APPENDIX C

GEOTECHNICAL ENGINEERING STUDY

GEOTECHNICAL ENGINEERING STUDY
FOR
BANNER SELF-STORAGE FACILITY
Corporate Way (APN 03100510190000)
Sacramento, California

Project No. E23314.000
November 2023

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ESTABLISHED 1984

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Project No. E23314.000
10 November 2023

Attention: Mr. Chris Tramonte

Subject: **BANNER SELF-STORAGE FACILITY**
Corporate Way (APN 03100510190000), Sacramento, California 95831
GEOTECHNICAL ENGINEERING STUDY

References: See page ii.

Dear Mr. Tramonte:

In accordance with your authorization, Youngdahl Consulting Group, Inc. has prepared this geotechnical engineering study for the project site located along Corporate Way at Sacramento Assessor's Parcel Number (APN) 03100510190000 in Sacramento, California. The purpose of this study was to prepare a site-specific geotechnical report that can be incorporated into design and construction of the proposed site. To complete this task, our firm completed a subsurface exploration, reviewed the referenced documents, and prepared this report in accordance with the Reference 3 services.

Based upon our observations, the subsurface conditions at the project site are prone to static settlements. These conditions are common in this region of Sacramento and efforts to accommodate for these conditions into the design and construction of a project are generally relative to the cost of the improvements while maintaining the requirements for life-safety. The risk of total settlement, differential settlement, and lateral spreading cannot be fully eliminated without remediation of the entire liquefiable soil column, which extends to depths on the order of 37 feet below the ground surface. For the purposes of this report, several mitigation methods and foundation systems are included in this report to aid in project planning. We should be contacted to provide additional recommendations for these types of mitigation methods and foundation systems should they be selected for the project.

Due to the non-uniform nature of soils, other geotechnical issues may become more apparent during grading operations which are not listed above. The descriptions, findings, conclusions, and recommendations provided in this report are formulated as a whole; specific conclusions or recommendations should not be derived or used out of context. Please review the limitations and uniformity of conditions section of this report.

This report has been prepared for the exclusive use of the addressee of this report and their consultants, for specific application to this project, in accordance with generally accepted geotechnical engineering practice. Should you have any questions or require additional information, please contact our office at your convenience.

Very truly yours,
Youngdahl Consulting Group, Inc.

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References:

1. Request for Proposal for Geotechnical Services, prepared by Banner Storage Group, LLC, dated 3 August 2023.
2. Boundary & Existing Conditions Plan for Corporate Way Self-Storage, prepared by TSD Engineering, Inc., dated 25 August 2023
3. Proposal Geotechnical Engineering Study for Banner Self-Storage Facility, prepared by Youngdahl Consulting Group, Inc., dated 15 August 2023 (Proposal No. PE23-436).

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GEOTECHNICAL ENGINEERING STUDY
FOR
BANNER SELF-STORAGE FACILITY

1.0 INTRODUCTION

This report presents the results of our geotechnical engineering study performed for the proposed improvements planned to be constructed along Corporate Way at Sacramento Assessor's Parcel Number (APN) 03100510190000 in Sacramento, California. The vicinity map provided on Figure A-1, Appendix A, shows the approximate project location.

Background

Based on a cursory review of historic aerial imagery, the project site consisted of agricultural land as early as 1947. Between 1984, Corporate Way and adjacent buildings to the south were constructed. Between August 1998 and May 2002, a parking lot and pre-school were constructed adjacent to the southeast and northeast of the site, respectively. Minor fills appear to have been placed along the northwest perimeter of the site. The site appears to have remained relatively unchanged since.

If studies or plans pertaining to the site exist and are not cited as a reference in this report, we should be afforded the opportunity to review and modify our conclusions and recommendations as necessary.

Project Understanding

We understand that proposed development will consist of the construction of a self-storage facility near Park City Drive and Corporate Way in Sacramento, California. We understand that this project is planned to consist of a three-story, ground-up, fully climate-controlled, self-storage facility with an office, trash enclosure, transformer, loading bay, and elevators which will have a base floor plate of 70,000 square feet. Appurtenant developments include a perimeter driveway, small parking lot, concrete hardscaping, and landscaping. The building is anticipated to be supported by shallow foundations and have concrete slab-on-grade floors.

Purpose and Scope

Youngdahl Consulting Group, Inc. has prepared this report to provide geotechnical engineering recommendations and considerations for incorporation into the design and development of the site. The following scope of services were developed and performed for preparation of this report:

- A review of geotechnical and geologic data available to us at the time of our study;
- Performance of a field study consisting of a site reconnaissance and subsurface explorations to observe and characterize the subsurface conditions;
- Evaluation of the data and information obtained from the field study, laboratory testing, and literature review for geotechnical conditions;
- Development of the following geotechnical recommendations and considerations regarding earthwork construction including, site preparation, engineered fill criteria, seasonal moisture conditions, excavation characteristics, and drainage;
- Development of geotechnical design criteria for code-based seismicity, shallow or mat conventional foundations with ground improvements, deep foundations, slabs on grade, and retaining walls;
- Preparation of this report summarizing our findings, conclusions, and recommendations regarding the above-described information.

2.0 SITE CONDITIONS

The following section describes our findings regarding the site conditions that we observed during our site reconnaissance and subsequent subsurface explorations.

Surface Observations

The project site is currently a grass-covered undeveloped lot. The site is bounded by Corporate Way to the northeast, a preschool to the northwest, and parking lots to the southeast and southwest. Topography at the site is relatively flat with slight undulations. Minor fill piles, likely from adjacent developments, are located along the northwest and northeast perimeters. At the time of our site visit on 8 September 2023, vegetation at the site consisted of short recently-mowed grasses and trees along the northwest, southwest, and southeast perimeters. No standing water was observed.

Subsurface Conditions

Our recent field study included a site reconnaissance by a representative of our firm and a subsurface exploration program. The exploration program included the advancement of four cone penetration test (CPT) soundings to refusal which occurred at depths of approximately 35 to 39 feet below ground surface (bgs) and two exploratory borings to depths of 25 to 50 feet bgs. The approximate locations of the soundings and borings are presented on Figure A-2, Appendix A.

The subsurface soils encountered in the CPT soundings and borings generally consisted of stiff to very stiff fine sandy clays and silts in the upper 2 to 7 feet. Groundwater was encountered at 6 ½ to 7 feet bgs. Then the subsurface soil became saturated and generally consisting of very soft to soft clays until 30 to 32 feet bgs where sandy silt to clayey sand was encountered. We encountered a rapid resistance increase at approximately 34 to 37 feet bgs within gravelly sand in a very dense condition with various amounts of sand and cobble to the maximum depth of exploration.

A more detailed description of the subsurface conditions encountered during our subsurface exploration is presented graphically in Appendix A.

Groundwater Conditions

At the time of our investigation, groundwater was encountered at the project site at an approximate depth of 6½ to 7 feet below current ground surface, based on wet cuttings on the auger observed during sampling. Historic groundwater from the Department of Water Resources website suggests that groundwater in the area may fluctuate between 4 and 20 feet bgs. Fluctuations in the level of groundwater may occur due to variations in rainfall, water levels of the nearby Sacramento River, and other factors not evident at the time measurements were made.

3.0 GEOTECHNICAL SOIL CHARACTERISTICS

Laboratory Testing

Laboratory testing of the collected samples was directed towards evaluating the physical and engineering properties of the soils underlying the site. The associated test results are presented in Appendix B. In summary, the following tests were performed for the preparation of this report:

Table 1: Laboratory Tests

Laboratory Test	Test Standard	Summary of Results	
Direct Shear	ASTM D3080	B-2 @ 0-5'	Φ = 31.7°, c = 174 psf (90% RC)
Maximum Dry Density	ASTM D1557	B-2 @ 0-5'	DD =101.3 pcf, MC = 17.8 %
Resistance Value	CTM 301	B-2 @ 0-5'	R-Value = 27
Unconfined Compressive Strength	ASTM D2166	B-1 @ 26-26.5' B-1 @ 51-51.5'	Compression Strength = 1137.5 psf Compression Strength = 6185.5 psf
Expansion Index	ASTM D4829	B-1 @ 0-5'	EI = 108 (High)
Atterberg Limits	ASTM D4318	B-1 @ 10.5-11' B-1 @ 30.5-31'	LL = 43, PI = 20 (CL) LL = 28, PI = 8 (SC)
Particle Size Distribution (Sieve)	ASTM D6913	B-2 @ 0-5' B-1 @ 40.5-41'	2% > No. 4, 81.3% < No. 200 (CH) 57% > No. 4, 5.5% < No. 200 (GW)
Finer Than No. 200	ASTM D1140	B-1 @ 20.5-21' B-1 @ 50.5-51'	72.5% < No. 200 88.0% < No. 200
Moisture Content & Dry Density	ASTM D2216 & D7263	B-1 @ 11-11.5' B-2 @ 16-16.5' B-1 @ 20.5-21' B-1 @ 21-21.5' B-2 @ 26-26.5' B-1 @ 31-31.5'	DD = 83.2 pcf, MC = 36.7% DD = 90.8 pcf, MC = 34.1% DD = 94.4 pcf, MC = 30.1% DD = 90.9 pcf, MC = 32.5% DD = 93.9 pcf, MC = 29.3% DD = 92.0 pcf, MC = 31.7%
Corrosivity Suite	CA DOT Tests 417, 422 and 643	See Soil Corrosivity Section	

Soil Expansion Potential
The plastic materials encountered within our explorations generally consisted of clay of moderate to high plasticity. Based upon the expansion index test results, and per Section 1803.5.3 2022 CBC, the clay encountered at the site is considered to have a high expansion potential. An expansion index test was conducted in the upper 5 feet which reflected this condition.

Questionable Soil Conditions
The Atterberg limit testing we performed returned high liquid limits and the moisture content testing yielded liquid limits near but still below the liquid limits. Clays with moisture contents near and sometimes above the liquid limit can behave like liquids. Where on-site clays are in this condition and further exacerbated by low blow counts and strengths, the site and building could be subject to excessive settlement when loaded.

Soil Corrosivity
A corrosivity testing suite consisting of soil pH, resistivity, sulfate, and chloride content tests were performed on selected soil samples collected during our site exploration. We are not corrosion specialists and recommend that the results be evaluated by a qualified corrosion expert. The laboratory test results (provided by Sunland Analytical, Inc.) are provided in Appendix B and are summarized in Table 2, below.

Table 2: Corrosivity Summary

Location	Depth (ft)	Soil pH	Minimum Resistivity ohm-cm (x1000)	Chloride (ppm)	Sulfate (ppm)	Caltrans Environment	ACI Environment
B-1	10-10.5	7.92	1.07	48.2	27.4	Non-Corrosive	S0 (Not a Concern)
B-2	3-3.5	7.36	1.69	14.8	13.8	Non-Corrosive	S0 (Not a Concern)

According to Caltrans Corrosion Guidelines Version 3.2, March 2021, the test results do not appear to indicate a potentially corrosive environment for steel used in mechanically stabilized earth elements and structural elements.

According to the 2022 California Building Code Section 1904.1 and ACI 318-14 Table 19.3.1.1, the test results indicate the onsite soils have a negligible potential for sulfide attack of concrete.

A certified corrosion engineer should be consulted to review the above tests and site conditions in order to develop specific mitigation recommendations if metallic pipes or structural elements are designed to be in contact with or buried in soil.

4.0 GEOLOGY AND SEISMICITY

The geologic portion of this report includes a review of geologic data pertinent to the site based on an interpretation of our observations of the surface exposures and our observations in our exploratory borings and CPT soundings.

Geologic Conditions

The site is located within the Sacramento Valley. According to the Generalized Geologic Map of Sacramento County (OFR 99-09) the project site is underlain by undivided alluvial deposits of the Holocene (Qha). The geologic map appears to suggest that the lower unit of the Riverbank Formation (Qrl) underlies the alluvial deposits. The lower Riverbank Formation generally consists of interbedded clays, sand, gravel, and cobble. The mapped geologic units correlate well with the logs of the subsurface conditions completed for this study.

Seismicity

Our evaluation of seismicity for the project site included reviewing existing fault maps and obtaining seismic design parameters from the USGS online calculators and databases. For the purpose of this study, we used a latitude and longitude of 38.492044, -121.517293 to identify the project site.

<u>Alquist-Priolo Regulatory Faults</u>

Based upon the records currently available from the California Department of Conservation, the project site is not located within an Alquist-Priolo Regulatory Review Zone and there are no known faults located at the subject site. We do not anticipate special design or construction requirements for faulting at this project site.

<u>Code Based Seismic Criteria</u>

The site should be classified as Site Class F. For these conditions, the building code assumes that the project site would be developed using site-specific design criteria based on the methodologies described in ASCE 7-16, Chapter 21 unless the structural engineer can apply for the exceptions listed in ASCE 7-16 Section 11.4.8.e2 and Section 20.3.1.e1. For the purpose of preparing the following table, our firm has assumed that these exceptions apply to this project. As such, the value of F_v was calculated using CBC Table 1613.2.3(2) since an evaluation of the site-specific ground motion response was not performed in accordance with ASCE 7-16 Chapter 21 and the design parameters were evaluated using Site Class D based on the seismic shear wave velocity from the CPT soundings. The structural engineer should review the conditions of the exception and the final choice of design parameters remains the purview of the project structural engineer.

Table 3: Seismic Design Parameters*

Reference		Seismic Parameter	Recommended Value
ASCE 7-16	Table 20.3-1	Site Class	F
		Site Class (Degraded for Exceptions)	D
	Figure 22-7	Maximum Considered Earthquake Geometric Mean (MCEC) PGA	0.255g
	Table 11.8-1	Site Coefficient F_{PGA}	1.345
	Equation 11.8-1	$PGA_M = F_{PGA}$ PGA	0.343g
2022 CBC	Figure 1613.2.1(1)	Short-Period MCE at 0.2s, S_S	0.611g
	Figure 1613.2.1(3)	1.0s Period MCE, S_1	0.263g
	Table 1613.2.3(1)	Site Coefficient, F_a	1.312
	Table 1613.2.3(2)	Site Coefficient, F_v	2.074
	Equation 16-20	Adjusted MCE Spectral Response Parameters, $S_{MS} = F_aS_s$	0.801g
	Equation 16-21	Adjusted MCE Spectral Response Parameters, $S_{M1} = F_vS_1$	0.545g
	Equation 16-22	Design Spectral Acceleration Parameters, $S_{DS} = ⅔S_{MS}$	0.534g
	Equation 16-23	Design Spectral Acceleration Parameters, $S_{D1} = ⅔S_{M1}$	0.364g
	Section 1613.2.5(1)	Seismic Design Category (Short Period), Occupancy I to III	D
	Section 1613.2.5(1)	Seismic Design Category (Short Period), Occupancy IV	D
	Section 1613.2.5(2)	Seismic Design Category (1-Sec Period), Occupancy I to IV	D

*Based on the online calculator available at https://earthquake.usgs.gov/ws/designmaps/

USGS Deaggregation

An evaluation of the design moment magnitude was evaluated using the online USGS deaggregation tool. Based on the results of the evaluation, the mean moment magnitude for the project site is 6.52 and this value was used in the evaluations presented by this report.

Earthquake Induced Liquefaction, Settlement, and Surface Rupture Potential

Liquefaction is the sudden loss of soil shear strength and sudden increase in porewater pressure caused by shear strains, as could result from an earthquake. Research has shown that saturated, loose to medium-dense sands with a silt content less than about 25 percent and located within the top 40 feet are most susceptible to liquefaction and surface rupture/lateral spreading. Typically, recent alluvial deposits such as those present on site are more susceptible to liquefaction. Lateral displacement was not considered since the site is relatively flat.

Earthquake induced settlement associated with liquefaction could be separated into free-field settlements (i.e., settlement of the ground surface), building settlement (i.e., settlement of the building relative to the ground surface), and ejecta (e.g., sand boils). The total settlement of the buildings is the combination of the free-field settlement, liquefaction induced building settlement, and ejecta.

Free-Field Settlement

An analysis of the liquefaction potential for these layers was performed using the computer-based program CLiq v.2.3.1.15 developed by Geologismiki, Inc. The CPT analysis was performed using the methods presented by Boulanger & Idriss (2014) and Robertson (NCEER 2001). We used a design earthquake moment magnitude of 6.52 and a peak ground acceleration of 0.255 based on the USGS deaggregation tool and ASCE 7-16, respectively. The groundwater elevation was set to a depth of 4 feet.

Based upon the CPT findings collected during our recent exploration of the upper 50 feet of site materials, liquefaction/seismic settlements ranging from about 1 to 2 inches were calculated, with an overall average from the four CPTs of about 1.7 inches.

Liquefaction Potential Index (Iwasaki, 1978)
We considered other methods of evaluation by using the liquefaction potential index (LPI) developed by Iwasaki, et al., 1978. This method considers depth and thickness of the liquefiable layer in respect to the surface effects of liquefaction. Based on this evaluation, the LPI is calculated to be between 1.7 and 4.1, depending on triggering method which is considered to have a low to high potential for liquefaction for the selected method.

Liquefaction Severity Number
We considered other manifestations method of evaluation using the liquefaction severity number (LSN). This method considers depth to liquefiable layers in respect to potential damage to surface layers. Based on this evaluation, the LSN is calculated to be between 1 and 9, depending on triggering method which is considered to have a moderate to major liquefaction for the selected method.

Differential Settlements
Based on the CPT findings, overall differential liquefaction settlement for the proposed structure is expected to be between 0.5 and 1 inches in 50 feet.

Ejecta
The ejection of sands or materials from the ground surface following a seismic event is referred to as ejecta. We are not currently aware of a methodology for determining the volume of potential ejecta. Based on engineering judgement, ejecta is not anticipated to be significant provided the recommendations presented in this report are applied to the development of the project site.

Lateral Displacement
Since the project site and the surrounding area is relatively flat; therefore, the potential for lateral displacement was not considered to have a potential impact for this project.

Static and Seismically Induced Slope Instability
The subject area is in an area of the site that is relatively flat; therefore, the potential for seismically induced slope instability for the existing slopes is considered negligible.

5.0 DISCUSSION AND CONCLUSIONS

Based on our findings, the project site could be subject to static settlement and some seismically induced settlements which could impact the support of the proposed structure. As such, we recommend mitigation measures be implemented to reduce the effects or presence of the settlement potential.

Static Settlement
Static settlement is anticipated based on the soft subsurface soil conditions. We have provided recommendations in the following sections of this report to overexcavate soils under the proposed

building, replace as engineered fills with increased relative compactions, incorporate ground improvement, and utilize rigid foundation approaches.

Mitigation Measures for Settlements Due to Soft Soils

Due to the potential for settlement conditions to affect site development and use mitigation measures are recommended. Measures to address settlement have a range of costs and complexity that can vary between projects and are generally selected based on acceptable amounts of risk and damage for the structure. We recognize that some mitigation measures can be cost prohibitive; however, the selected mitigation measures should at least provide protection for life safety. The selection of mitigation measure(s) is ultimately the decision of others such as the property owners and design build contract. For the purposes of this report, we have included a discussion of the following mitigation options to aid in project planning:

1. Deep foundations
2. Ground improvements
3. Conventional shallow or Mat foundations

Once a mitigation option is determined suitable, by others, additional recommendations can be provided by our firm under separate cover, if necessary.

<u>Deep Foundations</u>

One mitigation measure to address liquefaction is through the use of deep foundations, such as auger cast piles. The intent of this foundation system is to extend through the soil layer(s) which are susceptible to liquefaction and lateral displacement, and the transfer of building loads to a suitable bearing stratum, such as the sites' dense underlying gravels approximately 34 to 37 feet bgs. While this foundation system is considered to be effective in reducing the potential for seismically induced settlement, it may not be a cost-effective option for this project.

Due to the potential for negative skin friction, or downdrag, on the piles within the liquefiable soil layer(s) during a seismic event, the piles would need to be deepened below the dense layers to offset the effective downdrag loads, in addition to the loads of the structure itself. Additionally, due to the relatively shallow groundwater and the dense nature of the underlying gravels, constructability of a deep foundation system can be difficult.

We have assumed that a deep foundation system is not considered a cost-effective option to support the planned structure and mitigate the potential for settlement; therefore, this foundation type is not addressed within this report. If the owner desires a mitigation measure that includes a deep foundation design, we can prepare a proposal to provide that type of analysis under separate scope and contract.

<u>Ground Improvement</u>

Ground improvement techniques and their implementation are performed by a design-build contractor who specializes in the technique and can provide site specific designs to meet the desired conditions provided by the client and their geotechnical representative. Design-build operations are generally an iterative approach requiring consultations between the design-build contractor and the other professional team members. Of the options presented in this report, ground improvement is considered to provide the most protection against settlement.

Ground improvement techniques are generally based on changing the density or confinement of the soil through vibration or displacement via inclusions in the subsurface soils. The projected benefit is to potentially reduce the liquefaction settlement amount or provide a stiffer ground area for the settlements to occur in a more uniform fashion. In the Sacramento region these methods generally include the installation of stone columns or drilled displacement columns. Other methods exist such as rapid impact compaction and/or deep dynamic compaction; however, they are not generally suitable where existing structures are present or nearby. Based on the soil profile, we anticipate drilled displacement columns may be the most likely candidate. Ground improvement could allow for the use of conventional shallow foundations; however, the selection of the foundation system should be based on the acceptable settlement criteria. Some projects have elected to use mat foundations to further enhance performance.

If this method is desired for the development of this project, a design-build contractor would need to analyze the technique and prepare plans and specifications. Following development of the approach, our firm could review the prepared documentation and prepare supplemental recommendations, if necessary, to address any identified geotechnical concerns.

Conventional Shallow and Mat Foundations

If desired, conventional shallow or mat foundations could be used at the project site, *provided that they are used in conjunction with ground improvement techniques*. Considering the previous use of the site and the soft surface soils already present, the use of shallow or mat foundations would include overexcavation of the near-surface soils and placement of engineered fills prior to ground improvement. This would be needed to generate a working platform for the proposed improvements. This method also includes the placement of a crushed rock layer at the base of the excavation, which is intended to disperse any pore water pressure generated during or following a liquefaction event. The structural engineer should design the shallow or mat foundations to be sufficiently stiff to address the potential settlement of the soil and ultimate, differential settlement damages to the structure. Section 12.13.9.2 of ASCE 7-16 provides commentary regarding flexural demands for liquefaction design. The overexcavation conditions may be revisited depending upon the design-build ground improvement conditions.

Geotechnical Considerations for Development

Some geotechnical conditions should be considered for the development of the project site. The contractor and developer should consider these conditions when preparing the development and construction plans. Although additional items may arise, our firm has prepared the following summary of potential conditions below.

- The groundwater elevation likely fluctuates based on the time of year. The contractor should consider a preconstruction excavation test (i.e. test pit) prior to fully implementing the overexcavation and recompaction process to evaluate the subsurface moisture conditions at the time of construction.
- The project site is relatively flat which increases the potential for poor drainage practices. We recommend that the designers consider the grade designs to promote positive drainage away from the structural improvements.

6.0 SITE GRADING AND EARTHWORK IMPROVEMENTS

Excavation Characteristics

The uppermost site soils are anticipated to be excavatable with conventional earthwork equipment, such as a backhoe or mini-excavator. Sites with similar subsurface conditions generally resort to using mid-size excavators and larger dozers.

Soil Moisture Considerations

The compaction of soil to a desired relative compaction is dependent on conditioning the soil to a target range of moisture content. Moisture contents that are excessively dry or wet could limit the ability of the contractor to compact soils to the requirements for engineered fill. When dry, moisture should be added to the soil and the soils blended to improve consistency. Wet soil will need to be dried to become compactable. Generally, this includes blending and working the soil to avoid trapping moisture below a dryer surficial crust. Other options are available to reduce the time involved but typically have higher costs and require more evaluation prior to implementation.

The largest contributor to excessive soil moisture is generally precipitation and seepage during the rainy season. In recognition of this, we suggest that consideration be given to the seasonal limitations and costs of winter grading operations on the site. Special attention should be given regarding the drainage of the project site. If the project is expected to work through the wet season, the contractor should install appropriate temporary drainage systems at the construction site and should minimize traffic over exposed subgrades due to the moisture-sensitive nature of the on-site soils. During wet weather operations, the soil should be graded to drain and should be sealed by rubber tire rolling to minimize water infiltration.

Site Preparation

Preparation of the project site should involve demolition, site drainage controls, dust control, clearing and stripping, overexcavation and recompaction of loose/soft soils, exposed grade compaction, and expansive soil mitigation considerations. The following paragraphs state our geotechnical comments and recommendations concerning site preparation.

<u>Site Drainage Controls</u>

We recommend that initial site preparation involve intercepting and diverting any potential sources of surface or near-surface water within the construction zones. Because the selection of an appropriate drainage system will depend on the water quantity, season, weather conditions, construction sequence, and methods used by the contractor, final decisions regarding drainage systems are best made in the field at the time of construction. All drainage and/or water diversion performed for the site should be in accordance with the Clean Water Act and applicable Storm Water Pollution Prevention Plan.

<u>Dust Control</u>

Dust control provisions should be provided for as required by the local jurisdiction’s grading ordinance (i.e., water truck or other adequate water supply during grading). Dust control is the purview of the grading contractor.

<u>Clearing and Stripping of Organic Materials</u>

Clearing and stripping operations should include the removal of all organic laden materials including trees, bushes, root balls, root systems, and any soft or loose soil generated by the removal operations. Short or mowed dry grasses may be pulverized and lost within fill materials provided no concentrated pockets of organics result. It is the responsibility of the grading contractor to remove excess organics from the fill materials. **No more than 2 percent of organic material, by weight, should be allowed within the fill materials at any given location.** Preserved trees may require tree root protection which should be addressed on an individual basis by a qualified arborist.

<u>Overexcavation and Recompaction</u>

Following general site clearing, all existing loose/soft or saturated native soils within the development footprint should be overexcavated down to firm native materials approximately two feet bgs and recompacted. Chemical treatment may be considered for improvement to address the high moisture conditions. The overexcavation operations should be performed regardless of the planned foundation system.

Exposed Grade Compaction

Exposed soil grades following initial site preparation activities and overexcavation operations should be scarified to a minimum depth of 8 inches and compacted to the requirements for engineered fill. Prior to placing fill, the exposed grades should be in a firm and unyielding state. Any localized zones of soft or pumping soils observed within the exposed grade should either be scarified and recompacted or be overexcavated and replaced with engineered fill as detailed in the engineered fill section below.

Working Platform for Ground Improvement

Ground improvement techniques typically use tall, narrow equipment which can be subject to overturning. To reduce this risk, a working platform should be constructed to the requirements of the installation contractor. Some approaches have included limited over-excavation and recompaction efforts (e.g., 2 feet), chemical-treatment, and/or placement of stiff surface materials such as aggregate baserock. These approaches could also aid in limited support of slabs.

Engineered Fill Criteria

All materials placed as fills on the site should be placed as “Engineered Fill" which is observed, tested, and compacted as described in the following paragraphs.

Suitability of Onsite Materials

We expect that soil generated from excavations on the site, excluding deleterious material, may be used as engineered fill provided the material does not exceed 6 inches in maximum dimension.

Import Materials

The recommendations presented in this report are based on the assumption that the import materials will be similar to the materials present at the project site. High quality materials are preferred for import; however, these materials can be more dependent on source availability. Import material should be approved by our firm prior to transporting it to the project site.

Material for this project should consist of a material with the geotechnical characteristics presented below. If these requirements are not met, additional testing and evaluation may be necessary to determine the appropriate design parameters for foundations, pavement, and other improvements.

Table 4: Select Import Criteria

Behavior Property	Reference Document	Recommendation
Direct Shear Strength	ASTM D3080	≥ 30° when compacted
Plasticity Index	ASTM D4318	≤ 12
Expansion Index	ASTM D4829	≤ 20
Sieve Analysis	ASTM D1140	Not more than 30% Passing the No. 200 sieve
Maximum Aggregate Size	ASTM D1140	≤ 6”

Fill Placement and Compaction

Engineered fills should be placed in thin horizontal lifts not to exceed 8 inches in uncompacted thickness. If the contractor can achieve the recommended relative compaction using thicker lifts, the method may be judged acceptable based on field verification by a representative of our firm using standard density testing procedures. Lightweight compaction equipment may require thinner lifts to achieve the recommended relative compaction. Fills should have a maximum particle size of 8 inches unless approved by our firm.

Table 5: Recommended Relative Compaction

Fill Materials	Relative Compaction Private/Public	Method Private/Public
Engineered Fill	95 percent	ASTM D1557
Subgrade	95 percent	ASTM D1557
Aggregate Baserock Grade	95 percent	ASTM D1557

* Unless otherwise required by the utility or governing agency.

Depending on the moisture condition of the soils, the engineered fills may require moisture conditioning to be within a suitable compaction range.

Our firm should be requested for consultation, observation, and testing for the earthwork operations prior to the placement of any fills. Fill soil compaction should be evaluated by means of in-place density tests performed during fill placement so that adequacy of soil compaction efforts may be determined as earthwork progresses.

Underground Improvements

Trench Excavation

Trenches or excavations in soil should be shored or sloped back in accordance with current Cal/OSHA regulations prior to persons entering them. The potential use of a shield to protect workers cannot be precluded. Refer to the Excavation Characteristics section of Site Grading and Improvements of this report for anticipated excavation conditions.

Backfill Materials

Backfill materials for utilities should conform to the requirements of the local jurisdiction. It should be realized that permeable backfill materials will likely carry water at some time in the future.

When backfilling within structural footprints, compacted low permeability materials are recommended to be used a minimum of 5 feet beyond the structural footprint to minimize moisture intrusion.

Backfill Compaction

Backfill compaction should conform to the requirements of the local jurisdiction or to the recommendations of this report, whichever is greater. Where backfill compaction is not specified by the local jurisdiction, the backfill should be compacted to achieve the minimum relative compactions specified in Table 5 of this report.

Exposure to Water

The configuration of a trench increases the likelihood that the trench may be exposed to or retain water. The presence of water can adversely impact the performance of the trench by increasing the potential for the transmission of water to undesired outlets and settlement, even when compacted to the requirements of engineered fill. The contractor should consider these conditions when managing water during interim and post construction periods. This topic is discussed further in the Drainage section of this report.

Floatation
Based on the liquefaction evaluation, underground utilities may be susceptible to floatation as a result of a liquefaction event. The designer or manufacture of the utilities should be consulted regarding resisting elements or capabilities of the utility's elements.

7.0 DESIGN RECOMMENDATIONS

The contents of this section include recommendations for shallow foundations with ground improvement, deep foundations, slab-on-grade foundations, and drainage. We anticipate that the proposed self-storage structure is to be supported using conventional foundations with ground improvements or deep foundations such as auger cast piles or drilled displacement piles (DDC). The foundation designer should evaluate the conditions and prepare a design appropriate to their needs. Discussions regarding geotechnical elements are provided below

Ground Improvements

Auger Cast Pile or Drilled Displacement Column
Auger cast pile or column systems have been successfully used for ground improvements in the Sacramento area. Both densification of soil surrounding displacement elements and frictional or end bearing resistance occur when designing these systems. These systems are generally either continuous flight auger (CFA) piles, drilled displacement piles (DDP), auger pressure grouted displacement (APGD) piles, or drilled displacement columns (DDC). These systems are similar in installation procedures to cast-in-drilled-hole (CIDH) piles; however, the capacities can vary since pressure is used to potentially improve skin friction capacities by applying load to the sidewalls during installation. The volume of the hole can also vary due to the deformation of the sidewalls when uncased. The installation can be beneficial to control noise and vibration and generally have low spoils volumes, thus reducing the impact of foundation installation on the public and surrounding facilities.

Vibratory Stone Columns
Vibratory stone columns have been a successful ground improvement solution for projects in the region. However, the vibrations may be an issue for neighboring structures and difficult operation in clays. Consequently, we do not recommend this ground improvement method for this project without further review.

Field Evaluation of Ground Improvements
Ground improvement operations should be observed and documented by our firm during installation. Inspections should include the depth, spacing, material used, approximate dimensions, and other geotechnically related parameters established by the design. Post-installation CPT soundings may be performed following the completion of installation of ground improvement methods throughout the process. These services are not included in the current scope.

Implementation of the ground improvement technique should also include, at a minimum, the load testing of axial capacity of at least one improvement element (per type) in accordance with ASTM D1143 and D3689 (if appropriate) or other method approved by our firm.

Shallow Conventional Foundations with Ground Improvements

Shallow conventional foundation systems are considered suitable for construction of the proposed self-storage structure, provided that the site is prepared in accordance with the recommendations discussed in Section 6.0 of this report.

The provided values do not constitute a structural design of foundations which should be performed by the structural engineer. In addition to the provided recommendations, foundation design and construction should conform to applicable sections of the 2022 California Building Code.

Estimated Foundation Capacities

The estimated foundation bearing and lateral capacities are presented in the table below for planning purposes. *Final determination of these capacities should be provided by the design-build contractor and/or structural engineer based upon the utilized ground improvement technique and settlement criteria.* The allowable bearing capacity is for support of dead plus live loads based on the foundation configuration presented in this report. The allowable capacity may be increased by 1/3 for short-term wind and seismic loads. Lateral forces on structures may be resisted by passive pressure acting against the sides of shallow footings and/or friction between the foundation bearing material and the bottom of the footing. Section 1806.3 of the 2022 CBC allows for the combination of the friction factor and passive resistance value to lateral resistance. Consideration should be given to ignoring passive resistance where soils could be disturbed later or within 6 feet horizontally of the slope face.

Table 6: Estimated Foundation Capacities

Soil Type	Design Condition	Design Value
Engineered Fill	Allowable Bearing Capacity	2,000 psf
	Allowable Fiction Factor*	0.40
	Allowable Passive Resistance	230 psf/ft
* Friction Factor is calculated as tan(ϕ)		

Foundation Settlement

Acceptable settlement ranges should be based on the requirements of the structural engineer. The calculated settlement of the given site and structure is dependent on many factors, including total load, load configuration, and condition of the ground supporting the foundation system. The recommendations provided for the conventional footing systems uses ground improvement elements to provide support for the proposed improvements. As such, the ground improvement contractor should provide settlement estimates based on the selected technique. This generally includes the ground improvement contractor with information presented in this report and load data supplied by the structural engineer for design of their system.

Foundation Configuration

Conventional shallow foundations should be a minimum of 18 inches wide and founded a minimum of 24 inches below the lowest adjacent soil grade. Isolated pad foundations should be a minimum of 24 inches in plan dimension. All isolated pad footings should be interconnected on at least two sides by grade beams, having the same depth as the continuous footings. Foundation configuration and reinforcement should be provided by the structural engineer, taking into account the requirements of Section 12.13.9.2 of ASCE 7-16.

Foundation reinforcement should be provided by the structural engineer. The reinforcement schedule should account for typical construction issues such as load consideration, concrete cracking, and the presence of isolated irregularities. At a minimum, we recommend that continuous footing foundations be reinforced four No. 4 reinforcing bars, two located near the bottom of the footing and two near the top of the stem wall.

<u>Foundation Influence Line and Slope Setback</u>
All footings should be founded below an imaginary 2H:1V plane projected up from the bottoms of adjacent footings and/or parallel utility trenches, or to a depth that achieves a minimum horizontal clearance of 6 feet from the outside toe of the footings to the slope face, whichever requires a deeper excavation.

<u>Subgrade Conditions</u>
Footings should never be cast atop soft, loose, organic, slough, debris, nor atop subgrades covered by ice or standing water. A representative of our firm should be retained to observe all subgrades during footing excavations and prior to concrete placement so that a determination as to the adequacy of subgrade preparation can be made.

<u>Shallow Footing / Stemwall Backfill</u>
All footing/stemwall backfill soil should be compacted to the criteria for engineered fill as recommended in Section 6.0 of this report.

Mat Foundations with Ground Improvements
Soil-supported mat foundations could be used for the main floor of the proposed structure, provided the recommendations from this report are implemented and the foundation could accommodate the potential differential settlement.

The geotechnical issues regarding the use of this foundations include proper soil support and subgrade preparation, proper transfer of loads through the slab underlayment materials to the subgrade soils, and the anticipated presence or absence of moisture below, at, or above the subgrade level. We offer the following comments and recommendations concerning support of mat foundations. The concrete design (concrete mix, reinforcement, moisture protection, and underlayment materials) and possible chemical treatment of soils below the foundation is the purview of the project Structural Engineer.

<u>Estimated Bearing Capacity</u>
The bearing capacity of the mat foundation is expected to be controlled by settlement rather than localized bearing failures. *Final determination of these capacities should be provided by the design-build contractor and/or structural engineer based upon the utilized ground improvement technique and settlement criteria.* We anticipate the allowable pressures are for support of dead plus live loads and may be increased by 1/3 for short-term wind and seismic loads. An estimated allowable dead plus live load bearing pressure of 1,200 psf may be used for design of a mat foundation supported on engineered fills with ground improvement.

Lateral forces on structures may be resisted by passive pressure acting against the sides of mat and/or friction between the foundation bearing material and the bottom of the mat. Section 1806.3 of the 2022 CBC allows for the combination of the friction factor and passive resistance value to lateral resistance. Consideration should be given to ignoring passive resistance where soils could be disturbed later or within 6 feet horizontally of an open cut face.

Table 6: Estimated Foundation Capacities

Soil Type	Design Condition	Design Value
Engineered Fill	Allowable Bearing Capacity	1,200 psf
	Allowable Fiction Factor*	0.40
	Allowable Passive Resistance	250 psf/ft
* Friction Factor is calculated as tan(ϕ)		

Foundation Settlement

Acceptable settlement ranges should be based on the requirements of the structural engineer. The calculated settlement of the given site and structure is dependent on many factors, including total load, load configuration, and condition of the ground supporting the foundation system. The recommendations provided for the conventional footing systems uses ground improvement elements to provide support for the proposed improvements. As such, the ground improvement contractor should provide settlement estimates based on the selected technique. This generally includes the ground improvement contractor with information presented in this report and load data supplied by the structural engineer for design of their system.

Foundation Influence Line and Slope Setback

All footings should be founded below an imaginary 2H:1V plane projected up from the bottoms of adjacent footings and/or parallel utility trenches

Slab-on-Grade Construction with Ground Improvements

It is our opinion that soil-supported slab-on-grade floors could be used for the main floor of the structure, contingent on proper subgrade preparation and ground improvement. Often the geotechnical issues regarding the use of slab-on-grade floors include proper soil support and subgrade preparation, proper transfer of loads through the slab underlayment materials to the subgrade soils, and the anticipated presence or absence of moisture at or above the subgrade level. We offer the following comments and recommendations concerning support of slab-on-grade floors. The slab design (concrete mix design, curing procedures, reinforcement, joint spacing, moisture protection, and underlayment materials) is the purview of the project Structural Engineer.

Slab Subgrade Preparation

All subgrades proposed to support slab-on-grade floors should be prepared and compacted to the requirements of engineered fill as discussed in Section 6.0 of this report. To reduce the potential for drying following completion of grading, it is preferable that the grading operations be performed relatively close to the time of construction. If performed early, the building pads should be protected from loss of moisture.

Slab Underlayment

As a minimum for slab support conditions, the slab should be underlain by a minimum 4-inch-thick crushed rock layer that is covered by a minimum 10-mil thick moisture retarding plastic membrane. The membrane may only be functional when it is above the vapor sources. The bottom of the crushed rock layer should be above the exterior grade to act as a capillary break and not a reservoir, unless it is provided with an underdrain system. The slab design and underlayment should be in accordance with ASTM E1643 and E1745.

An optional 1-inch blotter layer (e.g., sand and pea gravel) placed above the plastic membrane, is sometimes used to aid in curing of the concrete. The blotter layer materials should be specified by the structural engineer. Although historically common, this blotter layer is not currently included in slabs designed according to the 2022 Green Building Code. When omitted, special wet curing procedures will be necessary. If installed, the blotter layer can become a reservoir for excessive moisture if inclement weather occurs prior to pouring the slab, excessive water collects in it from the concrete pour, or an external source of water enters above or bypasses the membrane. Development of appropriate slab mix design and curing procedures remains the purview of the project structural engineer.

Our experience has shown that vapor transmission through concrete is controlled through proper concrete mix design. As such, proper control of moisture vapor transmission should be considered in the design of the slab as provided by the project architect, structural or civil engineer. It should be noted that placement of the recommended plastic membrane, proper mix design, and proper slab underlayment and detailing per ASTM E1643 and E1745 will not provide a waterproof condition. If a waterproof condition is desired, we recommend that a waterproofing expert be consulted for slab design.

Slab Thickness and Reinforcement

Geotechnical reports have historically provided minimums for slab thickness and reinforcement for general crack control. The concrete mix design and construction practices can additionally have a large impact on concrete crack control. All concrete should be anticipated to crack. As such, these minimums should not be considered to be standalone items to address crack control, but are suggested to be considered in the slab design methodology.

In order to help control the growth of cracks in interior concrete from becoming significant, we suggest the following minimums. Interior concrete slabs-on-grade not subject to heavy loads, should be a minimum of 4-inches thick and reinforced. A minimum of No. 3 deformed reinforcing bars placed at 24 inches on center both ways, at the center of the structural section is suggested. Joint spacing should be provided by the structural engineer. Troweled joints recovered with paste during finishing or “wet sawn” joints should be considered every 10 feet on center. Expansion joint felt should be provided to separate floating slabs from foundations and at least at every third joint. Cracks will tend to occur at recurrent corners, curved or triangular areas and at points of fixity. Trim bars can be utilized at right angle to the predicted crack extending 40 bar diameters past the predicted crack on each side.

Vertical Deflections

Soil-supported slab-on-grade floors can deflect downward when vertical loads are applied, due to elastic compression of the subgrade. For preliminary design of concrete floors, a modulus of subgrade reaction of k = 100 psi per inch would be applicable for engineered fills.

Exterior Flatwork

Exterior concrete flatwork is recommended to have a 4-inch-thick rock cushion. This could consist of vibroplate compacted crushed rock or compacted ¾-inch aggregate baserock. If exterior flatwork concrete is against the floor slab edge without a moisture separator it may transfer moisture to the floor slab. Expansion joint felt should be provided to separate exterior flatwork from foundations and at least at every third joint. Contraction / groove joints should be provided to a depth of at least 1/4 of the slab thickness and at a spacing of less than 30 times the slab thickness for unreinforced flatwork, dividing the slab into nearly square sections. Cracks will tend to occur at recurrent corners, curved or triangular areas and at points of fixity. Trim bars can be utilized at right angle to the predicted crack extending 40 bar diameters past the predicted crack on each side.

Retaining Walls

Our design recommendations and comments regarding retaining walls for the project site are discussed below. *Retaining wall foundations should be designed in accordance with the Shallow Conventional Foundations section above.*

Retaining Wall Lateral Pressures

Based on our observations and testing, the retaining wall should be designed to resist lateral pressure exerted from a soil media having an equivalent fluid weight provided in the table below. The values presented below are not factored and are for conditions when firm native soil or engineered fill is used within the zone behind the wall defined as twice the height of the retaining wall. Additionally, the values do not account for the friction of the backfill on the retaining wall which may or may not be present depending on the wall materials and construction.

The lateral pressures presented in the table below include recommendations for earthquake loading which is required for structures to be designed in Seismic Design Categories D, E or F per Section 1803.5.12.1 of the 2022 California Building Code. The lateral pressures presented have been calculated using the Mononobe-Okabe Method derived from Wood (1973) and modified by Whitman et al. (1991). The values are intended to be used as the multiplier for uniformly distributed loads and the parameter "H" is the total height of the wall including the footing but excluding any key, if used.

Table 7: Retaining Wall Pressures

Wall Type	Wall Slope Configuration	Equivalent Fluid Weight (pcf)	Lateral Pressure Coefficient	Earthquake Loading (plf)	
Free Cantilever	Flat	40 (Drained) 78 (Undrained)	0.31	$8H^2$	Applied 0.6H above the base of the wall
Restrained*	Flat	60 (Drained) 90 (Undrained)	0.47	$28H^2$	

* Restrained conditions shall be defined as walls which are structurally connected to prevent flexible yielding, or rigid wall configurations (i.e., walls with numerous turning points) which prevent the yielding necessary to reduce the driving pressures from an at-rest state to an active state.

Design Values for Dry Stacked Walls

Dry stacked walls do not generally use the equivalent fluid weight method presented above; instead, they use design soil properties for a given soil condition such as the internal friction angle, cohesion, and bulk unit weight. The walls could include keyed or interlocking non-mortared walls such as segmental block (Basalite, Keystone, Allan Block, etc.), rockery walls, or specialty designs for proprietary systems. When this occurs, the following soil parameters would be applicable for design with the onsite native materials in a firm condition or for engineered fills. The seismic coefficient is considered to be ½ of the adjusted peak ground acceleration for the site conditions is given in Section 4.0 of this report. Some software allows for the extension of the Mononobe-Okabe Method beyond the conventional limitations and, if the method is applied, could calculate seismic values significantly higher than those provided by the multiplier method provided above.

Table 8: Generalized Design Parameters

Internal Angle of Friction	Cohesion	Bulk Unit Weight	Seismic Coefficient, Kh
31°	0 psf	120 psf	0.222g

Wall Drainage

The criteria presented above is based on fully drained conditions as detailed in the attached Figure C-1, Appendix C. For these conditions, we recommend that a blanket of filter material be placed behind all proposed walls. Permeable materials are specified in Section 68 of the California Department of Transportation Standard Specifications, current edition. The filter material should conform to Class 1, Type B permeable material in combination with a filter fabric to separate the open graded gravel/rock from the surrounding soils. Generally, a clean ¾ inch crushed rock should be acceptable. Consistent with Caltrans Standards, when Class 2 permeable materials are used, the filter fabric may be omitted unless otherwise designed.

The blanket of filter material should be a minimum of 12-inches thick and should extend from the bottom of the wall to within 12 inches of the ground surface. The top 12 inches of wall backfill should consist of a compacted soil cap. A filter fabric having specifications equal to or greater than those for Mirafi 140N should be placed between the gravel filter material and the surrounding soils to reduce the potential for infiltration of soil into the gravel. A 4-inch diameter drain pipe should be installed near the bottom of the filter blanket with perforations facing down. The drainpipe should be underlain by at least 4 inches of filter-type material. An adequate gradient should be provided along the top of the foundation to discharge water that collects behind the retaining wall to a controlled discharge system.

The configuration of a long retaining wall generally does not allow for a positive drainage gradient within the perforated drain pipe behind the wall since the wall footing is generally flat with no gradient for drainage. Where this condition is present, to maintain a positive drainage behind the walls, we recommend that the wall drains be provided with a discharge to an appropriate non-erosive outlet a maximum of 50 feet on center. **In addition, if the wall drain outlets are temporarily stubbed out in front of the walls for future connection during building construction, it is imperative that the outlets be routed into the tight pipe area drainage system and not buried and rendered ineffective**

Asphalt Concrete Pavement Design

We understand that asphalt pavements will be used for the associated roadways. The following comments and recommendations are given for pavement design and construction purposes. All pavement construction and materials used should conform to applicable sections of the latest edition of the California Department of Transportation Standard Specifications.

Relative Compaction

The asphalt concrete pavement section should be constructed to achieve the minimum relative compactions specified in Section 6.0 of this report. Deviation from the following values should be reviewed by the governing agency when the pavements are to be constructed within their right-of-way.

Subgrade Stability

All subgrades and aggregate base should be proof-rolled with a full water truck or equivalent immediately before paving, in order to evaluate their condition. If unstable subgrade conditions are observed, these areas should be overexcavated down to firm materials and the resulting excavation backfilled with suitable materials for compaction (i.e., drier native soils or aggregate base). Areas displaying significant instability may require geotextile stabilization fabric within the overexcavated area, followed by placement of aggregate base. Final determination of any required overexcavation depth and stabilization fabric should be based on the conditions observed during subgrade preparation.

Subgrade Resistance Value
Critical features that govern the durability of a pavement section include the stability of the subgrade; the presence or absence of moisture, free water, and organics; the fines content of the subgrade soils; the traffic volume; and the frequency of use by heavy vehicles. Soil conditions can be defined by a soil resistance value, or "R-Value," and traffic conditions can be defined by a Traffic Index (TI).

Laboratory testing was performed on bulk samples considered to be representative of the materials expected to be exposed at subgrade. An R-Value of 27 was identified for the tested soils and used this value for the pavement sections this report. Following the rough grading operations, the subgrade conditions should be evaluated to determine whether adjustments to the design R-value are warranted.

Design values provided are based upon properly drained subgrade conditions. Although the R-Value design to some degree accounts for wet soil conditions, proper surface and landscape drainage design is integral in performance of adjacent street sections with respect to stability and degradation of the asphalt. If clay soils are encountered and cannot be sufficiently blended with non-expansive soils, we should review pavement subgrades to determine the appropriateness of the provided sections, and provide additional pavement design recommendations as field conditions dictate. Even minor clay constituents will greatly reduce the design R-Value.

Section Thickness
The recommended design thicknesses presented in the following table were calculated in accordance with the methods presented in the Sixth Edition of the California Department of Transportation Highway Design Manual. A varying range of traffic indices are provided for use by the project Civil Engineer for roadway design.

Table 9: Asphalt Pavement Section Recommendations (R = 27)

Design Traffic Indices	Alternative Pavement Sections (Inches)				
	Standard Section		Cement-treated Section		
	Asphalt Concrete *	Aggregate Base **	Asphalt Concrete *	Aggregate Base **	Cement Treated Soil***
5.0	2.5	7.0	2.5	4.0	12.0
	3.0	6.0	2.5	4.0	12.0
6.0	3.0	9.0	2.5	4.0	12.0
	3.5	8.0	3.0	4.5	12.0
7.0	4.0	10.5	3.5	4.5	12.0
	4.5	9.5	4.0	5.0	12.0
8.0	4.5	12.5	4.0	5.0	12.0
	5.0	11.5	4.5	5.5	12.0
9.0	5.5	13.5	4.5	5.5	12.0
	6.0	13.0	5.0	6.0	12.0

* Asphalt Concrete: must meet specifications for Caltrans Hot Mix Asphalt Concrete
** Aggregate Base: must meet specifications for Caltrans Class II Aggregate Base (R-Value = minimum 78)
***Cement Treated Soil must meet a minimum 7-day compressive strength of 300 psi (R-Value = minimum 80)

Portland Cement Concrete Pavement Design
We understand that Portland cement concrete pavements may be considered for various aspects of the development, including the drive aisle at the trash enclosure and entry into the building.

The American Concrete Institute (ACI) Concrete Pavement Design method (ACI 330R-08) was used for design of the exterior concrete (rigid) pavements at the site.

Relative Compaction
The asphalt concrete pavement section should be constructed to achieve the minimum relative compactions specified in Section 6.0 of this report. Deviation from the following table should be reviewed by the governing agency when the pavements are to be constructed within their right-of-way. Final acceptance of the constructed pavement section is the purview of the governing agency.

Subgrade Stability
All subgrades and aggregate base should be proof-rolled with a full water truck or equivalent immediately before paving, in order to evaluate their condition.

Soil Design Parameters
The pavement thicknesses were evaluated based on the soil design parameters provided in the following table.

Table 10: Soil Parameters

Subgrade Soil Description	k, Modulus of Subgrade Reaction*	Base Course
CLAY	158 pci	6 inches

* Based on an R-Value of 20 as recommended above and correlated to a k-Value recommended by ACI 330R.

Section Thickness
Based on the subgrade soil parameters shown in the above table, the recommended concrete thicknesses for various traffic descriptions are presented in the table below. The recommended thicknesses provided below assume the use of plain (non-reinforced) concrete pavements.

Table 11: Concrete Pavement Section Recommendations

Category	ADTT*	Pavement Traffic Description	Thickness (inches)	
			3000 psi**	4000 psi**
A	1	Car parking areas and access lanes Autos, pickups, and panel trucks only	5.0	4.5
A	10		5.5	5.0
B	25	Shopping center entrance and service lanes Bus parking areas and interior lanes Single-unit truck parking areas and interior lanes	6.0	5.5
B	300		7.0	6.0
C	100	Roadway Entrances and Exterior Lanes	7.0	6.5
C	300		7.5	6.5
C	700		7.5	7.0

* Average Daily Truck Traffic
** 28-day concrete compressive strength

Jointing and Reinforcement
From a geotechnical perspective, contraction joints should be placed in accordance with the American Concrete Institute (ACI) recommendations which include providing a joint spacing about 30 times the slab thickness up to a maximum of 10 feet. The joint patterns should also divide the slab into nearly square panels. If increased joint spacing is desired, reinforcing steel should be installed within the pavement in accordance with ACI recommendations. Final determination of steel reinforcement configurations (if used within the pavements) remains the purview of the Project Structural Engineer.

Drainage

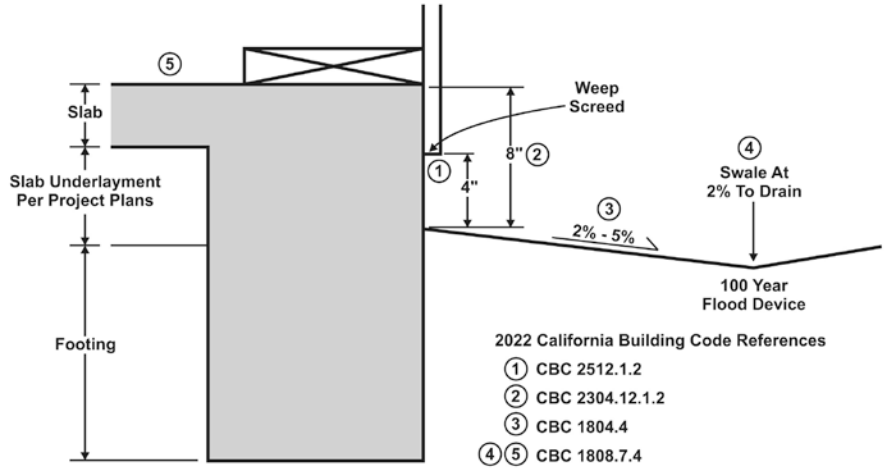
In order to maintain the engineering strength characteristics of the soil presented for use in this report, maintenance of the site will need to be performed. This maintenance generally includes, but is not limited to, proper drainage and control of surface and subsurface water which could affect structural support and fill integrity. A difficulty exists in determining which areas are prone to the negative impacts resulting from high moisture conditions due to the diverse nature of potential sources of water; some of which are outlined in the paragraph below. We suggest that measures be installed to minimize exposure to the adverse effects of moisture, but this will not guarantee that excessive moisture conditions will not affect the structure.

Some of the diverse sources of moisture could include water from landscape irrigation, annual rainfall, offsite construction activities, runoff from impermeable surfaces, collected and channeled water, and water perched in the subsurface soils. Some of these sources can be controlled through drainage features installed either by the owner or contractor. Others may not become evident until they, or the effects of the presence of excessive moisture, are visually observed on the property.

Some measures that can be employed to minimize the buildup of moisture include, but are not limited to proper backfill materials and compaction of utility trenches within the footprint of the proposed structures; grout plugs at foundation penetrations; collection and channeling of drained water from impermeable surfaces (i.e. roofs, concrete or asphalt paved areas); installation of subdrain/cut-off drain provisions; utilization of low flow irrigation systems; education to the proposed owners of proper design and maintenance of landscaping and drainage facilities that they or their landscaper installs.

Drainage Adjacent to Buildings

All grades should provide rapid removal of surface water runoff; ponding water should not be allowed on building pads or adjacent to foundations or other structural improvements (during and following construction). All soils placed against foundations during finish grading should be compacted to minimize water infiltration. Finish and landscape grading should include positive drainage away from all foundations. Section 1808.7.4 of the 2022 California Building Code (CBC) states that for graded soil sites, the top of any exterior foundation shall extend above the elevation of the street gutter at the point of discharge or the inlet of an approved drainage device a minimum of 12 inches plus 2 percent. If overland flow is not achieved adjacent to buildings, the drainage device should be designed to accept flows from a 100-year event. Grades directly adjacent to foundations should be no closer than 8 inches from the top of the slab (CBC 2304.12.1.2), and weep screeds are to be placed a minimum of 4 inches clear of soil grades and 2 inches clear of concrete or other hard surfacing. From this point, surface grades should slope a minimum of 2 percent away from all foundations for at least 5 feet but preferably 10 feet, and then 2 percent along a drainage swale to the outlet (CBC 1804.4). Downspouts should be tight piped via an area drain network and discharged to an appropriate non-erosive outlet away from all foundations.



Typical 2022 California Building Code
Drainage Requirements

The above referenced elements pertaining to drainage of the proposed structures is provided as general acknowledgement of the California Building Code requirements, restated and graphically illustrated for ease of understanding. Surface drainage design is the purview of the Project Architect/Civil Engineer. Review of drainage design and implementation adjacent to the building envelopes is recommended as performance of these improvements is crucial to the performance of the foundation and construction of rigid improvements.

ADA Compliance and Drainage

It should be noted that due to the Americans with Disabilities Act (ADA) requirements, design and construction of alternative site drainage configurations may be necessary, particularly for multi-family and commercial developments. In this case, design and construction of adequate drainage adjacent to foundations and slabs are essential to preserving foundation support and reducing the potential for wet slab related issues. A typical example of this condition occurs in commercial developments where the landscape grades are situated at the same elevation as the parking areas so as to not create a drop off between the grades. This condition subsequently results in flat grades between the building, landscape area, and parking lot which do not meet building code requirements and may require more substantial drain inlets.

Parking Area Landscaping Drainage

Prolonged water seepage into pavement sections can result in softening of subgrade soils and subsequent pavement distress. It is anticipated that heavy landscape watering could enter and pond within the aggregate base section as it permeates through the aggregate base under the sidewalks and/or curbs. Prolonged seepage within the pavement section could cause distress to pavements in heavy traffic areas. Some measures that can be employed to minimize the saturation of the subgrade and aggregate base materials include, but are not limited to, construction of cut-off drains or moisture barriers alongside the edge of the pavement, construction of subdrains within landscape areas and installation of plug and drain systems within utility trenches. Due to the elusive and discontinuous nature of drainage related issues, a risk-based approach should be determined by the developer based on consultation and discussions with the design professionals and the amount of protection of facilities that the developer may want to provide against potential moisture related issues.

Post Construction
All drainage related issues may not become known until after construction and landscaping are complete. Therefore, some mitigation measures may be necessary following site development. Landscape watering is typically the largest source of water infiltration into the subgrade. Given the soil conditions on site, excessive or even normal landscape watering could contribute to moisture related problems and/or cause distress to foundations and slabs, pavements, and underground utilities, as well as creating a nuisance where seepage occurs.

8.0 DESIGN REVIEW AND CONSTRUCTION MONITORING

Geotechnical engineering can be affected by natural variability of soils and, as with many projects, the contents of this report could be used and interpreted by many design professionals for the application and development of their plans. For these reasons, we recommend that our firm provide support through plan reviews and construction monitoring to aid in the production of a successful project.

Plan Review

The design plans and specifications should be reviewed and accepted by Youngdahl Consulting Group, Inc. prior to contract bidding. A review should be performed to determine whether the recommendations contained within this report are still applicable and/or are properly interpreted and incorporated into the project plans and specifications. Modifications to the recommendations provided in this report or to the design may be necessary at the time of our review based on the proposed plans.

Construction Monitoring

Construction monitoring is a continuation of geotechnical engineering to confirm or enhance the findings and recommendations provided in this report. It is essential that our representative be involved with all grading activities in order for us to provide supplemental recommendations as field conditions dictate. Youngdahl Consulting Group, Inc. should be notified at least two working days before site clearing or grading operations commence, and should observe the stripping of deleterious material, overexcavation of soft soils and existing fills (if present), and provide consultation, observation, and testing services to the grading contractor in the field. At a minimum, Youngdahl Consulting Group, Inc. should be retained to provide services listed in Table 10 below.

The recommendations included in this report have been based in part on assumptions about strata variations that may be tested only during earthwork. Accordingly, these recommendations should not be applied in the field unless Youngdahl Consulting Group, Inc. is retained to perform construction observation and thereby provide a complete professional geotechnical engineering service through the observational method. Youngdahl Consulting Group, Inc. cannot assume responsibility or liability for the adequacy of its recommendations when they are used in the field without Youngdahl Consulting Group, Inc. being retained to observe construction.

Post Construction Drainage Monitoring

Due to the elusive nature of subsurface water, the alteration of water features for development, and the introduction of new water sources, all drainage related issues may not become known until after construction and landscaping are complete. Youngdahl Consulting Group, Inc. can provide consultation services upon request that relate to proper design and installation of drainage features during and following site development.

9.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

1. This report has been prepared for the exclusive use of the addressee of this report for specific application to this project. The addressee may provide their consultants authorized use of this

report. Youngdahl Consulting Group, Inc. has endeavored to comply with generally accepted geotechnical engineering practice common to the local area. Youngdahl Consulting Group, Inc. makes no other warranty, expressed or implied.

2. As of the present date, the findings of this report are valid for the property studied. With the passage of time, changes in the conditions of a property can occur whether they be due to natural processes or to the works of man on this or adjacent properties. Legislation or the broadening of knowledge may result in changes in applicable standards. Changes outside of our control may cause this report to be invalid, wholly or partially. Therefore, this report should not be relied upon after a period of three years without our review nor should it be used or is it applicable for any properties other than those studied.

3. Section [A] 107.3.4 of the 2022 California Building Code states that, in regard to the design professional in responsible charge, the building official shall be notified in writing by the owner if the registered design professional in responsible charge is changed or is unable to continue to perform the duties.

   WARNING: Do not apply any of this report's conclusions or recommendations if the nature, design, or location of the facilities is changed. If changes are contemplated, Youngdahl Consulting Group, Inc. must review them to assess their impact on this report's applicability. Also note that Youngdahl Consulting Group, Inc. is not responsible for any claims, damages, or liability associated with any other party's interpretation of this report's subsurface data or reuse of this report's subsurface data or engineering analyses without the express written authorization of Youngdahl Consulting Group, Inc.

4. The analyses and recommendations contained in this report are based on limited windows into the subsurface conditions and data obtained from subsurface exploration. The methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Samples cannot be relied on to accurately reflect the strata variations that usually exist between sampling locations. Should any variations or undesirable conditions be encountered during the development of the site, Youngdahl Consulting Group, Inc. will provide supplemental recommendations as dictated by the field conditions.

Table 10: Checklist of Recommended Services

	Item Description	Recommended	Not Anticipated
1	Provide foundation design parameters	Included	
2	Review grading plans and specifications	✓	
3	Review foundation plans and specifications	✓	
4	Observe and provide recommendations regarding demolition	✓	
5	Observe and provide recommendations regarding site stripping	✓	
6	Observe and provide recommendations on moisture conditioning removal, and/or recompaction of unsuitable existing soils	✓	
7	Observe and provide recommendations on the installation of subdrain facilities		✓
8	Observe and provide testing services on fill areas and/or imported fill materials	✓	
9	Review as-graded plans and provide additional foundation recommendations, if necessary	✓	
10	Observe and provide compaction tests on storm drains, water lines and utility trenches		✓
11	Observe foundation excavations and provide supplemental recommendations, if necessary, prior to placing concrete	✓	
12	Observe and provide moisture conditioning recommendations for foundation areas and slab-on-grade areas prior to placing concrete		✓
13	Provide design parameters for retaining walls		✓
14	Provide finish grading and drainage recommendations	Included	
15	Provide geologic observations and recommendations for keyway excavations and cut slopes during grading		✓
16	Excavate and recompact all test pits within structural areas		✓

APPENDIX A

Field Study

Vicinity Map
Site Plan
Logs of Exploratory Boring Logs
Soil Classification Chart and Legend
Logs of CPT Soundings
Liquefaction Analyses

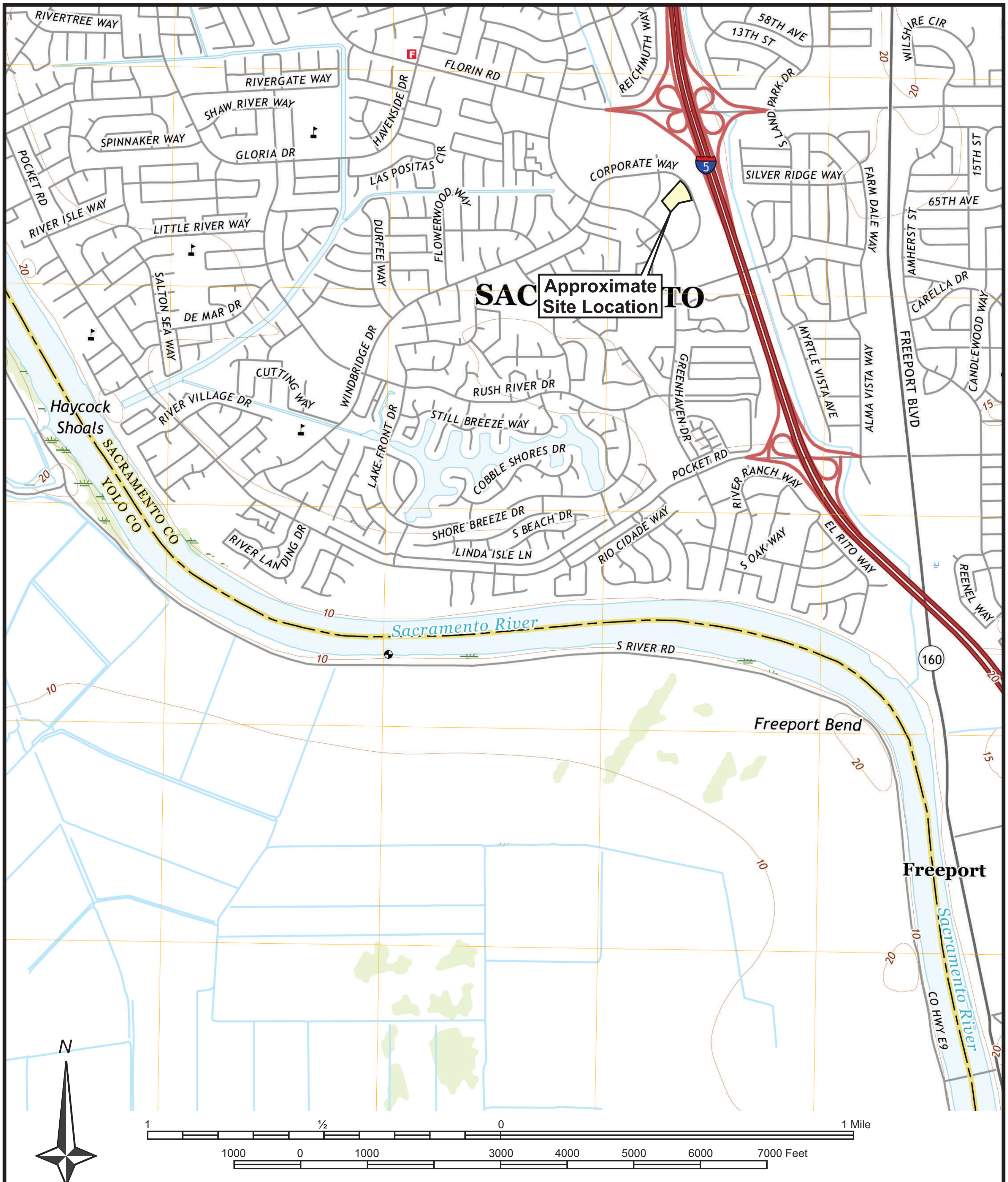
Introduction
The contents of this appendix shall be integrated with the Geotechnical Engineering Study of which it is a part. They shall not be used in whole or in part as a sole source for information or recommendations regarding the subject site.

Our field study included a site reconnaissance by a Youngdahl Consulting Group, Inc. representative followed by a subsurface exploration program conducted on 8 September 2023, which included the advancement of two (2) borings and the advancement of four (4) cone penetration test (CPT) soundings under his direction at the approximate locations shown on Figure A-2, this Appendix. Drilling was accomplished with a CME 55 truck mounted drill rig and CPTs were accomplished with a 20-ton electronic “push” CPT, 10-wheeled truck rig. The bulk and tube samples collected from the borings returned to our laboratory for further examination and testing.

The Exploratory Boring Logs describe the vertical sequence of soils and materials encountered in the borings, based primarily on our field classifications and supported by our subsequent laboratory examination and testing. Where a soil contact was observed to be gradual, our log indicates the average contact depth. Our log also graphically indicates the sample type, sample number, and approximate depth of each soil sample obtained from the boring.

The soils encountered were logged during this provide the basis for the “Exploratory Boring Logs”, Figures A-3 and A-4, this Appendix. This log shows a graphic representation of the soil profile, the location, and depths at which samples were collected.

The CPT data collected are provided in this section following Figure A-6. The enclosed CPT data describes the vertical sequence of soil behavior which was encountered during exploration based on cone resistance, sleeve friction, and pore water pressure.



Scale: 1:24,000

BASE MAP REFERENCE: *U.S.G.S. 7.5 Minute Topographic Series, Clarksburg Quadrangle, Dated 2021*

YOUNGDAHL CONSULTING GROUP, INC. ESTABLISHED 1984	Project No.: E23314.000	**VICINITY MAP**	**FIGURE**
	October 2023	**Banner Self-Storage Facility** Sacramento California	**A-1**

REFERENCE: *Conceptual Site Plan, provided by Client; Google Earth, Aerial Data Dated 2/15/2022*

YOUNGDAHL CONSULTING GROUP, INC. ESTABLISHED 1984	Project No.: E23314.000	SITE PLAN	FIGURE
	October 2023	Banner Self-Storage Facility Sacramento California	A-2

Logged By: **ITK**	Date: **8 September 2023**	Lat / Lon: **N 38.491872° / W 121.517478°**	Boring No. **B-1**
Equipment: **CME 55 Drill Rig - 4" Solid Stem Auger / Mud Rotary**		Elevation: **~**	

Depth (Feet)	Graphic Log	Ground Water	Geotechnical Description & Unified Soil Classification	Sample	Blow Counts	Pocket Pen (tsf)	Dry Density (pcf)	Moisture Content (%)	Tests & Comments
0-1			Grey to olive brown **CLAY (CL)**, medium stiff to stiff, slightly moist						**Bulk B-1** @ 0' - 5' Partial Recovery EI = 108 (high)
3.5			*Grades moist*		10	2.5			
6			*Grades very soft, moist to wet*		3	<.25			
7		▽	*Grades wet*						Switch to Mud Rotary
10.5			*Grades olive grey, fine grained, sandy*		4	<.25	83.2	36.7	LL = 43, PI = 20
16					3	<.25 <.25			
20.5			Olive grey **SILT (ML)** with sand, very soft, wet		2	<.25	94.4	30.1	72.5% < No. 200
21			Olive grey **CLAY (CL)**, very soft, wet				90.9	32.5	
25									

Boring Continued on Figure A-3b

Note: The boring log indicates subsurface conditions only at the specific location and time noted. Subsurface conditions, including groundwater levels, at other locations of the subject site may differ significantly from conditions which, in the opinion of Youngdahl Consulting Group, Inc., exist at the sampling locations. Note, too, that the passage of time may affect conditions at the sampling locations.

YOUNGDAHL CONSULTING GROUP, INC. ESTABLISHED 1984	Project No.: E23314.000 October 2023	**EXPLORATORY BORING LOG** **Banner Self-Storage Facility** Sacramento California	FIGURE **A-3a**

Logged By: **ITK**	Date: **8 September 2023**	Lat / Lon: **N 38.491872° / W 121.517478°**	Boring No. **B-1**
Equipment: **CME 55 Drill Rig - 4" Solid Stem Auger / Mud Rotary**		Elevation: **~**	

Depth (Feet)	Graphic Log	Ground Water	Geotechnical Description & Unified Soil Classification	Sample	Blow Counts	Pocket Pen (tsf)	Dry Density (pcf)	Moisture Content (%)	Tests & Comments
25–30.5			Brown sandy **SILT (ML)**, very soft, wet		4	<.25			
30.5–31			Brown clayey **SAND (SC)**, very loose, wet						LL = 28, PI = 8
31–35			Light olive brown **SILT (ML)**, very soft, wet		4		92.0	31.7	
35–37			Brown silty **SAND (SM)**, very loose, wet *Grades olive grey to grey*		4	<.25 <.25			
37–46.3			Olive grey to grey sandy **GRAVEL (GW)** with silt, subrounded, 2" max clast size, dense, wet						
41.5			*Grades very dense*		97				57.0% > No. 4 5.5% < No. 200
46					48	4.25			
46.3–51.5			Olive to olive yellow **CLAY (CH)**, hard, moist						
50.5			*Grades green grey to blue grey*		48				88.0% < No. 200
51.5			Boring terminated at 51.5' Groundwater encountered at 6.5'						

Note: The boring log indicates subsurface conditions only at the specific location and time noted. Subsurface conditions, including groundwater levels, at other locations of the subject site may differ significantly from conditions which, in the opinion of Youngdahl Consulting Group, Inc., exist at the sampling locations. Note, too, that the passage of time may affect conditions at the sampling locations.

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October 2023

EXPLORATORY BORING LOG

Banner Self-Storage Facility
Sacramento California

FIGURE **A-3b**

Logged By: **ITK**	Date: **8 September 2023**	Lat / Lon: **N 38.492170° / W 121.517120°**	Boring No. **B-2**
Equipment: **CME 55 Drill Rig - 4" Solid Stem Auger / Mud Rotary**		Elevation: **~**	

Depth (Feet)	Ground Water	Geotechnical Description & Unified Soil Classification	Sample	Blow Counts	Pocket Pen (tsf)	Dry Density (pcf)	Moisture Content (%)	Tests & Comments
0–1		Olive brown sandy **CLAY (CL)**, medium stiff to stiff, slightly moist						**Bulk B-2** @ 0' - 5' Partial Recovery ϕ = 31.7°, c = 174 psf DDmax = 101.3 pcf MCopt = 17.8% R-Value = 27
3		*Grades moist*	2.5'–4'	14	4.5+			
5		*Grades soft*	5'–6.5'	6	3.5			
7	▽	*Grades very soft, wet*						Switch to Mud Rotary
10			10'–11.5'	0				No Recovery
16		*Grades olive grey, low plasticity*	15'–16.5'	2	<.25	90.8	34.1	
20			20'–21.5'	0				No Recovery Sand Catcher
25		Brown silty **SAND (SM)**, very loose, wet	25'–26.5'	2		93.9	29.3	
26		Olive grey sandy **CLAY (CL)**, very soft, wet						
26.5		Boring terminated at 26.5' Groundwater encountered at 7'						

Note: The boring log indicates subsurface conditions only at the specific location and time noted. Subsurface conditions, including groundwater levels, at other locations of the subject site may differ significantly from conditions which, in the opinion of Youngdahl Consulting Group, Inc., exist at the sampling locations. Note, too, that the passage of time may affect conditions at the sampling locations.

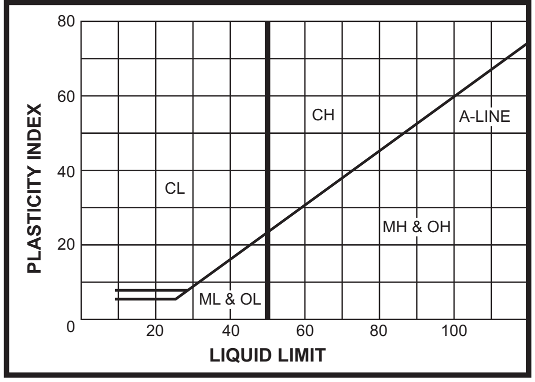
YOUNGDAHL CONSULTING GROUP, INC. ESTABLISHED 1984	Project No.: E23314.000 October 2023	**EXPLORATORY BORING LOG** **Banner Self-Storage Facility** Sacramento California	FIGURE **A-4**

UNIFIED SOIL CLASSIFICATION SYSTEMS

MAJOR DIVISION			SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS Over 50% > #200 sieve	GRAVELS Over 50% > #4 sieve	Clean **GRAVELS** With Little Or No Fines	GW	Well graded **GRAVELS**, **GRAVEL-SAND** mixtures
			GP	Poorly graded **GRAVELS**, **GRAVEL-SAND** mixtures
		GRAVELS With Over 12% Fines	GM	Silty **GRAVELS**, poorly graded **GRAVEL-SAND-SILT** mixtures
			GC	Clayey **GRAVELS**, poorly graded **GRAVEL-SAND-CLAY** mixtures
	SANDS Over 50% < #4 sieve	Clean **SANDS** With Little Or No Fines	SW	Well graded **SANDS**, gravelly **SANDS**
			SP	Poorly graded **SANDS**, gravelly **SANDS**
		SANDS With Over 12% Fines	SM	Silty **SANDS**, poorly graded **SAND-SILT** mixtures
			SC	Clayey **SANDS**, poorly graded **SAND-CLAY** mixtures
FINE GRAINED SOILS Over 50% < #200 sieve	**SILTS & CLAYS** Liquid Limit < 50		ML	Inorganic **SILTS**, silty or clayey fine **SANDS**, or clayey **SILTS** with plasticity
			CL	Inorganic **CLAYS** of low to medium plasticity, gravelly, sandy, or silty **CLAYS**, lean **CLAYS**
			OL	Organic **CLAYS** and organic silty **CLAYS** of low plasticity
	SILTS & CLAYS Liquid Limit > 50		MH	Inorganic **SILTS**, micaceous or diamacious fine sandy or silty soils, elastic **SILTS**
			CH	Inorganic **CLAYS** of high plasticity, fat **CLAYS**
			OH	Organic **CLAYS** of medium to high plasticity, organic **SILTS**
HIGHLY ORGANIC CLAYS			PT	PEAT & other highly organic soils

PLASTICITY CHART

USED FOR CLASSIFICATION OF FINE GRAINED SOILS



SAMPLE DRIVING RECORD

BLOWS PER FOOT	DESCRIPTION
25	25 Blows drove sampler 12 inches, after initial 6 inches of seating
50/7"	50 Blows drove sampler 7 inches, after initial 6 inches of seating
50/3"	50 Blows drove sampler 3 inches during or after initial 6 inches of seating

Note: To avoid damage to sampling tools, driving is limited to 50 blows per 6 inches during or after seating interval.

SOIL GRAIN SIZE

U.S. STANDARD SIEVE	6"		3"	¾"	4	10	40	200		
	BOULDER	COBBLE	GRAVEL		SAND			SILT	CLAY
			COARSE	FINE	COARSE	MEDIUM	FINE		
SOIL GRAIN SIZE IN MILLIMETERS	150		75	19	4.75	2.0	.425	0.075	0.002

KEY TO PIT & BORING SYMBOLS

- Standard Penetration test
- 2.5" O.D. Modified California Sampler
- 3" O.D. Modified California Sampler
- Shelby Tube Sampler
- 2.5" Hand Driven Liner
- Bulk Sample
- Water Level At Time Of Drilling
- Water Level After Time Of Drilling
- P Perched Water

KEY TO PIT & BORING SYMBOLS

	Joint
	Foliation
	Water Seepage
NFWE	No Free Water Encountered
FWE	Free Water Encountered
REF	Sampling Refusal
DD	Dry Density (pcf)
MC	Moisture Content (%)
LL	Liquid Limit
PI	Plasticity Index
PP	Pocket Penetrometer
UCC	Unconfined Compression (ASTM D2166)
TVS	Pocket Torvane Shear
EI	Expansion Index (ASTM D4829)
Su	Undrained Shear Strength

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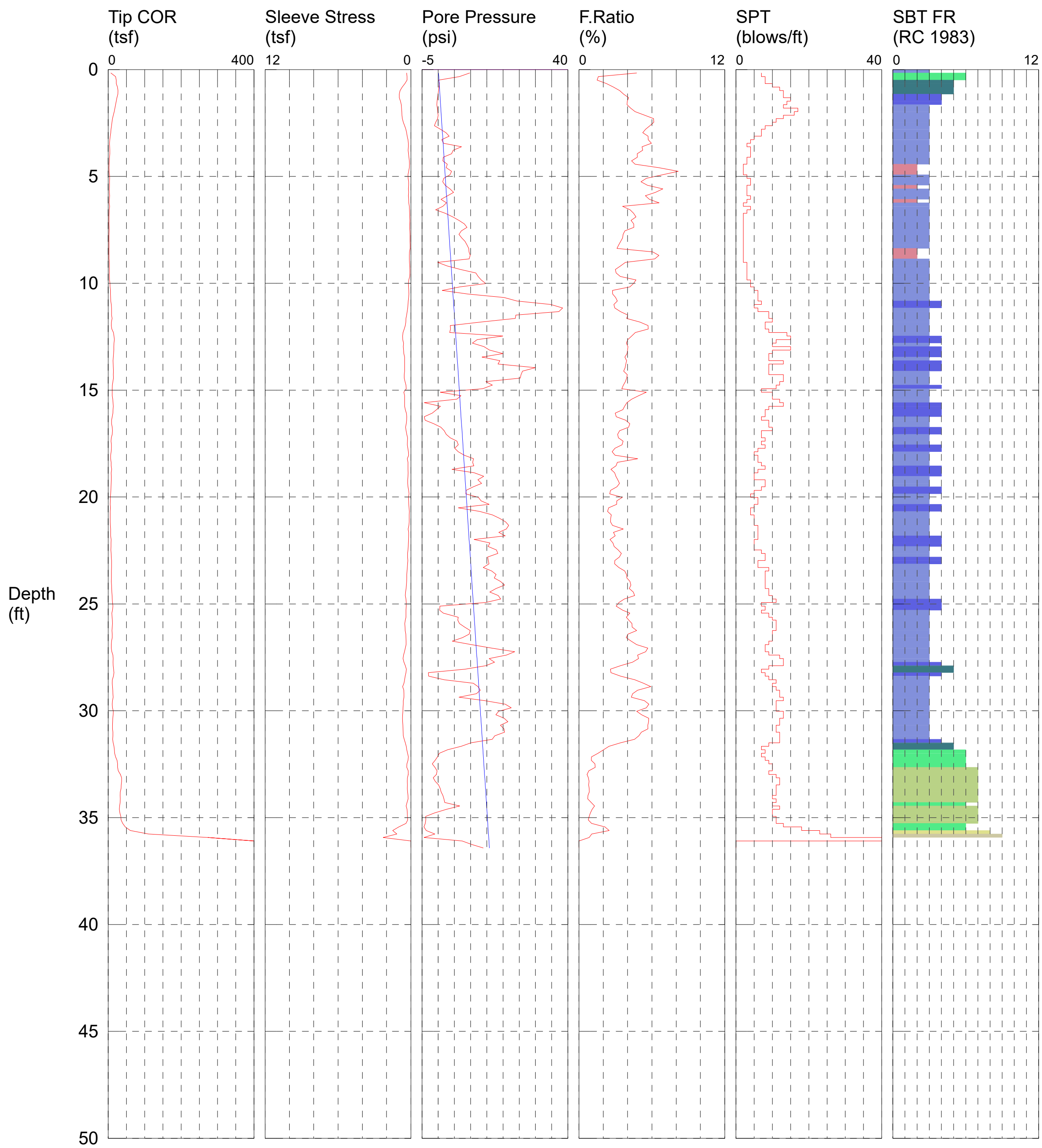
October 2023

SOIL CLASSIFICATION CHART AND LOG EXPLANATION
Banner Self-Storage Facility
Sacramento California

FIGURE A-5

SOUNDING

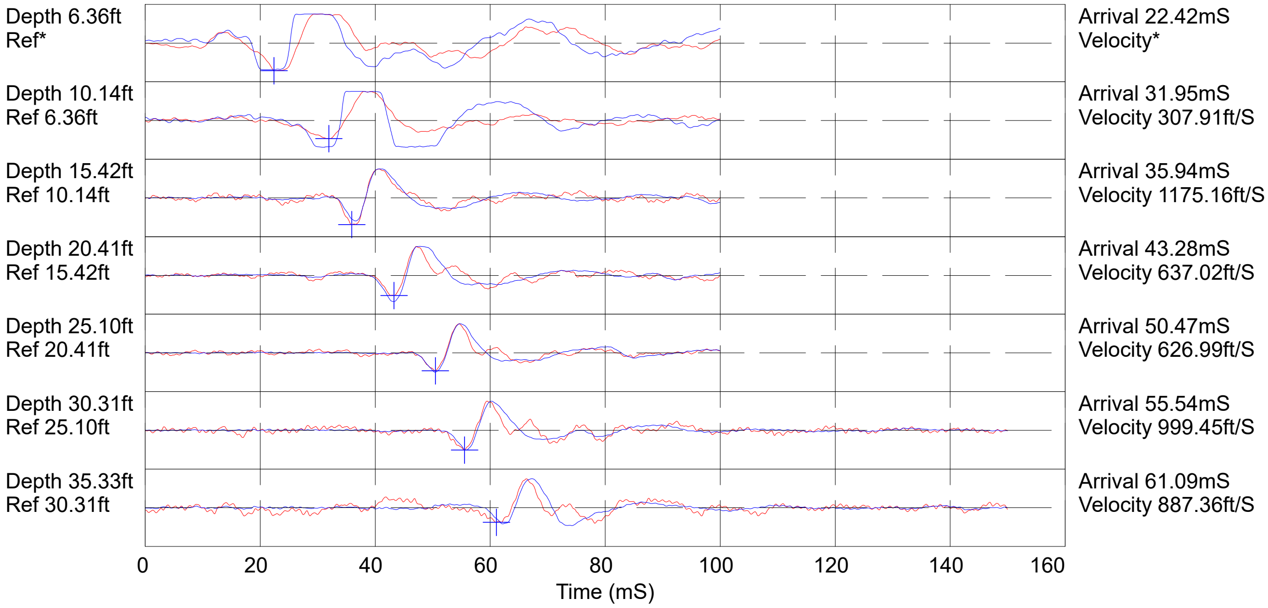
SOUNDING	JOB NUMBER:	COMMENT:
CUSTOMER: Taber Drilling	HOLE NUMBER: CPT-1	GPS (LAT,LON,ALT): 0.00,0.00,0.0
OPERATOR: David	TEST DATE: 9/8/2023 8:01:01 AM	LOCATION:
CONE ID: DDG1570	COMMENT: Auto Enhance On	LOCATION:
LOCATION:	COMMENT: Filter On	LOCATION:



1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

SEISMIC TEST



Hammer to Rod String Distance (ft): 6.56
* = Not Determined

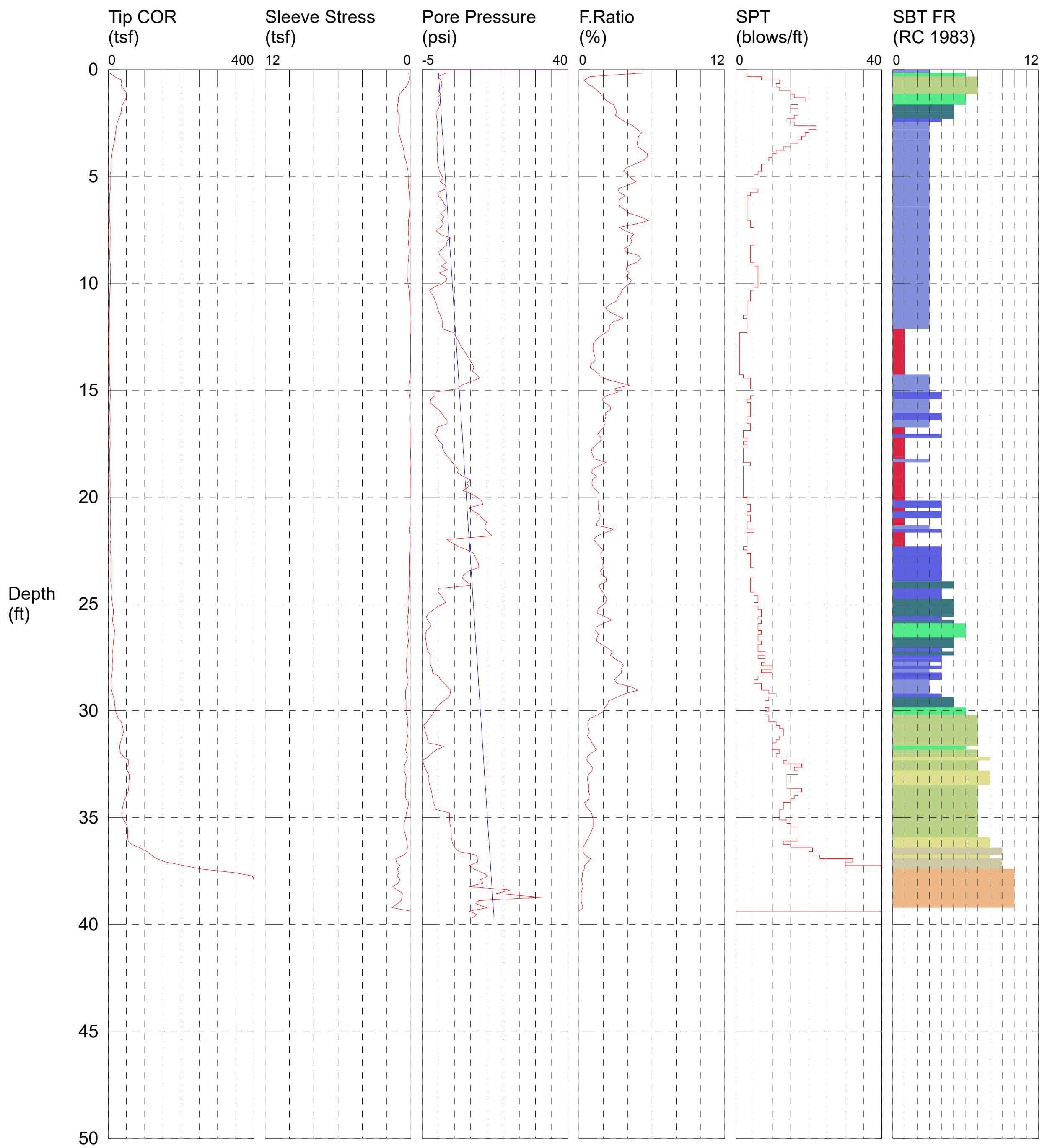
COMMENT:

SOUNDING

SOUNDING
CUSTOMER: Taber Drilling
OPERATOR: David
CONE ID: DDG1570
LOCATION:

JOB NUMBER:
HOLE NUMBER: CPT-2
TEST DATE: 9/8/2023 9:20:12 AM
COMMENT: Auto Enhance On
COMMENT: Filter On

COMMENT:
GPS (LAT,LON,ALT): 0.00,0.00,0.0
LOCATION:
LOCATION:
LOCATION:



1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy silt	10	gravelly sand to sand
2	organic material	5	clayey silt to silty clay	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey silt	9	sand	12	sand to clayey sand (*)

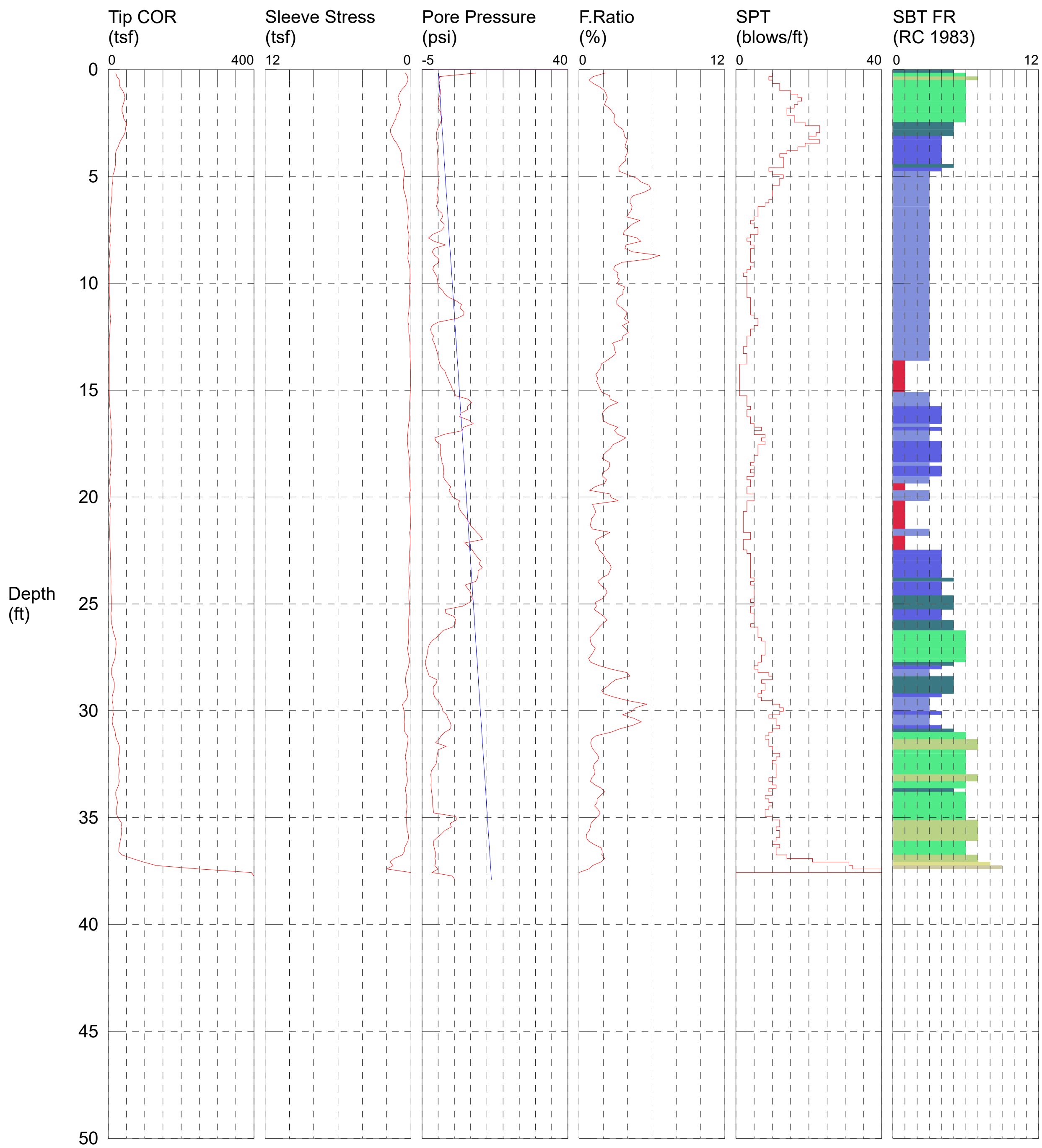
*SBT/SPT CORRELATION: UBC-1983

SOUNDING

SOUNDING
CUSTOMER: Taber Drilling
OPERATOR: David
CONE ID: DDG1570
LOCATION:

JOB NUMBER:
HOLE NUMBER: CPT-3
TEST DATE: 9/8/2023 10:31:38 AM
COMMENT: Auto Enhance On
COMMENT: Filter On

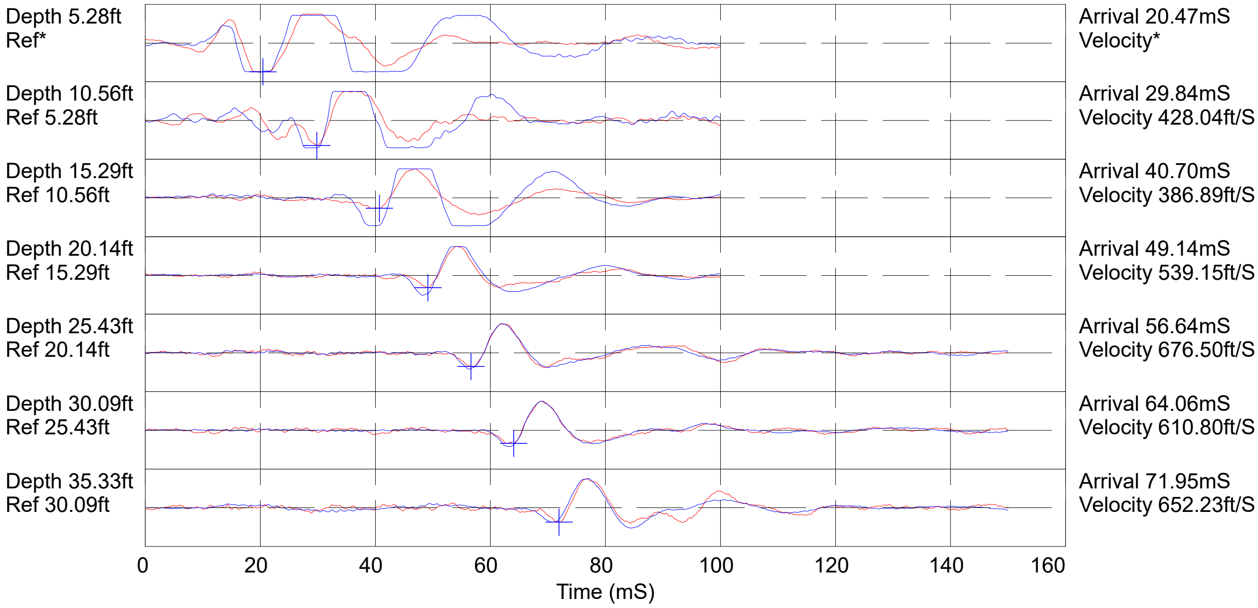
COMMENT:
GPS (LAT,LON,ALT): 0.00,0.00,0.0
LOCATION:
LOCATION:
LOCATION:



1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

*SBT/SPT CORRELATION: UBC-1983

SEISMIC TEST



Hammer to Rod String Distance (ft): 6.56
* = Not Determined

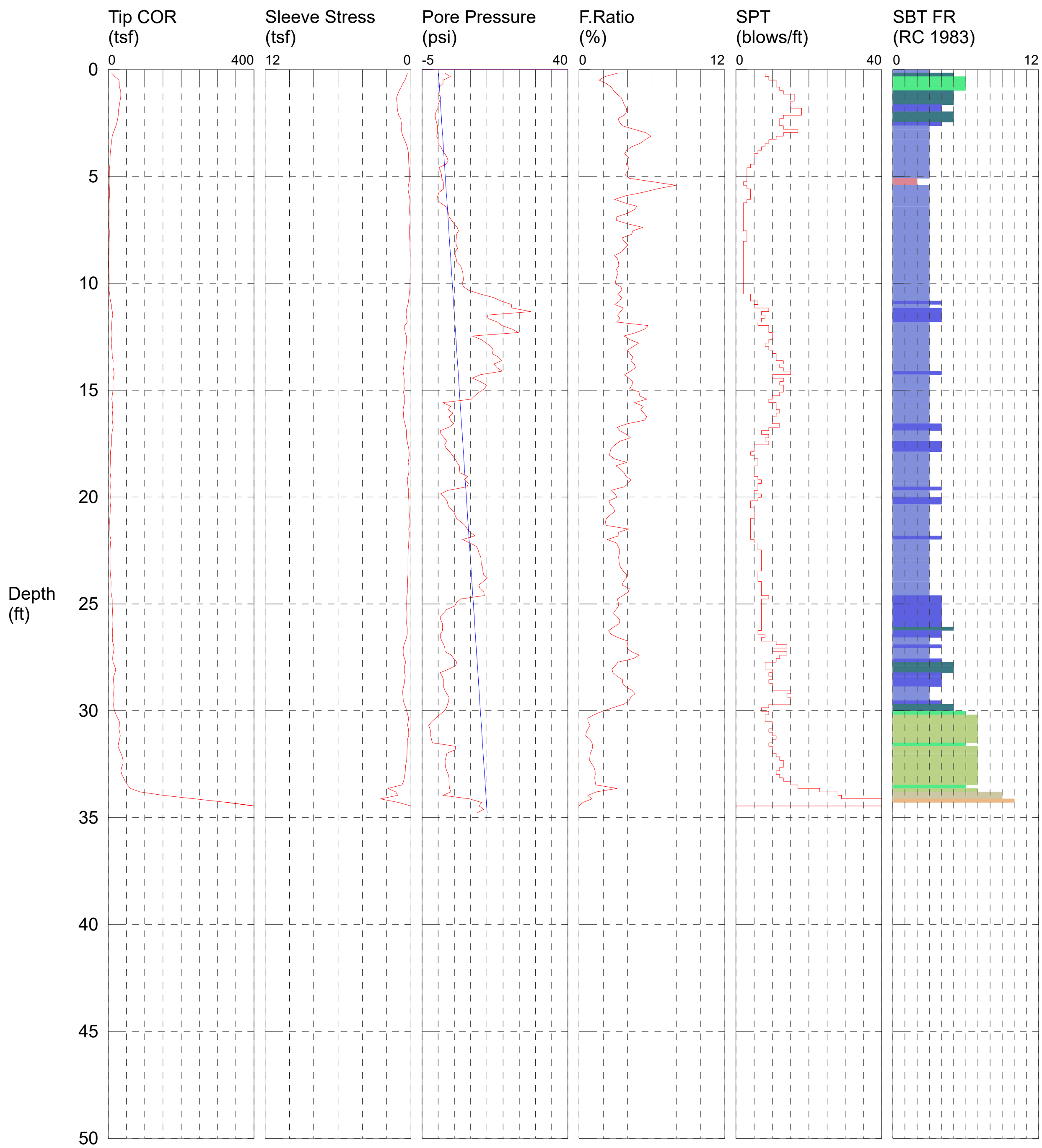
COMMENT:

SOUNDING

SOUNDING
CUSTOMER: Taber Drilling
OPERATOR: David
CONE ID: DDG1570
LOCATION:

JOB NUMBER:
HOLE NUMBER: CPT-4
TEST DATE: 9/8/2023 11:28:49 AM
COMMENT: Auto Enhance On
COMMENT: Filter On

COMMENT:
GPS (LAT,LON,ALT): 0.00,0.00,0.0
LOCATION:
LOCATION:
LOCATION:



1	sensitive fine grained	4	silty clay to clay	7	silty sand to sandy silt	10	gravelly sand to sand
2	organic material	5	clayey silt to silty clay	8	sand to silty sand	11	very stiff fine grained (*)
3	clay	6	sandy silt to clayey silt	9	sand	12	sand to clayey sand (*)

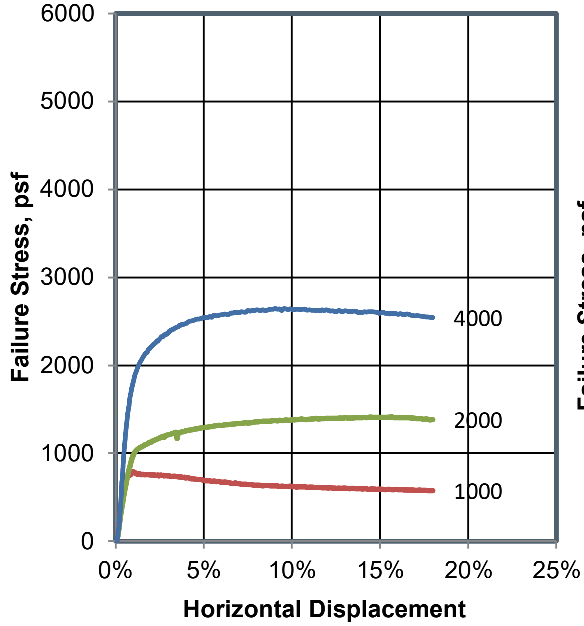
*SBT/SPT CORRELATION: UBC-1983

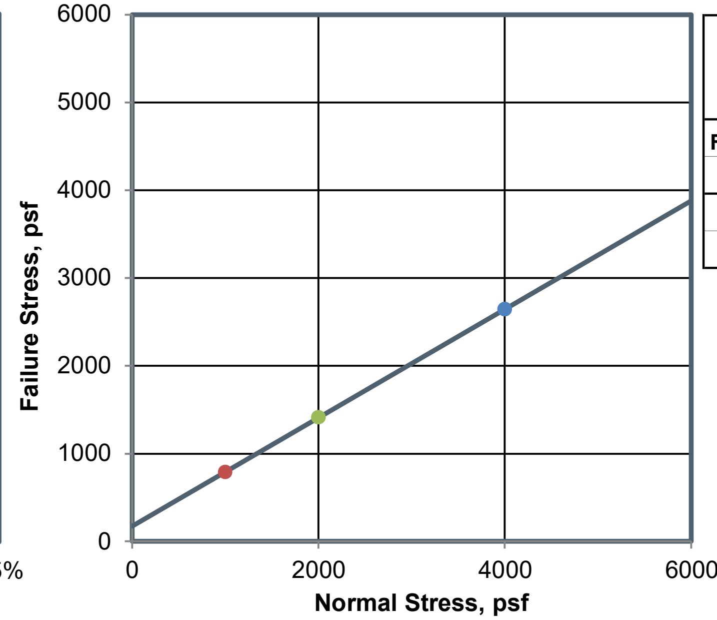
APPENDIX B

Laboratory Testing

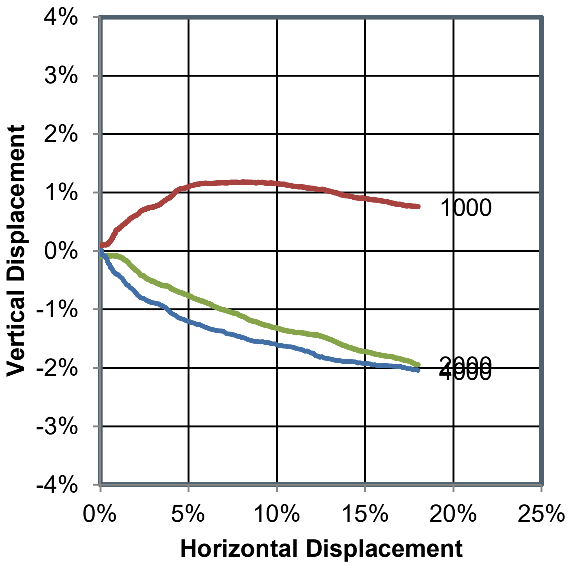
Direct Shear Test
Modified Proctor Test
Unconfined Compressive Strength Tests
No. 200 Wash Tests
Particle Size Analysis Tests
Atterberg Limits Tests
Corrosivity Tests

Direct Shear Test of Soils Under Consolidated Drained Conditions, ASTM D3080





Direct Shearbox Results
Friction Angle
31.7°
Cohesion
174 psf



	Test No.	1	2	3
Initial	Wet Density, pcf	107.4	107.4	107.4
	Dry Density, pcf	91.2	91.2	91.2
	Moisture Content, %	17.8	17.8	17.8
	Diameter, in	2.50	2.50	2.50
	Height, in	1.00	1.00	1.00
Pre Shear	Wet Density, pcf	121.6	122.5	121.4
	Dry Density, pcf	91.5	91.6	92.7
	Moisture Content, %*	32.9	33.8	30.9
	Diameter, in	2.50	2.50	2.50
	Height, in	1.00	1.00	0.98
Normal Stress, psf		1000	2000	4000
Failure Stress, psf		790	1415	2646
Failure Strain, %		0.96	15.68	9.07
Rate, in/min			0.001	

*Based on post shear moisture content

Sample Type: Remolded to 90% RC

Material Description: **Olive Brown Sandy CLAY**

Source: Curve 1

Notes: Gravel removed from test sample.

Sample No./Depth:	B-2 @ 0-5'			USCS Class.	Liquid Limit	Plasticity Index	% Greater than No. 4	% Less than No. 200
Date Sampled:	9/8/2023	Date Test Started:	9/15/2023				2	81.3

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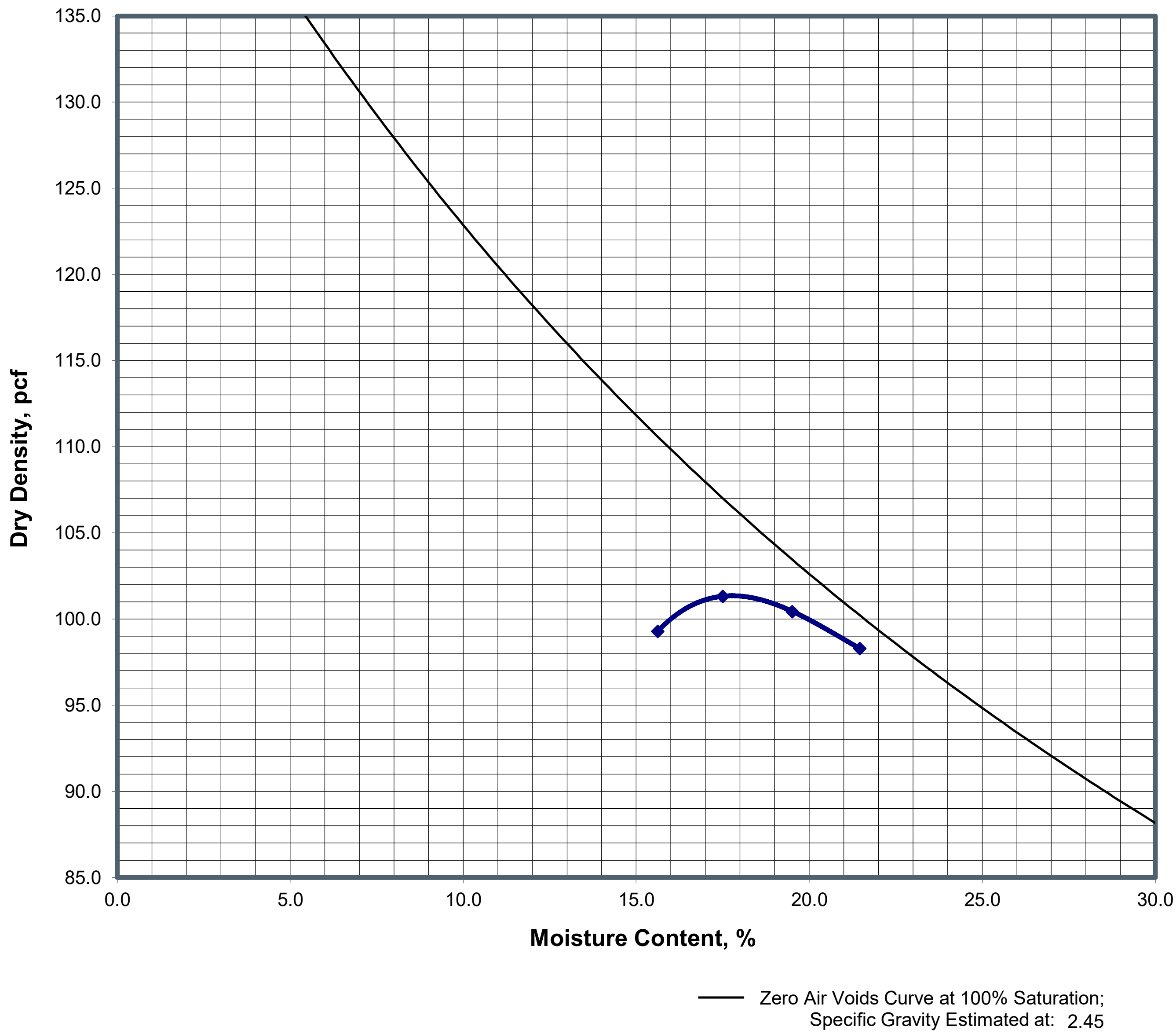
ph 916.933.0633 ▪ fx 916.933.6482 ▪ www.youngdahl.net

Project:	**Banner Self-Storage Facility GES**			
Project No.:	**E23314.000**			Figure B-1
Reviewed By:	DN	Date:	9/19/2023	

Laboratory Compaction Characteristics of Soil
Using Modified Effort (56,000 lf-lbf/ft3), ASTM D1557, Method A



Maximum Dry Density, pcf:	**101.3**	Optimum Moisture Content, %:	**17.8**

Material Description: **Olive Brown Sandy CLAY**

Source: B-2 @ 0-5'

Notes:

Sample No./Depth:	Curve 1			USCS Class.	Liquid Limit	Plasticity Index	% Greater than No. 4 :	% Less than No. 200
Date Sampled:	9/8/2023	Date Test Started:	9/14/2023				2	81.3

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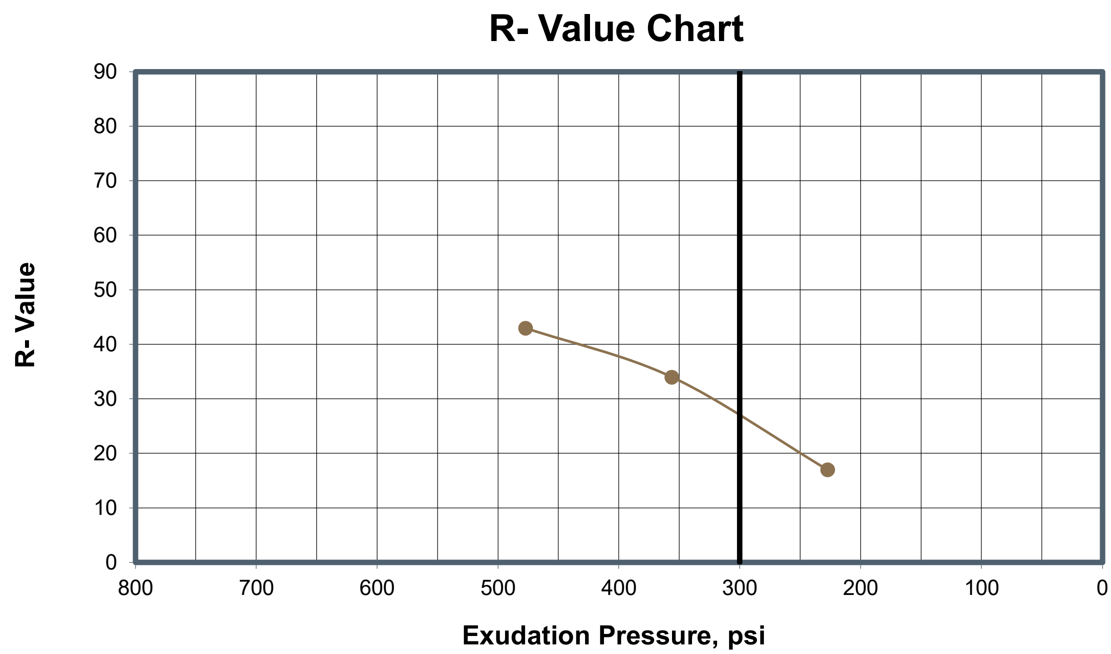
Project:	**Banner Self-Storage Facility**		
Project No.:	**E23314.000**		Figure
Reviewed By: BLCC	Date: 9/15/2023		B-2

Resistance "R" Value of Soil and Soil-Aggregate Mixtures, CTM 301



Test Specimen No.:	1	2	3
Moisture Content at Test, %	19.5	21.7	22.8
Dry Density at Test, pcf	105.7	103.1	101.5
Expansion Pressure, psf	1173	450	398
Exudation Pressure, psi	477	356	227
Resistance "R" Value	43	34	17
"R" Value at 300 psi Exudation Pressure			**27**

Material Description: **Olive Brown Sandy CLAY**

Source: B-2 @ 0-5'

Notes:

Sample No./Depth:	RV-1			USCS Class.	Liquid Limit	Plasticity Index	% Greater than No. 4	% Less than No. 200
Date Sampled:	9/8/2023	Date Test Started:	9/18/2023				2	81.3

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Project:	**Banner Self-Storage Facility GES**			Figure
Project No.:	**E23314.000**			
Reviewed By:	JLC	Date:	9/19/2023	B-3

Unconfined Compressive Strength of Cohesive Soil, ASTM D2166

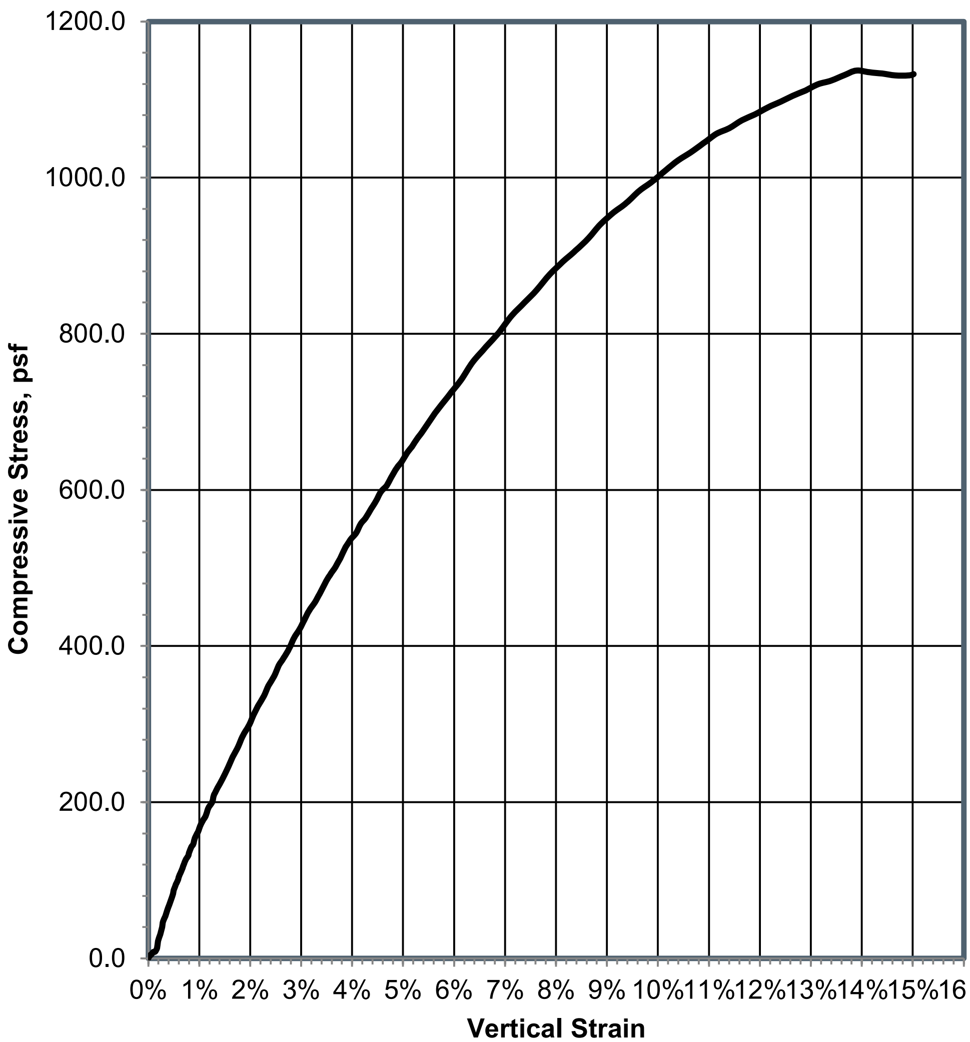
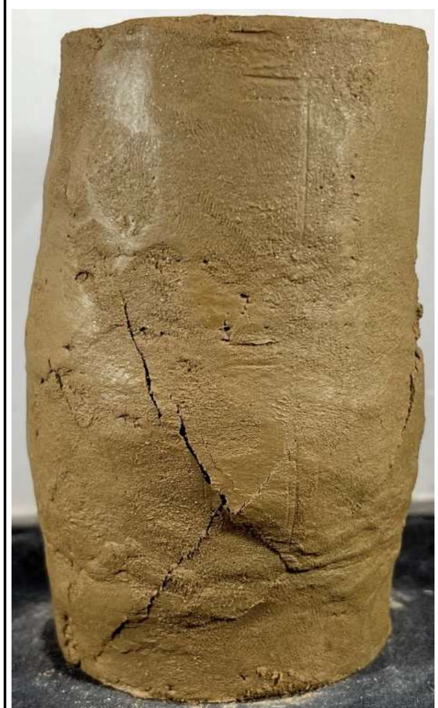


Image of Failed Specimen



Unconfined Compression Results	Compression Strength	1137.5 psf	**Specimen Parameters**	Wet Density, pcf	126.5	Diameter, in	2.39
	Shear Strength	568.75 psf		Dry Density, pcf	96.9	Height/Diameter	2.3
	Failure Strain, %	13.9 %		Moisture Content, %	30.6	Strain Rate, %/min	2.0
				Saturation, %	Not Evaluated	Sensitivity:	Not Evaluated
				Void Ratio	80.5	Specimen Type:	Insitu
				Height, in	5.48		

Material Description: **Brown Sandy Lean CLAY**

Source:

Notes: *Moisture content based on after test sample.

Sample No./Depth:	B-1 @ 26-26.5'			USCS Class.	Liquid Limit	Plasticity Index	% Greater than No. 4	% Less than No. 200
Date Sampled:	9/8/2023	Date Test Started:	9/20/2023					

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Project:	**Banner Self-Storage Facility GES**		
Project No.:	**E23314.000**		Figure
Reviewed By: DN	Date: 9/21/2023		B-4

Unconfined Compressive Strength of Cohesive Soil, ASTM D2166

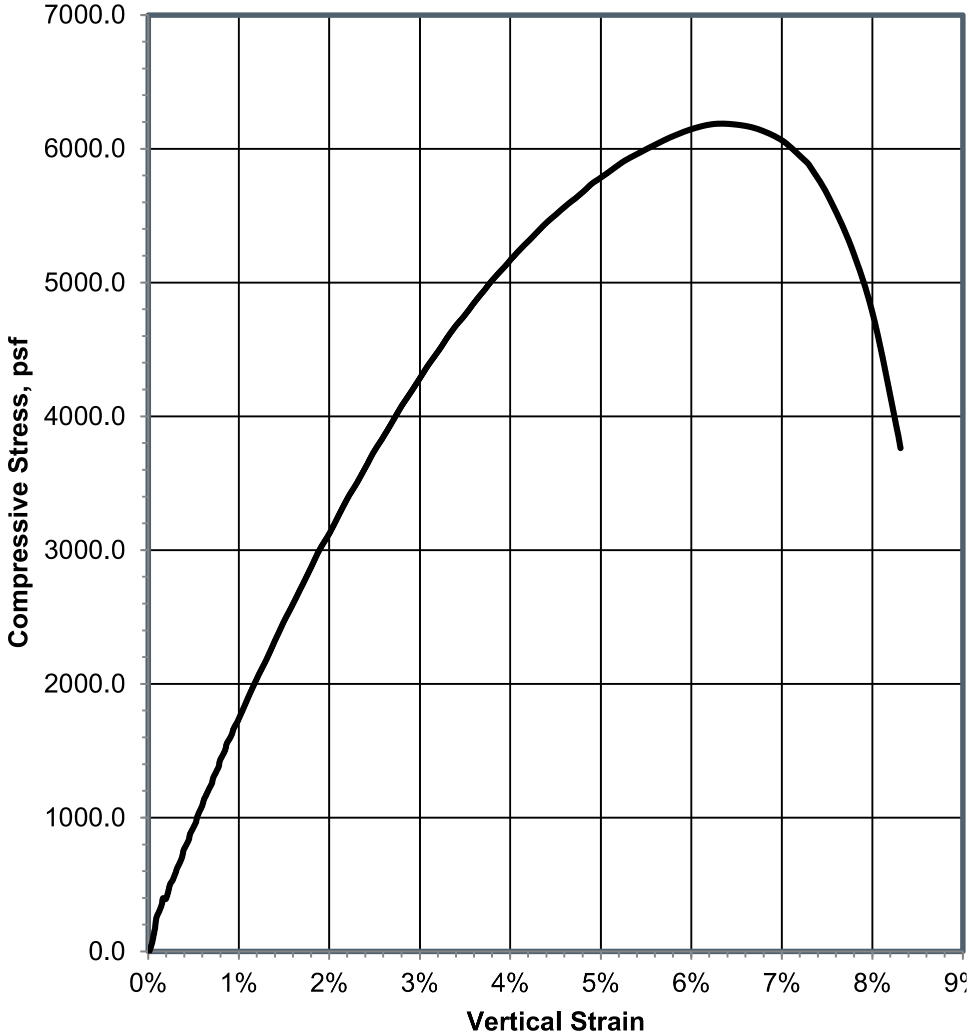

Image of Failed Specimen

Unconfined Compression Results	Compression Strength	6185.5 psf	**Specimen Parameters**	Wet Density, pcf	127.4	Diameter, in	2.40
				Dry Density, pcf	103.7	Height/Diameter	2.4
	Shear Strength	3092.75 psf		Moisture Content, %	22.8	Strain Rate, %/min	1.5
				Saturation, %	Not Evaluated	Sensitivity:	Not Evaluated
	Failure Strain, %	6.3 %		Void Ratio	75.7	Specimen Type:	Insitu
				Height, in	5.81		

Material Description: **Green Gray Lean CLAY with Sand**

Source:

Notes: *Moisture content based on after test sample.

Sample No./Depth:	B-1 @ 51-51.5'			USCS Class.	Liquid Limit	Plasticity Index	% Greater than No. 4	% Less than No. 200
Date Sampled: 9/8/2023		Date Test Started:	9/19/2023					

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Project:	**Banner Self-Storage Facility GES**		
Project No.:	**E23314.000**		Figure
Reviewed By: DN	Date: 9/21/2023		B-5

Amount of Material Finer than No. 200 (75-µm) Sieve in Soils by Washing, ASTM D1140, Method A

Sample No.	Depth	Sample Description	Material Finer than No. 200 Sieve, %
B-1	20.5-21'	Olive Gray SILT with Sand	72.5
B-1	50.5-51'	Green Gray Grading to Blue Gray Fat CLAY	88.0

Notes:

Date Sampled:	9/8/2023	Date Test Started:	9/14/2023

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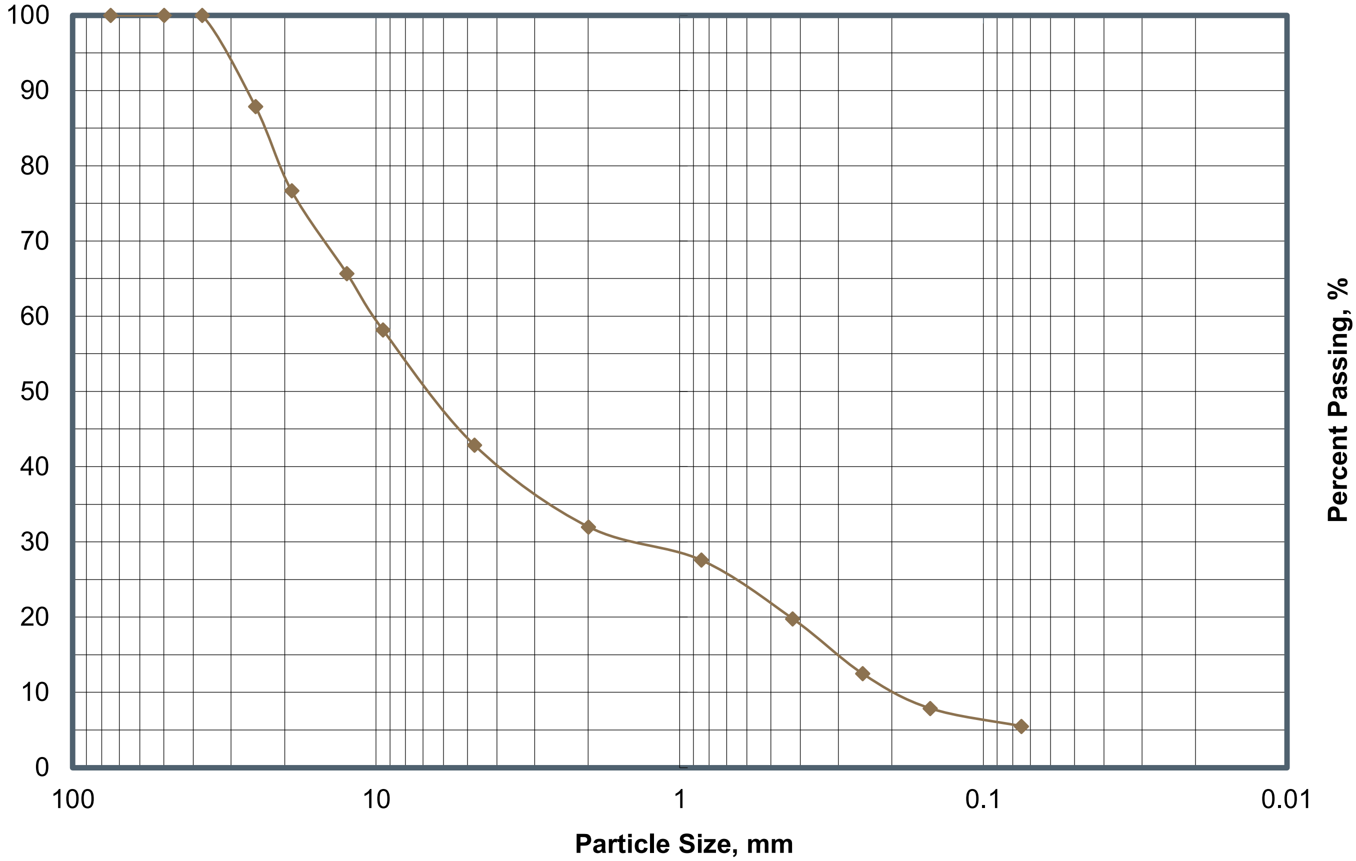
Project:	**Banner Self-Storage Facility**		
Project No.:	**E23314.000**		Figure
Reviewed By: DN		Date: 9/15/2023	B-6

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis, ASTM D6913, Method A



U.S. Standard Sieve Size	Combined % Passing	U.S. Standard Sieve Size	Combined % Passing
3 Inch (75 mm)	100	No. 4 (4.75 mm)	43
2 Inch (50 mm)	100	No. 10 (2 mm)	32
1 1/2 Inch (37.5 mm)	100	No. 20 (850 μm)	28
1 Inch (25 mm)	88	No. 40 (425 μm)	20
3/4 Inch (19 mm)	77	No. 60 (250 μm)	12
1/2 Inch (12.5 mm)	66	No.100 (150 μm)	8
3/8 Inch (9.5 mm)	58	No. 200 (75 μm)	5.5

Material Description: **Olive Gray Sandy GRAVEL**

Source:

Notes:

Sample No./Depth:	B-1 @ 40.5-41'			USCS Class.	Liquid Limit	Plasticity Index	% Great than No. 4	% Less than No. 200
Date Sample:	9/8/2023	Date Test Started:	9/15/2023				57	5.5

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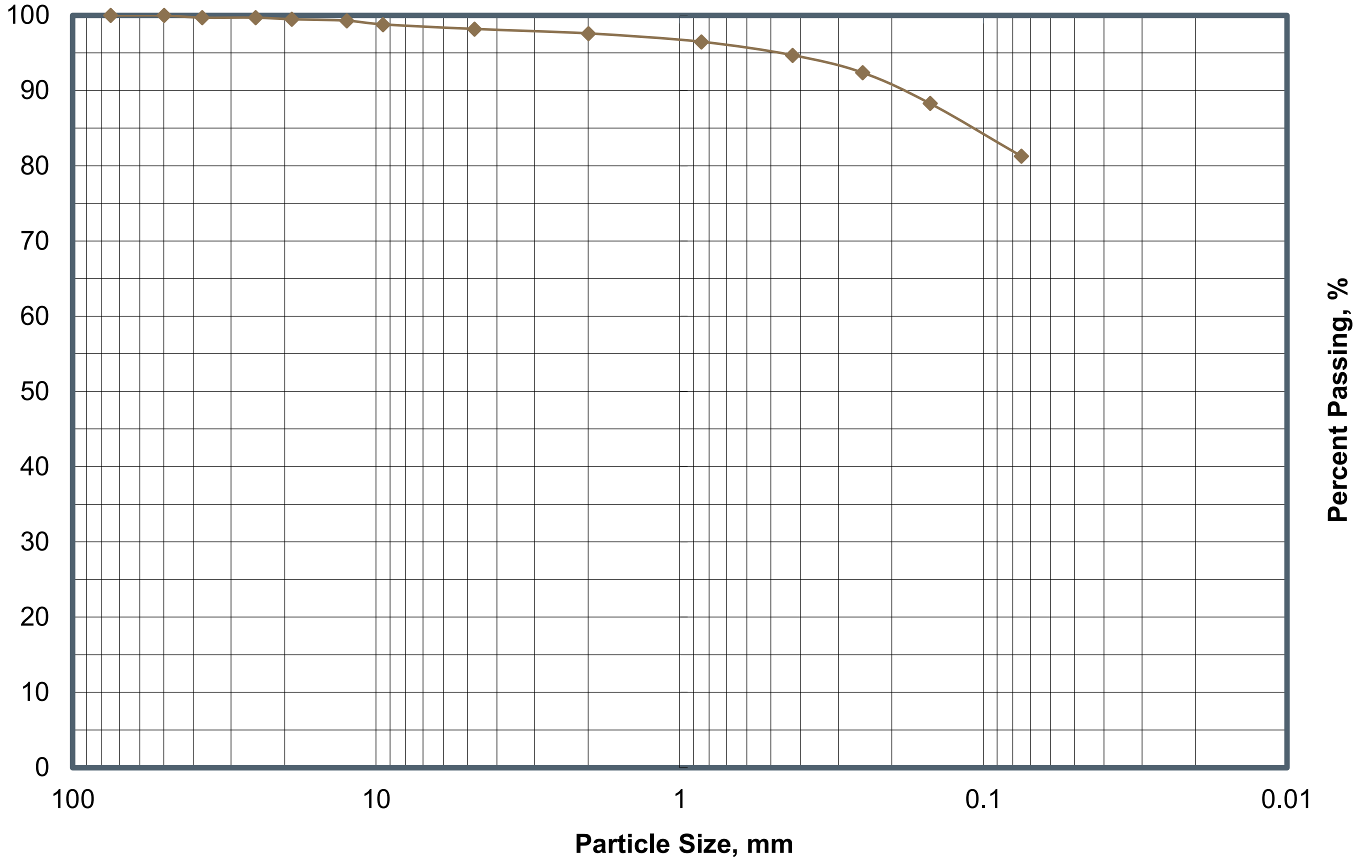
Project:	**Banner Self-Storage Facility**		
Project No.:	**E23314.000**		Figure
Reviewed By: DN	Date:	9/15/2023	B-7

Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis, ASTM D6913, Method A



U.S. Standard Sieve Size	Combined % Passing	U.S. Standard Sieve Size	Combined % Passing
3 Inch (75 mm)	100	No. 4 (4.75 mm)	98
2 Inch (50 mm)	100	No. 10 (2 mm)	98
1 1/2 Inch (37.5 mm)	100	No. 20 (850 μm)	96
1 Inch (25 mm)	100	No. 40 (425 μm)	95
3/4 Inch (19 mm)	100	No. 60 (250 μm)	92
1/2 Inch (12.5 mm)	99	No.100 (150 μm)	88
3/8 Inch (9.5 mm)	99	No. 200 (75 μm)	81.3

Material Description: **Olive Brown Sandy CLAY**

Source:

Notes:

Sample No./Depth:	B-2 @ 0-5'			USCS Class.	Liquid Limit	Plasticity Index	% Great than No. 4	% Less than No. 200
Date Sample:	9/8/2023	Date Test Started:	9/13/2023				2	81.3

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Project: **Banner Self-Storage Facility**

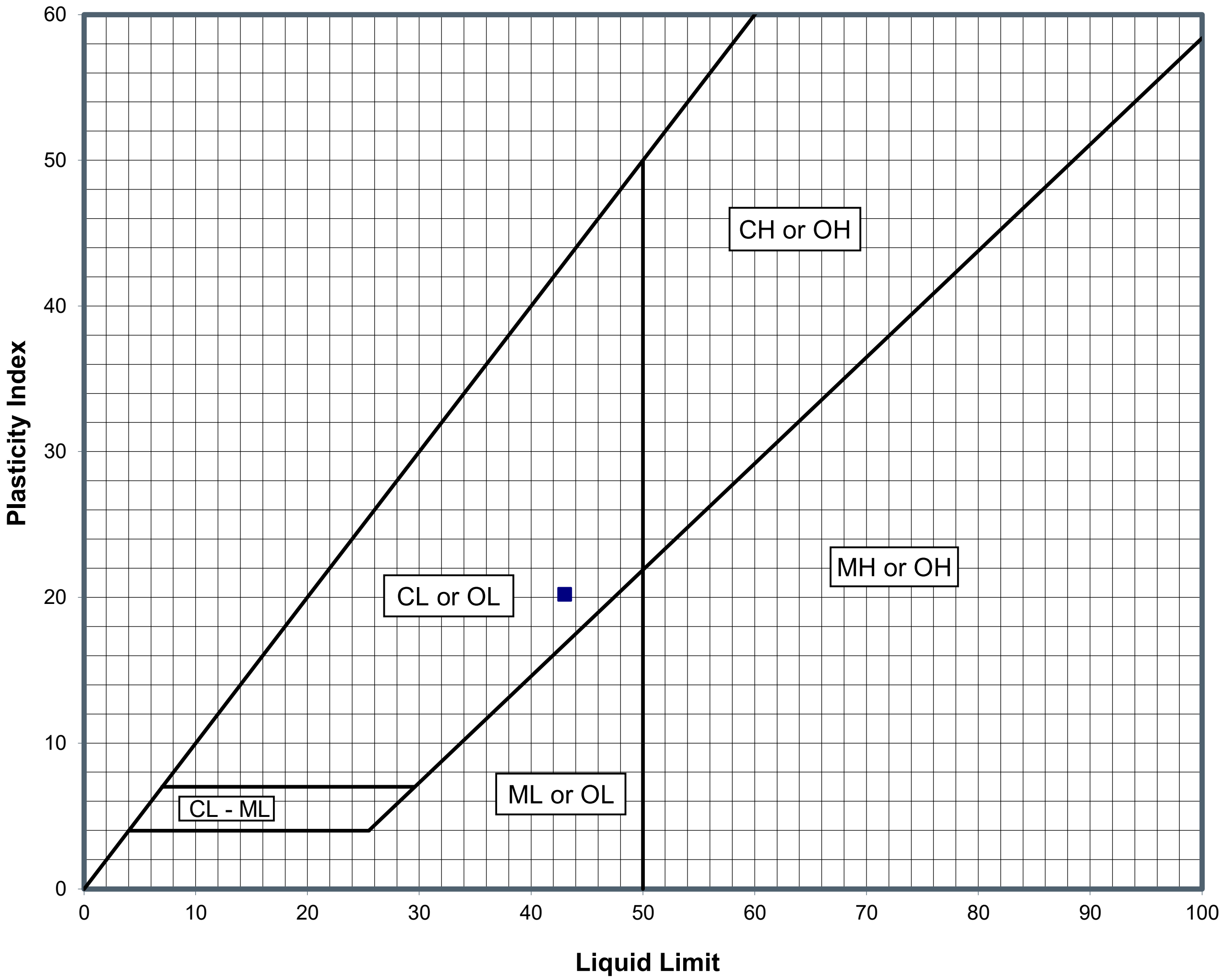
Project No.: **E23314.000**

Reviewed By: DN

Date: 9/15/2023

Figure B-8

Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D4318, Method A



Liquid Limit	Plastic Limit	Plasticity Index	Unified Soil Classification, ASTM D2487
43	23	20	CL

Material Description: **Gray Lean CLAY with Sand**

Source:

Notes:

				USCS Class.	Liquid Limit	Plasticity Index	% Greater than No. 4	% Less than No. 200
Sample No./Depth:	B-1 @ 10.5-11'			CL	43	20	0	78.7
Date Sampled:	9/8/2023	Date Test Started:	9/18/2023					

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Project: **Banner Self-Storage Facility GES**

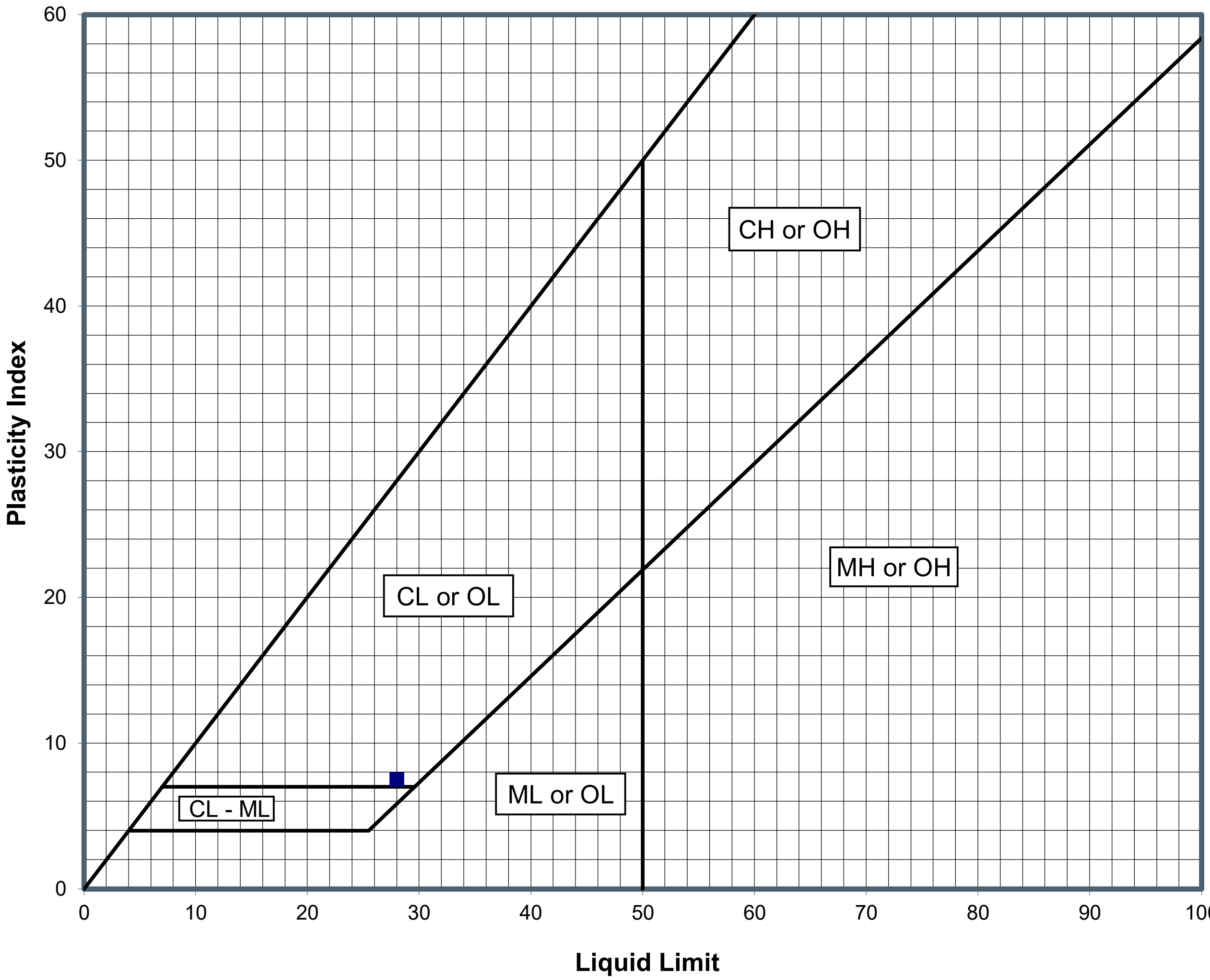
Project No.: **E23314.000**

Reviewed By: DN

Date: 9/19/2023

Figure B-9

Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D4318, Method A



Liquid Limit	Plastic Limit	Plasticity Index	Unified Soil Classification, ASTM D2487
28	21	8	CL

Material Description:	**Brown Lean CLAY with Sand**						
Source:							
Notes:							
Sample No./Depth:	B-1 @ 30.5-31'			USCS Class.	Liquid Limit	Plasticity Index	% Greater than No. 4	% Less than No. 200
Date Sampled:	9/8/2023	Date Test Started:	9/18/2023	CL	28	8	0	83.4

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Project:	**Banner Self-Storage Facility GES**		
Project No.:	**E23314.000**		Figure
Reviewed By: DN	Date: 9/21/2023		B-10

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/20/2023
Date Submitted 09/13/2023

To: Jeffry Cannon
Youngdahl Consulting Group
1234 Glenhaven Ct.
El Dorado Hills, CA 95630

From: Gene Oliphant, Ph.D. \ Randy Horney RH
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : E23314.000 Site ID : B-1 @ 10-10.5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 90574-187927.

EVALUATION FOR SOIL CORROSION

Soil pH	7.92		
Minimum Resistivity	1.07	ohm-cm (x1000)	
Chloride	48.2 ppm	00.00482	%
Sulfate	27.4 ppm	00.00274	%

METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

Sunland Analytical
11419 Sunrise Gold Circle, #10
Rancho Cordova, CA 95742
(916) 852-8557

Date Reported 09/20/2023
Date Submitted 09/13/2023

To: Jeffry Cannon
Youngdahl Consulting Group
1234 Glenhaven Ct.
El Dorado Hills, CA 95630

From: Gene Oliphant, Ph.D. \ Randy Horney
General Manager \ Lab Manager

The reported analysis was requested for the following location:
Location : E23314.000 Site ID : B-2 @ 3-3.5FT.
Thank you for your business.

* For future reference to this analysis please use SUN # 90574-187928.

EVALUATION FOR SOIL CORROSION

Soil pH	7.36		
Minimum Resistivity	1.69 ohm-cm (x1000)		
Chloride	14.8 ppm	00.00148	%
Sulfate	13.8 ppm	00.00138	%

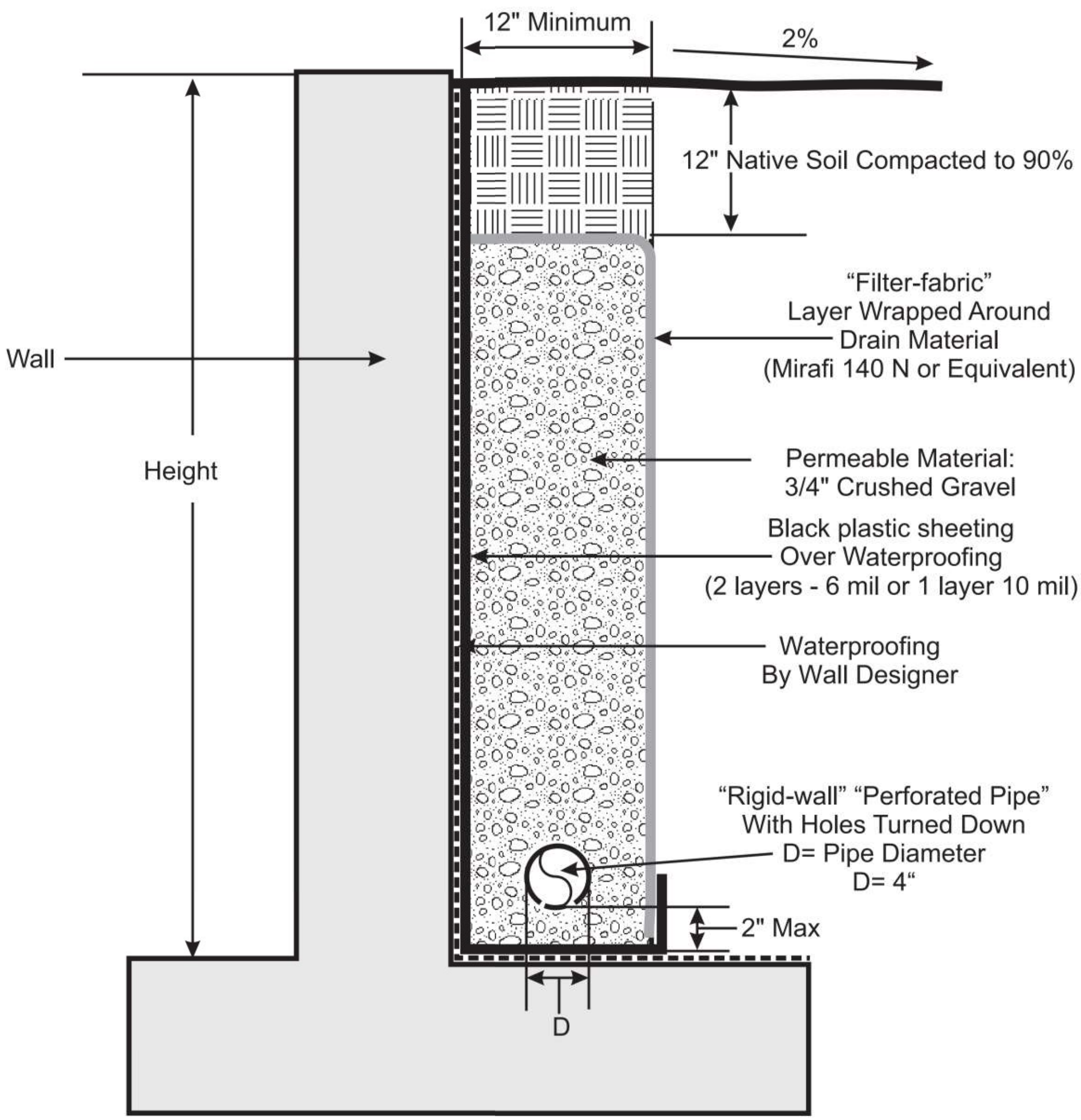
METHODS
pH and Min.Resistivity CA DOT Test #643
Sulfate CA DOT Test #417, Chloride CA DOT Test #422m

APPENDIX C
Details

Site Wall Drainage

Retaining Wall With "Perforated Pipe Sub-Drain"

(Typical Cross Section)



Notes:
1. Slope footing and "rigid-wall" pipes along flow line parallel to wall at least 1% gradient to drain to an appropriate outfall area away from residence.
2. Use "sweeps" for directional changes in pipe flow **(do not use 90°elbows)**.
3. Provide periodic "clean-outs".
4. Washed clean permeable material.

Not To Scale

YOUNGDAHL CONSULTING GROUP, INC. GEOTECHNICAL • ENVIRONMENTAL • MATERIALS TESTING	Project No.: E23314.000	**RETAINING WALL DRAIN DETAIL**	**FIGURE**
	October 2023	**Banner Self-Storage Facility** Sacramento, California	**C-1**