REPORT OF SUBSURFACE EXPLORATION
AND GEOTECHNICAL EVALUATION
OBVA MEDICAL FACILITY WATER STORAGE TANKS
SHREVEPORT, LA

PROJECT NUMBER: DRGEOES 22060G

DATE: NOVEMBER 17, 2023

PREPARED FOR:

DESIGN BUILD CONSORTIUM, LLC

PREPARED BY:

11.17.23

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November 17, 2023

Design Build Consortium, LLC
1111 Hawn Avenue
Shreveport, LA

Attention: Mr. John A. Avolio, PMP, REA, CQCM
Chief Executive Officer

Re: Report of Subsurface Exploration and
Geotechnical Consultation Services
VA Medical Facility Water Storage Tanks
Shreveport, LA
Project No. DRGeoES 22060G

Dear Mr. John A. Avolio:

Dave Rambaran Geosciences, LLC (DRGeoES) has completed the authorized geotechnical engineering evaluation for the above referenced project. Our services were performed in substantial accordance with our Proposal Number 22060G.

The purpose of our evaluation was to determine the general subsurface conditions at the site, perform an analysis to establish the potential impact these conditions will have on site grading and foundation design for the proposed structure. The recommendations in this report are based on a physical reconnaissance of the site, performed August 29-30, 2022, October 14, 2022, as well as observation and classification of samples obtained from test borings conducted at the site. Confirmation of the anticipated subsurface conditions during construction is an essential part of geotechnical services.

We appreciate the opportunity to provide consultation services for the proposed development. If you have any questions regarding the information in this report or need any additional information, please call us.

Respectfully Submitted,
DAVE RAMBARAN GEOSCIENCES, LLC

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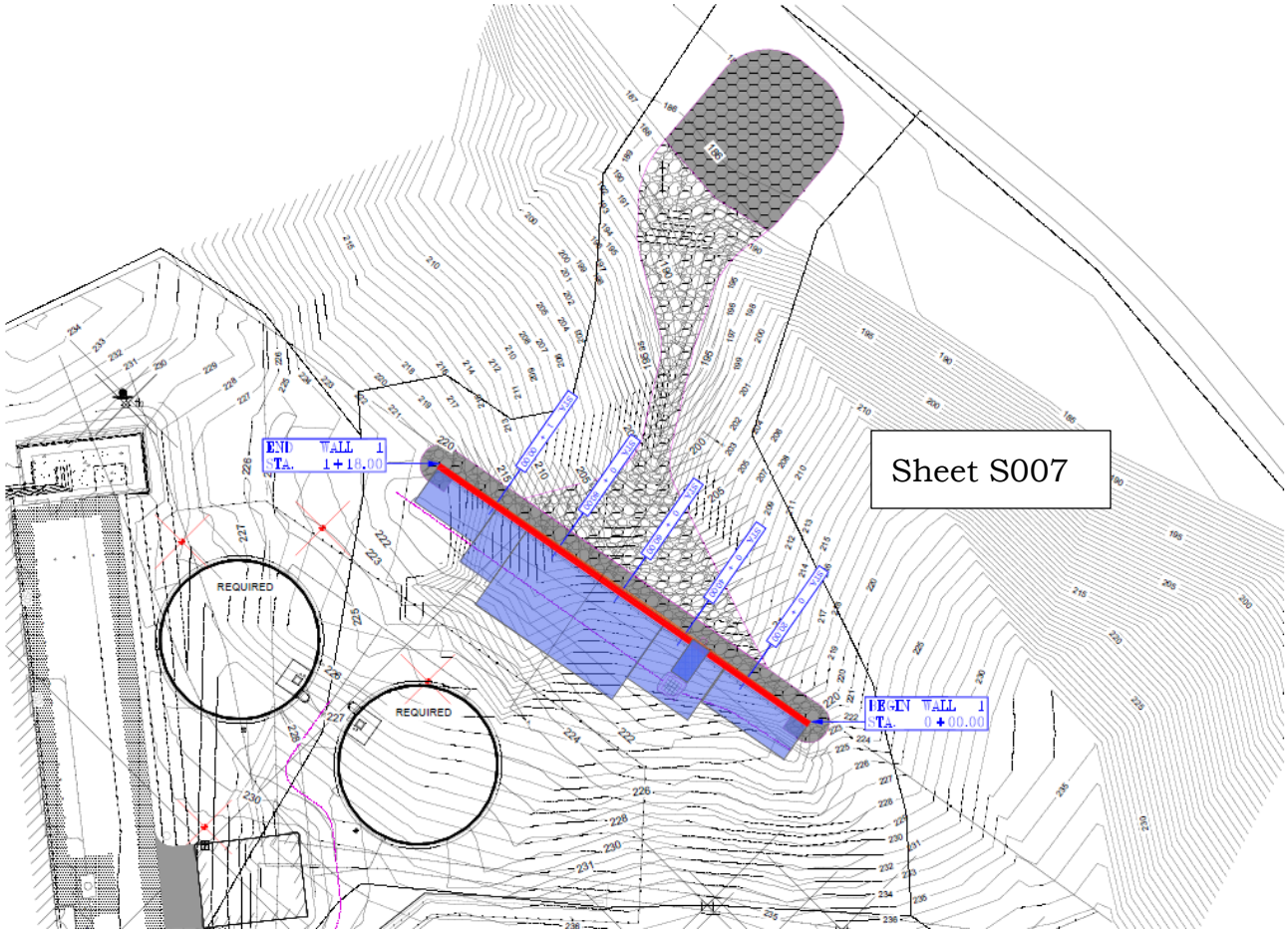
APPENDIX

1.0 PROJECT AND SITE DESCRIPTION

Dave Rambaran Geosciences, LLC (DRGeoES) has completed the geotechnical exploration for the VA Medical Facility Water Storage Tanks in Shreveport, Louisiana. Our services were authorized by Design Build Consortium, LLC. The exploration was accomplished in general accordance with DRGeoES Proposal Number 22060G.

The proposed project will consist of the construction of two new above ground water storage tanks at the Overton Brooks VA Medical Center in Shreveport, Louisiana. The two water storage tanks will be placed at the location shown in Figure 1. Each tank will be 41 feet in diameter, approximately 30 feet in height and hold 300,000 gallons each. This will also consist of a pump and equipment building approximately 20 by 25 feet and access pavement. The tanks will be located at the top of the existing slope as indicated. The slope is being washed out and will need repair and a retaining wall has been added in the design strategically to stabilize the slope for the purpose of constructing the tanks.

FIGURE I



The geotechnical recommendations presented in this report are based on the available project information and the subsurface materials described in this report. If any of the noted information is incorrect, DRGeoES should be informed in writing so that we may amend the recommendations presented in this report, if appropriate and desired by the client. DRGeoES will not be responsible for the implementation of report recommendations when not notified of changes in the project and not engaged to monitor construction.

2.0 SCOPE OF SERVICES

The purpose of the geotechnical exploration was to determine the general subsurface conditions and to gather data on which to base a geotechnical evaluation with respect to the proposed structure. The information gathered during the geotechnical exploration was evaluated to determine if any special site development issues should be addressed during the site-grading phase of the project. The results of the field and laboratory work presented within this report are as follows:

- Summary of existing surface conditions.
- A description of the subsurface conditions encountered at the soil test boring location.
- Presentation of laboratory test results.
- Site preparation considerations including material types to be expected at the site, and treatment of unsuitable soils, if encountered.
- Lateral earth pressure coefficients.
- Recommendations to be used for foundation design includes appropriate foundation types, bearing pressures.
- A description of the current groundwater conditions as observed in the boreholes during drilling at completion of the drilling. Long-term monitoring is not included in this proposal.

The scope of services did not include an environmental assessment for determining the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or around this site. Any statements in this report or on the boring log regarding odors, colors, and unusual or suspicious items or conditions are strictly for informational purposes.

3.0 SITE GEOLOGY

According to the "Generalized Geologic Map of Louisiana," the site is underlain by Paleocene and Holocene geologic age; Major Lithologic Constituents of Clay and Silt; Minor Lithologic Constituents of Sand and Limestone. United States Geological Survey, 1998, Digital Overlay of the Geologic Map of Louisiana: U.S. Geological Survey, Biological Resources Division, National Wetlands Research Center, Product Id USGS-NWRC 1984-02-0001, http://www.nwrc.usgs.gov/.

The scope of our work did not include a geologic investigation to address any faults, large scale subsidence, or macro geologic features not specifically addressed in this report or in the agreement between DRGeoES and the client.

FIGURE II

4.0 LABORATORY ANALYSIS & FIELD TESTING

The laboratory analysis provided consists of moisture profiles of the natural moisture content, #200 washes, and Atterberg limits tests determinations. The results of the laboratory analysis are presented on the boring log in the Appendix of this report. A detailed description of the general tests that may be performed is attached in the Appendix. The tests performed are listed below.

- DESCRIPTION OF SOILS (VISUAL-MANUAL PROCEDURE) (ASTM D 2488)
- NATURAL MOISTURE CONTENT (ASTM D 2216)
- ATTERBERG LIMITS (ASTM D-4318)
- WASH #200 TEST (ASTM D 422)
- DCP FIELD TEST (ASTM D 6915 & MDOT 93-05)
- STANDARD TEST METHOD FOR FIELD VANE SHEAR TEST (ASTM D4767 – 04 ASTM D2573 – 08)

5.0 SUBSURFACE CONDITIONS

5.1 INFORMATION

General site information was provided with the general layout location of the proposed structure. The borings were located using drawings provided by the client. Six boring locations were estimated using known features at the site and are approximate. Borings were sampled continuously to a depth of 10 feet and then at 5 feet intervals to the termination depth or refusal depth. Boring B1 was drilled to a depth of 60 feet and B2-B5 was drilled to a depth of 30 feet. Borings B6 and B7 was augured continuously to a depth 10 feet.

At the boring locations, soil samples were retrieved at standard sampling intervals using general accepted standards practiced in this area. The project engineer prepared Boring Logs summarizing the subsurface conditions at the boring locations. The boring logs and site layout are attached to this report.

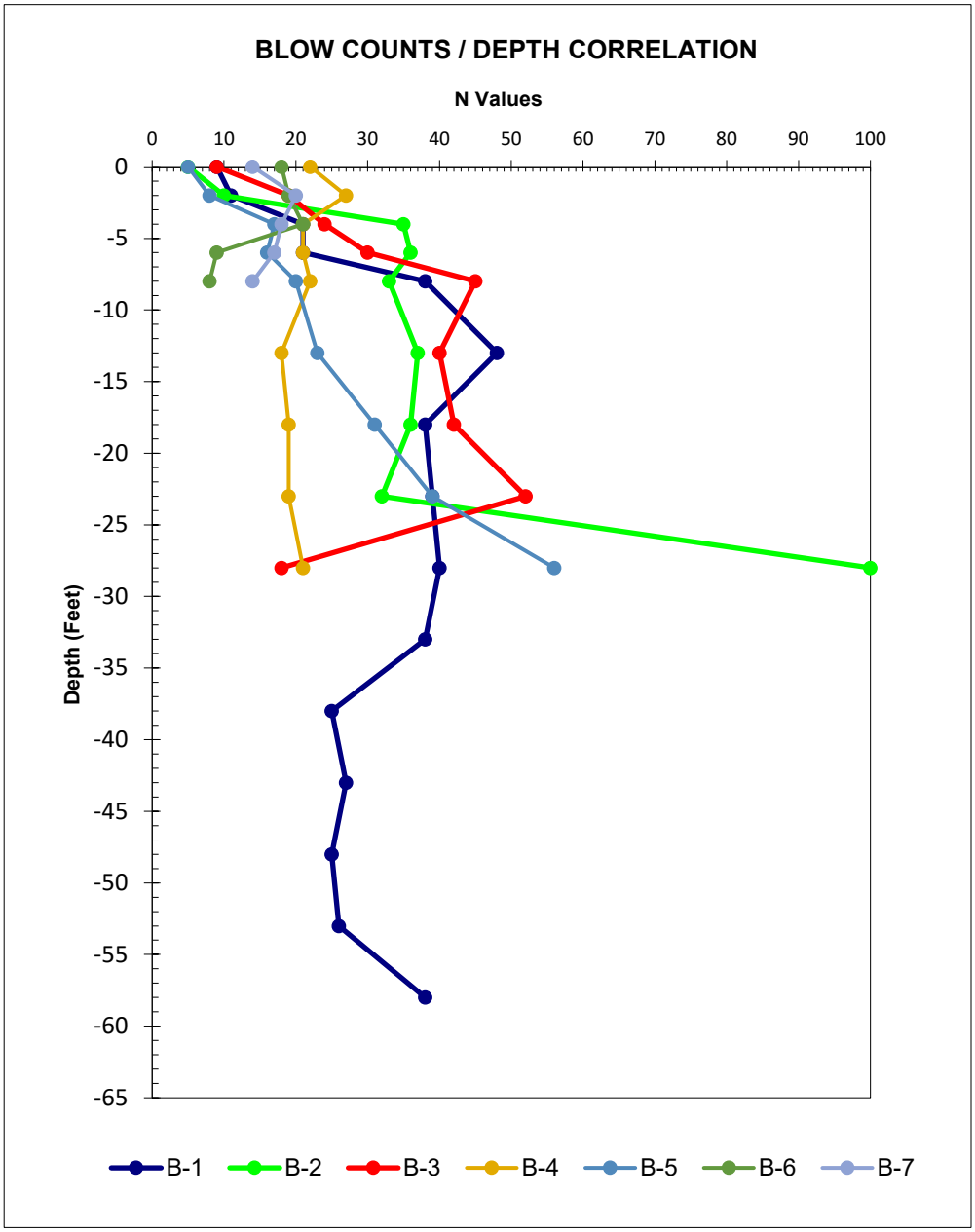
5.2 CONDITIONS

The subsurface conditions at the site were evaluated based on the borings performed at the site. The following conditions and subsequent recommendations are based on the information from the borings obtained during the field investigation. Anomalous conditions may occur due to the geologic conditions at the site. It will be necessary to verify the conditions during site grading and prior to foundation installation - **DRGeoES offers construction materials testing and inspection services and shall be retained to perform these services during construction.**

Table I - Generalized Soil Profile

Stratum No.	Approx. Depth Range (ft)		Soil Description
	From	To	
I	0	18	Sandy Clay (CL); FIRM to HARD Silty Sand (SM); LOOSE to DENSE
II	18	38	Fat Clay (CH); VERY STIFF to HARD Silty Sand (SM); MEDIUM DENSE to DENSE
III	38	60	Clay (CL); VERY STIFF to HARD with Shale and Sand

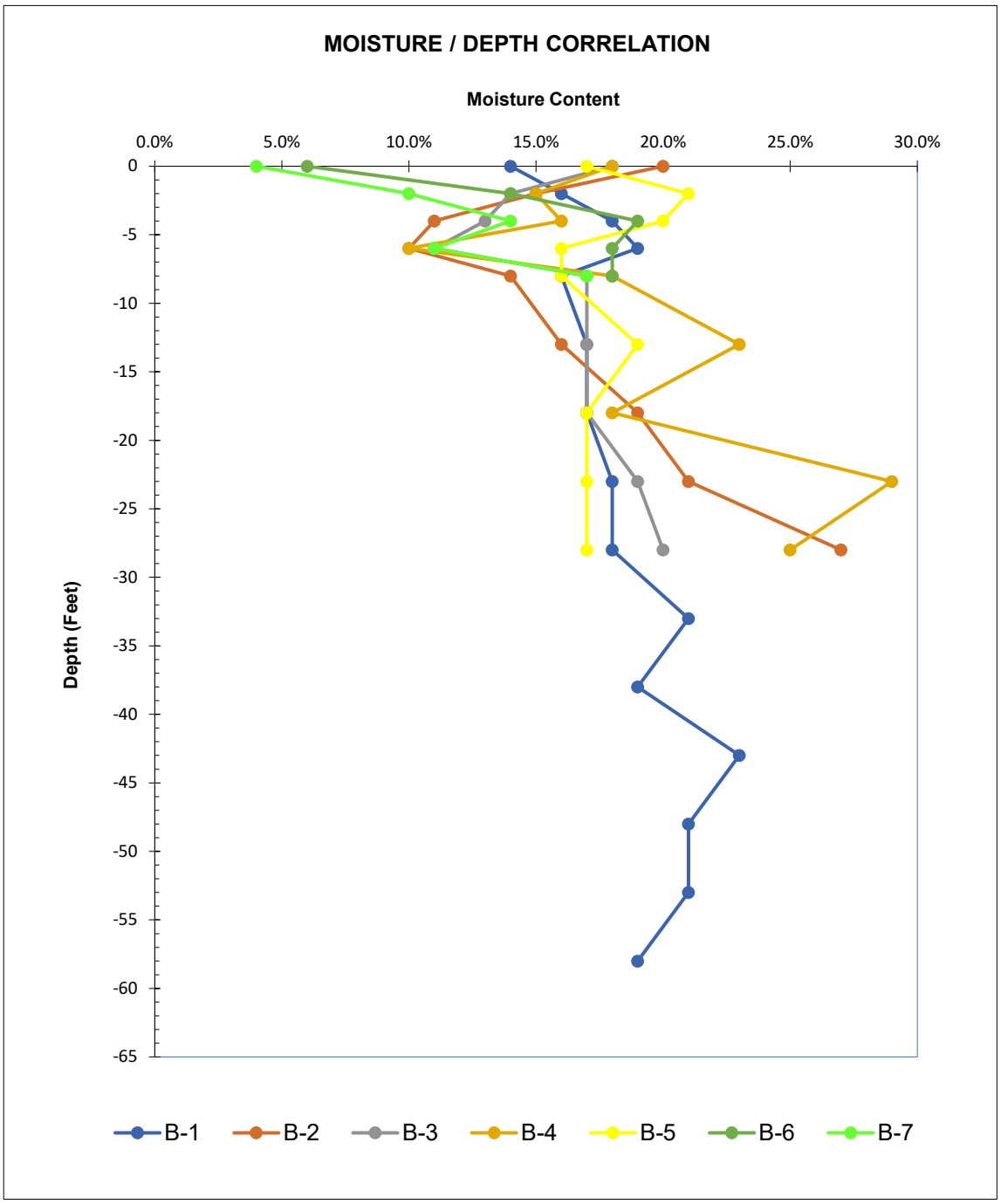
FIGURE III



5.3 GROUNDWATER

Groundwater was encountered at 23 feet in boring B1 and at 7 to 8 feet in borings B2-B5. A moisture profile was estimated using the natural moisture content of the soil samples obtained from the boring. The natural moisture content increases from 6 feet. The subsurface groundwater level is subject to change with variation in climate condition. Future construction activities may also alter the surface and/or subsurface drainage patterns of this site. Therefore, groundwater conditions should be explored at the start of construction by others. The time of year and quantity of precipitation should be considered when using this information.

FIGURE IV



6.0 SITE GRADING CONSIDERATIONS

The means, methods and sequence of construction including site preparation shall be the responsibility of the project contractor who shall specialize in this type of construction. However, general geotechnical related considerations are given herein for general guidance.

Preliminary site plan was reviewed for location of the proposed tanks. However, based on information obtained from the borings, sandy clay (CL) and silty sand (SM) materials are encountered at the site. A 41 feet diameter, 30 feet high, two above ground storage tanks are to be constructed in the general area of boring B-1 through B-5.

6.1 SITE PREPARATION

DRGeoES recommend vegetation, topsoil, or any other deleterious materials be removed up to 12 inches including tree roots, stumps, pre-existing structures, foundations, and utilities. Unsuitable materials should be removed prior to construction. The geotechnical engineer shall be retained for observation during construction.

After stripping and excavation are completed, areas that will require fill or support structures should be observed and proof rolled or probed by the geotechnical engineer's representative prior to fill placement or construction. A consistent horizontal fill shall be placed under the tanks to limit differential settlement. The project geotechnical engineer shall be retained to observe the compaction operations.

Due to the potential to failure of slope failure during construction measures shall be taken to protect the slope prior to construction commencing.

If the site becomes saturated due to the sensitive nature of the clay material, DRGeoES recommend equipment not be allowed to travel directly on the exposed subgrade once the area is cleared and undercut. The structural fill can be pushed into place using a light-weight bulldozer (i.e., Caterpillar D3 - <18-20,000 pounds). DRGeoES recommend backfilling this area with stone material as recommended below. The bulldozer or other equipment should not travel directly on the newly exposed subgrade.

The bulldozer should work from the first lift and not impact the subgrade. At all times, the bulldozer should maintain at least 12 inches of material between it and the undisturbed subgrade, excluding the final leveling of each lift. A non-vibratory roller should be used to compact the remaining lifts if unstable conditions are encountered. It is important to note traffic on the pad should be confined to only the necessary equipment needed to be on the pad. The contractor may forgo the proof-rolling provided the geotechnical engineer or his representative is present to perform field verification of the subgrade, compaction testing, and the stone base material is verified to be compacted to the recommended compaction.

The geotechnical engineer's representative shall be present to perform field verification of the subgrade, compaction testing, and the select fill material to be compacted to the recommended 95 percent standard Proctor (ASTM D-

698). However, if rutting and pumping of the subgrade is apparent, then the geotechnical engineer should be contacted to provide additional recommendations. The site contractor shall be monitored by the geotechnical engineer.

6.2 UNDERCUTTING/STABILIZATION

Instability may exist during construction, depending on climatic conditions. Loose and very soft and/or unstable surficial silty or clayey soils may be encountered across the project site during wet weather or high-water table levels. Very soft and/or unstable materials also may be present in unexplored areas of the site. Soft and/or unstable soils identified should be undercut, replaced, and recompacted in lifts or stabilized. Where unstable soils are encountered, they should be removed and replaced with select fill.

6.3 EXCAVATION CONSIDERATIONS

DRGeoES recommend low plasticity clay (CL/SC) fill should be used to replace the excavated material in accordance with **Section 6.5** of this report. Groundwater may be encountered during the excavation depending on the time of year and precipitation. Levels will depend on the rainfall amounts prior to and during construction and appropriate measures should be taken during construction to prevent water infiltration into the excavations. Shoring should be used to protect the sides of the excavation from sloughing and caving. Dewatering may be needed.

6.4 SUBGRADE EVALUATION

The geotechnical engineer shall be retained to evaluate the subgrade of the site after the initial site preparation is completed. All unsuitable material identified during the subgrade evaluation shall be removed prior to the placement of fill. Areas that require fill placement must be stable and the stability must be confirmed by the geotechnical engineer. The first lift of fill placed on the approved subgrade should not exceed 8 to 12 inches loose measure and should be compacted to the specifications provided in the following section.

6.5 STRUCTURAL FILL

DRGeoES recommend select fill at the site be composed of material with Plasticity Index (PI) between 5 and 20, and a Liquid Limit (LL) less than 40. Also, the material should be classified as a clay, sandy clay (CL) or clayey sand (SC). The percent silt shall not exceed 30% of the material passing the #200 sieve. Any fill to be placed at the site should be approved by the geotechnical engineer.

The select fill should be compacted as shown in Table II of the Standard Proctor maximum dry density and within -2 to +2% of the optimum moisture as determined by ASTM D-698. The specifications should state density should be met. The lifts should not exceed 8 to 12 inches thick, depending on the compaction equipment used. Density and moisture tests should be performed on each lift prior to placement of subsequent lifts.

Table II – Structural Fill Compaction

Location or Area	Standard Proctor Density ASTM D698	Testing Frequency 1 per lift per
Structures	98%	1,000 sf
Trenches	95%	75 lf
Scarified Subgrade recompacted and Proof-roll	95%	2,500 sf

6.6 UTILITY TRENCH BACKFILL

All utility trenches must be backfilled and compacted in the manner specified above for structural fill. It may be necessary to reduce the lift thickness to 4 to 6 inches to achieve compaction using hand-operated equipment. Due to the potential of moisture migration through the utility trenches, we recommend that all trenches be filled and compacted on the day they are opened. Future moisture migration through the trenches should be prevented by protecting and sealing the trenches**.** The backfill should be sandy clays or lean clays placed and compacted in accordance with the recommendations in the "Site Preparation" section of this report. Utility trench fill placement shall be monitored and confirmed by the geotechnical engineer.

6.7 LANDSCAPING AND DRAINAGE CONSIDERATIONS

The potential for soil moisture fluctuations within the subgrade areas should be minimized in order to reduce the potential of subgrade movement. Site grading should include positive drainage away from the structure. Ponding of water adjacent to structure could result in soil moisture increases and subsequent degradation and/or heave of the soils. Trees can develop large root systems which can draw water from subgrade soils, resulting in subsequent shrinkage of the soils. Frequent irrigation of landscaping poses a risk of saturating and softening soils below foundations, which could result in settlement or heave and premature failure.

6.8 WET WEATHER CONSTRUCTION

Cooler temperatures and shorter days during the winter season significantly reduce the capacity to dry out wet clayey soils. Additionally, excessive movement of construction equipment across the site during wet weather will result in ruts, which will collect rainwater, prolonging the time required to dry the subgrade soils.

During rainy periods, additional effort will be required to properly prepare the site and establish/maintain an acceptable subgrade. The difficulty will increase in areas where clay or silty soils are exposed at the subgrade elevation. Grading contractors typically postpone grading operations during wet weather to wait for conditions that are more favorable. Contractors can typically disk or aerate the upper soils to promote drying during intermittent periods of favorable weather. When deadlines restrict postponement of grading operations, additional measures, such as undercutting and replacing saturated soils or chemical stabilization, can be utilized to facilitate placement of additional fill material.

Very loose and very soft/or unstable surficial silty or clayey soils may be encountered across the project site during wet weather or high-water table levels. Very soft and/or unstable materials also may be present in unexplored areas of the site. Soft and/or unstable soils should be undercut and recompacted in lifts or chemically stabilized. The project geotechnical engineer should observe these operations.

These design recommendations have been developed based on the previously described project characteristics and subsurface conditions encountered. If there are any changes in these project criteria, the geotechnical engineer should be retained to perform a review to determine if any modifications to the recommendations will be required. The findings of such a review should be presented in a supplemental report. If additional investigation is needed during construction, the geotechnical engineer should be retained to perform the additional services.

6.9 LATERAL EARTH PRESSURE COEFFICIENTS

Two water storage tanks are to be constructed as shown in figure 1. Based on the borings from this exploration, laboratory testing of the samples and reasonably similar soils on other projects, the following earth pressure coefficients are recommended for the in-place soils at the site for design purposes. The table below can be utilized for design of the long-term horizontal components of the at-rest earth pressure for the naturally occurring clayey and sandy soils, below and above the water table, respectively.

TABLE III

MATERIAL	Depth of Layer	UNIT WEIGHT, PCF wet	COHESION, C_U AND FRICTION ANGLE, ϕ	COEFFICIENT OF EARTH PRESSURE			EQUIVALENT FLUID DENSITY[1]	WALL ADHESION and FRICTION COEFFICIENT fs
	Feet			Ko	Ka	Kp	Pcf	
In-situ CLs	**Existing grade 0 to 4**	**115**	**0 psf, 25°**	**0.58**	**0.41**	**2.46**	**67, 93**	**0 psf, 0.46**
In-situ SMs	**4 to 18**	**116**	**0 psf, 25°**	**0.58**	**0.41**	**2.46**	**66, 93**	**0 psf , 0.47**
In-situ SMs	**18 to 38**	**117**	**0 psf, 30°**	**0.50**	**0.33**	**3.00**	**59, 90**	**0 psf, 0.58**
In-situ CLs	**38 to 60**	**118**	**0 psf, 27°**	**0.55**	**0.38**	**2.66**	**64, 93**	**0 psf, 0.51**

NOTES:1. AT-REST EARTH PRESSURE CONDITIONS, ABOVE AND BELOW THE WATER TABLE, RESPECTIVELY.

The earth pressure coefficients presented in the preceding table are based on our experience with similar projects with similar soil conditions and on the laboratory test results. These coefficients were estimated based on an assumed angle of internal friction for active pressure and passive pressure.

Walls or surfaces such as exterior retaining walls, which are permitted to rotate at the top, may be designed to resist “active” lateral earth pressure. Typically, a top rotation of about 0.5 inches per 10 feet height of wall is sufficient to develop active pressure conditions in soils similar to those that will be utilized for backfill of the wall.

The passive earth pressure coefficient (Kp) is normally used to evaluate the ultimate lateral resistance of the soil against the side of a foundation that aids in preventing sliding of the foundation. It should be noted that a considerable amount of deflection is required to fully mobilize the passive resistance. Therefore, we recommend that the passive earth pressure coefficient indicated in the above table be divided by a factor of safety of two to limit the amount of lateral deflection required to mobilize the passive resistance. Additional resistance to sliding is obtained due to friction along the base of the footing. The coefficient of sliding resistance is designated as “fs” in the preceding table.

The active and passive earth pressure coefficients presented herein are based upon the Rankine Theory, which assumes a vertical surface with horizontal backfill and neglects wall friction. A sloping soil backfill, or inclined surface will influence the values presented above.

6.10 RETAINING WALL

It is recommended that the walls be backfilled with a free draining material such as crushed stone, crushed concrete or coarse clean sand and compacted (reinforced fill). If water from the tanks are to be drained through the wall then crushed concrete shall be used for the reinforced fill. A drainage system may be necessary near or at the base of the walls to collect and remove water and prevent build-up of hydrostatic pressures.

The compacted mass unit weight of the backfill soil should be used with the above earth pressure coefficients to calculate lateral earth pressures. We recommend an in-place unit weight of 125 pcf be used for design. Lateral pressures arising from surcharge loading, earthquake loading, and ground water should be added to the above soil earth pressures to determine the total lateral pressures, which the structures must resist. In addition, transient loads imposed on the structures by construction equipment during backfilling should be taken into consideration during design and construction. Excessively heavy grading equipment (that could impose temporary excessive lateral pressures) shall not be allowed within ten feet (horizontally) of the`` structure.

The wall should be backfilled with free-draining granular fill such as sands (SW/SP), crushed stone, crushed concrete, No. 57 stone (ASTM D 448) or similar material (retained fill). The fill should be placed as a minimum in the zone defined by projecting a 1(H):1(V) line from the base of the wall to the finished subgrade elevation / ground surface. A layer of suitable filtration fabric, such as Amoco 4545, should be placed between the soil (retained soil) and backfill (retained fill) to reduce migration of soil fines into the drainage zone behind the wall. The recommended earth pressure coefficients are provided below:

TABLE IV

MATERIAL	DRY UNIT WEIGHT, PCF	FRICTION ANGLE, ϕ	COEFFICIENT OF EARTH PRESSURE			FRICTION COEFFICIENT
			Ko	Ka	Kp	
Reinforced Fill	125	30°	0.5	0.33	3.0	0.58
Retained Fill	125	30°	0.5	0.33	3.0	0.58
Retained Soils	125	25°	0.58	0.41	2.46	0.47
Foundation Soils	120	25°	0.58	0.41	2.46	0.47

The fill must be placed and compacted in a systematic manner. The fill lifts should not exceed eight (8) to ten (10) inches in loose thickness, depending on the compaction equipment used. The fill should be placed in lifts not exceeding four (4) inches when hand compaction equipment is utilized. Density tests should be performed on each lift. Because density tests cannot be performed in open graded aggregate, if utilized, the fill placement and compaction of open graded aggregate should be visually monitored.

The fill should be compacted to a minimum of 95% of the Standard Proctor value, and within ±2% of the optimum moisture as determined by ASTM D-698. The specifications should state that both density and moisture requirements should be met. Moisture conditioning of the soil may be necessary to achieve the desired moisture content and will be dependent upon weather conditions at the time of construction. The contractor should have equipment on site to both wet and dry the soil.

Due to the high velocities within the ditch during rains, a grouted Rip-Rap bedding underlain by a filter fabric should be strategically placed within the ditch outfall to protect the slopes and bottom of the ditch from scouring. Rip-Rap stone shall consist of field stone or rough unhewn quarry stone as nearly uniform in section as is practical. Stone shall be dense, resistant to air and water, and suitable for purpose intended. Unless otherwise specified, stone shall weigh up to 150 pounds and at least 60 percent of stones shall be over 100 pounds each.

In areas where the velocities of the flow decrease because of turns in the ditch and scour, may be of major concern we recommend a grout slurry be poured in the crevices of the stone bedding. The grout slurry should have a strength value of at least 3000 psi and weather the elements.

At the commencement and termination of the riprap armoring, the stone and filter fabric should be keyed to prevent scouring at those points. Also, at the discharge and inlet areas, the armoring should be keyed, grout should be placed to prevent seepage and undermining. These areas should be checked to confirm seepage has not weakened the natural subgrade.

6.11 PIPE LINE TRENCH & BEDDING MATERIAL FOR PIPE

Based on the subsurface exploration from the ground surface, soft to very stiff clay/sandy clay and silty sand materials were encountered to 60 feet. The CL/ SM material encountered in conjunction with high water table shall require additional steps to keep the banks of the excavations from caving. The onsite material was classified in accordance with ASTM D2487 & D2321. Also, the onsite material in the top two feet falling in the range of lean clay (CL) with Liquid Limit under 45 and Plasticity Index between 8 & 20 is acceptable and

has been classified as Class III if placed within 2 to 3 percent of the optimum moisture. Generally competent bearing material was encountered; however, if soft or loose material is encountered, then thicker bedding below the pipes may be necessary and should be checked by the geotechnical engineer at the time of placement. For this site, we generally recommend the following to be used:

a. A minimum of 4 inches below the bottom of the pipe, place **BEDDING** and then **HAUNCHING** material consisting of Class I material (Angular crushed stone or rock or crushed concrete, dense or open graded with little or no fines 1/4 to 1 ½ inches in size). Compacted to a minimum of 95 percent (ASTM D1557). If the bottom of the trench is found to be unstable, then the Geotechnical Engineer should be retained to check the bottom of the trench for stability and to determine if the bedding thickness should be increased.
b. **INITIAL BACKFILL** may be placed from the middle of the pipe to 12 inches above the pipe consist of Class I, II, III or IV or excavated material free from debris, organic materials and stone and must be placed and compacted to 95 percent compaction (ASTM D698).
c. **FINAL BACKFILL** material may be the excavated material free from debris, organic material and stone and compacted to a minimum 95 percent compaction (ASTM D698).

Note: In areas where heavy loading may affect pipe, then the Class I material should be placed to at least 12 inches above the top of the pipe and compacted to 95 percent of ASTM 1557.

6.12 MANHOLE BEDDING MATERIAL

a. A minimum of 12 inches of **BEDDING** material consisting of Class I material (Angular crushed stone or rock or crushed concrete, dense or open graded with little or no fines 1/4 to 1 ½ inches in size) extends to limits of excavation, loosely placed, then compacted to a minimum of 90 percent (ASTM D1557).
d. **BACKFILL** material may be the excavated material free from debris, organic materials and stone and must be placed and compacted to 95 percent compaction (ASTM D698).

Note: If compaction of the material is not easily achievable then Class I material may be used or a gap graded gravel material that allows self-compaction.

In "The Federal Register", Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P." This document was issued to better ensure the safety of workmen entering trenches or excavations. This federal regulation mandates that

excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the applicable OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if not closely followed, the owner and the contractor could be liable for substantial penalties.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations to maintain the stability of both the excavation sides and bottom. The contractor's "responsible person," as defined in the "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P," should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

7.0 SETTLEMENT & SLOPE STABILITY ANALYSIS

7.1 SETTLEMENT ANALYSIS

As requested, DRGeoES performed settlement analysis in settle3 program using existing soil profiles and parameters. Settle3 Performs soil settlement analysis of embankments, foundations, and surface loads. And also analyze immediate settlement, primary consolidation, and secondary settlement.

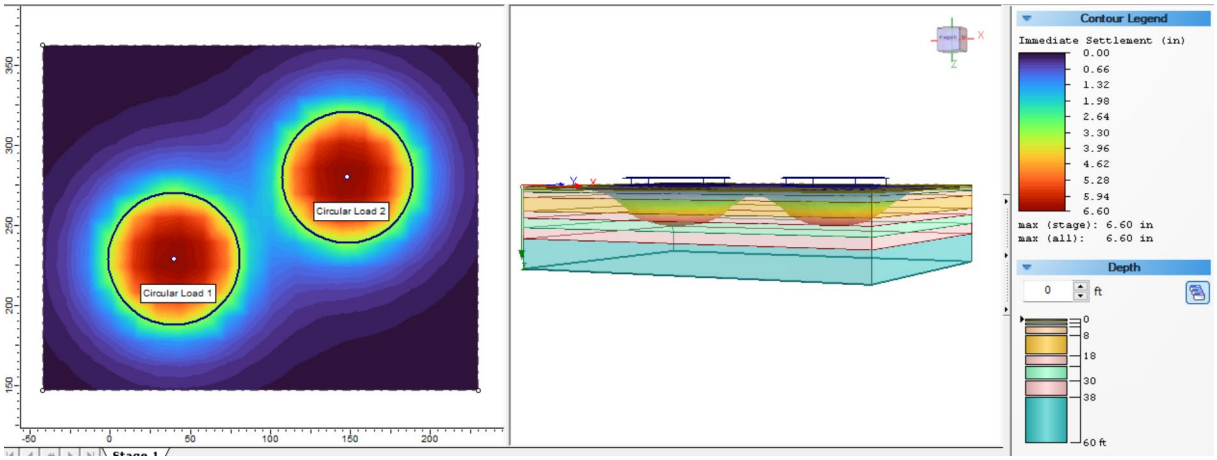
Based on the anticipated loads and existing soil parameters, the estimated foundation settlement to be 6.60 inches as shown below. However the majority of this settlement shall occur during construction due to the high sand content of the materials encountered.

Existing section soil property table and Settlement analysis

TALE IV

Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	mv (ft2/kips)	mvur (ft2/kips)	Su A (kips/ft2)	Su S	Su m
CLS1		0.086	0.115	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CLS 2		0.095	0.116	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CLS 3		0.096	0.111	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CLS 4		0.098	0.115	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
SM		0.1	0.117	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CH		0.098	0.129	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CL with Shale & Sand		0.095	0.118	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8

FIGURE V



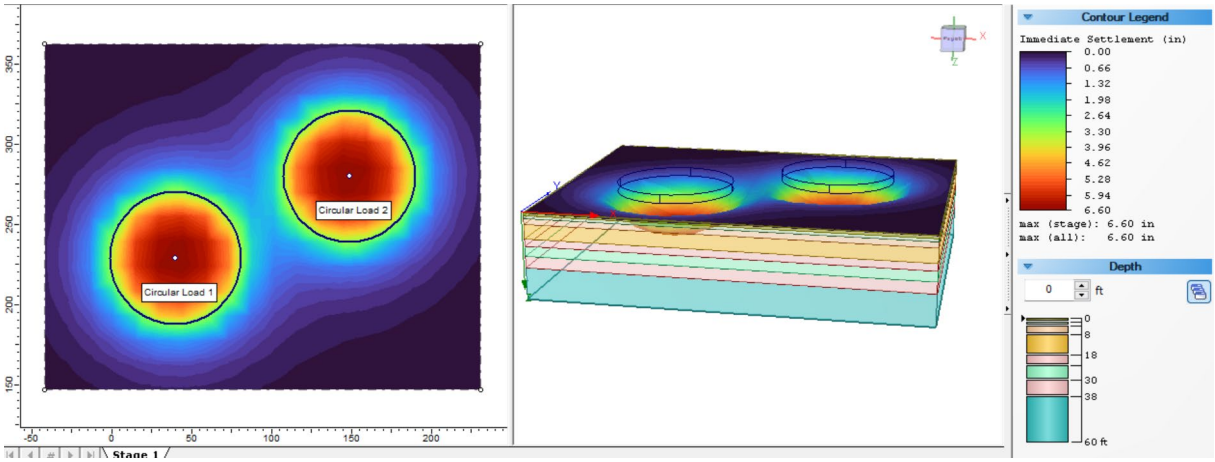
Section 2 with top 2 feet excavation and replacing with fill

Soil property table and Settlement analysis

TABLE V

Material Name	Color	Unit Weight (kips/ft3)	Sat. Unit Weight (kips/ft3)	Es (ksf)	Eur (ksf)	Material Type	mv (ft2/kips)	mvur (ft2/kips)	Su A (kips/ft2)	Su S	Su m
CLS1 FILL		0.096	0.11	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CLS 2		0.095	0.116	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CLS 3		0.096	0.111	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CLS 4		0.098	0.115	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
SM		0.1	0.117	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CH		0.098	0.129	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8
CL with Shale & Sand		0.095	0.118	208.9	208.9	Linear	0.00978	0.00978	0	0.2	0.8

FIGURE VI



7.2 SLOPE STABILITY ANALYSIS

The Slope Stability Analysis is based on subsurface conditions encountered on site during exploration, the observations during our site visit, and the slope surface profiles provided by the client. Stability Analysis of the slopes were carried out using the computer program SLIDE 2 with the Spencer method of Analysis. It is a simplified program using the limit equilibrium method which includes built-in finite element groundwater seepage analysis, probabilistic analysis, multi-scenario modeling, and support design. SLIDE 2 can be used for all types of soil and rock slopes, embankments, earth dams, and retaining walls. The safety factor is determined by the ratio of available resistance force versus the driving force in the horizontal direction.

One critical section is used to model the slope. Recommended new section of the slope is provided below.

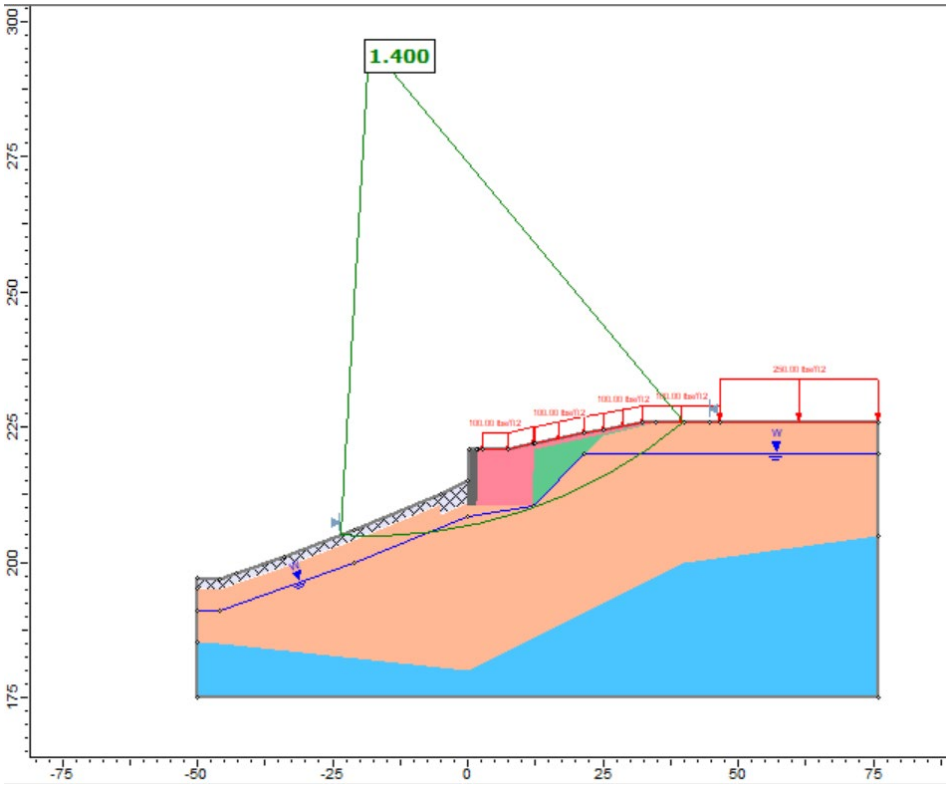
SOIL PARAMETERS USED FOR ANALYSIS

Table VI

Material Name	Color	Unit Weight (lbs/ ft3)	Sat. Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Grouted RIP RAP		105		Mohr-Coulomb	0	69	None			0
Facing		100	110	Mohr-Coulomb	0	85	None			0
Reinforced Fill		95	125	Mohr-Coulomb	0	30	None			0
Retained Fill		95	125	Mohr-Coulomb	0	30	None			0
Retained Soil		90	120	Mohr-Coulomb	0	25	Water Surface	Custom	1	
Foundation Soil		98	118	Mohr-Coulomb	2500	15	Water Surface	Custom	1	

Section 2: Recommended New Section

FIGURE VII



GENERAL COMMENTS ON SLOPE STABILITY

Additional comments related to the slope stability at the site are as follows:

• In order to prevent soil erosion at the slope surface, the vegetation on the existing
slopes must be preserved.
• Surface water must be directed away from the slope or carried down the slope in suitable conduits.

For this site, we recommend stabilizing the slopes by adding a surface cover to the slope, excavating, and changing the slope geometry, and using drainage to control the groundwater in slope material. Generally, conditions encountered consisted of rotational slide, cohesive soil, and presence of groundwater at 4 feet with surface water above grade.

Therefore, the recommended stabilization approach is to remove and replace (regrade and recompact) the existing grade to increase soil strength, factor of safety, and therefore stabilize the slope. However, necessary measures must be taken to prevent continued and further failures.

8.0 FOUNDATION RECOMMENDATIONS

As requested by the client a deep foundation information has been provided for this storage tanks. Ring wall structure with a tank floor (6 to 8 inches thick reinforced concrete floor) and circumferential ring wall footing, 12-15 inches thick and approximately 2 to 3 feet wide along the perimeter of the tank. The interior slab should be placed on compacted subgrade/fill at least 5 feet outside the footprint of the storage tanks. Please see the following sections for information.

Auger cast pile (ACP) foundation system has been considered for the above ground water storage tanks in this report. The structural design loading for the approximately 41 feet diameter, 30 feet high above ground water storage tanks was provided as 75K-150K of allowable vertical load per pile. Also, drilled shaft system may be used in lieu of ACP.

The following items should be considered during the preparation of construction documents and foundation installation:

- The geotechnical engineer or his representative shall be retained to observe the exposed bearing surface prior to fill placement. He should observe fill placement for the entire section and foundation concrete placement in order to verify that the conditions anticipated from the subsurface exploration are encountered. Any unsuitable bearing materials that are found should be removed and replaced with select fill in accordance with the section 6.5 of this report. The geotechnical engineer should be contacted for additional recommendations if needed.
- Wherever possible, the foundation concrete should be placed "neat," using the sides of the excavations as forms. Where this is not possible, the excavations created by forming the foundations must be backfilled with suitable structural fill and properly compacted. The building pad should be sloped to drain away from the building foundations.
- All bearing surfaces must be free of soft or loose soil prior to placing concrete.
- Concrete should be placed the same day the excavation is completed; and bearing materials should be verified by the geotechnical engineer's representative. If the excavation is left open for an extended period, or if the bearing surface is disturbed after the initial observation, then the bearing surface should be re-evaluated prior to concrete placement.

  Water should not be allowed to pond in the excavation prior to the concrete placement or above the concrete after the foundation is completed. Therefore, measures should be taken to direct water away from the foundation.

8.1 AUGER PRESSURE GROUTED PILES FOUNDATION (ACP)

Based on the soil conditions, auger cast piles can be used for the storage tanks in lieu of shallow foundation system. Auger Cast Pile (ACP) capacities were computed for 16 and 18-inch diameter piles of various lengths of embedment. Axial ACP capacities are presented in the table below. We recommend a minimum of two (2) piles be placed under spread footings. We recommend the pile foundations for the structure be firmly embedded into the dense silty sands at least 30 feet below existing grade (elevation 205 feet appx). Exact loads were unknown at the time of this report; therefore, pile recommendations were provided to cover a wide range of potential load conditions. Pile foundations at this site will derive their capacity from both frictional resistance between the piles and surrounding soils, and end bearing in very dense silty sands, at least 30 feet below existing grade.

Table IX Recommended Capacity Values 30 feet

DIAMETER (IN)	ULTIMATE COMPRESSION RESISTANCE (KIPS)	ULTIMATE TENSION RESISTANCE (KIPS)
16	234	146
18	276	164
20	280	182

Notes: 2. Auger cast piles are not recommended in permanent uplift unless the piles are properly designed in tension. Piles should be designed for full length reinforcing.

Provided below are the recommended factors of safety that may be considered for this project.

Table X

FIELD CAPACITY VERIFICATION METHOD	FACTOR OF SAFETY
Installation of 1 test pile. Static Load Test on minimum 1 test pile (after 14-day set-up time). Full time observation of production pile installation.	2
No static load test, but full-time observation of production pile installation	3

8.2 PILE GROUP & GROUP EFFICENCY

Pile supported structures consisting of groups greater than 4 to 6 piles, should have piles spaced at a minimum of three (3) pile diameters or 3 feet, whichever is greater, to minimize group effects. Consideration should be given to the group effect of piles in clusters of closely spaced piles in accordance with the building code. Experience has shown that the group effect of large clusters of piles is best accounted for in such materials through the use of the "Perimeter

Shear" formula. This formula assumes the material enclosed within the pile cluster tends to act as a large block and the forces resisting the movement of this block are compared with the total load on the block.

8.3 STRUCTURAL CAPACITY

The structural capacity of the piles should be verified by the designer at the time design loads are finalized. Pile capacities in excess of 100 kips have been predicted, these capacities should not be used in design until verified by a pile load test. Designer must adequately reinforce auger cast piles for tension, bending, and shear.

8.4 UPLIFT & TEMPORAY LOADS

Auger cast piles are not recommended in permanent uplift unless the piles are properly designed in tension. The recommended axial compressive capacities may be increased by 30 percent for temporary loads such as wind or seismic loads.

8.5 PILE SETTLEMENT, DRAG LOAD AND DOWN DRAG

Piles installed into the medium dense to dense sands below approximately 30 feet from existing grade at the site should experience minimal settlements. Top of pile movements of less than 1 inch are expected for the allowable design loads. These movements are associated with the loading from the structure and possible drown drag load due to liquefaction of the very soft silts and sand layer. This would be in addition to any fill-induced settlement, where applicable. Piles installed into medium dense to dense sands are not particularly susceptible to group settlement effects. Immediate settlements are estimated to be less than ½ inch. In addition to the immediate settlement, the estimated settlements should include any settlement caused by the elastic shortening of the pile itself.

8.6 ACP INSTALLATION

It is our opinion that the method, means, and sequence of the ACP installation should be the responsibility of the foundation contractor who should be experienced in auger cast pile installation. Piles at this site will derive their capacity from both frictional resistance between the piles and the surrounding soils and end bearing in the medium dense to dense sands at depths below 55 feet. We recommend all ACP be inspected by DRGeoES qualified geotechnical inspector. Care should be taken so that the ACP are not "over drilled" or grout flow stopped prematurely during withdrawal, as this could result in loss of ground, settlement at the surface, or necking of grout near the surface. Records of drilling times, grout placement time, drilled diameter, pile length, pumping rates, volumes, grout pressure, and relative difficulty should be

permanently kept. Grout mixes used in ACP should be designed to achieve the required strengths at the recommended flow rate/slumps.

8.7 PILE TEST PROGRAM

Design loads in this report should not be used without the verification of a pile test program. When using a factor of safety of 2, field load testing of test piles, Pile Integrity Testing (PIT) of the test piles on a minimum 2% of the production piles should be performed to verify the integrity of the piles and validate the predicted axial capacities. The test pile(s) will be used to determine the continuity and load capacity of the subsurface. Pile test locations should be selected based on the lowest strength profile and/or the least favorable soil conditions. The pile load tests should be conducted in accordance with accepted standards or in general accordance with accepted standards. The same type piles, pile sizes, ACP installation equipment, and installation procedures that will be used for the production piles should also be used for the test piles. The test piles should not be considered as job piles.

The contractor should be notified that overage factors achieved for the test piles should be consistent with those anticipated for their production piling. Production piling that exhibits overage factors that are notably less than those measured during the test pile program may require further evaluation of the production piles including additional PIT and possibly PDA Testing with a drop hammer to validate achieving the design capacity. The test piles should be loaded to a minimum of 250% and maximum 300% of its design capacity after verifying the grout strength, but not sooner than 14 days after installation. Ideally, the piles should be loaded to produce top of pile movement of 1.0 and 1.25 inches for 16-inch and 18-inch diameter piles respectively. This will allow for an accurate interpretation of the ultimate capacity.

8.8 RETAINING WALL FOUNDATION

Based on the anticipated loads and the results of our field exploration and laboratory analysis, the vertical reinforced “T” retaining wall loads may be supported by shallow footings. Strip footings (minimum 4 feet wide) for the wall may be designed for a maximum net allowable soil bearing pressure of 1,500 psf, respectively based on dead load plus design live load.

The design of reinforved “T’ walls should include provisions for liberally spaced weep holes to minimize the effects of overturning and lateral sliding.

9.0 CONSTRUCTION MONITORING

The recommendations presented in this report are based on information obtained from test borings. It is always possible that variations can occur between or away from test locations. The actual subsurface conditions should be confirmed during pipe installation. Field verification of site conditions is an essential part of the services provided by the geotechnical consultant. The geotechnical engineer shall be retained to confirm the provided recommendations and to make periodic visits to the site during site grading and foundation installation. If any variances from the described conditions are encountered, the geotechnical engineer of record shall be notified.

10.0 LIMITATIONS

The analyses, conclusions, and recommendations presented in this report are based upon the preceding project information and the results of the investigation. The data collected is representative of the locations tested. Variations are likely to occur at other locations throughout the site.

This report has been prepared in accordance with generally accepted standards of geotechnical engineering practice. No other warranty is expressed or implied.

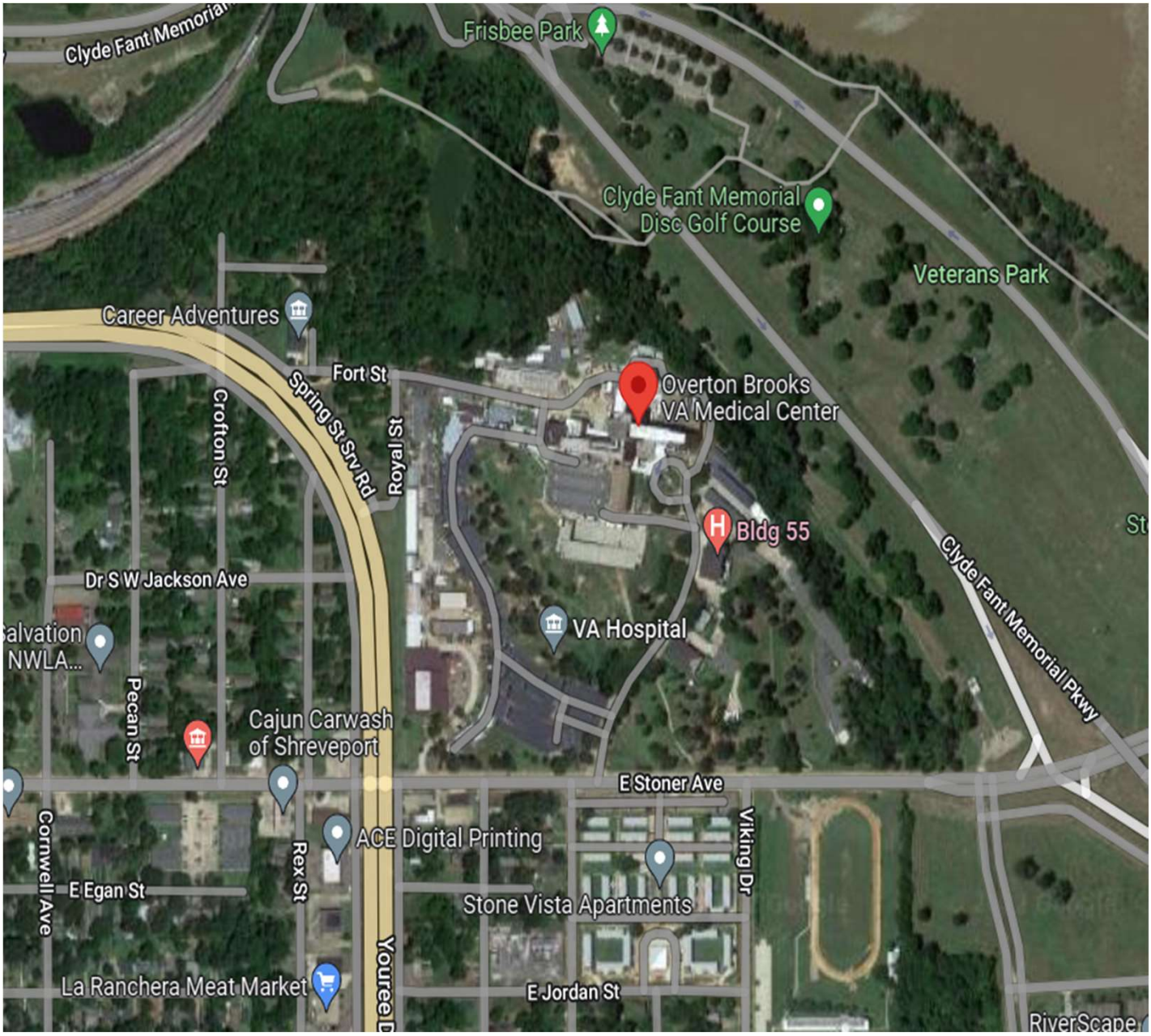
11.0 CLOSING

This report was prepared for Civil Design Group, LLC, and their design team for specific application to this project. The information in this report is not transferable. This report should not be used for a different development on the same property without first being evaluated by the engineer. The recommendations in this report were based on the information obtained from our field exploration, laboratory analysis, and engineering judgment regarding conditions encountered in the soil boring. It will be necessary to confirm the anticipated subsurface conditions during site grading and pavement installation.

This report is intended for use during design and preparation of specifications and may not address all conditions at the site during construction. Contractors reviewing this information should acknowledge that this document is for design information only.

An article published by the Association of Engineering Firms Practicing in the Geosciences (ASFE), titled *Important Information about Your Geotechnical Report*, has been included in the Appendix. We encourage all individuals to become familiar with the article.

APPENDIX



N

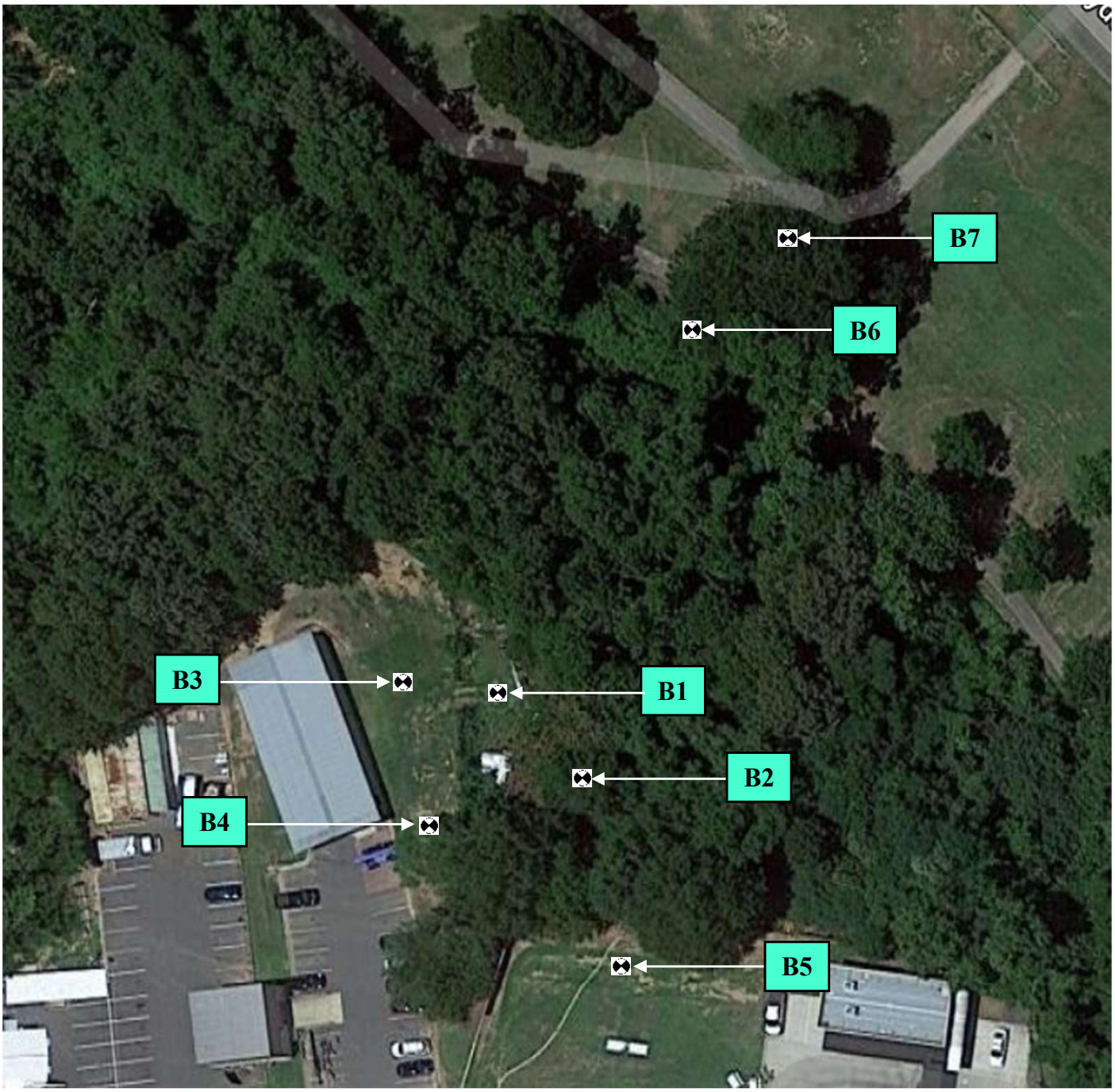
VICINITY MAP

DRGeoES, LLC

VA Medical Facility
Water Storage Tanks
Shreveport, LA

Scale: NTS

Project Number: DRGeoES 22060G



Source: Site Map provided by Client.

Estimated Boring Location -----

DRGeoES, LLC

VA Medical Facility
Water Storage Tanks
Shreveport, LA

Scale: NTS

Project Number: DRGeoES 22060G

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Dave Rambaran GEOSCIENCES, LLC
DRGeoES, LLC
9053 Mansfield Rd, Suite A
Shreveport, LA - 71118
Telephone: 318-671-1760
Email: contactus@drgeosciences.com

BORING NUMBER B1

CLIENT KSA
PROJECT NAME VA Medical Facility Water Storage Tanks
PROJECT NUMBER 22060G
PROJECT LOCATION Shreveport, LA
DATE STARTED 8/29/22 **COMPLETED** 8/29/22
GROUND ELEVATION 224 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Dave Rambaran Geosciences, LLC
GROUND WATER LEVELS:
DRILLING METHOD Drill rig
AT TIME OF DRILLING 23.00 ft / Elev 201.00 ft
LOGGED BY CC **CHECKED BY** DRR
AT END OF DRILLING ---
NOTES Hole caved in at 40 feet
24hrs AFTER DRILLING 25.00 ft / Elev 199.00 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL	% ML	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0		Silty Sand (SM); LOOSE, Brown	SS 1		4-5-4 (9)			14					
		MEDIUM DENSE, Brown	SS 2		3-4-7 (11)			16					
		MEDIUM DENSE, Tan	SS 3		5-9-12 (21)			18					
		MEDIUM DENSE, Tan	ST 4	2.8	10-10-11 (21)		100	19					
		DENSE, Tan	SS 5		15-20-18 (38)			16					
10													
		DENSE, Tan	SS 6		11-28-20 (48)			17		35			35
		DENSE, Tan	SS 7		11-18-20 (38)			17					
20													
		DENSE, Tan	SS 8		15-18-21 (39)			18					
		DENSE, Tan	SS 9		13-18-22 (40)		100	18					
30													
		DENSE, Tan	SS 10		15-18-20 (38)			21	9	19			28
		Clay (CL); VERY STIFF with Sand, Grey Shale	ST 11	3.3	15-12-13 (25)			19					
40													
		VERY STIFF with Sand, Grey Shale	ST 12	3.5	13-13-14 (27)			23					
		VERY STIFF with Sand, Grey Shale	ST 13	3.1	13-12-13 (25)		96	21	62	15	43	20	77
50													
		VERY STIFF with Sand, Grey Shale	ST 14	3.4	13-13-13 (26)			21					
		HARD with Sand, Grey Shale	SS 15		15-18-20 (38)			19					
60													

(Continued Next Page)

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Dave Rambaran GEOSCIENCES, LLC

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BORING NUMBER B1

CLIENT KSA

PROJECT NAME VA Medical Facility Water Storage Tanks

PROJECT NUMBER 22060G

PROJECT LOCATION Shreveport, LA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL % ML	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
		Bottom of borehole at 60.0 feet.										

DRGeoES, LLC
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BORING NUMBER B2

CLIENT KSA
PROJECT NAME VA Medical Facility Water Storage Tanks
PROJECT NUMBER 22060G
PROJECT LOCATION Shreveport, LA
DATE STARTED 8/30/22 **COMPLETED** 8/30/22
GROUND ELEVATION 223 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Dave Rambaran Geosciences, LLC
GROUND WATER LEVELS:
DRILLING METHOD Drill rig
AT TIME OF DRILLING 7.00 ft / Elev 216.00 ft
LOGGED BY CC **CHECKED BY** DRR
AT END OF DRILLING ---
NOTES Hole caved in at 18 feet
24hrs AFTER DRILLING 13.00 ft / Elev 210.0 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL	% ML	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0		Sandy Clay (CL); FIRM, Brown	SS 1		2-2-3 (5)		86	20	40	11	40	22	51
		Silty Sand (SM); MEDIUM DENSE, Tan	SS 2		2-3-7 (10)			15					
5		DENSE, Tan	SS 3		10-18-17 (35)			11					
	▽	DENSE, Tan	SS 4		13-16-20 (36)		103	10					40
		DENSE, Tan	SS 5		16-15-18 (33)			14	14	26			
10													
	▼	DENSE, Tan	SS 6		15-17-20 (37)			16					
15													
		DENSE, Tan	SS 7		17-18-18 (36)			19					
20													
		DENSE, Tan	SS 8		12-15-17 (32)		103	21	10	23			33
25													
		DENSE, Tan	SS 9		20-50-50 (100)			27					
30													

Bottom of borehole at 30.0 feet.

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DRGeoES, LLC
9053 Mansfield Rd, Suite A
Shreveport, LA - 71118
Telephone: 318-671-1760
Email: contactus@drgeosciences.com

BORING NUMBER B3

CLIENT KSA
PROJECT NAME VA Medical Facility Water Storage Tanks
PROJECT NUMBER 22060G
PROJECT LOCATION Shreveport, LA
DATE STARTED 8/30/22 **COMPLETED** 8/30/22
GROUND ELEVATION 227 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Dave Rambaran Geosciences, LLC
GROUND WATER LEVELS:
DRILLING METHOD Drill rig
AT TIME OF DRILLING 8.00 ft / Elev 219.00 ft
LOGGED BY CC **CHECKED BY** DRR
AT END OF DRILLING ---
NOTES Hole caved in at 20 feet
24hrs AFTER DRILLING 15.00 ft / Elev 212.00 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL	% ML	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0		Silty Sand (SM); MEDIUM DENSE, Tan	SS 1		2-3-6 (9)			18					
		MEDIUM DENSE, Tan	SS 2		9-9-10 (19)			14	15	26			41
5		MEDIUM DENSE, Tan	SS 3		8-11-13 (24)		103	13					
		MEDIUM DENSE, Tan	SS 4		11-13-17 (30)			11					
		DENSE, Brown	SS 5		15-18-27 (45)			17					
10													
		DENSE, Tan	SS 6		13-19-21 (40)			17					
15													
		DENSE, Tan	SS 7		16-20-22 (42)		103	17					
20													
		DENSE, Tan	SS 8		19-24-28 (52)			19	10	28			38
25													
		MEDIUM DENSE, Brown	SS 9		8-9-9 (18)			20					
30													

Bottom of borehole at 30.0 feet.

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Shreveport, LA - 71118
Telephone: 318-671-1760
Email: contactus@drgeosciences.com

BORING NUMBER B4

CLIENT KSA
PROJECT NAME VA Medical Facility Water Storage Tanks
PROJECT NUMBER 22060G
PROJECT LOCATION Shreveport, LA
DATE STARTED 8/30/22 **COMPLETED** 8/30/22
GROUND ELEVATION 230 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Dave Rambaran Geosciences, LLC
GROUND WATER LEVELS:
DRILLING METHOD Drill rig
AT TIME OF DRILLING 8.00 ft / Elev 222.00 ft
LOGGED BY CC **CHECKED BY** DRR
AT END OF DRILLING ---
NOTES Hole caved in at 20 feet
24hrs AFTER DRILLING 10.00 ft / Elev 220.00 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL	% ML	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0		Sandy Clay (CL); VERY STIFF, Tan	SS 1		5-10-12 (22)			18					
		VERY STIFF, Tan	SS 2		11-14-13 (27)		96	15			31	15	
5		VERY STIFF, Tan	SS 3		9-11-10 (21)			16	50	11			61
		Silty Sand (SM); MEDIUM DENSE, Tan	SS 4		10-11-10 (21)			10					
		MEDIUM DENSE, Tan	SS 5		11-12-10 (22)		103	18					
10													
15		MEDIUM DENSE, Tan	SS 6		9-11-7 (18)			23		43			43
20		MEDIUM DENSE, Tan	SS 7		9-9-10 (19)			18					
25		Fat Clay (CH); VERY STIFF, Tan	SS 8		7-7-12 (19)			29	70	27	59	38	97
30		HARD, Brown	ST 9	5.4			100	25					

Bottom of borehole at 30.0 feet.

COPY DMCP LOG WITH N VALUE FOR LA - COMPANY TEMPLATE FOR PROJECT.GDT - 11/19/22 13:36 - C:\USERS\RANAR\DROPBOX\OLD DROPBOX COPIED\DRGEOES\PROJECT FOLDER 2022\22060G VA MEDICAL FACILITY WATER STORAGE TANKS SHREVEPORT

COPY DMCP LOG WITH N VALUE FOR LA - COMPANY TEMPLATE FOR PROJECT.GDT - 11/19/22 13:36 - C:\USERS\RANAR\DROPBOX\OLD DROPBOX COPIED\DRGEOES\PROJECT FOLDER 2022\22060G VA MEDICAL FACILITY WATER STORAGE TANKS SHREVEPORT

Dave Rambaran GEOSCIENCES, LLC
DRGeoES, LLC
9053 Mansfield Rd, Suite A
Shreveport, LA - 71118
Telephone: 318-671-1760
Email: contactus@drgeosciences.com

BORING NUMBER B5

CLIENT KSA
PROJECT NAME VA Medical Facility Water Storage Tanks
PROJECT NUMBER 22060G
PROJECT LOCATION Shreveport, LA
DATE STARTED 8/30/22 **COMPLETED** 8/30/22
GROUND ELEVATION 237 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Dave Rambaran Geosciences, LLC
GROUND WATER LEVELS:
DRILLING METHOD Drill rig
▽ **AT TIME OF DRILLING** 8.00 ft / Elev 229.0 ft
LOGGED BY CC **CHECKED BY** DRR
AT END OF DRILLING ---
NOTES
▼ **24hrs AFTER DRILLING** 8.00 ft / Elev 229.0 ft

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL	% ML	ATTERBERG LIMITS LIQUID LIMIT	ATTERBERG LIMITS PLASTICITY INDEX	FINES CONTENT (%)
0		Sany Clay (CL); FIRM, Brown	SS 1		1-3-2 (5)			17					
		VERY STIFF, Grey and Red	ST 2	2.1				21	50	14	41	24	64
5		HARD, Tan	ST 3	4.6			96	20					
		VERY STIFF, Tan	SS 4		7-8-8 (16)			16					
▼		VERY STIFF, Tan	SS 5		7-9-11 (20)			16	50	13	29	13	63
10													
		VERY STIFF, Tan	SS 6		10-12-11 (23)			19					
15													
		Silty Sand (SM); MEDIUM DENSE, Tan	SS 7		12-10-21 (31)			17					
20													
		MEDIUM DENSE, Tan	SS 8		13-19-20 (39)		103	17					
25													
		MEDIUM DENSE, Tan	SS 9		19-27-29 (56)			17	11	28			39
30													

Bottom of borehole at 30.0 feet.

COPY DMCP LOG WITH N VALUE FOR LA - COMPANY TEMPLATE FOR PROJECT.GDT - 11/19/22 13:36 - C:\USERS\RANAR\DROPBOX\OLD DROPBOX COPIED\DRGEOES\PROJECT FOLDER 2022\22060G VA MEDICAL FACILITY WATER STORAGE TANKS SHREVEPORT

DRGeoES, LLC
9053 Mansfield Rd, Suite A
Shreveport, LA - 71118
Telephone: 318-671-1760
Email: contactus@drgeosciences.com

BORING NUMBER B6

CLIENT KSA
PROJECT NAME VA Medical Facility Water Storage Tanks
PROJECT NUMBER 22060G
PROJECT LOCATION Shreveport, LA
DATE STARTED 10/14/22 **COMPLETED** 10/14/22
GROUND ELEVATION 186 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Dave Rambaran Geosciences, LLC
GROUND WATER LEVELS:
DRILLING METHOD Auger Boring
AT TIME OF DRILLING --- None Encountered
LOGGED BY CC **CHECKED BY** DRR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL	% ML	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		Silty Sand (SM); MEDIUM DENSE, Brown	AU 1		8-9-9 (18)			6					
2.5		MEDIUM DENSE, Brown	AU 2		9-9-10 (19)		103	14		36			36
5.0		Clayey Sand (SC); MEDIUM DENSE, Brown	AU 3	2.6	10-10-11 (21)			19					
7.5		MEDIUM DENSE, Brown	AU 4	3.6	4-5-4 (9)		103	18	36	18			44
		MEDIUM DENSE, Brown	AU 5	5.0	5-4-4 (8)			18			33	15	
10.0													

Bottom of borehole at 10.0 feet.

COPY DMCP LOG WITH N VALUE FOR LA - COMPANY TEMPLATE FOR PROJECT.GDT - 11/19/22 13:36 - C:\USERS\RANAR\DROPBOX\OLD DROPBOX COPIED\DRGEOES\PROJECT FOLDER 2022\22060G VA MEDICAL FACILITY WATER STORAGE TANKS SHREVEPORT

Dave Rambaran GEOSCIENCES, LLC

DRGeoES, LLC
9053 Mansfield Rd, Suite A
Shreveport, LA - 71118
Telephone: 318-671-1760
Email: contactus@drgeosciences.com

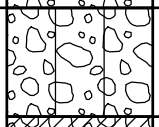
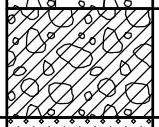
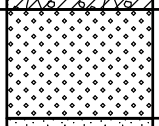
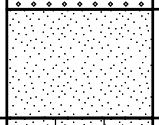
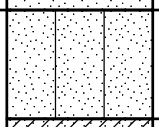
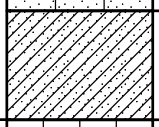
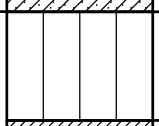
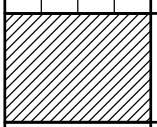
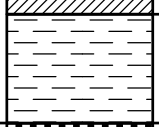
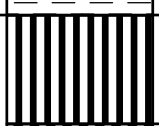
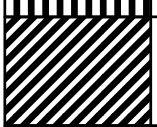
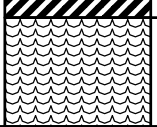
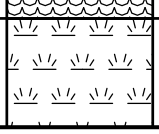
BORING NUMBER B7

CLIENT KSA
PROJECT NAME VA Medical Facility Water Storage Tanks
PROJECT NUMBER 22060G
PROJECT LOCATION Shreveport, LA
DATE STARTED 10/14/22 **COMPLETED** 10/14/22
GROUND ELEVATION 169 ft **HOLE SIZE** 4 inches
DRILLING CONTRACTOR Dave Rambaran Geosciences, LLC
GROUND WATER LEVELS:
DRILLING METHOD Auger Boring
AT TIME OF DRILLING --- None Encountered
LOGGED BY CC **CHECKED BY** DRR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

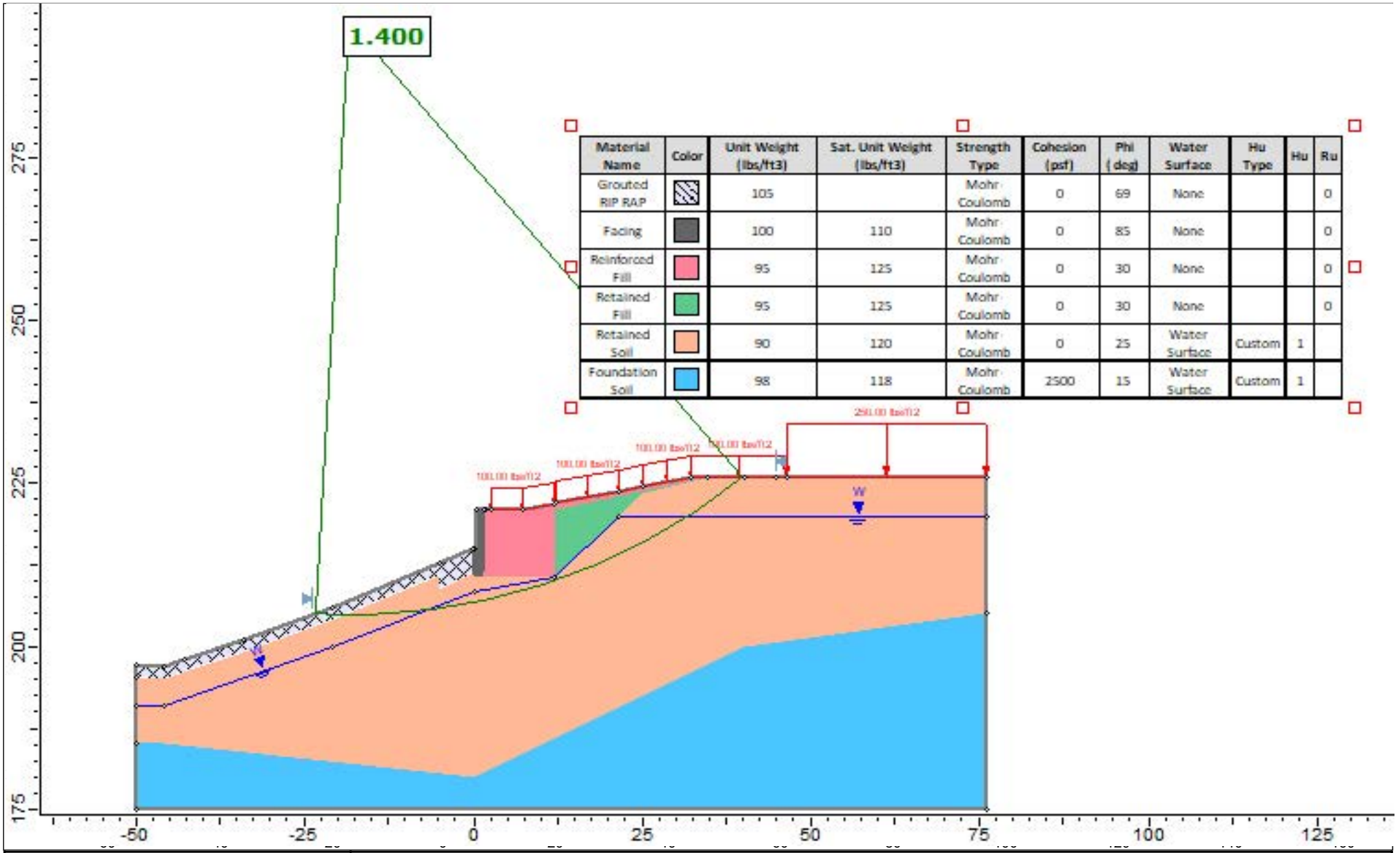
DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	TORVANE Su (ksf)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% CL	% ML	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0.0		Silty Sand (SM); MEDIUM DENSE, Brown	AU 1		6-6-8 (14)			4					
2.5		MEDIUM DENSE, Tan	AU 2		9-10-10 (20)		103	10		15			15
5.0		Silt (ML); VERY STIFF with Sand, Brown	AU 3	2.9	5-8-10 (18)			14					
7.5		VERY STIFF with Sand, Brown	AU 4	3.3	6-8-9 (17)		96	11		84			84
10.0		VERY STIFF with Sand, Brown	AU 5	3.8	5-7-7 (14)			17					

Bottom of borehole at 10.0 feet.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu	Ru
Grouted RIP RAP		105		Mohr-Coulomb	0	69	None			0
Facing		100	110	Mohr-Coulomb	0	85	None			0
Reinforced Fill		95	125	Mohr-Coulomb	0	30	None			0
Retained Fill		95	125	Mohr-Coulomb	0	30	None			0
Retained Soil		90	120	Mohr-Coulomb	0	25	Water Surface	Custom	1	
Foundation Soil		98	118	Mohr-Coulomb	2500	15	Water Surface	Custom	1	

rocscience

SLIDEINTERPRET 9.010

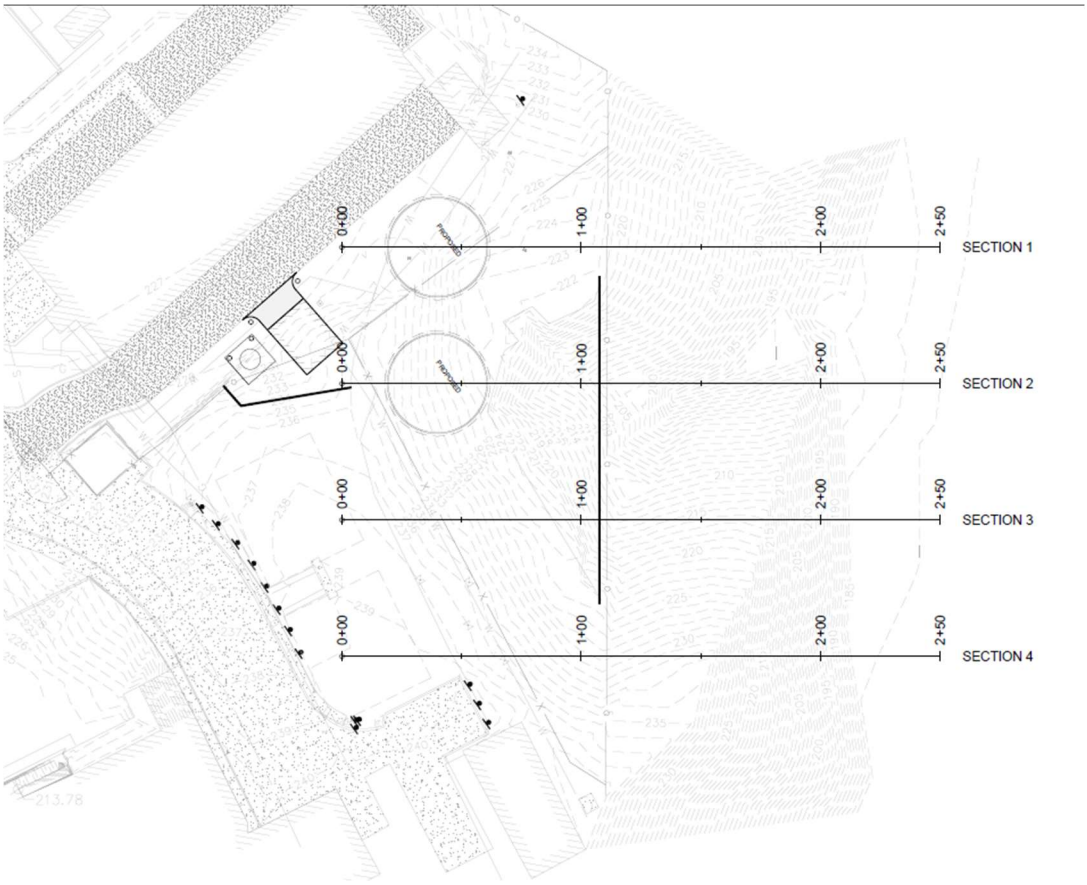
Project	VA Med Storage Tanks		
Group	Group 1	Scenario	Master Scenario
Drawn By		Company	DRGeoES, LLC
Date	5/30/2023, 1:41:01 PM	File Name	SSA Slide2 FOS.slmd

1.1 SLOPE STABILITY ANALYSIS EXISTING CONDITION

The Slope Stability Analysis is based on subsurface conditions encountered on site during exploration, the observations during our site visit, and the slope surface profiles provided by the client. Stability Analysis of the slopes were carried out using the computer program SLIDE 2 with the Spencer method of Analysis. It is a simplified program using the limit equilibrium method which includes built-in finite element groundwater seepage analysis, probabilistic analysis, multi-scenario modeling, and support design. SLIDE 2 can be used for all types of soil and rock slopes, embankments, earth dams, and retaining walls. The safety factor is determined by the ratio of available resistance force versus the driving force in the horizontal direction.

One critical section named section 2 is used to model the slopes. Two cases were analyzed: 1) existing section 2) recommended new section of the slope.

FIGURE I



SOIL PARAMETERS USED FOR ANALYSIS

Existing slope soil property Table I

Material Name	Color	Unit Weight (lbs/ ft3)	Sat. Unit Weight (lbs/ ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
CLS		86	115	Mohr-Coulomb	600	4	Water Surface	Custom	1
CLS.1		96	115	Mohr-Coulomb	1500	14	Water Surface	Custom	1
SM		105	125	Mohr-Coulomb	0	15	Water Surface	Custom	1
Weak Layer		80	120	Mohr-Coulomb	300	4	Water Surface	Custom	1
CL with Sand/ Shale		103	130	Mohr-Coulomb	3000	14	Water Surface	Custom	1

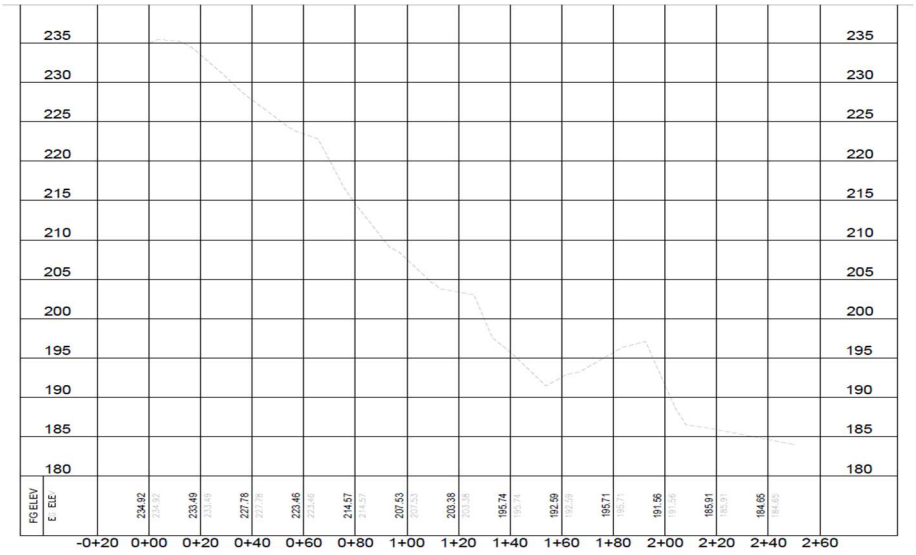
New slope with fill, soil property Table II

Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Water Surface	Hu Type	Hu
FILL		96	110	Mohr-Coulomb	2000	14	Water Surface	Custom	1
CLS		80	95	Mohr-Coulomb	600	4	Water Surface	Custom	1
CLS.1		96	115	Mohr-Coulomb	1500	14	Water Surface	Custom	1
SM		105	125	Mohr-Coulomb	0	15	Water Surface	Custom	1
CL with Sand/Shale		96	118	Mohr-Coulomb	3000	14	Water Surface	Custom	1
Weak Layer		80	120	Mohr-Coulomb	300	4	Water Surface	Custom	1

Table-III Factor of Safety

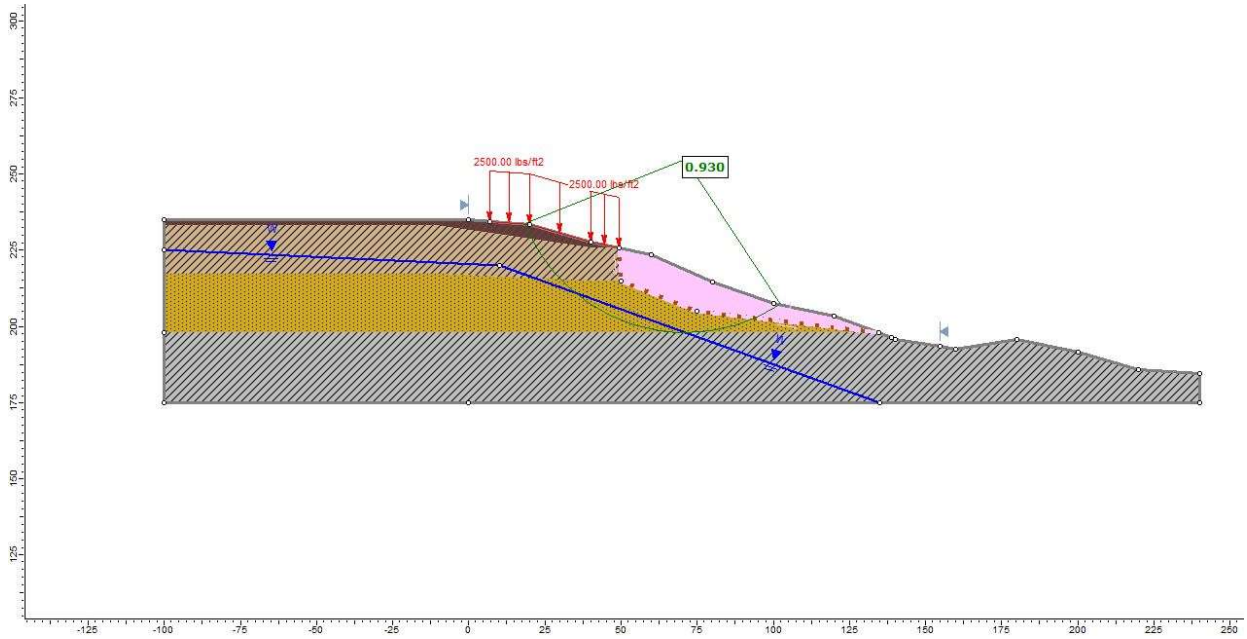
Serial No	Section	Existing Section (FS)	New Section with Fill (FS)
1	2	0.930	1.022

Critical Section No. 2



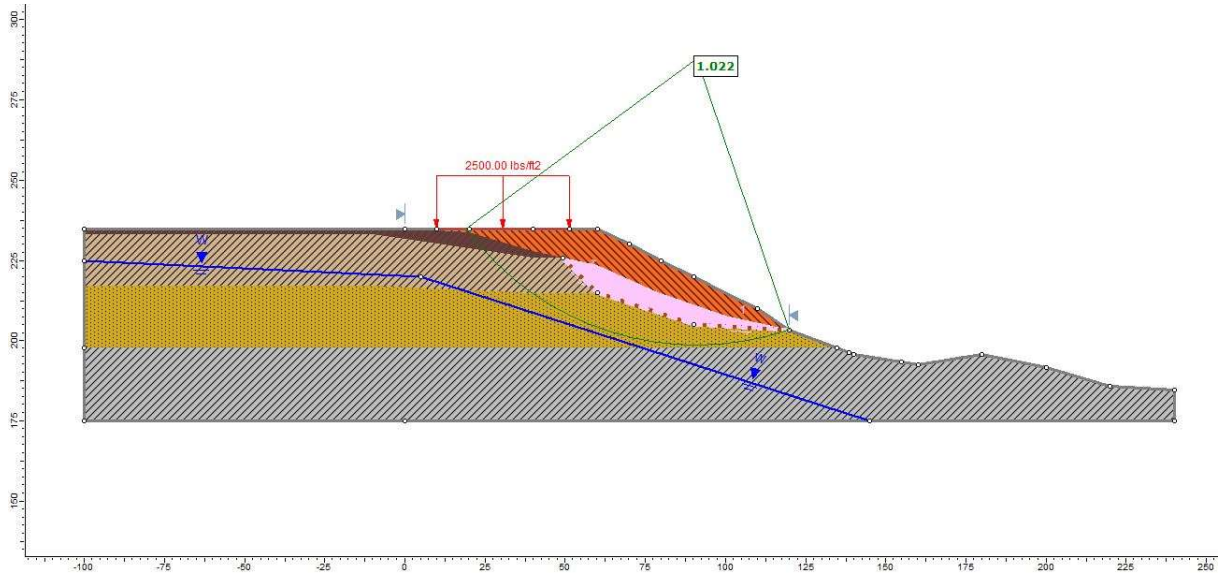
Section 2: Existing Section

FIGURE IX



Section 2: Recommended New Section with Select Fill

FIGURE X



DESIGN CALCULATIONS

FOR

VA MEDICAL FACILITY STORAGE TANKS

510 E STONER AVE
SHREVEPORT, CADDO PARISH

PROJECT NO.: 23ERS002
PLANS & CALCULATIONS DATED 07/19/2023, REV 2

PREPARED BY:

Earth Retention
Design + Supply + Support

231 ROPE MILL PARKWAY, WOODSTOCK, GA 30188 | T: (678) 903-3614 | F: (678) 550-9862

Project: 23ERS002 - VA MEDICAL STORAGE TANKS

Site: SHREVEPORT, LA

Date: 7/19/2023

Wall: 1_HDGWWF_F2

Earth Retention

231 Rope Mill Parkway
Woodstock, GA 30188
T) 678-903-3614
www.EARTHRETENTION.com

Project: 23ERS002 - VA MEDICAL STORAGE TANKS

Site: SHREVEPORT, LA

Date: 7/19/2023

Section Summary Report

Earth Retention

231 Rope Mill Parkway
Woodstock, GA 30188
T) 678-903-3614
www.EARTHRETENTION.com

Project Design Inputs

Design Standard AASHTO 2020 (LRFD) Simplified

Conventional/Common Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.55	0.00
SLIDING	Resistance - Sliding	0.90	0.00

Conventional/Common Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake Load	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Reinforced Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFCT	Load - Vehicular Collision Force	0.00	1.00
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.65	0.00
TCONN	Resistance - Connection	0.80	0.00
PULLOUT	Resistance - Pullout	0.70	0.00
SLIDING	Resistance - Sliding	1.00	0.00
TAL	Resistance - Tensile	0.80	0.00

Reinforced Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.90	0.00
COMBIMPCO	Resistance - Combined Static/EQ Connection	1.00	0.00
COMBIMPPO	Resistance - Combined Static/EQ Pullout	1.00	0.00
COMBIMPTO	Resistance - Combined Static/EQ Tensile	1.00	0.00
RFconn	Resistance - Seismic Frictional Connection Factor	0.80	0.00
RFpull	Resistance - Seismic Frictional Pullout Factor	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Design Factors

Term	Description	Minimum (as appl.)	Maximum (as appl.)
RC	Reinforced coverage ratio	1.00	0.00

Selected Facing Unit

Licensor/Product Line: User defined
Name: WWF - 1/2 Length

Facing Height	Hu	1.50 ft
Facing Width	Lu	4.83 ft
Facing Depth	Wu	1.50 ft
Facing Weight	Xu	115 lb/ft³
Center of Gravity	Gu	0.75 ft
Setback	Δu	0.08 ft

Batter	ω	3.18 °	
Cap Height	Hcu	0.00 ft	
Initial Shear Capacity	au	800.00 lb/ft	
Apparent Shear Angle	λu	30.00 °	
Maximum Shear Capacity	Vu(max)	2000.00 lb/ft	

Selected Reinforcement Types

Reinforcements

ERS-3 - ERS-3 **Supplier: User defined, Fill Type: Sands**

Tult	3,600.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	1,667.21 lb/ft	Cds	0.80	Ci	0.80	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

ERS-2 - ERS-2 **Supplier: User defined, Fill Type: Sands**

Tult	2,700.00 lb/ft	RFcr	1.51	RFid	1.11	RFd	1.30
LTDS	1,239.14 lb/ft	Cds	0.80	Ci	0.80	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

ERS-5 - ERS-5 **Supplier: User defined, Fill Type: Sands**

Tult	5,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	2,500.81 lb/ft	Cds	0.85	Ci	0.85	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

ERS-8 - ERS-8 **Supplier: User defined, Fill Type: Sands**

Tult	7,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	3,427.04 lb/ft	Cds	0.80	Ci	0.80	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

Selected Soil Types

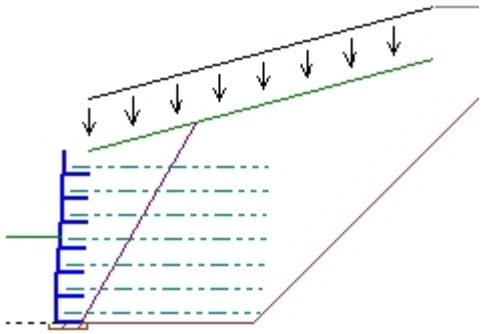
Soil Zone	Soil Type	Friction Angle φ	In Situ Density γ [lb/ft³]	Cohesion Cf [lb/ft²]
Infill (i)	REINF	30°	125.00	n/a
Retained (r)	RET	25°	125.00	n/a
Foundation (f)	FDN	25°	120.00	0.00
Base (b)	GW	36°	140.00	n/a
Drainage (d)	GP	38°	110.00	n/a

Soil Glossary

CH:	Inorganic clays, high plasticity
CL:	Inorganic clays, low to medium plasticity, gravelly, sandy, silty, lean clays
FDN:	FOUNDATION SOIL
GC:	Clayey gravels, poorly graded gravel-sand-clay mixtures
GM:	Silty gravels, poorly graded gravel-sand-silt mixtures
GP:	1/2"-3/4" clean crushed stone or crushed gravel
GW:	Well-graded gravels, gravel-sand. Little or no fines.
MH:	Inorganic clayey silts, elastic silts

ML:	Inorganic silts, very fine sands, silty or clayey, slight plasticty
REINF:	REINFORCED FILL
RET:	RETAINED SOIL
SC:	Clayey sands, poorly graded sand-clay mixtures
SM:	Silty sands, poorly graded sand-silt mixtures
SP:	Poorly-graded sands, gravelly sands. Little or no fines.
SW:	Well-graded sands, gravelly sands. Little or no fines.

Section #3 at Station 19.33
Report Date July 19, 2023
Designer RLC
Design Standard AASHTO 2020 (LRFD) Simplified
Design Static and Seismic
Unit of Measure U.S./Imperial
Selected Facing Unit
Licensor/Product Line: User defined
Name: WWF - 1/2 Length
Seismic As 0.13



Soil Parameters

Soil Zone	Soil Type	Friction Angle φ	In Situ Density γ [lb/ft³]	Cohesion Cf [lb/ft²]
Infill (i)	REINF	30°	125.00	n/a
Retained (r)	RET	25°	125.00	n/a
Foundation (f)	FDN	25°	120.00	0.00
Base (b)	GW	36°	140.00	n/a
Drainage (d)	GP	38°	110.00	n/a

Section Details

Section Height	10.50	Back Slope	14.93°	LL Surcharge	100	DL Surcharge	250
Design Height	10.50 ft	Crest Offset	30.00 ft	LL Offset	0.00 ft	DL Offset	44.50 ft
Embedment	5.16 ft	Wall Batter	3.18°	Toe Slope	0.00°	Toe Offset	0.00 ft

Reinforced Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFCT	Load - Vehicular Collision Force	0.00	1.00
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.65	0.00
TCONN	Resistance - Connection	0.80	0.00
PULLOUT	Resistance - Pullout	0.70	0.00
SLIDING	Resistance - Sliding	1.00	0.00
TAL	Resistance - Tensile	0.80	0.00

Reinforced Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.90	0.00
COMBIMPCO	Resistance - Combined Static/EQ Connection	1.00	0.00
COMBIMPPO	Resistance - Combined Static/EQ Pullout	1.00	0.00
COMBIMPTO	Resistance - Combined Static/EQ Tensile	1.00	0.00
RFconn	Resistance - Seismic Frictional Connection Factor	0.80	0.00
RFpull	Resistance - Seismic Frictional Pullout Factor	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Reinforcements

ERS-5 - ERS-5 **Supplier: User defined, Fill Type: Sands**

Tult	5,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	2,500.81 lb/ft	Cds	0.85	Ci	0.85	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

Analysis Results

*** Embedment is not included in Bearing Capacity**

*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

External Static	CDR			
Bearing Capacity	1.86	Bearing Pressure - Strength	2471.87	lb/ft²
Overturning	3.52	Bearing Pressure - Service	1745.33	lb/ft²
Base Sliding	1.09	Eccentricity for Overturning	0.59	ft
Internal Sliding	1.38	Max Eccentricity	3.00	ft

External Seismic	CDR			
Bearing Capacity	3.63	Bearing Pressure	1731.91	lb/ft²
Overturning	4.32	Eccentricity for Overturning	0.65	ft
Base Sliding	1.33	Max Eccentricity	3.96	ft

Internal Static Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
7	219.50	ERS-5	12.00	4.05	5.81
6	218.00	ERS-5	12.00	5.84	5.29
5	216.50	ERS-5	12.00	6.37	4.03
4	215.00	ERS-5	12.00	6.94	3.25
3	213.50	ERS-5	12.00	7.53	2.73
2	212.00	ERS-5	12.00	8.12	2.35
1	210.50	ERS-5	12.00	18.00	4.25

Internal Seismic Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
7	219.50	ERS-5	12.00	3.99	6.56
6	218.00	ERS-5	12.00	5.82	6.03
5	216.50	ERS-5	12.00	6.55	4.69
4	215.00	ERS-5	12.00	7.27	3.83
3	213.50	ERS-5	12.00	7.99	3.24
2	212.00	ERS-5	12.00	8.71	2.81
1	210.50	ERS-5	12.00	18.39	4.93

Internal Compound Stability

10 Lowest Static

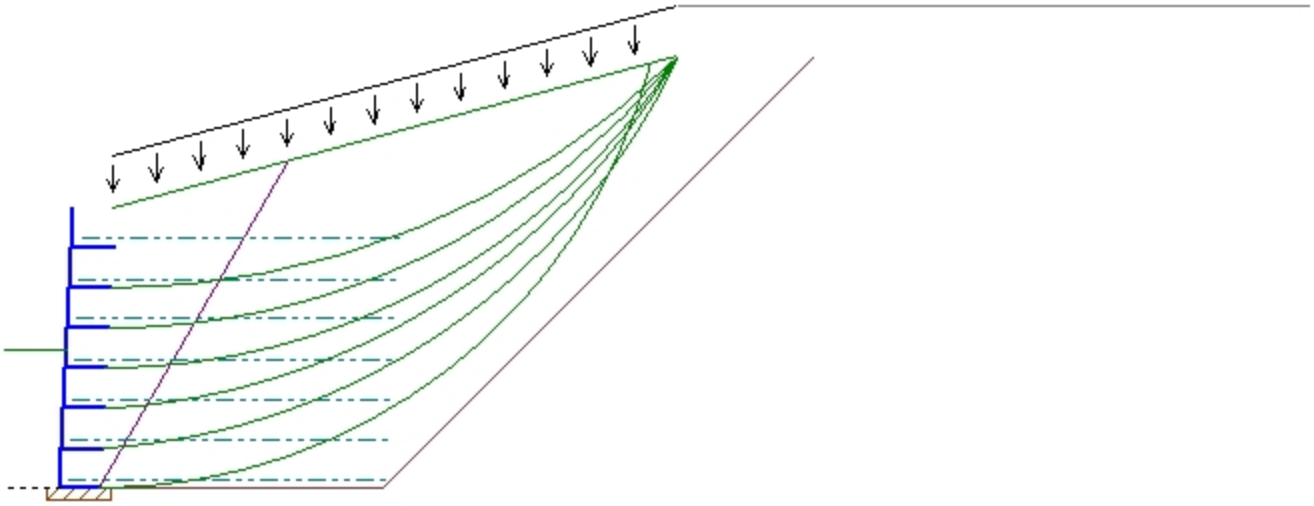
Radius Point	Entry Point	Exit Point	Result	Status
1	9	1	1.55	Pass
2	10	2	1.69	Pass
2	10	3	1.81	Pass
1	10	4	1.91	Pass
1	10	5	2.01	Pass
1	10	6	2.14	Pass

10 Lowest Seismic

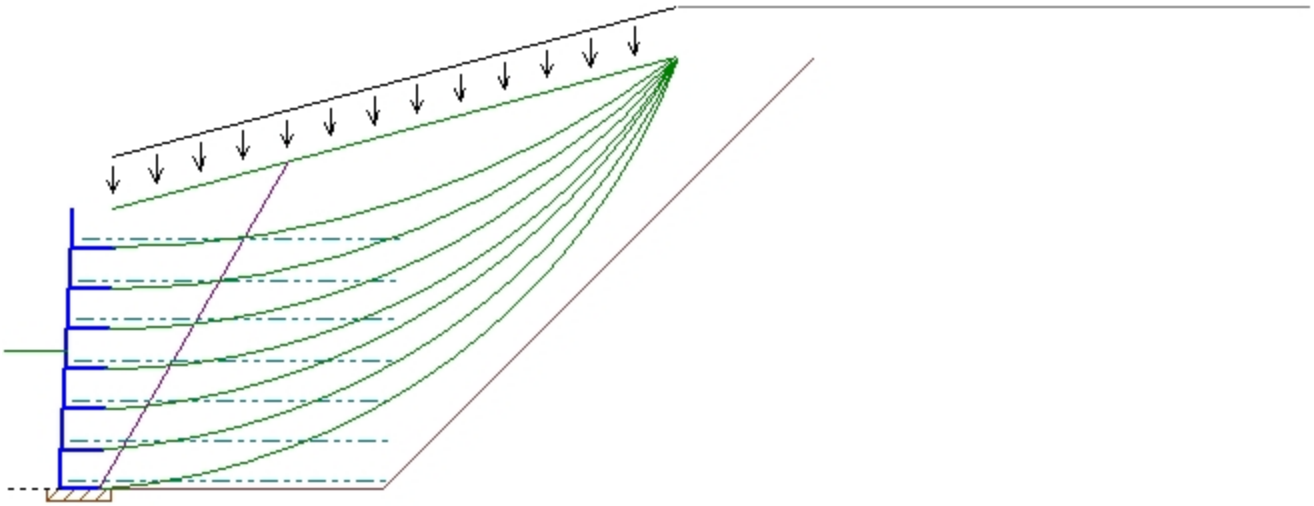
2	10	1	1.53	Pass
2	10	2	1.68	Pass
2	10	3	1.80	Pass
1	10	4	1.88	Pass
1	10	5	1.97	Pass
1	10	6	2.08	Pass
1	10	7	2.16	Pass

Section 3 Details

Section 3 Static ICS Cross-section

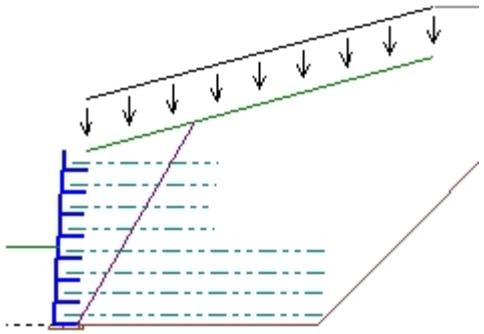


Section 3 Seismic ICS Cross-section



*** Embedment is not included in Bearing Capacity**
*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

Section #4 at Station 26.58
Report Date July 19, 2023
Designer RLC
Design Standard AASHTO 2020 (LRFD) Simplified
Design Static and Seismic
Unit of Measure U.S./Imperial
Selected Facing Unit
Licensor/Product Line: User defined
Name: WWF - 1/2 Length
Seismic As 0.13



Soil Parameters

Soil Zone	Soil Type	Friction Angle φ	In Situ Density γ [lb/ft³]	Cohesion Cf [lb/ft²]
Infill (i)	REINF	30°	125.00	n/a
Retained (r)	RET	25°	125.00	n/a
Foundation (f)	FDN	25°	120.00	0.00
Base (b)	GW	36°	140.00	n/a
Drainage (d)	GP	38°	110.00	n/a

Section Details

Section Height	12.00	Back Slope	14.93°	LL Surcharge	100	DL Surcharge	250
Design Height	12.00 ft	Crest Offset	30.00 ft	LL Offset	0.00 ft	DL Offset	44.50 ft
Embedment	5.36 ft	Wall Batter	3.18°	Toe Slope	0.00°	Toe Offset	0.00 ft

Reinforced Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFCT	Load - Vehicular Collision Force	0.00	1.00
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.65	0.00
TCONN	Resistance - Connection	0.80	0.00
PULLOUT	Resistance - Pullout	0.70	0.00
SLIDING	Resistance - Sliding	1.00	0.00
TAL	Resistance - Tensile	0.80	0.00

Reinforced Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.90	0.00
COMBIMPCO	Resistance - Combined Static/EQ Connection	1.00	0.00
COMBIMPPO	Resistance - Combined Static/EQ Pullout	1.00	0.00
COMBIMPTO	Resistance - Combined Static/EQ Tensile	1.00	0.00
RFconn	Resistance - Seismic Frictional Connection Factor	0.80	0.00
RFpull	Resistance - Seismic Frictional Pullout Factor	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Reinforcements

ERS-5 - ERS-5 **Supplier: User defined, Fill Type: Sands**

Tult	5,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	2,500.81 lb/ft	Cds	0.85	Ci	0.85	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

Analysis Results

*** Embedment is not included in Bearing Capacity**

*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

External Static	CDR			
Bearing Capacity	2.77	Bearing Pressure - Strength	2693.54	lb/ft²
Overturning	5.11	Bearing Pressure - Service	1927.72	lb/ft²
Base Sliding	1.31	Eccentricity for Overturning	0.21	ft
Internal Sliding	1.54	Max Eccentricity	4.50	ft

External Seismic	CDR			
Bearing Capacity	5.41	Bearing Pressure	1895.35	lb/ft²
Overturning	6.27	Eccentricity for Overturning	0.28	ft
Base Sliding	1.58	Max Eccentricity	5.94	ft

Internal Static Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
8	219.50	ERS-5	10.00	9.24	5.54
7	218.00	ERS-5	10.00	12.05	5.14
6	216.50	ERS-5	10.00	12.19	3.94
5	215.00	ERS-5	10.00	12.51	3.19
4	213.50	ERS-5	18.00	12.92	2.69
3	212.00	ERS-5	18.00	13.40	2.32
2	210.50	ERS-5	18.00	13.91	2.04
1	209.00	ERS-5	18.00	29.68	3.74

Internal Seismic Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
8	219.50	ERS-5	10.00	9.03	6.22
7	218.00	ERS-5	10.00	11.90	5.82
6	216.50	ERS-5	10.00	12.43	4.56
5	215.00	ERS-5	10.00	13.02	3.75
4	213.50	ERS-5	18.00	13.65	3.18
3	212.00	ERS-5	18.00	14.30	2.77
2	210.50	ERS-5	18.00	14.96	2.45
1	209.00	ERS-5	18.00	30.44	4.34

Internal Compound Stability

10 Lowest Static

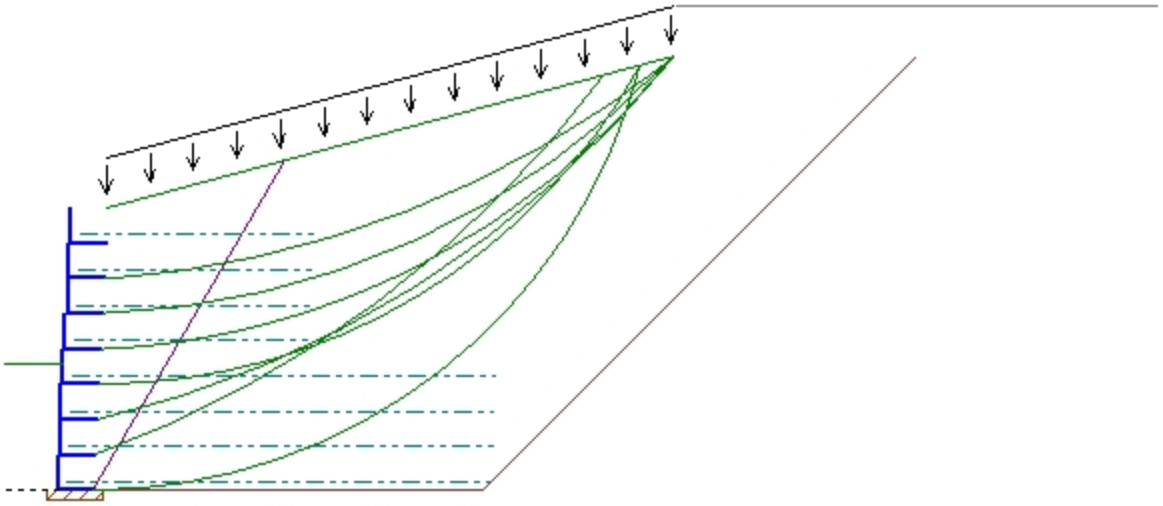
Radius Point	Entry Point	Exit Point	Result	Status
7	10	3	1.59	Pass
1	8	4	1.61	Pass
10	6	2	1.62	Pass
2	10	5	1.69	Pass
2	10	6	1.81	Pass
1	8	1	1.85	Pass
4	10	7	1.97	Pass

10 Lowest Seismic

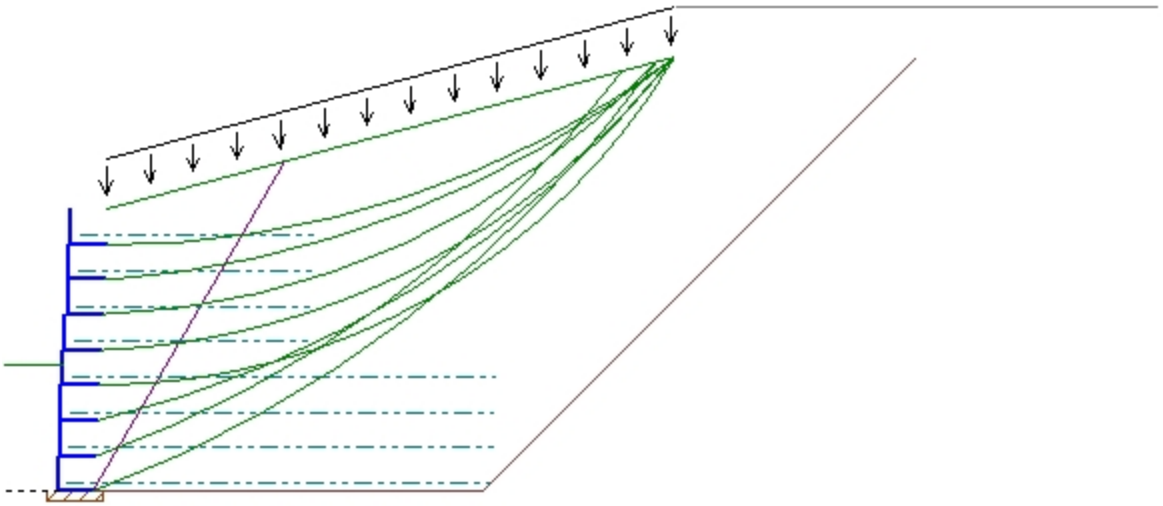
1	10	4	1.51	Pass
7	10	3	1.52	Pass
10	9	1	1.54	Pass
2	10	5	1.57	Pass
10	7	2	1.57	Pass
2	9	6	1.68	Pass
4	10	7	1.84	Pass
1	10	8	2.10	Pass

Section 4 Details

Section 4 Static ICS Cross-section

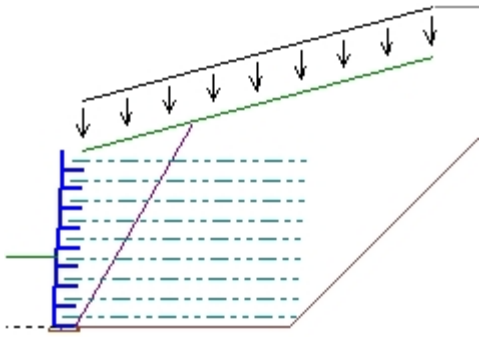


Section 4 Seismic ICS Cross-section



*** Embedment is not included in Bearing Capacity**
*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

Section #5 at Station 36.25
Report Date July 19, 2023
Designer RLC
Design Standard AASHTO 2020 (LRFD) Simplified
Design Static and Seismic
Unit of Measure U.S./Imperial
Selected Facing Unit
Licensor/Product Line: User defined
Name: WWF - 1/2 Length
Seismic As 0.13



Soil Parameters

Soil Zone	Soil Type	Friction Angle φ	In Situ Density γ [lb/ft³]	Cohesion Cf [lb/ft²]
Infill (i)	REINF	30°	125.00	n/a
Retained (r)	RET	25°	125.00	n/a
Foundation (f)	FDN	25°	120.00	0.00
Base (b)	GW	36°	140.00	n/a
Drainage (d)	GP	38°	110.00	n/a

Section Details

Section Height	13.50	Back Slope	14.93°	LL Surcharge	100	DL Surcharge	250
Design Height	13.50 ft	Crest Offset	30.00 ft	LL Offset	0.00 ft	DL Offset	44.50 ft
Embedment	5.30 ft	Wall Batter	3.18°	Toe Slope	0.00°	Toe Offset	0.00 ft

Reinforced Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFCT	Load - Vehicular Collision Force	0.00	1.00
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.65	0.00
TCONN	Resistance - Connection	0.80	0.00
PULLOUT	Resistance - Pullout	0.70	0.00
SLIDING	Resistance - Sliding	1.00	0.00
TAL	Resistance - Tensile	0.80	0.00

Reinforced Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.90	0.00
COMBIMPCO	Resistance - Combined Static/EQ Connection	1.00	0.00
COMBIMPPO	Resistance - Combined Static/EQ Pullout	1.00	0.00
COMBIMPTO	Resistance - Combined Static/EQ Tensile	1.00	0.00
RFconn	Resistance - Seismic Frictional Connection Factor	0.80	0.00
RFpull	Resistance - Seismic Frictional Pullout Factor	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Reinforcements

ERS-5 - ERS-5 **Supplier: User defined, Fill Type: Sands**

Tult	5,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	2,500.81 lb/ft	Cds	0.85	Ci	0.85	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

Analysis Results

*** Embedment is not included in Bearing Capacity**

*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

External Static	CDR			
Bearing Capacity	2.38	Bearing Pressure - Strength	3052.31	lb/ft²
Overturning	4.43	Bearing Pressure - Service	2180.30	lb/ft²
Base Sliding	1.23	Eccentricity for Overturning	0.44	ft
Internal Sliding	1.44	Max Eccentricity	4.50	ft

External Seismic	CDR			
Bearing Capacity	4.58	Bearing Pressure	2170.18	lb/ft²
Overturning	5.37	Eccentricity for Overturning	0.54	ft
Base Sliding	1.48	Max Eccentricity	5.94	ft

Internal Static Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
9	219.50	ERS-5	18.00	8.40	5.30
8	218.00	ERS-5	18.00	11.14	5.00
7	216.50	ERS-5	18.00	11.35	3.86
6	215.00	ERS-5	18.00	11.71	3.14
5	213.50	ERS-5	18.00	12.16	2.65
4	212.00	ERS-5	18.00	12.65	2.29
3	210.50	ERS-5	18.00	13.17	2.01
2	209.00	ERS-5	18.00	13.72	1.80
1	207.50	ERS-5	18.00	29.26	3.33

Internal Seismic Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
9	219.50	ERS-5	18.00	8.16	5.93
8	218.00	ERS-5	18.00	10.91	5.63
7	216.50	ERS-5	18.00	11.49	4.44
6	215.00	ERS-5	18.00	12.11	3.67
5	213.50	ERS-5	18.00	12.76	3.12
4	212.00	ERS-5	18.00	13.43	2.72
3	210.50	ERS-5	18.00	14.11	2.41
2	209.00	ERS-5	18.00	14.79	2.16
1	207.50	ERS-5	18.00	30.09	3.88

Internal Compound Stability

10 Lowest Static

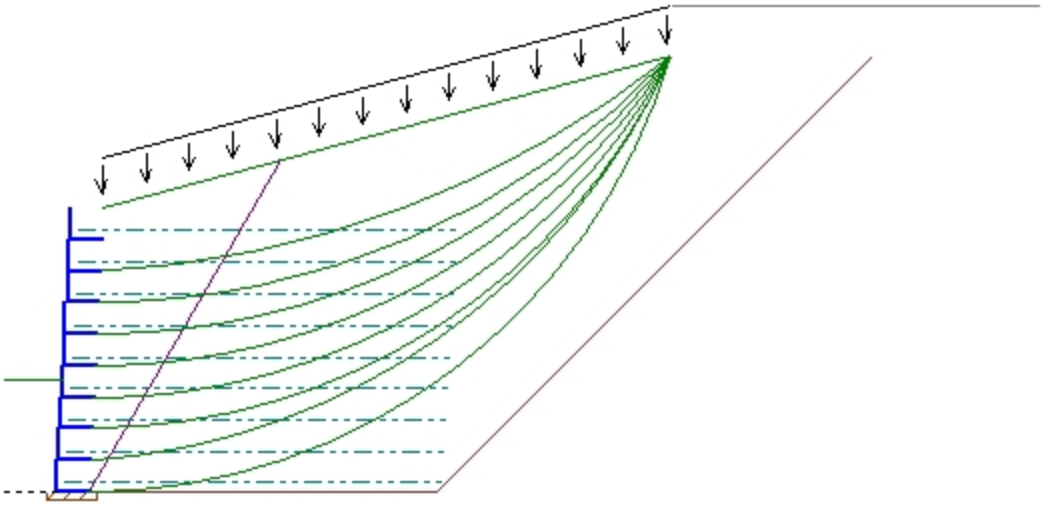
Radius Point	Entry Point	Exit Point	Result	Status
1	10	1	1.72	Pass
2	10	2	1.83	Pass
1	10	3	1.92	Pass
1	10	4	2.03	Pass
1	10	5	2.15	Pass
1	10	6	2.32	Pass
1	10	7	2.50	Pass
3	10	8	2.60	Pass

10 Lowest Seismic

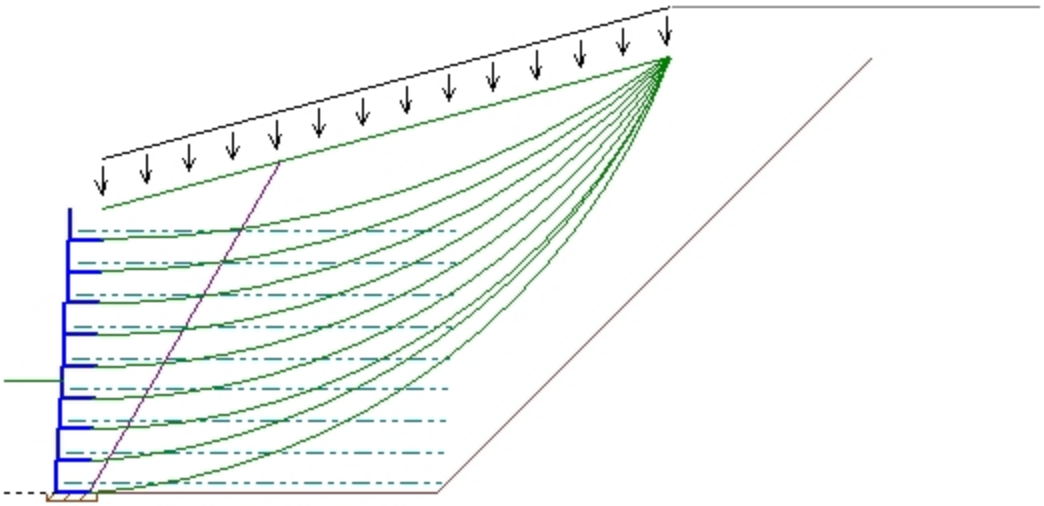
2	10	1	1.66	Pass
2	10	2	1.78	Pass
1	10	3	1.85	Pass
1	10	4	1.95	Pass
1	10	9	2.04	Pass
1	10	5	2.06	Pass
1	10	6	2.22	Pass
1	10	7	2.38	Pass
1	10	8	2.48	Pass

Section 5 Details

Section 5 Static ICS Cross-section

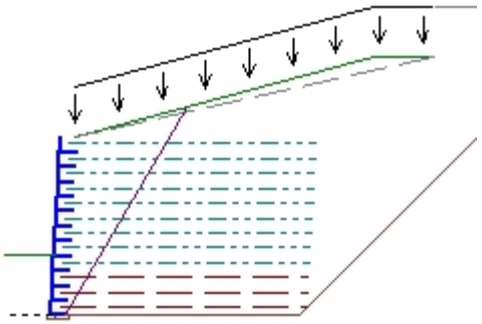


Section 5 Seismic ICS Cross-section



*** Embedment is not included in Bearing Capacity**
*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

Section #8 at Station 62.83
Report Date July 19, 2023
Designer RLC
Design Standard AASHTO 2020 (LRFD) Simplified
Design Static and Seismic
Unit of Measure U.S./Imperial
Selected Facing Unit
Licensor/Product Line: User defined
Name: WWF - 1/2 Length
Seismic As 0.13



Soil Parameters

Soil Zone	Soil Type	Friction Angle φ	In Situ Density γ [lb/ft³]	Cohesion Cf [lb/ft²]
Infill (i)	REINF	30°	125.00	n/a
Retained (r)	RET	25°	125.00	n/a
Foundation (f)	FDN	25°	120.00	0.00
Base (b)	GW	36°	140.00	n/a
Drainage (d)	GP	38°	110.00	n/a

Section Details

Section Height	18.00	Back Slope	14.93°	LL Surcharge	100	DL Surcharge	250
Design Height	18.00 ft	Crest Offset	30.00 ft	LL Offset	0.00 ft	DL Offset	44.50 ft
Embedment	6.15 ft	Wall Batter	3.18°	Toe Slope	0.00°	Toe Offset	0.00 ft

Reinforced Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFCT	Load - Vehicular Collision Force	0.00	1.00
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.65	0.00
TCONN	Resistance - Connection	0.80	0.00
PULLOUT	Resistance - Pullout	0.70	0.00
SLIDING	Resistance - Sliding	1.00	0.00
TAL	Resistance - Tensile	0.80	0.00

Reinforced Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.90	0.00
COMBIMPCO	Resistance - Combined Static/EQ Connection	1.00	0.00
COMBIMPPO	Resistance - Combined Static/EQ Pullout	1.00	0.00
COMBIMPTO	Resistance - Combined Static/EQ Tensile	1.00	0.00
RFconn	Resistance - Seismic Frictional Connection Factor	0.80	0.00
RFpull	Resistance - Seismic Frictional Pullout Factor	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Reinforcements

ERS-8 - ERS-8 **Supplier: User defined, Fill Type: Sands**

Tult	7,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	3,427.04 lb/ft	Cds	0.80	Ci	0.80	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

ERS-5 - ERS-5 **Supplier: User defined, Fill Type: Sands**

Tult	5,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30

LTDS	2,500.81 lb/ft	Cds	0.85	Ci	0.85	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

Analysis Results

*** Embedment is not included in Bearing Capacity**

*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**

*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

External Static	CDR			
Bearing Capacity	2.59	Bearing Pressure - Strength	3971.17	lb/ft²
Overturning	5.00	Bearing Pressure - Service	2855.40	lb/ft²
Base Sliding	1.35	Eccentricity for Overturning	0.38	ft
Internal Sliding	1.42	Max Eccentricity	6.25	ft

External Seismic	CDR			
Bearing Capacity	4.91	Bearing Pressure	2861.85	lb/ft²
Overturning	6.01	Eccentricity for Overturning	0.53	ft
Base Sliding	1.63	Max Eccentricity	8.25	ft

Internal Static Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
12	219.50	ERS-5	25.00	13.78	4.68
11	218.00	ERS-5	25.00	17.72	4.62
10	216.50	ERS-5	25.00	17.42	3.62
9	215.00	ERS-5	25.00	17.45	2.98
8	213.50	ERS-5	25.00	17.65	2.53
7	212.00	ERS-5	25.00	17.97	2.20
6	210.50	ERS-5	25.00	18.35	1.95
5	209.00	ERS-5	25.00	18.79	1.75
4	207.50	ERS-5	25.00	19.26	1.58
3	206.00	ERS-8	25.00	18.59	1.98
2	204.50	ERS-8	25.00	19.08	1.83
1	203.00	ERS-8	25.00	39.88	3.45

Internal Seismic Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
12	219.50	ERS-5	25.00	13.17	5.18
11	218.00	ERS-5	25.00	16.98	5.12
10	216.50	ERS-5	25.00	17.29	4.12
9	215.00	ERS-5	25.00	17.73	3.44
8	213.50	ERS-5	25.00	18.24	2.96
7	212.00	ERS-5	25.00	18.80	2.60
6	210.50	ERS-5	25.00	19.40	2.31
5	209.00	ERS-5	25.00	20.01	2.08
4	207.50	ERS-5	25.00	20.64	1.90
3	206.00	ERS-8	25.00	20.04	2.38
2	204.50	ERS-8	25.00	20.65	2.20
1	203.00	ERS-8	25.00	41.25	4.03

Internal Compound Stability

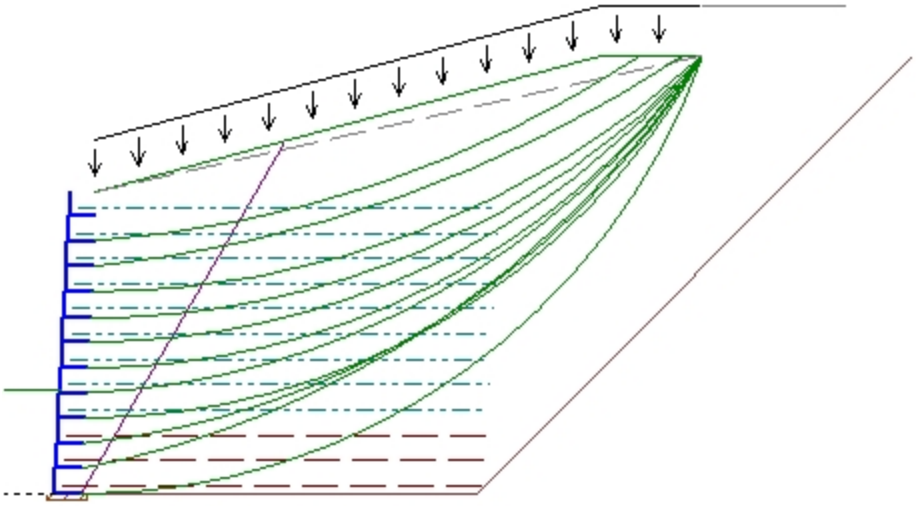
10 Lowest Static

Radius Point	Entry Point	Exit Point	Result	Status
1	10	1	1.78	Pass
5	10	2	1.80	Pass
3	10	3	1.84	Pass
1	10	4	1.93	Pass
2	10	5	2.03	Pass
1	10	6	2.14	Pass
1	10	7	2.25	Pass

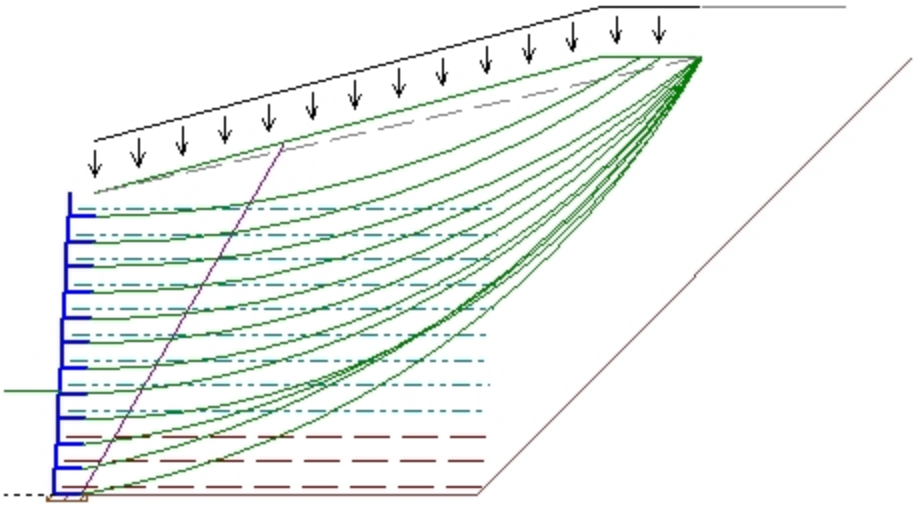
Radius Point	Entry Point	Exit Point	Result	Status
1	10	8	2.39	Pass
5	7	11	2.56	Pass
1	10	9	2.58	Pass
10 Lowest Seismic				
4	10	1	1.66	Pass
5	10	2	1.72	Pass
3	10	3	1.75	Pass
1	10	4	1.82	Pass
2	10	5	1.91	Pass
1	10	6	2.00	Pass
2	7	12	2.00	Pass
1	10	7	2.08	Pass
1	10	8	2.20	Pass
1	8	11	2.32	Pass

Section 8 Details

Section 8 Static ICS Cross-section

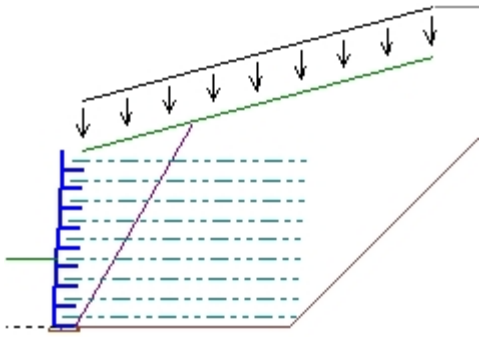


Section 8 Seismic ICS Cross-section



*** Embedment is not included in Bearing Capacity**
*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

Section #11 at Station 89.42
Report Date July 19, 2023
Designer RLC
Design Standard AASHTO 2020 (LRFD) Simplified
Design Static and Seismic
Unit of Measure U.S./Imperial
Selected Facing Unit
Licensor/Product Line: User defined
Name: WWF - 1/2 Length
Seismic As 0.13



Soil Parameters

Soil Zone	Soil Type	Friction Angle φ	In Situ Density γ [lb/ft³]	Cohesion Cf [lb/ft²]
Infill (i)	REINF	30°	125.00	n/a
Retained (r)	RET	25°	125.00	n/a
Foundation (f)	FDN	25°	120.00	0.00
Base (b)	GW	36°	140.00	n/a
Drainage (d)	GP	38°	110.00	n/a

Section Details

Section Height	13.50	Back Slope	14.93°	LL Surcharge	100	DL Surcharge	250
Design Height	13.50 ft	Crest Offset	30.00 ft	LL Offset	0.00 ft	DL Offset	44.50 ft
Embedment	5.24 ft	Wall Batter	3.18°	Toe Slope	0.00°	Toe Offset	0.00 ft

Reinforced Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFCT	Load - Vehicular Collision Force	0.00	1.00
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.65	0.00
TCONN	Resistance - Connection	0.80	0.00
PULLOUT	Resistance - Pullout	0.70	0.00
SLIDING	Resistance - Sliding	1.00	0.00
TAL	Resistance - Tensile	0.80	0.00

Reinforced Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.90	0.00
COMBIMPCO	Resistance - Combined Static/EQ Connection	1.00	0.00
COMBIMPPO	Resistance - Combined Static/EQ Pullout	1.00	0.00
COMBIMPTO	Resistance - Combined Static/EQ Tensile	1.00	0.00
RFconn	Resistance - Seismic Frictional Connection Factor	0.80	0.00
RFpull	Resistance - Seismic Frictional Pullout Factor	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Reinforcements

ERS-5 - ERS-5 **Supplier: User defined, Fill Type: Sands**

Tult	5,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	2,500.81 lb/ft	Cds	0.85	Ci	0.85	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

Analysis Results

*** Embedment is not included in Bearing Capacity**

*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

External Static	CDR			
Bearing Capacity	2.38	Bearing Pressure - Strength	3052.31	lb/ft²
Overturning	4.43	Bearing Pressure - Service	2180.30	lb/ft²
Base Sliding	1.23	Eccentricity for Overturning	0.44	ft
Internal Sliding	1.44	Max Eccentricity	4.50	ft

External Seismic	CDR			
Bearing Capacity	4.58	Bearing Pressure	2170.18	lb/ft²
Overturning	5.37	Eccentricity for Overturning	0.54	ft
Base Sliding	1.48	Max Eccentricity	5.94	ft

Internal Static Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
9	219.50	ERS-5	18.00	8.40	5.30
8	218.00	ERS-5	18.00	11.14	5.00
7	216.50	ERS-5	18.00	11.35	3.86
6	215.00	ERS-5	18.00	11.71	3.14
5	213.50	ERS-5	18.00	12.16	2.65
4	212.00	ERS-5	18.00	12.65	2.29
3	210.50	ERS-5	18.00	13.17	2.01
2	209.00	ERS-5	18.00	13.72	1.80
1	207.50	ERS-5	18.00	29.26	3.33

Internal Seismic Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
9	219.50	ERS-5	18.00	8.16	5.93
8	218.00	ERS-5	18.00	10.91	5.63
7	216.50	ERS-5	18.00	11.49	4.44
6	215.00	ERS-5	18.00	12.11	3.67
5	213.50	ERS-5	18.00	12.76	3.12
4	212.00	ERS-5	18.00	13.43	2.72
3	210.50	ERS-5	18.00	14.11	2.41
2	209.00	ERS-5	18.00	14.79	2.16
1	207.50	ERS-5	18.00	30.09	3.88

Internal Compound Stability

10 Lowest Static

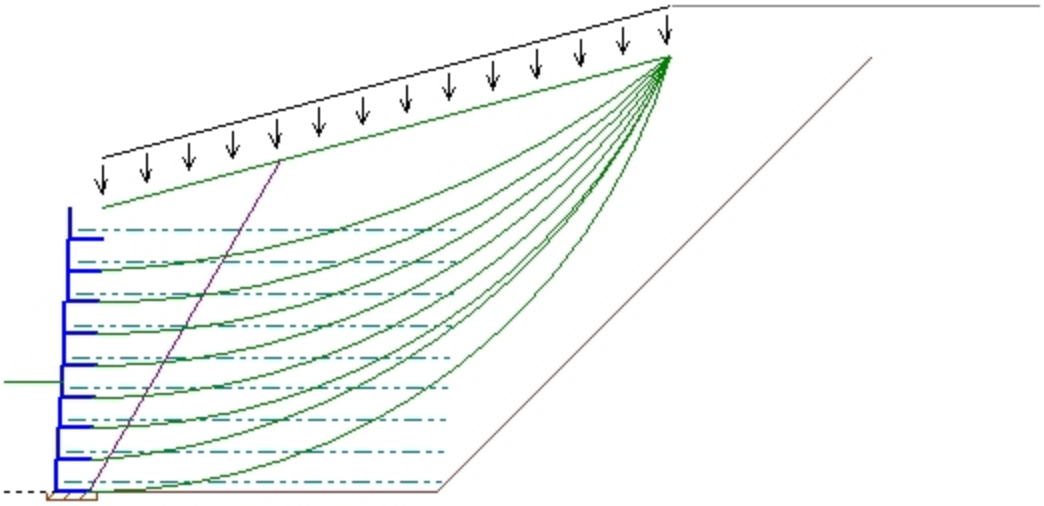
Radius Point	Entry Point	Exit Point	Result	Status
1	10	1	1.72	Pass
2	10	2	1.83	Pass
1	10	3	1.92	Pass
1	10	4	2.03	Pass
1	10	5	2.15	Pass
1	10	6	2.32	Pass
1	10	7	2.50	Pass
3	10	8	2.60	Pass

10 Lowest Seismic

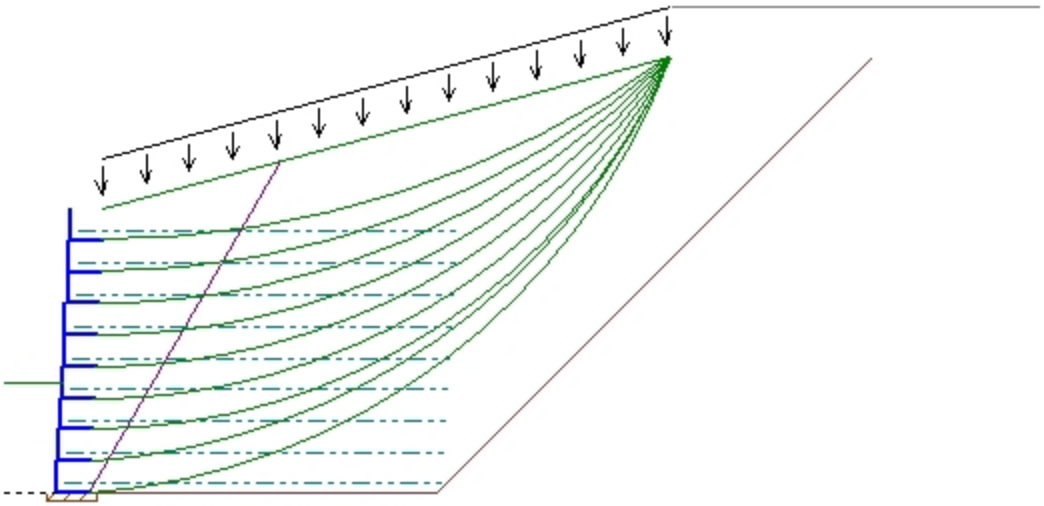
2	10	1	1.66	Pass
2	10	2	1.78	Pass
1	10	3	1.85	Pass
1	10	4	1.95	Pass
1	10	9	2.04	Pass
1	10	5	2.06	Pass
1	10	6	2.22	Pass
1	10	7	2.38	Pass
1	10	8	2.48	Pass

Section 11 Details

Section 11 Static ICS Cross-section

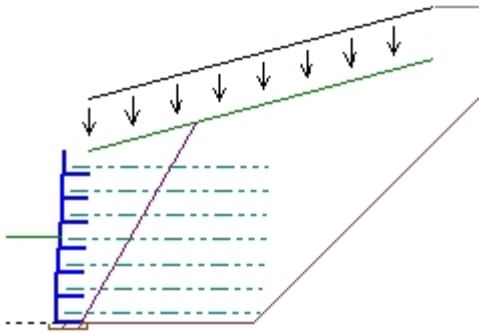


Section 11 Seismic ICS Cross-section



*** Embedment is not included in Bearing Capacity**
*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

Section #13 at Station 99.08
Report Date July 19, 2023
Designer RLC
Design Standard AASHTO 2020 (LRFD) Simplified
Design Static and Seismic
Unit of Measure U.S./Imperial
Selected Facing Unit
Licensor/Product Line: User defined
Name: WWF - 1/2 Length
Seismic As 0.13



Soil Parameters

Soil Zone	Soil Type	Friction Angle φ	In Situ Density γ [lb/ft³]	Cohesion Cf [lb/ft²]
Infill (i)	REINF	30°	125.00	n/a
Retained (r)	RET	25°	125.00	n/a
Foundation (f)	FDN	25°	120.00	0.00
Base (b)	GW	36°	140.00	n/a
Drainage (d)	GP	38°	110.00	n/a

Section Details

Section Height	10.50	Back Slope	14.93°	LL Surcharge	100	DL Surcharge	250
Design Height	10.50 ft	Crest Offset	30.00 ft	LL Offset	0.00 ft	DL Offset	44.50 ft
Embedment	5.23 ft	Wall Batter	3.18°	Toe Slope	0.00°	Toe Offset	0.00 ft

Reinforced Load and Resistance Factors - Static

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFDC	Load - Dead Load (Structure)	0.90	1.25
LFES	Load - Earth Surcharge Load	0.75	1.50
LFEH	Load - Horz. Pressure of earth fill	0.90	1.50
LFCT	Load - Vehicular Collision Force	0.00	1.00
LFLL	Load - Vehicular Live Load	0.00	1.75
LFEV	Load - Vert. Pressure of earth fill	1.00	1.35
BEARING	Resistance - Bearing	0.65	0.00
TCONN	Resistance - Connection	0.80	0.00
PULLOUT	Resistance - Pullout	0.70	0.00
SLIDING	Resistance - Sliding	1.00	0.00
TAL	Resistance - Tensile	0.80	0.00

Reinforced Load and Resistance Factors - Seismic

Term	Description	Minimum (as appl.)	Maximum (as appl.)
LFEQ	Load - Earthquake	0.00	1.00
LFEQLL	Load - Live Load Seismic Factor	0.00	0.50
BEARING	Resistance - Bearing	0.90	0.00
COMBIMPCO	Resistance - Combined Static/EQ Connection	1.00	0.00
COMBIMPPO	Resistance - Combined Static/EQ Pullout	1.00	0.00
COMBIMPTO	Resistance - Combined Static/EQ Tensile	1.00	0.00
RFconn	Resistance - Seismic Frictional Connection Factor	0.80	0.00
RFpull	Resistance - Seismic Frictional Pullout Factor	0.80	0.00
SLIDING	Resistance - Sliding	1.00	0.00

Reinforcements

ERS-5 - ERS-5 **Supplier: User defined, Fill Type: Sands**

Tult	5,400.00 lb/ft	RFcr	1.51	RFid	1.10	RFd	1.30
LTDS	2,500.81 lb/ft	Cds	0.85	Ci	0.85	α Correction	0.80

Connection/Shear Properties

αcs1	0.00 lb/ft	IP-1	0.00 lb/ft	αcs2	0.00 lb/ft	IP-2	0.00 lb/ft
αcs max	0.00 lb/ft	au	800.00 lb/ft	λu	30.00 lb/ft	Vu(max)	2,000.00 lb/ft
TLot Reduct.	1.00	cn RFcr	1.51	cn RFd	1.15		

Analysis Results

*** Embedment is not included in Bearing Capacity**

*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**

*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

External Static	CDR			
Bearing Capacity	1.86	Bearing Pressure - Strength	2471.87	lb/ft²
Overturning	3.52	Bearing Pressure - Service	1745.33	lb/ft²
Base Sliding	1.09	Eccentricity for Overturning	0.59	ft
Internal Sliding	1.38	Max Eccentricity	3.00	ft

External Seismic	CDR			
Bearing Capacity	3.63	Bearing Pressure	1731.91	lb/ft²
Overturning	4.32	Eccentricity for Overturning	0.65	ft
Base Sliding	1.33	Max Eccentricity	3.96	ft

Internal Static Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
7	219.50	ERS-5	12.00	4.05	5.81
6	218.00	ERS-5	12.00	5.84	5.29
5	216.50	ERS-5	12.00	6.37	4.03
4	215.00	ERS-5	12.00	6.94	3.25
3	213.50	ERS-5	12.00	7.53	2.73
2	212.00	ERS-5	12.00	8.12	2.35
1	210.50	ERS-5	12.00	18.00	4.25

Internal Seismic Layer	Elevation [ft]	Rein.	Length [ft]	Pullout CDR	Tensile Overstress CDR
7	219.50	ERS-5	12.00	3.99	6.56
6	218.00	ERS-5	12.00	5.82	6.03
5	216.50	ERS-5	12.00	6.55	4.69
4	215.00	ERS-5	12.00	7.27	3.83
3	213.50	ERS-5	12.00	7.99	3.24
2	212.00	ERS-5	12.00	8.71	2.81
1	210.50	ERS-5	12.00	18.39	4.93

Internal Compound Stability

10 Lowest Static

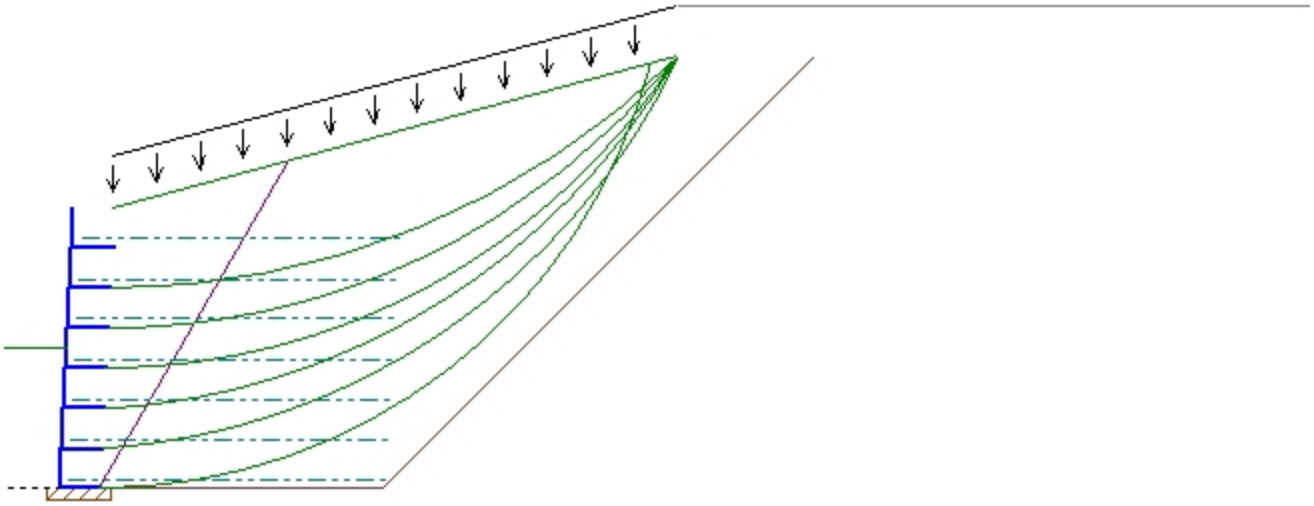
Radius Point	Entry Point	Exit Point	Result	Status
1	9	1	1.55	Pass
2	10	2	1.69	Pass
2	10	3	1.81	Pass
1	10	4	1.91	Pass
1	10	5	2.01	Pass
1	10	6	2.14	Pass

10 Lowest Seismic

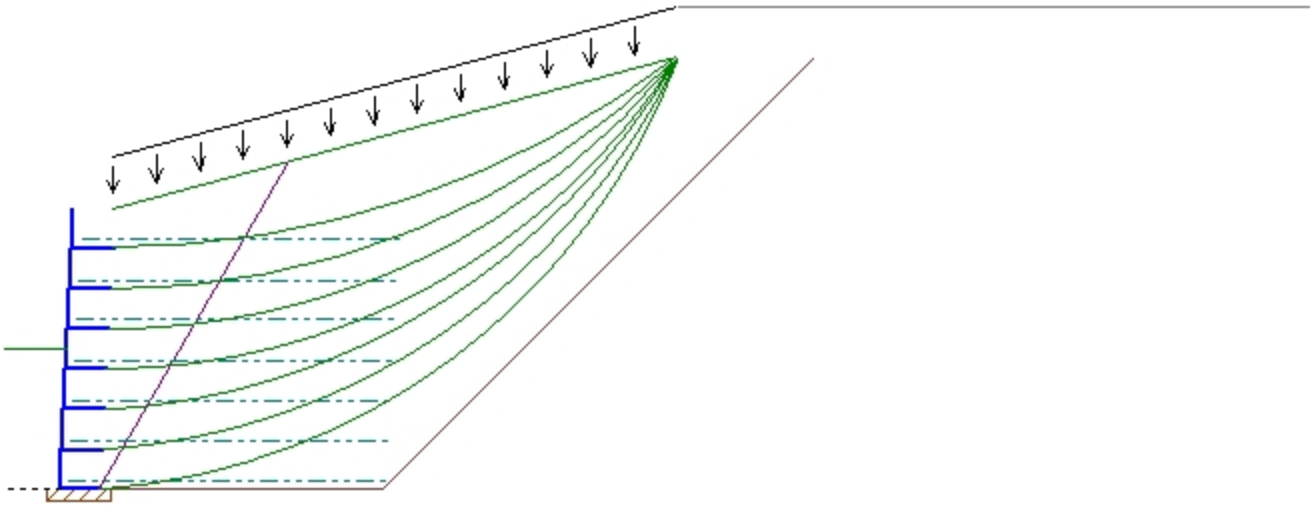
2	10	1	1.53	Pass
2	10	2	1.68	Pass
2	10	3	1.80	Pass
1	10	4	1.88	Pass
1	10	5	1.97	Pass
1	10	6	2.08	Pass
1	10	7	2.16	Pass

Section 13 Details

Section 13 Static ICS Cross-section



Section 13 Seismic ICS Cross-section



*** Embedment is not included in Bearing Capacity**
*** Analysis uses Vertical Earth Pressure Factor, EV, for internal tension**
*** Analysis uses Auto-run Trial Wedge method for Seismic when ϕ – aTan(kh) – β < 0**

LABORATORY ANALYSIS

After the soil samples are visually classified, specific samples are selected by the geotechnical engineer for laboratory analysis. The laboratory analysis may consisted of moisture profiles of the natural moisture content, #200 washes, and Atterberg limits tests determinations. The results of the laboratory analysis are presented on the boring logs in the Appendix of this report. A brief description of the laboratory tests performed is provided in the following sections. Tests follow the general procedures of ASTM.

DESCRIPTION OF SOILS (VISUAL-MANUAL PROCEDURE) (ASTM D 2488)

The soil samples were visually examined by our engineer and soil descriptions were provided. Representative samples were then selected and tested in accordance with the aforementioned laboratory-testing program to determine soil classifications and engineering properties. This data was used to correlate our visual descriptions with the Unified Soil Classification System (USCS).

NATURAL MOISTURE CONTENT (ASTM D 2216)

Natural moisture content (M%) was determined on selected samples. The natural moisture content is the ratio, expressed as a percentage, of the weight of water in a given amount of soil to the weight of solid particles. The results are indicated on the boring logs and table in the Appendix of this report.

ATTERBERG LIMITS (ASTM D-4318)

The Atterberg Limits test was performed to evaluate the soil's plasticity characteristics. The soil Plasticity Index (PI) is representative of this characteristic and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). The Liquid Limit is the moisture content at which the soil will flow as a heavy viscous fluid. The Plastic Limit is the moisture content at which the soil is between "plastic" and the semi-solid stage. The Plasticity Index (PI = LL - PL) is a frequently used indicator for a soil's potential for volume change. Typically, a soil's potential for volume change increases with higher plasticity indices. The results of Atterberg limit testing are presented on the boring logs and summary table in the Appendix of this report.

WASH #200 TEST (ASTM D 422)

Grain-size tests were performed to determine the partial soil particle size distribution. The amount of material finer than the openings on the No. 200 sieve (0.074 mm) was determined by washing soil over the No. 200 sieve. The results of wash #200 tests are presented on the boring logs included in the Appendix of this report.

POCKET PENETROMETER

Pocket penetrometer tests were performed on cohesive soil samples. The pocket penetrometer provides a consistency classification and an estimate of the soil's unconfined compressive strength. The pocket penetrometer data are presented as Qp in units of tons per square foot (tsf) on the logs in the Appendix.

STANDARD TEST METHOD FOR FIELD VANE SHEAR TEST ASTM D4767 – 04 ASTM D2573 - 08

The vane shear test is an in-situ geotechnical testing methods used to estimate the undrained shear strength of fully saturated clays without disturbance. The test is relatively simple, quick, and provides a cost effective way of estimating the soil shear strength; therefore, it is widely used in geotechnical investigations. Under special condition, the vane shear test can be also carried out in the laboratory on undisturbed soil specimens; however, the use of the vane shear test in in-situ testing is much more common.

IMPORTANT INFORMATION ABOUT YOUR

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Speciltc Factors

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engi- neers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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