ARKANSAS DEPARTMENT OF TRANSPORTATION

SUBSURFACE INVESTIGATION

STATE JOB NO. 101000

FEDERAL AID PROJECT NO. NHPP-0028(52)

VILLAGE CREEK STR. & APPRS. (S)

STATE HIGHWAY 69 **SECTION** 10

IN GREENE **COUNTY**

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Geotechnical Exploration
Highway 69 Over Village Creek Str. & Apprs. (S)
Greene County, Arkansas

Arkansas Department of Transportation
State Project No. 101000
Federal Aid Project No. 9990

Prepared for:

Garver, LLC
North Little Rock, Arkansas

Prepared by:

Geotechnology, Inc.
Memphis, Tennessee

Date:

August 10, 2020

Geotechnology Project No.:

J034363.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

GEOTECHNOLOGY INC
FROM THE GROUND UP

August 10, 2020

Mr. John Ruddell, P.E., S.E.
Vice President - Bridge Design Manager
Garver, LLC
4701 Northshore Drive
North Little Rock, Arkansas 72118

Re: Geotechnical Exploration
ARDOT 101000
Highway 69 Over Village Creek Structures and Approaches (S)
Greene County, Arkansas
Geotechnology Project No. J034363.01

Dear Mr. Ruddell:

Presented in this report are the results of the geotechnical exploration performed by Geotechnology, Inc. for the referenced project. The report includes our understanding of the project, observed site conditions, conclusions and/or recommendations, and support data as listed in the Table of Contents.

We appreciate the opportunity to provide geotechnical services for this project. If you have any questions regarding this report, or if we can be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,

GEOTECHNOLOGY, INC.

Dale M. Smith, P.E.
Geotechnical Manager

STATE OF ARKANSAS
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1.0 SCOPE OF SERVICES

Presented in this report are the results of the geotechnical exploration and recommendations for design and construction for the proposed approach improvements and bridge replacement over Village Creek. The referenced features include demolition of the existing bridge and construction of a new bridge (Structure No. M3808). It is our understanding the anticipated foundation type for support of the new bridges is driven, closed-ended, concrete-filled, pipe piles. The existing bridge approaches will be modified to facilitate traffic flow over the new bridges. A general overview of the project is shown on Figure 1 included in Appendix B.

The recommendations presented in this report are based on the geology, topography, and the results of the geotechnical exploration. Results of borings, in-situ testing, sampling, and laboratory testing are included in the report. A total of three borings were drilled in the vicinity of the site as shown on Figure 2 included in Appendix B. The boring logs, along with field and laboratory test results, are enclosed. The collected data have been analyzed and the physical properties of the in-situ soils summarized. General site conditions are discussed, along with recommendations for subgrade preparation. Important information prepared by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association for studies of this type is presented in Appendix A for your review.

2.0 GENERAL INFORMATION

Planned Modifications

The existing 2-lane, 116-foot long, 34.7-foot wide, 4-span, Highway 69 bridge over Village Creek will be replaced with a 2-lane, 126.3-foot long, 32.5-foot wide, 3-span bridge. The existing timber-pile bents will be removed and closed-ended, concrete-filled, pipe piles of 16- and 20-inch diameter will be driven at the abutments and bents, respectively. Riprap is planned along the toe of the abutment (spill) slopes based on the provided plans[1]. Spill slopes are anticipated to be two and one-half horizontal units for every vertical unit (2.5H:1V) and side slopes are anticipated to be 3H:1V and 4H:1V. The intersections of County Road 933 and access drives will be modified to accommodate the new alignment. Up to approximately 15 and 6 feet of cut and fill, respectively, is required to meet design grades.

Topography

According to the provided plans, ground surface elevations vary from approximately El 268[2] along the existing highway centerline to approximately El 252 along Village Creek at its intersection with Highway 69 Bridge No. M3808.

Drainage

The drainage system in the project area consists of the Lower St. Francis Watershed. The Lower St. Francis Watershed, in turn, is part of the overall drainage system of the Mississippi River Basin.

Physiographic Setting & Geology

Greene County is located in northeastern Arkansas, in the Mississippi Embayment. The Mississippi Embayment is a trough-like depression containing thousands of feet of sediment and plunging southward along an axis approximating the present course of the Mississippi River. The deposits in the area consist of Holocene epoch alluvial gravel and sand. These materials are typically white to brown or gray, poorly to well sorted, fine- to coarse-quartz sand and gravel with minor silts and clays. These deposits form a broad terrace among the west side of the Mississippi River flood plan, and include both glacial outwash and non-glacial alluvium. Thickness can vary from 10 to 130 meters and may include loessal colluvium from nearby Crowley's Ridge.

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[1] *Arkansas Department of Transportation Construction Plans for State Highway, Village Creek STRS. & APPRS. (S), Greene County, Route 69 Section 10, Job 101000*, Federal Aid Project 9990, provided by Garver, LLC on February 3, 2020.

[2] Elevations are in units of feet referenced to the mean sea level datum.

3.0 GEOTECHNICAL EXPLORATION

The borings were drilled between January 28th and February 3rd, 2020 with a rotary drill rig (CME 55) using hollow-stem auger and wash rotary drilling methods. The borings were drilled to a maximum depth of 100 feet. An additional boring was drilled adjacent to Boring VC-3 on March 23, 2020 to collect additional samples and extend the soil profile to 120 feet. The information obtained from this boring was added to the VC-3 boring log. Sampling procedures included Standard Penetration Test (SPT) and thin-wall (Shelby) tube methods. SPT's were conducted at 2.5-, 5-, and 10-foot depth intervals using automatic hammers. Thin-walled Shelby tube samples were collected in cohesive soils at selected depths. Groundwater observations were made during drilling operations.

The collected samples were visually examined by field staff and transported to our laboratory for further evaluation and testing. The samples were examined in the laboratory by a geotechnical professional who prepared descriptive logs of the materials encountered. The boring logs are presented in Appendix C. An explanation of the terms and symbols used on the boring logs is also provided in Appendix C. Included on each boring log are ground surface elevation, station and offset provided by representatives of Garver. Included in Table 1 are in situ tests and measurements made as part of the fieldwork and recorded on the boring logs.

Table 1. Field Tests and Measurements.

Item	Test Method
Soil Classification	ASTM D 2488/ D 3282
Standard Penetration Test (SPT)	ASTM D 1586/ AASHTO T206
Thin-Walled (Shelby) Tube Sampling	ASTM D 1587/ AASHTO T207

4.0 LABORATORY REVIEW AND TESTING

Laboratory testing was performed on soil samples to assess engineering and index properties. Most of the laboratory test results are presented on the boring logs in Appendix C. The Atterberg limits, grain size analyses, resistivity, unconsolidated-undrained triaxial compression (UU), and consolidated-undrained triaxial compression test results are also provided in Appendix D. The laboratory tests and corresponding test method standards are presented in Table 2.

Table 2. Summary of Laboratory Tests and Methods.

Laboratory Test	ASTM	AASHTO
Moisture Content	D 2216	T 265
Atterberg Limits	D 4318	T 98
Grain Size Analysis by Sieving	D 6913	T 88
Grain Size Analysis by Hydrometer	D 7928	T 88
Unconsolidated-Undrained Triaxial Compression	D 2850	T 296
Consolidated-Undrained Compression	D 4767	T 297
One-Dimensional Consolidation	D 2435	T 216
Soil Electrical Resistivity	G 57	T 288
Soil pH	D 4972	T 289

The boring logs were prepared by a geotechnical engineer from the field logs, visual classification of the soil samples in the laboratory, and laboratory test results. Terms and symbols used on the boring logs are presented on the Boring Log: Terms and Symbols in Appendix C. Stratification lines on the boring logs indicate approximate changes in strata. The transition between strata could be abrupt or gradual.

5.0 SUBSURFACE CONDITIONS

The borings at this site include Borings VC-1 through -3. Borings VC-1 and -3 were drilled in the southbound lane of existing, south and north approaches, respectively. Boring VC-2 was drilled through the bridge deck of the northbound lane. Asphalt and base material thicknesses encountered in the borings are shown in Table 3.

Table 3. Surficial Materials and Thicknesses.

Boring	Material	Thickness (inches)
VC-1	Asphalt	7
	Gravel and Sand	14
VC-3	Asphalt	9
	Gravel and Sand	18

Approximately 9 inches of clayey sand was encountered at the surface of the creek bed in Boring VC-2. Underlying the surficial materials, the stratigraphy generally consisted of predominantly fine-grained soils underlain by intermixed fine- and coarse-grained soil at the depths shown in Table 4. More detailed descriptions of the stratigraphy encountered at each bridge are included below and on the boring logs in Appendix C.

Table 4. Depths of Fine- and Coarse-Grained Soils.

Stratum	Depth (feet) Boring VC-1	VC-2[a]	VC-3
Predominantly Fine-Grained Soils	1 – 68 | 78 – 100	2 – 78	3 – 68 | 78 – 118
Predominantly Coarse-Grained Soils	68 – 78	1 – 2 | 78 – 80	2 – 3 | 68 – 78 | 118 – 120

[a] Depths are referenced from ground surface of creek bed.

The predominantly fine-grained soils were classified as lean clay (CL), fat clay (CH), and silt (ML) with varying amounts of sand by the Unified Soil Classification System (USCS) and A-6, A-7-6, or A-4 by the AASHTO classification method. The fine-grained soils were very soft to hard based on SPT N-values and the results of UU tests. The laboratory testing used to determine USCS and AASHTO classifications are presented in Appendix D.

The predominantly coarse-grained soils were classified as silty sand (SM by USCS; A-1-a, A-1-b, A-2-4, A-2-6, or A-4 by AASHTO), clayey sand (SC by USCS; A-6, A-7-6, or A-4 by AASHTO), poorly graded sand with silt (SP-SM by USCS; A-3 by AASHTO), and poorly graded gravel with sand and clay (GP-GC by USCS; A-2-6 by AASHTO). Based on field test results, the coarse-grained soils were medium dense.

Corrosion Potential

In addition to laboratory soil classification and strength testing, soil pH and resistivity testing were also conducted. The purpose of corrosion and deterioration testing is to provide soil data for use by a structural engineer for analysis of any necessary protection to the piling, concrete, or reinforcing steel. Corrosion and deterioration protection requirements and guidelines for piling are set forth in Section 10.7.5 of the AASHTO LRFD Bridge Design Specifications. The corrosion and deterioration testing results are summarized below and are included in Appendix D.

Table 5. Results of Soil Resistivity Testing.

Boring	Sample No.	Sample Depth (foot)	pH	Soil Resistivity (ohm-cm)
VC-1	SS4 & SS6 | ST5 | SS7 | SS8 | SS10	8.5 & 13.5 | 10 | 18.5 | 23.5 | 28.5	4.82 | 5.06 | 6.65 | 8.32 | 2.37	1,482 | 1,026 | -- | 1,767 | 2,565
VC-2[a]	SS1 | SS2 | SS3	1 | 13.5 | 18.5	6.40 | 8.44 | 8.81	4,788 | 1,653 | 1,995
VC-3	SS5 & SS6 | SS7 & SS8	13.5 & 18.5 | 23.5 & 28.5	6.82 & 7.77 | 8.27 & 8.65	1,140 | 2,565

[a] Depths referenced from ground surface.

The following soil conditions should be considered as indicative of a potential pile deterioration or corrosion:

- Resistivity values less than 2,000 ohms-cm.
- pH less than 5.5.
- pH between 5.5 and 8.5 in soils with high organic content.

The following soil conditions should be considered as indicative of a potential steel reinforcement corrosion or deterioration situation:

- Resistivity less than 3,000 ohm-cm.
- pH less than 5.5

Results of the corrosion and deterioration testing indicate the site has moderate potential for pile or steel reinforcement deterioration with the exception of SS10 at a depth of 28.5 feet in Boring VC-1 where pH test results indicate a strong potential for pile corrosion and deterioration. Interpretation of the data and corrosion protection of the bridge structural components should be performed by the design team.

Groundwater

Groundwater was encountered during drilling operations in Boring VC-1 at a depth of 26 feet. The presence of higher groundwater levels in Borings VC-2 and -3 could have been masked by the use of mud rotary drilling methods, which introduces fluid to the borehole. Groundwater levels could vary significantly over time due to water levels in Village Creek and seasonal variation in precipitation, recharge, or other factors not evident at the time of exploration.

6.0 ENGINEERING EVALUATION, ANALYSIS, AND RECOMMENDATIONS

Site Preparation and Earthwork

The following procedures are recommended for site preparation in cut and fill areas. These recommendations do not supersede ARDOT standards and specifications. Site preparation and compaction requirements must conform to the latest ARDOT standards.

Site Preparation. In general, cut areas and areas to receive new fill should be stripped of pavements, topsoil, vegetation, and other deleterious materials. Topsoil should be placed in landscape areas or disposed of off-site. Vegetation and tree roots should be over-excavated.

The exposed subgrade should be proof-rolled using a tandem-axle dump truck loaded to approximately 20,000 pounds per axle (or equivalent proof-rolling equipment). Soft areas that develop should be over-excavated and backfilled with select fill, which is defined as soil conforming to A-4 or better material, and compacted to the unit weights specified in subsequent paragraphs.

Side Slopes. Slopes steeper than 4H:1V must be benched prior to placing new fill. Slope ratios of 3H:1V or flatter with the exception of the 2.5H:1V spill slopes are recommended for all cut and fill slopes along the proposed alignment, based on the results of global stability analyses (discussed in a subsequent section).

Cut Areas. After excavation, the top 6 inches of the resulting subgrade should be compacted to a minimum of 95% of the maximum dry unit weight as determined by a standard Proctor test (ASTM D 698/AASHTO T 99). Areas supporting pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

Fill Materials. Fill material can consist of natural soils classified as AASHTO A-6 or better. Soils classified as AASHTO A-4 or better are considered to be select fill. Fine-grained soils (A-4 through A-6) and coarse-grained soils with fines can have a maximum liquid limit (LL) of 45 percent and a plasticity index (PI) between 5 and 20 percent. Such materials should be free from organic matter, debris, or other deleterious materials and have a maximum particle size of 2 inches.

Fill and Backfill Placement. Fill and backfill should be placed in level lifts up to 8 inches in loose thickness. For fill and backfill exhibiting a well-defined moisture-density relationship, each lift should be moisture-conditioned to within ±2% of the optimum moisture content and compacted with a sheepsfoot roller or self-propelled compactor to a minimum of 98% of the maximum dry unit weight as determined by the standard Proctor test. Moisture-conditioning can include: aeration and drying of wetter soils, wetting drier soils, and/or mixing wetter and drier soils into a uniform blend. The upper three feet of fill and backfill beneath the base of pavement should be compacted to 98% of the maximum unit weight as determined by the standard Proctor test.

For fill and backfill that do not exhibit a well-defined moisture-density relationship, each lift should be compacted to at least 70% of the relatively density as evaluated from the maximum and minimum index densities measured by ASTM D4253 and D4254, respectively. The upper 3 feet of fill and backfill beneath the base of pavement should be compacted to at least 75% of the relatively density.

Moisture Considerations. The soils encountered in the borings are relatively wet and will most likely require drying. The time for drying will depend on the weather conditions during grading activities. We recommend construction take place during dry weather conditions. Wet weather conditions can cause rutting of the surficial soils which will require drying and recompacting.

Maintaining the moisture content of bearing and subgrade soils within the acceptable range is important during and after construction for the proposed structure. Silty and clayey subgrade soils should not be allowed to become wet or dry during or after construction, and measures should be taken to hinder water from ponding on these soils.

Water from surface runoff, downspouts, and subsurface drains should be collected and discharged through a storm water collection system. Positive drainage should be established around the proposed structures to promote drainage of surface water away from the structures and reduce ponding of water adjacent to these structures.

Seismic Considerations

Earthquake Risk. The project area is located within the New Madrid Seismic Zone (NMSZ). The NMSZ is located in the northern part of the Mississippi Embayment and trends in a northeast to southwest direction from southern Illinois to northeast Arkansas. In December 1811, a series of large magnitude earthquakes occurred, which were centered near New Madrid, Missouri. Three strong earthquakes occurred over a 3-month period and smaller aftershocks continued until at least 1817. According to researchers, the magnitudes of these three events ranged from 7.5 to 8.0.

Seismic Design Parameters. It is our understanding liquefaction hazard and dynamic settlement potential will be evaluated using published values. A peak ground acceleration of 0.604g was obtained from published values.

Liquefaction and Dynamic Settlement. A study was performed to evaluate the liquefaction and dynamic settlement potential at the site. Both field and laboratory data were used to perform the analysis. The field measurements included the assumed depth of the water table and the SPT N-values. The laboratory data included USCS/AASHTO classification and soil unit weight. An earthquake magnitude (Mw) of 7.7 was considered. A peak ground acceleration of 0.604g was utilized as obtained from the USGS via the Applied Technologies Council (ATC). Groundwater was set at a depth of approximately 26 feet measured from the approximate ground surface at the locations of Borings VC-1 and VC-3.

Subsurface conditions (as characterized by field and laboratory data) and earthquake characteristics were used to estimate the safety factors against liquefaction in each soil layer, as well as the associated dynamic settlement during the design seismic event. Based on the analysis, there is potential for liquefaction at the site. The analysis results are presented in Table 6.

Table 6. Results of Liquefaction Analyses.

Boring No.	Depth of Boring (feet)	Depth Intervals with Liquefaction Factor of Safety Less than 1.0		Estimated Dynamic Settlement (inches)	
		Depth (feet)	Elevation	Upper 50 Feet	Total Depth of Boring
VC-1	100	68-78	199-189	0	2
VC-2	80	78-80	174-172	0	2
VC-3	120	28-33	239-234	3	3

Soils considered susceptible to liquefaction are saturated, loose sand, gravel, and low plasticity silt (ML; PI < 7) and silty clay (CL-ML; PI < 5). The liquefiable layer identified in Boring VC-3 was classified as a soft, silt layer with a PI of 5 percent. However, liquefaction potential is estimated in a single soil sample in one boring indicating limited vertical and horizontal extent at the site. These soil deposits are Pleistocene aged and based on that age are considered to have a low liquefaction susceptibility. Therefore, it is our opinion is the site has a low potential for liquefaction.

The current state of practice for liquefaction hazard assessment is based on what is known as “the Simplified Method” as introduced by Seed (1971) and subsequent modifications/revisions by many researchers (Seed 1982, Idriss 1999, Youd 2001, and Idriss and Boulanger 2014, among others). The simplified method was based on observations and assessments of soil zones that either liquefied or did not liquefy in the upper 40 feet (12 m). There are reported uncertainties in the values of one of the inputs to the method (the stress reduction factor, or r_d) at depths greater than 50 feet. The occurrence of significant liquefaction in deeper sand deposits is unlikely. Therefore, we recommend not considering potentially liquefiable zones below a depth of 50 feet when determining pile embedment lengths. A discussion of the downdrag potential due to dynamic settlement is included in a subsequent section.

Liquefaction hazard mitigation can be be accomplished using compaction piles (large displacement piles) or proprietary ground improvement techniques such as earthquake drains or stone columns. Proprietary ground improvement techniques are typically performed by specialty firms on a design/build basis.

<u>Lateral Spreading</u>. Lateral spreading is triggered and sustained by earthquake ground motions. Based on our seismic slope stability analyses, it is our professional opinion the potential for lateral

spreading is low. Geotechnology evaluated this condition, and more information is provided in the Global Stability section of this report.

Approach Embankment Settlement

Based on the plans provided, it appears up to approximately 6 feet of fill will be required at the proposed abutments to bring the site to grade; we have assumed cohesive, engineered fill will be used for the fill material. Settlement analyses were performed to assess fill-induced settlement for the approaches at each site. The results of the settlement analyses are shown in Table 7. If grade changes will require the placement of additional fill, Geotechnology should be contacted to perform additional settlement analyses for fill-induced settlement at the approaches.

Table 7. Summary of Estimated Settlement.

South Abutment		North Abutment	
Max Fill (feet)	Estimated Settlement (inches)	Max Fill (feet)	Estimated Settlement (inches)
3	Less than 1	6	1

Discussion of Fill-Induced Settlement. The results of the settlement analyses indicate up to 1 inch of total settlement. We anticipate practical completion of settlement to occur within 2 to 3 weeks of fill placement.

Global Stability

Based on the provided plans, abutment fill will be placed at a 2.5H:1V slope and side slopes will be constructed at 3H:1V and 4H:1V slopes. We have assumed cohesive, lean clay will be used for the fill material. Geotechnology performed stability analyses for deep-seated, global failure of bridge abutment slopes using the computer program Slide. Short-term, long-term, seismic, and post seismic (residual strength) conditions were considered using the Spencer method and the software Slide by RocScience to compute factors of safety for the proposed slopes.

The models used in this computation did not consider the relative stabilizing effect of foundation piles driven to support the abutments or cladding of abutments with rip rap or concrete. In general, foundation piles may provide additional stabilizing force to the abutment slopes, resulting in a factor of safety higher than those presented in Table 8.

Calculated minimum factors of safety are summarized in Table 8. A pseudo-static seismic acceleration of 0.302g, corresponding to one-half the peak ground acceleration (per FHWA Publication NHI-11-032) was utilized for the seismic condition. An estimated residual shear strength was used to model the potentially liquefiable silt layer in Boring VC-3 for the post-seismic condition. Section profiles with calculated critical failure surfaces and utilized soil parameters are presented in Appendix E for selected analyses.

Table 8. Results of Slope Stability Analyses.

Location	Description	Approximate Slope Height (feet)	Calculated Factor of Safety			
			Short-Term Static[a]	Long-Term Static[a]	Seismic[b]	Post-Seismic[b]
South Abutment Side Slope	3:1 Cut	6	5.2	1.9	1.8	--
South Abutment Spill Slope	2½:1 Cut | Fill	17	2.7	1.5	1.4	--
North Abutment Side Slope	3:1 Cut	9½	5.1	1.9	1.8	4.0
North Abutment Spill Slope	2½:1 Cut | Fill	17	3.2	1.5	1.1	1.7

[a] Target factor of safety = 1.5, approximately equivalent to a global stability resistance factor = 0.65.
[b] Target factor of safety = 1.1, approximately equivalent to a global stability resistance factor = 0.9.

Sufficient factors of safety were calculated for all conditions. However, the critical failure surfaces for the seismic condition are unrealistically large. The displacement that would occur as the result of a seismic event was evaluated using Newmark's simplified displacement method outlined in FHWA Publication NHI-11-032. This method requires a yield acceleration which is defined as the horizontal ground acceleration required to bring the factor of safety (capacity to demand ratio) to 1.0; the yield accelerations used in the displacement analysis were calculated using the software Slide by RocScience. Based on the results of the evaluation, the yield accelerations are 0.31 and 0.32g for the southern and northern spill slopes, respectively, and 0.54 and 0.6g for the southern and northern side slopes, respectively. Based on the Newmark simplified displacement method, displacements are not expected to be substantial.

Deep Foundations

Foundation design recommendations are provided herein based on the AASHTO LRFD Bridge Design Specifications (2014). It is our understanding concrete filled, closed-end, steel, pipe piles will be used for support of the proposed bridge. We understand 16- and 20-inch diameter piles will be driven at the abutments and intermediate supports, respectively. Geotechnology should be notified if a different foundation type is being considered.

Synthetic profiles have been compiled for each abutment and bent locations based upon the soil profile encountered in the borings, approximate boring elevations, and the proposed final grade. Soil parameters, including LPILE parameters, for each structure are included in Appendix G.

Nominal resistance curves showing axial resistance from skin friction and total axial capacity (skin friction + end bearing) for the bents and abutments are presented in Appendix F. Nominal capacities at each bridge support are presented in Table 9. Uplift (tension) capacities may be calculated using the resistance provided by skin friction.

Table 9. Axial Pile Resistance.

Pile Diameter (inches)	Location	Embedment Length (feet)	Compression Total (tons)
16	South Abutment	70	200
		80	230
		90	260
	North Abutment	70	220
		80	330
		90	320
20	Center Bents	60	250
		70	290
		80	330

Resistance Factors. Resistance factors should be applied to the nominal resistances provided. Based solely on the static analysis methods used to calculate nominal pile resistances, the factors presented in Table 10 may be applied.

Table 10. Resistance Factors Based on Static Analysis Methods.

Deep Foundation and Condition	Clay		Sand	
	Side Resistance	End-Bearing	Side Resistance	End-Bearing
Nominal Compressive Resistance of Single Pile	0.35	0.35	0.45	0.45
Uplift Resistance of Single Pile	0.25	--	0.35	--

Based on AASHTO LRFD (2014) Table 10.5.5.2.3-1, a higher resistance factor can be used in accordance with the method of pile testing performed as indicated in Table 11.

Table 11. Resistance Factors for Driven Piles.

Condition/Resistance Determination Method		Resistance Factor
Nominal Bearing Resistance of Single Pile – Dynamic Analysis and Static Load Test Methods	Driving criteria established by successful static load test of at least one pile per site condition and dynamic testing of at least two piles per site, but no less than 2% of the production piles*	0.80
	Driving criteria established by successful static load test of at least one pile per site condition without dynamic testing	0.75
	Driving criteria established by dynamic testing conducted on 100% of production piles*	0.75
	Driving criteria established by dynamic testing, quality control by dynamic testing of at least two piles per site condition, but no less than 2% of production piles*	0.65
	Wave equation analysis, without pile dynamic measurements or load test but with field confirmation of hammer performance	0.50
	FHWA-modified Gates dynamic pile formula (End of Drive condition only)	0.40
Uplift Resistance of Single Pile	Dynamic test with signal matching	0.50

* Dynamic testing requires signal matching, and estimates of nominal resistance are made from a restrike. Dynamic tests are calibrated to a static load test, when available.

Pile Group Considerations. The settlement of pile groups should be evaluated as per AASHTO LRFD (2014) section 10.7.2.3. Settlement analysis of the pile groups can be performed when the foundation configurations and service loads are available. AASHTO LRFD (2014) section 10.7.3.9 addresses pile group resistance. Group capacity considerations for different pile groups, center-to-center spacings, and other conditions (cap contact with ground, softness of surface soil, etc.) are given in AASHTO LRFD (2014) sections 10.7.3.9 and 10.7.3.11.

Driven Pile Construction Considerations. Minimum hammer energies required to drive the piles were evaluated using the computer software WEAP. The recommended minimum hammer energies for each pile type are provided in Table 12.

Table 12. Minimum Hammer Energies.

Pile Diameter [a] (inches)	Location	Embedment Length (feet)	Required Capacity (tons/kips)	Minimum Rated Hammer Energy (kip-feet)
16	North and South Abutment (Bent Nos. 1 and 4)	63	142/284	14
20	Center Bents (Bent Nos. 2 and 3)	73	239/478	28

[a] Closed-ended pipe piles with ½-inch thick walls.

Piles should be driven with a pile hammer developing appropriate energy that will not cause damage to the pile. Alternatively, potential driving criteria can be developed using wave equation analyses after the pile hammer is selected.

Static Pile Load Testing. At least one static pile compression load test should be performed for each bent or abutment location. The testing should be performed in accordance with ASTM D 1143 using the quick loading procedure and AASHTO LRFD (2014) section 10.7.3.8.2. Please refer to the previous Resistance Factors table for additional guidance regarding the minimum number of tests and alternate resistance factors associated with other field methods for determining resistance.

If the piles are to support net uplift loads, at least one tension load test should be performed for each location. The test should be performed in accordance with ASTM D 3689. Piles should be tested to the required nominal uplift resistances.

Load tests are required to verify recommended nominal pile resistance and will not be used to increase the design pile resistance. The piles used in the load tests should not be used for support of any structures. Geotechnology should be consulted regarding the locations of the test piles.

Dynamic Testing of Driven Piles. As an alternative to static pile load testing, high-strain dynamic pile testing can be performed according to AASHTO LRFD (2014) section 10.7.3.8.3 and the procedures given in ASTM D4945. Different resistance factors correspond to different load testing combinations as illustrated in the previous table. We recommend that the test piles be identified according to AASHTO LRFD (2014) Table 10.5.5.2.3-1 or 2 percent of the production piles, whichever results in a larger number of tests. We recommend that the identified piles be tested at the end of initial drive (EOID) and a restrike performed at a minimum seven days after EOID.

Pile driving monitoring should be performed by an engineer with a minimum three years dynamic pile testing and analysis experience and who has achieved Basic or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA. Pile driving modeling and analyses should be performed by an engineer with a minimum five years dynamic pile testing and analysis experience and who has achieved Advanced or better certification under the High-Strain Dynamic Pile Testing Examination and Certification process of the Pile Driving Contractors Association and Foundation QA.

Dynamic tests are required to monitor hammer and drive system performance, assess driving stresses and structural integrity and to evaluate pile resistance, and should not be used to increase design pile resistance. Dynamic tests should be performed on production piles with the lowest driving resistance. Geotechnology will be available to assist with development of specifications for this program and should be on site to perform or observe the testing and establish the pile driving criteria.

Settlement. Settlement of pile foundations depends on the loads applied and the foundation configuration. In general, settlement of deep foundations designed in accordance with the recommendations provided in this report is not expected to exceed 1-inch. However, a calculation of the expected settlement of the pile foundations can be performed when the applied service loads and foundation configuration are available.

Uplift Resistance. Uplift forces can be resisted by the effective weight of the piles and caps, and frictional resistance between the piles and surrounding soil. If the anticipated maximum level of groundwater is higher than the tip of the pile then the buoyant unit weight of the pile must be used in computing uplift resistance for pile lengths extending below the design groundwater level.

Lateral Resistance. The lateral resistance of pile foundations depends on the length and dimensions of the foundation and the soil characteristics. The lateral resistance of pile foundations can be computed using the computer program LPILE to model the behavior of a single pile or shaft. Soil parameters are provided in Appendix G for the various strata and soil strengths present at the site. Soil parameters are based on field and laboratory test results and empirical correlations with SPT N-values.

The effects of group interaction must be considered when evaluating pile/shaft group horizontal movement. The lateral resistance for individual piles calculated by LPILE must be reduced by the P-multipliers provided in Section 10.7.2.4 of the AASHTO LRFD (2014) to determine lateral resistance of a pile group. Alternatively, the GROUP software can be used to evaluate the lateral resistance of the pile/shaft groups. The resistance factor for lateral resistance of single piles or pile groups is 1.0.

Downdrag

The AASHTO LRFD (2014) suggests that settlement of 0.4-inch or greater could produce downdrag on pile foundations. Downdrag occurs as the soil strata move downward relative to the foundations due to settlement of the soil layers. The relative movement of the soil layers versus the shaft depends on the final foundation configuration.

Downdrag Due to Fill-Induced Settlement. Based on settlement analysis performed for the 6-foot maximum fill placement at the abutments, up to 1 inch of settlement is predicted. Pile driving should not begin until settlement is practically complete, which is estimated to be approximately 2 to 3 weeks after fill placement. Piles driven immediately after fill placement will be subject to drag loads as the soil consolidates due to the weight of the fill.

Downdrag Due to Dynamic Settlement. Based on liquefaction analysis results, up to 3 inches of dynamic settlement was estimated within the upper 50 feet of soil of the northern abutment during the design earthquake event. However, due to the reasons stated on page 8 of this report, it is our professional opinion liquefaction potential is low at this site, hence liquefaction-induced drag loads should not be considered.

Pre-drilling or applying bituminous or viscous coatings are not recommended to reduce liquefaction-induced downdrag because such methods will reduce the nominal static compressive resistance of the piles. If potential downdrag forces are not tolerable, consideration should be given to methods which mitigate dynamic settlement by reducing pore pressure. Such techniques are performed by specialty contractor; if more information is desired, please contact Geotechnology.

7.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on: Geotechnology's understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend Geotechnology be included in the final design and construction process, and be retained to review the project plans and specifications to confirm the recommendations given in this report have been correctly implemented. We recommend Geotechnology be retained to participate in pre-bid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the subject project.

Since actual subsurface conditions between boring locations could vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

8.0 LIMITATIONS

This report has been prepared on behalf of, and for the exclusive use of, the client for specific application to the named project as described herein. If this report is provided to other parties, it should be provided in its entirety with all supplementary information. In addition, the client should make it clear the information is provided for factual data only, and not as a warranty of subsurface conditions presented in this report.

Geotechnology has attempted to conduct the services reported herein in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions. The recommendations and conclusions contained in this report are professional opinions. The report is not a bidding document and should not be used for that purpose.

Our scope for this phase of the project did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site. Any statements in this report or on the boring logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client. Our scope did not include an assessment of the effects of flooding and erosion of creeks or rivers adjacent to or on the project site.

Our scope did not include: any services to investigate or detect the presence of mold or any other biological contaminants (such as spores, fungus, bacteria, viruses, and the by-products of such organisms) on and around the site; or any services, designed or intended, to prevent or lower the risk of the occurrence of an infestation of mold or other biological contaminants.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from the geotechnical exploration. The field exploration methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Consequently, subsurface conditions could vary gradually, abruptly, and/or nonlinearly between sample locations and/or intervals.

The conclusions or recommendations presented in this report should not be used without Geotechnology's review and assessment if the nature, design, or location of the facilities is changed, if there is a lapse in time between the submittal of this report and the start of work at the site, or if there is a substantial interruption or delay during work at the site. If changes are contemplated or delays occur, Geotechnology must be allowed to review them to assess their impact on the findings, conclusions, and/or design recommendations given in this report. Geotechnology will not be responsible for any claims, damages, or liability associated with any other party's interpretations of the subsurface data or with reuse of the subsurface data or engineering analyses in this report.

The recommendations included in this report have been based in part on assumptions about variations in site stratigraphy that can be evaluated further during earthwork and foundation construction. Geotechnology should be retained to perform construction observation and continue its geotechnical engineering service using observational methods. Geotechnology cannot assume liability for the adequacy of its recommendations when they are used in the field without Geotechnology being retained to observe construction.

Appendix A
IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

Important Information about This

Geotechnical-Engineering Report

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

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

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

Understand the Geotechnical-Engineering Services Provided for this Report

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

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

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

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

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

Do <u>not</u> rely on this report if your geotechnical engineer prepared it:

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

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

Read this Report in Full

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

You Need to Inform Your Geotechnical Engineer About Change

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

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

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

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

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

This Report's Recommendations Are Confirmation-Dependent

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

This Report Could Be Misinterpreted

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

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

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

Give Constructors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are <u>Not</u> Covered

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

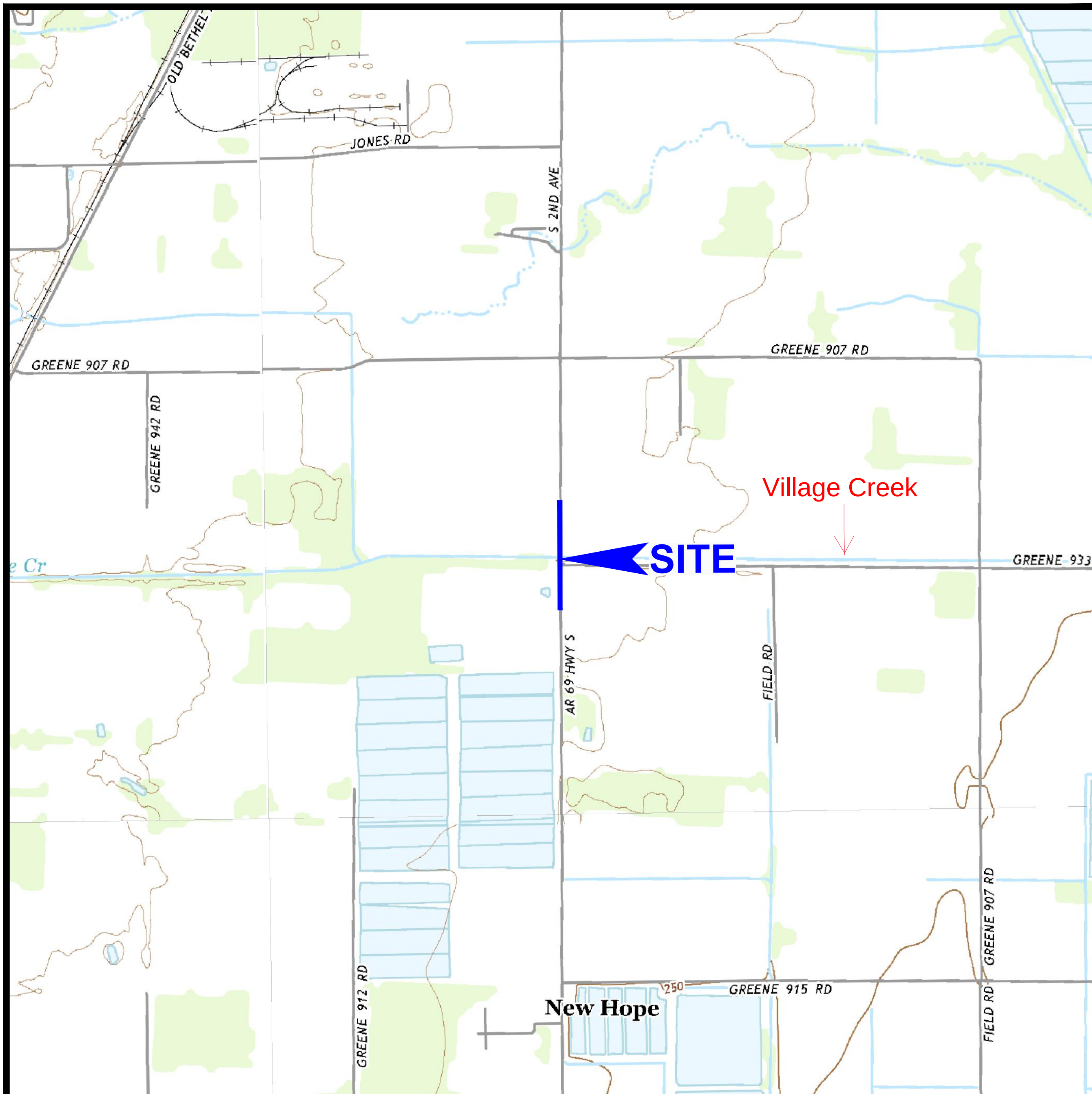
Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

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

GEOPROFESSIONAL BUSINESS ASSOCIATION

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

Appendix B
FIGURES

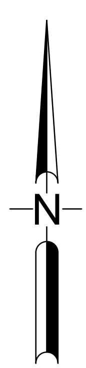


NOTES

1. Plan adapted from 7.5 minute U.S.G.S. maps for Paragould East, Pargould West, Brookland and Dixie, Arkansas quadrangles, last revised in 2014.

0 2,000 4,000

SCALE IN FEET



Drawn By: WAH	Ck'd By: ASM	App'vd By: DBA
Date: 2-20-20	Date: 3-20-20	Date: 3-20-20

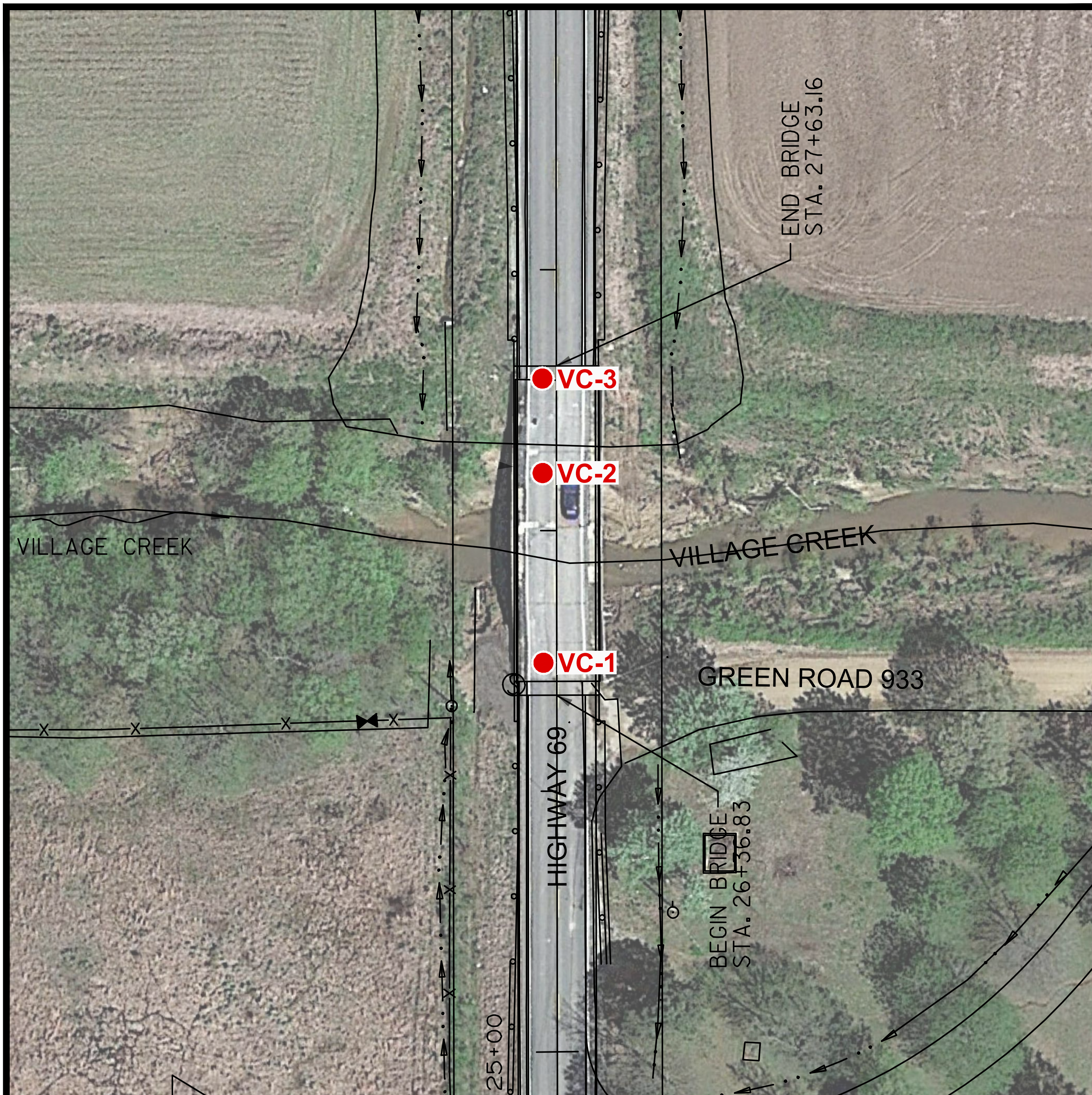
GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Project No. 101000
Highway 69 Bridge over Village Creek
Greene County, Arkansas

SITE LOCATION AND TOPOGRAPHY

Project Number J034363.01

FIGURE 1



NOTES

1. Plan adapted from an April 11, 2019 aerial photograph courtesy of Google Earth and a drawing dated December 16, 2019, titled "Plan and Profile - Highway 69", supplied by the client.

2. Borings were located in the field with reference to site features and are shown approximate only.

LEGEND

● Boring Location

0 60 120

SCALE IN FEET

N

Drawn By: WAH	Ck'd By: ASM	App'vd By: DBA
Date: 2-20-20	Date: 3-20-20	Date: 3-20-20

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Project No. 101000
Highway 69 Bridge over Village Creek
Greene County, Arkansas

AERIAL PHOTOGRAPH OF SITE AND BORING LOCATIONS

Project Number
J034363.01

FIGURE 2

Appendix C
BORING INFORMATION

Surface Elevation: 267	Completion Date: 1/30/20
Datum msl	Station: 26+30
	Offset: -8

DESCRIPTION OF MATERIAL

Depth in feet: 5, 10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60, 65, 70, 75, 80, 85, 90, 95, 100

Elevation in feet: 262, 257, 252, 247, 242, 237, 232, 227, 222, 217, 212, 207, 202, 197, 192, 187, 182, 177, 172, 167

Asphalt: 7 inches

Base Materials: 14 inches of gravel and sand

Very stiff to medium stiff, red and brown, LEAN CLAY - (CL)
trace sand
little sand, trace organics
88% passing No. 200 sieve
resistivity = 1,482 ohms-cm
pH = 4.82

Brown and gray, FAT CLAY - (CH)
pH = 5.06
resistivity = 1,026 ohms-cm
trace sand
98% passing No. 200 sieve

Medium stiff to very stiff, brown and gray, LEAN CLAY - (CL)
trace sand
98% passing No. 200 sieve
pH = 6.65
trace sand and organics
96% passing No. 200 sieve
resistivity = 1,767 ohms-cm
pH = 8.32
trace sand
99% passing No. 200 sieve
pH = 2.37
trace sand
93% passing No. 200 sieve

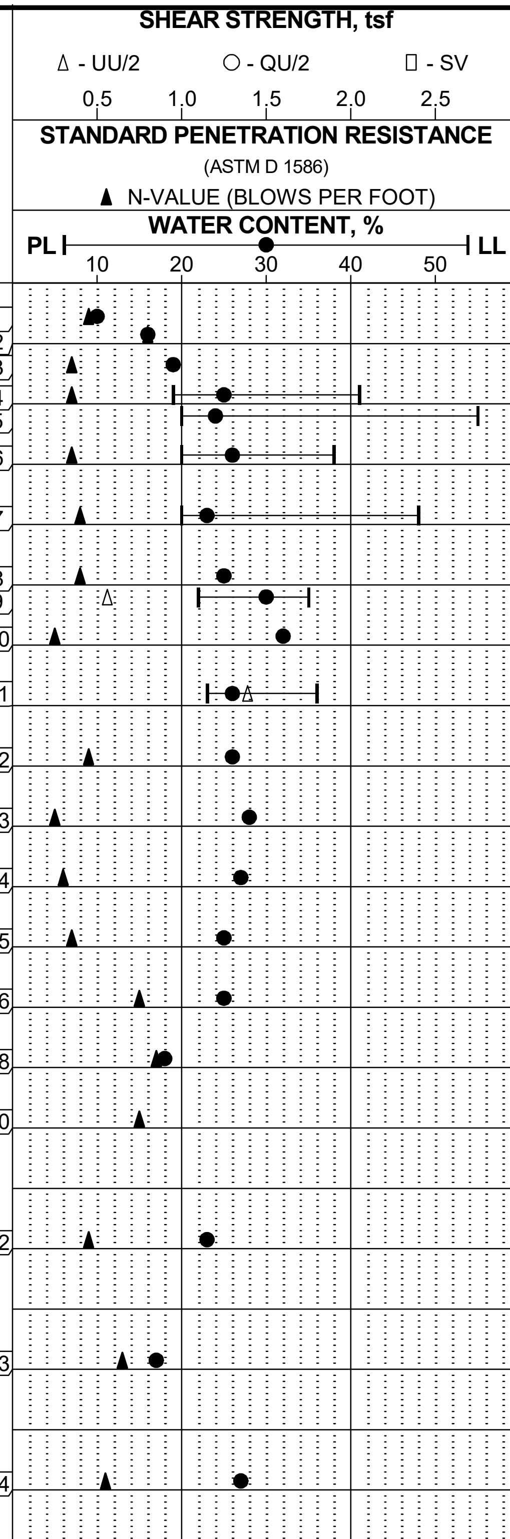
Medium dense, gray, CLAYEY SAND - (SC)
48% passing No. 200 sieve

Stiff, gray, LEAN CLAY - CL

Stiff, gray, sandy, LEAN CLAY - CL

Stiff, tan, FAT CLAY - CH
Boring terminated at 100 feet.

DRY UNIT WEIGHT (pcf) / SPT BLOW COUNTS / CORE RECOVERY/RQD	SAMPLES
2-4-5	SS1
4-7-9	SS2
3-2-5	SS3
3-3-4	SS4
101	ST5
3-3-4	SS6
2-4-4	SS7
3-3-5	SS8
94	ST9
3-3-2	SS10
102	ST11
5-4-5	SS12
2-2-3	SS13
2-3-3	SS14
3-3-4	SS15
4-6-9	SS16
4-9-8	SS18
4-8-7	SS20
3-4-5	SS22
2-5-8	SS23
4-5-6	SS24



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

ENCOUNTERED AT 26 FEET ∇

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 35 FEET
JCG DRILLER DLB LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS: Elevation provided by Garver in feet and assumed to reference mean sea level (msl).

Drawn by: AIM	Checked by: ASM	App'vd. by: DBA
Date: 2/3/20	Date: 3/10/20	Date: 3/10/20

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas

LOG OF BORING: VC-1

Geotechnology Project No. J034363.01

LOG OF BORING 2002 WL J034363.01.GPJ GTINC 0638301.GPJ 4/8/20

Surface Elevation: 267	Completion Date: 2/3/20
Datum msl	Station: 27+09
	Offset: -7

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	GRAPHIC LOG	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Height from top of bridge deck to ground surface.			
5	262				
10	257				
15	252	Gray, CLAYEY SAND, some gravel - (SC) 20% passing No. 200 sieve resistivity = 4,788 ohms-cm pH = 6.40		1-1-0	SS1
20	247				
25	242	Medium stiff to very stiff, brown to brown and gray, LEAN CLAY - (CL) silt pockets			
30	237	trace sand and organics 97% passing No. 200 sieve		4-5-6	SS2
35	232	resistivity = 1,653 ohms-cm pH = 8.44		3-2-3	SS3
40	227	resistivity = 1,995 ohms-cm pH = 8.81		3-6-2	SS4
45	222			4-4-5	SS5
50	217			2-2-2	SS6
55	212			1-2-4	SS7
60	207	with sand, trace organics 83% passing No. 200 sieve		3-4-4	SS8
65	202	little sand		4-6-10	SS9
70	197				
75	192			2-4-3	SS10
80	187				
85	182	Medium stiff, brown, FAT CLAY, trace sand - CH		2-3-3	SS11
90	177				
95	172	Medium dense, gray, SILTY SAND, trace clay - SM Boring terminated at 95 feet from top of bridge deck (80 feet from ground surface).		9-8-7	SS12
100	167				

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 □ - SV

0.5 1.0 1.5 2.0 2.5

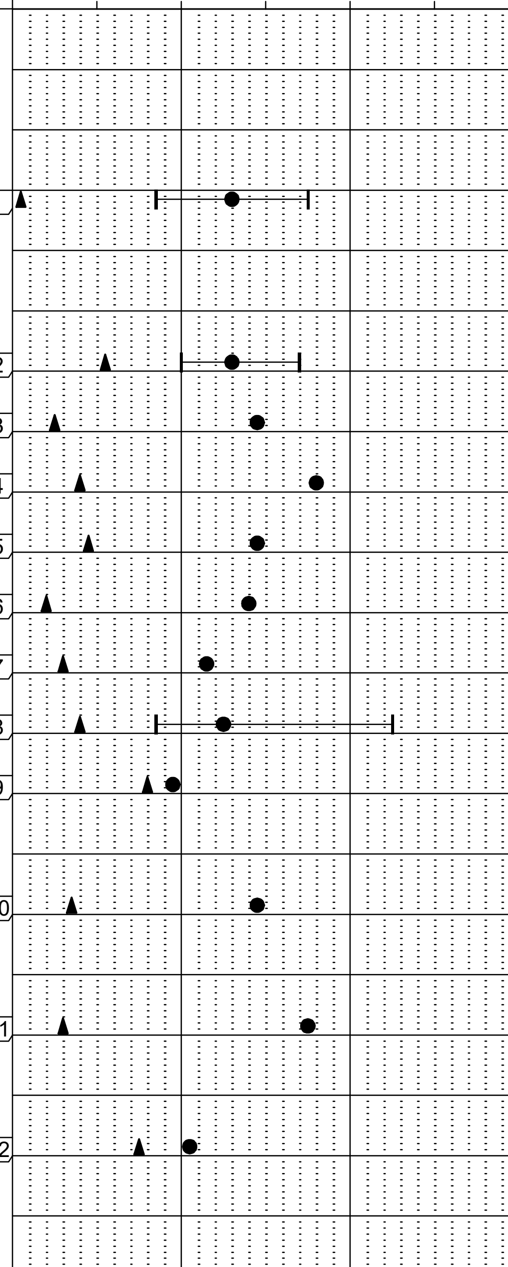
STANDARD PENETRATION RESISTANCE (ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ● LL

10 20 30 40 50



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 15 FEET

JCG DRILLER DLB LOGGER

CME 55 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 92 %

REMARKS: Boring drilled through bridge deck using a casing extending 15 feet to the ground surface.
Elevation provided by Garver in feet and assumed to reference mean sea level (msl).

Drawn by: AIM	Checked by: ASM	App'vd. by: DBA
Date: 2/6/20	Date: 3/10/20	Date: 3/10/20

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas

LOG OF BORING: VC-2

Geotechnology Project No. J034363.01

Surface Elevation: 267

Datum: msl

Completion Date: 1/28/20

Station: 27+61

Offset: -6

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
		Asphalt: 9 inches		
		Base Materials: 18 inches of gravel and sand	112	ST1
		Red and brown, CLAYEY SAND - (SC) 29% passing No. 200 sieve	3-6-4	SS2
5	262	Stiff to medium stiff, brown to gray, LEAN CLAY - (CL)	104	ST3
		3 inches of wood, some sand with sand	102	ST4
10	257	74% passing No. 200 sieve		
		trace sand 97% passing No. 200 sieve	2-2-3	SS5
15	252	trace sand 97% passing No. 200 sieve resistivity = 1,140 ohms-cm pH = 6.82	3-5-6	SS6
20	247	trace sand and organics 98% passing No. 200 sieve pH = 7.77		
25	242	Stiff to soft, brown SILT - (ML) trace organics resistivity = 2,565 ohms-cm pH = 8.27	4-8-7	SS7
30	237	100% passing No. 200 sieve pH = 8.65	2-2-2	SS8
		Soft, brown, LEAN CLAY - (CL)	2-2-2	SS9
35	232	Stiff, brown SILT - (ML)	96	ST10
40	227	Medium stiff to very stiff, gray, LEAN CLAY - (CL)	3-5-4	SS11
			101	ST12
45	222			
			2-3-4	SS13
50	217		107	ST14
55	212	Medium stiff, gray, sandy, LEAN CLAY, trace gravel - (CL)	2-3-4	SS15
60	207	Stiff to hard, gray to brown and gray, LEAN CLAY - (CL)	5-5-6	SS16
			108	ST17

SHEAR STRENGTH, tsf

Δ - UU/2 ○ - QU/2 ▯ - SV

0.5 1.0 1.5 2.0 2.5

STANDARD PENETRATION RESISTANCE

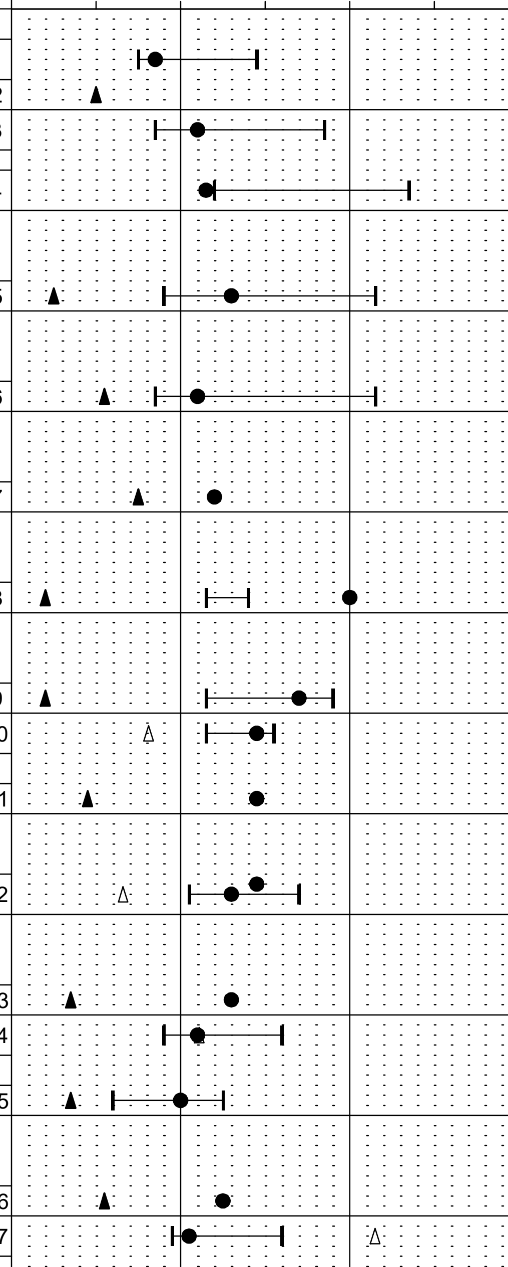
(ASTM D 1586)

▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %

PL ● LL

10 20 30 40 50



NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

___ AUGER 3 3/4" HOLLOW STEM

WASHBORING FROM 10 FEET

JCG DRILLER DLB LOGGER

CME 55 DRILL RIG

HAMMER TYPE Auto

HAMMER EFFICIENCY 92 %

REMARKS: Elevation provided by Garver in feet and assumed to reference mean sea level (msl).

Drawn by: AIM	Checked by: ASM	App'vd. by: DBA
Date: 1/29/20	Date: 3/10/20	Date: 3/10/20

GEOTECHNOLOGY INC FROM THE GROUND UP

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas

LOG OF BORING: VC-3

Geotechnology Project No. J034363.01

Surface Elevation: 267
Datum: msl
Completion Date: 1/28/20
Station: 27+61
Offset: -6

SHEAR STRENGTH, tsf: Δ - UU/2 ○ - QU/2 ▯ - SV (0.5, 1.0, 1.5, 2.0, 2.5)

STANDARD PENETRATION RESISTANCE (ASTM D 1586) ▲ N-VALUE (BLOWS PER FOOT)

WATER CONTENT, %: PL ├──●──┤ LL (10, 20, 30, 40, 50)

DEPTH IN FEET	ELEVATION IN FEET	DESCRIPTION OF MATERIAL	DRY UNIT WEIGHT (pcf) SPT BLOW COUNTS CORE RECOVERY/RQD	SAMPLES
65	202	Stiff to hard, gray to brown and gray, LEAN CLAY - (CL) *(continued)*	4-7-15	SS18
70	197	Medium dense, gray and tan, CLAYEY SAND - (SC) 43% passing No. 200 sieve	11-10-9	SS19
75	192	Medium dense, tan GRAVEL with sand and clay - (GP-GC) 7% passing No. 200 sieve	30-19-11	SS20
80	187	Medium stiff, brown and gray, LEAN CLAY, trace sand - CL	3-4-4	SS21
85	182	Medium stiff to very soft, brown and gray to gray, sandy, FAT CLAY, trace gravel - CH	3-3-3	SS22
90	177		0-0-1	SS23
95	172	Stiff, gray, sandy, LEAN CLAY - CL	5-6-9	SS24
100	167	Stiff to medium stiff, gray, FAT CLAY - CH trace sand 91% passing No. 200 sieve	7-5-6	SS25
105	162			
110	157	with sand 84% passing No. 200 sieve	4-3-5	SS6
115	152			
120	147	Medium dense, brown SAND with silt - (SP-SM) 10% passing No. 200 sieve Boring terminated at 120 feet.	7-8-10	SS7
125	142			

NOTE: STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARIES BETWEEN SOIL TYPES AND THE TRANSITION MAY BE GRADUAL. GRAPHIC LOG FOR ILLUSTRATION PURPOSES ONLY.

GROUNDWATER DATA

X FREE WATER NOT ENCOUNTERED DURING DRILLING

DRILLING DATA

__ AUGER 3 3/4" HOLLOW STEM
WASHBORING FROM 10 FEET
JCG DRILLER DLB LOGGER
CME 55 DRILL RIG
HAMMER TYPE Auto
HAMMER EFFICIENCY 92 %

REMARKS: Elevation provided by Garver in feet and assumed to reference mean sea level (msl).

Drawn by: AIM	Checked by: ASM	App'vd. by: DBA
Date: 1/29/20	Date: 3/10/20	Date: 3/10/20

GEOTECHNOLOGY INC
FROM THE GROUND UP

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas

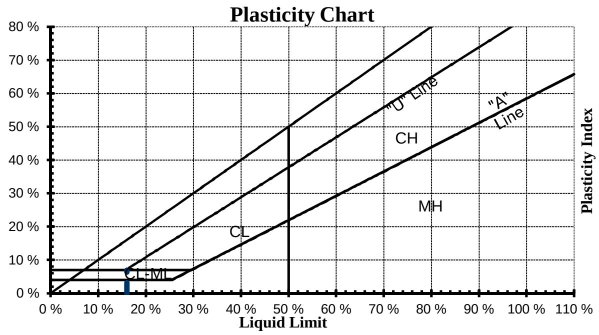
CONTINUATION OF LOG OF BORING: VC-3

Geotechnology Project No. J034363.01

BORING LOG: TERMS AND SYMBOLS

LEGEND

CS	Continuous Sampler
GB	Grab Sample
NQ	NQ Rock Core
PST	Three-Inch Diameter Piston Tube Sample
SS	Split-Spoon Sample (Standard Penetration Test)
ST	Three-Inch Diameter Shelby Tube Sample
*	Sample Not Recovered
PL	Plastic Limit (ASTM D4318)
LL	Liquid Limit (ASTM D4318)
SV	Shear Strength from Field Vane (ASTM D2573)
UU	Shear Strength from Unconsolidated-Undrained Triaxial Compression Test (ASTM D2850)
QU	Shear Strength from Unconfined Compression Test (ASTM D2166)



SOIL GRAIN SIZE

US STANDARD SIEVE

12" 3" 3/4" 4 10 40 200

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		COARSE	FINE	COARSE	MEDIUM	FINE		

300 76.2 19.1 4.76 2.00 0.42 0.074 0.005

SOIL GRAIN SIZE IN MILLIMETERS

UNIFIED SOIL CLASSIFICATION SYSTEM

Major Divisions			***Symbol***	***Description***
Coarse-Grained Soils (More than 50% Larger than No. 200 Sieve Size)	Gravel and Gravelly Soil	Clean Gravels Little or no Fines	GW	Well-Graded Gravel, Gravel- Sand Mixture
			GP	Poorly-Graded Gravel, Gravel-Sand Mixture
		Gravels with Appreciable Fines	GM	Silty Gravel, Gravel-Sand-Silt Mixture
			GC	Clayey-Gravel, Gravel-Sand-Clay Mixture
	Sand and Sandy Soils	Clean Sands Little or no Fines	SW	Well-Graded Sand, Gravelly Sand
			SP	Poorly-Graded Sand, Gravelly Sand
		Sands with Appreciable Fines	SM	Silty Sand, Sand-Silt Mixture
			SC	Clayey-Sand, Sand-Clay Mixture
Fine-Grained Soils (More than 50% Smaller than No. 200 Sieve Size)	Silts and Clays	Liquid Limit Less Than 50	ML	Silt, Sandy Silt, Clayey Silt, Slight Plasticity
			CL	Lean Clay, Sandy Clay, Silty Clay, Low to Medium Plasticity
			OL	Organic Silts or Lean Clays, Low Plasticity
	Silts and Clays	Liquid Limit Greater Than 50	MH	Silt, High Plasticity
			CH	Fat Clay, High Plasticity
			OH	Organic Clay, Medium to High Plasticity
	Highly Organic Soils		PT	Peat, Humus, Swamp Soil

STRENGTH OF COHESIVE SOILS

Consistency	***Undrained Shear Strength (tsf)***	***Unconfined Comp. Strength (tsf)***
Very Soft	less than 0.125	less then 0.25
Soft	0.125 to 0.25	0.25 to 0.5
Medium Stiff	0.25 to 0.5	0.5 to 1.0
Stiff	0.5 to 1.0	1.0 to 2.0
Very Stiff	1.0 to 2.0	2.0 to 3.0
Hard	greater than 2.0	greater than 4.0

DENSITY OF GRANULAR SOILS

Descriptive Term	***Approximate N_{60}-Value Range***
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	>50

N-Value (Blow Count) is the last two, 6-inch drive increments (i.e. 4/7/9, N = 7 + 9 = 16). Values are shown as a summation on the grid plot and shown in the Unit Dry Weight/SPT column.

RELATIVE COMPOSITION

Trace	0 to 10%
Little	10 to 20%
Some	20 to 35%
And	35 to 50%

OTHER TERMS

Layer - Inclusion greater than 3 inches thick.

Seam - Inclusion 1/8-inch to 3 inches thick

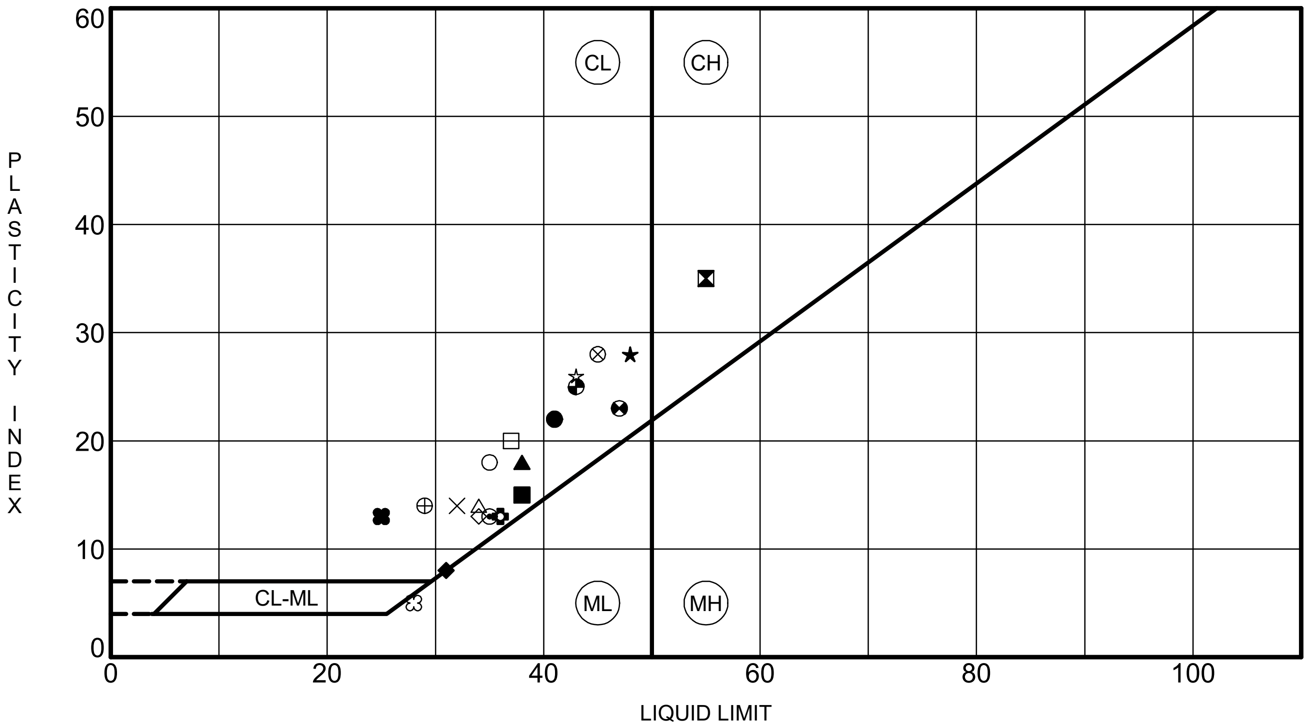
Parting - Inclusion less than 1/8-inch thick

Pocket - Inclusion of material that is smaller than sample diameter

GEOTECHNOLOGY INC
FROM THE GROUND UP

Relative composition and Unified Soil Classification System (USCS) designations are based on visual descriptions and are approximate only. If laboratory tests were performed to classify the soil, the USCS designation is shown in parenthesis.

Appendix D
LABORATORY TEST DATA

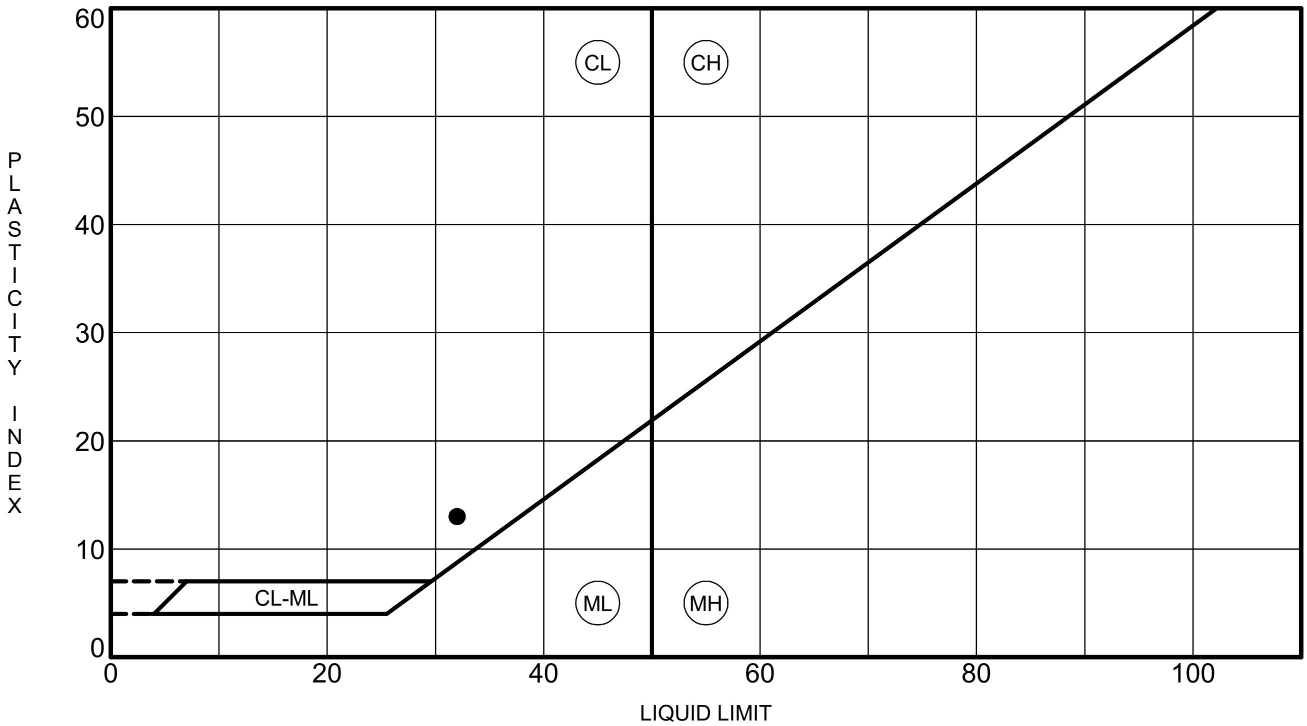


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**VC-1**	**8.5**	**41**	**19**	**22**	**88**	**LEAN CLAY(CL), A-7-6 (20)**
⧗	**VC-1**	**10.0**	**55**	**20**	**35**	**98**	**FAT CLAY(CH), A-7-6 (38)**
▲	**VC-1**	**13.5**	**38**	**20**	**18**	**98**	**LEAN CLAY(CL), A-6 (0)**
★	**VC-1**	**18.5**	**48**	**20**	**28**	**98**	**LEAN CLAY(CL), A-7-6 (30)**
⊙	**VC-1**	**25.0**	**35**	**22**	**13**	**99**	**LEAN CLAY(CL), A-6 (14)**
✿	**VC-1**	**33.0**	**36**	**23**	**13**	**93**	**LEAN CLAY(CL), A-6 (13)**
○	**VC-2**	**15.0**	**35**	**17**	**18**	**20**	**CLAYEY SAND(SC), A-2-6 (0)**
△	**VC-2**	**28.5**	**34**	**20**	**14**	**97**	**LEAN CLAY(CL), A-6 (14)**
⊗	**VC-2**	**58.5**	**45**	**17**	**28**	**83**	**LEAN CLAY with SAND(CL), A-7-6 (23)**
⊕	**VC-3**	**2.5**	**29**	**15**	**14**	**29**	**CLAYEY SAND(SC), A-2-6 (1)**
□	**VC-3**	**5.0**	**37**	**17**	**20**	**74**	**LEAN CLAY with SAND(CL), A-6 (13)**
◐	**VC-3**	**8.0**	**47**	**24**	**23**	**97**	**LEAN CLAY(CL), A-7-6 (25)**
◑	**VC-3**	**13.5**	**43**	**18**	**25**	**97**	**LEAN CLAY(CL), A-7-6 (26)**
☆	**VC-3**	**18.5**	**43**	**17**	**26**	**98**	**LEAN CLAY(CL), A-7-6 (27)**
✣	**VC-3**	**28.5**	**28**	**23**	**5**	**100**	**SILT(ML), A-4 (5)**
■	**VC-3**	**33.5**	**38**	**23**	**15**		**LEAN CLAY(CL), A-6**
◆	**VC-3**	**35.0**	**31**	**23**	**8**		**SILT(ML), A-4**
◇	**VC-3**	**43.0**	**34**	**21**	**13**		**LEAN CLAY(CL), A-6**
×	**VC-3**	**50.0**	**32**	**18**	**14**		**LEAN CLAY(CL), A-6**
✖	**VC-3**	**53.5**	**25**	**12**	**13**		**SANDY LEAN CLAY(CL), A-6**

GEOTECHNOLOGY INC
FROM THE GROUND UP

ATTERBERG LIMITS RESULTS

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas
J034363.01

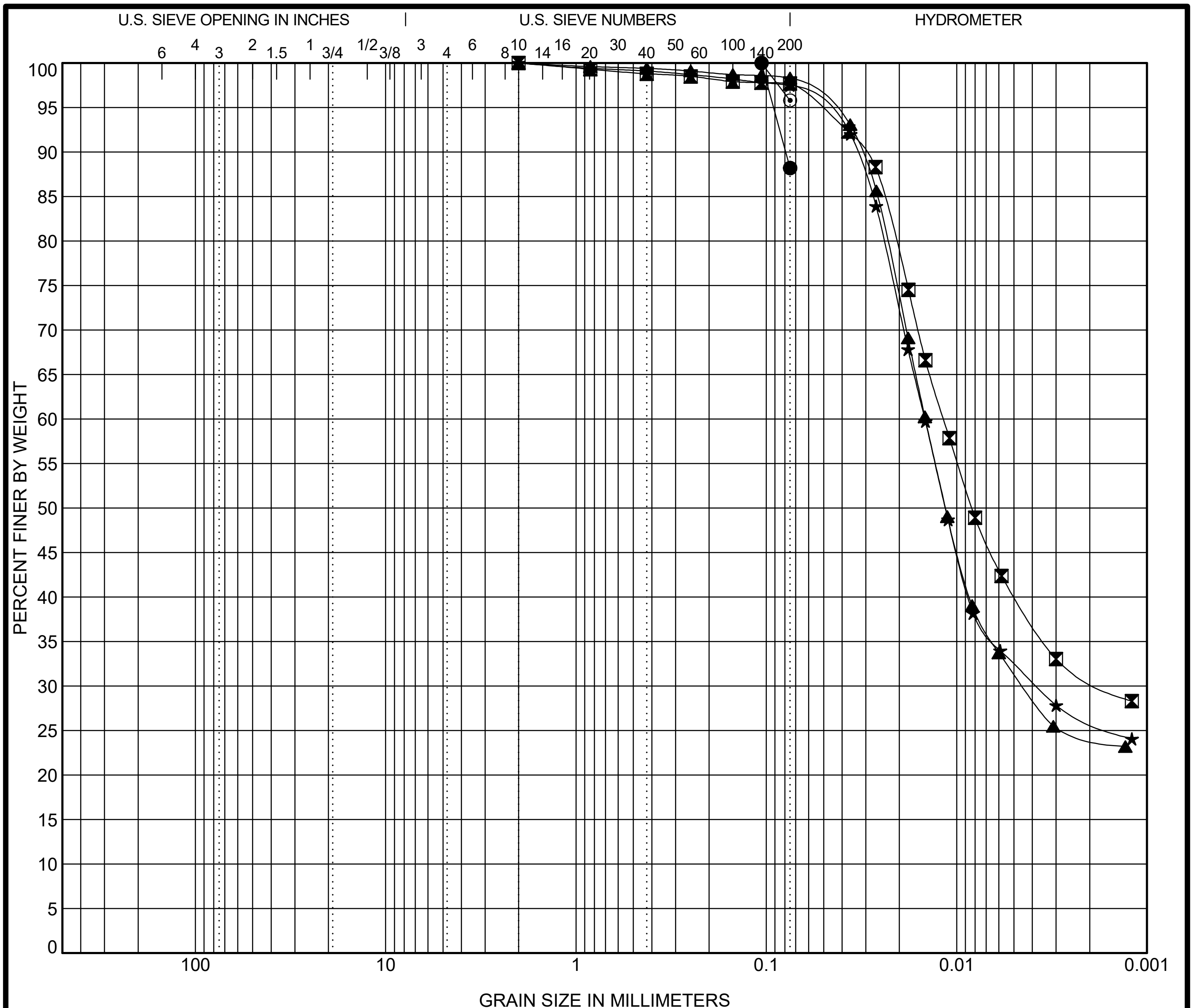


	Specimen Identification		LL	PL	PI	Fines	Classification
●	**VC-3**	**60.0**	**32**	**19**	**13**		**LEAN CLAY(CL), A-6**

GEOTECHNOLOGY INC
FROM THE GROUND UP

ATTERBERG LIMITS RESULTS

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas
J034363.01



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	VC-1	8.5	LEAN CLAY(CL), A-7-6 (20)	41	19	22		
⧗	VC-1	10.0	FAT CLAY(CH), A-7-6 (0)	55	20	35		
▲	VC-1	13.5	LEAN CLAY(CL), A-6 (0)	38	20	18		
★	VC-1	18.5	LEAN CLAY(CL), A-7-6 (0)	48	20	28		
⊙	VC-1	23.5	LEAN CLAY(CL), A-6					

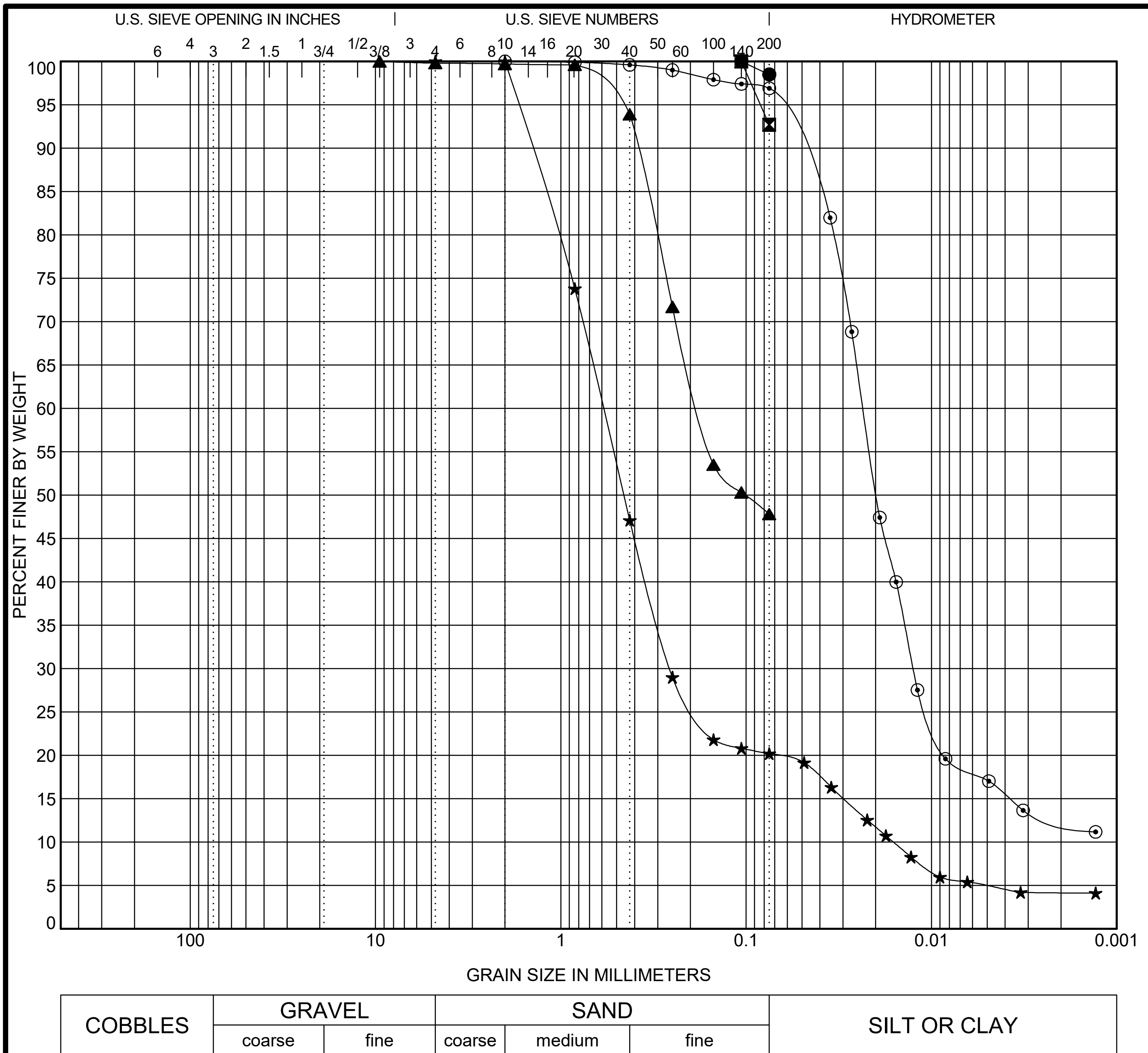
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	VC-1	8.5	0.106				0.0	11.8	88.2	
⧗	VC-1	10.0	2	0.012	0.002		0.0	2.3	57.4	40.3
▲	VC-1	13.5	2	0.015	0.004		0.0	1.7	66.9	31.4
★	VC-1	18.5	2	0.015	0.004		0.0	2.5	65.0	32.5
⊙	VC-1	23.5	0.106				0.0	4.2	95.8	

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas
J034363.01

US GRAIN SIZE J034363.01.GPJ US LAB.GDT 4/2/20

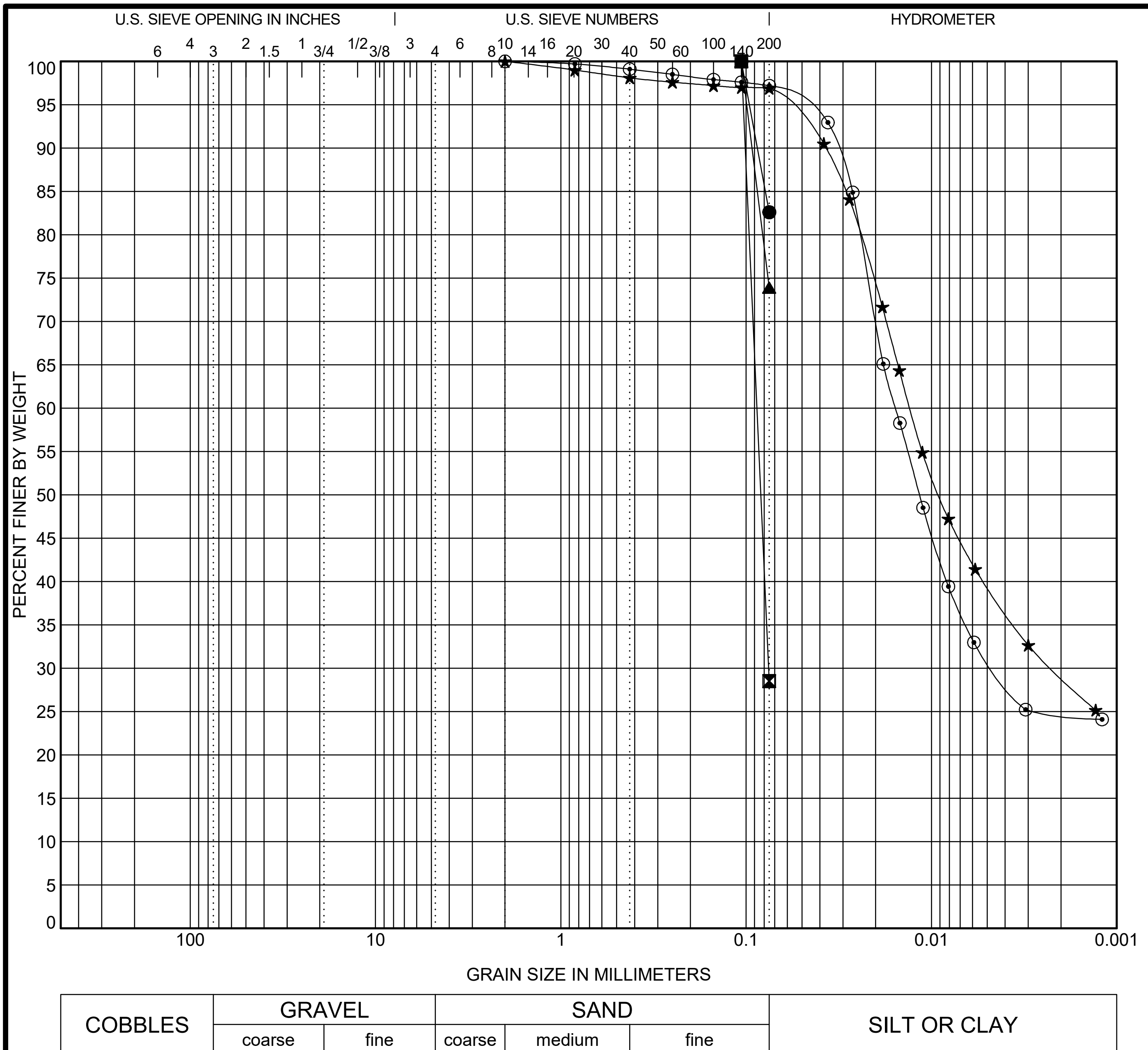


Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**VC-1**	**25.0**	**LEAN CLAY(CL), A-6 (14)**	**35**	**22**	**13**		
⧗	**VC-1**	**33.0**	**LEAN CLAY(CL), A-6 (13)**	**36**	**23**	**13**		
▲	**VC-1**	**68.5**	**CLAYEY SAND(SC), A-6**					
★	**VC-2**	**15.0**	**CLAYEY SAND(SC), A-2-6 (0)**	**35**	**17**	**18**	**6.99**	**36.82**
⊙	**VC-2**	**28.5**	**LEAN CLAY(CL), A-6 (14)**	**34**	**20**	**14**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**VC-1**	**25.0**	**0.106**				**0.0**	**1.5**	**98.5**	
⧗	**VC-1**	**33.0**	**0.106**				**0.0**	**7.3**	**92.7**	
▲	**VC-1**	**68.5**	**9.5**	**0.18**			**0.2**	**52.0**	**47.8**	
★	**VC-2**	**15.0**	**4.75**	**0.591**	**0.257**	**0.016**	**0.0**	**79.8**	**15.2**	**5.0**
⊙	**VC-2**	**28.5**	**2**	**0.023**	**0.013**		**0.0**	**3.1**	**79.8**	**17.1**

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION
ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas
J034363.01



Specimen Identification		Classification	LL	PL	PI	Cc	Cu
● VC-2	58.5	**LEAN CLAY with SAND(CL), A-7-6 (23)**	45	17	28		
⧗ VC-3	2.5	**CLAYEY SAND(SC), A-2-6 (1)**	29	15	14		
▲ VC-3	5.0	**LEAN CLAY with SAND(CL), A-6 (13)**	37	17	20		
★ VC-3	8.0	**LEAN CLAY(CL), A-7-6 (25)**	47	24	23		
⊙ VC-3	13.5	**LEAN CLAY(CL), A-7-6 (26)**	43	18	25		

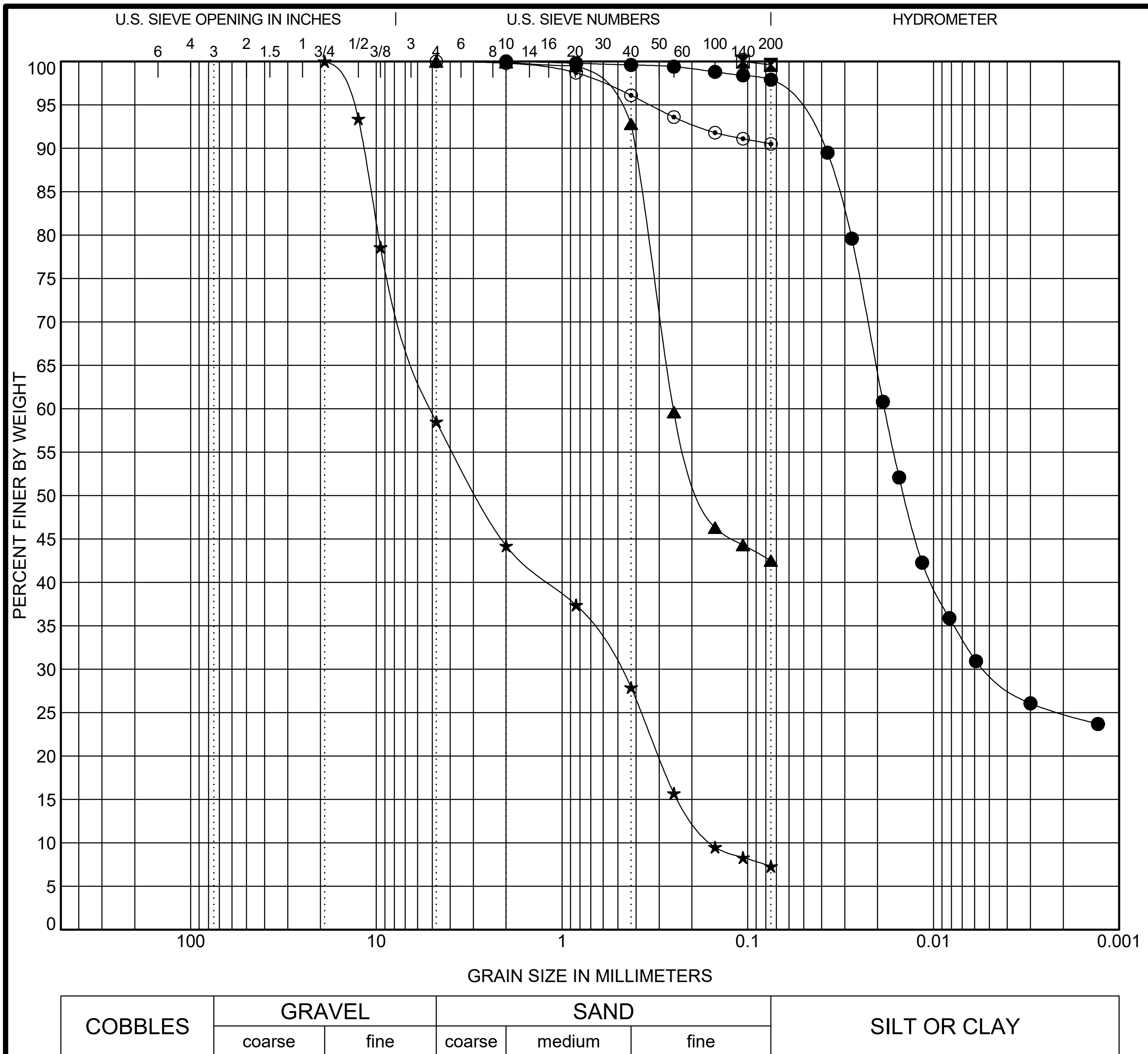
Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
● VC-2	58.5	0.106				0.0	17.4	82.6	
⧗ VC-3	2.5	0.106	0.087	0.076		0.0	71.5	28.5	
▲ VC-3	5.0	0.106				0.0	26.1	73.9	
★ VC-3	8.0	2	0.013	0.002		0.0	3.1	57.4	39.5
⊙ VC-3	13.5	2	0.016	0.005		0.0	2.8	66.2	31.0

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas
J034363.01

US GRAIN SIZE J034363.01.GPJ US LAB.GDT 4/2/20



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	VC-3	18.5	LEAN CLAY(CL), A-7-6 (27)	43	17	26		
⧗	VC-3	28.5	SILT(ML), A-4 (5)	28	23	5		
▲	VC-3	68.5	CLAYEY SAND(SC), A-6					
★	VC-3	73.5	P GRADED GRAVEL with SAND and CLAY(GP-GC), A-2-6				0.31	32.00
⊙	VC-3	98.5	FAT CLAY(CH), A-7-6					

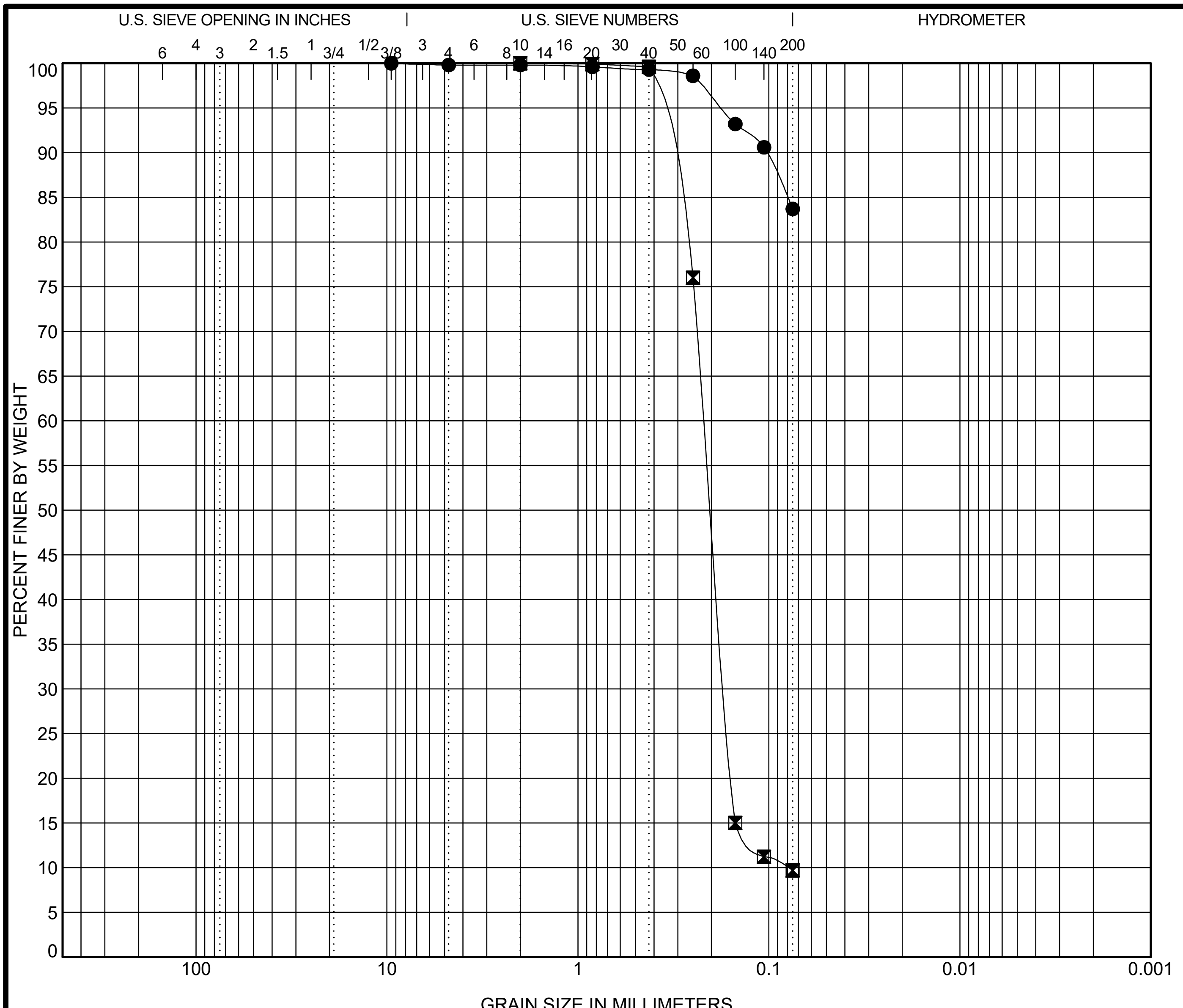
Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	VC-3	18.5	2	0.018	0.005		0.0	2.1	68.2	29.7
⧗	VC-3	28.5	0.106				0.0	0.4	99.6	
▲	VC-3	68.5	4.75	0.252			0.0	57.5	42.5	
★	VC-3	73.5	19	5.002	0.494	0.156	41.5	51.2	7.3	
⊙	VC-3	98.5	4.75				0.0	9.5	90.5	

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION

ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas
J034363.01

US GRAIN SIZE J034363.01.GPJ US LAB.GDT 4/2/20



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

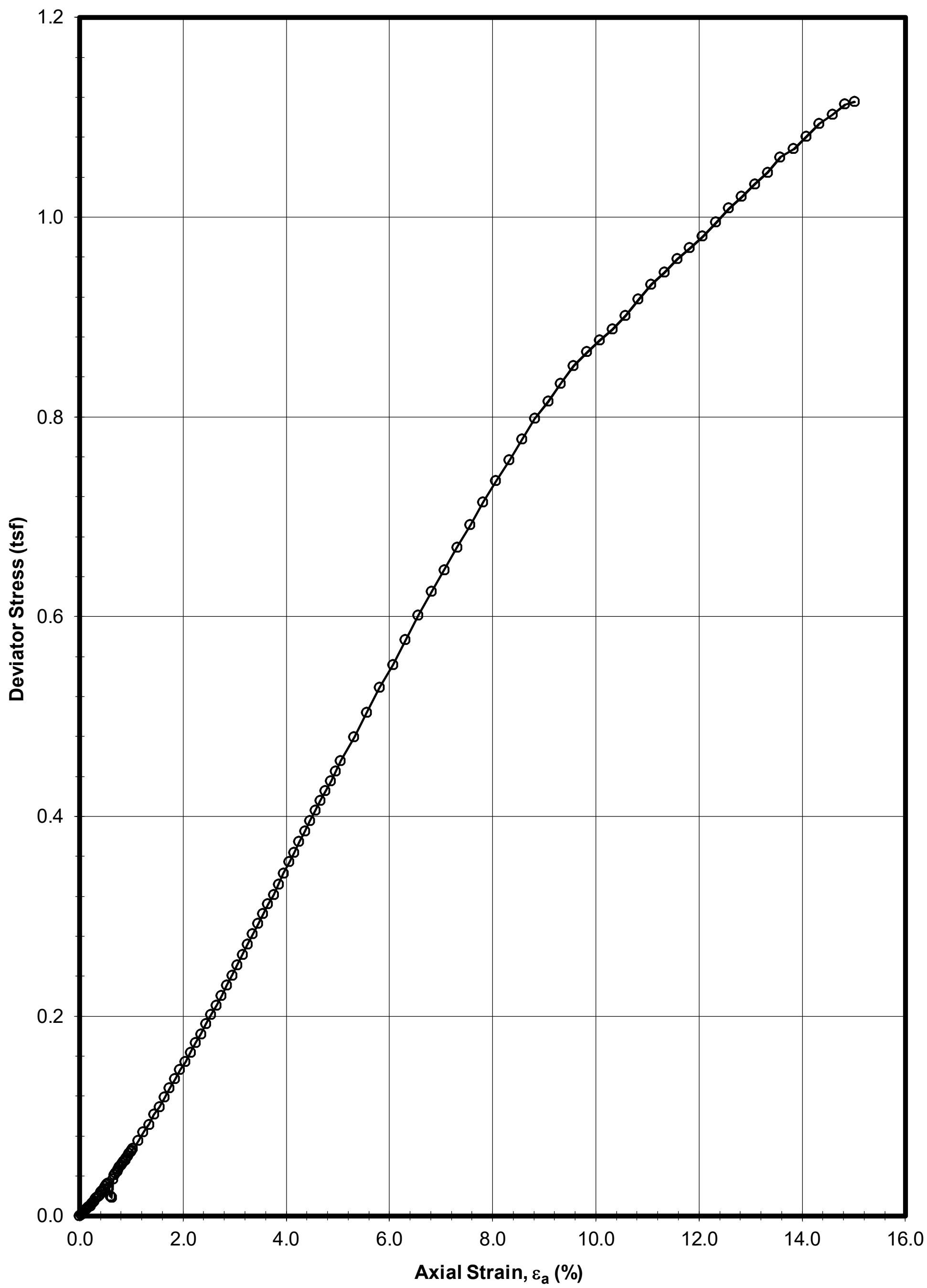
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	VC-3	108.5	**FAT CLAY with SAND(CH), A-7-6**					
⧗	VC-3	118.5	**POORLY GRADED SAND with SILT(SP-SM), A-3**				**1.65**	**2.72**

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	VC-3	108.5	**9.5**				**0.2**	**16.1**	**83.7**	
⧗	VC-3	118.5	**2**	**0.219**	**0.17**	**0.08**	**0.0**	**90.3**	**9.7**	

US_GRAIN_SIZE_J034363.01.GPJ_US_LAB.GDT_4/2/20

GEOTECHNOLOGY INC
FROM THE GROUND UP

GRAIN SIZE DISTRIBUTION
ARDOT Project No. 101000
Highway 69 Over Village Creek
Greene County, Arkansas
J034363.01



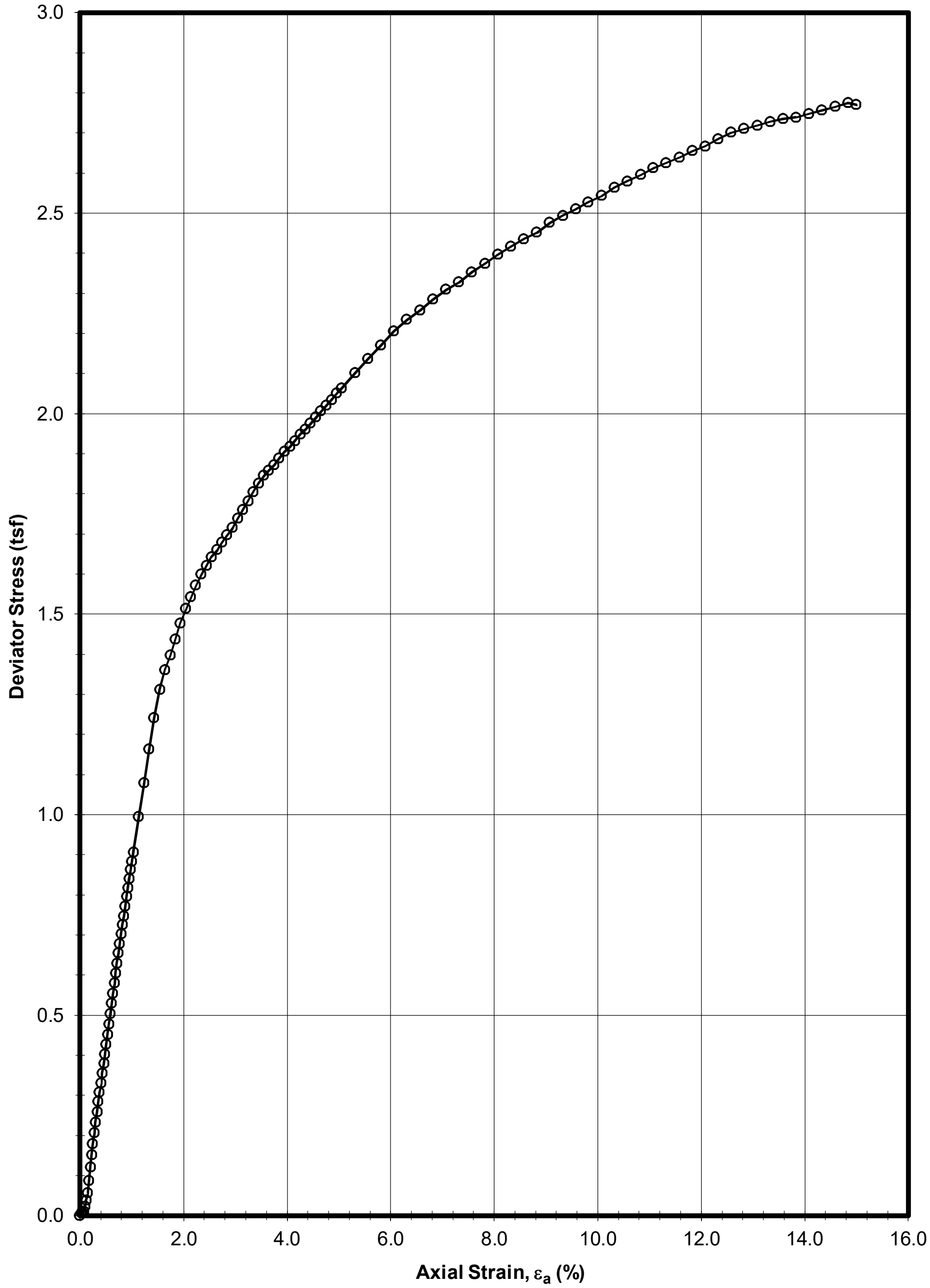
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034363.01

Boring: VC-1

Sample: ST-9 - Depth: 25 ft.



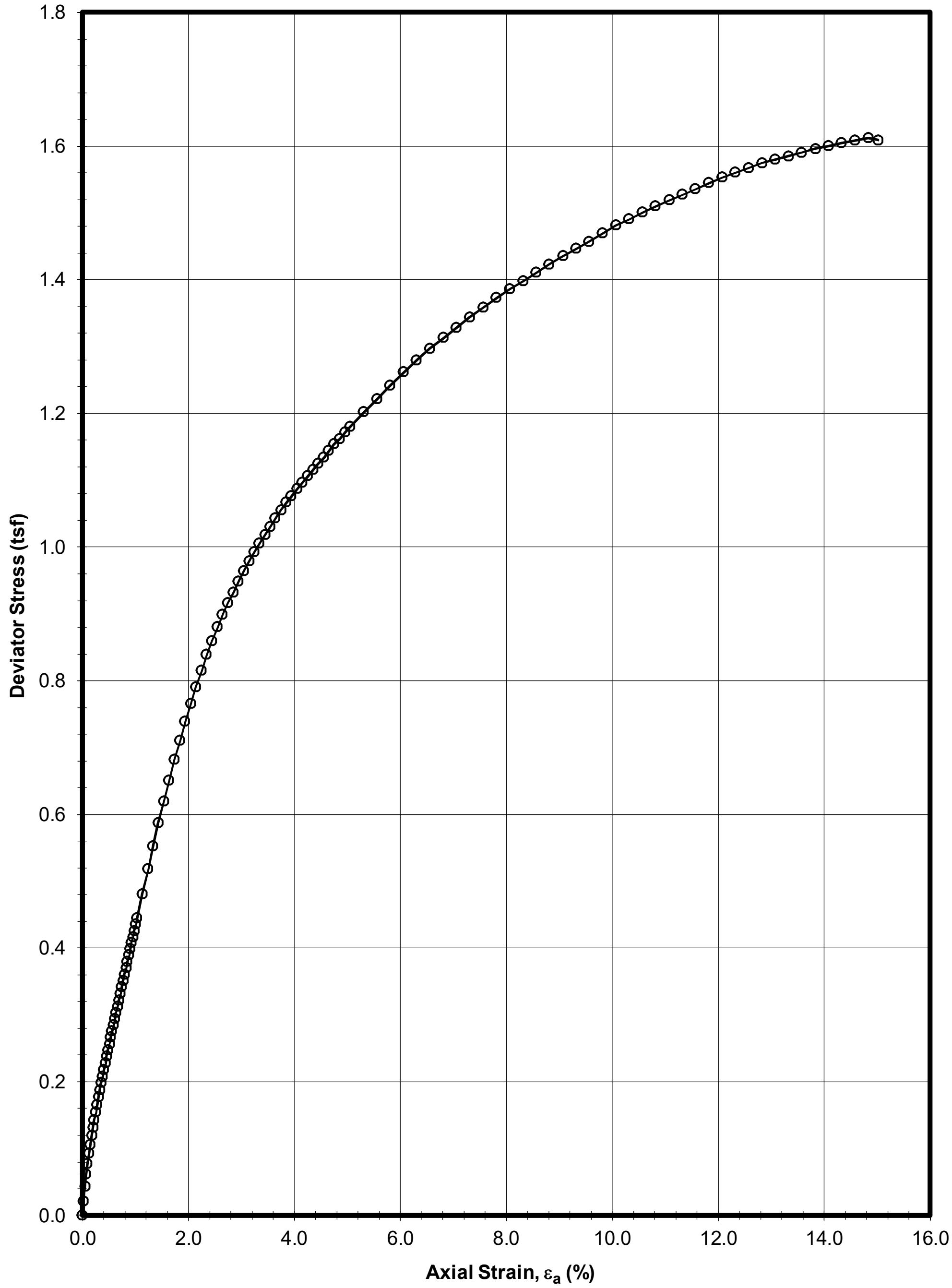
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034363.01

Boring: VC-1

Sample: ST-11 - Depth: 33 ft.



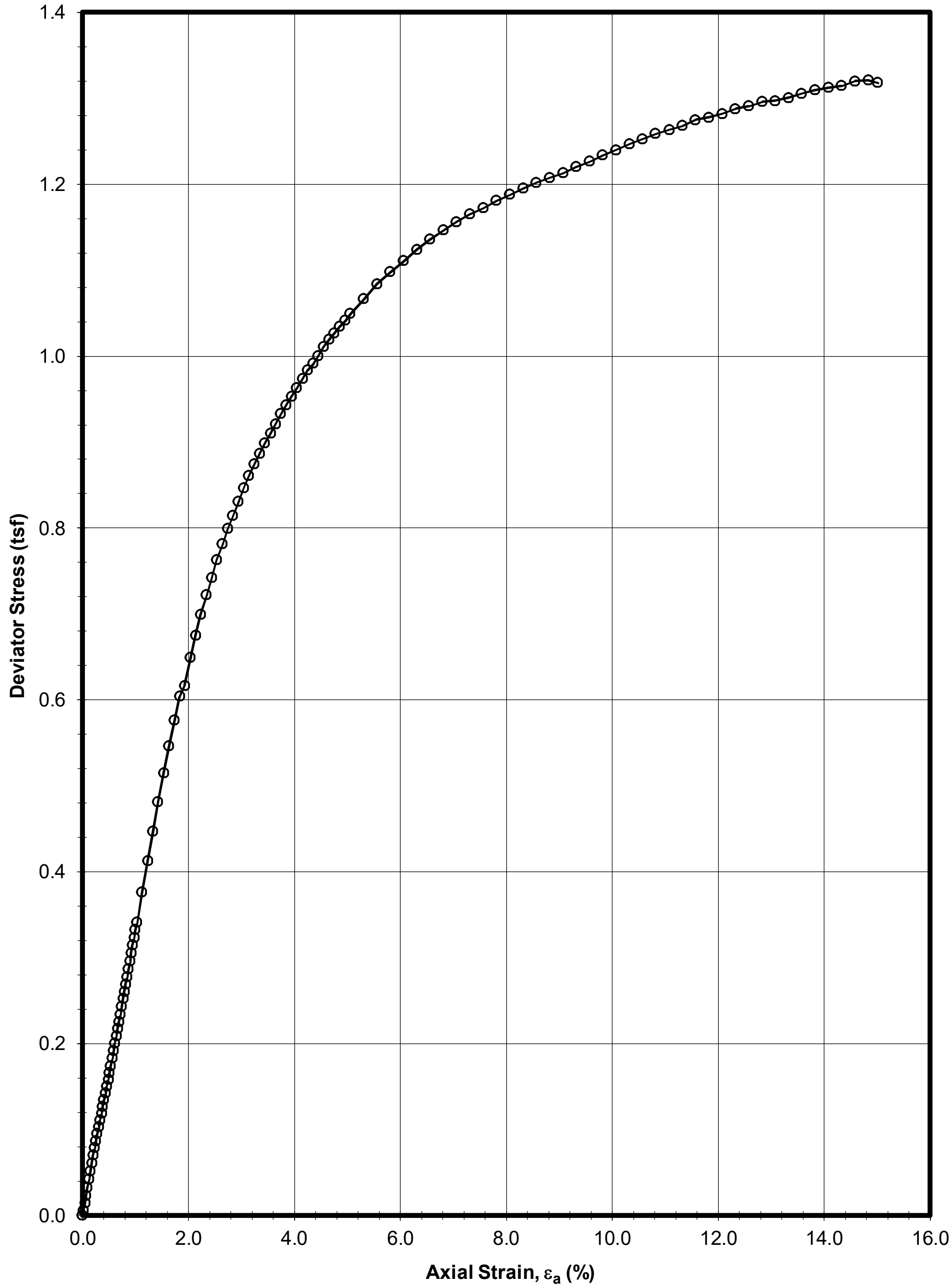
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034363.01

Boring: VC-3

Sample: ST-10 - Depth: 35 ft.



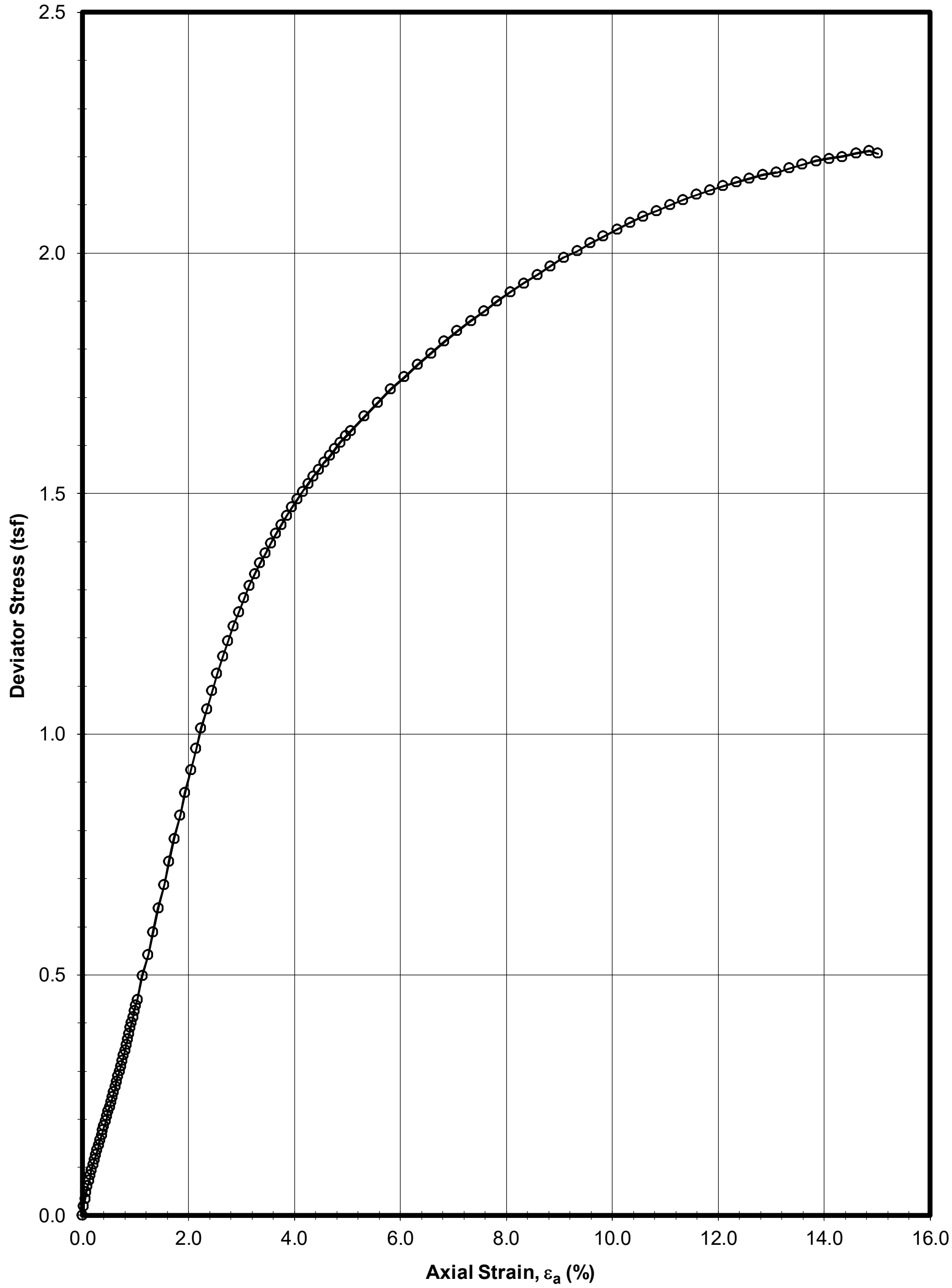
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034363.01

Boring: VC-3

Sample: ST-12 - Depth: 43 ft.



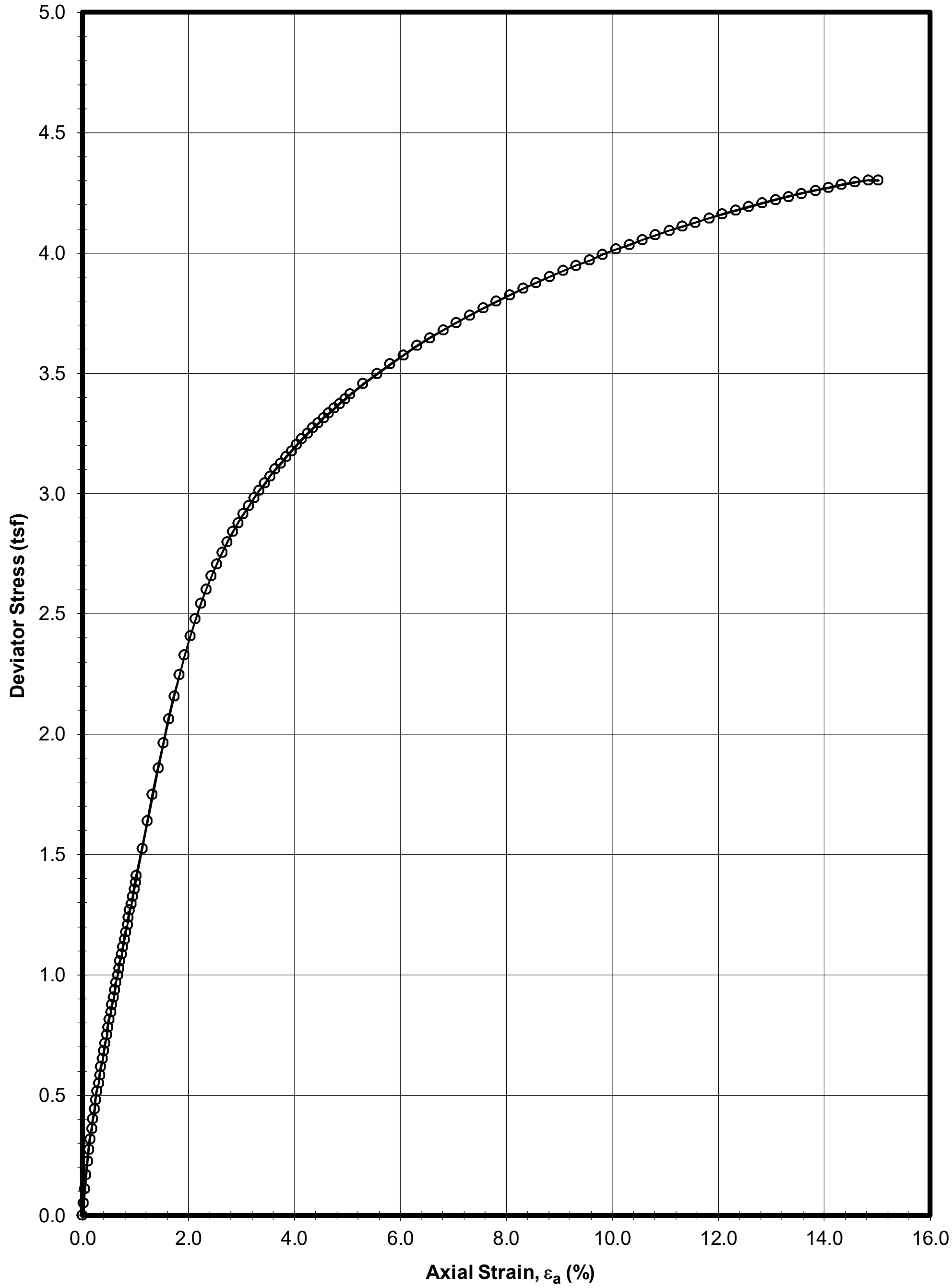
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.: J034363.01

Boring: VC-3

Sample: ST-14 - Depth: 50 ft.



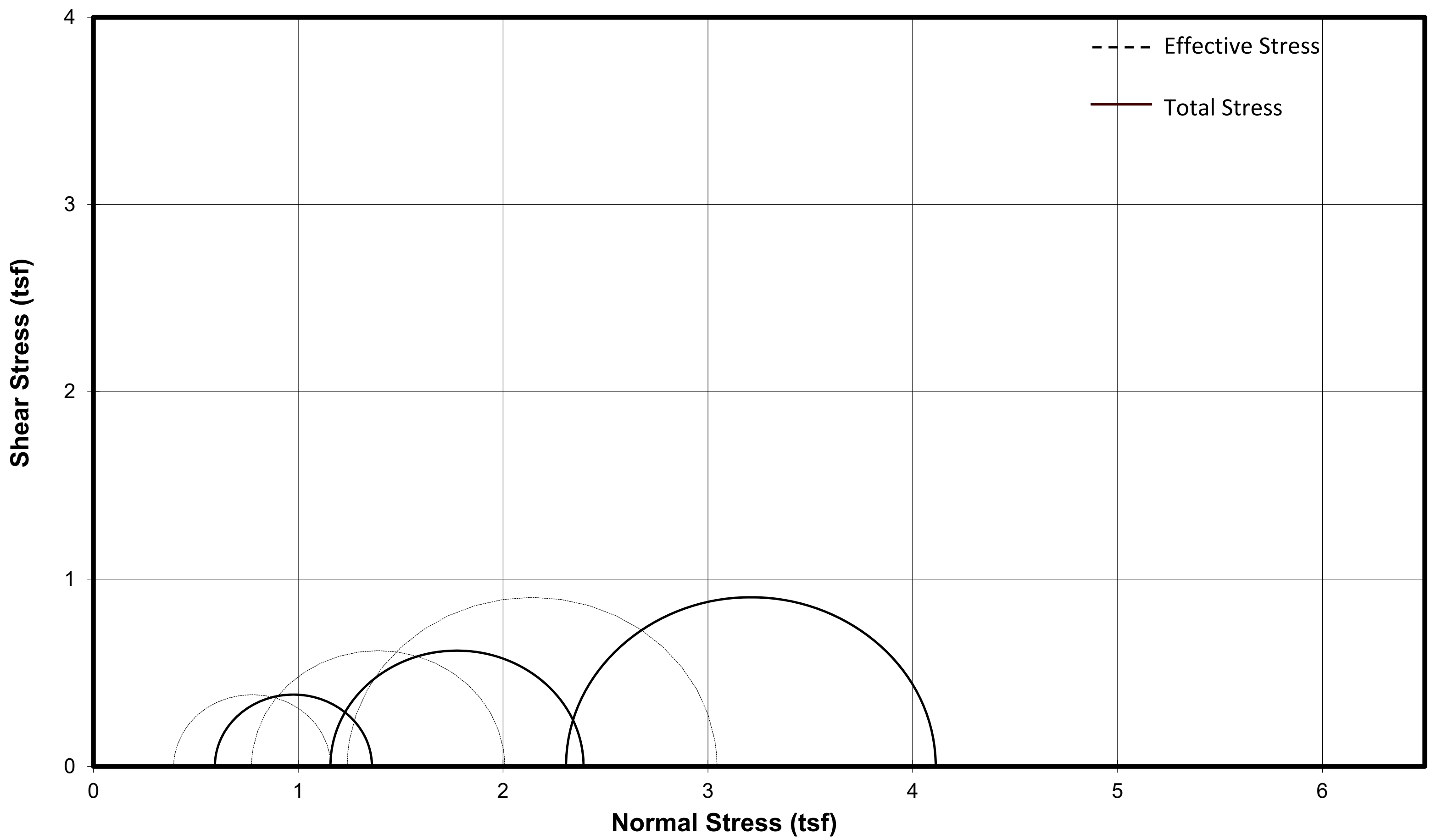
UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 2850

Project No.:

Boring: VC-3

Sample: ST-17 - Depth: 60 ft.



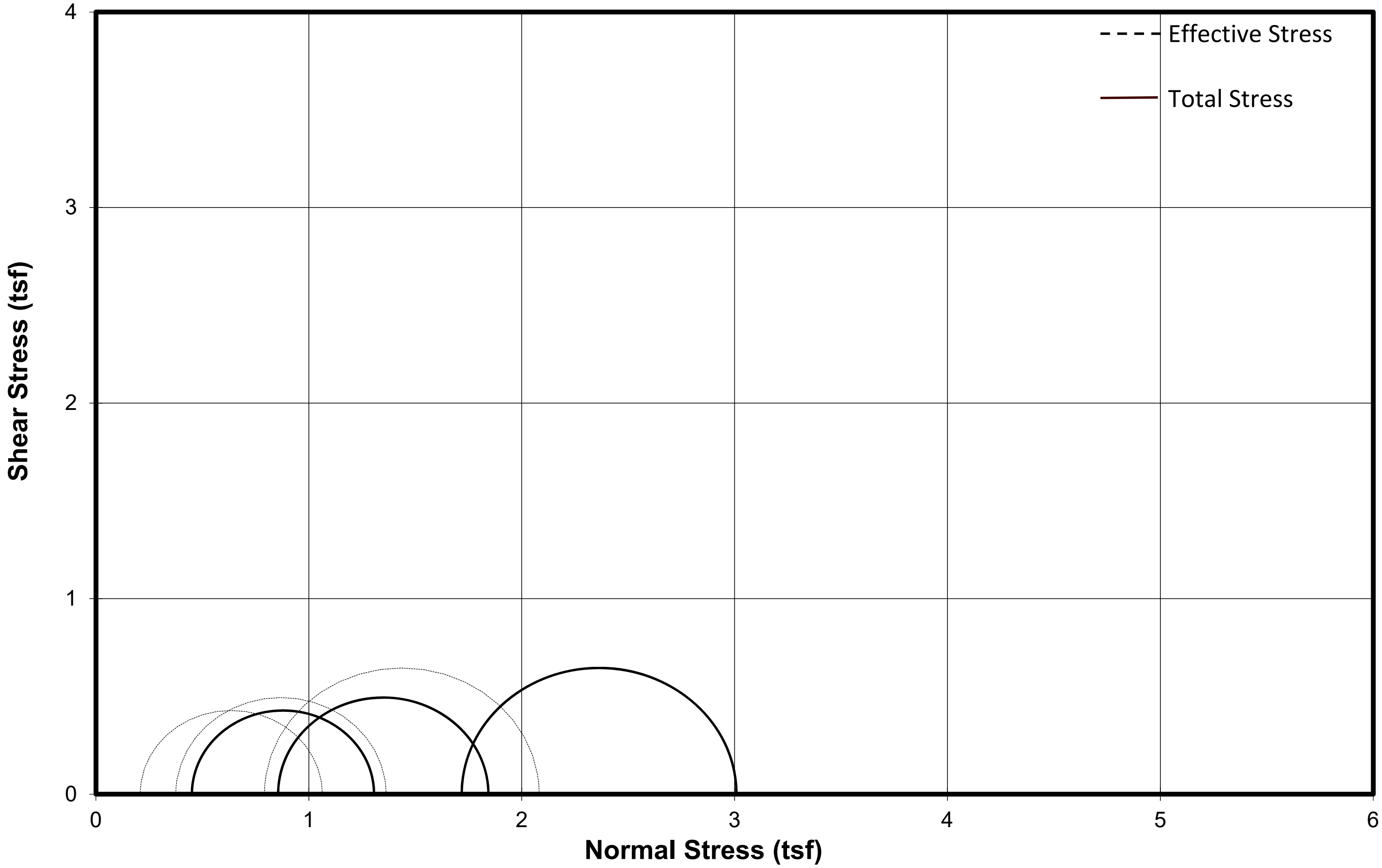
CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 4767

Project No.: J034363.01

Boring: VC-1

Sample: ST-1 - Depth: 10.0



CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST

ASTM D 4767

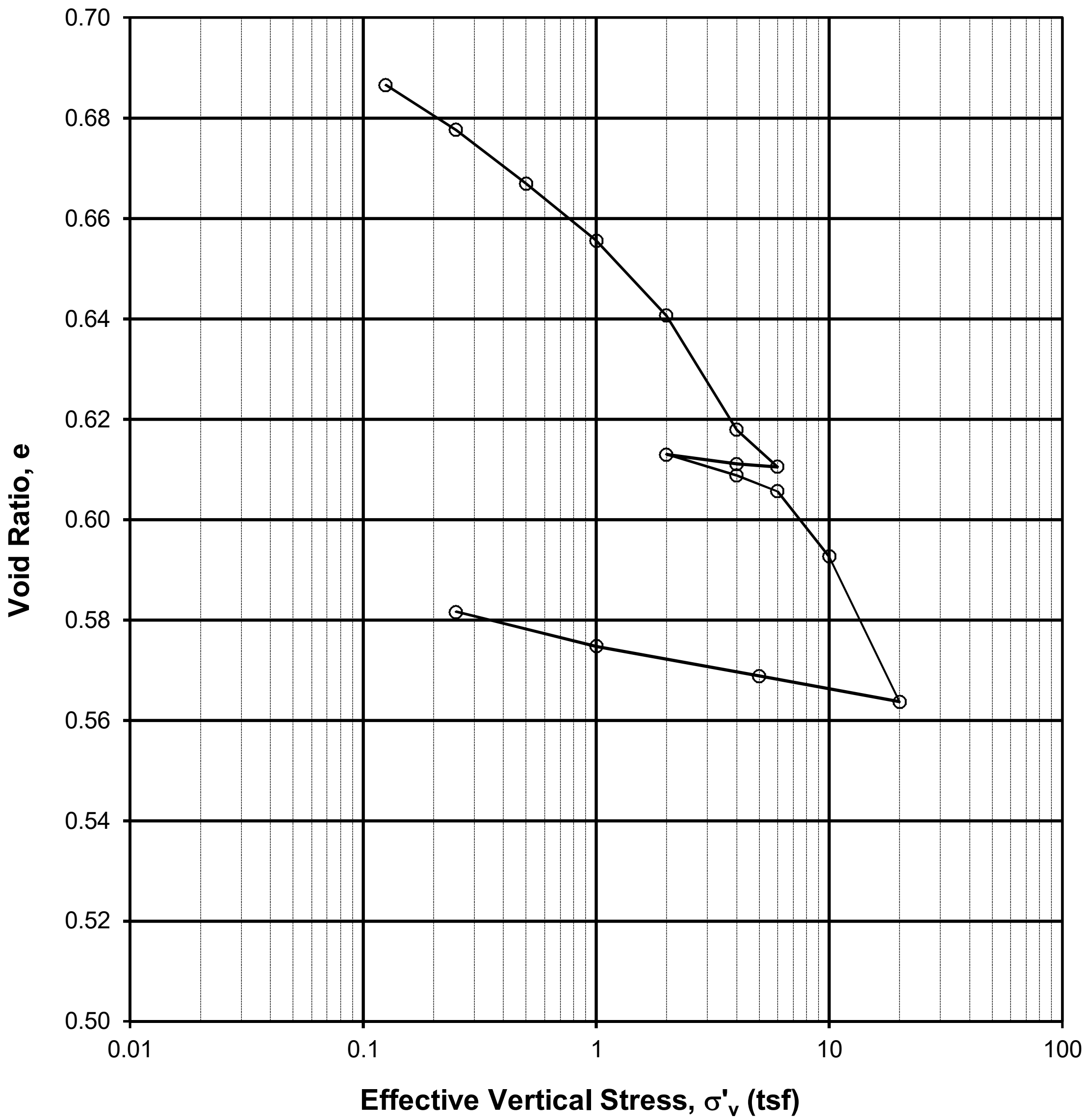
Project No.: J034363.01

Boring: VC-3

Sample: ST-4 - Depth: 8.0

Liquid Limit= 36	Plastic Limit= 23	Plasticity Index = 13	USCS: CL

Compression Index, C_c = 0.08	Void Ratio, e_o = 0.718
Recompression Index, C_r = 0.04	Preconsolidation Pressure = 3 tsf



1-D CONSOLIDATION TEST: INCREMENTAL

ASTM D 2435

Project No.: J034363.01

Boring: VC-1

Sample: ST-11 - Depth: 33.0

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**March 31, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	**Page 1 of 1**
Boring Number:	**VC-1**	
Sample ID:	**SS- 4-6**	
Depth (ft):	**8.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	7,700	0.57	4,389.00	14.4
#2	2,600	0.57	1,482.00	21.5
#3	2,900	0.57	1,653.00	29.4

Minimum Soil Resistivity **1,482.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**February 28, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	**Page 1 of 1**
Boring Number:	**VC-1**	
Sample ID:	**ST-5**	
Depth (ft):	**10.0**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	4,500	0.57	2,565.00	12.8
#2	2,000	0.57	1,140.00	21.1
#3	1,800	0.57	1,026.00	27.5
#4	2,100	0.57	1,197.00	35.0

Minimum Soil Resistivity **1,026.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**March 30, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	
Boring Number:	**VC-1**	
Sample ID:	**SS- 7-8**	
Depth (ft):	**23.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	6,100	0.57	3,477.00	12.8
#2	3,500	0.57	1,995.00	21.1
#3	3,100	0.57	1,767.00	27.5
#4	3,500	0.57	1,995.00	35.0

Minimum Soil Resistivity **1,767.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**March 31, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	**Page 1 of 1**
Boring Number:	**VC-1**	
Sample ID:	**SS- 7-8**	
Depth (ft):	**28.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	8,000	0.57	4,560.00	12.0
#2	5,600	0.57	3,192.00	19.0
#3	4,500	0.57	2,565.00	26.5
#4	4,900	0.57	2,793.00	34.6

Minimum Soil Resistivity **2,565.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**April 3, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	
Boring Number:	**VC-2**	
Sample ID:	**SS-1**	
Depth (ft):	**15.0**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	14,000	0.57	7,980.00	10.0
#2	9,000	0.57	5,130.00	16.5
#3	8,400	0.57	4,788.00	23.4
#4	10,000	0.57	5,700.00	24.2

Minimum Soil Resistivity **4,788.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**April 3, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	**Page 1 of 1**
Boring Number:	**VC-2**	
Sample ID:	**SS-2**	
Depth (ft):	**28.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	5,500	0.57	3,135.00	12.4
#2	3,200	0.57	1,824.00	19.5
#3	2,900	0.57	1,653.00	30.9
#4	3,300	0.57	1,881.00	33.9

Minimum Soil Resistivity **1,653.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**April 3, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	
Boring Number:	**VC-2**	
Sample ID:	**SS-3**	
Depth (ft):	**33.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	7,100	0.57	4,047.00	12.2
#2	5,100	0.57	2,907.00	19.1
#3	3,500	0.57	1,995.00	27.2
#4	3,800	0.57	2,166.00	35.0

Minimum Soil Resistivity **1,995.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**March 31, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	**Page 1 of 1**
Boring Number:	**VC-3**	
Sample ID:	**SS- 5-6**	
Depth (ft):	**13.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	3,300	0.57	1,881.00	13.5
#2	2,000	0.57	1,140.00	20.7
#3	2,200	0.57	1,254.00	28.1

Minimum Soil Resistivity **1,140.00**

TEST REPORT

Prepared For:
Garver USA
4701 Northshore Drive
North Little Rock, Arkansas 72118

Project No.:	**J034363.01**	**March 31, 2020**
Project Name:	**ARDOT 101000 Hwy 69, Village Creek**	**Page 1 of 1**
Boring Number:	**VC-3**	
Sample ID:	**SS- 7-8**	
Depth (ft):	**23.5**	

MINIMUM LABORATORY SOIL RESISTIVITY
AASHTO T288

Reading	Resistance Measurement	Soil Box Factor (cm)	Soil Resistivity (ohms-cm)	Moisture Content (%)
#1	8,000	0.57	4,560.00	12.0
#2	5,600	0.57	3,192.00	19.0
#3	4,500	0.57	2,565.00	26.5
#4	4,900	0.57	2,793.00	34.6

Minimum Soil Resistivity **2,565.00**

Appendix E
SELECTED GLOBAL STABILITY ANALYSES

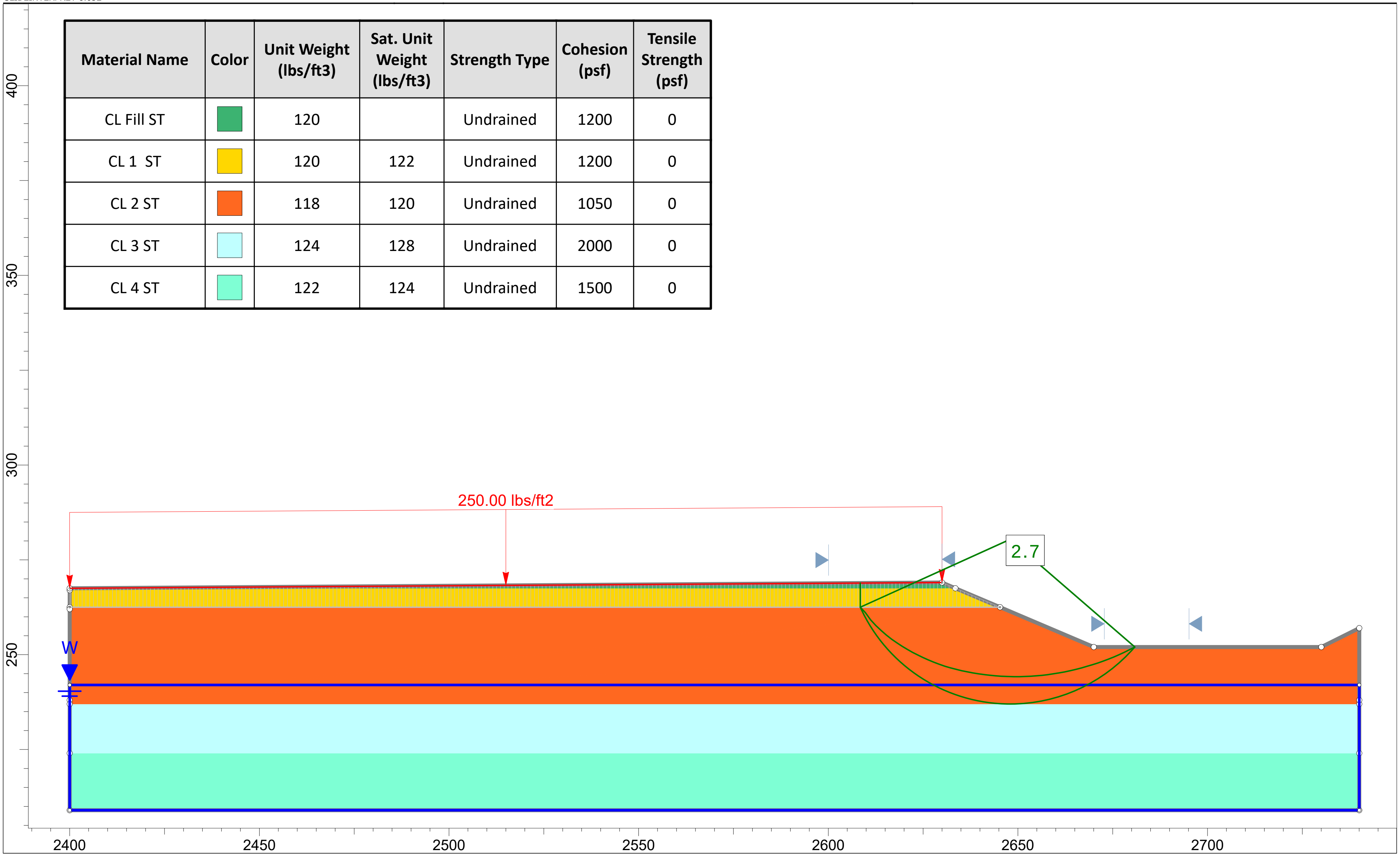
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Abutment - Spill Slope.slmd
Name: Spill Slope
Description: Short Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology. Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/31/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
CL 1 ST		120	122	Undrained	1200	0
CL 2 ST		118	120	Undrained	1050	0
CL 3 ST		124	128	Undrained	2000	0
CL 4 ST		122	124	Undrained	1500	0



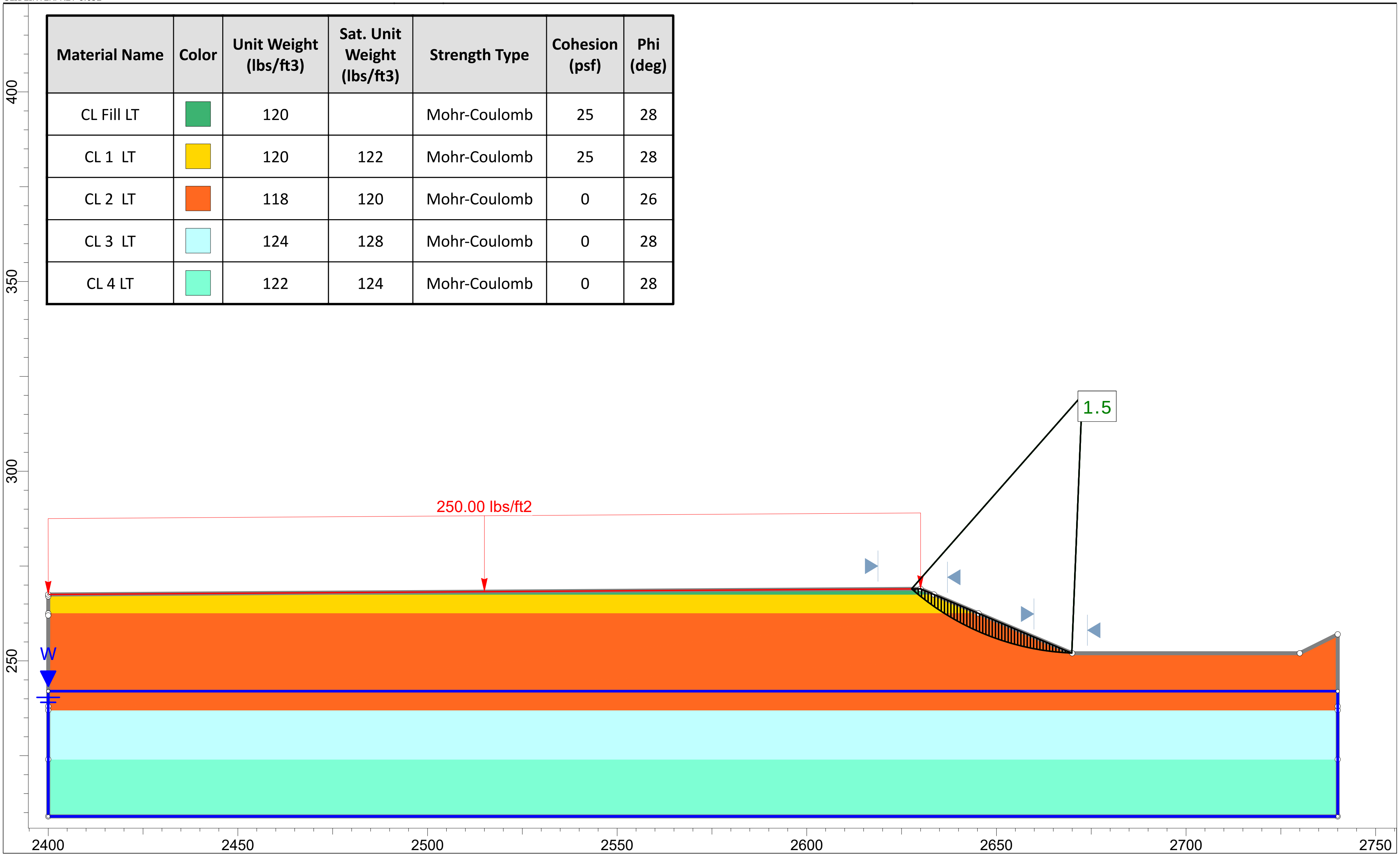
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Abutment - Spill Slope.slmd
Name: Spill Slope
Description: Long Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology. Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/31/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CL Fill LT		120		Mohr-Coulomb	25	28
CL 1 LT		120	122	Mohr-Coulomb	25	28
CL 2 LT		118	120	Mohr-Coulomb	0	26
CL 3 LT		124	128	Mohr-Coulomb	0	28
CL 4 LT		122	124	Mohr-Coulomb	0	28



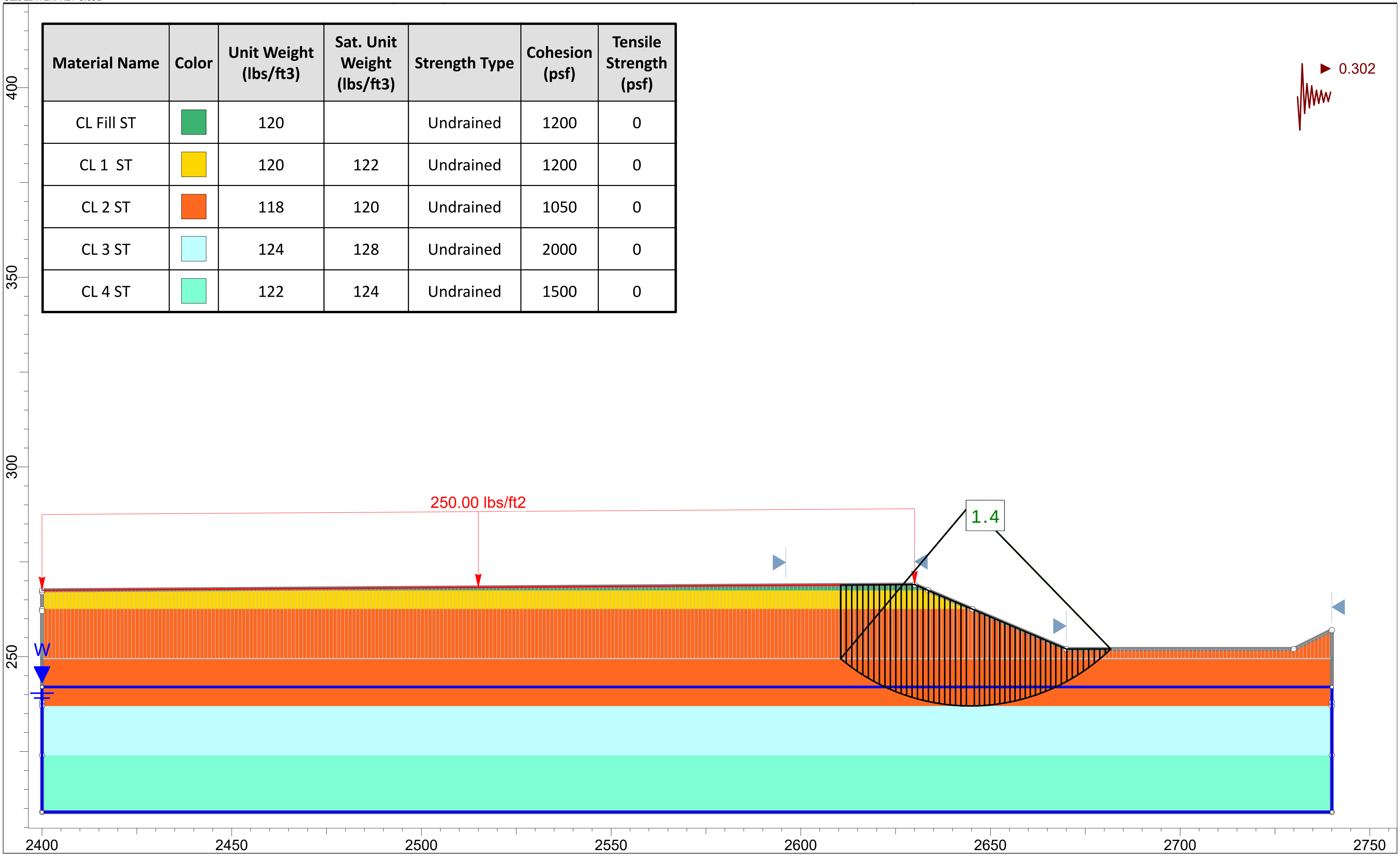
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Abutment - Spill Slope.slmd
Name: Spill Slope
Description: Seismic
Method: Spencer

Project Number: J034363.01
Client: Geotechnology. Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/31/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
CL 1 ST		120	122	Undrained	1200	0
CL 2 ST		118	120	Undrained	1050	0
CL 3 ST		124	128	Undrained	2000	0
CL 4 ST		122	124	Undrained	1500	0



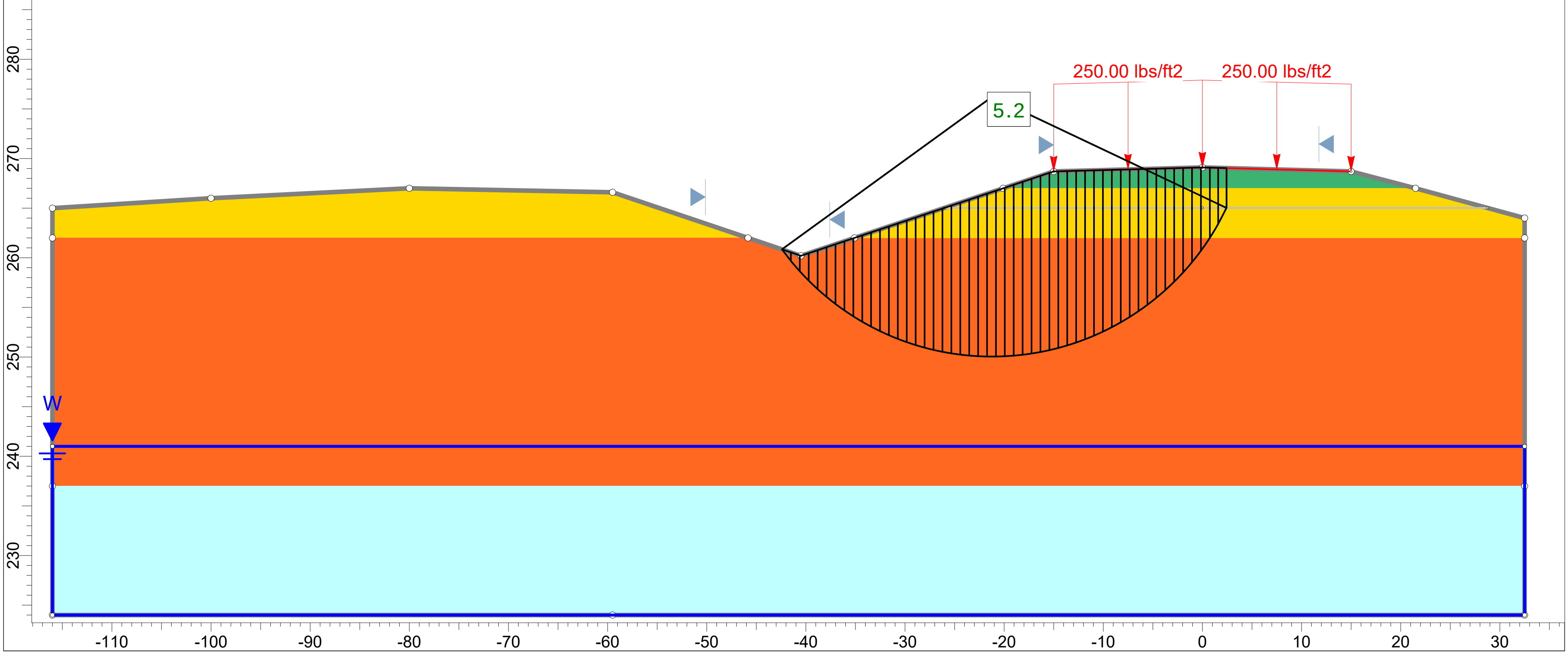
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Abutment - Side Slope.slmd
Name: Sta 26+36.83
Description: Short Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology, Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/19/2020

SLIDEINTERPRET 8.031

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
CL 1 ST		120	122	Undrained	1200	0
CL 2 ST		116	120	Undrained	1050	0
CL 3 ST		124	128	Undrained	2000	0



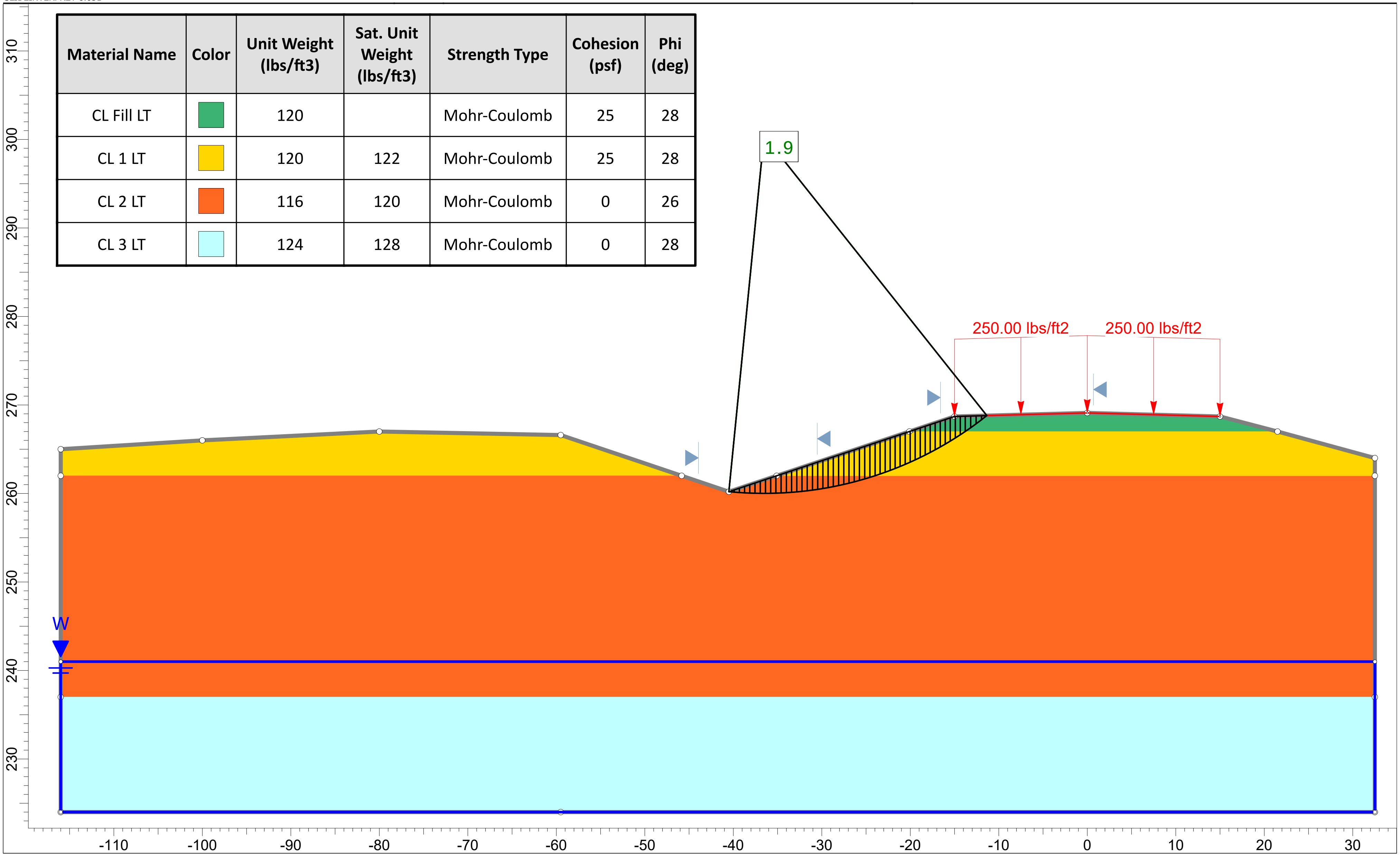
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: South Abutment - Side Slope.slmd
Name: Sta 26+36.83
Description: Long Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology, Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/19/2020

SLIDEINTERPRET 8.031

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CL Fill LT		120		Mohr-Coulomb	25	28
CL 1 LT		120	122	Mohr-Coulomb	25	28
CL 2 LT		116	120	Mohr-Coulomb	0	26
CL 3 LT		124	128	Mohr-Coulomb	0	28

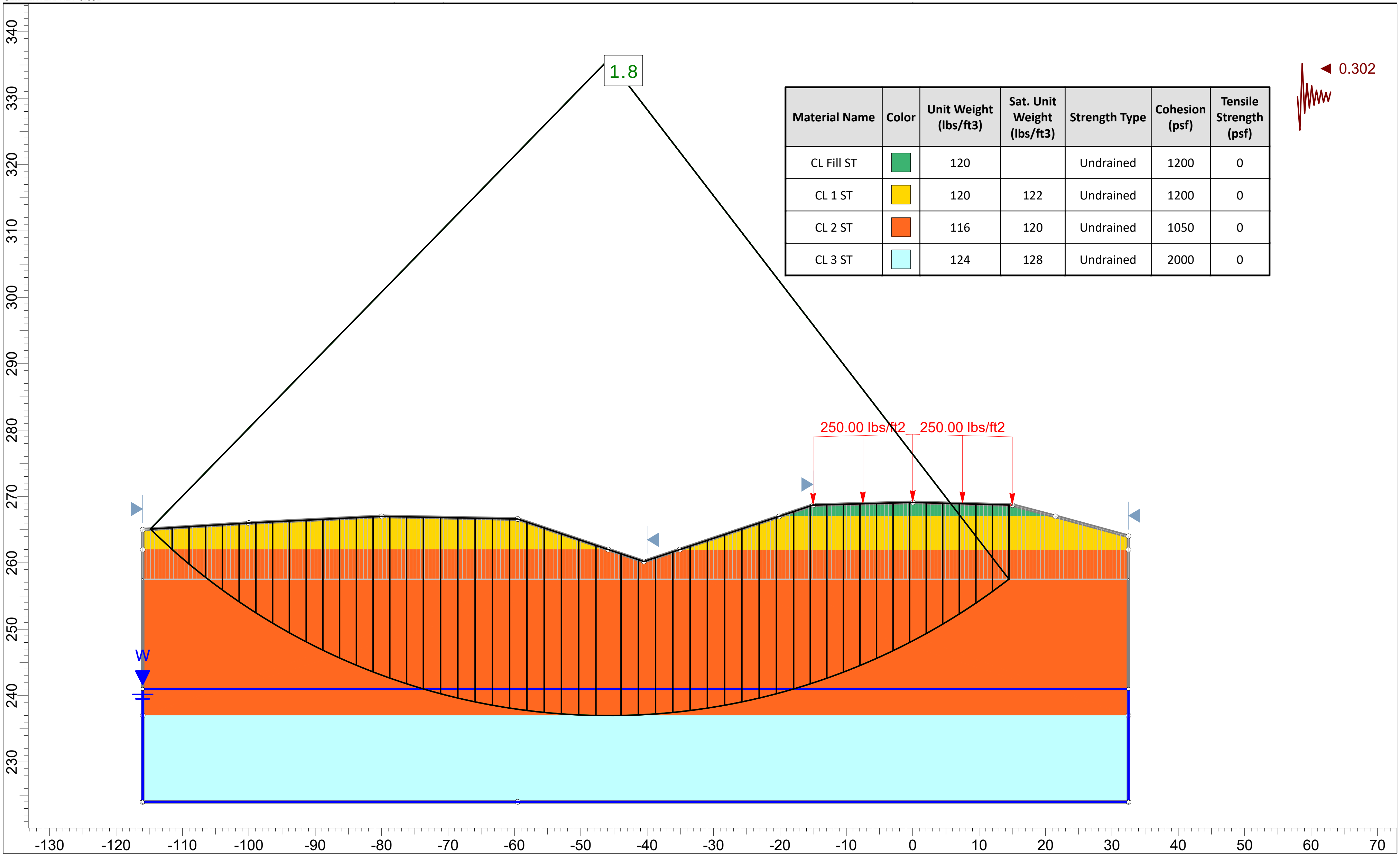


File Name: South Abutment - Side Slope.slmd
Name: Sta 26+36.83
Description: Seismic
Method: Spencer

Project Number: J034363.01
Client: Geotechnology, Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/31/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
CL 1 ST		120	122	Undrained	1200	0
CL 2 ST		116	120	Undrained	1050	0
CL 3 ST		124	128	Undrained	2000	0



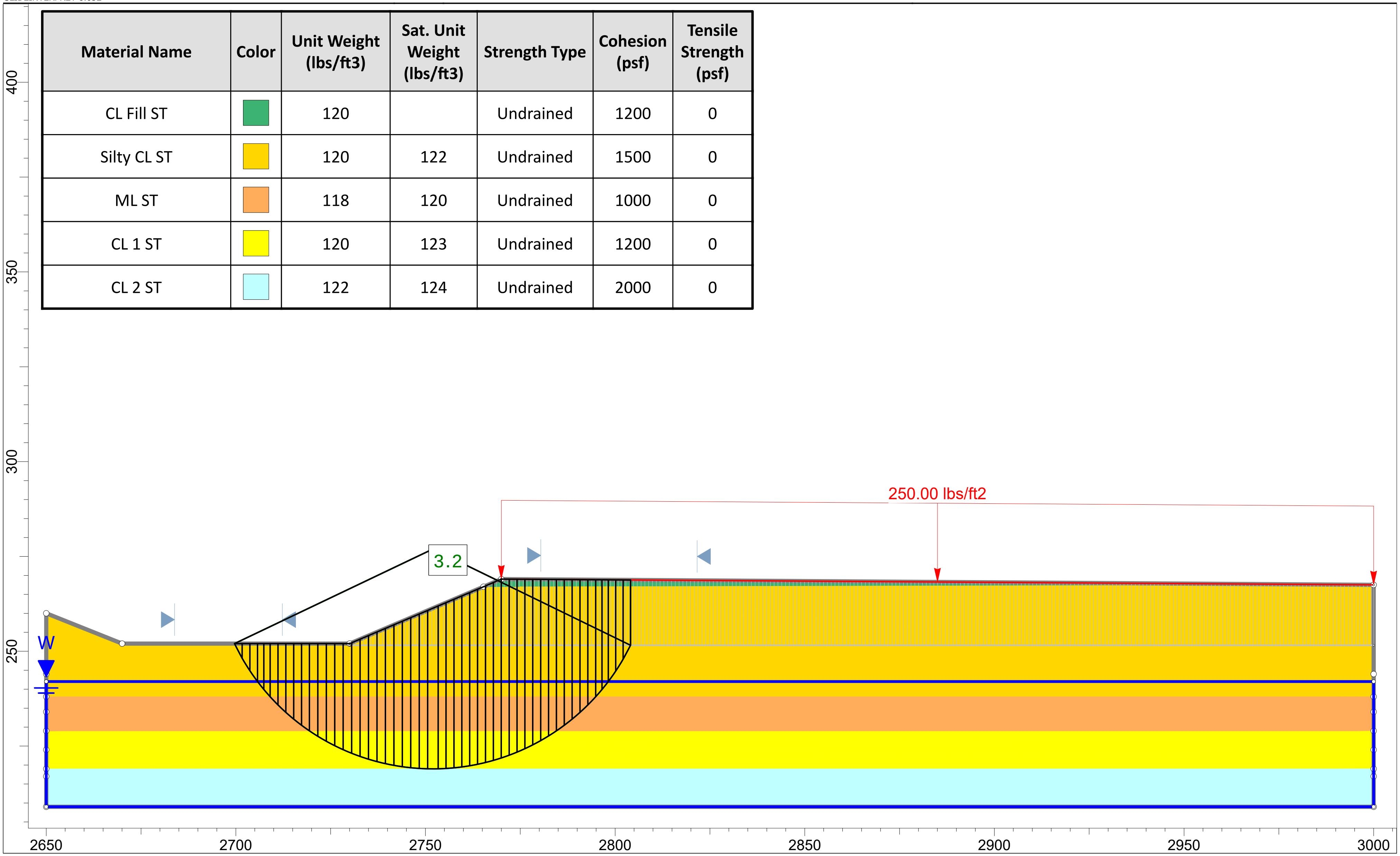
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: North Abutment - Spill Slope.slmd
Name: Spill Slope
Description: Short Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology. Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 4/6/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
Silty CL ST		120	122	Undrained	1500	0
ML ST		118	120	Undrained	1000	0
CL 1 ST		120	123	Undrained	1200	0
CL 2 ST		122	124	Undrained	2000	0



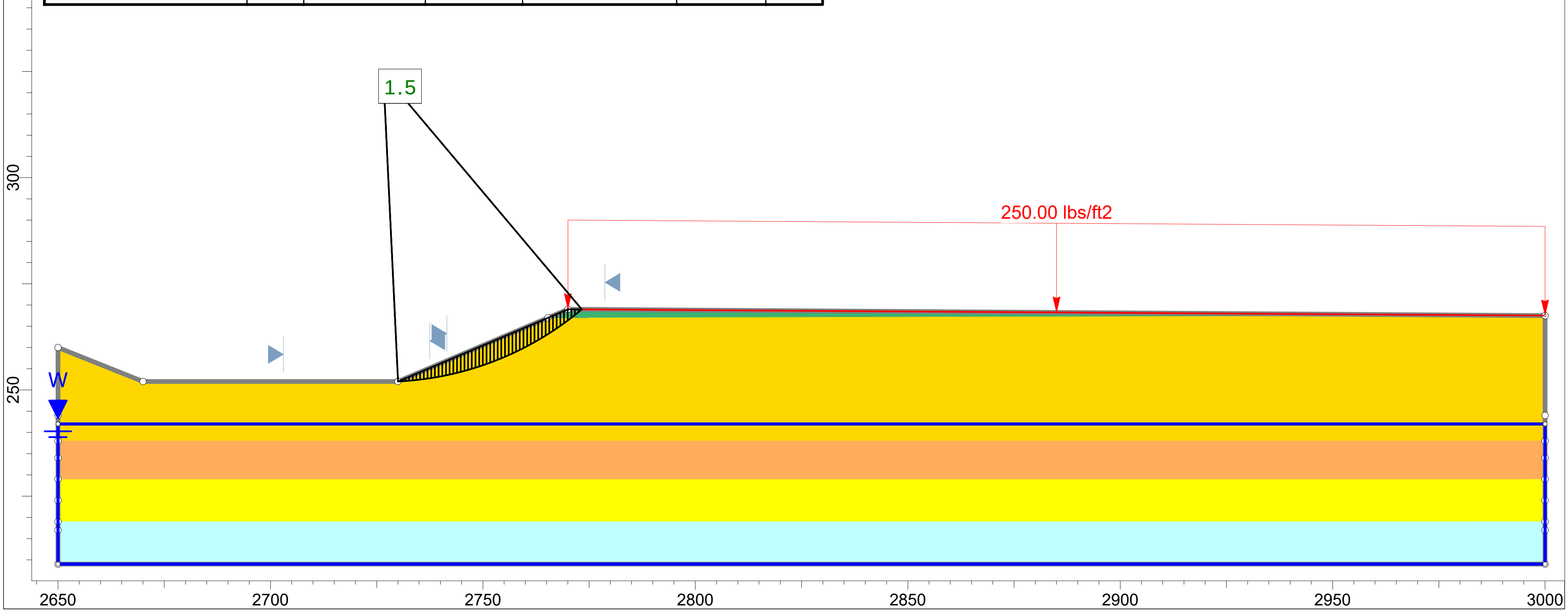
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: North Abutment - Spill Slope.slmd
Name: Spill Slope
Description: Long Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology. Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/31/2020

SLIDEINTERPRET 8.032

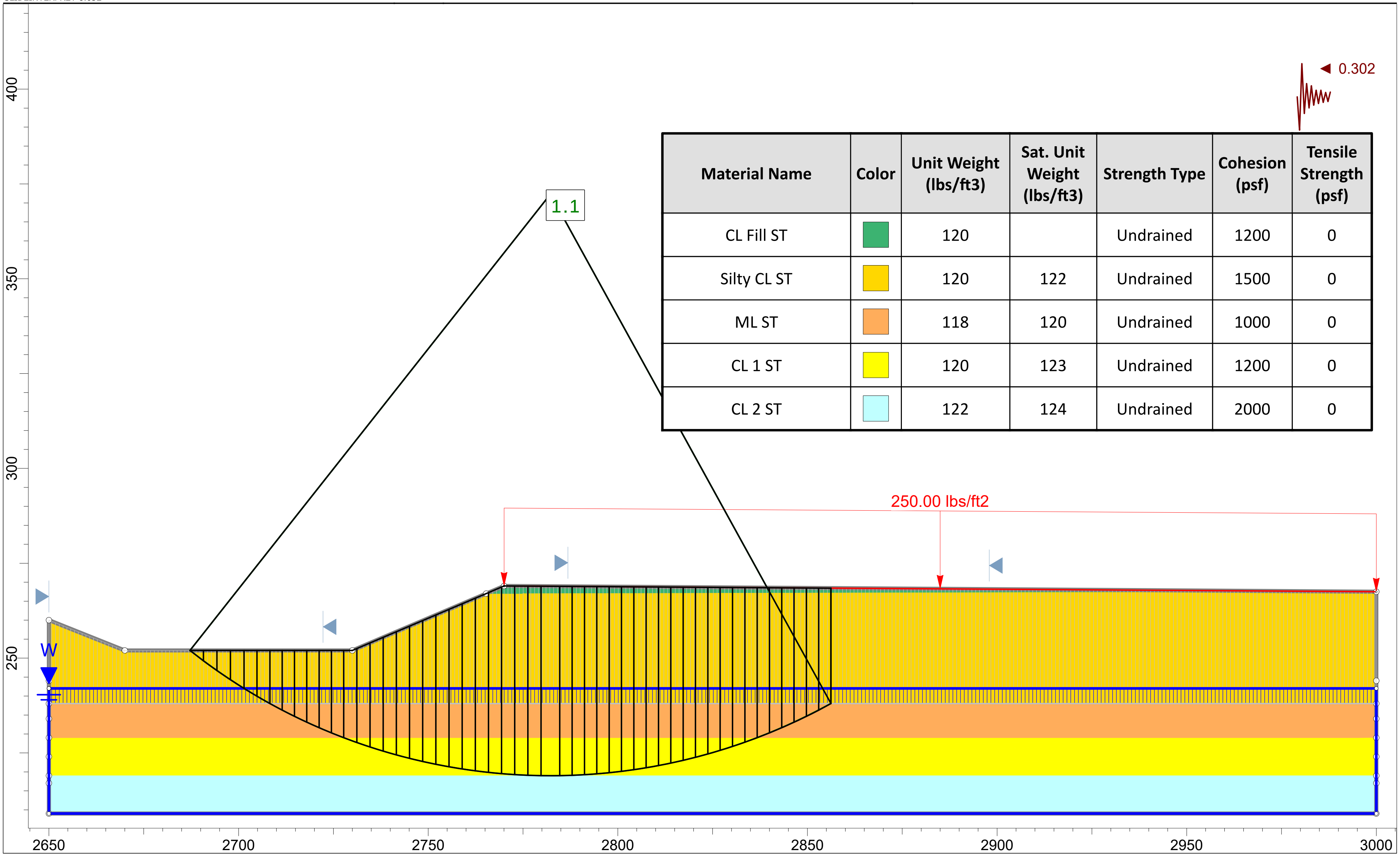
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CL Fill LT		120		Mohr-Coulomb	25	28
Silty CL LT		120	122	Mohr-Coulomb	0	26
ML LT		118	120	Mohr-Coulomb	0	30
CL 1 LT		120	123	Mohr-Coulomb	0	28
CL 2 LT		122	124	Mohr-Coulomb	0	28



File Name: North Abutment - Spill Slope.slmd
Name: Spill Slope
Description: Seismic
Method: Spencer

Project Number: J034363.01
Client: Geotechnology. Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 4/6/2020

SLIDEINTERPRET 8.032



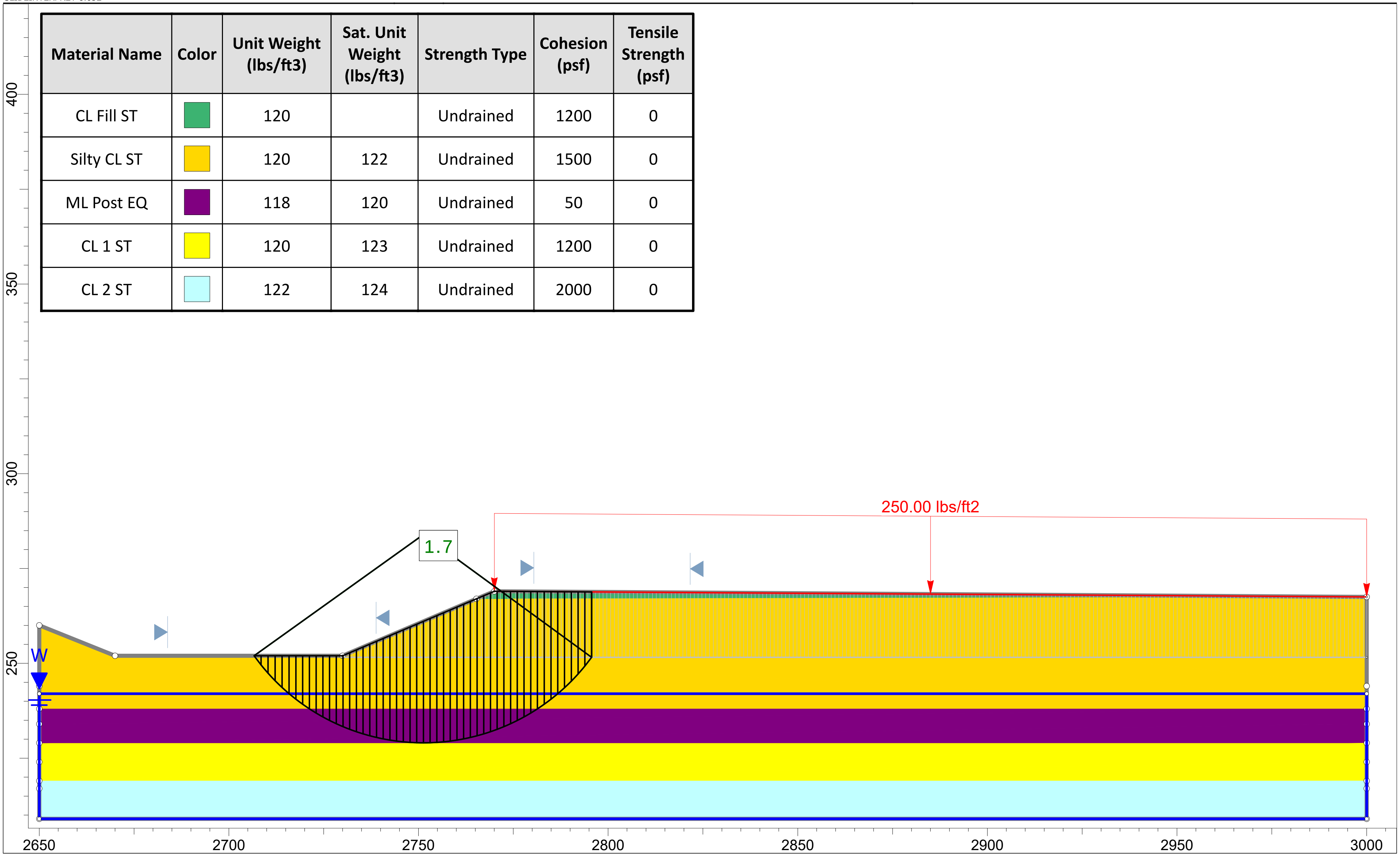
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
Silty CL ST		120	122	Undrained	1500	0
ML ST		118	120	Undrained	1000	0
CL 1 ST		120	123	Undrained	1200	0
CL 2 ST		122	124	Undrained	2000	0

File Name: North Abutment - Spill Slope.slmd
Name: Spill Slope
Description: Post Seismic
Method: Spencer

Project Number: J034363.01
Client: Geotechnology. Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 4/6/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
Silty CL ST		120	122	Undrained	1500	0
ML Post EQ		118	120	Undrained	50	0
CL 1 ST		120	123	Undrained	1200	0
CL 2 ST		122	124	Undrained	2000	0



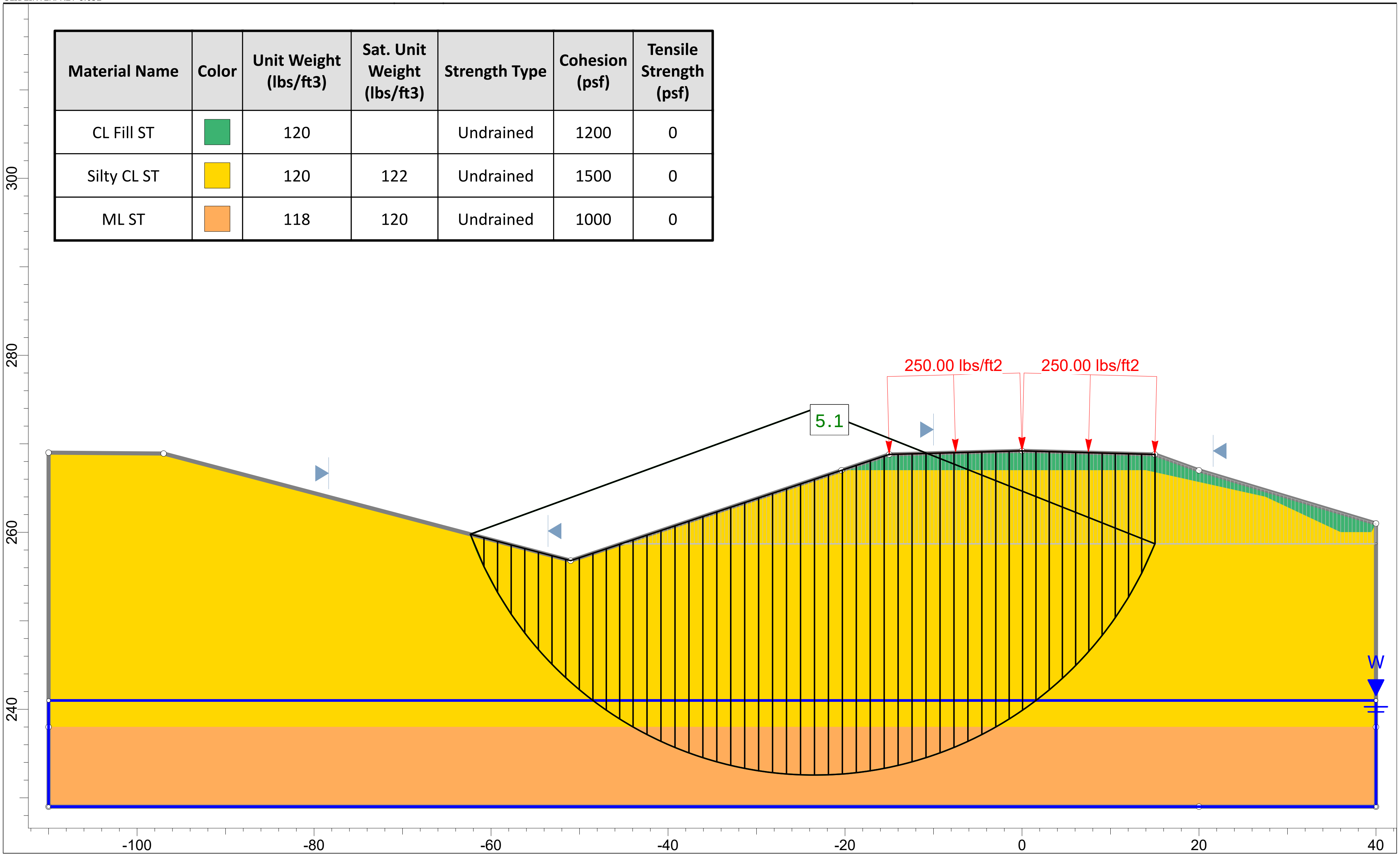
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: North Abutment - Side Slope.slmd
Name: Sta 27+63.16
Description: Short Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology, Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 4/6/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST	green	120		Undrained	1200	0
Silty CL ST	yellow	120	122	Undrained	1500	0
ML ST	orange	118	120	Undrained	1000	0

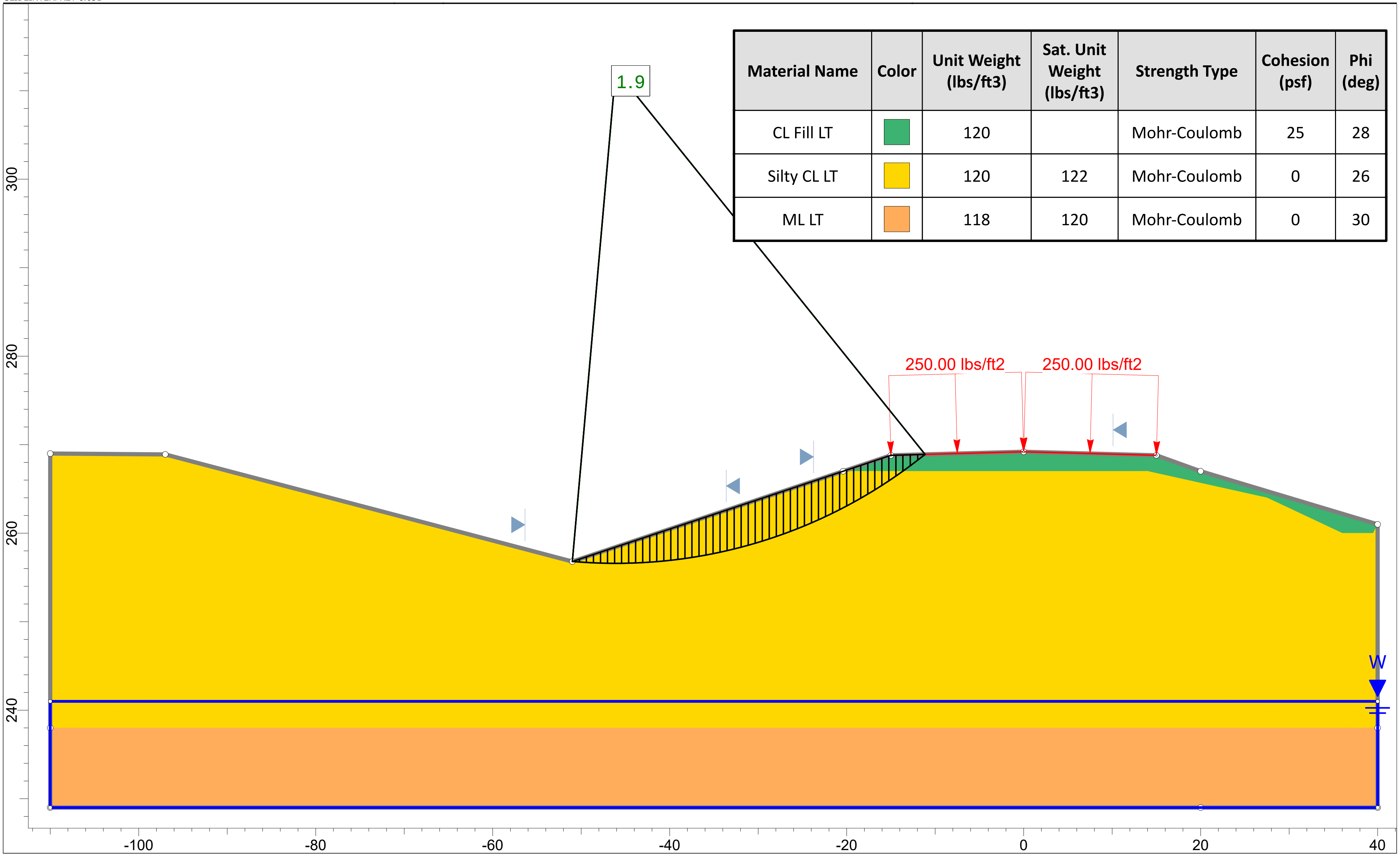


GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: North Abutment - Side Slope.slmd
Name: Sta 27+63.16
Description: Long Term
Method: Spencer

Project Number: J034363.01
Client: Geotechnology, Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 3/19/2020

SLIDEINTERPRET 8.031



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
CL Fill LT		120		Mohr-Coulomb	25	28
Silty CL LT		120	122	Mohr-Coulomb	0	26
ML LT		118	120	Mohr-Coulomb	0	30

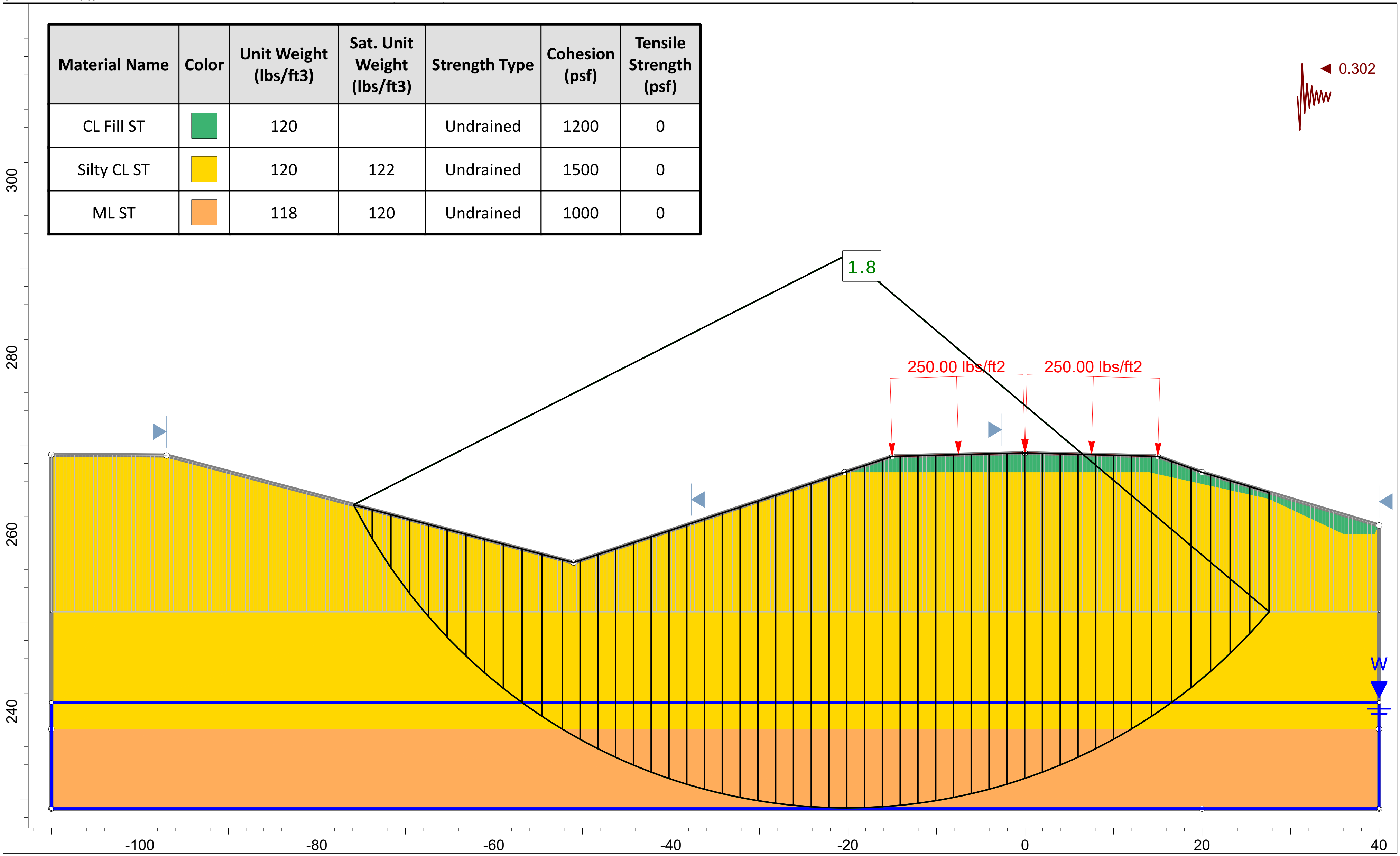
GEOTECHNOLOGY INC
FROM THE GROUND UP

File Name: North Abutment - Side Slope.slmd
Name: Sta 27+63.16
Description: Seismic
Method: Spencer

Project Number: J034363.01
Client: Geotechnology, Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 4/6/2020

SLIDEINTERPRET 8.032

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST	green	120		Undrained	1200	0
Silty CL ST	yellow	120	122	Undrained	1500	0
ML ST	orange	118	120	Undrained	1000	0

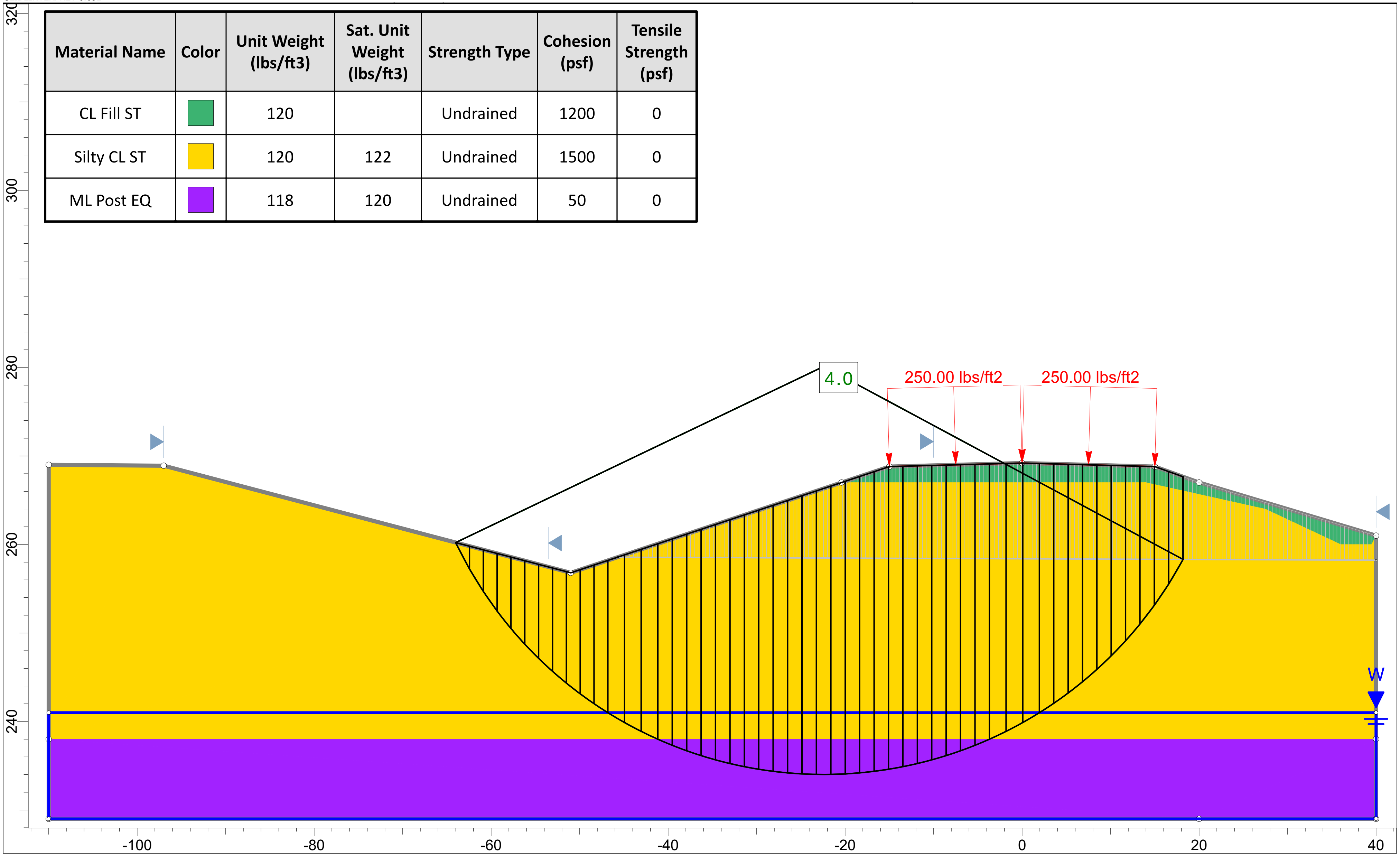


File Name: North Abutment - Side Slope.slmd
Name: Sta 27+63.16
Description: Post Seismic
Method: Spencer

Project Number: J034363.01
Client: Geotechnology, Inc.
Project: ARDOT 101000 Highway 69 Over Village Creek
Date: 4/6/2020

SLIDEINTERPRET 8.032

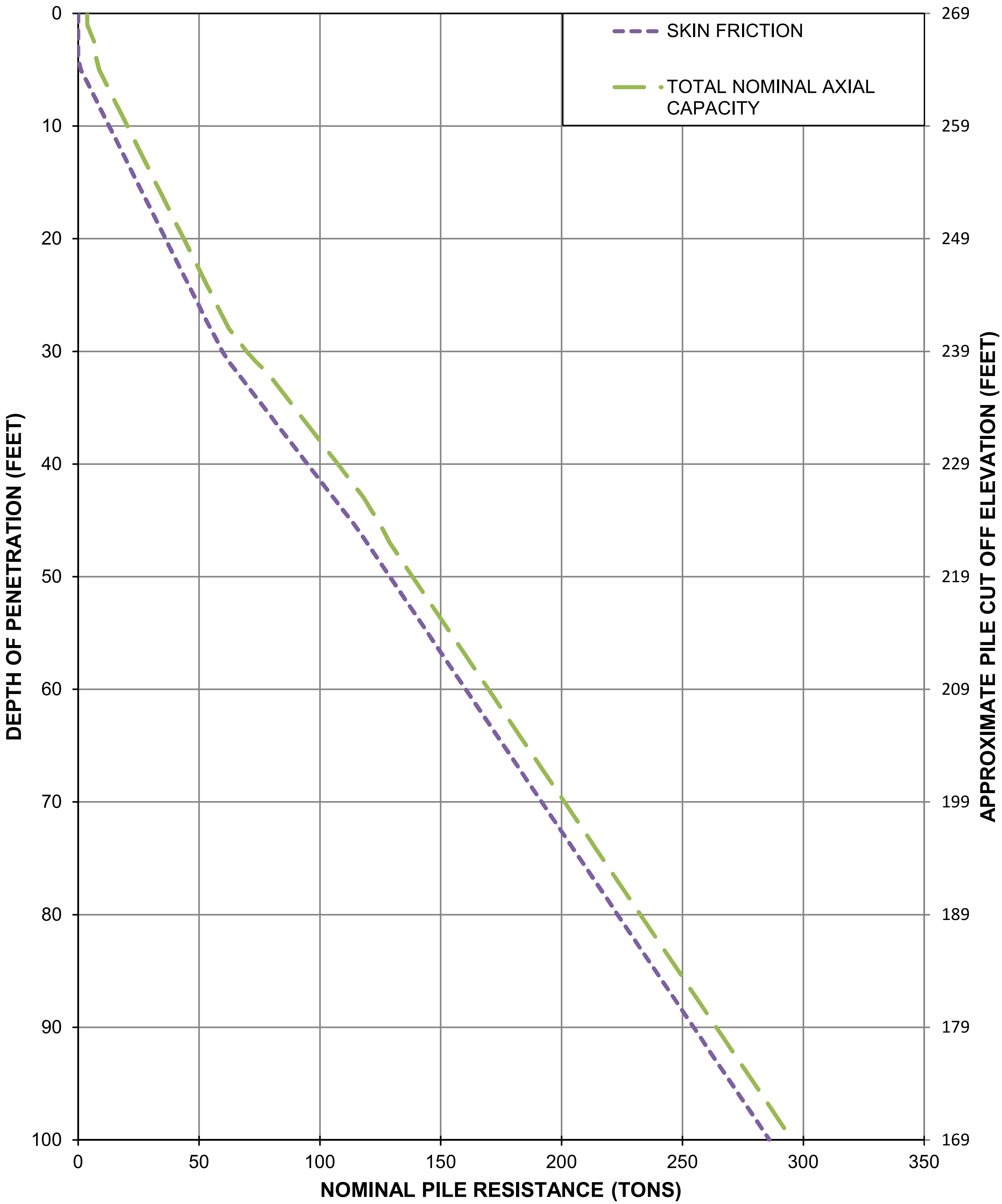
Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Tensile Strength (psf)
CL Fill ST		120		Undrained	1200	0
Silty CL ST		120	122	Undrained	1500	0
ML Post EQ		118	120	Undrained	50	0



Appendix F
NOMINAL RESISTANCE CURVES FOR DRIVEN PILES

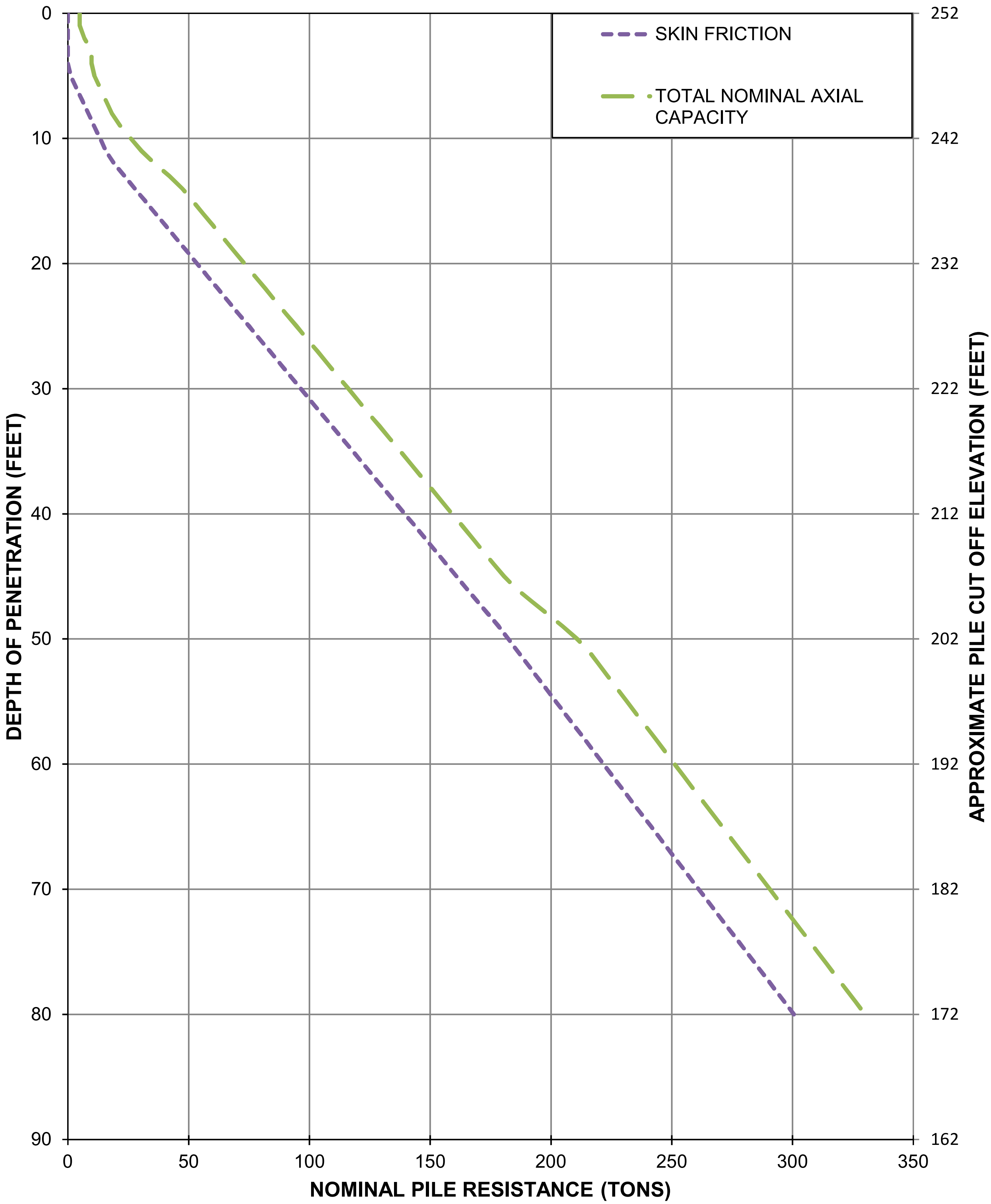
SOUTH ABUTMENT
HWY 69 OVER VILLAGE CREEK

NOMINAL RESISTANCE CURVES
DRIVEN 16-INCH, CLOSED-ENDED, PIPE PILES



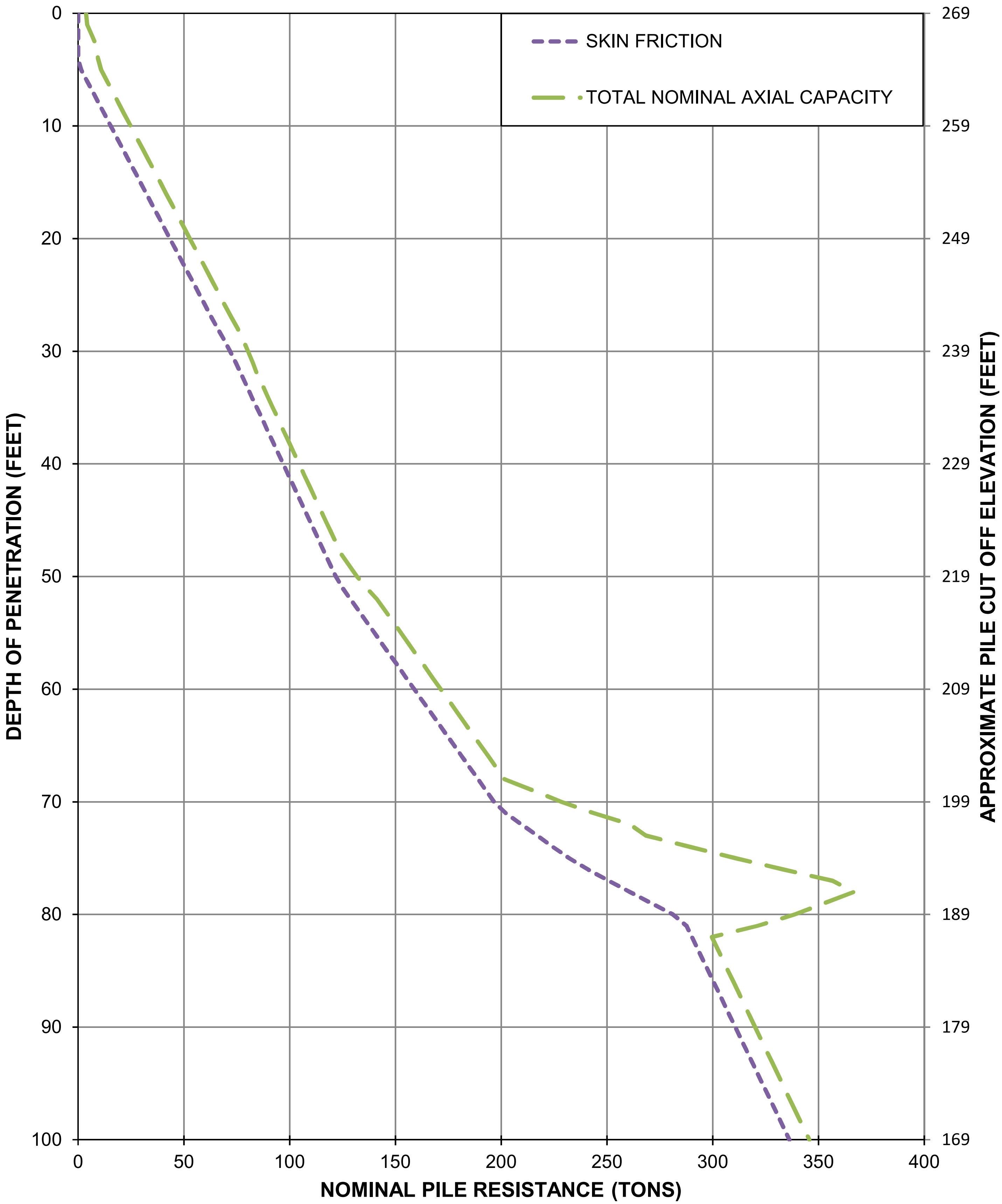
CENTER BENTS
HWY 69 OVER VILLAGE CREEK

NOMINAL RESISTANCE CURVES
DRIVEN 20-INCH, CLOSED-ENDED, PIPE PILES



NORTH ABUTMENT
HWY 69 OVER VILLAGE CREEK

NOMINAL RESISTANCE CURVES
DRIVEN 16-INCH, CLOSED-ENDED, PIPE PILES



Appendix G
SOIL PARAMETERS FOR SYNTHETIC PROFILES

SOUTH ABUTMENT - BORING VC-1										
Zone	Soil Types	Elevation[a]		Total Unit Weight (pcf)	Shear Strength Parameters				Lateral Load Parameters[d]	
					Undrained (Short Term)		Drained (Long Term)			
		From	To		Cohesion (psf)	Φ (Degree)	Effective Cohesion (psf)	Φ’ (Degree)	Soil Strain, E_{50}	Static Soil Modulus (pci)[c]
1	Engineered Fill (Cohesive)	269[b]	267	120	1,200	--	25	28	0.007	500
2	Lean Clay	267	262	122	1,200	--	25	28	0.007	500
3	Lean Clay	262	237	118	1,000	--	--	26	0.01	100
4	Lean Clay	237	224	124	2,000	--	--	28	0.007	500
5	Lean Clay	224	199	122	1,500	--	--	28	0.007	500
6	Clayey Sand	199	189	126	--	32	--	32	--	20
7	Lean Clay	189	167	125	1,500	--	--	28	0.007	500

[a] Elevations are approximated from the provided drawing
[b] Approximate final grade at south abutment
[c] Pounds per cubic inch
[d] For lateral load analysis only

CENTER BENTS – BORING VC-2										
Zone	Soil Types	Elevation[a]		Total Unit Weight (pcf)	Shear Strength Parameters				Lateral Load Parameters[d]	
					Undrained (Short Term)		Drained (Long Term)			
		From	To		Cohesion (psf)	Φ (Degree)	Effective Cohesion (psf)	Φ' (Degree)	Soil Strain, E_{50}	Static Soil Modulus (pci)[c]
1	Lean Clay	252[b]	241	118	1,000	--	--	28	0.01	100
2	Lean Clay	241	204	123	2,000	--	--	28	0.007	500
3	Lean Clay	204	174	116	3,000	--	--	24	0.007	500
4	Silty Sand	174	172	124	--	32	--	32	--	60

[a] Elevations are approximated from the provided drawing
[b] Approximate final grade at center bents
[c] Pounds per cubic inch
[d] For lateral load analysis only

SOUTH ABUTMENT - BORING VC-3										
Zone	Soil Types	Elevation[a]		Total Unit Weight (pcf)	Shear Strength Parameters				Lateral Load Parameters[d]	
					Undrained (Short Term)		Drained (Long Term)			
		From	To		Cohesion (psf)	Φ (degree)	Effective Cohesion (psf)	Φ' (degree)	Soil Strain, E_{50}	Static Soil Modulus (pci)[c]
1	Engineered Fill (Cohesive)	269[b]	267	120	1,200	--	25	28	0.01	100
2	Lean Clay/Silt	267	238	120	1,500	--	--	28	0.007	500
3	Silt/Lean Clay	238	229	118	1,000	--	--	30	0.01	100
4	Lean Clay	229	219	123	1,200	--	--	28	0.007	500
6	Lean Clay	219	199	118	2,000	--	--	28	0.007	500
7	Clayey Sand	199	194	126	--	34	--	34	--	60
8	Gravel with Clay and Sand	194	189	128	--	36	--	36	--	60
9	Lean Clay	189	167	120	1,500	--	--	26	0.007	500

[a] Elevations are approximated from the provided drawing
[b] Approximate final grade at north abutment
[c] Pounds per cubic inch
[d] For lateral load analysis only