GEOTECHNICAL ENGINEERING STUDY

FOR

HICKORY RIDGE – PHASE 1 UNIT 2
ELMENDORF, TEXAS

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Project No. ASA22-114-00
January 9, 2023

Mr. Kirk Breitenwischer
Executive Vice President
Castlerock Communities
2401 Fountain View Drive, Suite 215
Houston, Texas 77057

RE: Geotechnical Engineering Study
Hickory Ridge – Phase 1 Unit 2
Elmendorf, Texas

Dear Mr. Breitenwischer:

Raba Kistner, Inc. (RKI) is pleased to submit the report of our Geotechnical Engineering Study for the above-referenced project. This study was performed in accordance with RKI Proposal No. PSA22-172-00, dated November 7, 2022. The purpose of this study was to drill borings within the proposed Hickory Ridge Unit 2 roadways, to perform laboratory testing to evaluate and characterize subsurface conditions, and to prepare an engineering report presenting pavement design and construction guidelines.

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

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

Very truly yours,

RABA KISTNER, INC.

Ryan DeWees
Graduate Engineer

Isaac Molina, P.E.
Project Engineer

STATE OF TEXAS
ISAAC MOLINA
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LICENSED
PROFESSIONAL ENGINEER

RD/IM/kv

Attachments

Copies Submitted: Above (Electronic)

GEOTECHNICAL ENGINEERING STUDY

For

HICKORY RIDGE – PHASE 1 UNIT 2
ELMENDORF, TEXAS

Prepared for

CASTLEROCK COMMUNITIES
Houston, Texas

Prepared by

RABA KISTNER, INC.
San Antonio, Texas

PROJECT NO. ASA22-114-00

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RABA KISTNER

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ATTACHMENTS

The following figures are attached and complete this report:

INTRODUCTION

RABA KISTNER, Inc. (RKI) has completed the authorized subsurface exploration for the proposed Hickory Ridge Unit 2 roadways located in Elmendorf, Texas. This report briefly describes the procedures utilized during this study and presents our findings along with our recommendations for pavement design and construction guidelines.

PROJECT DESCRIPTION

To be considered in this study are the roadways interior to Phase 1 Unit 2 of the Hickory Ridge Subdivision in Elmendorf, Bexar County, Texas. The site is located approximately 0.8 miles southwest of the intersection of Loop 1604 and U.S. Highway 181. We understand that the proposed roadways will be classified as Local Type A roadways using the City of San Antonio Pavement Design standards.

LIMITATIONS

This engineering report has been prepared in accordance with accepted Geotechnical Engineering practices in the region of south/central Texas and for the use of Castlerock Communities (CLIENT) and its representatives for design purposes. This report may not contain sufficient information for purposes of other parties or other uses. This report is not intended for use in determining construction means and methods.

The recommendations submitted in this report are based on the data obtained from 5 borings drilled at this site, our understanding of the project information provided to us. If the project information described in this report is incorrect, is altered, or if new information is available, we should be retained to review and modify our recommendations.

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

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

If final grade elevations for the proposed roadways are significantly different existing surface elevations (more than plus or minus 1 ft), our office should be informed about these changes. If needed and/or if desired, we will reexamine our analyses and make supplemental recommendations.

BORINGS AND LABORATORY TESTS

Subsurface conditions at the site were evaluated by 5 borings drilled at the locations shown on the Boring Location Map, Figure 1. These locations are approximate and distances were measured using a recreational-grade, hand-held GPS locator. The borings were drilled using a truck-mounted drilling rig to an approximate depth of 14 ft below the existing ground surface. During drilling operations, Split-Spoon samples (with Standard Penetration Testing) and auger cutting samples were collected. Each sample was visually classified in the laboratory by a member of our Geotechnical Engineering staff. The geotechnical engineering properties of the strata were evaluated by the natural moisture content, percent passing a No. 200 sieve, Atterberg limits, Laboratory CBR (with moisture density relationship), sulfate content, ph-Lime series, and Soil-lime compression strength testing.

Laboratory test results are presented in graphical or numerical form on the boring logs illustrated on Figures 2 through 6. A key to classification terms and symbols used on the logs is presented on Figure 7. The results of the laboratory and field testing are also tabulated on Figure 8 for ease of reference.

Standard penetration test results are noted as “blows per ft” on the boring logs and Figure 8, where “blows per ft” refers to the number of blows by a falling hammer required for 1 ft of penetration into the soil/weak rock. Where hard or dense materials were encountered, the tests were terminated at 50 blows even if one foot of penetration had not been achieved. When all 50 blows fall within the first 6 in. (seating blows), refusal “ref” for 6 in. or less will be noted on the boring logs and on Figure 8.

In addition to the above listed testing and sampling, a bulk sample of the anticipated subgrade soils was collected from an area near Boring P-2 for use in a moisture-density relationship ASTM D698 and Tex-113-E Proctor (Figure 9), CBR (Figure 10) test, pH-Lime Series test (Figure 11), Soil-lime compression strength test, and a sulfate content test. A summary of the bulk sample testing results are presented in the following table:

Material Type and Location	Proctor Max Dry Density and Optimum Moisture	Laboratory CBR	Average Percent Swell	Sulfate Content (ppm)	Unconfined Compressive Strength (psi)
Tan Clay (Boring B-2)	120.4 pcf @ 11.4%	12.4	0.3	160	-
Lime Treated Tan Clay (Boring B-2)	121.3 pcf @ 9.4%	-	-	-	148 (1)

(1)The results of the testing show that a 3% mixture by unit weight of the soil has an average compressive strength of 148 psi which exceeds the CoSA standard of 50 psi for structural credit.

Results of the soil-lime test are also discussed in the *Pavement Design* section of this report.

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

GENERAL SITE CONDITIONS

GEOLOGY

A review of the *Geologic* the *Atlas of Texas, Austin Sheet*, indicates that this site is naturally underlain with soils/rocks of the Carrizo Sand, which is composed of Sandstone, fine to coarse grained, poorly sorted and thickly bedded. The grains may be friable to locally indurated and noncalcareous. Light yellow to orange and brown in color. Heavily Iron–oxide banded. In the upper part of the formation carbonaceous clay and silts are present and weathered, characterized by reddish brown ferric deposits and some beds of ironstone. In the eastern zones the formation is thickly forested with oak.

STRATIGRAPHY

In general, the existing natural stratigraphy can be described as a relatively thin layer of surficial dark brown clay, reddish brown sands, or sandy clays. The surficial layers are underlain by either a tan and gray clay or a dark gray clay layer. Each stratum has been designated by grouping soils that possess similar physical and engineering characteristics. The boring logs should be consulted for more specific stratigraphic information. Unless noted on the boring logs, the lines designating the changes between various strata represent approximate boundaries. The transition between materials may be gradual or may occur between recovered samples. The stratification given on the boring logs, or described herein, is for use by RKI in its analyses and should not be used as the basis of design or construction cost estimates without realizing that there can be variation from that shown or described.

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

GROUNDWATER

Groundwater was not encountered during drilling or immediately upon completion of the drilling operations. However, it is possible for groundwater to exist beneath this site at shallow depths on a transient basis, particularly following periods of precipitation and within granular layers. Fluctuations in groundwater levels occur due to variation in rainfall and surface water run-off. The construction process itself may also cause variations in the groundwater level.

SULFATE TESTING

Sulfate testing was performed a bulk sample taken from an area close to Boring P-2. The results of the sulfate content test are presented in the table in the *Borings and Laboratory Tests* section of this report.

The purpose of the sulfate testing was to determine the concentration of soluble sulfates in the subgrade soils, in order to investigate the potential for an adverse reaction to lime in sulfate-containing soils. The adverse reaction, referred to as sulfate-induced heave, has been known to cause cohesive subgrade soils to swell in short periods of time, resulting in pavement heaving and possible failure. Sulfates can also affect the durability of concrete when encountered in high concentrations.

Our single soil sulfate concentration test result showed a "*Negligible*" potential to cause sulfate induced heave, however, if subgrade treatment is considered additional sulfate content testing is recommended during the construction phase. Reported sulfate concentrations above 3,000 ppm are known to cause sulfate induced heaving when the soils are mixed with lime. If the option for lime is considered, a quality assurance program should be implemented to assist in reducing the risk of sulfate induced heaving.

FLEXIBLE PAVEMENT RECOMMENDATIONS

SUBGRADE STRENGTH CHARACTERIZATION

We have assumed the pavement subgrade will consist of recompacted on-site soils. The CBR will be measured using *ASTM D 1883 Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils* and was calculated to be 12.4 using the soaked sample methodology for material obtained from an area near Boring P-2. Swell was also measured as part of the CBR procedure and the average swell was determined to be 0.3 percent. Based on the laboratory CBR results, soil classification test results, and our experience with similar subgrade soils, we have assigned a design CBR value of 5.0 for the soil subgrade to be used in pavement thickness design analyses. If soils are imported for the purpose of constructing the roadbed then imported materials must be selected that have a CBR value of at least 5.0 and a PI of less than 20. If lower quality fill materials are utilized, the pavement sections will have to be increased based on the quality (tested CBR value) of the imported materials.

DESIGN PARAMETERS

The proposed roadways were evaluated in general accordance with the City of San Antonio's Design (CoSA) Guidance Manual. We understand that the proposed roadways will be classified as Local A. Based on information provided by the City of San Antonio, we understand that the following design parameters are required for use in the design of pavement sections for this street classification:

Pavement Design Parameters	Design Input	
	Flexible Pavement	Rigid Pavement
18-kip ESALs Local A without Bus Traffic Local A with Bus Traffic	 100,000 1,000,000	 150,000 1,500,000
CBR	5.0(1)	
Initial Serviceability Index	4.2	4.5
Terminal Serviceability Index	2.5	
Overall Standard Deviation	0.45	0.35
Reliability Local A without Bus Traffic Local A with Bus Traffic	70%	
Modulus of Subgrade reaction (k-value)	-	140 to 300 pci
28-day Concrete Modulus of Rupture	-	600 psi
28-day Concrete Elastic Modulus	-	4,000,000 psi
Load Transfer Coefficient Local A without Bus Traffic Local A with Bus Traffic	-	 3.7 3.7
Drainage Coefficient	-	1.02
Roadbed Soil Resilient Modulus	7,500(1)	-
Minimum Structural Number Local A without Bus Traffic Local A with Bus Traffic	 2.05 2.95	-
Maximum Structural Number Local A without Bus Traffic Local A with Bus Traffic	 5.08 5.76	-
Minimum Pavement Thickness (in.) Local A without Bus Traffic Local A with Bus Traffic	-	 5.0 6.0
Maximum Pavement Thickness (in.) Local A without Bus Traffic Local A with Bus Traffic	-	 6.0 8.0

(1)The design CBR and corresponding Soil Resilient Modulus values was estimated based on laboratory CBR results and our experience with soils in the area.

The required structural number (SN) is related to the CBR value of the pavement subgrade and the amount of traffic that the pavement will carry over its service life. The required structural number for flexible pavement design was determined to be 2.05 for Local A without Bus Traffic and 2.95 for Local A with Bus Traffic. The CBR provides an estimate of the relative strength of the subgrade and consequently indicates the ability of the pavement section to carry load. If clay soils are imported for the purpose of constructing the roadbed then imported materials must be selected that have a CBR value of at least 5.0. If lower quality clay fill materials are utilized, the pavement sections will have to be increased based on the quality (tested CBR value) of the clays imported. The selected design CBR value was utilized in conjunction with the above specified parameters to determine the required SN for use in the design of the pavement section.

Concrete slab support is characterized by the k-value (modulus of subgrade/subbase reaction). The selected k-value for the subgrade material at this site was estimated based our experience with similar soils and laboratory CBR results. The k-value for the subbase material was selected based on the *City of San Antonio's Design Guidance Manual* recommended subbase layer options for rigid pavement design.

We recommend that subgrade soils with a plasticity index (PI) greater than 20 be treated with lime or other proven methods of treatment to reduce the PI of the soil to less than 20. **Based on the results of our Atterberg limits testing performed in the upper 5 ft of our borings, the PI values of the subgrade materials are generally above 20.**

RECOMMENDED PAVEMENT SECTIONS

Utilizing the design SN values discussed above and minimum layer thicknesses, the optional pavement sections presented in the following table are recommended. Soil lime testing (Tex-121-E) was completed on a sample for an area near Boring P-2. The results of the testing show that a 3% mixture by unit weight of the soil has an average compressive strength of 148 psi which exceeds the CoSA standard of 50 psi. Based on this result, structural credit can be given to lime treated onsite soils and is applied to Flexible Base Option 2 for each roadway classification presented in the following table.

Street Classification	Subgrade	Options	Layer Description	Layer Thickness(1)	Recommended SN Coeff.	S.N. Extension
Local A without Bus Traffic	Soil (CBR=5.0)	**Flexible Base Option 1**	Type C or D Surface Course	2.0 in.	0.44	0.88
			Flexible (Granular) Base	9.0 in.	0.14	1.26
			Low PI Subgrade Material(2)	6.0 in.	0.00	0.00
			Combined Total	**17.0 in.**		**2.14**
		Flexible Base Option 2	Type C or D Surface Course	2.0 in.	0.44	0.88
			Flexible (Granular) Base	6.0 in.	0.14	0.84
			Treated Subgrade(2)	6.0 in.	0.08	0.48
			Combined Total	**14.0 in.**		**2.20**
		Full Depth Asphalt Option(4)	Type C or D Surface Course	2.0 in.	0.44	0.88
			Type B Asphaltic Base	4.0 in.	0.38	1.52
			Combined Total	**6.0 in.**		**2.40**
		Mechanically Stabilized Layer Option	Type C or D Surface Course	2.0 in.	0.44	0.88
			Mechanically Stabilized Layer(3)	7.0 in.	0.17	1.19
			Low PI Subgrade Material(2)	6.0 in.	0.00	0.00
			Combined Total	**15.0 in.**		**2.07**
		Rigid Pavement Option 1	Concrete	5.0 in.	-	-
			Treated Subgrade(2)	6.0 in.		
			Combined Total	**11.0 in.**		
		Rigid Pavement Option 2	Concrete	5.0 in.	-	-
			HMA Bond Breaker	1.0 in.		
			Cement Treated Base	6.0 in.		
			Low PI Subgrade Material(2)	6.0 in.		
			Combined Total	**18.0 in.**		
Local A with Bus Traffic	Soil (CBR=5.0)	**Flexible Base Option 1**	Type C or D Surface Course	3.0 in.	0.44	1.32
			Flexible (Granular) Base	12.0 in.	0.14	1.68
			Low PI Subgrade Material(2)	6.0 in.	0.00	0.00
			Combined Total	**21.0 in.**		**3.00**
		Flexible Base Option 2	Type C or D Surface Course	3.0 in.	0.44	1.32
			Flexible (Granular) Base	6.0 in.	0.14	0.84
			Treated Subgrade(2)	10.0 in.	0.08	0.80
			Combined Total	**19.0 in.**		**2.96**
		Full Depth Asphalt Option(4)	Type C or D Surface Course	3.0 in.	0.44	1.32
			Type B Asphaltic Base	5.0 in.	0.38	1.90
			Combined Total	**8.0 in.**		**3.22**
		Mechanically Stabilized Layer Option	Type C or D Surface Course	3.0 in.	0.44	1.32
			Mechanically Stabilized Layer(3)	10.0 in.	0.17	1.70
			Low PI Subgrade Material(2)	6.0 in.		0.00
			Combined Total	**19.0 in.**		**3.02**
		Rigid Pavement Option 1	Concrete	7.5 in.	-	-
			Treated Subgrade(2)	6.0 in.		
			Combined Total	**13.5 in.**		
		Rigid Pavement Option 2	Concrete	7.0 in.	-	-
			HMA Bond Breaker	1.0 in.		
			Cement Treated Base	6.0 in.		
			Low PI Subgrade Material(2)	6.0 in.		
			Combined Total	**20.0 in.**		

(1) Alternative layer thicknesses are available and can be provided upon request.
(2) Treated subgrade or replacement with low PI material is only required in areas where the PI is greater than 20.
(3) We recommend Tensar TX-5 or TxDOT Type II geogrid.
(4) For use in areas with depth limitations, such as over culverts.

A Mechanically Stabilized Layer (MSL) is a composite layer consisting of flexible (granular) base and geogrid. Geogrid provides lateral restraint to the flexible base by confining aggregate particles within the plane of the geogrid, thereby creating a reinforced, or mechanically stabilized layer. Geogrid allows the thickness of the reinforced layer to be optimized which reduces the thickness of the required flexible base and provides a stronger, more resilient structure. It will also help reduce, but not eliminate the potential for cracking. The geogrid reinforcement should conform to TxDOT DMS – 6240 Type II.

Rigid Pavement Consideration

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

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

Isolation joints are used to separate concrete slabs from other structures or fixed objects within or abutting the paved area to offset the effects of expected differential horizontal and vertical movements. Such structures include, but are not limited to, buildings, light standard foundations, and drop inlets. Isolation joints are also used at “T” intersections to accommodate differential movement along the different axes. Isolations joints are sometimes referred to as expansion joints. However, they are rarely needed to accommodate concrete expansion so they are not typically recommended for use as regularly spaced joints.

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

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

PAVEMENT CONSTRUCTION CONSIDERATIONS

SITE PREPARATION

The roadways and all areas to support fill should be stripped of all vegetation, organic topsoil, and rootmass. Exposed subgrades should be thoroughly proofrolled in order to locate any weak, compressible zones. A fully-loaded tandem-wheeled dump truck or a similar heavily-loaded piece of construction equipment should be used for planning purposes. Proofrolling operations should be observed by the Geotechnical

Engineer or their representative to document subgrade condition and preparation. Weak or soft areas identified during proofrolling should be removed and replaced with a suitable, compacted backfill.

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

DRAINAGE CONSIDERATIONS

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

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

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

ON-SITE SOIL FILL

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

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

LIME TREATMENT OF SUBGRADE

Lime treatment of the subgrade soils, if any, should be in accordance with the TxDOT Standard Specifications, Item 260. Lime-treated subgrade soils should be compacted to a minimum of 95 percent of the maximum density at a moisture content within the range of optimum moisture content to 3 percentage points above the optimum moisture content as determined by Tex-113-E. Based on the results of the pH-Lime Series and soil-lime Atterberg limits tests, we recommend that at least 3 percent hydrated lime by weight be used to increase the pH of the subgrade clays to 12.4. Assuming a total density of 134 pcf, 3 percent of hydrated lime corresponds to approximately 18 lbs per square yard (6 in. thickness). If dry placement of lime is used during construction, an additional 1 percent of lime should be added to account for expected losses and/or inefficiencies associated with field mixing operations.

FLEXIBLE BASE COURSE

The flexible base course should be crushed limestone conforming to TxDOT Standard Specifications, Item 247, Type A, Grade 1-2. The base course should be placed in lifts with a maximum compacted thickness of 8 in. (10 inches loose) and compacted to a minimum of 95 percent of the maximum density determined by Tex-113-E at a moisture content within the range of 2 percentage points below to 2 percentage points above the optimum moisture content as determined by Tex-113-E.

PRIME COAT

A prime coat should be placed on top of a flexible base course (if used) and should be a MC-30 or AE-P conforming to the TxDOT Standard Specifications 2014, Item 300 – Asphalts, Oils or Emulsions. Prime coat application rates are typically between 0.1 to 0.3 gal/yd^2 and are generally dependent upon the absorption rate of the granular base and other environmental conditions at the time of placement.

TACK COAT

A tack coat should be placed between asphaltic concrete base and/or surface lifts and should be a PG binder with a minimum high-temperature grade of PG 58, SS-1H, CSS-1H, or EAP&T conforming to TxDOT Standard Specifications 2014, Item 300 – Asphalts, Oils or Emulsions. For construction, the application rate shall not exceed 0.1 gal/yd^2.

ASPHALTIC CONCRETE COURSES

The asphaltic concrete surface, binder, and/or base courses should conform to TxDOT Standard Specifications 2014, Item 340 – Dense Graded Hot Mix Asphalt (Method) or 341 – Dense Graded Hot-Mix Asphalt (QC/QA). Recycled asphalt pavement (RAP) should be limited to 20 percent of the total weight of the mix for Types C and D mixes, and 30 percent for Type B mixes. Higher percentages of RAP may be permissible depending on the material source. If higher percentages of RAP are desired, contact RKI for consideration. Asphalt cement grades should conform to the following table:

	Minimum PG Asphalt Cement Grade		
Street Classifications	**Surface Courses**	**Binder & Level Up Courses**	**Base Courses**
Local A	PG 64-22	PG 64-22	PG 64-22

The asphaltic concrete should be compacted on the roadway to contain between 5 to 9 percent air voids computed using the maximum theoretical specific gravity (Rice) of the mixture determined according to Test Method Tex-227-F. Pavement specimens, which shall be either cores or sections of asphaltic pavement, will be tested according to Test Method Tex-207-F. The nuclear-density gauge or other methods which correlate satisfactorily with results obtained from project roadway specimens may be used when approved by the Engineer. Unless otherwise shown on the plans, the Contractor shall be responsible for obtaining the required roadway specimens at their expense and in a manner and at locations selected by the Engineer.

It is recommended that the hot mix asphalt concrete pavement be placed with a paving machine only and not with a motor grader unless prior approval is granted by the Engineer for special circumstances.

PAVEMENT RELATED CONSIDERATIONS

Longitudinal Cracking

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

Pavements that do not have a protective barrier to reduce moisture fluctuation of the expansive clay subgrade between the exposed pavement edge and that beneath the pavement section tend to develop longitudinal cracks 1 to 4 ft from the edge of the pavement. Once these cracks develop, further degradation and weakening of the underlying granular base may occur due to water seepage through the cracks. The occurrence of these cracks can be more prevalent in the absence of lateral restraint and embankments. This problem can best be addressed by providing either a horizontal or vertical moisture barrier at the unprotected pavement edge.

At a minimum, we recommend that the curbs are constructed such that the depth of the curb extends through the entire depth of the granular base material and into the subgrade to act as a protective barrier against the infiltration of water into the granular base.

In most cases, a longitudinal crack does not immediately compromise the structural integrity of the pavement system. However, if left unattended, infiltration of surface water runoff into the crack will result in isolated saturation of the underlying base. This will result in pumping of the flexible base, which could lead to rutting, cracking, and potholes. For this reason, we recommend that cracks be immediately sealed.

Pavement Maintenance

Regular pavement maintenance is critical in maintaining pavement performance over a period of several years. All cracks that develop in asphalt pavements should be regularly sealed. Areas of moderate to severe fatigue cracking (also known as alligator cracking) should be sawcut and removed. The underlying base should be checked for contamination or loss of support and any insufficiencies fixed or removed and the entire area patched. Other typical maintenance techniques should be followed as required.

Utilities

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

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

- All backfill materials should be placed and compacted in controlled lifts appropriate for the type of backfill and the type of compaction equipment being utilized and all backfilling procedures should be tested and documented.
- Consideration should be given to wrapping free-draining bedding gravels with a geotextile fabric (similar to Mirafi 140N) to reduce the infiltration and loss of fines from backfill material into the interstitial voids in bedding materials.

Curb and Gutter

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

Construction Traffic

Construction traffic on prepared subgrade, granular base or asphalt treated base (black base) should be restricted as much as possible until the protective asphalt surface pavement is applied. Significant damage to the underlying layers resulting in weakening may occur if heavily loaded vehicles are allowed to use these areas.

CONSTRUCTION RELATED SERVICES

CONSTRUCTION MATERIALS TESTING AND OBSERVATION SERVICES

As presented in the attachment to this report, *Important Information About Your Geotechnical Engineering Report*, subsurface conditions can vary across a project site. The conditions described in this report are based on interpolations derived from a limited number of data points. Variations will be encountered during construction, and only the geotechnical design engineer will be able to determine if these conditions are different than those assumed for design.
Construction problems resulting from variations or anomalies in subsurface conditions are among the most prevalent on construction projects and often lead to delays, changes, cost overruns, and disputes. These variations and anomalies can best be addressed if the geotechnical engineer of record, RKI is retained to

perform construction observation and testing services during the construction of the project. This is because:

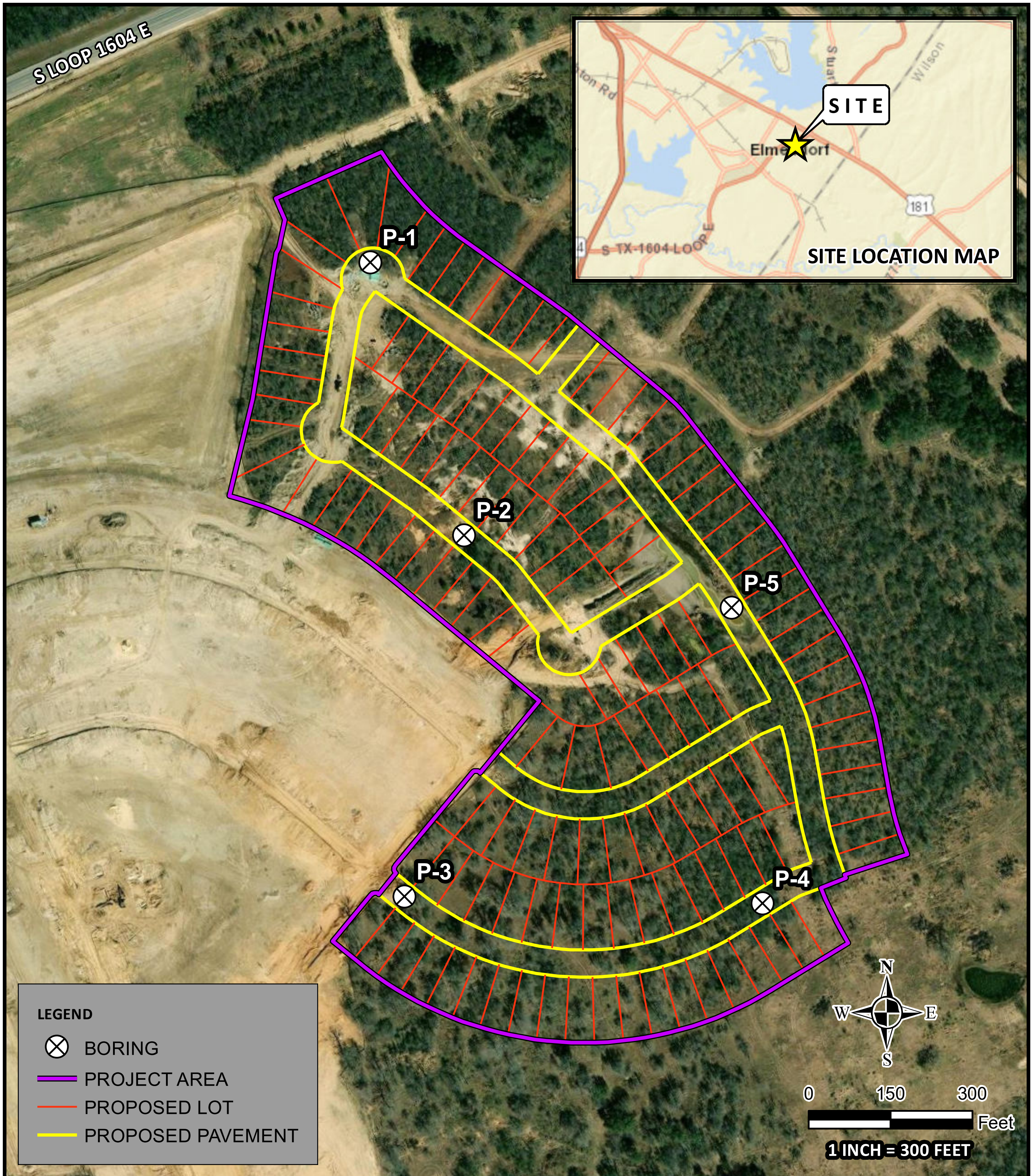
- RKI has an intimate understanding of the geotechnical engineering report's findings and recommendations. RKI understands how the report should be interpreted and can provide such interpretations on site, on the client's behalf.
- RKI knows what subsurface conditions are anticipated at the site.
- RKI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

BUDGETING FOR CONSTRUCTION TESTING

Appropriate budgets need to be developed for the required construction testing and observation activities. At the appropriate time before construction, we advise that RKI and the project designers meet and jointly develop the testing budgets, as well as review the testing specifications as it pertains to this project.
Once the construction testing budget and scope of work are finalized, we encourage a preconstruction meeting with the selected contractor to review the scope of work to make sure it is consistent with the construction means and methods proposed by the contractor. RKI looks forward to the opportunity to provide continued support on this project, and would welcome the opportunity to meet with the Project Team to develop both a scope and budget for these services.

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

ATTACHMENTS



RABA KISTNER
Building Better Tomorrows
12821 West Golden Lane
San Antonio, Texas 78249
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(210)699-6426 FAX
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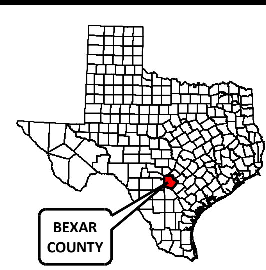
TBPE Firm Number 3257

SOURCE: Aerial photography obtained from Google Earth - 2018

BORING LOCATION MAP

Hickory Ridge – Phase 1 Unit 2
Elmendorf, Texas



PROJECT No.: ASA22-114-00

ISSUE DATE:	11/14/2022
DRAWN BY:	BM
CHECKED BY:	SS
REVIEWED BY:	IM

FIGURE

1

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

LOG OF BORING NO. P-1

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.26180; W 98.31355

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

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

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
			FAT CLAY, Stiff, Brown	14			
			LEAN CLAY, Blocky, Very Stiff, Gray and Tan, with ferric staining and gypsum deposits	15		32	
5				16			
				22			
10				21			
			FAT CLAY, Very Stiff, Gray, with ferric staining	23			
15			Boring Terminated				

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

DEPTH DRILLED: 15.0 ft	**DEPTH TO WATER:** DRY	**PROJ. No.:** ASA22-114-00	
DATE DRILLED: 11/28/2022	**DATE MEASURED:** 11/28/2022	**FIGURE:** 2	

LOG OF BORING NO. P-2

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

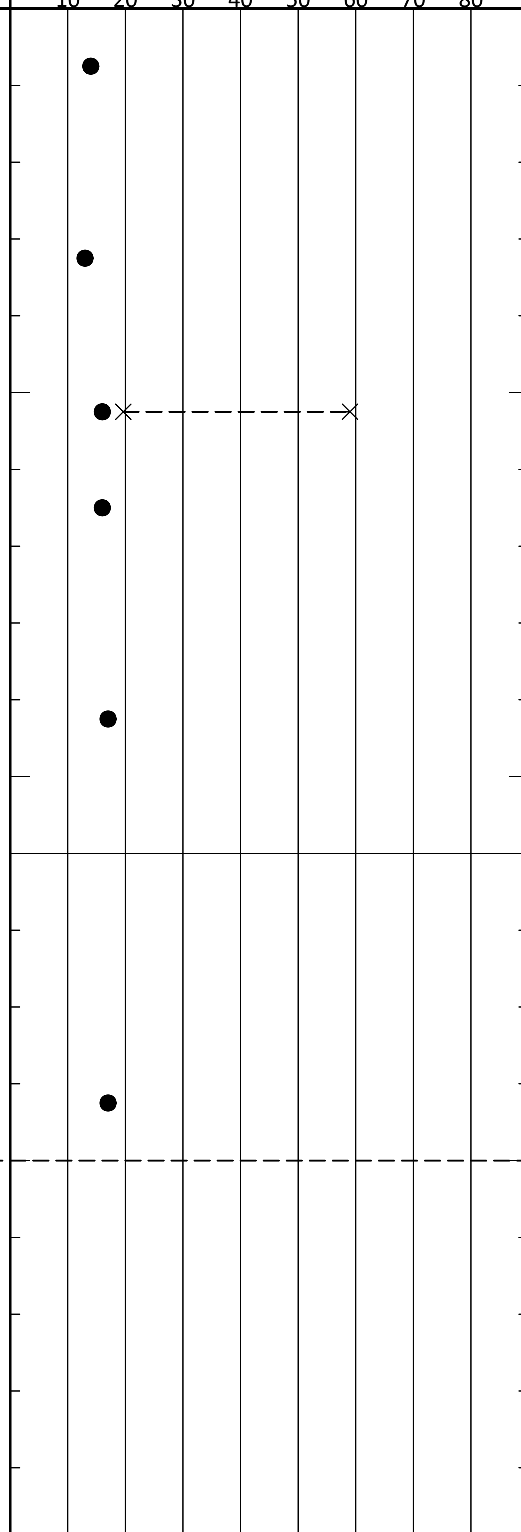
DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.26048; W 98.31281

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

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

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0–11	FAT CLAY, Very Stiff, Brownish Gray to Gray, with ferric staining and deposits	20			
		25			
5		25		39	
		27			
11–15	FAT CLAY, Hard, Dark Gray, with ferric staining	41			
15	Boring Terminated				



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

DEPTH DRILLED:	15.0 ft	**DEPTH TO WATER:**	DRY	**PROJ. No.:**	ASA22-114-00
DATE DRILLED:	11/28/2022	**DATE MEASURED:**	11/28/2022	**FIGURE:**	3

LOG OF BORING NO. P-3

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.25860; W 98.31339

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

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

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
	SAND, Clayey, Medium Dense to Dense, Reddish Brown	18			
		42		15	
5		50/9"			35
		50/11"			
		50/10"			
10					
	LEAN CLAY, Hard, Tan, with sand	50/8"			
	Boring Terminated				
15					

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

DEPTH DRILLED: 14.2 ft
DATE DRILLED: 11/28/2022
DEPTH TO WATER: DRY
DATE MEASURED: 11/28/2022
PROJ. No.: ASA22-114-00
FIGURE: 4

LOG OF BORING NO. P-4

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger

LOCATION: N 29.25833; W 98.31125

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

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

DEPTH, FT	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	PLASTICITY INDEX	% -200
0–6	LEAN CLAY, Sandy, Stiff to Hard, Dark Brown, with ferric staining	12			55
		30		23	
5		26			
6–15	LEAN CLAY, Blocky, Very Stiff, Tan and Gray, with ferric staining	15			
10		24			
		24			
15	Boring Terminated				

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

DEPTH DRILLED: 15.0 ft
DATE DRILLED: 12/6/2022
DEPTH TO WATER: DRY
DATE MEASURED: 12/6/2022
PROJ. No.: ASA22-114-00
FIGURE: 5

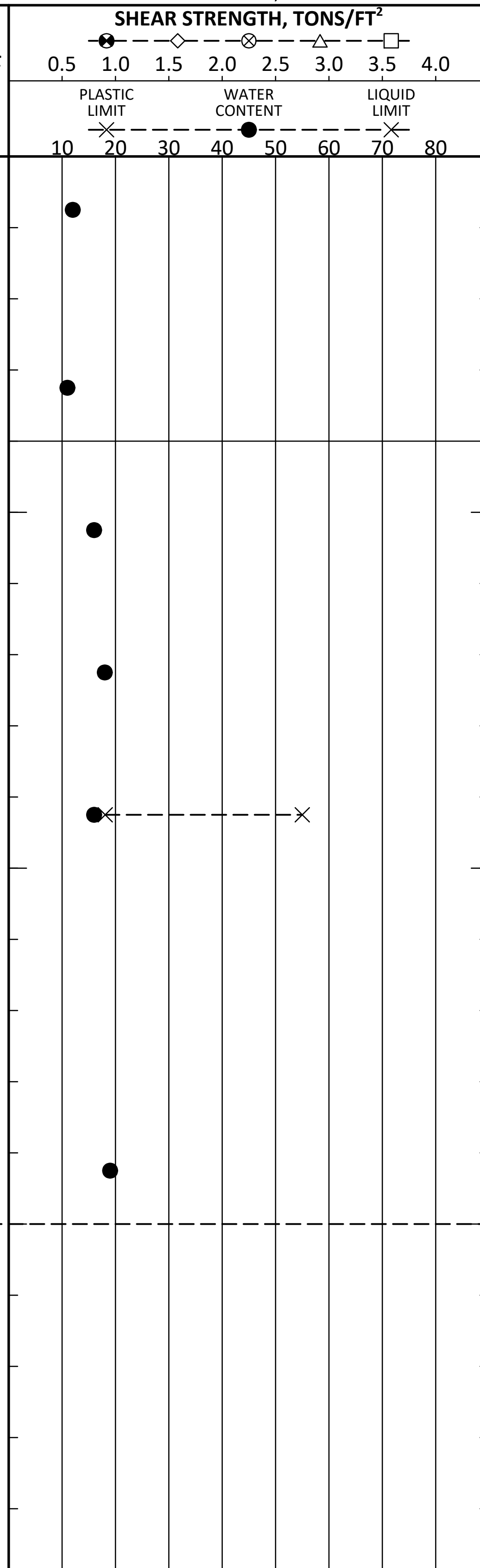
LOG OF BORING NO. P-5

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas

RABA KISTNER
TBPE Firm Registration No. F-3257

DRILLING METHOD: Straight Flight Auger **LOCATION:** N 29.25986; W 98.31137

DEPTH, FT	SYMBOL	SAMPLES	DESCRIPTION OF MATERIAL	BLOWS PER FT	UNIT DRY WEIGHT, pcf	SHEAR STRENGTH, TONS/FT² / PLASTIC LIMIT – WATER CONTENT – LIQUID LIMIT	PLASTICITY INDEX	% -200
0–4			FAT CLAY, Stiff to Very Stiff, Reddish Tan, with sand	14				
				24				
4–15			FAT CLAY, Blocky, Very Stiff to Hard, Tan and Gray, with gypsum deposits, ferric staining, and yellow stains	19				
				26				
				28			37	
				30				
15			Boring Terminated					



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

DEPTH DRILLED: 15.0 ft **DEPTH TO WATER:** DRY **PROJ. No.:** ASA22-114-00
DATE DRILLED: 12/6/2022 **DATE MEASURED:** 12/6/2022 **FIGURE:** 6

KEY TO TERMS AND SYMBOLS

MATERIAL TYPES

SOIL TERMS

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

ROCK TERMS

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

OTHER

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

WELL CONSTRUCTION AND PLUGGING MATERIALS

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

SAMPLE TYPES

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

STRENGTH TEST TYPES

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

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

PROJECT NO. ASA22-114-00

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

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

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

RELATIVE DENSITY

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

COHESIVE STRENGTH

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

PLASTICITY

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

ABBREVIATIONS

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

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

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

KEY TO TERMS AND SYMBOLS (CONT'D)

TERMINOLOGY

SOIL STRUCTURE

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

SAMPLING METHODS

RELATIVELY UNDISTURBED SAMPLING

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

STANDARD PENETRATION TEST (SPT)

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

SPLIT-BARREL SAMPLER DRIVING RECORD

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

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

RESULTS OF SOIL SAMPLE ANALYSES

PROJECT NAME: Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas

FILE NAME: ASA22-114-00 HICKORY RIDGE SUBDIVISION PH 1, U2.GPJ 1/5/2023

Boring No.	Sample Depth (ft)	Blows per ft	Water Content (%)	Liquid Limit	Plastic Limit	Plasticity Index	USCS	Dry Unit Weight (pcf)	% -200 Sieve	Shear Strength (tsf)	Strength Test
P-1	0.0 to 1.5	14	8								
	2.5 to 4.0	15	14	48	16	32	CL				
	4.0 to 4.5										
	4.5 to 6.0	16	19								
	6.5 to 8.0	22	16								
	8.5 to 10.0	21	21								
	13.5 to 15.0	23	18								
P-2	0.0 to 1.5	20	14								
	2.5 to 4.0	25	13								
	4.5 to 6.0	25	16	59	20	39	CH				
	6.5		16								
	8.5 to 10.0	27	17								
	13.5 to 15.0	41	17								
P-3	0.0 to 1.5	18	6								
	2.0 to 2.5										
	2.5 to 4.0	42	7	29	14	15	CL				
	4.5 to 5.8	50/9"	5						35		
	6.5 to 7.9	50/11"	7								
	8.5 to 9.8	50/10"	5								
	13.5 to 14.7	50/8"	4								
P-4	0.0 to 1.5	12	17						55		
	2.5 to 4.0	30	5	35	12	23	CL				
	4.5 to 6.0	26	9								
	6.5 to 8.0	15	15								
	8.5 to 10.0	24	18								
	13.5 to 15.0	24	17								
P-5	0.0 to 1.5	14	12								
	2.5 to 4.0	24	11								
	4.5 to 6.0	19	16								
	6.5 to 8.0	26	18								
	8.5 to 10.0	28	16	55	18	37	CH				
	13.5 to 15.0	30	19								

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

CU = Consolidated Undrained Triaxial

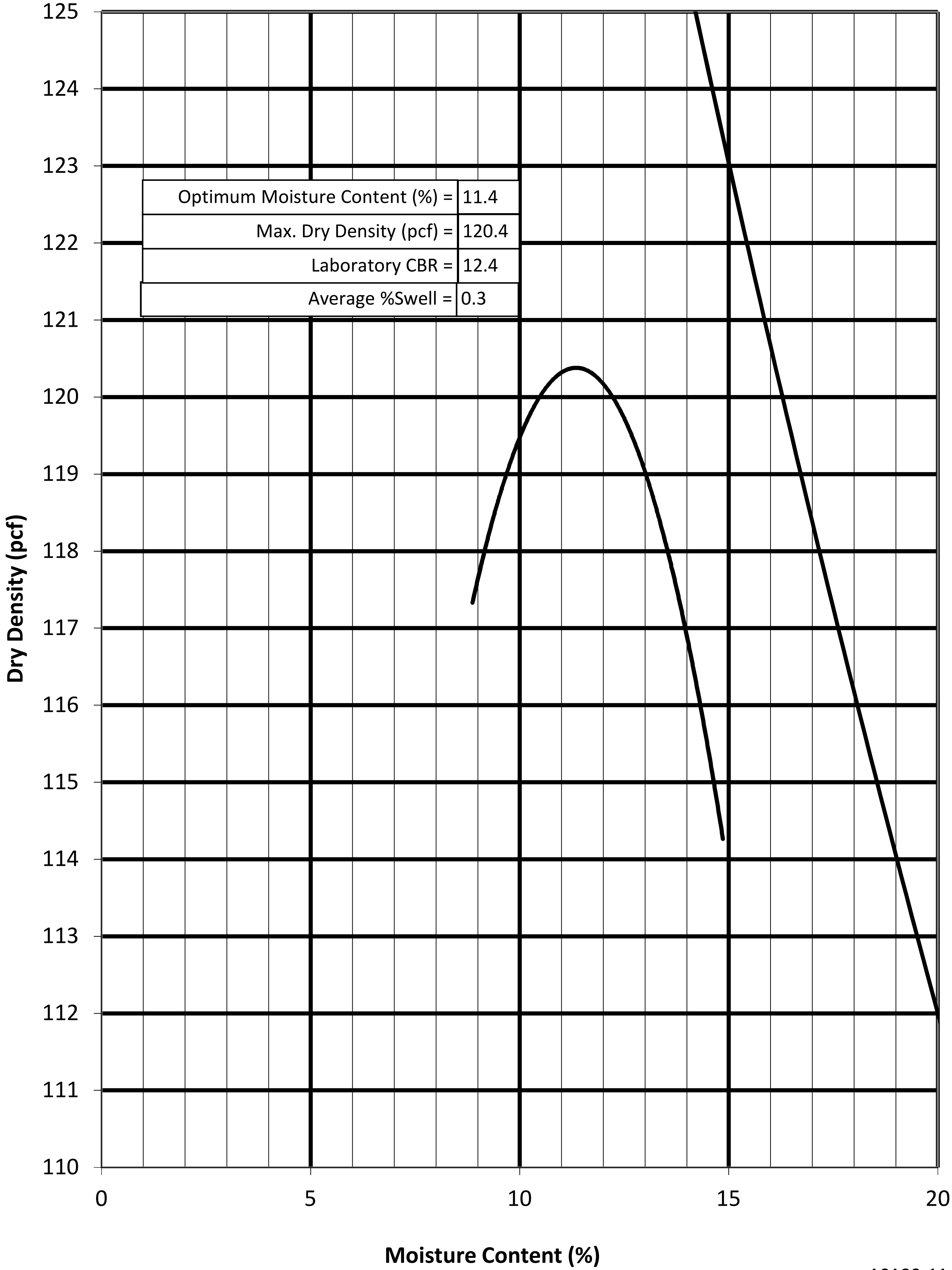
PROJECT NO. ASA22-114-00

RABAKISTNER

MOISTURE DENSITY RELATIONSHIP CURVE (ASTM D698)

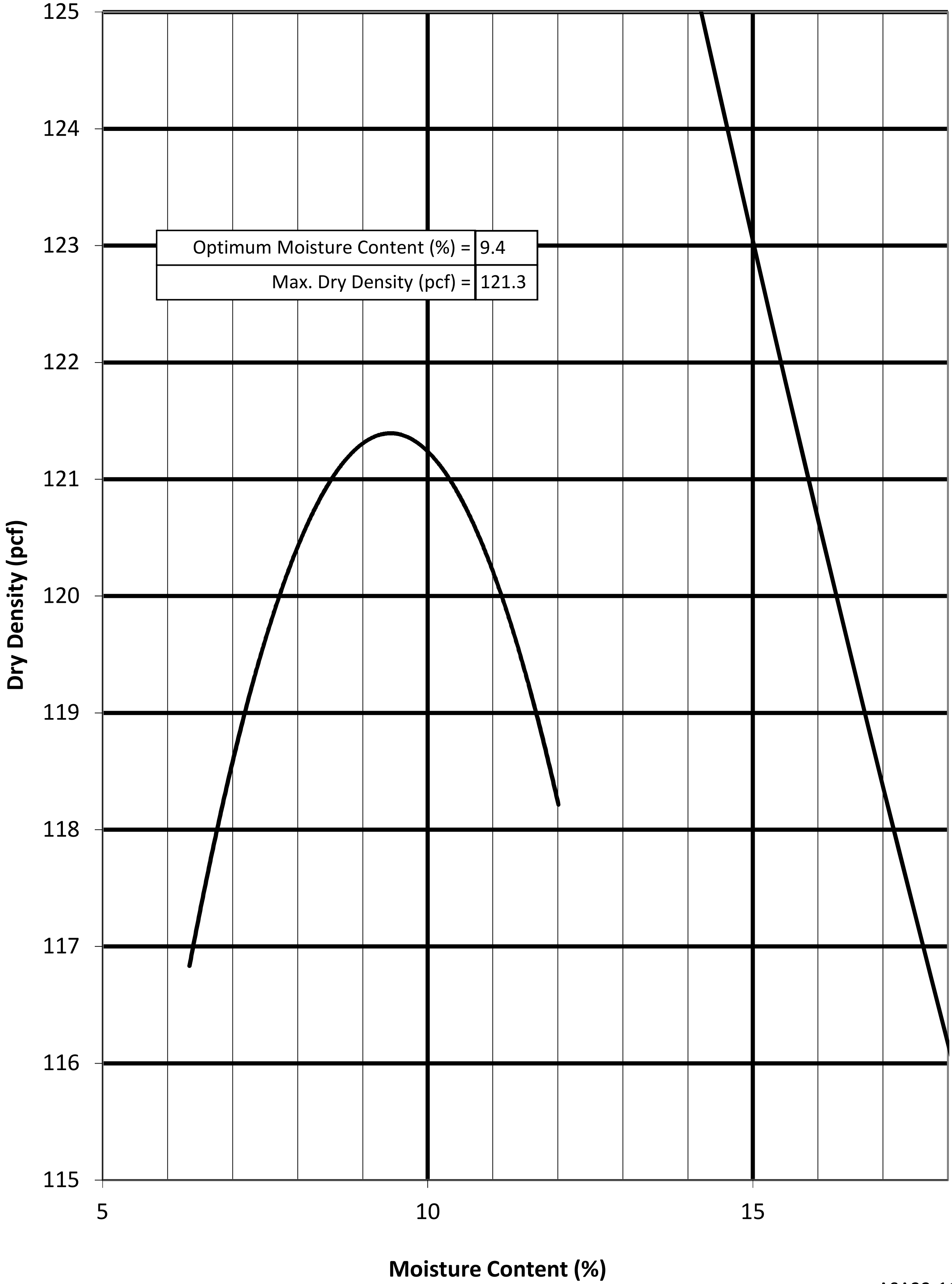
Hickory Ridge - Phase 1 Unit 2

Elmendorf, Texas



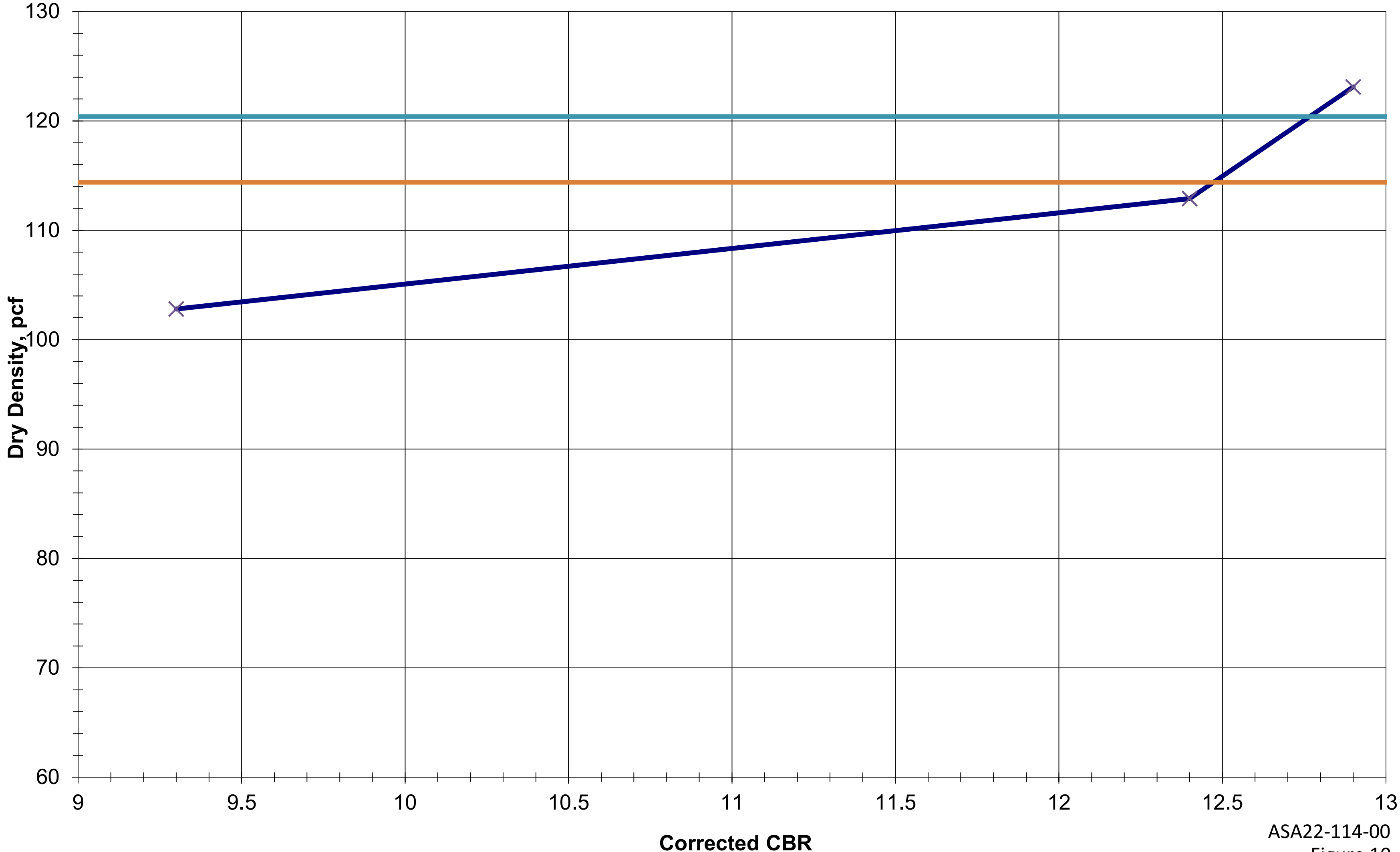
MOISTURE DENSITY RELATIONSHIP CURVE (Tex-113-E) @ 3% LIME

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas



CORRECTED CBR VS. DRY DENSITY

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas

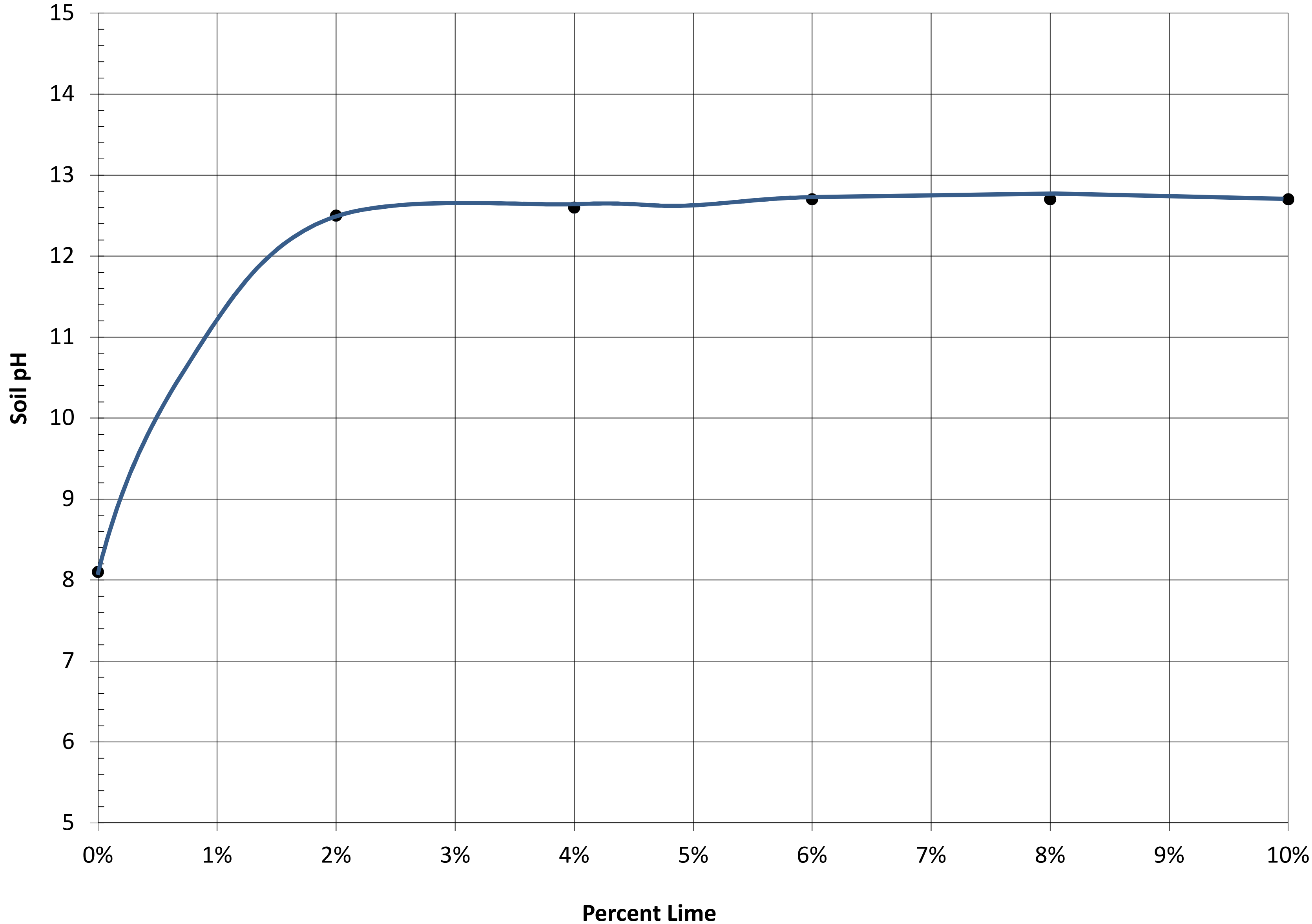


ASA22-114-00
Figure 10

1/5/2023

pH-LIME SERIES CURVE

Hickory Ridge - Phase 1 Unit 2
Elmendorf, Texas



Important Information about This

Geotechnical-Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are Not Final

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

A Geotechnical-Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Environmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

GBC GEOTECHNICAL BUSINESS COUNCIL
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