GEOTECHNICAL ENGINEERING REPORT

Miro Meadows – Lift Station and Pavement Design

New Sulphur Springs Road near Blandford Road
San Antonio ETJ, Bexar County, Texas

Prepared for:
Lennar
San Antonio, Texas

Prepared by:
TTL, Inc.
San Antonio, Texas

Project No. 00210903116.02
December 12, 2022

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RE: **Revised Geotechnical Engineering Report**
Miro Meadows Subdivision – Lift Station and Pavement Design
New Sulphur Springs Road near Blandford Road
San Antonio ETJ, Bexar County, Texas
TTL Project No. 00210903116.02

Dear Mr. Mott:

TTL, Inc. (*TTL*) is pleased to submit this revised Geotechnical Engineering Report for the above-referenced project. If you have any questions regarding our report, or if additional services are needed, please do not hesitate to contact us.

The enclosed report contains a brief description of the site conditions and our understanding of the project. The geotechnical recommendations contained within this report are based on our understanding of the proposed development, the results of our field exploration and laboratory tests and our experience with similar projects.

We appreciate the opportunity to be of professional service during this phase of the project, and look forward to working with you in the future.

Respectfully submitted,
TTL, Inc.

JUNE M. POTTER
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12/12/2022

June Potter. P.E.
Senior Project Engineer
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1.0 PROJECT INFORMATION

1.1 Project Description

Item	Description
Project Location	The project site is located on New Sulphur Springs Road near Blandford Road in the San Antonio ETJ, Bexar County, Texas. The Site Location Map is provided in Appendix A.
Proposed Development	Based on the plan developed by KFW Engineers and Surveying, we understand that the Miro Meadows subdivision will consist of approximately 3,500 linear feet of streets, compliable with City of San Antonio (COSA) regulations. The project also includes the design and construction of a lift station on the southwestern portion of the project site. We anticipate the lift stations will be a cast-in-place concrete wet well about 30 feet deep. We anticipate the grade supported structures will be supported by slab-on-grade foundations. A mat foundation will support the wet well foundation.
Maximum Loads	Loads were not provided to TTL as a part of this project.

If the above information is not correct, please contact us so that we can make the necessary modifications to this document and our recommendations, if needed.

1.2 Authorization

This Project was authorized on October 8, 2021, by Mr. Richard Mott, P.E. with Lennar by acceptance of our Agreement for Services, TTL Proposal No. P00210901531.00, dated October 7, 2021.

2.0 EXPLORATION FINDINGS

2.1 Site Conditions

Item	Description
Existing Conditions	Based on imagery obtained from Google Maps, the Site is undeveloped and moderately to heavily vegetated with trees, crops, and unmaintained grasses.
Site Topography	Based on the preliminary site layout provided to us by KFW Engineers + Surveying. The site generally slopes downward from approximate El. 560 feet on its eastern side to approximate El. 550 feet on its western side.

2.2 Site Geology

We reviewed the Geologic Atlas of Texas to determine the geologic setting of the project site and surrounding area. Our review indicated the Project Site is located over the Midway Group (Emi) and the Wilcox Group (Ewi) of Tertiary geologic age. The Midway Group generally consists of clay and sand. This formation ranges from approximately 100 to 400 feet in thickness. The Wilcox Group generally consists of clay, sand, mudstone, and sandstone with various amounts of lignite.

The Wilcox Group ranges from approximately 440 to 1200 feet in thickness. These formations are known to contain expansive clay soils within the project area.

2.3 Subsurface Stratigraphy

Subsurface conditions within the limits of the project were evaluated by drilling Seven (7) exploratory borings at the approximate locations shown on the Boring Location Plan in Appendix A. Out of the seven (7) borings, two (2) borings (boring B-1 and B-2) were drilled at the lift station location, and the remaining five (5) borings were drilled along the pavement alignments provided by KFW Engineers + Surveying.

Samples obtained during our field exploration were transported to our laboratory, where they were reviewed by geotechnical engineering personnel. Representative samples were selected and tested to determine pertinent engineering properties and characteristics for use in our evaluation of the project site. Based on the information developed during our field exploration and laboratory testing, we have determined the stratigraphy of the site is generally as shown on the logs of boring as shown in Appendix A.

The boring logs presented in Appendix A represent our interpretation of the subsurface conditions at each individual boring location. Our interpretation is based on tests and observations performed during drilling operations, visual examination of the soil samples by a geotechnical engineer, and laboratory tests conducted on the retrieved soil samples. The USCS classifications shown on the boring logs represent classifications based on either visual examination, laboratory testing, or both. The lines designating the interfaces between various strata on the boring logs represent the approximate strata boundary. The transition between strata may be more gradual than shown, especially where indicated by a broken line. All data should only be considered accurate at the exact boring location as subsurface variations may occur between and away from boring locations.

SANDY LEAN CLAY (CL) materials, along with various percentages of gravel, were encountered in borings B-1 and B-3 between the ground surface to 28 feet. These materials are granular in nature and can slough during excavation operations. While these materials were not encountered in the remaining borings, it is likely that similar materials will be encountered in other areas of this project site. Therefore, the contractor should check soil conditions before the commencement of excavation activities and be prepared to control sloughing.

2.4 Subsurface Water Conditions

Subsurface water was encountered in one (1) of the seven (7) borings drilled for this project. **At boring B-1, subsurface water was encountered at a depth of approximately 23 feet.** Drilling operations were suspended for a period of 15 minutes, after which the subsurface water level was observed at a depth of approximately 24 feet. Subsurface water was not encountered during or upon completion of drilling operations in the remaining borings. Upon completion of subsurface water observations, the boreholes were backfilled with soil cuttings produced during our drilling operations. More detailed information on subsurface water conditions and caving within our exploratory borings is included in the boring logs in Appendix A.

The presence or absence of subsurface water during a geotechnical exploration may not be indicative of long-term subsurface water conditions at the project site. Subsurface water may exist as ‘true” or permanent water sources or as temporary ‘perched’ sources. Furthermore, these water sources may or may not be contiguous across a given project site. Subsurface water may exist year-round or may appear intermittently. The presence or absence of subsurface water and the elevations at which it may be encountered can be influenced by a wide range of factors that often include seasonal and climatic changes, vegetation, surface runoff, and the proximity of the site to nearby water bodies.

As was mentioned above, SANDY LEAN CLAY (CL) materials were encountered in borings B-1 and B-3. These materials are granular in nature and will transmit water easily. Accordingly, it should be expected that subsurface water will be encountered during excavation operations in other areas of the project site. While subsurface water was not encountered in borings B-2 thru B-7, it is possible that those materials and subsurface water may be encountered during excavation in the vicinity of these borings as well.

It should be noted that subsurface water levels will fluctuate with the seasons and variations in precipitation. Furthermore, based on Google images and the FEMA Flood Map, a 100-year floodplain exists on the western side of the project and numerous Calaveras Lake tributaries traverse the project site. Furthermore, an unidentified dam is located near the northern edge of the project boundary. As a result, there is an increased likelihood that subsurface water may be encountered during construction. Therefore, the contractor should check for subsurface water before the commencement of excavation activities and be prepared to control subsurface water infiltration.

2.5 Subsurface Water Control

At the time of our drilling operations (March 2022), subsurface was encountered at a depth of approximately 23 feet below the existing ground surface at boring B-1. As a result of the presence of subsurface water at the lift station location, it will likely be necessary to control subsurface water infiltration during construction of the wet well. The control of subsurface water will likely be coupled with the control of sloughing soils as gravel materials were encountered between depths of 20 and 28 feet in boring B-1 and could be encountered at depths above and below that zone within the vicinity of the wet well location. Control of subsurface water infiltration and sloughing soils are means and methods issues that are beyond the scope of this project. However, TTL would be happy to assist in the design and construction of a dewatering system should that become necessary for the successful completion of this project.

Due to the presence of subsurface water and the potential for sloughing soils in the vicinity of the wet well, we recommend the contractor check soil and subsurface water conditions

prior to the commencement of excavation activities. The information developed by such a reconnaissance will likely be very useful with regard to the development of appropriate construction plans and sequences that will effectively control subsurface water infiltration and soil sloughing during excavation and construction of the wet well.

2.6 Buoyancy

We understand that the lift station will be founded at a depth of approximately 30 to 35 feet below the existing grade. Permanent structures submerged below the water table will be subject to uplift forces caused by buoyancy. The mat foundation bearing below the water table should be designed to resist buoyancy due to groundwater. The deadweight of the structure can be used in resisting hydrostatic uplift. **The project structural engineer should consider buoyancy while designing mat foundations for the wet well foundation.**

3.0 GEOTECHNICAL CONSIDERATIONS

The expansive potential of a given soil profile may be characterized using the Potential Vertical Rise (PVR) methodology as described in the Texas Department of Transportation (TxDOT) Method TEX-124-E. This methodology is used to estimate how much a given point located on the ground surface may move vertically due to volumetric changes in the soil resulting from fluctuations in soil moisture content. Based on our laboratory test results, the estimated PVR of the lift station location ranged from about one (1) and one and one-half (1½) inches in its present condition. **These estimated PVR values indicate the soils at the lift station location possess a low to moderate expansive potential. Care should be taken to carefully evaluate the potential effects of these movements on the structural design of the shallow foundations planned for the lift station.**

4.0 EARTHWORK RECOMMENDATIONS

4.1 General Site Preparation

Finished Floor Elevations (FFE’s) for any planned equipment shelters or other foundations at the ground surface were not provided to us at the time of this report. The recommendations below are based on existing conditions with the assumption that the top of slab will be at least six (6) inches above existing grade. The mass grading of the subdivision was not provided at the time of our investigations; once any ground surface FFEs are finalized TTL can provide revised recommendations. Wet well structures can be supported by mat foundations. For mat foundations, the building pad area should be stripped as recommended in Section 4.3.1 of this report. Upon completion of stripping operations, the site may be either excavated or filled as necessary to achieve the desired site elevation. After site stripping and excavation, or prior to placement of any fill materials, the exposed subgrade should be prepared as recommended in Section 4.3.2 and proof-rolled as recommended in Section 4.3.3 of this report. For the grade supported slab foundations, the building pad area should be prepared as recommended in

Section 4.2 of this report. Grade adjustments **outside** the building pad areas can be made using select or general fill materials in accordance with Section 4.4 of this report.

4.2 Building Pad Preparation

Grade supported slab foundations may be used to support equipment shelters near the lift station. We understand the equipment supporting the lift station may be sensitive to movements and, as a result, we understand the "design" PVR should be limited to about one (1) inch. Based on our laboratory test results, the estimated PVR of the site ranged from about one (1) and one and one-half (1½) inches in its present condition. Therefore, special steps will be required to reduce the estimated PVR of the building pads for the grade supported foundations to acceptable levels.

Please note that our laboratory testing indicated that the soils within the upper six (6) to eight (8) feet of borings B-1 and B-2 generally indicated the Plasticity Indices of those soils were at or slightly above the maximum allowable values applicable to select fil material as discussed in Section 4.4 of this report. Similarly, the soils encountered in borings B-3 through B-7 possess similar Plasticity Index values. However, below a depth of six (6) to eight (8) feet in borings B-1 and B-2, a layer of FAT CLAY (CH) was present and resulting in the estimated PVR values greater than one (1) inch as discussed above. **The recommendations for preparation of the building pad(s) presented below are designed to reuse the surficial soils and condition them such that the estimated PVR of the building pad is reduced to about one (1) inch.**

4.2.1 Building Pad PVR of one (1) inch

- Strip the building pad area as recommended in Section 4.3.1 of this report. Continue to excavated to a depth of approximately three (3) feet below existing grade.
- The soil excavated from the building pad may be reused within the building pad as select fill provided it meets the requirements for select fill as specified in Section 4.4 of this report. **Any soils that do not meet those requirements shall not be reused in the building pad. If additional soils beyond those excavated from the building pad are required to achieve the desired FFE, those soils should consist of select fill as described in Section 4.4 of this report.**
- Upon completion of excavation to the appropriate depth, the exposed subgrade should be prepared as recommended in Section 4.3.2 of this report.
- After the completion of subgrade preparation and prior to the placement of select fill (either soils excavated from the building pad that meet the requirements for select fill or imported select fill), the exposed subgrade should be proof-rolled as recommended in Section 4.3.3 of this report.
- Place select fill (either soils excavated from the building pad that meet the requirements for select fill or imported select fill) as necessary to achieve desired foundation elevation for the applicable structure. Select fill should be placed as

recommended in Section 4.4 of this report. **The placement of select fill as recommended herein should result in a minimum select fill body thickness of at least three (3) feet below the underside of the grade supported slab foundation.**

4.2.2 Limits of Building Pad Preparation

The building pad for the foundation should extend at least two (2) feet beyond the limits of the valve vault, generator pad or other grade supported structure foundation.

4.2.3 Clay Cap

For building pads constructed using granular fill materials (materials with less than 50 percent passing the No. 200 sieve), a clay cap should be installed around the building perimeter. The intent of this cap is to minimize the potential for infiltration of moisture into the fill materials comprising the building pad. This clay cap should begin against the building foundation and extend at least three (3) feet beyond the edge of the limits of the building pad. Installation of this cap should occur after the structure and any associated appurtenances have been constructed. The composition of the materials comprising this clay cap and its placement should be as recommended in Section 4.4 of this report. This clay cap may be replaced with concrete flatwork or pavements extending to the edge of the valve vault, generator pad or other grade supported structure.

4.3 Site Preparation Details

4.3.1 Stripping Operations

Stripping operations should remove all topsoil, vegetation, roots, loose or soft soils, and any other deleterious materials. The stripped materials should be removed from the site and properly disposed of.

4.3.2 Subgrade Preparation

Exposed subgrade soils should be scarified to a depth of at least eight (8) inches. The scarified soil should be moisture conditioned to between optimum and +4 percentage points of the optimum moisture content just prior to compaction. The soils should then be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 698.

4.3.3 Proof-Rolling Operations

Proof-rolling should be accomplished with construction equipment weighing at least 20 tons. Soils that are observed to rut or deflect excessively under the moving load should be removed and replaced with select fill materials compacted as recommended in Section 4.4 of this report.

4.4 Compacted Fill Materials

Compacted fill materials may consist of select or general fill depending upon its intended use. General fill materials may consist of onsite soils, select fill materials, or clean imported fill soils possessing good compaction characteristics that will provide suitable, uniform support for pavements and other non-habitable facilities which are not extremely sensitive to movements. General fill material may also be used in open areas where such facilities will not be constructed. Select fill material, on the other hand, is typically selected based on specific engineering characteristics and performance criteria. These selection characteristics and criteria typically depend on the requirements of the pavements, structures, or other facilities they are intended to support.

General and select fill materials should be clean and free of any vegetation, roots, organic materials, trash or garbage, construction debris, or other deleterious materials. These materials should contain stones no larger than two and one-half (2½) inches in maximum dimension. The following table provides more specific requirements for general and select fill materials.

MATERIAL TYPE	CHARACTERISTICS	COMPACTION PROCEDURES	COMPACTION CONTROL [1, 2]
GENERAL FILL	Shall consist of CH, CL, SC, GC, SM, GM, SW, or GW as defined by ASTM D 2487. Plasticity Index: Not more than 35. Maximum allowable organic content: 3 percent by weight. **This fill material type shall not be used in areas where select fill materials are specified. It is not the intent of this material to control differential soil movements and it shall not be used in areas where control of soil movements is required.**	Maximum loose lift thickness: 8 inches. Compaction requirement: Compaction should be at least 95 percent of the standard Proctor (ASTM D 698) maximum dry density for fill bodies less than 5 feet in thickness. Compaction should be at least 95 percent of the modified Proctor (ASTM D 1557) maximum dry density for fill bodies 5 feet or greater in thickness. Moisture content at time of compaction: within plus to minus 3 percent of the material's optimum moisture content.	General Fill Areas: One field test for every 10,000 square feet per lift, with a minimum of two tests per lift. Utility Trenches (in areas where Select Fill is not required): One field density test per every 100 linear feet, per lift.
SELECT FILL (FLEXIBLE BASE)	2014 Texas Department of Transportation (TxDOT) Item 247, Types A or B, Grades 1 or 2.	Maximum loose lift thickness: 8 inches. Compaction requirement: Compaction should be at least 95 percent of the standard Proctor (ASTM D 698) maximum dry density for fill bodies less than 5 feet in thickness. Compaction should be at least 95 percent of the modified Proctor (ASTM D 1557) maximum dry density for fill bodies 5 feet or greater in thickness. Moisture content at time of compaction: within plus to minus 3 percent of the material's optimum moisture content.	Building Area: One field density test every 5,000 square feet per lift, with a minimum of two tests per lift. Pavement Areas and Slopes: One field density test every 10,000 square feet per lift, with a minimum of two tests per lift. Utility Trenches (where select fill is specified): One field density test per structure or one test per every 100 linear feet, per lift.
SELECT FILL	Shall consist of CL, SC, GC, or GM as defined by ASTM D 2487. Maximum allowable organic content: 3 percent by weight. Liquid Limit: Not more than 40. Plasticity Index: Between 8 and 20.		

MATERIAL TYPE	CHARACTERISTICS	COMPACTION PROCEDURES	COMPACTION CONTROL [1, 2]
CLAY CAPS AND UTILITY PLUGS	Shall consist of a cohesive clay soil with a maximum Plasticity Index of 25 percent. At least 70 percent (by weight) should pass the No. 200 sieve and no more than 15 percent (by weight) should be retained on the No. 4 sieve.	Maximum loose lift thickness: 8 inches. Compaction requirement: Compaction should be between 93 and 98 percent of the standard Proctor (ASTM D 698) maximum dry density. This material shall be moisture conditioned as necessary to achieve the target compaction.	Clay Cap Areas: One field test for every 10,000 square feet per lift, with a minimum of two tests per lift. Utility Plugs: One field density test per lift at each plug.

[1] For preliminary planning only. Our technician/engineer should determine the actual test frequency.

[2] In addition, the fill must be stable under the influence of compaction equipment. Heavy construction traffic should not be allowed to travel on compacted fill areas, except on designated haul roads, to reduce the potential for damaging a previously compacted fill subgrade.

4.5 Excavation Conditions

4.5.1 Temporary Slopes and OSHA Soil Types

The Occupational Safety and Health Administration (OSHA) Safety and Health Standards (29 CFR Part 1926) require that excavations be constructed in accordance with the current OSHA guidelines. The contractor is **solely** responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. To that end, the contractor's 'responsible person' as defined in 29 CFR Part 1926 should evaluate the required excavations and the soils exposed by those excavations and determine appropriate means as part of the contractor's safety procedures.

OSHA requires that excavations in excess of five (5) feet be shored or appropriately sloped. Currently available and practiced methods for achieving excavation stability include sloping, benching, shoring, and the use of trench shields. In excavations that are less than 20 feet deep, OSHA addresses maximum allowable slopes on OSHA Table B-1 as reproduced below.

Soil or Rock Type	Maximum Allowable Slopes (H:V)[1] for Excavations Less Than 20 Feet Deep[2]	
Stable Rock	Vertical	90°
Type A[3]	¾:1	53°
Type B	1:1	45°
Type C	1½:1	34°

1. Numbers shown in parentheses next to maximum allowable slopes are angles expressed in degrees from the horizontal. Angles have been rounded off.
2. Slopes or benching for excavations that exceed 20 feet shall be designed by a licensed professional engineer.
3. For Type A soils, a short-term maximum allowable slope of ½:1 (63°) is allowed in excavations that are 12 feet deep or less. For excavations deeper than 12 feet, the short-term allowable slope shown above applies. OSHA defines short-term as a period of 24 hours or less.

Based on the results of our field and laboratory testing, it is our opinion that the FAT CLAY (CH) LEAN CLAY (CL) and SANDY LEAN CLAY (CL) soils encountered in our soil borings may be considered as Type B soils. If those clay soils become saturated or submerged, they should be downgraded to Type C soils. We have provided this information solely as a service to our client. The actual OSHA regulations should be consulted prior to any excavations that would be subject to OSHA regulations. TTL does not assume responsibility for any construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

We have provided this information solely as a service to our client. The actual OSHA regulations should be consulted prior to any excavations that would be subject to OSHA regulations. TTL does not assume responsibility for any construction site safety or the contractor's or other parties' compliance with local, state, and federal safety or other regulations.

4.5.2 Anticipated Excavation Conditions

As is shown on the boring logs presented in Appendix A, clay and sand materials were encountered at this site. The soils encountered at the borings can generally be excavated by conventional earthmoving equipment. However, the hard clay encountered at deeper depths (below approximately 28 feet at boring B-1) will likely be difficult to excavate with conventional earthmoving equipment. **Heavy-duty rock excavation equipment and techniques may be required to excavate the hard/cemented clay encountered at this site.** However, the final selection of appropriate excavation equipment is a "means and methods" issue and is the responsibility of the project contractor.

4.5.3 Drainage During Construction

Water should not be allowed to collect in foundation excavations, on foundation surfaces, or on prepared subgrades within the construction area during construction. Excavated areas should be sloped toward designated drainage points to facilitate removal of any collected rainwater, subsurface water, or surface runoff. Positive surface drainage at the site should be provided to reduce infiltration of surface water into subgrades and fill bodies during construction and promote prompt removal of water from the project site.

4.6 Long-Term Drainage Considerations

Long-term drainage conditions can have a significant impact on the performance of structures, utilities, and other ancillary facilities on a project site. We recommend that site drainage be developed such that long-term ponding does not occur except in areas specifically designed for such purposes. When establishing final grades, the design team should be remined that in expansive clay environments, it is common for ground surface movements to occur that could potentially cause reversal of site drainage patterns and unwanted ponding of surface water. We recommend the following be considered:

- Elevation of the ground surface adjacent to foundations should be at least six (6) inches below the Finished Foundation Elevation unless measures are taken to ensure long-term positive drainage away from the structure.
- The slope of the ground surface away from any structures (if not covered with pavement) should be a minimum of five (5) percent for a distance of at least 10 feet unless measures are taken to ensure long-term positive drainage away from the structures.
- Gutter downspouts should extend at least five (5) feet past the edge of the foundations.
- Sufficient slope of the ground surface should be maintained around pavements and other ancillary facilities to ensure long-term positive drainage.

5.0 INFRASTRUCTURE RECOMMENDATIONS

5.1 Utilities

Various utilities will be installed at these Sites. The utilities will likely include sanitary sewer lines, electrical lines, and possibly telecommunication lines. Installation of these utilities should conform to the applicable specifications of the appropriate utility entities. At a minimum, all utilities should meet the following installation guidelines.

- The bottoms of the utility trench excavations should be clean of loose soils and debris prior to placement of the utility pipe or cable.
- Utility trenches may be backfilled with general or select fill in accordance with Section 4.4 of this report.
- As an alternate, utility trenches may be backfilled with flowable fill materials that terminate at a depth sufficient to allow for the construction of structure foundations or any pavements constructed as a part of this project. Flowable fill should have a minimum 28-day compressive strength of 100 psi. The flowable fill should not have an unreasonably high compressive strength to ensure that it remains excavatable should the need arise in the future. Flowable fill is defined as materials complying with Item 401 of the 2014 TxDOT Standard Specifications.
- Where granular bedding is used for pipe bedding, consideration should be given to the placement of filter fabric around the bedding materials within the trench to reduce the potential for piping fines through the bedding material. Piping of fines within utility trenches often results in pronounced subsidence of the ground surface over time that could affect foundations and pavements constructed over the utility trenches.

5.2 Final Pavement Design Considerations

Based on the COSA design guidelines, the following design parameters were used for design of the pavement sections:

	Local Type A without Bus Traffic	Local Type A with Bus Traffic	Local Type B	Collector Street
Reliability, %	70	70	90	90
Initial Serviceability Index, p_0	4.2	4.2	4.2	4.2
Terminal Serviceability Index, p_t	2.0	2.0	2.0	2.5
Standard Deviation, S_0	0.45	0.45	0.45	0.45
Design Life, years	20	20	20	20
18-kip ESALs	100,000	1,000,000	2,000,000	2,000,000
Minimum Structural Number	2.02	2.58	2.92	2.92
Maximum Structural Number	3.18	4.20	5.08	5.08

Soil bulk sample was collected to determine the California Bearing Ratio (CBR) value to be used for our pavement design recommendations. The location at which the CBR bulk sample was taken is indicated on the Boring Location Plan in Appendix A. We performed CBR tests at three compaction levels (i.e., 90%, 95% and 100%). Based on laboratory test results, a CBR value of about 3.0 percent was obtained for the existing untreated subgrade compacted to at least 95 percent of the maximum dry density determined in accordance with ASTM D 698. TTL recommends that a CBR value of 3.0 percent be used to represent the pavement subgrade conditions at this site. There are a number of published correlations relating CBR to the Resilient Modulus (MR). In accordance with the COSA and Bexar County design guidelines, we used a Resilient Modulus (MR) = 1,500 times the CBR in psi, to convert CBR to MR.

The COSA pavement guidelines require lime treatment of clay subgrades with a PI greater than 20. CBR and the boring logs samples obtained from this subdivision indicate a PI value over 20. Therefore, the subgrade at this site shall be treated with hydrated lime in accordance with TxDOT Item 260. We anticipate that approximately six (6) percent of hydrated lime will be required (about 35 pounds per square yard). It is anticipated that even after the mass grading is completed that the soils will require lime treatment. Furthermore, we understand the lime-treated subgrade will not be treated to meet the COSA requirement for lime stabilization.

However, it should be noted that, upon completion of the grading operations at the site, the index properties of the subgrade soils should be checked to determine whether or not lime treatment is required. This is because mass grading operations may have removed lower PI material to expose higher PI material, or higher PI fill may have been placed over lower PI materials.

5.2.1 Pavement Sections

Following are the recommended pavement sections for Local Type A without Bus Traffic, Local Type A with Bus Traffic, Local Type B, Collector, and Secondary Arterial.

Flexible Pavement System	
Component	Local Type A without Bus Traffic
	Pavement Material Thickness
Hot Mixed Asphaltic Concrete – Type D	2 inches
Prime Coat	Yes
Granular Base Course (Type A, Grade 1 or 2)	12 inches
Lime Treated Subgrade[1]	6 inches
Required Structural Number	2.49
Provided Structural Number[1]	2.56
Required 18-kip ESALs	100,000
Estimated Provided 18-kip ESALs	121,000

[1] Structural Number for Lime Treated Subgrade was not used in the Pavement Section Calculations.

Flexible Pavement System	
Component	Local Type A with Bus Traffic
	Pavement Material Thickness
Hot Mixed Asphaltic Concrete – Type D	2 inches
Prime Coat	Yes
Granular Base Course (Type A, Grade 1 or 2)	19 inches
Lime Treated Subgrade[1]	6 inches
Required Structural Number	3.53
Provided Structural Number[1]	3.54
Required 18-kip ESALs	1,000,000
Estimated Provided 18-kip ESALs	1,027,000

[1] Structural Number for Lime Treated Subgrade was not used in the Pavement Section Calculations.

Flexible Pavement System		
Component	Local Type B	
	Pavement Material Thickness	
Hot Mixed Asphaltic Concrete – Type D	2-½ inches	2 inches
Hot Mixed Asphaltic Concrete – Type C	----	
Dense-Grade Hot-Mix Asphaltic Concrete Base Course (Type B, Item- 341)	4 inches	4 inches
Prime Coat	Yes	Yes
Granular Base Course (Type A, Grade 1 or 2)	13 inches	15 inches
Lime Treated Subgrade[1]	6 inches	6 inches
Required Structural Number	4.37	4.37
Provided Structural Number[1]	4.44	4.50
Required 18-kip ESALs	2,000,000	2,000,000
Estimated Provided 18-kip ESALs	2,274,700	2,504,500

[1] Structural Number for Lime Treated Subgrade was not used in the Pavement Section Calculations.

Flexible Pavement System	
Component	Collector
	Pavement Material Thickness
Hot Mixed Asphaltic Concrete – Type D	2 inches
Hot Mixed Asphaltic Concrete – Type C	----
Dense-Grade Hot-Mix Asphaltic Concrete Base Course (Type B, Item- 341)	4 inches
Prime Coat	Yes
Granular Base Course (Type A, Grade 1 or 2)	17 inches
Lime Treated Subgrade[1]	6 inches
Required Structural Number	4.67
Provided Structural Number[1]	4.78
Required 18-kip ESALs	2,000,000
Estimated Provided 18-kip ESALs	2,371,000

[1] Structural Number for Lime Treated Subgrade was not used in the Pavement Section Calculations.

5.2.2 General Guidelines for Pavements

All pavement design and construction shall conform to the latest edition of Bexar County/ City of San Antonio Design and Construction guidelines. Pavement design methods are intended to provide structural sections with adequate thickness over a particular subgrade such that wheel loads are reduced to a level the subgrade can support. **The support characteristics of the subgrade for pavement design do not account for shrink/swell movements of an expansive clayey subgrade. Thus, the pavement may be adequate from a structural standpoint, yet still experience cracking and deformation due to shrink/swell-related movement of the subgrade. It is, therefore, important to minimize moisture changes in the subgrade to reduce shrink/swell movements.**

On most projects, rough site grading is accomplished relatively early in the construction phase. However, as construction proceeds, excavations are made into these areas; dry weather may desiccate some areas; rainfall and surface water may saturate some areas; heavy traffic from concrete and other delivery vehicles disturbs the subgrade, and many surface irregularities are filled in with loose soils to improve trafficability temporarily. As a result, the pavement subgrade should be carefully evaluated as the time for pavement construction approaches. This is particularly important in and around utility trench cuts.

Thorough proof-rolling of pavement areas using appropriate construction equipment weighing at least 20 tons should be performed no more than 24 hours prior to surface paving. Any problematic areas should be reworked and compacted at that time.

Long-term pavement performance will be dependent upon several factors, including maintaining subgrade moisture levels and providing for preventive maintenance. The following recommendations should be considered at a minimum:

- Maintain and promote proper surface drainage away from pavement edges;
- Consider appropriate edge drainage systems;
- Install drainage in areas anticipated for frequent wetting (e.g., landscape beds, discharge area, collection areas, etc.).
- Place joint sealant and seal cracks immediately;
- Seal all landscaped areas in, or adjacent to pavements, to minimize or prevent moisture migration to subgrade soils;
- Placing compacted, low permeability backfill against the exterior side of curb and gutter; and,
- Extending the base of the curb and gutter system through the pavement base material and at least 6 inches into lime-treated subgrade soils.

Preventive maintenance should be planned and provided for through an on-going pavement management program. These activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. This consists of both localized maintenance (e.g., crack and joint sealing and patching) and global maintenance (e.g., surface sealing). Preventive maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements. Prior to implementing any maintenance, additional engineering observation is recommended to determine the type and extent of preventive maintenance.

5.2.3 Drainage Adjacent to Pavements

The performance of the pavement system will not only be dependent upon the quality of construction but also upon the stability of the moisture content of the soils and the base underlying the pavement surface. Proper drainage along or adjacent to the pavement edge or curbs is very important and should be provided, so infiltration of surface water from unpaved areas surrounding the pavement is minimized. **TTL observed Calaveras Lake tributary areas within the subdivision crossing pavements at several locations.** The Project Civil Engineer should design final grades so that there is positive drainage away from the pavement/curb edge. Also, surface slopes for asphaltic concrete pavement areas should be no flatter than two (2) percent to reduce the potential for ponding of water on the asphaltic concrete surface. The importance of proper runoff and drainage cannot be overemphasized and should be thoroughly considered by the Project Civil Engineer. Post-construction accumulation or ponding of surface runoff near structures must be avoided.

Since water penetration usually results in degradation of the pavement section with time as vehicular traffic traverses the affected area, we recommend that the curbs extend vertically through the aggregate base course into the lime stabilized layer of more than six (6) inches below pavement base layers.

5.2.4 Pavement Section Materials

All pavement materials shall conform to the latest edition of City of San Antonio/ Bexar County design and construction guidelines. 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 TTL and any appropriate members of the Project Team. The submittals should provide test information necessary to verify full compliance with the recommended or specified material properties.

Hot Mix Asphaltic Concrete Surface - The paving mixture and construction methods shall conform to Item 340, “Hot Mix Asphaltic Concrete, Type D” of the Standard Specifications by TxDOT. The mix should be compacted between 91 and 95 percent of the maximum theoretical density as measured by TEX-227-F. The asphalt cement content by percent of total mixture weight should fall within a tolerance of ±0.3 percent asphalt cement from the specific mix. In addition, the mix should be designed so 75 to 85 percent of the voids in the mineral aggregate (VMA) are filled with asphalt cement. The asphalt cement grades should conform to the table shown below.

Asphalt Cement Grades			
Street Classifications	**Minimum PG Asphalt Cement Grade**		
	Surface Courses	**Binder and Level up courses**	**Base Courses**
Arterials	PG 76-22	PG 70-22	PG 64-22
Collector and Local Type B Streets	PG 70-22		
Local Type A Street with Bus Traffic		PG 64-22	
Local Type A Street without Bus Traffic	PG 64-22		

Aggregates known to be prone to stripping should not be used in the hot mix. If such aggregates are used measures should be taken to mitigate this concern. The mix should have at least 70 percent strength retention when tested in accordance with TEX-531-C.

Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method TEX-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from Project pavement specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required pavement specimens at their expense and in a manner and at locations selected by the Engineer.

Asphaltic Base Course – The asphaltic concrete base materials should be plant mixed, hot laid Type A or B Base meeting all the requirements of TxDOT Item 341 of the 2014 Standard Specifications. The asphaltic concrete base course of the pavement section should be constructed in accordance with this Item.

Prime Coat - The prime coat should consist of sealing the base with an oil such as MC-30 or AE-P asphalt cement. The prime coat should be applied at a rate not to exceed 0.35 gallons per square yard with materials which meet TxDOT Item 300. The prime coat will help to minimize the penetration of rainfall and other moisture that penetrates the base.

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 or 2; and should have no more than 15 percent of the material passing the No. 200 sieve. The base should be compacted to at least 95 percent of the maximum dry density determined in accordance with test method TEX-113-E at moisture contents ranging between -2 and +3 percentage points of the optimum moisture content.

Lime Treatment - Lime treatment shall be performed only on the dark brown clay subgrade. The subgrade shall be treated with hydrated lime in accordance with TxDOT Item 260. We anticipate that approximately six (6) percent hydrated lime will be required (approximately 35 pounds per square yard). The optimum hydrated lime content should result in a soil-lime mixture with a pH of at least 12.4 when tested in accordance with ASTM C 977, Appendix XI.

The hydrated lime should initially be blended with a mixing device such as a pulvermixer. After sufficient moisture conditioning, the treated soil mixture shall be compacted to at least 95 percent of the maximum dry density as determined in accordance with the Standard effort (ASTM D 698) at moisture contents from optimum to +4 percentage points of the optimum moisture content. If the in-place gradation requirements can be achieved during initial mixing, the remixing after the curing period can be eliminated.

Details regarding subgrade preparation are presented in Pavement Earthwork Section below.

5.2.5 Pavement Earthwork

The intended performance of roadway pavement is contingent upon following the earthwork recommendations and guidelines outlined in this section. Earthwork activities on the Project should be observed and evaluated by *TTL* personnel. The evaluation of earthwork should include observation and testing of all fill and backfill soils placed at the Site, and subgrade preparation beneath the streets.

The clay soils across the site have a moderate potential to undergo expansion and contraction with fluctuations in their moisture content. Expansion and contraction of the clay subgrade can lead to cracking and undulating/corrugation in the pavement and curbs. Remedial methods to address this issue include: removing the expansive soils and replacing them with non-expansive cohesive soil; chemical injection of the expansive soils; a combination of moisture conditioning, lime or cement treatment, and installation of a vertical moisture barrier; other subgrade preparation methods are also available.

This report provides recommendations to help mitigate some effects of soil shrinkage and expansion. However, even if these procedures are followed, movement and cracking in the pavements from the expensive soils should be anticipated. The severity of cracking and other damage will probably increase if any modification of the site results in excessive wetting or drying of the expansive soils. Eliminating the risk of movement and distress may not be feasible, but it may be possible to further reduce the risk of movement if significantly more expensive measures are used during construction. We would be pleased to discuss other construction alternatives with you upon request*. If additional earthwork preparation methods will be used or evaluated, please contact us.*

The pavement sections represent minimum recommended thicknesses to satisfy city/county requirements, and as such, periodic maintenance should be anticipated. Therefore, an on-going pavement management program should be planned and provided for preventive maintenance. Maintenance activities are intended to slow the pavement deterioration rate and preserve the pavement investment.

The following earthwork recommendations must be performed prior to pavement construction.

- Strip vegetation, loose topsoil, vegetation, and any otherwise unsuitable materials from the pavement area. The pavement area is defined as the area that extends at least three

(3) feet (horizontal) beyond the perimeter of the proposed pavement and any adjacent flatwork (sidewalks).

- Perform cut and fill to accommodate the design pavement subgrade elevation (also referenced as the bottom of the base course). Onsite soils can be used for grade adjustments in fill areas. Refer to Section 4.0 of this report for requirements for the placement of onsite soils and select fill materials.
- After achieving the required excavation depth, and before placing any fill, the exposed excavation subgrade should be proof-rolled with at least a 20-ton roller, or equivalent equipment, to evidence any weak yielding zones. A technical representative of our firm should be present to observe the proof-rolling operations. If any weak yielding zones are present, they should be over-excavated, both vertically and horizontally, until competent soils are exposed. The excavated soil can be used to restore the excavation subgrade, provided that the soils are relatively free and clean of deleterious material or materials exceeding 3 inches in maximum dimension. The excavated soil or imported fill soil shall be placed in maximum of 6-inch compacted lifts. Each lift of soil shall be moisture conditioned and compacted as described in Section 4.0.
- After proof-rolling and replacing any weak yielding zones, the clay subgrade should be lime treated in accordance with TxDOT Item 260. The lime shall be in slurry form. It is anticipated that approximately six (6) percent hydrated lime will be required (approximately 22 pounds per square yard). The soil-lime mixture shall be placed between optimum and +4 percentage points of the optimum moisture content and shall be compacted to at least 95 percent of the maximum dry density determined in accordance with the Standard compaction effort (ASTM D 698).
- For pavement subgrades, the earth work described here should result in approximately six (6) inches of lime-treated soil below the design pavement subgrade elevation.
- For the pavements located in the flood hazard area or natural drainage path areas, one of the following additional measures should be constructed beneath the soil subgrade level:
    - Prepare the subgrade with 12 inches of moisture conditioned soils beneath 6 to 8 inches of lime treated soils, or
    - Prepare the subgrade with at least 12 inches of lime-treated soils.

6.0 STRUCTURAL RECOMMENDATIONS

6.1 Seismic Design Parameters

Presented below are the seismic design criteria for the Project site and immediate area.

Description	Value
2015 International Building Code Site Classification (IBC)[1]	D[2]
Site Latitude	29.358561°
Site Longitude	-98.340265°

Description	Value
Maximum Considered Earthquake 0.2 second Design Spectral Response Acceleration (S_{DS})	0.055 g
Maximum Considered Earthquake 1.0 second Design Spectral Response Acceleration (S_{D1})	0.037 g
[1] As per the requirements of Section 1613.3.2 in the 2015 IBC, the site class definition was determined using SPT N-values in conjunction with Table 20.3-1 of the ASCE 7 manual. The Spectral Acceleration values were determined using publicly available information provided on the United States Geological Survey (USGS) website.	
[2] Note: Chapter 20 of ASCE 7 requires a site soil profile determination extending to a depth of 100 feet for seismic site classification. The current scope does not include the required 100-foot soil profile determination. The borings extended to a maximum depth of ***40 feet***, and this seismic site class definition considers that similar soils continue below the maximum depth of the subsurface exploration. Additional exploration to deeper depths would be required to confirm the conditions below the current depth of exploration.	

6.2 Corrosion Considerations

According to the 2015 IBC, concrete that is exposed to sulfate-containing solutions should be designed in accordance with ACI 318. To evaluate if sulfate exposure was a concern at this site, laboratory testing was conducted on soil samples recovered during the field exploration to assess the corrosivity risk of the soil at the boring locations. The soil samples were submitted to an analytical lab to determine the sulfate content. The results of the laboratory tests are presented in the following table.

Boring No.	Sample Depth (ft.)	Sulfate (ppm)
B-1	6½ to 8	281
B-5	2½ to 4	74.8

The sulfate test results indicate that the sulfate exposure level is Class S0, which infers that sulfate exposure to concrete is not an issue. Therefore, Type I/II cement may be used.

6.3 Shallow Foundations

6.3.1 Mat Foundation

We understand the wet well planned for this project will bear at a depth of approximately 30 feet below grade as it existed at the time of our drilling operations. The wet well foundation bearing at or below this depth may be designed using maximum contact pressures of 7,500 psf based on total loads or 5,000 psf based on dead plus long-term live loads. The bearing capacities discussed above include design factors of safety of two (2) and three (3), respectively.

The design of a mat foundation typically involves the use of a Winkler foundation type model. The Winkler foundation model assumes the soil can be modeled using an infinite number of elastic springs. The spring constant in this model is known as the modulus of subgrade reaction. For wet well foundation designed as mats bearing at the anticipated bearing depths discussed in this report may be designed using a modulus of subgrade reaction value of 150 pounds per cubic inch.

If necessary, mat foundations may be used to resist uplift loads induced by the planned structures or buoyancy. Uplift resistance for a mat foundation generally consists of the dead load from the structure and the weight of the mat. If more resistance is necessary, the mat can be geometrically shaped so the weight of any soil overlying the mat foundation can also be considered. In such a case, a unit weight of 100 pcf may be used for backfill soils placed over the footing in accordance with the requirements for select fill in Section 4.4 of this report.

6.3.2 Slab and Grade Beam Foundations

A slab foundation with interior and exterior grade beams can be used to support the valve vault, generator pad, or other grade supported structures provided the pads for these foundations are prepared as discussed in Section 4.2 of this report. Grade beams may bear on properly placed select fill materials and can be designed using the parameters shown in the following tables.

CRSI (Concrete Reinforcing Steel Institute) Method[1, 4, 5, 6]

Net Allowable Bearing Pressures[3]	
Total Load Conditions (psf):	2,500
Dead Load Plus Gravity Live Load Conditions (psf):	1,700
Maximum Allowable Deflection Ratio of Beam Footing:	1/360
Subgrade Modulus, *k* (pci):	50

WRI (Wire Reinforcement Institute) Method[1, 4, 5, 6]

Effective Plasticity Index (EPI)[2]:	28
Climatic Rating (C_w):	16
Soil Climate Support Index (1-C):	0.13
Site Slope (%):	0.0
Soil Unconfined Compressive Strength (q_u) in tsf:	1.0
Net Allowable Bearing Pressures[3]:	
Total Load Conditions in psf:	2,500
Dead Load Plus Gravity Live Load Conditions in psf:	1,700
Maximum Allowable Deflection Ratio of Foundation Beam:	1/360

PTI (Post-Tensioning Institute) Method; 3rd Edition[1, 2, 3, 4, 7]

Vertical Moisture Barrier Depth (ft.):	2½
Edge Moisture Variation Distance (e_m):	
Center Lift (ft.):	9.0
Edge Lift (ft.):	4.5
Maximum Unrestrained Differential Soil Movement or Swell (y_m):	
Center Lift (in.):	1.1
Edge Lift (in.):	1.6
Coefficient of Slab-Subgrade Friction (μ):	0.75
Net Allowable Bearing Pressures[5]:	
Total Load Conditions (psf):	2,500
Dead Load Plus Gravity Live Load Conditions (psf):	1,700
Maximum Allowable Deflection Ratio of Foundation Beam:	1/360

Slab Foundation Design Notes. The following notes apply to the slab and grade beam design methods presented above.

1	Slab Design parameters are based on preparing the subgrade and placing any fill materials (if necessary) as recommended in Sections 4.1, 4.2, 4.3 and 4.4 of this report. Mat Foundation parameters are based on preparing the subgrade and place any fill materials (if necessary) as recommended in Sections 4.1, 4.3 and 4.4 of this report.
2	This is essentially an empirical design method and the recommended design parameters are based on our understanding of the proposed Project, our interpretation of the information and data collected as a part of this study, our area experience, and the criteria published in the CRSI and WRI design manuals.
3	The width of foundation beams should not be less than 12 inches. The minimum bearing depth below the adjacent ground surface (also referred to as “final grade”) should not be less than ***30 inches*** for perimeter and interior foundation beams. These foundation dimension recommendations are for the proper development of bearing capacity for the foundations and to reduce the potential for water to migrate beneath the foundation. These recommendations are not based on structural considerations of the applicable design method. Actual foundation depths and widths may need to be greater than the minimum recommended herein for structural considerations, which should be properly evaluated and designed by the Structural or Foundation Engineer.
4	Includes a factor of safety (FS) of at least 2 for total load conditions and at least 3 for dead load plus long-term live load conditions.
5	Based on the weighted average method for a depth of 15 feet.

6.3.3 Shallow Foundation Construction Considerations

Excavations for shallow foundations and grade beams shall be neat excavated with a smooth-mouthed bucket. If a toothed bucket is used, excavation with this bucket should be stopped six (6) inches above the final foundation bearing surface and the excavation completed with a smooth-mouthed bucket or by hand labor. Debris in the bottom of the excavations should be removed prior to steel placement. If neat excavation is not possible, the foundation should be overexcavated and formed. All loose materials should be removed from the overexcavated areas and filled with lean concrete or flowable fill as described in ACI 229R.

Reinforcing steel should be placed and the foundation constructed as quickly as possible to avoid exposure of the foundation bottoms to wetting and draying. The excavations should be sloped sufficiently to create internal sumps for runoff collection and removal of water. If surface runoff or subsurface water seepage in excess of one (1) inch accumulates at the bottom of the excavation, it should be collected and removed so that ponding water does not adversely affect the quality of the bearing surfaces. Special care should be taken to protect exposed bearing surfaces from disturbance or drying out prior to the placement of concrete.

While the wet well planned for this project will be founded on a shallow foundation, the bearing depth of that foundation will be relatively deep. As a result, additional considerations will be necessary for the successful installation of that foundation. In particular, consideration should be given to the means and methods necessary for the control of subsurface water infiltration and soil sloughing. The foundation contractor should carefully evaluate soil and subsurface water conditions at the wet well location prior to commencement of excavation activities and plan accordingly.

6.3.4 Settlement of Grade Supported Foundations

Total settlement of grade supported foundations designed and constructed as recommended in this report is expected to be about one (1) inch or less. The settlement of the foundations is expected to be elastic in nature with most of the observed settlement occurring during construction. Differential settlement approaching ½ to ¾ of the total foundation settlement should be expected to occur between load bearing foundation elements. The settlement response of grade supported foundations is impacted more by the quality of construction than by soil-structure interaction. The improper installation of foundation elements can result in differential settlements that are greater than we have estimated.

6.4 Drilled Pier Foundations with Structurally Suspended Slab

We understand that consideration will be given to structurally suspending the 16’ x 43’ pad (adjacent to the lift station) planned for this project using drilled pier foundations and either void boxes or forms, or crawlspaces. The use of structurally suspended slab systems will significantly reduce the risk of slab distress. Recommendations for the design of drilled pier foundations and structurally suspended slabs are presented below.

6.4.1 Drilled Pier Foundation Recommendations

Drilled piers can be used to support the structure column loads supporting 16’ x 43’ suspended pad/slab. If void boxes or carton forms are used, a minimum void space of four (4) inches should be provided below the lowest foundation elements. **For this project, only straight-sided piers should be considered, and pier depths should not extend below the elevations shown in the following table without contacting our office prior to excavation.**

Compressive axial loads on drilled pier foundations are resisted by both side friction along the drilled pier shaft and by end bearing at the base of the drilled pier. Uplift loads are resisted solely by side friction along the drilled pier shaft, the weight of the drilled pier, and the structure dead load acting on the drilled pier.

Drilled pier foundations shall be designed based on the following table. Please note that the allowable side friction values shown below may be used to evaluate the drilled piers for both compressive and tensile axial load conditions. See Section 6.4.2 of this report for further details on uplift on drilled piers due to the action of expansive soils.

COMPRESSION LOADING			
Approximate Elevation (feet)	**Allowable Side Friction (f_s), psf (Safety Factor = 2)**	**Allowable End Bearing, psf**	
		Total Load (Safety Factor = 2)	**Dead Load + Gravity Live Load (Safety Factor = 3)**
0 - 5	----	----	----
5 – 11	400	----	----
11 – 22	400	5,000	3,500
22 – 32	600	9,000	6,000
32 - 40	800	16,500	11,000

6.4.2 Drilled Pier Design Considerations in Expansive Soils

Drill pier foundations installed in expansive soils can be subjected to tensile forces arising from the swelling action of those soils imposing uplift forces on the pier structure itself through skin friction. Within the area of the project, these tensile forces will generally be generated within the upper 15 feet of the soil profile, assuming that proper drainage is utilized under and around the structures and under structurally suspended structures. In order to counteract these forces, drilled piers must be installed to a sufficient depth such that the skin friction developed by the soil below that depth of 15 feet (as measured from final grade) is greater than or equal to the uplift forces generated by the expansive soils. **For this project, we estimate uplift forces (in kips) acting on the piers as a result of the action of expansive clays to be approximately 20 times the diameter of the pier (in feet) being considered.**

6.4.3 Interactions with Adjacent Piers

Drilled pier foundations supporting the pad structure that are in close proximity to other drilled pier foundations could potentially experience foundation interactions. In order to avoid this interaction, we recommend that drilled piers be spaced a distance of at least three (3) pier diameters, center to center, from the nearest adjacent piers. This spacing should be based on the largest diameter of the piers in question. If drilled piers are to be placed within three (3) pier diameters of each other, the end bearing parameters provided will generally not require a reduction. However, the skin friction action on both piers will be affected. We recommend the values shown in the following table be used to reduce the available skin friction on both piers for design purposes.

Center to Center Pier Spacing/Pier Diameter	Allowable Skin Friction Reduction Factor
3.0 or greater	1.00
2.5	0.89
2.0	0.78
1.5	0.67
1.0	0.53
Note: If necessary, intermediate values may be interpolated linearly.	

6.4.4 Drilled Pier Construction Considerations

The design and installation of drilled pier foundations should proceed as discussed in this report and in the U.S. Department of Transportation Federal Highway Administration (FHWA) publication entitled Drilled Shafts: Construction Procedures and Design Methods by Dan A. Brown, John P. Turner, Raymond J. Castelli, and Erik J. Loehr (Report No. FHWA NHI-18-024 dated September 2018). The design and construction of drilled shafts should also be accomplished as discussed in the applicable sections of the latest editions of the American Concrete Institute (ACI) 318 Building Code Requirements for Structural Concrete and ACI 336.3R Report on Design and Construction of Drilled Piers.

As was discussed previously, subsurface water was not encountered either during or upon completion of our soil borings. However, granular soils were encountered at boring B-1 and may be present in other areas of the project site. It is possible that subsurface water could be encountered, especially after periods of heavy rainfall or prolonged wet weather. Therefore, the foundation contractor should be prepared to address subsurface water issues during construction.

For minor amounts of subsurface water infiltration into the drilled shaft with negligible sloughing of the drilled shaft sidewalls, the foundation contractor may be able to install the planned foundations without the need for casing or slurry. This is predicated on the ability of the contractor to remove any accumulated water from the bottom of the drilled shaft prior to placement of concrete or the use of tremies or pumps to place concrete below any accumulated water. For significant water infiltration and/or sloughing of the drilled shaft sidewalls, the installation of casing or use of drilling slurry should be considered. We recommend that the guidelines suggested in the FHWA publication Drilled Shafts: Construction Procedures and Design Methods be implemented when installing the drilled pier foundations for this project, particularly with regard to the use of casing or drilling slurry. If casing is used during construction, it must be removed and cannot remain in-place. Casing that remains in-place will degrade the side friction capacity of the drilled pier foundations.

We recommend that foundation construction be monitored by TTL to document compliance of drilled pier installation activities with appropriate construction specifications and with the recommendations provided in this report. For this project, particular attention will be required to monitor:

- Vertical alignment of drilled shafts;
- Proper evaluation of drilled shaft depths, bottoms, and side walls with regard to design requirements;
- Proper installation (and removal) of casing, if used;
- Proper mixing and maintenance of slurry, and application, if used;
- Proper installation of reinforcing steel and/or structure placement; and
- Proper concrete properties and placement.

6.5 Lateral Earth Pressures

6.5.1 Lift Station

The proposed wet well walls should be designed to resist lateral earth pressure from soils and hydrostatic pressure. Groundwater flow in the native alluvial soils will be influenced primarily by the creek situated just east of the project Site, surface water infiltration**. Due to the presence of granular material, ground water at 23 feet and the location of the Calaveras Lake and its associated tributaries, we recommend that the wet well walls be designed to resist full hydrostatic conditions.** Lateral earth pressures on these walls should be designed using at-rest earth pressures.

The table below may be used to design soil retaining wet well walls for 'At-rest' earth pressures. The equivalent fluid densities shown in the table do not include any safety factors.

Backfill Material Type	At-Rest Equivalent Fluid Density, pcf With Hydrostatic Pressure
Fat Clay Soils	125
Lean Clay Soils	125
Clayey Sand	92
Gravel	88

The at-rest equivalent fluid densities assume that the wet well walls are restrained from lateral or rotational movement. These pressures do not include the influence of any surcharge pressures (such as those due to equipment loading) which should be added. In addition to the lateral earth pressure, we recommend evaluating the wet well walls within the upper 10 feet for the effects of swell pressure resulting from the presence of expansive clays at the project site. The swell pressure within the upper 10 feet may be taken as 500 psf. This swell pressure is unfactored.

6.6 Light Pole Foundations

Lateral loads on the light pole can be resisted by passive soil pressures on the sides of the foundations. The allowable passive resistance provided by the soils surrounding foundations can be estimated using the following recommended allowable passive soils pressure per foot of embedment:

Soil Type	Allowable Passive Soil Pressure (psf/ft)
Topsoil/ Organic Soils	0
Sandy Lean Clay (CL); light brown	150
Fat Clay (CH); dark brown	200

Surficial topsoil or organic soil deposits and the top 12 inches of soil adjacent to the light pole foundations should not be relied upon to provide passive earth pressure against the foundations. Deepened circular footings should extend at least 5 feet below finished grades and have a diameter of at least 18 inches. Footings founded in at least 5 feet below finished grade should be proportioned using an allowable bearing pressure of 2,000 psf. Groundwater encountered at a depth of about 25 feet below the existing grade at boring B-1 will not likely interfere during the excavation of light pole structures.

7.0 LIMITATIONS

This geotechnical engineering report has been prepared for the exclusive use of our Client for specific application to this Project. This geotechnical engineering report has been prepared in accordance with generally accepted geotechnical engineering practices using that level of care and skill ordinarily exercised by licensed members of the engineering profession currently practicing under similar conditions in the same locale. No warranties, express or implied, are intended or made.

TTL understands that this geotechnical engineering report will be used by the Client and various individuals and firms' designers and contractors involved with the design and construction of the Project. TTL should be invited to attend Project meetings (in person or teleconferencing) or be contacted in writing to address applicable issues relating to the geotechnical engineering aspects of the Project. TTL should also be retained to review the final construction plans and specifications to evaluate if the information and recommendations in this geotechnical engineering report has have been properly interpreted and implemented in the design and specifications. This report has not been prepared as, and should not be used as, a design or specification document to be directly implemented by the contractor. The contractor and applicable subcontractors should familiarize themselves with this report prior to the start of their construction activities and contact TTL for any interpretation or clarification of the report.

This geotechnical engineering report is based upon the information provided to us by the Client and various other individuals and entities associated with the Project, along with the field exploration, laboratory testing, and engineering analyses and evaluations performed by TTL as described in this report. The Client and readers of this geotechnical engineering report should realize that subsurface variations and anomalies may exist across the site which may not be revealed by our field exploration. Furthermore, the Client and readers should realize that site conditions can change due to the modifying effects of seasonal and climatic conditions and conditions at times after our exploration may be different than reported herein.

The nature and extent of such site or subsurface variations may not become evident until construction commences or is in progress. If site and subsurface anomalies or variations exist or develop, TTL should be contacted immediately so that the situation can be properly evaluated and, if necessary, addressed with provide applicable recommendations.

Unless stated otherwise in this report or in the contract documents between TTL and Client, our scope of services for this Project did not include, either specifically or by implication, any environmental or biological assessment of the site or buildings, or any identification or prevention of pollutants, hazardous materials or conditions at the site or within buildings. If the Client is concerned about the potential for such contamination or pollution, TTL should be contacted to provide a scope of additional services to address the environmental concerns. In addition, TTL is not responsible for permitting, site safety, excavation support, and dewatering requirements.

Should the nature, design, or location of the Project, as outlined in this geotechnical engineering report be modified, the geotechnical engineering recommendations and guidelines provided in this document will not be considered valid unless TTL is authorized to review the changes and either verifies or modifies the applicable Project changes in writing.

Additional information about the use and limitations of a geotechnical report is provided within the Geoprofessional Business Association document included at the end of this report.

Important Information about This

Geotechnical-Engineering Report

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

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

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

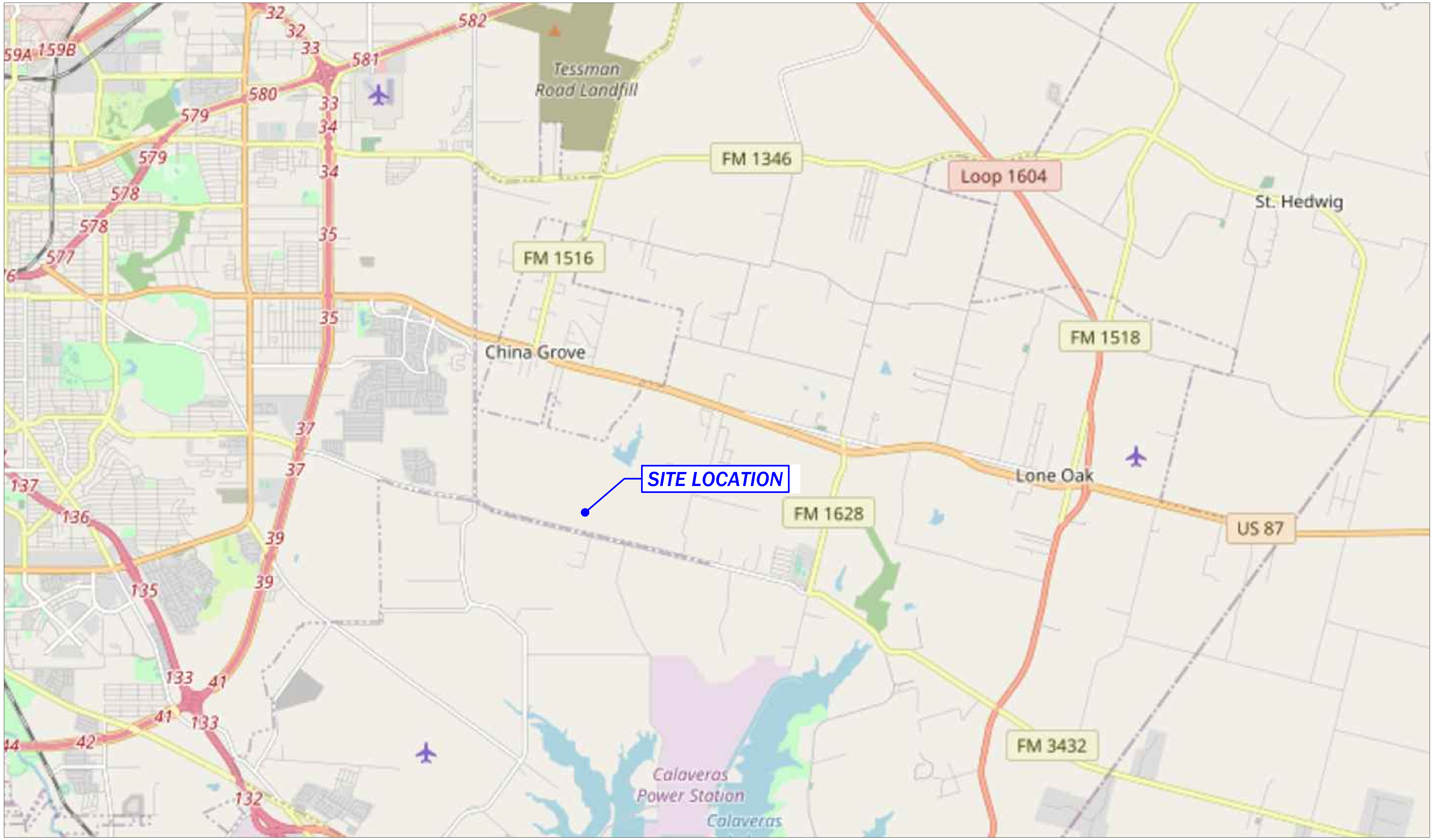
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL
BUSINESS
ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX A
ILLUSTRATIONS



Legend

SITE LOCATION MAP

LENNAR
MIRO MEADOWS SUBDIVISION - LIFT STATION AND PAVEMENT DESIGN

NEW SULPHUR SPRINGS ROAD NEAR BLANDFORD ROAD
SAN ANTONIO ETJ, BEXAR COUNTY, TEXAS

Date:	04/08/2022
Drawn By:	AM
Checked By:	AB
Approved By:	AB
Project No.:	00210903116.02

Exhibit 1

TTL

17215 Jones Maltsberger Rd.,
Suite 101 San Antonio, TX 78247
210.888.6100
TBPE Registration: F-12622
TBPG Registration: 50456

Legend

B–X	Boring Location and Identifier
CBR–X	California Bearing Ratio Sample Location and Identifier

BORING LOCATIONS

LENNAR
MIRO MEADOWS SUBDIVISION - LIFT STATION AND PAVEMENT DESIGN

NEW SULPHUR SPRINGS ROAD NEAR BLANDFORD ROAD
SAN ANTONIO ETJ, BEXAR COUNTY, TEXAS

Date:	04/08/2022
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Exhibit 2

TTL
17215 Jones Maltsberger Rd.,
Suite 101 San Antonio, TX 78247
210.888.6100
TBPE Registration: F–12622
TBPG Registration: 50456

SOIL LEGEND

FINE- AND COARSE-GRAINED SOIL INFORMATION

FINE-GRAINED SOILS (SILTS AND CLAYS)

SPT N-Value	Consistency	Estimated Q_u (TSF)
0 - 1	Very Soft	0 - 0.25
2 - 4	Soft	0.25 - 0.5
5 - 8	Firm	0.5 - 1.0
9 - 15	Stiff	1.0 - 2.0
16 - 30	Very Stiff	2.0 - 4.0
31+	Hard	4.0+

Q_u = Unconfined Compression Strength

COARSE-GRAINED SOILS (SANDS AND GRAVELS)

SPT N-Value	Relative Density
0 - 4	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
51+	Very Dense

PARTICLE SIZE

Name	Size (US Std. Sieve)
Boulders	>300 mm (>12 in.)
Cobbles	75 mm to 300 mm (3 - 12 in.)
Coarse Gravel	19 mm to 75 mm (3/4 - 3 in.)
Fine Gravel	4.75 mm to 19 mm (#4 - 3/4 in.)
Coarse Sand	2 mm to 4.75 mm (#10 - #4)
Medium Sand	0.425 mm to 2 mm (#40 - #10)
Fine Sand	0.075 mm to 0.425 mm (#200 - #40)
Silts and Clays	< 0.075 mm (< #200)

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Terms	Percent of Dry Weight
"Trace"	< 15
"With"	15 - 30
Modifier	> 30

RELATIVE PROPORTIONS OF CLAYS AND SILTS

Descriptive Terms	Percent of Dry Weight
"Trace"	< 5
"With"	5 - 12
Modifier	> 12

CRITERIA FOR DESCRIBING MOISTURE CONDITION

Description	Criteria
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp, but no visible water
Wet	Visible free water, usually soil is below water table

CRITERIA FOR DESCRIBING CEMENTATION

Description	Criteria
Weak	Crumbles or breaks with handling or little finger pressure
Moderate	Crumbles or breaks with considerable finger pressure
Strong	Will not crumble or break with finger pressure

CRITERIA FOR DESCRIBING STRUCTURE

Description	Criteria
Stratified	Alternating layers of varying material or color with layers at least 6 mm thick; note the thickness
Laminated	Alternating layers of varying material or color with the layers less than 6 mm thick; note thickness
Fissured	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided	Fracture planes appear polished or glossy, sometimes striated
Blocky	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed	Inclusion of small pockets of different soils such as small lenses of sand scattered through a mass of clay; note thickness
Homogeneous	Same color and appearance throughout

SAMPLERS AND DRILLING METHODS

▮	AUGER CUTTINGS
◇	BAG/BULK SAMPLE
✋	GRAB SAMPLE
▤	CONTINUOUS SAMPLES
■	SHELBY TUBE SAMPLE
⧗	PITCHER SAMPLE
⊠	STANDARD PENETRATION SPLIT-SPOON SAMPLE
○	SPLIT-SPOON SAMPLE WITH NO RECOVERY
▼	DYNAMIC CONE PENETROMETER
▥	ROCK CORE

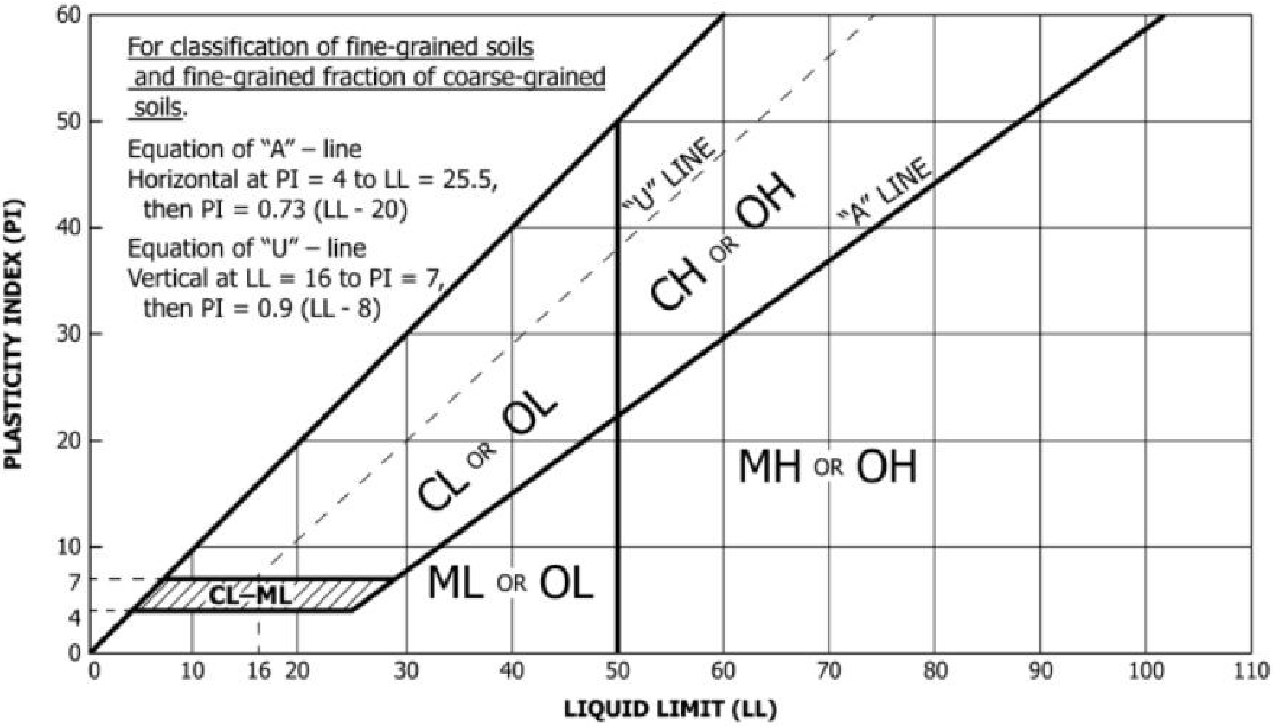
WATER LEVEL SYMBOLS

▽	WATER LEVEL AT TIME OF DRILLING
≡	PERCHED WATER OBSERVED AT DRILLING
▼	DELAYED WATER LEVEL OBSERVATION
▨	CAVE-IN DEPTH
≈	OBSERVED SEEPAGE

ABBREVIATIONS AND ACRONYMS

WOH	Weight of Hammer	N-Value	Sum of the blows for last two 6-in increments of SPT
WOR	Weight of Rod		
Ref.	Refusal	NA	Not Applicable or Not Available
ATD	At Time of Drilling	OD	Outside Diameter
DCP	Dynamic Cone Penetrometer	PPV	Pocket Penetrometer Value
Elev.	Elevation	SFA	Solid Flight Auger
ft.	feet	SH	Shelby Tube Sampler
HSA	Hollow Stem Auger	SS	Split-Spoon Sampler
ID	Inside Diameter	SPT	Standard Penetration Test
in.	inches	USCS	Unified Soil Classification System
lbs	pounds		

PLASTICITY CHART FOR USCS CLASSIFICATION OF FINE-GRAINED SOILS



IMPORTANT NOTES ON TEST BORING RECORDS

1) The report and graphics key are an integral part of these logs. All data and interpretations in this log are subject to the explanations and limitations stated in the report.

2) Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual or differ from those shown. Solid lines are used to indicate a change in the material type, particularly a change in the USCS classification. Dashed lines are used to separate two materials that have the same material type, but that differ with respect to two or more other characteristics (e.g. color, consistency).

3) No warranty is provided as to the continuity of soil or rock conditions between individual sample locations.

4) Logs represent general soil and rock conditions observed at the point of exploration on the date indicated.

5) In general, Unified Soil Classification System (USCS) designations presented on the logs were based on visual classification in the field and were modified where appropriate based on gradation and index property testing.

6) Fine-grained soils that plot within the hatched area on the Plasticity Chart, and coarse-grained soils with between 5% and 12% passing the #200 sieve require dual USCS symbols as presented on the previous page.

7) If the sampler is not able to be driven at least 6 inches, then 50/X" indicates that the sampler advanced X inches when struck 50 times with a 140-pound hammer falling 30 inches.

8) If the sampler is driven at least 6 inches, but cannot be driven either of the subsequent two 6-inch increments, then either 50/X" or the sum of the second 6-inch increment plus 50/X" for the third 6-inch increment will be indicated.

Example 1: Recorded SPT blow counts are 16 - 50/4", the SPT N-value will be shown as N = 50/4"
Example 2: Recorded SPT blow counts are 18 - 25 - 50/2", the SPT N-value will be shown as N = 75/8"

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New Sulphur Springs Road near Blandford Road

San Antonio ETJ, Bexar County, Texas

Log of B-1

Page 1 of 1

Drilling Co.:	*Gainco*	**TTL** Project No.:	*00210903116.02*
Driller:	*M. Balderama*	Date Drilled:	*3/28/2022*
Logged by:	*T. Burford*	Boring Depth:	*40 feet*
Equipment:	*Mobile B-61*	Boring Elevation:	*Ground Surface*
Hammer Type:	*Automatic*	Coordinates:	*Longitude: -98.34028 Latitude: 29.35858*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling*	∇ Water Level at Time of Drilling:	*23.5 ft BGS*
		Cave-In at Time of Drilling:	*N/A*

▼ Delayed Water Level: *24 ft BGS*

Delayed Water Observation Date: *3/28/2022*

Remarks:
Subsurface water was encountered at 23½ feet. After a 15-minute delay, subsurface water was measured at 24 feet. The borehole was backfilled with soil cuttings after drilling activities were completed.

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	MATERIALS DESCRIPTION	TYPE	BORE/CORE DATA (1st 6" - 2nd 6" - 3rd 6"; N-VALUE BLOWS/FT)	P: TONS/SQFT	RQD / % REC	MOISTURE CONTENT (%)	LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY (psf)	SHEAR STRENGTH (psf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	% PASSING #200 SIEVE
		SANDY LEAN CLAY; stiff to hard, very dark brown to light brown (CL)	☒	6 - 6 - 7 N = 13			9	42	18	24					
			☒	34 - 50/3 N = 50/3"			12								
	5		☒	5 - 9 - 13 N = 22			14	38	16	22					67.6
		- calcareous between 6½ and 10 feet	☒	6 - 10 - 13 N = 23			15								
	10	FAT CLAY; very stiff to stiff, mottled gray and brown (CH)	☒	6 - 8 - 13 N = 21			20	63	25	38					
	15		☒	5 - 7 - 10 N = 17			21								88.6
	20		☒	5 - 5 - 9 N = 14			25								
		SANDY LEAN CLAY WITH GRAVEL; very stiff, mottled gray and brown (CH)													
	25		☒	8 - 10 - 15 N = 25			21								
	30	LEAN CLAY; very stiff to hard, gray (CL)	☒	8 - 11 - 16 N = 27			20	43	18	25					
	35		☒	50/2 N = 50/2"			31								89.8
	40		☒	12 - 16 - 28 N = 44			22								
		Boring terminated at 40 feet.													

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New Sulphur Springs Road near Blandford Road

San Antonio ETJ, Bexar County, Texas

Log of B-2

Page 1 of 1

Drilling Co.:	*Gainco*	**TTL** Project No.:	*00210903116.02*
Driller:	*M. Balderama*	Date Drilled:	*3/28/2022*
Logged by:	*T. Burford*	Boring Depth:	*15 feet*
Equipment:	*Mobile B-61*	Boring Elevation:	*Ground Surface*
Hammer Type:	*Automatic*	Coordinates:	*Longitude: -98.34008 Latitude: 29.35872*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling*	∇ Water Level at Time of Drilling:	*Not Encount.*
		Cave-In at Time of Drilling:	*N/A*

Remarks:
Subsurface water was not encountered during drilling. The borehole was backfilled with soil cuttings after drilling activities were completed.

▼ Delayed Water Level: *N/A*

Delayed Water Observation Date: *N/A*

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	SAMPLE DATA TYPE	BORE/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE BLOWS/FT	P: TONS/SOFT	RQD / % REC	MOISTURE CONTENT (%)	ATTERBERG LIMITS (%) LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY (psf)	SHEAR STRENGTH (psf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	% PASSING #200 SIEVE
		▨	LEAN CLAY; firm to stiff, very dark brown to mottled gray and yellow (CL)	☒	2 - 2 - 3 N = 5			14								88.7
		▨		☒	4 - 6 - 8 N = 14			16	35	15	20					
	5	▨	- calcareous between 4½ and 8 feet	☒	3 - 6 - 7 N = 13			18								
		▨	FAT CLAY; very stiff to stiff, mottled gray and yellow to gray (CH)	☒	5 - 7 - 11 N = 18			21	67	27	40					
		▨		☒	5 - 6 - 13 N = 19			23								
	10	▨														
	15	▨		☒	4 - 6 - 9 N = 15			22								93.9
			Boring terminated at 15 feet.													
	20															
	25															
	30															
	35															
	40															

TTL

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New Sulphur Springs Road near Blandford Road
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Log of B-3
Page 1 of 1

Drilling Co.: *Gainco*
Driller: *M. Balderama*
Logged by: *T. Burford*
Equipment: *Mobile B-61*
Hammer Type: *Automatic*
Drilling Method: *Hollow Stem Auger w/SPT Sampling*

TTL Project No.: *00210903116.02*
Date Drilled: *3/28/2022*
Boring Depth: *10 feet*
Boring Elevation: *Ground Surface*
Coordinates: *Longitude: -98.3399 Latitude: 29.36033*

Remarks:
Subsurface water was not encountered during drilling. The borehole was backfilled with soil cuttings after drilling activities were completed.

▽ Water Level at Time of Drilling: *Not Encount.*
Cave-In at Time of Drilling: *N/A*
▼ Delayed Water Level: *N/A*
Delayed Water Observation Date: *N/A*

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	SAMPLE DATA: TYPE	BORE/CORE DATA: 1st 6" 2nd 6" 3rd 6" N-VALUE BLOWS/FT	P: TONS/SQFT	RQD / % REC	MOISTURE CONTENT (%)	ATTERBERG LIMITS (%): LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY (psf)	SHEAR STRENGTH (psf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	% PASSING #200 SIEVE
		▨	SANDY LEAN CLAY; firm to very stiff, light brown to yellowish-brown (CL)	☒	3 - 2 - 6 N = 8			13								53.9
		▨		☒	8 - 11 - 12 N = 23			9	30	15	15					
	5	▨		☒	10 - 10 - 13 N = 23			8								
		▨		☒	11 - 11 - 10 N = 21			9	26	13	13					
		▨	- becomes mottled gray and yellow below 8½ feet	☒	6 - 6 - 6 N = 12			16								
	10		Boring terminated at 10 feet.													
	15															
	20															
	25															
	30															
	35															
	40															

R:\GINT\TTL\PROJECTS\2021\00210903116 -- MIRO MEADOWS.GPJ 5/5/22 Report:AEP-GEOTECH LOG - LAT LONG

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New Sulphur Springs Road near Blandford Road
San Antonio ETJ, Bexar County, Texas

Log of B-4
Page 1 of 1

Drilling Co.: *Gainco*

Driller: *M. Balderama*

Logged by: *T. Burford*

Equipment: *Mobile B-61*

Hammer Type: *Automatic*

Drilling Method: *Hollow Stem Auger w/SPT Sampling*

TTL Project No.: *00210903116.02*

Date Drilled: *3/28/2022*

Boring Depth: *10 feet*

Boring Elevation: *Ground Surface*

Coordinates: *Longitude: -98.33974 Latitude: 29.35846*

▽ Water Level at Time of Drilling: *Not Encount.*

Cave-In at Time of Drilling: *N/A*

▼ Delayed Water Level: *N/A*

Delayed Water Observation Date: *N/A*

Remarks:
Subsurface water was not encountered during drilling. The borehole was backfilled with soil cuttings after drilling activities were completed.

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	SAMPLE DATA TYPE	BORE/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE BLOWS/FT	P: TONS/SQFT	RQD % REC	MOISTURE CONTENT (%)	ATTERBERG LIMITS (%) LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY (psf)	SHEAR STRENGTH (psf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	% PASSING #200 SIEVE
		▨	LEAN CLAY; firm to hard, very dark brown to mottled yellowish-brown and gray (CL)	☒	3 - 3 - 4 N = 7			16	27	13	14					
		▨		☒	20 - 25 - 7 N = 32			12								
	5	▨	- calcareous between 4½ and 8 feet	☒	7 - 7 - 9 N = 16			15	41	15	26					
		▨		☒	6 - 8 - 12 N = 20			21								91.2
		▨		☒	5 - 9 - 12 N = 21			23								
	10		Boring terminated at 10 feet.													
	15															
	20															
	25															
	30															
	35															
	40															

This boring log shall not be separated from the corresponding Instrument of Service; no third party may rely upon this boring log or the corresponding Instrument of Service absent a written TTL Secondary Client Agreement.

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Miro Meadows Subdivision - Lift Station and Pavement Design
New Sulphur Springs Road near Blandford Road

San Antonio ETJ, Bexar County, Texas

Log of B-5

Drilling Co.:	*Gainco*	**TTL** Project No.:	*00210903116.02*
Driller:	*M. Balderama*	Date Drilled:	*3/28/2022*
Logged by:	*T. Burford*	Boring Depth:	*10 feet*
Equipment:	*Mobile B-61*	Boring Elevation:	*Ground Surface*
Hammer Type:	*Automatic*	Coordinates:	*Longitude: -98.33887 Latitude: 29.36143*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling*	▽ Water Level at Time of Drilling:	*Not Encount.*
		Cave-In at Time of Drilling:	*N/A*
▼ Delayed Water Level:	*N/A*	Delayed Water Observation Date:	*N/A*

Remarks:
Subsurface water was not encountered during drilling. The borehole was backfilled with soil cuttings after drilling activities were completed.

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	TYPE	BORE/CORE DATA: 1st 6" 2nd 6" 3rd 6" N-VALUE BLOWS/FT	P: TONS/SQFT	RQD / % REC	MOISTURE CONTENT (%)	ATTERBERG LIMITS (%) LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY (psf)	SHEAR STRENGTH (psf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	% PASSING #200 SIEVE
		▨	LEAN CLAY; stiff to hard, brown to yellowish-brown (CL)	☒	5 - 4 - 6 N = 10			3								93.8
				☒	9 - 10 - 14 N = 24			11	33	15	18					
	5			☒	10 - 11 - 10 N = 21			12								
				☒	8 - 10 - 12 N = 22			11	37	16	21					
				☒	7 - 50/2 N = 50/2"			9	31	17	14					
	10		Boring terminated at 10 feet.													
	15															
	20															
	25															
	30															
	35															
	40															

Lennar
Miro Meadows Subdivision - Lift Station and Pavement Design
New Sulphur Springs Road near Blandford Road
San Antonio ETJ, Bexar County, Texas

Log of
B-6

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Drilling Co.:	*Gainco*	**TTL** Project No.:	*00210903116.02*
Driller:	*M. Balderama*	Date Drilled:	*3/28/2022*
Logged by:	*T. Burford*	Boring Depth:	*10 feet*
Equipment:	*Mobile B-61*	Boring Elevation:	*Ground Surface*
Hammer Type:	*Automatic*	Coordinates:	*Longitude: -98.33901 Latitude: 29.36267*
Drilling Method:	*Hollow Stem Auger w/SPT Sampling*	∇ Water Level at Time of Drilling:	*Not Encount.*
		Cave-In at Time of Drilling:	*N/A*

Remarks:
Subsurface water was not encountered during drilling. The borehole was backfilled with soil cuttings after drilling activities were completed.

▼ Delayed Water Level: *N/A*

Delayed Water Observation Date: *N/A*

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	TYPE	BORE/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE BLOWS/FT	P: TONS/SQFT	RQD % REC	MOISTURE CONTENT (%)	ATTERBERG LIMITS (%) LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY (psf)	SHEAR STRENGTH (psf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	% PASSING #200 SIEVE
		▨	LEAN CLAY WITH SAND; stiff to hard, brown to light brown (CL)	☒	3 - 6 - 8 N = 14			13	39	18	21					
		▨		☒	20 - 16 - 17 N = 33			7								73.6
	5	▨	- trace gravel between 4½ and 6 feet	☒	50 N = 50"			6	27	14	13					
		▨		☒	50/4 N = 50/4"			6								
		▨		☒	45 - 11 - 16 N = 27			14								86.5
	10		Boring terminated at 10 feet.													
	15															
	20															
	25															
	30															
	35															
	40															

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This boring log shall not be separated from the corresponding Instrument of Service; no third party may rely upon this boring log or the corresponding Instrument of Service absent a written TTL Secondary Client Agreement.

Lennar
Miro Meadows Subdivision - Lift Station and Pavement Design
New Sulphur Springs Road near Blandford Road

San Antonio ETJ, Bexar County, Texas

Log of
B-7

Page 1 of 1

Drilling Co.: *Gainco*

Driller: *M. Balderama*

Logged by: *T. Burford*

Equipment: *Mobile B-61*

Hammer Type: *Automatic*

Drilling Method: *Hollow Stem Auger w/SPT Sampling*

TTL Project No.: *00210903116.02*

Date Drilled: *3/28/2022*

Boring Depth: *10 feet*

Boring Elevation: *Ground Surface*

Coordinates: *Longitude: -98.33775 Latitude: 29.36352*

∇ Water Level at Time of Drilling: *Not Encount.*

Cave-In at Time of Drilling: *N/A*

Remarks:
Subsurface water was not encountered during drilling. The borehole was backfilled with soil cuttings after drilling activities were completed.

▼ Delayed Water Level: *N/A*

Delayed Water Observation Date: *N/A*

SAMPLE DATA

ELEVATION (ft)	DEPTH (ft)	GRAPHIC LOG	MATERIALS DESCRIPTION	TYPE	BORE/CORE DATA 1st 6" 2nd 6" 3rd 6" N-VALUE BLOWS/FT	P: TONS/SQFT	RQD / % REC	MOISTURE CONTENT (%)	ATTERBERG LIMITS (%) LIQUID LIMIT LL	PLASTIC LIMIT PL	PLASTICITY INDEX PI	DRY DENSITY (psf)	SHEAR STRENGTH (psf)	FAILURE STRAIN (%)	CONFINING PRESSURE (psi)	% PASSING #200 SIEVE
		▨	LEAN CLAY WITH SAND; stiff to hrad, brown to mottled brown and gray (CL)	☒	5 - 6 - 9 N = 15			10								
		▨		☒	9 - 11 - 14 N = 25			8	32	15	17					
	5	▨		☒	16 - 13 - 13 N = 26			9								81.1
		▨		☒	26 - 28 - 29 N = 57			13								
		▨		☒	6 - 12 - 18 N = 30			15								
	10		Boring terminated at 10 feet.													
	15															
	20															
	25															
	30															
	35															
	40															

Boring	Depth	USCS	AASHTO	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	% Gravel	% Sand	Maximum Size (mm)	% Passing #200 % Silt % Clay (If hydrometer data available)	D50 (mm)
B-1	0.5 - 2	---	---	9	42	18	24	---	---	---	---	---
B-1	2.5 - 4	---	---	12	---	---	---	---	---	---	---	---
B-1	4.5 - 6	CL	A-6 (13)	14	38	16	22	0.0	0.0	0.075	67.6	---
B-1	6.5 - 8	---	---	15	---	---	---	---	---	---	---	---
B-1	8.5 - 10	---	---	20	63	25	38	---	---	---	---	---
B-1	13.5 - 15	---	---	21	---	---	---	0.0	0.0	0.075	88.6	---
B-1	18.5 - 20	---	---	25	---	---	---	---	---	---	---	---
B-1	23.5 - 25	---	---	21	---	---	---	---	---	---	---	---
B-1	28.5 - 30	---	---	20	43	18	25	---	---	---	---	---
B-1	33.5 - 35	---	---	31	---	---	---	0.0	0.0	0.075	89.8	---
B-1	38.5 - 40	---	---	22	---	---	---	---	---	---	---	---
B-2	0.5 - 2	---	---	14	---	---	---	0.0	0.0	0.075	88.7	---
B-2	2.5 - 4	---	---	16	35	15	20	---	---	---	---	---
B-2	4.5 - 6	---	---	18	---	---	---	---	---	---	---	---
B-2	6.5 - 8	---	---	21	67	27	40	---	---	---	---	---
B-2	8.5 - 10	---	---	23	---	---	---	---	---	---	---	---
B-2	13.5 - 15	---	---	22	---	---	---	0.0	0.0	0.075	93.9	---
B-3	0.5 - 2	---	---	13	---	---	---	0.0	0.0	0.075	53.9	---
B-3	2.5 - 4	---	---	9	30	15	15	---	---	---	---	---
B-3	4.5 - 6	---	---	8	---	---	---	---	---	---	---	---
B-3	6.5 - 8	---	---	9	26	13	13	---	---	---	---	---
B-3	8.5 - 10	---	---	16	---	---	---	---	---	---	---	---
B-4	0.5 - 2	---	---	16	27	13	14	---	---	---	---	---
B-4	2.5 - 4	---	---	12	---	---	---	---	---	---	---	---
B-4	4.5 - 6	---	---	15	41	15	26	---	---	---	---	---
B-4	6.5 - 8	---	---	21	---	---	---	0.0	0.0	0.075	91.2	---
B-4	8.5 - 10	---	---	23	---	---	---	---	---	---	---	---
B-5	0.5 - 2	---	---	3	---	---	---	0.0	0.0	0.075	93.8	---
B-5	2.5 - 4	---	---	11	33	15	18	---	---	---	---	---
B-5	4.5 - 6	---	---	12	---	---	---	---	---	---	---	---
B-5	6.5 - 8	---	---	11	37	16	21	---	---	---	---	---
B-5	8.5 - 10	---	---	9	31	17	14	---	---	---	---	---
B-6	0.5 - 2	---	---	13	39	18	21	---	---	---	---	---
B-6	2.5 - 4	---	---	7	---	---	---	0.0	0.0	0.075	73.6	---
B-6	4.5 - 6	---	---	6	27	14	13	---	---	---	---	---
B-6	6.5 - 8	---	---	6	---	---	---	---	---	---	---	---
B-6	8.5 - 10	---	---	14	---	---	---	0.0	0.0	0.075	86.5	---
B-7	0.5 - 2	---	---	10	---	---	---	---	---	---	---	---
B-7	2.5 - 4	---	---	8	32	15	17	---	---	---	---	---
B-7	4.5 - 6	---	---	9	---	---	---	0.0	0.0	0.075	81.1	---
B-7	6.5 - 8	---	---	13	---	---	---	---	---	---	---	---
B-7	8.5 - 10	---	---	15	---	---	---	---	---	---	---	---

Summary of Laboratory Test Results

Client: Lennar
Project: Miro Meadows Subdivision - Lift Station and Pavement Design
Location: San Antonio ETJ, Bexar County, Texas
Project Number: 00210903116.02

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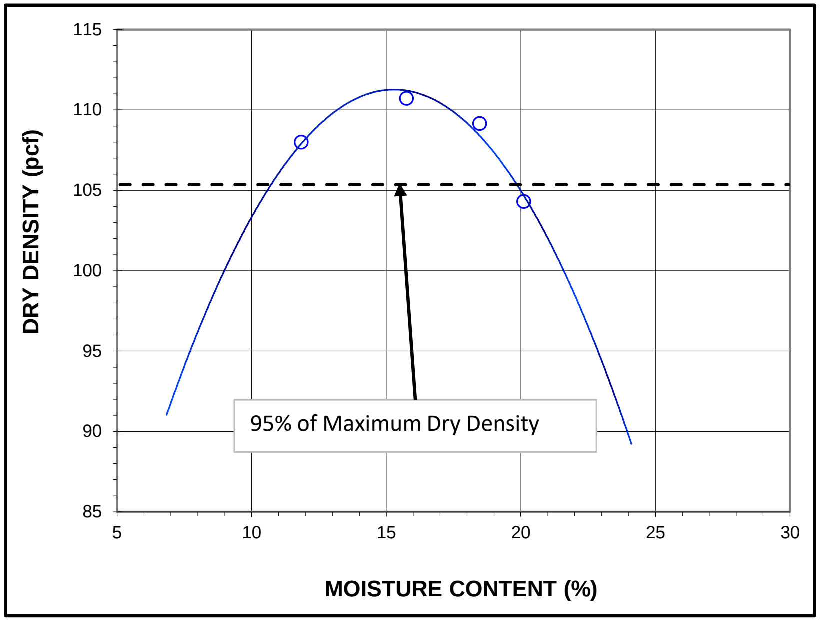

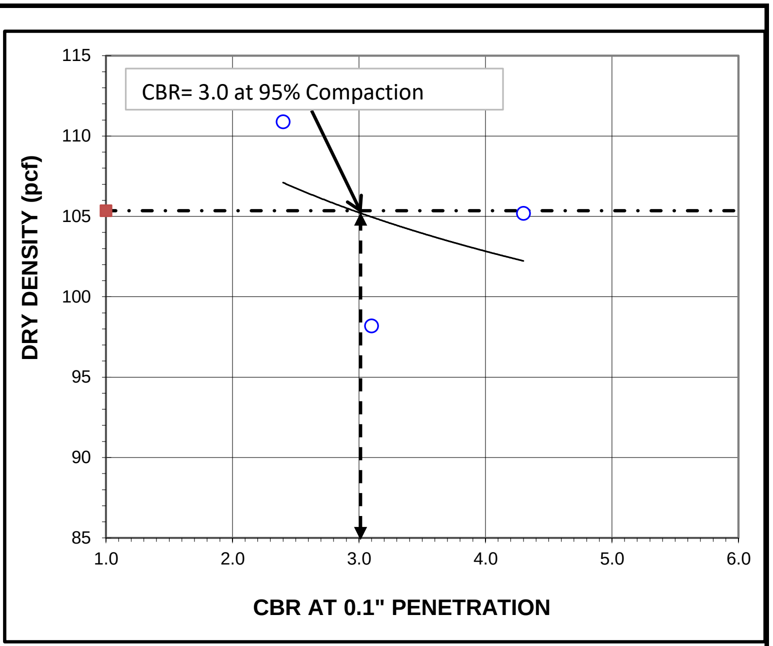

CBR Sample Location: 29.362553°, -98.338518°

Sample Depth: Between 0 and 5 feet below existing ground surface

Optimum Moisture Content: 16.6 %

Maximum Dry Unit Weight: 110.9 pcf

% Passing # 200 Sieve 44.8 %

Atterberg Limits: LL = 23 , PL = 16 , PI = 7

Sample: **CBR Sample No. 1**

Proctor Test Method: Standard Proctor (ASTM D-698)

CBR Test Method: California Bearing Ration (ASTM D-1883)

Material: CLAYEY SAND (SC), brown

17215 Jones Maltsberger Rd, Suite 101
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T: 210-888-6100
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MIRO MEADOWS SUBDIVISION - LIFT STATION AND PAVEMENT DESIGN
NEW SULPHUR SPRINGS ROAD NEAR BLANDFOR ROAD

SAN ANTONIO ETJ, BEXAR COUNTY, TEXAS

Drawn By: AM
Checked By: AB
Proj No:00210903116.02
File Name

CBR PLOT

APPENDIX B
REFERENCE MATERIALS

EXPLORATION PROCEDURES

General

Various drill equipment and procedures are used to obtain soil or rock specimens during geotechnical engineering exploration activities. The drill equipment typically consists of fuel powered machinery that is mounted on a flat-bed truck or an all-terrain vehicle. The ground surface conditions at the site generally determine the type of vehicle to use.

Borings can be drilled either dry or wet. The drilling technique depends on the type of subsurface materials (clays, sands, silts, gravels, rock) encountered and whether or not subsurface water is present during the drilling operations. Sometimes a combination of both techniques is implemented.

The dry method can generally be employed when subsurface water or granular soils are not present. The dry method generally consists of advancing the augers without the use of water or drilling fluids. Air can be employed as necessary to remove cuttings from the borehole or cool the drilling bits during some drilling applications. The wet rotary process is generally used when subsurface water, rock or granular soils are present. The wet rotary process utilizes water or drilling fluids to advance the augers, remove cuttings from the borehole, and cool the drilling bits during drilling.

Sampling

Various sampling devices are available to recover soil or rock specimens during the geotechnical exploration program. The type of sampling apparatus to employ depends on the subsurface materials (clays, sands, silts, gravels, rock) encountered and on their consistency or strength. Most commonly used samplers are Shelby tubes, split-spoons or split-barrels, and NX core barrels. Depending on the subsurface conditions, sampling apparatus such as the Pitcher barrel, Osterberg sampler, Dennison barrel, or California sampler are sometimes used. The procedures for using and sampling subsurface materials with most of these samplers are described in detail by the American Society for Testing and Materials (ASTM). Sampling is generally performed on a two (2) foot continuous interval to a depth of about ten (10) feet, followed by five (5) foot intervals between the depths of about ten (10) to 50 feet, and on ten (10) foot intervals thereafter to the termination depth of the borings. However, sampling intervals may change depending on the project scope and actual subsurface conditions encountered.

If cohesive soils (clays and some silts) are present during drilling, samples are retrieved by using the Shelby tube sampler (ASTM D 1587) or the split-barrel sampler (ASTM D 1586). The Shelby tube is used to recover “virtually” undisturbed soil specimens that can be returned to the laboratory for strength and compressibility testing. The Shelby tube is a three (3) inch nominal diameter, thin-walled tube that is advanced hydraulically into the soil by a single stroke of the drill equipment.

The split-barrel sampler is used when performing the Standard Penetration Test (SPT). The recovered sample is considered to be a “disturbed” specimen due to the SPT procedure. The split-barrel is advanced into the soil by driving the sampler with blows from a 140-pound hammer free falling 30 inches. The SPT procedure is performed to evaluate the strength or competency of the material being sampled. This evaluation is based on the material sampled, depth of the sample, and the number of blows required to obtain full penetration of the split-barrel sampler. This blow count or penetration resistance is referred to as the “N” value.

The split-barrel is typically used when cohesionless soils (sands, silts, gravels) are encountered or when good quality cohesive soils cannot be recovered with the Shelby tube sampler. The SPT procedure can be employed when rock or cemented zones are encountered. However, the split-barrel may not penetrate the rock or cemented zone if the layer is extremely hard, thus resulting in no sample recovery.

When rock or cemented zones are present, and depending on the type of project and engineering testing required, rock coring may be implemented to recover specimens of the particular layer. Typically, an NX double tube core barrel (ASTM D 2113) is used.

Logging

During the drilling activities, one of our geologists or engineering technicians is present to make sure that the appropriate sampling techniques are employed and to extrude or remove all materials from the samplers. The samples are then visually classified by our field representative who records the information on a field boring log. Our field representative may perform pocket penetrometer, hand torvane, or field vane tests on the subsurface materials recovered from the Shelby tube samplers. If the SPT procedure is employed, our field representative will record the N values or blow counts that are germane to that particular field test. If rock coring is utilized, our field representative will calculate the percent recovery and Rock Quality Designation (RQD). The test data for all the field tests will be noted on the appropriate field boring log. Upon completion of the logging activities and field testing of the recovered soil or rock samples, representative portions of the specimens were placed in appropriately wrapped and sealed containers to preserve their natural moisture condition and to minimize disturbance during handling and transporting to our laboratory for additional testing.

When subsurface water is observed during the drilling and sampling operations, drilling will be temporarily delayed so the subsurface water level can be monitored for a period of at least 15 to 30 minutes. Depending on the rise of the subsurface water in the borehole and project requirements, subsurface water measurements may be monitored for periods of 24 hours or more. Generally, observation wells or piezometers are installed in the completed boreholes to monitor subsurface water levels for periods longer than 24 hours.

Following completion of drilling, sampling, and subsurface water monitoring, all boreholes are backfilled with soil cuttings from the completed borings unless the client requests or local ordinance requires special backfilling requirements. If there are not enough soil cuttings available, clean sand will be used to backfill the completed boreholes.

Details concerning the subsurface conditions are provided on each individual boring log presented in this Appendix. The terms and symbols used on each boring log are defined in the Legend Sheet which is also presented in this Appendix.

LABORATORY TESTING PROCEDURES

Classification and Index Testing

The recovered soil samples were classified in the laboratory by a geoprofessional using the USCS as a guide. Samples were tested for the following properties in general accordance with the applicable ASTM standards:

- Moisture content (ASTM D2216),
- Atterberg Limits (ASTM D4318),
- Percent material passing the No. 200 sieve (ASTM D1140),
- Grain Size Analysis (ASTM D6913 or D1140), and
- California Bearing Ratio test (ASTM D1883).
- Soluble Sulfates (M4500-SO4 E).

Results of tests for moisture content, Atterberg Limits, and percent material passing the No. 200 sieve are presented on individual boring logs in Appendix A. The results are also tabulated on the Summary of Laboratory Results sheet in Appendix A.