

GEOTECHNICAL FEASIBILITY EVALUATION

Flying W Ranch New Braunfels, Texas

Report For:

Continental Homes of Texas, LP - San Marcos 1306 East McCarty Lane, Suite 100 San Marcos, Texas 78666

April 2024

Engineer's Job # 24201102.004

MLA Geotechnical TBPE FIRM # F-2684

Geotechnical Engineering and Construction Materials Testing "put us to the test"

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GEOTECHNICAL FEASIBILITY EVALUATION

Flying W Ranch New Braunfels, Texas

BACKGROUND

This report presents the results of a soil exploration and analysis for the proposed *Flying W Ranch Feasibility* project in New Braunfels, Texas. Authorization to perform this exploration and analysis was by Agreement for Engineering Services with Mr. Devin Lee Kleinfelder of Continental Homes of Texas, LP - San Marcos.

The purpose of this study was to evaluate the feasibility in regard to the underlying soils for constructing single-family residential foundations for structures up to 3 stories in height. Sources of information included publicly available geologic maps, USGS topography maps and firm experience in the area ^(1,2). Thirteen exploratory borings were placed across the site. Borings were located using standard field locating techniques. Please refer to *Appendix A* for the location of the site and boring layout.

SUBSURFACE CONDITIONS AND GEOLOGY

Natural Soil Profiles

The native soil profile encountered in the borings generally consists of a surface layer of brown to pale brown low plasticity clay (CL) that varies in color to light yellowish brown with depth. The surface soils are underlain by limestone in many of the borings. However, some boring lack limestone and revealed yellowish brown high plasticity clay (CH) at depth. A single boring revealed a surface layer of dark brown high plasticity clay (CH). Flying W Ranch Engineer's Job No.: 24201102.004

Geologic Profile

Geologic maps indicate outcrops of the Person Formation, Kp, and the Austin Group, Kau, on this site and the Eagle Ford Formation, Kef, was identified in the borings at depth ^(1,2).

The Person Formation is from the Lower Cretaceous period and is a significant member of the Edwards Group. In typical geologic sequence the Person is overlain by the Georgetown Formation, *Kgt*, and underlain by the Kainer Formation, *Kk*, the later being a member of the Edwards Group. This formation outcrops along the Balcones Fault zone in a band stretching from San Antonio up into the Austin area where it is mapped collectively with the Edwards Formation. The Person is predominately a limestone formation but has zones of dolomitic limestone and argillaceous (clay containing) limestone with cherty zones being common. It was deposited cyclically in a shallow marine environment. It displays the characteristic karst honey-combing common to the limestones of the Edwards Plateau. Pockets of red clay commonly infill these karst features and collect along fractures and bedding planes. Fossils include pelecypodes, gastropods, and rudistids. Reference material generally equates this formation with the Segovia Formation of the eastern Edwards Plateau. The approximate thickness of the Person ranges from 130 feet to 150 feet.

The Austin Group is overlain by the Taylor Group and underlain by the Eagle Ford Formation (of the Woodbine Group). These groups are all from the Gulf Series. The Austin Group is composed of seven fairly distinct member formations but can be generally described as follows. The Austin is composed of alternating, thick to thin beds of chalky limestone and calcareous clays. The near surface weathered limestone and marls are white and light yellowish brown, respectively. Reddish brown streaks caused by pyrite inclusions are often encountered on freshly exposed chalky limestone surfaces. Both limestone and marl are dark gray to very dark gray in their unweathered state usually found at great depths. Fossils such as *Inocerami* are often on freshly exposed surfaces. Full sections may range up to 420 feet in thickness although specific locations may be significantly different. Some parts of the Austin Group are stable hard rock, other parts are made up of calcareous clays that can shrink and swell.

The Eagle Ford Formation is a member of the Lower Gulf Series and was deposited during Upper Cretaceous Period. The Eagle Ford is overlain by the Austin Group and underlain by the Buda Limestone and ranges up to 50 feet in thickness. The Eagle Ford is a geologically complex formation and often weathers into three strata. The upper stratum is characterized by a zone of low plasticity clay with light reddish brown silt/fine sand seams with very thin light gray clay layers. This stratum contains layers of impure limestone/sandstone called "flagstone" that range up to 5 inches in thickness. Firm experience has shown this flag zone to be more dense and thicker in southern Central Texas. The middle stratum is characterized by brownish yellow and dark yellowish brown, mottled, high plasticity clay with dark red silt/fine sand seams. The lowest stratum of the Eagle Ford is a very dark gray to black claystone or shale that is interbedded with layers of hard flagstone and is the least weathered section of the Eagle Ford. It is highly variable: some portions are a well-cemented medium gray sandstone/limestone while other portions are stiff to hard clay/shale. The shear strengths of these two materials vary dramatically. This lowest zone also has thin layers of light gray bentonite that have a high shrink/swell potential.

The Eagle Ford is well known as a troublesome formation having high shrink/swell properties that are not always borne out in standard laboratory index tests. Residences constructed over this formation often undergo differential movements that lead to serious detrimental cracking. This characteristic of the Eagle Ford formation prompts special attention to be given to the design of foundations.

Faults

Published geology maps do indicate the presence of faults and suspected faults on the project site. (The geologic map of this site depicts faults between geologic formations with solid black lines and suspected faults with dashed lines). While the faults in Central Texas are not seismically active, their presence can be indicative of variability in the soil profiles at the site.

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Ground Water

Ground water was not encountered in the borings during the field investigation. However, ground water may be encountered at other portions of the site and at different times of the year, especially at lower elevations near the existing pond. The presence of ground water is seasonal and random, depending on the amount of preceding rainfall, weather patterns, and changes in land use.

TOPOGRAPHY, DRAINAGE AND VEGETATION

The site is situated on variably sloping topography with typical slopes varying from approximately 2 percent to 8 percent with isolated slopes of approximately 12 percent. Regionally, the tract drains to the northeast. The predeveloped vegetation on this site consists of mature trees and grasses.

DISCUSSION

Foundation Design

1. Slab on ground foundations are expected to be financially feasible for this site. The estimated design parameters will vary according to the character and thickness of the surface clays as well as the relative variation in soil profile across borings. Approximate Equivalent B.R.A.B. #33 PI values and PTI Parameters are presented in the following table for use as a guide for cost estimating. These values will be replaced by the final foundation design parameters derived from the more detailed investigation required for actual foundation design recommendations.

		P.T.I. Pa	arameters ⁽³⁾		B.R.A.B. #33 Parameter ⁽⁴⁾	PVR/ PVM ⁽⁵⁾
Boring	Em- center, ft	Em- edge, ft	Ym- Ym-edge, čt center, in in		Estimated Equivalent P.I.	inches
B-1	7.5	3.5	1.1	1.4	38	< 1
B-2	7.5	3.5	1.1	1.4	38	< 1
B-3	7.5	3.5	1.6	2.0	52	2 1/4
B-4	7.5	3.5	1.4	1.8	45	< 1
B-5	7.5	3.5	1.6	2.0	52	< 1
B-6	7.5	3.5	1.4	1.8	45	< 1
B-7	7.5	3.5	1.1	1.4	38	< 1
B-8	7.5	3.5	1.1	1.4	38	< 1
B-9	7.5	3.5	1.1	1.4	38	< 1
B-10	7.5	3.5	1.6	2.0	52	1
B-11	7.5	3.5	1.6	2.0	52	1 1/4
B-12	7.5	3.5	1.1	1.4	38	< 1
B-13	7.5	3.5	1.6	2.0	52	2 1/2

Table 1: Expected Foundation Design Parameters

2. The soils at this project site exhibited plasticity indices ranging from 7 to 45. Point estimates of the potential vertical rise, PVR, and potential vertical movement, PVM, are shown in Table 1. PVR was calculated using the TxDOT Method and is widely used and

should be considered an index property of the site. Other magnitudes of PVR/PVM may be estimated by other methods and at other locations with varying results.

- 3. We expect cuts and fills will be necessary to develop this tract for residential use. All grading should be monitored for proper material selection and compaction. Significant cuts where the shallow limestone is removed could expose the Eagle Ford Formation at depth resulting in a higher design PI due to the risks associated with expansive soils. Lots on which significant fill is placed may be subject to settlement. Even properly compacted fill may be subject to potential settlement. As such, it is expected that some lots will require design parameters higher than those shown in the table above to account for the potential settlement.
- 4. We do not anticipate the need for moisture conditioning or injection for the majority subdivision. However, if the Eagle Ford is encountered near the surface, it is possible those areas may need moisture conditioning. Moisture injection is not expected to be feasible on any portion of this tract.
- 5. It is recommended that all foundations be designed by competent foundation engineers using generally accepted foundation procedures.

Construction

- 1. Excavation may be performed using ordinary power equipment for the construction of slabon-ground foundations at this site.
- Areas of limited extent used for rural residential garbage disposal or incineration were not encountered during this feasibility study but have been encountered on similar sites. Such areas could be called the "rancher's landfill" and maybe encountered on this site.
- 3. Ground water was not encountered in the borings during the field investigation. However, ground water may be encountered at other portions of the site and at different times of the year, especially at lower elevations. Trench drains may be required to address ground water issues and should be considered in preliminary design and cost estimating.

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4. Pavement sections at this site will require base course possibly with a lime stabilized subgrade. For example, local streets will likely require 2 to 3 inches of HMAC and 8 to 10 inches of crushed limestone base course with the possibility of a layer of lime stabilized subgrade. A geotechnical pavement study will be required to develop pavement thickness sections.

REFERENCES

- Local geologic maps published by The Bureau of Economic Geology. Austin, Texas including: "Geologic Atlas of Texas" 15-minute quadrangles. March 9, 2004 geospatial data. "Geologic Map of the Austin Area, Texas 1992" Geology of Austin Area Plate VII. "Geologic Map of the West Half of Taylor Texas, 30 x 60 min quad." 2005. misc. map 43 "Geologic Map of the New Braunfels, Texas 30 x 60 min quad." 2000. misc. map 39
- 2. "The Geology of Texas, Volume I, Stratigraphy", The University of Texas Bulletin No. 3232: August 22, 1932, The University of Texas, Austin, Texas, 1981.
- 3. "Design of Post-Tensioned Slabs-on-Ground", Third Edition. Post-Tensioning Institute, Farmington Hills, MI, 2008.
- 4. "Criteria for Selection and Design of Residential Slabs-on-Ground", Building Research Advisory Board, National Research Council of the National Academy of Sciences, Report #33, 1968.
- 5. "Method for Determining Potential Vertical Rise, PVR, Test Method Tex-124-E", Manual of Testing Procedures, Texas Department of Transportation Materials and Tests Division, September 1995.

LIMITATIONS OF REPORT

Conditions of the site at locations other than the boring locations are not expressed or implied, and conditions may be different at different times from the time of drilling. Contractors or others desiring more complete information are advised to secure their own supplemental borings. This investigation and report, do not, and are not intended to determine the environmental conditions or evaluate possible hazardous or toxic waste conditions on this site or adjacent sites. Interested persons requiring this information are advised to contact MLA Geotechnical. The analysis and recommendations contained herein are based on the available data as shown in this report and the writer's professional expertise, experience and training, and no other warranty is expressed or implied concerning the satisfactory use of these recommendations or data.

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APPENDIX A

GEOTECHNICAL DATA



Approximate location of site in yellow

NAPP Aerial Photograph of Site – 1995

Source: TEXAS NATURAL RESOURCES INFORMATION SYSTEM 3.75-minute DOQQ. 1-meter ground resolution. apx. date 1995-6 (http://www.tnris.state.tx.us/digital.htm)





Approximate location of site in yellow. 5-foot contours in orange (2021)

NAPP Aerial Photograph of Site – 1995

Source: TEXAS NATURAL RESOURCES INFORMATION SYSTEM 3.75-minute DOQQ. 1-meter ground resolution. apx. date 1995-6 (http://www.tnris.state.tx.us/digital.htm)





Approximate location of site in yellow

Aerial Photograph of Site – 2022

Source: TEXAS NATURAL RESOURCES INFORMATION SYSTEM Apx. Date - 2022 (www.tnris.org)





Approximate location of site in yellow. 5-foot contours in orange (2021)

Aerial Photograph of Site – 2022

Source: TEXAS NATURAL RESOURCES INFORMATION SYSTEM Apx. Date - 2022 (www.tnris.org)





Approximate location of site in blue

U.S. 7.5 Minute Series Topographic Map Hunter Quadrangle, Texas Contour Interval = 10 feet Source: TEXAS NATURAL RESOURCES INFORMATION SYSTEM (http://www.tnris.state.tx.us/digital.htm)





Approximate location of site in yellow

Geologic Setting of Site Geologic Map of the New Braunfels Quad

Original Source: Geologic Map of the New Braunfels, Texas 30 x 60 min quad" 2000. misc. map 39. Source: The Bureau of Economic Geology. Austin, Texas







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	Termination Depth: 15.0 feet	Kef						

SOIL CLASSIFICATION CHART

M		IONS	SYM	BOLS	TYPICAL	
			GRAPH	LETTER	DESCRIPTIONS	
	GRAVEL AND	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES	
	GRAVELLY SOILS	(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES	
COARSE GRAINED SOILS	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES	
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES	
MORE THAN 50% OF MATERIAL IS	SAND AND	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	
LARGER THAN NO. 200 SIEVE SIZE	SANDY SOILS	(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES	
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES	
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES	
FINE GRAINED		LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY	
SOILS	SILTS AND CLAYS			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
UILL	CLAYS	LIQUID LIMIT GREATER THAN 50		СН	INORGANIC CLAYS OF HIGH PLASTICITY	
SOILS C	OF MODERATE PL	ASTICITY		CL-CH	LOW PI CLAYS WITH APPRECIABLE HIGH PI MOTTLING, CLAY WITH BORDERLINE CLASSIFICATION	
	OTHER MATERIAI	_S		FILL	MATERIAL NOT NATURALLY DEPOSITED	
		HHH	Ie	WEATHERED LIMESTONE		
1778-0-111-0-11-1-1-1-1-1-1-1-1-1-1-1-1-1-1	MMMM			LJ		

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Key to Terms and Abbreviations

Descriptive Terms Characterizing Soils and Rock	Standard Description	Symbols and
	Abbreviations and	Abbreviations for
	Terms	Test Data
Argillaceous – having appreciable amounts of clay in the	brn = brown	LL = Liquid Limit
soil or rock mass. Used most often in describing	dk = dark	PL = Plastic Limit
limestones, occasionally sandstones.	lt = light	PI = Plasticity Index
Calcareous – containing appreciable quantities of calcium	wx = weathered	(LL-PL)
carbonate. Can be either nodular or "powder."	calc = calcareous	NP = non-plastic
Crumbly – cohesive soils which break into small blocks or	sw = severely weathered	$\gamma_d = dry unit weight$
crumbs on drying.	cw = completely	$q_u = unconfined$
Evaporite – deposits of salts and other soluble compounds.	weathered	compressive
Most commonly calcium carbonate or gypsum. May be	n/a = not available	strength
in either "powder" or visible crystal form.	b. = below	$q_c = confined$
Ferruginous – having deposits of iron or nodules, typically		compressive
oxidized and dark red in color.		strength
Ferrous – see Ferruginous	Engineering Units	SPT = standard
Fissured – containing shrinkage cracks frequently filled	pcf = pounds per cubic	penetration test
with fine sand or silt, usually more or less vertical.	foot	TCP = Texas cone
Fossiliferous – containing appreciable quantities of fossils,	psf = pounds per square	penetration test
fossil fragments, or traces of fossils	foot	(Texas Highway
Laminated – composed of thin layers of varying color or	tsf = tons per square foot	Department)
texture. Layers are typically distinct and varying in	pF = picofarad	N or $N_{SPT} =$ blows per
composition from sand to silt and clay.	psi = pounds per square	foot from SPT
Mottled – characterized as having multiple colors organized	inch	$N_{TCP} =$ blows per foot
in a marbled pattern.	kips = thousand pounds	from TCP
Slickensided – having inclined planes of weakness that are	(force)	SCR = standard core
slick and glossy in appearance.	ksf = kips per square	recovery
Varved – see Laminated.	foot	RQD = rock quality
		designation
		RQI = see RQD

Terms Describing Consistency of Soil and Rock

COARSE GRAI	NED MATERIAL	SEDIMENTARY ROCK			
DESCRIPTIVE	BLOWS/FT (SPT)	DESCRIPTIVE	STRENGTH, TSF		
TERM		TERM			
very loose	0 - 4	soft	4 - 8		
loose	4 - 10	medium	8-15		
firm (medium)	10-30	hard	15 - 50		
dense	30 - 50	very hard	over 50		
very dense	over 50				

Describing Consistency of Fine Grained Soil

DESCRIPTIVE	BLOWS/FT (SPT)	UNCONFINED COMPRESSION, TSF
TERM		
very soft	< 2	< 0.25
soft	2-4	0.25 - 0.50
medium stiff	4 - 8	0.50 - 1.00
stiff	8-15	1.00 - 2.00
very stiff	15 - 30	2.00 - 4.00
hard	over 30	over 4.00

Auger CuttingsShelby TubeSplit Spoon
(SPT)Texas Cone
(TCP)Rock Core

No Sample

Revised: October 2018

Sample Type Key

APPENDIX B

STANDARD FIELD AND LABORATORY PROCEDURES

STANDARD FIELD AND LABORATORY PROCEDURES

STANDARD FIELD PROCEDURES

Drilling and Sampling

Borings and test pits are typically staked in the field by the drillers, using simple taping or pacing procedures and locations are assumed to be accurate to within several feet. Unless noted otherwise, ground surface elevations (GSE) when shown on logs are estimated from topographic maps and are assumed to be accurate to within a foot. A Plan of Borings or Plan of Test Pits showing the boring locations and the proposed structures is provided in the Appendix.

A log of each boring or pit is prepared as drilling and sampling progressed. In the laboratory, the driller's classification and description is reviewed by a Geotechnical Engineer. Individual logs of each boring or pit are provided in the Appendix. Descriptive terms and symbols used on the logs are in accordance with the Unified Soil Classification System (ASTM D-2487). A reference key is also provided. The stratification of the subsurface material represents the soil conditions at the actual boring locations, and variations may occur between borings. Lines of demarcation represent the approximate boundary between the different material types, but the transition may be gradual.

A truck-mounted rotary drill rig utilizing rotary wash drilling or continuous flight hollow or solid stem auger procedures is used to advance the borings, unless otherwise noted. A backhoe provided by others is used to place test pits. Test pits are advanced to the required depth, refusal (typically bedrock) or to the limits of the equipment. Samples of soil are obtained from the borings or test pit spoils for subsequent laboratory study. Samples are sealed in plastic bags and marked as to depth and boring/pit locations in the field. Cores are wrapped in a polyethylene wrap to preserve field moisture conditions, placed in core boxes and marked as to depth and core runs. Unless notified to the contrary, samples and cores will be stored for 90 days, then discarded.

Standard Penetration Test and Split-Barrel Sampling of Soils (ASTM D-1586) (SPT)

This sampling method consists of driving a 2 inch outside diameter split barrel sampler using a 140 pound hammer freely falling through a distance of 30 inches. The sampler is first seated 6 inches into the material to be sampled and then driven an additional 12 inches. The number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance. The results of the SPT is recorded on the boring logs as "N" values.

Thin-Walled Tube Sampling of Soils (ASTM D-1587) (Shelby Tube Sampling)

This method consists of pushing thin walled steel tubes, usually 3 inches in diameter, into the soils to be sampled using hydraulic pressure or other means. Cohesive soils are usually sampled in this manner and relatively undisturbed samples are recovered.

B-1

Soil Investigation and Sampling by Auger Borings (ASTM D-1452)

This method consists of auguring a hole and removing representative soil samples from the auger flight or bit at intervals or with each change in the substrata. Disturbed samples are obtained and this method is, therefore, limited to situations where it is satisfactory to determine the approximate subsurface profile and obtain samples suitable for Index Property testing.

Diamond Core Drilling for Site Investigation (ASTM D-2113)

This method consists of advancing a hole into hard strata by rotating a single or double tube core barrel equipped with a cutting bit. Diamond, tungsten carbide, or other cutting agents may be used for the bit. Wash water or air is used to remove the cuttings and to cool the bit. Normally, a 3 inch outside diameter by 2-1/8 inch inside diameter coring bit is used unless otherwise noted. The rock or hard material recovered within the core barrel is examined in the field and in the laboratory and the cores are stored in partitioned boxes. The intactness of all rock core specimens is evaluated in two ways. The first method is the Standard Core Recovery (SCR) expressed as the length of the total core recovered divided by the length of the core run, expressed as a percentage:

> SCR =<u>total core length recovered</u> x 100% length of core run

This value is exhibited on the boring logs as the Standard Core Recovery (SCR).

The second procedure for evaluating the intactness of the rock cores is by Rock Quality Designation (RQD). The RQD provides an additional qualitative measure of soundness of the rock. This index is determined by measuring the intact recovered core unit which exceed four inches in length divided by the total length of the core run:

 $RQD = \frac{all core lengths greater than 4"}{length of core run} \times 100\%$

The RQD is also expressed as a percentage and is shown on the boring logs.

Vane Shear Tests

In-situ vane shear tests may be used to determine the shear strength of soft to medium cohesive soil. This test consists of placing a four-bladed vane in the undisturbed soil and determining the torsional force applied at the ground surface required to cause the cylindrical perimeter surface of the vane to be sheared. The torsional force sufficient to cause shearing is converted to a unit of shearing resistance or cohesion of the soil surrounding the cylindrical surface.

THD Cone Penetrometer Test

The THD Cone Penetrometer Test is a standard field test to determine the relative density or consistency and load carrying capacity of foundation soils. This test is performed in much the same manner as the Standard Penetration Test described above. In this test, a 3 inch diameter penetrometer cone is used in place of a split-spoon sampler. This test calls for a 170-pound weight falling 24 inches. The actual test in hard materials consists of driving the penetrometer cone and accurately recording the inches of penetration for the first and second 50 blows for a total of 100 blows. These results are then correlated using a table of load capacity vs. number of inches penetrated per 100 blows.

Pocket Penetrometer Test

A pocket penetrometer or hand penetrometer is a small device used to estimate the shear capacity or unconfined compressive strength of a soil sample. The device consists of a spring-loaded probe which measures the pressure required to penetrate the probe into a soil sample for specified depth. This test can only be performed on cohesive soil samples. This pressure is reported in tons per square foot (tsf) on the Logs of Boring. A hyphen (-) indicates that the soil sample was too loose or too soft to perform the test. This test is considered rudimentary and too inaccurate to be used for direct design parameters; however, this test is useful for correlations among soil strata and general stiffness descriptions.

Ground Water Observation

Ground moisture observations are made during the operations and are reported on the logs of boring or pit. Moisture condition of cuttings are noted, however, the use of water for circulation precludes direct observation of wet conditions. Water levels after completing the borings or pits are noted. Seasonal variations, temperatures and recent rainfall conditions may influence the levels of the ground water table and water may be present in excavations, even though not indicated on the logs.

STANDARD LABORATORY PROCEDURES

To adequately characterize the subsurface material at this site, some or all of the following laboratory tests are performed. The results of the actual tests performed are shown graphically on the Logs of Boring or Pit.

Moisture Content - ASTM D-2216

Natural moisture contents of the samples (based on dry weight of soil) are determined for selected samples at depths shown on the respective boring logs. These moisture contents are useful in delineating the depth of the zone of moisture change and as a gauge of correlation between the various index properties and the engineering properties of the soil. For example, the relationship between the plasticity index and moisture content is a source of information for the correlation of shear strength data.

Dry Density - ASTM D-7263

The dry density, γ_d , (bulk density or unit weight) of the samples is determined for selected samples at depths shown on the respective boring logs using Method B of the aforementioned ASTM standard. The in-situ density was determined from undisturbed SPT samples and the dry density was calculated using moisture content results. These dry density values are useful for calculating other characteristic values such as porosity, void ratio, and mass composition of soil. Additionally, these values can also be used to assess the degree of compaction or consolidation of fill materials.

Atterberg Limits - ASTM D-4318

The Atterberg Limits are the moisture contents at the time the soil meets certain arbitrarily defined tests. At the moisture content defined as the plastic limit, Pw, the soil is assumed to change from a semi-solid state to a plastic state. By the addition of more moisture, the soil may be brought up to the moisture content defined as the liquid limit, Lw, or that point where the soil changes from a plastic state to a liquid state. A soil existing at a moisture content between these two previously described states is said to be in a plastic state. The difference between the liquid limit, Lw, and the plastic limit, Pw, is termed the plasticity index, Iw. As the plasticity index increases, the ability of a soil to attract water and remain in a plastic state increases. The Atterberg Limits that were determined are plotted on the appropriate log.

The Atterberg Limits are quite useful in soil exploration as an indexing parameter. Using the Atterberg Limits and grain size analysis, A. Casagrande developed the Unified Soils Classification System (USCS) which is widely used in the geotechnical engineering field. This system related the liquid limit to the plasticity index by dividing a classification chart into various zones according to degrees of plasticity of clays and silts. Although the Atterberg Limits are an indexing parameter, K. Terzaghi has related these limits to various engineering properties of a soil. Some of these relationships are as follows:

- 1. As the grain size of the soil decreases, the Atterberg Limits increase.
- 2. As the percent clay in the soil increases, the Atterberg Limits increase.
- 3. As the shear strength increases, the Atterberg Limits decrease.
- 4. As the compressibility of a soil increases, the Atterberg Limits increase.

Free Swell Test - ASTM D-4546-96

The free swell test assesses the potential for swell of soil. This value is useful for the design of various structures such as slab-on-ground foundations, piers and piles, and underground utilities. Method B of the aforementioned ASTM standard determines the amount of swell (vertical heave) of a sample. This is done by placing the sample in a consolidometer under a seating load equal to the overburden pressure and giving the sample free access to water. The height is measured and the swell is calculated as the vertical displacement divided by the original height of the specimen. The results of these tests are presented on the Logs of Boring at the depth of the samples tested.

Swell Pressure Test - ASTM D-4546-96

The swell pressure test assesses the potential for swell of soil. This value is useful for the design of various structures such as slab-on-ground foundations, piers and piles, and underground utilities. Method C of the aforementioned ASTM standard determines the pressure required to keep a soil sample at equilibrium under swelling conditions. This is done by placing the sample in a consolidometer under a seating load and giving the sample free access to water. A constant height of the sample is maintained and the vertical pressure on the sample is adjusted until equilibrium is reached. The vertical pressure on the sample at equilibrium is reported as the swell pressure. The results of these tests are presented on the Logs of Boring at the depth of the samples tested.

Soil Suction Test - ASTM D-5298-94

Soil suction (potential) tests are performed to determine both the matric and total suction values for the samples tested. Soil suction measures the free energy of the pore water in a soil. In a practical sense, soil suction is an indication of the affinity of a given soil sample to retain water. Soil suction provides useful information on a variety of characteristics of the soil that are affected by the soil water including volume change, deformation, and strength.

Soil suction tests are performed using the filter paper method per ASTM D-5298. Results of these tests are shown graphically on the logs of boring and tabulated in summary sheet of laboratory data.

For matric suction values found using this method, it should be noted that when the soil is in a dry state adequate contact between the filter paper and the soil may not be possible. This lack of contact may result in the determination of total suction instead of matric suction.

Triaxial Shear Test - ASTM D-2850-70

Triaxial tests may be performed on samples that are approximately 2.83 inches in diameter, unless a smaller diameter sample was necessary to achieve a more favorable length:diameter (L:D) ratio. A minimum length to diameter ratio (L:D) of 2.0 is maintained to reduce end effects.

The triaxial tests are typically unconsolidated-undrained using nitrogen gas for chamber confining pressure. Confining pressures are selected to conform to in-situ hydrostatic pressure considering the earth to be a fluid of 120 pcf. In this test, undisturbed Shelby tube samples are trimmed so that their ends are square and then pressed in a triaxial compression machine. The load at which failure occurs is the compressive strength. The results of the triaxial tests and the correlated hand penetrometer strengths can be utilized to develop soil shear strength values. These test provide the confined compressive strength, q_c , which are presented on the Logs of Boring at the depth of the samples tested.

Unconfined Compressive Strength of Rock Cores - ASTM D-2938

The unconfined compressive strength, q_u , is a valuable parameter useful in the design of foundation footings. This value, qu, is related to the shearing resistance of the rock and thus to the capacity of the rock to support a load. In completing this test it is imperative that the length: diameter ratio of the core specimens are maintained at a minimum of 2:1. This ratio is set so that the shear plane will not extend through either of the end caps. If the ratio is less than 2.0 a correction is applied to the result.

Grain Size Analysis - ASTM D-421 and D-422

Grain size analysis tests are performed to determine the particle size and distribution of the samples tested. The grain size distribution of the soils coarser than the Standard Number 200 sieve is determined by passing the sample through a standard set of nested sieves, and the distribution of sizes smaller than the No. 200 sieve is determined by a sedimentation process, using a hydrometer. The results are given on the log of Boring/Pit or on Grain Size Distribution semi-log graphs within the report.

Slake Durability Test - ASTM D-4644

The slake durability test provides an index for the durability of a shale, or similar rock, considering the effects of wetting, drying, and abrasion. This index is used to quantify the strength of weak rock formations when exposed to natural wetting and drying cycles, especially in the context of underground tunneling and excavation. The index, $I_d(2)$, represents the percentage, by mass, of rock material retained after two wetting and drying cycles. These cycles are simulated by oven drying the sample followed by ten minutes of tumbling and soaking in water within a drum and trough apparatus. After tumbling and soaking, the sample is oven-dried and the mass of the sample is recorded. The results of these tests are presented on the Logs of Boring at the depth of the samples tested.

Brazilian Tensile Strength - ASTM D-3967

The Brazilian (splitting) tensile strength, σ_t , is useful in rock mechanics design, especially in regard to tunneling. This value is an indirect representation of the true uniaxial tensile strength. The Brazilian test is typically used more commonly than direct tensile strength tests because it is less difficult, more cost effective, and more represented of in-situ conditions. The test is conducted by mechanically compressing a rock core sample along its vertical diameter, causing the sample to fail due to tension along the horizontal diameter caused by the Poisson effect.

CERCHAR Abrasivity Index (CAI) Test - ASTM D-7625

The CERCHAR Abrasivity Index (CAI) is used to determine the abrasivity of rocks. This is particularly useful in assessing the potential wearing on cutting tools during excavation. The CAI of a rock is determined by the CERCHAR test, which consists of scraping steel pins across a rock surface and measuring the wear of each pin. The rock specimen is held in a mechanical vice, while a conical steel pin fastened to a 15-pound head is drug across the face of the specimen using a lever being pulled 1 centimeter in 1 second. The CAI is calculated based on the resultant diameter on the end of the pin.