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GEOTECHNICAL FEASIBILITY EVALUATION

Elizondo Tract - REVISED
San Antonio, Texas

Report For:

Forestar Group, Inc.
2221 E. Lamar Boulevard Suite 790
Arlington, Texas 76006

May 2022

Engineer's Job # 20201100.003

MLA Geotechnical TBPE FIRM # F-2684
Geotechnical Engineering and Construction Materials Testing
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TABLE OF CONTENTS

GEOTECHNICAL FEASIBILITY EVALUATION

Elizondo Tract - REVISED
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BACKGROUND

This report presents the results of a soil exploration and analysis for the proposed Elizondo Tract project in San Antonio, Texas. Authorization to perform this exploration and analysis was by Agreement for Engineering Services signed by Ms. Lauren Anderson of Forestar Group, Inc. on September 23, 2020.

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 in-house geologic maps, USGS topography maps and firm experience in the area [1,2]. Seven 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 consists mainly of dark gray high plasticity clay (CH) that varies in color to yellowish tan and gray with depth. A layer of tan low plasticity clay (CL) was encountered in Boring B-4.

Geologic Profile

Geologic maps indicate the Leona Formation, *Qle*, and the Marlbrook Marl, *Kknm*, beneath the subject site [1,2]. A description of each geology follows.

The Leona Formation, *Qle,* is part of a series of Terrace Deposits of the late Quaternary Period. The composition of these terrace deposits was dependent on the depositional energy of the flood plain and the composition of the material eroded by the rivers. Locally, terrace deposits from the Quaternary Period are quite variable horizontally due the nature of fluvial systems.

The older portion of these terrace deposits generally consists of coarse sand with thick layers of coarse gravel. Often, very little clay is present in the lower (oldest) terrace deposits. As the geologic record progresses to the present, the sand becomes finer and gravel layers become less frequent and thinner and the gravel sizes are smaller. Larger proportions of clay are typically realized in the upper portions of terrace deposits. These terrace deposits typically terminate in the Pleistocene Epoch where they consist of fine sand and clay. More recent alluvial material is not often mapped separately from the older terrace deposits because the boundary is not sufficiently distinct.

Calcareous zones occur occasionally in terrace deposits and are thought to be caused by the precipitation of salts in ground water from extreme drying periods. The extreme drying periods resulted in the sediments of that time being over-consolidated. Cemented conglomerate is often encountered in this formation. Terrace deposits generally have moderate bearing capacity and a variable shrink and swell potential. Due to their nature, terrace deposits have the ability to store and transmit large quantities of ground water. The thickness of the Leona Formation ranges up to 50 feet.

The Marlbrook Marl, *Kknm*, commonly referred to as the Upper Taylor Marl is from the Cretaceous Period and is overlain by the Eocene age formations and underlain by the Austin Group of the Cretaceous Period. The thickness of the Marlbrook Marl or the "Upper Taylor Marl" is approximately 600 feet.

The Upper Taylor Marl is characterized by dark brown to dark gray and blue gray, montmorillonitic clay. The Taylor in the unweathered state is usually hard, dry shale. Within a 30 to 70 foot surficial zone, weathering usually forms a high plasticity, mottled light brown and light gray tan with reddish and yellowish brown seams. Thin layers of quartz and calcareous siltstone are common. Layers of soft calcium and other readily soluble minerals left behind from evaporation processes are common in the upper part. Some silt and fine sand layers are present in the clay matrix which causes this formation to be highly susceptible to lateral moisture penetration. This can be result in extensive volume changes with varying soil water contents. Structures built on this formation without proper consideration of these effects often undergo significant movement resulting in damage to the structure.

Faults

Geologic maps do not indicate a fault on the site and faulted conditions were not noted in the borings.

Ground Water

Ground water was not encountered in the borings during the field investigation. However, based on the nearby ponds and tributaries, groundwater should be anticipated. The presence of ground water is seasonal and random, depending on the amount of preceding rainfall, weather patterns, and changes in land use. Therefore, groundwater may be present at various depths after rainfall.

TOPOGRAPHY, DRAINAGE AND VEGETATION

The site is situated on variably sloping topography with typical slopes varying from approximately 1 to 14 percent. Regionally, the site drains to the southeast, south, and southwest. The predeveloped vegation on this site consists of scattered mature trees and wild 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.

Table 1: Expected Foundation Design Parameters

P.T.I. Parameters (3)				B.R.A.B. #33 Parameter (4)	PVM (5)
Em-center, ft	Em-edge, ft	Ym-center, in	Ym-edge, in	Estimated Equivalent P.I.	inches
7.5	3.8	2.1	2.4	60*	5+

2. * - Local topography maps indicate that this site is sloping with slopes ranging up to 14 percent. Significant earthwork may be required on this site due to the slope. Deep cuts in expansive clay can cause extensive slab on ground foundation tilt and/or cracking which will lead to unsatisfactory foundation performance. Depending on the final grading plan, additional earthwork or foundation recommendations may be required on individual lots. Once a grading plan is finalized, lots with significant cuts can be identified. These lots will likely require moisture conditioning or moisture/chemical injection to a depth of 5 to 7 feet.

3. The soils at this project site exhibited plasticity indices ranging from 20 to 55. Point estimates of the potential vertical movement, PVM, of the in-situ soil profile range up to 5 or more inches. Thus, the potential for disruptive foundation movements due to expansive soils may be categorized as very high. PVM was calculated considering the

potential vertical rise (PVR) method, free swell testing of undisturbed soil samples at in-situ moisture conditions, and firm experience in similar sites. The PVR method is a TxDOT procedure that only considers Atterberg Limits and moisture content of the soil [(5)]. The PVR method is widely used and should be considered an index property of the site; however, it has been known to underestimate the swell potential of highly expansive soils. Other magnitudes of PVR/PVM may be estimated by other methods and at other locations with varying results. It is our understanding that PVM reduction is desired in order to reduce the PVM to 4 ½ inches or less. As such, moisture conditioning or moisture/chemical injection to a depth of 7 feet will be required. Other magnitudes of PVR may be estimated by other methods and at other locations with varying results. However, the TxDOT Method is widely used and should be considered an index property of the site.

4. It is recommended that all foundations be designed by competent foundation engineers using generally accepted foundation procedures.
5. This site is highly expansive. Foundation footprints at this site should be kept relatively compact and rectangular. A typical foundation footprint in these areas should be limited to the following:
    - Nearly rectangular shape with the shape factor less than 20, where the shape factor is defined as the perimeter of the footprint squared divided by the footprint area.
    - No offsets greater than 12 feet.
    - No concrete length greater than 70 feet.

Construction

1. Excavation may be performed using ordinary power equipment for the construction of slab-on-ground foundations at this site.

2. 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. Public pavement sections at this site will require a thickened base course and a lime stabilized subgrade. For example, local streets will likely require 2 inches of HMAC, 8 to 10 inches of crushed limestone base course, and 8 to 10 inches of lime stabilized subgrade. A geotechnical pavement study will be required to develop pavement thickness sections.

4. Private pavement sections at this site will depend on the traffic loading and intended use. Typical sections for lightly loaded parking and access ways will likely require 2 inches of HMAC, 10 to 15 inches of crushed limestone base course, and 8 to 10 inches of lime stabilized subgrade. Rigid pavement sections will likely require 6 inches of concrete and 8 to 10 inches of lime stabilized subgrade.

REFERENCES

1. 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.

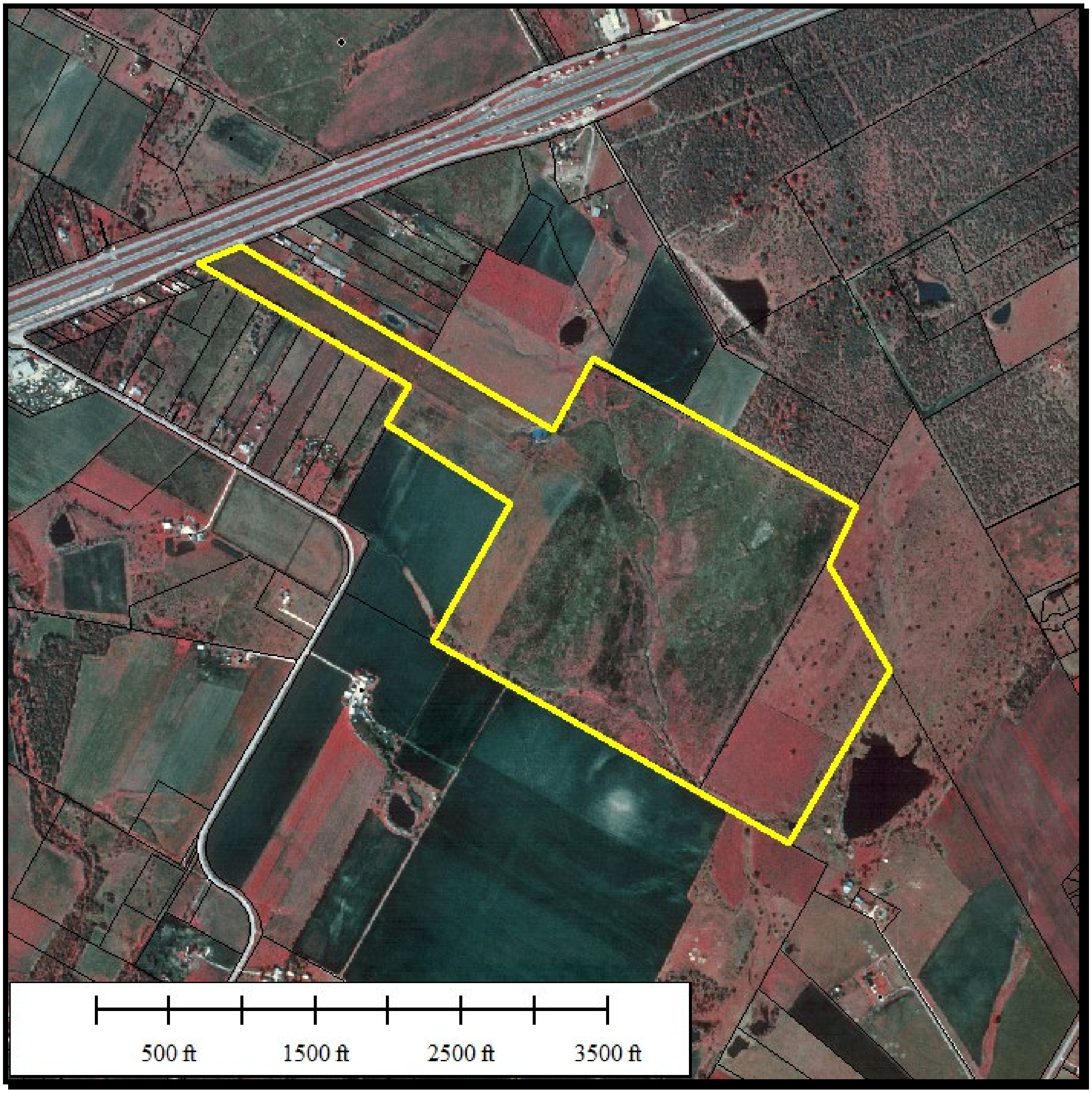
5. "Method for Determining Potential Vertical Rise, PVR, Test Method Tex-124-E", Manual of Testing Procedures, Texas Department of Transportation Materials and Tests Division, September 1995.

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 borings. 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.

APPENDIX A

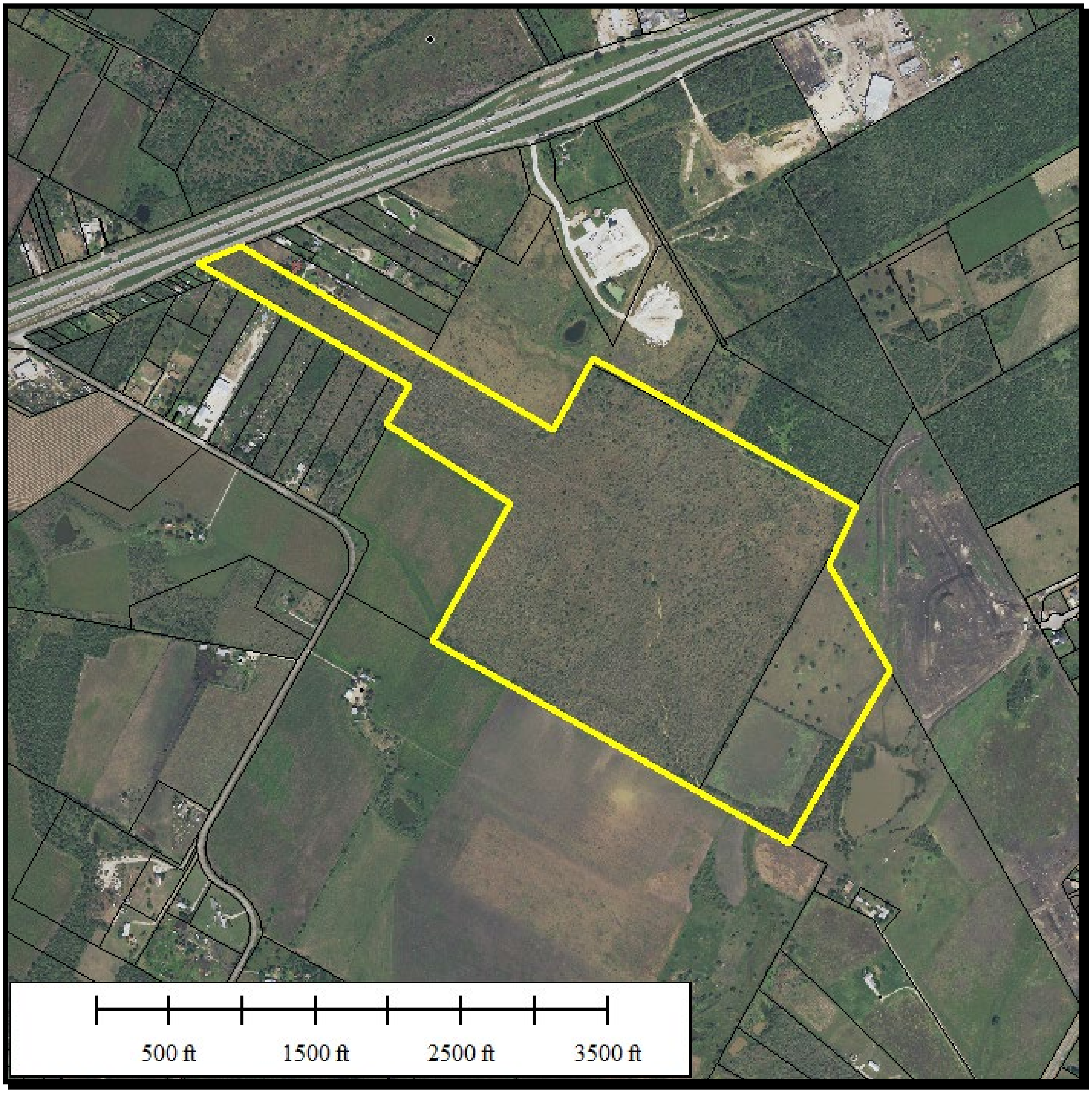
GEOTECHNICAL DATA



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)

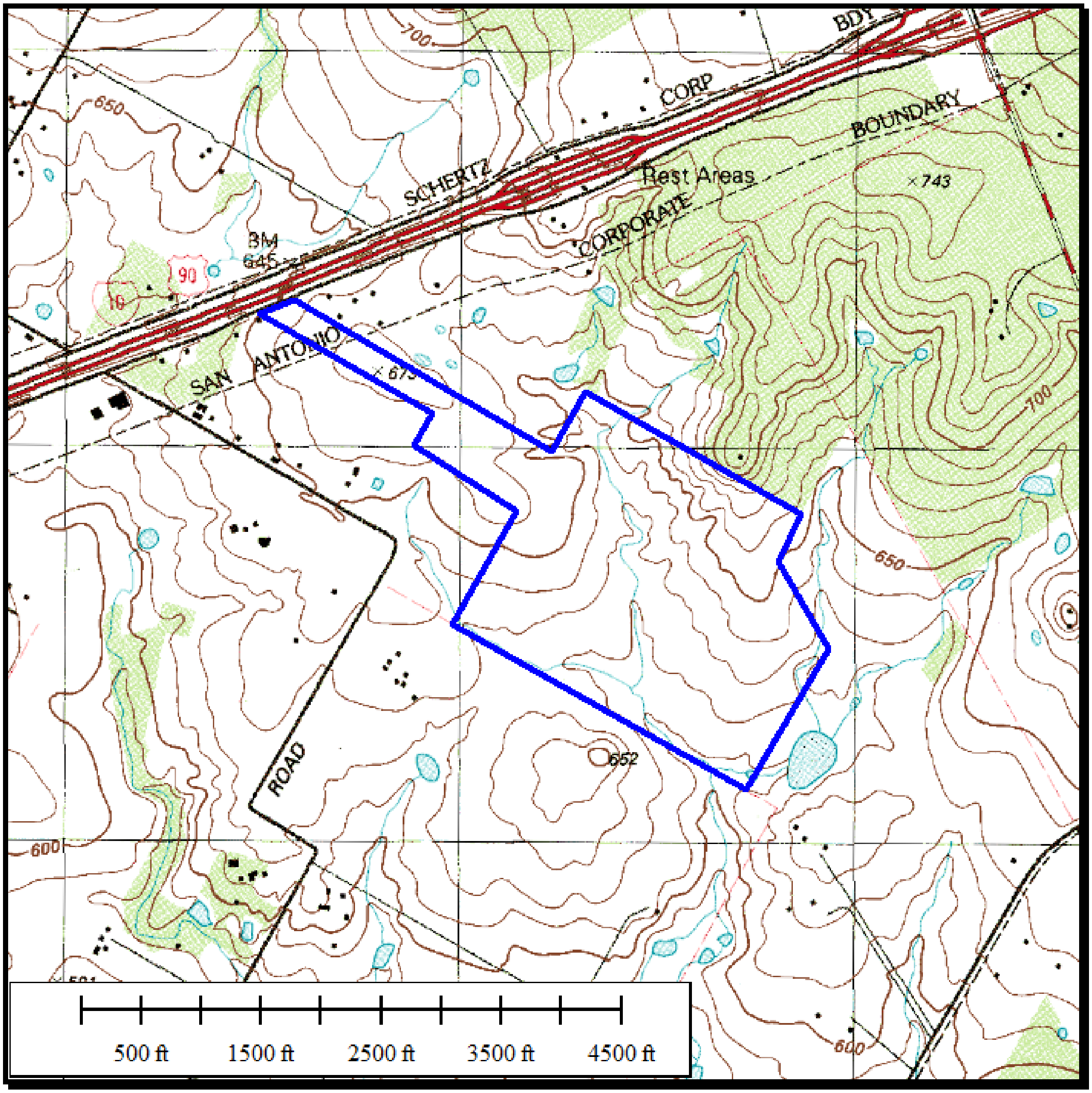


Approximate location of site in yellow

Aerial Photograph of Site – 2018

Source: TEXAS NATURAL RESOURCES INFORMATION SYSTEM
Apx. Date - 2018
(www.tnris.org)

N

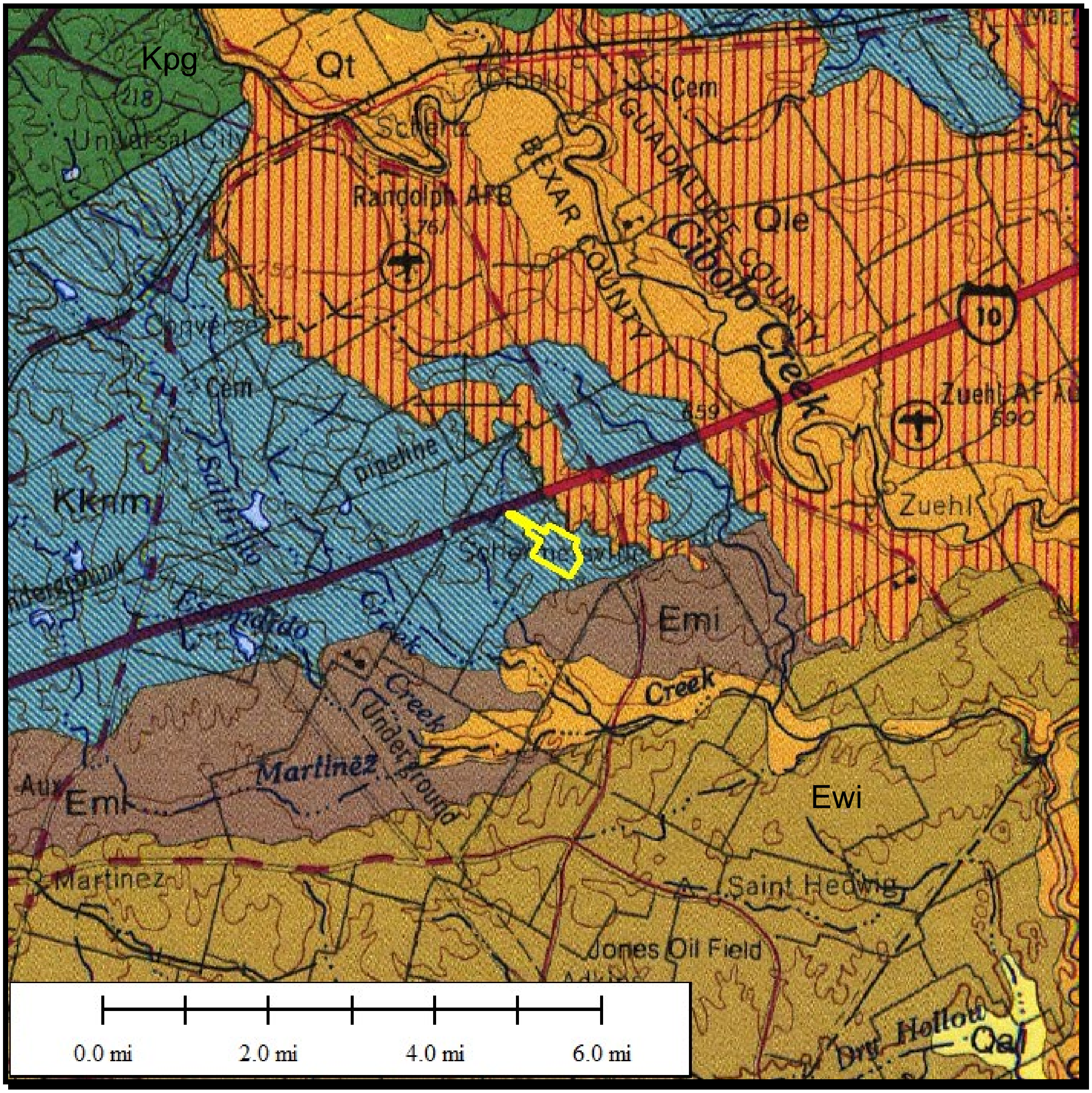


Approximate location of site in blue

U.S. 7.5 Minute Series Topographic Map
Saint Hedwig Quadrangle, Texas
Contour Interval = 10 feet

Source: TEXAS NATURAL RESOURCES INFORMATION SYSTEM
(http://www.tnris.state.tx.us/digital.htm)

N



Approximate location of site in yellow

Geologic Setting of Site
Geologic Atlas of Texas

N

Original Source: Bureau of Economic Geology, The University of Texas at Austin, latest version
Digital Source: 15-minute Digital GAT Quads. TCEQ March 9, 2004

SCALE = N.T.S.

PAGE 1 OF 1

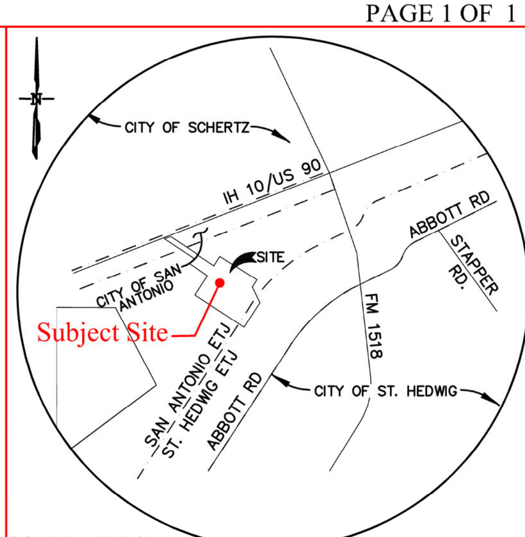
PLAN OF BORINGS

Elizondo Tract
San Antonio, Texas
Job. No.: 20201100.003
Client: Forestar Group, Inc.

LEGEND

B-#	Boring Number
⊕	Approx. Boring Location

VICINITY MAP



SCALE = N.T.S.

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LOG OF BORING

Boring B-1
PAGE 1 OF 1

Job Name: Elizondo Tract
Job Location: San Antonio, Texas
Engineer's Job #: 20201100.003
Client: Forestar Group, Inc.

Drill Date: September 17, 2020
Ground Elevation: n/a
Hole Size: 4.5 in.
Notes:

Ground Water Levels:
AT TIME OF DRILLING: ---
AT END OF DRILLING: ---
AFTER DRILLING: ---

DEPTH, ft.	GRAPHIC LOG	MATERIAL DESCRIPTION	GEOLOGY U.S.C.S.	POCKET PEN. (tsf)	-2um / -#200 / -#4; Moisture Content, %; PL / LL (0 20 40 60 80 100)	Plasticity Index, %
0		CLAY, dark gray, hard, damp	CH	4.5		
				4.5		
		...yellowish tan and gray below 4.2'				
5				4.5		
				4.5		
				4.5		
10						
				4.5		
				4.5		
			Kknm	4.5		
15		Termination Depth: 15.0 feet				
20						
25						
30						

20201100.003 - ELIZONDO TRACT - LOGS.GPJ 10/7/20

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LOG OF BORING

Boring B-2

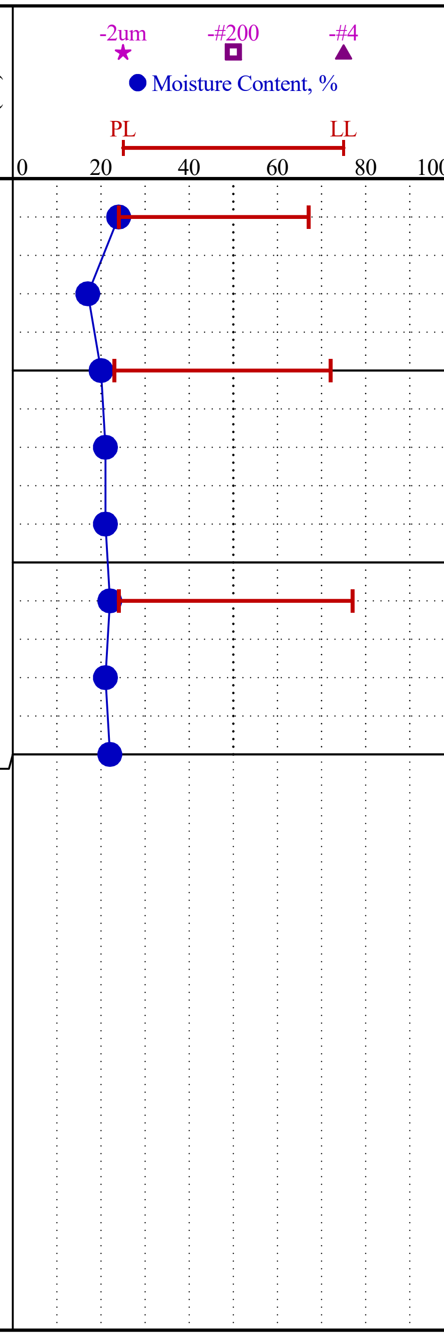
Job Name: Elizondo Tract
Job Location: San Antonio, Texas
Engineer's Job #: 20201100.003
Client: Forestar Group, Inc.

Drill Date: September 17, 2020 **Ground Elevation:** n/a
Hole Size: 4.5 in.

Notes:

Ground Water Levels:
AT TIME OF DRILLING: ---
AT END OF DRILLING: ---
AFTER DRILLING: ---

DEPTH, ft.	GRAPHIC LOG	MATERIAL DESCRIPTION	GEOLOGY U.S.C.S.	POCKET PEN. (tsf)	Plasticity Index, %
0		CLAY, dark gray, hard, damp	CH	4.5	43
				4.5	
		...yellowish tan and gray below 3.9'			
5				4.5	49
				4.5	
				4.5	
10				4.0	53
				4.5	
15			Kknm	4.5	
		Termination Depth: 15.0 feet			
20					
25					
30					



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LOG OF BORING

Boring B-3

Job Name: Elizondo Tract
Job Location: San Antonio, Texas
Engineer's Job #: 20201100.003
Client: Forestar Group, Inc.

Drill Date: September 17, 2020
Hole Size: 4.5 in.
Ground Elevation: n/a
Notes:

Ground Water Levels:
AT TIME OF DRILLING: ---
AT END OF DRILLING: ---
AFTER DRILLING: ---

DEPTH, ft.	GRAPHIC LOG	MATERIAL DESCRIPTION	GEOLOGY U.S.C.S.	POCKET PEN. (tsf)	-2um ★ / -#200 ◻ / -#4 ▲ / ● Moisture Content, % / PL — LL (0 20 40 60 80 100)	Plasticity Index, %
0		CLAY, dark gray, very stiff to hard, damp ...yellowish tan and gray below 1.8'	CH	3.5		
				4.5		
5				4.5		
				4.5		
				4.5		
10				4.0		
				4.5		
15			Kknm	4.5		
15		Termination Depth: 15.0 feet				
20						
25						
30						

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LOG OF BORING

Boring B-4

PAGE 1 OF 1

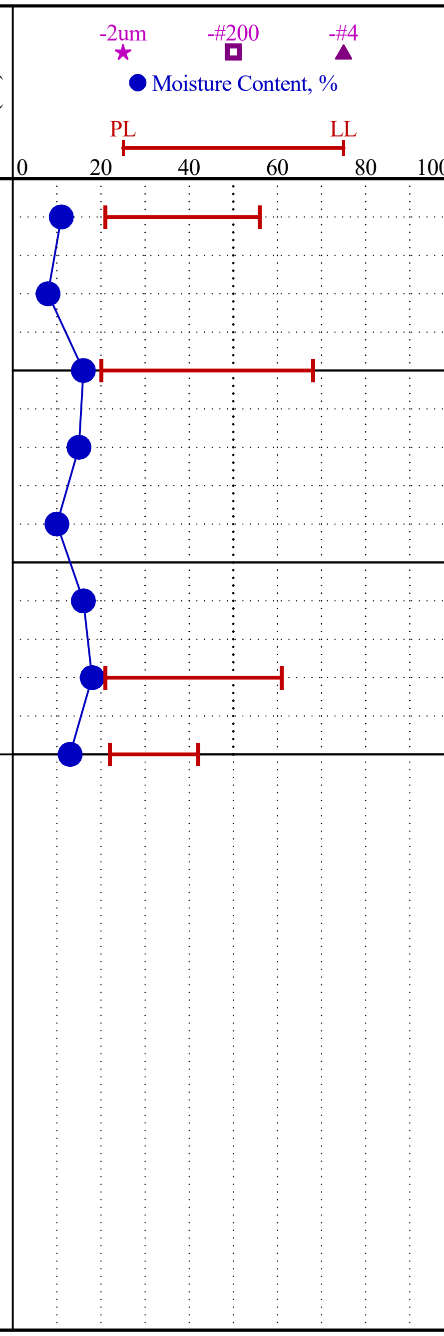
Job Name: Elizondo Tract
Job Location: San Antonio, Texas
Engineer's Job #: 20201100.003
Client: Forestar Group, Inc.

Drill Date: September 17, 2020
Ground Elevation: n/a
Hole Size: 4.5 in.

Notes:

Ground Water Levels:
AT TIME OF DRILLING: ---
AT END OF DRILLING: ---
AFTER DRILLING: ---

DEPTH, ft.	MATERIAL DESCRIPTION	GEOLOGY U.S.C.S.	POCKET PEN. (tsf)	Plasticity Index, %
0	CLAY, dark gray, with gravel, damp	CH	4.5	35
	...push refusal at 2.3'			
	...tan below 2.5'		4.5	
5				48
10				
				40
	CLAY, tan, with gravel, damp	CL		
15		Qle		20
	Termination Depth: 15.0 feet			
20				
25				
30				



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LOG OF BORING

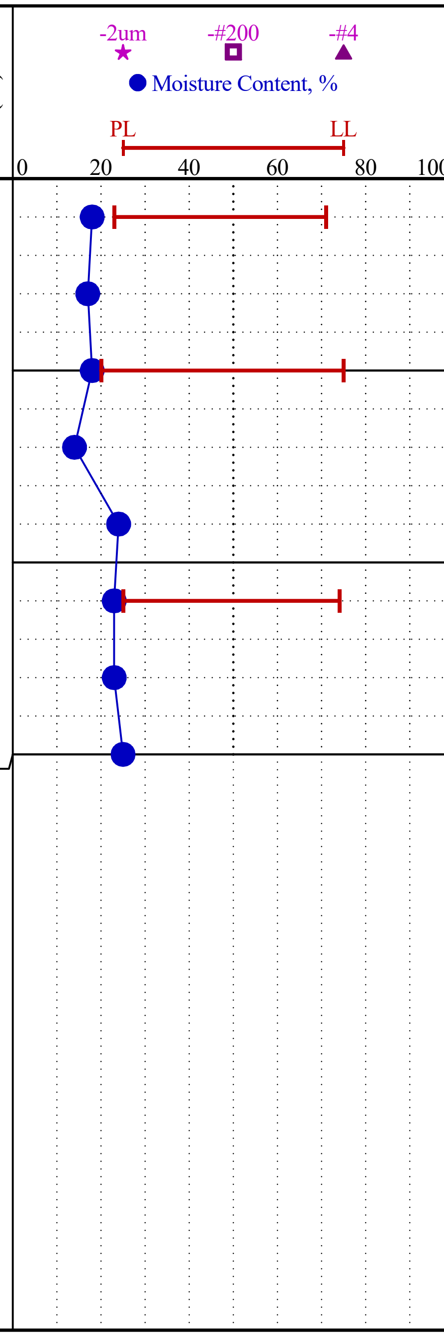
Boring B-5

Job Name: Elizondo Tract
Job Location: San Antonio, Texas
Engineer's Job #: 20201100.003
Client: Forestar Group, Inc.

Drill Date: September 17, 2020
Ground Elevation: n/a
Hole Size: 4.5 in.
Notes:

Ground Water Levels:
AT TIME OF DRILLING: ---
AT END OF DRILLING: ---
AFTER DRILLING: ---

DEPTH, ft.	MATERIAL DESCRIPTION	GEOLOGY U.S.C.S.	POCKET PEN. (tsf)	Plasticity Index, %
0	CLAY, dark gray, very stiff to hard, damp	CH	3.5	48
			4.5	
5	...push refusal at 5.9' ...with gravel from 6.0' to 7.8' ...yellowish tan and gray below 7.8'		4.5	55
			4.5	
10			4.0	49
			4.5	
15		Kknm	4.5	
	Termination Depth: 15.0 feet			
20				
25				
30				



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LOG OF BORING

Boring B-6

Job Name: Elizondo Tract
Job Location: San Antonio, Texas
Engineer's Job #: 20201100.003
Client: Forestar Group, Inc.

Drill Date: September 17, 2020
Ground Elevation: n/a
Hole Size: 4.5 in.

Ground Water Levels:
AT TIME OF DRILLING: ---
AT END OF DRILLING: ---
AFTER DRILLING: ---

Notes:

DEPTH, ft.	GRAPHIC LOG	MATERIAL DESCRIPTION	GEOLOGY U.S.C.S.	POCKET PEN. (tsf)	-2um ★ -#200 ◻ -#4 ▲ ● Moisture Content, % PL LL (0 20 40 60 80 100)	Plasticity Index, %
0		CLAY, dark gray, very stiff to hard, damp	CH	4.5		
				4.5		
		...yellowish tan and gray below 3.9' ...with gravel from 3.9' to 6.2'				
5				4.5		
				4.5		
				4.5		
10				2.5		
				4.5		
15			Kknm	4.5		
		Termination Depth: 15.0 feet				
20						
25						
30						

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LOG OF BORING

Boring B-7
PAGE 1 OF 1

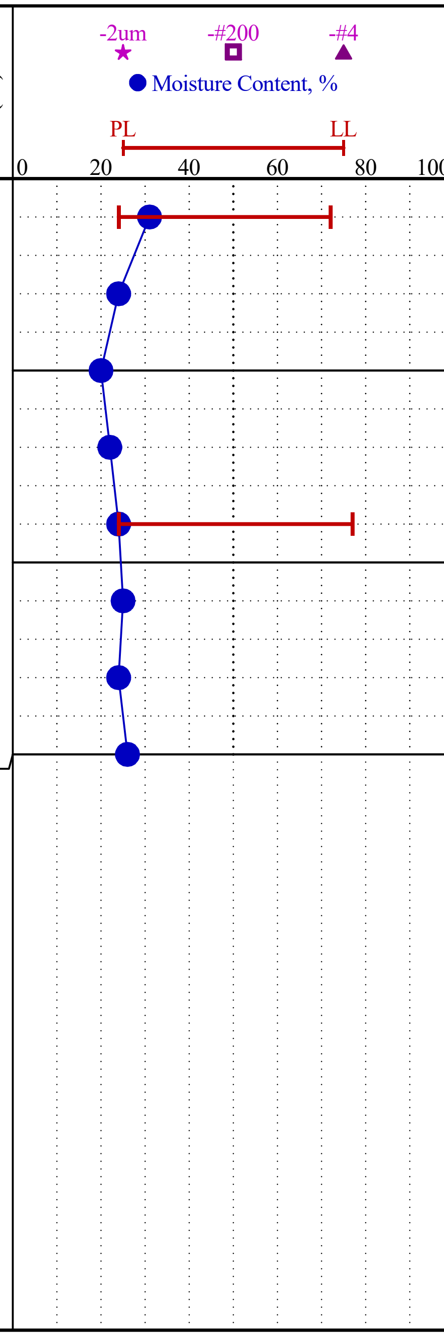
Job Name: Elizondo Tract
Job Location: San Antonio, Texas
Engineer's Job #: 20201100.003
Client: Forestar Group, Inc.

Drill Date: September 17, 2020
Ground Elevation: n/a
Hole Size: 4.5 in.

Ground Water Levels:
AT TIME OF DRILLING: ---
AT END OF DRILLING: ---
AFTER DRILLING: ---

Notes:

DEPTH, ft.	MATERIAL DESCRIPTION	GEOLOGY U.S.C.S.	POCKET PEN. (tsf)	Plasticity Index, %
0	CLAY, dark gray, very stiff to hard, damp	CH		
			3.0	48
			4.5	
5	...with gravel from 5.0' to 6.0' ...yellowish tan and gray below 5.5'		4.5	
			4.5	
			4.5	53
10				
			3.0	
			4.0	
15		Kknm	3.5	
	Termination Depth: 15.0 feet			
20				
25				
30				



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SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS: GRAPH	SYMBOLS: LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	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
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	CLAYS	LIQUID LIMIT GREATER THAN 50		CH	INORGANIC CLAYS OF HIGH PLASTICITY
SOILS OF MODERATE PLASTICITY				CL-CH	LOW PI CLAYS WITH APPRECIABLE HIGH PI MOTTLING, CLAY WITH BORDERLINE CLASSIFICATION
OTHER MATERIALS				FILL	MATERIAL NOT NATURALLY DEPOSITED
				LS	WEATHERED LIMESTONE INTACT LIMESTONE

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

Key to Terms and Abbreviations

Descriptive Terms Characterizing Soils and Rock

Argillaceous – having appreciable amounts of clay in the soil or rock mass. Used most often in describing limestones, occasionally sandstones.

Calcareous – containing appreciable quantities of calcium carbonate. Can be either nodular or "powder."

Crumbly – cohesive soils which break into small blocks or crumbs on drying.

Evaporite – deposits of salts and other soluble compounds. Most commonly calcium carbonate or gypsum. May be in either "powder" or visible crystal form.

Ferruginous – having deposits of iron or nodules, typically oxidized and dark red in color.

Ferrous – see Ferruginous

Fissured – containing shrinkage cracks frequently filled with fine sand or silt, usually more or less vertical.

Fossiliferous – containing appreciable quantities of fossils, fossil fragments, or traces of fossils

Laminated – composed of thin layers of varying color or texture. Layers are typically distinct and varying in composition from sand to silt and clay.

Mottled – characterized as having multiple colors organized in a marbled pattern.

Slickensided – having inclined planes of weakness that are slick and glossy in appearance.

Varved – see Laminated.

Standard Description Abbreviations and Terms

brn = brown
dk = dark
lt = light
wx = weathered
calc = calcareous
sw = severely weathered
cw = completely weathered
n/a = not available
b. = below

Engineering Units

pcf = pounds per cubic foot
psf = pounds per square foot
tsf = tons per square foot
pF = picofarad
psi = pounds per square inch
kips = thousand pounds (force)
ksf = kips per square foot

Symbols and Abbreviations for Test Data

LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index (LL-PL)
NP = non-plastic
γ_d = dry unit weight
q_u = unconfined compressive strength
q_c = confined compressive strength
SPT = standard penetration test
TCP = Texas cone penetration test (Texas Highway Department)
N or N_{SPT} = blows per foot from SPT
N_{TCP} = blows per foot from TCP
SCR = standard core recovery
RQD = rock quality designation
RQI = see RQD

Terms Describing Consistency of Soil and Rock

COARSE GRAINED MATERIAL		SEDIMENTARY ROCK	
DESCRIPTIVE TERM	BLOWS/FT (SPT)	DESCRIPTIVE TERM	STRENGTH, TSF
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 TERM	BLOWS/FT (SPT)	UNCONFINED COMPRESSION, TSF
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

Sample Type Key

Auger Cuttings
Shelby Tube
Split Spoon (SPT)
Texas Cone (TCP)
Rock Core
No Sample

Revised: October 2018

APPENDIX B

STANDARD FIELD AND LABORATORY PROCEDURES

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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.

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:

$$SCR = \frac{\text{total core length recovered}}{\text{length of core run}} \times 100\%$$

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:

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

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.