mm):	70	Volume Calibration ID:	V 20a (1)
Pressuremeter model:	TEXAM^{e}	Calibration tube I.D. (mm):	76.2	Calibration coefficient, c (cm^3/kPa):	0.008717
Test zone drilling method:	Shelby Tube Sampling	Calibration tube O.D. (mm):	101.6	Reload calibration coefficient, c_r (cm^3/kPa):	0.010091
Poisson's Ratio of soil/rock:	0.33	Tubing length (m):	50	Contact volume, V_a (cm^3):	236
Method for estimating P_L:	1/V vs. P			Initial volume of probe, V_0 (cm^3):	1682

Raw Test Data		Pressure Calibration		Volume Calibration		Corrected Data		
Volume	Pressure	Volume	Pressure	Volume	Pressure	Volume	$\Delta R/R_o$	Pressure
cm^3	kPa	cm^3	kPa	cm^3	kPa	cm^3	%	kPa
0.0	1	0.0	12	0.0	12	0.0	0.00	119
35.0	8	75.5	20	359.1	509	34.9	1.03	122
75.6	13	155.9	26	384.1	1016	75.5	2.22	123
118.8	21	236.3	28	403.7	1509	118.6	3.47	128
156.5	32	317.2	31	418.2	2012	156.2	4.54	136
196.8	50	397.6	33	428.9	2535	196.4	5.68	153
237.2	86	478.0	35	436.5	3026	236.4	6.80	188
276.8	153	558.8	37	448.9	4013	275.5	7.88	253
317.4	258	640.7	38	460.3	5068	315.1	8.96	357
357.1	398	722.1	39	469.5	6091	353.6	10.01	496
397.7	526	802.3	41	478.4	7099	393.1	11.07	623
437.2	678	882.7	41	487.5	8064	431.2	12.09	774
477.2	835	964.0	43	496.3	9039	470.0	13.11	930
517.2	1008	1043.9	43	504.2	10071	508.4	14.12	1102
557.2	1170	1124.2	45			547.0	15.12	1263
597.8	1290	1205.7	45			586.5	16.13	1382
588.0	1101	1285.2	48			578.4	15.93	1194
577.8	983	1365.8	48			569.3	15.69	1076
567.9	887	1447.6	49			560.1	15.46	980
577.6	1079					568.2	15.66	1172
587.8	1192					577.4	15.90	1285
597.9	1276					586.8	16.14	1368
638.6	1439					626.1	17.14	1531
679.1	1561					665.5	18.14	1652
719.9	1671					705.3	19.14	1762
759.8	1756					744.5	20.11	1846
799.6	1831					783.6	21.07	1920
879.4	1932					862.6	23.00	2021
959.1	2023					941.4	24.89	2110
1038.8	2101					1020.5	26.76	2188
1118.3	2165					1099.5	28.59	2250
				Reload Cycle				
				434.4	2504			
				460.3	5051			



Interpreted Test Results

Deformation Modulus, E_P	21,963	kPa	3,186	psi
Reload Modulus, E_R	89,641	kPa	13,001	psi
At-Rest Horizontal Pressure, P_{OH}	150	kPa	22	psi
Yield Pressure, P_F	1,263	kPa	183	psi
Limit Pressure, P_L	2,660	kPa	386	psi
Net Limit Pressure, P_L*	2,510	kPa	364	psi
E_P / P_L	8.3			
P_L / P_F	2.1			

Remarks

Supporting Information

Contents:

Note: All attachments are one page unless noted above.

Terracon
6000 Northwest Pkwy Ste 100
San Antonio, TX

General Notes

Sampling	Water Level	Field Tests
Shelby Tube Split Spoon	Water Initially Encountered Water Level After a Specified Period of Time Water Level After a Specified Period of Time Cave In Encountered Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	N — Standard Penetration Test Resistance (Blows/Ft.) (HP) — Hand Penetrometer (T) — Torvane (DCP) — Dynamic Cone Penetrometer UC — Unconfined Compressive Strength (PID) — Photo-Ionization Detector (OVA) — Organic Vapor Analyzer

Descriptive Soil Classification

Soil classification as noted on the soil boring logs is based Unified Soil Classification System. Where sufficient laboratory data exist to classify the soils consistent with ASTM D2487 "Classification of Soils for Engineering Purposes" this procedure is used. ASTM D2488 "Description and Identification of Soils (Visual-Manual Procedure)" is also used to classify the soils, particularly where insufficient laboratory data exist to classify the soils in accordance with ASTM D2487. In addition to USCS classification, coarse grained soils are classified on the basis of their in-place relative density, and fine-grained soils are classified on the basis of their consistency. See "Strength Terms" table below for details. The ASTM standards noted above are for reference to methodology in general. In some cases, variations to methods are applied as a result of local practice or professional judgment.

Location And Elevation Notes

Exploration point locations as shown on the Exploration Plan and as noted on the soil boring logs in the form of Latitude and Longitude are approximate. See Exploration and Testing Procedures in the report for the methods used to locate the exploration points for this project. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

Strength Terms

Relative Density of Coarse-Grained Soils (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		Consistency of Fine-Grained Soils (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Relative Density	Standard Penetration or N-Value (Blows/Ft.)	Consistency	Unconfined Compressive Strength Qu (tsf)	Standard Penetration or N-Value (Blows/Ft.)
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

Relevance of Exploration and Laboratory Test Results

Exploration/field results and/or laboratory test data contained within this document are intended for application to the project as described in this document. Use of such exploration/field results and/or laboratory test data should not be used independently of this document.

Unified Soil Classification System

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]				Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	Cu≥4 and 1≤Cc≤3 [E]	GW	Well-graded gravel [F]
			Cu<4 and/or [Cc<1 or Cc>3.0] [E]	GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH	GM	Silty gravel [F, G, H]
			Fines classify as CL or CH	GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	Cu≥6 and 1≤Cc≤3 [E]	SW	Well-graded sand [I]
			Cu<6 and/or [Cc<1 or Cc>3.0] [E]	SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH	SM	Silty sand [G, H, I]
			Fines classify as CL or CH	SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	PI > 7 and plots above "A" line [J]	CL	Lean clay [K, L, M]
			PI < 4 or plots below "A" line [J]	ML	Silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OL	Organic clay [K, L, M, N]
					Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line	CH	Fat clay [K, L, M]
			PI plots below "A" line	MH	Elastic silt [K, L, M]
		Organic:	$\frac{LL\ oven\ dried}{LL\ not\ dried} < 0.75$	OH	Organic clay [K, L, M, P]
					Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.
[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.
[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.
[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$
[F] If soil contains ≥ 15% sand, add "with sand" to group name.
[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H] If fines are organic, add "with organic fines" to group name.
[I] If soil contains ≥ 15% gravel, add "with gravel" to group name.
[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L] If soil contains ≥ 30% plus No. 200 predominantly sand, add "sandy" to group name.
[M] If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N] PI ≥ 4 and plots on or above "A" line.
[O] PI < 4 or plots below "A" line.
[P] PI plots on or above "A" line.
[Q] PI plots below "A" line.