

Geotechnical Investigation

E
APPENDIX



GEOTECHNICAL INVESTIGATION



AL SHUGART DOG PARK
SCOTTS VALLEY, CALIFORNIA

FOR
VERDE DESIGN INC.
SANTA CLARA, CALIFORNIA



CONSULTING GEOTECHNICAL ENGINEERS

25116-SZ42-C17
DECEMBER 2025
www.4pacific-crest.com

December 23, 2025

Project No. 25116-SZ42-C17

Mr. Rico Lardizabal
Verde Design Inc
2455 The Alameda
Santa Clara, CA 95050

Subject: **Geotechnical Investigation – Design Phase**
Al Shugart Dog Park
Glenwood Drive
Scotts Valley, California

Dear Mr. Lardizabal,

In accordance with your authorization, we have performed a geotechnical investigation for the proposed improvements at Al Shugart Park in Scotts Valley, California.

The accompanying report presents our conclusions and recommendations as well as the results of the geotechnical investigation on which they are based. The conclusions and recommendations presented in this report are contingent upon our review of the plans during the design phase of the project, and our observation and testing during the construction phase of the project.

Very truly yours,

PACIFIC CREST ENGINEERING INC.

Prepared by:



Ben Andrews, PE
Senior Engineer
PE 97525
Expires 6/30/27

Reviewed by:



Chris Johnson, PE
Principal Civil Engineer
PE 82630
Expires 9/30/26

TABLE OF CONTENTS

I.	INTRODUCTION	1
	PURPOSE AND SCOPE	1
	PROJECT LOCATION	2
	PROPOSED IMPROVEMENTS	2
	PREVIOUS INVESTIGATIONS	2
II.	INVESTIGATION METHODS	2
	FIELD INVESTIGATION	2
	INFILTRATION TESTING	3
	LABORATORY TESTING	4
III.	FINDINGS AND ANALYSIS	4
	GEOLOGIC SETTING	4
	SURFACE CONDITIONS	4
	SUBSURFACE SOIL CONDITIONS	5
	GROUNDWATER CONDITIONS	5
	STORM WATER INFILTRATION	6
	FAULTING AND SEISMICITY	6
	GEOTECHNICAL HAZARDS	7
IV.	DISCUSSION AND CONCLUSIONS	9
	GENERAL	9
	PRIMARY GEOTECHNICAL CONSIDERATIONS	9
V.	RECOMMENDATIONS	10
	EARTHWORK	10
	FOUNDATIONS – SPREAD FOOTINGS	15
	FLATWORK	16
	PAVEMENT DESIGN	16
	SURFACE DRAINAGE	18
	STORM WATER INFILTRATION	19
	EROSION CONTROL	20
	PLAN REVIEW	20
VI.	LIMITATIONS AND UNIFORMITY OF CONDITIONS	21
	APPENDIX A – FIGURES, BORING LOGS AND LABORATORY RESULTS	22
	APPENDIX B – BORINGS LOGS FROM PREVIOUS INVESTIGATIONS	35
	APPENDIX C – INFILTRATION TEST RESULTS	42
	APPENDIX D – LIQUEFACTION RESULTS	46

GEOTECHNICAL INVESTIGATION REPORT AL SHUGART DOG PARK IMPROVEMENTS SCOTTS VALLEY, CALIFORNIA

I. INTRODUCTION

PURPOSE AND SCOPE

The City of Scotts Valley looks to improve the Al Shugart Park area to include facilities common to a dog park, including but not limited to new benches and pathways, a restroom, a plaza shade structure, minor regrading and new landscaping. This report describes the geotechnical investigation and presents our findings, conclusions, and recommendations for the proposed improvements. For purposes of this report, "site" refers to Al Shugart Park.

Our scope of services for this project has consisted of:

1. Site reconnaissance to observe the existing conditions.
2. Review of the following published maps:
 - Geologic Map of Santa Cruz County, California, Brabb, 1997.
 - Preliminary Map of Landslide Deposits in Santa Cruz County, California, Cooper-Clark and Associates, 1975.
 - Map Showing Geology and Liquefaction Potential of Quaternary Deposits in Santa Cruz County, California, Dupré, 1975.
 - U.S. Geological Survey (and the California Geologic Survey), 2018, Quaternary fault and fold database for the United States, accessed January 2020, from USGS web site: <http://earthquake.usgs.gov/hazards/qfaults/>.
 - Geographic Information System - Santa Cruz County, "GISWEB Interactive Mapping Application" <http://gis.co.santa-cruz.ca.us/internet/wwwgisweb/viewer.htm>
3. The drilling and logging of five (5) exploratory borings.
4. Infiltration testing of three (3) test holes in accordance with the Central Coast Low Impact Development Initiative, with procedures outlined in the report titled "Native Soil Assessment For Small Infiltration-Based Storm Water Control Measures". Our infiltration study followed the "Shallow Quick Infiltration Test" method, as described within Attachment 1 of that document.
5. Laboratory analysis of retrieved soil samples.
6. Engineering analysis of the field and laboratory test results.
7. Review of preliminary plans and sections showing the locations of the proposed improvements.
8. Preparation of this report documenting our investigation and presenting geotechnical recommendations for the design and construction of the project.



PROJECT LOCATION

Al Shugart Park is surrounded by rural residential neighborhoods on the northern side of Scotts Valley, with large open spaces of the East Glenwood Preserve to the northeast. The site is currently undeveloped with the exception of walking trails/bridges, drainage improvements, and an unpaved parking lot. Riparian growth within the drainage areas consisting of a dense growth of willow trees and native brush intersect the generally meadow like landscape. Please refer to Figure No. 1, Regional Site Map, for the general vicinity of the site. The site is approximately located at the following coordinates:

Latitude = 37.069289 degrees
Longitude = -122.003950 degrees

PROPOSED IMPROVEMENTS

Based on our review of preliminary plans and discussions with the City of Scotts Valley and Verde Design personnel, it is our understanding that the primary objective of the project is to improve the Al Shugart Park area with two separate off leash spaces for large and small dogs, pet proof fencing and gates to cordon off the two areas, a perimeter walking path, and new site amenities such as; a shade structure, picnic tables, benches, lighting, waste bins, bike racks, water fountains, and a restroom structure.

PREVIOUS INVESTIGATIONS

Pacific Crest Engineering, Inc. has completed a previous subsurface exploration at the proposed site for the pedestrian pathway and foot bridges, as denoted below:

- Pacific Crest Engineering, Inc. *Geotechnical Investigation for Shugart Park Pedestrian Pathway and Foot Bridges*, Project No. 1293-SZ42-C16

The above referenced geotechnical investigation included the advancement of four (4) exploratory borings along the northern limit of the park. Please refer to Figure No. 2, Appendix A of this report for the general locations of these borings. The individual boring logs from the referenced investigations are included within Appendix B of this report.

II. INVESTIGATION METHODS

FIELD INVESTIGATION

Two (2) exploratory borings and three (3) infiltration test borings were drilled at the site on November 3rd, 2025. The approximate locations of the test borings are shown on Figure No. 2, in Appendix A. The drilling method used was hydraulically operated continuous flight augers on a track mounted drill rig. An engineer from Pacific Crest Engineering Inc. was present during the drilling operations to log the soil encountered and to choose sampler type and locations.

Relatively undisturbed soil samples were obtained at various depths by driving a split spoon sampler 18 inches into the ground. This was achieved by dropping a 140-pound hammer a vertical height of 30 inches. The drill rig was equipped with an automatic trip hammer. The number of blows required to drive the sampler each 6-inch increment and the total number of blows required to drive the last 12 inches was recorded by the field engineer/geologist. The outside diameter of the samplers used was 3-inch or 2-inch and is designated on the Boring Logs as "L" or "T", respectively.



The field blow counts in 6-inch increments are reported on the Boring Logs adjacent to each sample as well as the Standard Penetration Test data (SPT). All SPT data has been normalized to a 2-inch O.D. sampler and is reported on the Boring Logs as SPT "N" values. The normalization method used was derived from the second edition of the Foundation Engineering Handbook (H.Y. Fang, 1991). The method utilizes a Sampler Hammer Ratio which is dependent on the weight of the hammer, height of hammer drop, outside diameter of sampler, and inside diameter of sample.

The soils encountered in the borings were continuously logged in the field and visually described in accordance with the Unified Soil Classification System (ASTM D2488) as modified in the Boring Log Explanation, Figures No. 3 and 4, in Appendix A. The soil classification was verified upon completion of laboratory testing in accordance with ASTM D2487.

Appendix A contains the site plan showing the locations of the test borings, our borings logs and an explanation of the soil classification system used. Stratification lines on the boring logs are approximate as the actual transition between soil types may be gradual.

INFILTRATION TESTING

Infiltration testing was performed within each of the three (3) proposed infiltration areas as identified by Verde Design. The general locations of the infiltration tests are illustrated on Figure No. 2, Appendix A of this report. The infiltration test borings were advanced to depths of 0 to 2 feet below the invert elevation of the proposed stormwater infiltration feature (approximately 5 feet below the existing ground surface elevation), as specified by the "Native Soil Assessment For Small Infiltration-Based Storm water Control Measures" test procedure.

All infiltration test holes were drilled using a track-mounted drill rig equipped with 6-inch or 8-inch diameter solid flight augers. An engineer and/or geologist from Pacific Crest Engineering Inc. were present during the drilling operations to log the soil encountered and to verify the infiltration test depths. Approximately 1 to 2 inches of clean crushed ½-inch diameter gravel was placed at the bottom of each boring. A 4-inch diameter perforated pipe was then placed within each test hole, and the annular space backfilled with gravel. The test holes were presoaked for approximately 24 hours prior to infiltration testing.

The infiltration tests were performed in accordance with the Central Coast Low Impact Development Initiative, with procedures outlined in the report titled "Native Soil Assessment For Small Infiltration-Based Storm Water Control Measures". Our infiltration study followed the "Shallow Quick Infiltration Test" method, as described within Attachment 1 of the above referenced document. This procedure is generally described as follows:

1. At the commencement of each test, the water level within the infiltration test boring was adjusted to the top of the test zone (approximately 2 feet above the bottom of the boring). This was accomplished by using graduated cylinders, allowing the volume of water placed within the test boring to be recorded.
2. The water level within each test boring was maintained at a constant head for the initial 30 minutes of the test. The volume of water required to maintain the constant head was recorded.



3. Following the initial 30-minute constant head period, the water elevation was allowed to fall. This portion of the test was continued for a minimum of 2 hours, with water elevation readings being taken every 2 to 30 minutes contingent on the infiltration rate. The difference in water elevation was then used to compute the infiltration rate at each time interval.
4. If the test boring were to run out of water during the 2-hour test, it would be refilled to the initial elevation. If the infiltration rate was such that the test boring was to run dry following 2 refills (not including the initial fill-up), then the test was concluded.
5. If the drop in elevation at any time was less than 6 inches in 2 hours, or if the readings were not stable at the end of the 2-hour test, then the test was continued for an additional 2-hour interval (4 hours total).
6. The final infiltration rate was defined as the average infiltration rate during the last time interval. The last time interval is considered to be the last refill cycle or the last 2 hours of a 4-hour test. All final infiltration rates (I_f) are calculated in $(\text{in}^3/\text{in}^2)/\text{hr.}$ or $(\text{in}/\text{hr.})$. The factored infiltration rate (k_f), which includes a factor of safety of 2, was also calculated from the final interval.

A summary of the infiltration test results is provided below. The complete infiltration test sheets are provided within Appendix C of this report.

LABORATORY TESTING

The laboratory testing program was developed to aid in evaluating the engineering properties of the materials encountered at the site. Laboratory tests performed include:

- Moisture Density relationships in accordance with ASTM D2937.
- Gradation testing in accordance with ASTM D1140.
- Atterberg Limits testing in accordance with ASTM D4318.
- "R" Value testing in accordance with California 301.

The results of the laboratory testing are presented on the boring logs opposite the sample tested and/or presented graphically in Appendix A.

III. FINDINGS AND ANALYSIS

GEOLOGIC SETTING

The surficial geology within the park is mapped as Alluvial Deposits underlain by Santa Cruz Mudstone at depth (Brabb, 1997). The Alluvial Deposits are described by Brabb as moderately graded, generally unconsolidated deposits of silt and sand that contain discontinuous lenses of clay and silty clay. The Santa Cruz Mudstone is described by Brabb as slightly laminated, pale yellowish brown siliceous mudstone that is medium to thickly bedded. The native soils encountered in the test borings are consistent with these descriptions.

SURFACE CONDITIONS

The site is relatively flat with a mild hummocky surface and that slopes gently towards the south. Two heavily vegetated riparian corridors cut through the park, the eastern corridor separates the parking lot from the main park area. Public access is provided to the main park area via two existing pedestrian bridges



that cross the northern end of the site. The southern boundary is walled by vegetation at the confluence of the two riparian corridors.

Surface drainage within the park is directed by the sloping surface towards the riparian corridors. Stormwater runoff from the housing development to the north appears to drain into the park and saturate the northern end of the site. An approximately 2 foot deep drainage ditch appeared to be freshly cut north of the pathway and had visible water several days after the previous rain. The walkway adjacent to the drainage ditch shows longitudinal distress cracking characteristic of ground movement. The likely cause of this ground movement is expansive subgrade material undergoing seasonal drying and wetting which results in cyclic settlement and heave distressing the pavement surface. Site improvements will need to address this hazard through proper site drainage, the use of non-expansive engineered fill materials, and proper compaction of engineered fill in improved areas.

SUBSURFACE SOIL CONDITIONS

Our subsurface exploration consisted of two (2) exploratory borings and three (3) infiltration test borings drilled as close to the proposed improvement areas as possible. The exploratory borings extended 5 feet into the contact with hard rock (approximately 40 feet below ground surface) and infiltration test borings extended 5 feet below ground surface. The soil profile classifications, laboratory test results and groundwater conditions encountered for each test boring are presented in the Logs of Test Borings, in Appendix A. The general subsurface conditions are described below.

The park subgrade is generally comprised of material consistent with Alluvial Deposits. These deposits were generally described as interbedded layers of sand, clayey sand, and clay with varying amounts of silts and gravels. The sand within these borings was predominantly well graded and fine to coarse grained. The clay soils were found to have intermediate to high plasticity, which typically corresponds to moderate to high expansive potential. The material encountered in these areas was stiff to hard at the surface, with a layer of very soft material above the contact with hard rock.

Please refer the Logs of Test Borings in Appendix A and Appendix B, for a more detailed description of the subsurface conditions encountered in each of our test borings at the subject site.

GROUNDWATER CONDITIONS

The groundwater conditions, as encountered in our test borings, are generally described in the following Table:

Table No. 1 – Groundwater Measurements

Boring Location	Groundwater Depth	Date
B2	14	11/3/25
B4	14	11/3/25
1	20	1/11/13
2	20	1/11/13
3	15	1/31/13
4	7	1/31/13

In general, groundwater was encountered at depths ranging between 7 and 20 feet below existing grades. It should be noted that the groundwater level was not allowed to stabilize for more than a few hours; therefore, the actual groundwater level may be higher or lower than initially encountered. The



groundwater conditions described in this report reflect the conditions encountered relative to the respective drill date at the specific locations drilled. It must be anticipated that the perched and regional groundwater tables may vary with location and could fluctuate with variations in rainfall, runoff, irrigation and other changes to the conditions existing at the time our measurements were made.

Please refer the Logs of Test Borings in Appendix A and Appendix B, for a more detailed description of the groundwater conditions encountered in each of our test borings at the subject site.

STORM WATER INFILTRATION

A summary of the infiltration test results is provided below. The complete infiltration test sheets are provided within Appendix C of this report.

Table No. 2 – Summary of Infiltration Test Results

Test No.	Depth of Test Zone Below Existing Grade (ft.)	Soil Type within Test Zone	Soil Gradation			Infiltration Rate, I_t (in/hr.)	Factored Infiltration Rate, K_f (in/hr.)
			Gravel (%)	Sand (%)	Fines (%)		
B1	4.5	Sandy Lean Clay	0.3	36.2	63.5	0.0	0.0
B3	4.5	Clayey Sand	4.8	67.6	27.6	0.0	0.0
B5	4.5	Sandy Lean Clay	1.2	36.9	61.9	0.0	0.0

FAULTING AND SEISMICITY

Faulting

Mapped faults which have the potential to generate earthquakes that could significantly affect the subject site are listed in Table No. 4. The fault distances are approximate distances based on the U.S. Geological Survey and California Geological Survey, Quaternary fault and fold database, accessed November 2025, from the USGS website and overlaid onto Google Earth.

Table No. 3 - Distance to Significant Faults

Fault Name	Distance (miles)	Direction
Butano	5	Northeast
San Andreas	5½	Northeast
Sargent	6	Northeast
Zayante-Vergeles	7½	Southeast
San Gregorio	13½	Southwest

Seismic Shaking and California Building Code (CBC) Design Parameters

Due to the proximity of the site to active and potentially active faults, it is reasonable to assume the site will experience high intensity ground shaking during the lifetime of the project. Structures founded on thick soft soil deposits are more likely to experience more destructive shaking, with higher amplitude and lower frequency, than structures founded on bedrock. Generally, shaking will be more intense closer to earthquake epicenters. Thick soft soil deposits large distances from earthquake epicenters, however, may result in seismic accelerations significantly greater than expected in bedrock.



Selection of seismic design parameters should be determined by the project structural designer. The site coefficients and seismic ground motion values shown in the table below were developed based on CBC 2025 incorporating the ASCE 7-22 standard, and the project site location.

Table No. 4 - 2025 CBC Seismic Design Parameters¹

Seismic Design Parameter	ASCE 7-22 Values
Site Class	D
Spectral Acceleration for Short Periods	$S_s = 1.83g$
Spectral Acceleration for 1-second Period	$S_1 = 0.71g$
MCE Spectral Response Acceleration for Short Period	$S_{MS} = 1.88g$
MCE Spectral Response Acceleration for 1-Second Period	$S_{M1} = 2.14$
Design Spectral Response Acceleration for Short Period	$S_{DS} = 1.25g$
Design Spectral Response Acceleration for 1-Second Period	$S_{D1} = 1.43^2$

Note 1: Design values have been obtained by using the ASCE Hazard Tool

The recommendations of this report are intended to reduce the potential for structural damage to an acceptable risk level, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs. It should be assumed that exterior improvements such as pavements or sidewalks may need to be repaired or replaced following strong seismic shaking.

GEOTECHNICAL HAZARDS

A quantitative analysis of geotechnical hazards was beyond our scope of services for this project. In general, however, the geotechnical hazards associated with the project site include seismic shaking (discussed above), ground surface fault rupture, liquefaction, lateral spreading, landsliding and expansive soils. A discussion of these hazards is presented below.

Ground Surface Fault Rupture

Pacific Crest Engineering Inc. has not performed a specific investigation for the presence of active faults at the project site. Based upon our review of the Santa Cruz County GIS Hazard Maps, the project site is not mapped within a fault hazard zone.

Ground surface fault rupture typically occurs along the surficial traces of active faults during significant seismic events. Since the nearest known active, or potentially active fault trace is mapped approximately 5 miles from the site, it is our opinion that the potential for ground surface fault rupture to occur at the site should be considered low.

Liquefaction

Liquefaction is the process in which the seismic shear waves cause an increase in the pore water pressure in a cohesionless (sand and some non-cohesive silts) soil strata. This increase in pore water pressure reduces the effective stress that confines the soil. The reduction in effective stress causes a reduction in the shear modulus of the soil, which in turn, results in increased soil deformation. Also associated with liquefaction is a loss in bearing strength. In the case of full liquefaction, when the increase in pore water pressure reduces the confining stress to zero, the soil experiences a full loss of strength and undergoes large viscous deformations. Lateral spreading is possible when liquefaction occurs in ground having even



minimal slope. Primary factors that can trigger liquefaction are moderate to strong ground shaking, relatively clean and loose granular soils, and saturated soil conditions. Liquefaction is generally limited to the upper saturated 50 feet of ground surface due to the increasing overburden pressure with depth. Based upon our review of the regional geologic hazard maps, soils that underlay the project site are mapped to have a moderate liquefaction potential.

Our analysis utilized the software program LiqSVs 1.3.3.1 by Geologismiki, which is based upon the most recent recommendations of the NCEER Workshop. The program calculates a factor of safety against liquefaction and estimates seismically induced settlement due to both liquefaction and dynamic compaction of loose, dry sands above the design water table. The following criteria were used for our analysis:

1. Estimated mean peak ground accelerations (PGAM) of 0.65g and a 7.6 magnitude (M) earthquake occurring on the San Andreas Fault, as derived from a deaggregation tool available from the USGS website.
2. We have assumed a design groundwater at 14 feet below the ground surface.

The results of our liquefaction analysis indicate that a cumulative settlement on the of 1 to 3 inches may be realized at the site during a seismic event. Therefore, it is our opinion that there is a moderate probability that liquefaction will occur and negatively impact the proposed improvements to the site. Refer to Appendix D of this report for the results of our liquefaction analysis.

Lateral Spreading

Liquefaction induced lateral spreading occurs when a liquefied soil mass fails toward an open slope face, or fails on an inclined topographic slope. Our quantitative liquefaction analysis indicates that the site has a moderate potential for liquefaction, consequently the potential for lateral spreading is also considered moderate.

Landsliding

The subject site and immediate vicinity are relatively flat to gently sloping. It is our opinion that the potential for shallow landsliding to occur and adversely affect the proposed development should be considered low.

Slope failures can occur where surface drainage is allowed to concentrate onto unprotected slopes. Appropriate landscaping and good control of surface drainage around the project area becomes very important to reduce potential for shallow slumping of slopes. Erosion control measures should be implemented and maintained. Under no circumstances should surface runoff be directed toward, or discharged upon, any topographic slopes.

Expansive Soils

Expansive soils have an ability to absorb relatively large amounts of water and subsequently undergo relatively large changes in volume. Expansive soils can exert considerable expansion pressure on confining elements such as foundations or pavements. The cyclic nature of expansive soils can fatigue supported elements over time through seasonal drying and wetting. The soil expansion potential is influenced by the



type of clay mineral present, the amount of water available for absorption, and the confining pressure on the soil. When the confining pressure exceeds the expansion pressure, the volume change of the soil is greatly diminished and consequently the distress to supported elements is diminished. Soils tested exhibit considerable expansion pressure therefore the risk of expansive soils adversely affecting the proposed improvements is considered moderate.

IV. DISCUSSION AND CONCLUSIONS

GENERAL

1. The results of our investigation indicate that the proposed improvements are feasible from a geotechnical engineering standpoint, provided our recommendations are included in the design and construction of the project.
2. Grading and foundation plans should be reviewed by Pacific Crest Engineering Inc. during their preparation and prior to contract bidding.
3. Pacific Crest Engineering Inc. should be notified at least four (4) working days prior to any site clearing and grading operations on the property in order to observe the stripping and disposal of unsuitable materials, and to coordinate this work with the grading contractor. During this period, a pre-construction conference should be held on the site, with at least the client or their representative, the grading contractor, a City representative and one of our engineers present. At this meeting, the project specifications and the testing and inspection responsibilities will be outlined and discussed.
4. The findings, conclusions and recommendations provided in this report are based on the understanding that Pacific Crest Engineering will remain as Geotechnical Engineer of Record throughout the design and construction phase of the project. The validity of the findings, conclusions and recommendations contained in this report are dependent upon our review of project plans as well as an adequate testing and observation program during the construction phase. Field observation and testing must therefore be provided by a representative of Pacific Crest Engineering Inc., to enable us to form an opinion as to whether the extent of work related to earthwork or foundation excavation complies with the project plans, specifications and our geotechnical recommendations. Pacific Crest Engineering assumes no responsibility for any site work that is performed without the full knowledge and direct observation of Pacific Crest Engineering Inc.

PRIMARY GEOTECHNICAL CONSIDERATIONS

5. Based upon the results of our investigation, it is our opinion that the primary geotechnical issues associated with the design and construction of the proposed project are the following:
 - a. Expansive Soils: The native clay soils at the site exhibit high expansive potential. Seasonal shrinking and swelling of these soils could result in heave or settlement of the ground surface and subsequent damage to improvements. To reduce this potential, we recommend that foundations and concrete flatwork be underlain by non-expansive engineered fill. Refer to the Earthwork and Foundations sections of this report for details.



- b. Compressible Soils: The native clay soils at the site are considered significantly compressible and subject to consolidation under loading. Structures, concrete slabs-on-grade, and pavements underlain by compressible material may be subject to settlement and distress. Recommendations for subgrade preparation are presented in the Earthwork section of this report.
- c. Soil Creep: The surficial soils at the site are comprised of highly expansive clay, which may be subject to soil creep. Consequently, the proposed pedestrian trail, benches, tables and bathroom structure planned near the riparian corridor will likely be impacted by the effects of soil creep. To reduce the potential of damage to the subject improvements, we recommend that the all cut/fill slopes have a gradient no steeper than 3:1 (H:V), and that the trail and other improvements be set back a minimum of 3 feet horizontally from the crown of any slope. Detailed recommendations are presented in the Earthwork section of this report.
- d. Liquefaction: The subsurface investigation found layers of potentially liquefiable material at the project site. The site is located in an area of California that is capable of producing strong enough seismic shaking to trigger liquefaction. The liquefaction analysis, detailed above, found a potential for 1 to 3 inches of post-earthquake settlements at the site. The structural designer should be aware of the liquefaction hazard, and the potential need for post-earthquake repairs.
- e. Strong Seismic Shaking: The project site is located within a seismically active area and strong seismic shaking is expected to occur within the design lifetime of the project. Improvements should be designed and constructed in accordance with the most current California Building Code and the recommendations of this report to minimize reaction to seismic shaking. Structures built in accordance with the latest edition of the CBC have an increased potential for experiencing relatively minor damage which should be repairable, however strong seismic shaking could result in architectural damage and the need for post-earthquake repairs.

V. RECOMMENDATIONS

EARTHWORK

Clearing and Stripping

1. The initial preparation of the site may consist of demolition of portions of any existing flatwork and removal of designated trees and debris. All flatwork and any encountered foundation elements must be completely removed from the development areas. Tree removal should include the entire stump and root ball. Septic tanks and leaching lines, if found, must be completely removed. The extent of this soil removal will be designated by a representative of Pacific Crest Engineering Inc. in the field. This material must be removed from the site.
2. Any voids created by the removal of flatwork, foundations, tree and root balls, septic tanks, and leach lines must be backfilled with properly compacted engineered fill which meets the requirements of this report.
3. Any wells encountered shall be capped in accordance with the requirements and approval of the County Health Department. The strength of the cap shall be equal to the adjacent soil and shall not be located within 5 feet of a structural footing.



4. Surface vegetation, tree roots and organically contaminated topsoil should then be removed ("stripped") from the area to be graded. In addition, any remaining debris or large rocks must also be removed (this includes asphalt or rocks greater than 2 inches in greatest dimension). This material may be stockpiled for future landscaping.

5. It is anticipated that the depth of stripping may be 2 to 4 inches. Final required depth of stripping must be based upon visual observations by a representative of Pacific Crest Engineering Inc., in the field. The required depth of stripping will vary based upon the type and density of vegetation across the project site and with the time of year.

Subgrade Preparation

6. Areas of undocumented fill, if encountered in areas of structural foundations, will need to be completely excavated to undisturbed native material. The excavation process should be observed and the extent designated by a representative of Pacific Crest Engineering Inc., in the field. Any voids created by fill removal must be backfilled with properly compacted engineered fill.

7. After clearing and stripping are completed the following subexcavation depths are recommended:

Pedestrian Pathway: 12 inches below the bottom of pavement

Shallow foundations: 18 inches below bottom of footing

Vehicular pavements and parking areas: 18 inches below the bottom of pavement

8. Subexcavations should extend at least 5 feet horizontally beyond all footings and at least 24 inches horizontally beyond flatwork, pavement, curbs and gutters. Final depth of subexcavation should be determined by a representative of Pacific Crest Engineering Inc., in the field.

9. Wet and/or unstable soils may be encountered at the bottom of the excavations. If wet and/or unstable subgrades are encountered, they may need to further subexcavated and replaced with stabilization fabric, crushed rock or other materials to create a stable working surface. The depth of over-excavations and method used should be determined in the field at the time of construction. All subexcavations should be observed by a representative of Pacific Crest Engineering Inc. and modified as necessary to establish a stable subgrade.

Material for Engineered Fill

10. As noted above, a significant portion of the native surficial soils are highly expansive. Expansive native soils are not suitable as engineered fill within the subexcavations noted above. Therefore, we recommend the following options be considered:

- a. Segregate expansive and non-expansive material under the oversight of Pacific Crest Engineering, Inc. This option will likely require the import and off haul of some material; however, we are unable to quantify import and off haul quantities.
- b. Remove expansive native material and import non-expansive fill to provide the non-expansive sections described in this report.
- c. Chemically treat the native material with lime (Calcium Oxide) to reduce the expansive potential of the soil. Our office should be contacted for additional recommendations if this option is selected.



11. Native or imported soil proposed for use as engineered fill should meet the following requirements:

- a. free of organics, debris, and other deleterious materials,
- b. free of "recycled" materials such as asphaltic concrete, concrete, brick, etc.,
- c. granular in nature, and well graded
- d. free of rocks in excess of 2 inches in size,
- e. Plasticity Index between 4 and 12, and
- f. non-expansive.

12. Samples of any proposed imported fill planned for use on this project should be submitted to Pacific Crest Engineering Inc. for appropriate testing and approval not less than ten (10) working days before the anticipated jobsite delivery. This includes proposed import trench sand, drain rock and for aggregate base materials. Imported fill material delivered to the project site without prior submittal of samples for appropriate testing and approval must be removed from the project site.

Engineered Fill Placement and Compaction

13. Following any necessary subexcavations and/or subgrade preparation, areas should be brought up to design grades with engineered fill that is moisture conditioned and compacted according to the recommendations of this report. ***This should result in a minimum of 18 inches of engineered fill beneath all new footings, slab-on-grade floors, concrete flatwork, pavement, curbs and gutters, as well as 12 inches of engineered fill below new pedestrian pathways.*** Recompacted sections should extend at least 5 feet horizontally beyond all footings, slabs and 24 inches beyond the edges of flatwork and pavement areas, where possible.

14. Engineered fill should be placed in maximum 8 inch lifts, before compaction, at a water content which is within 1 to 3 percent of the laboratory optimum value. Expansive subgrade soils should be moisture conditioned to between 3 to 5 percent above the laboratory optimum.

15. The soil on the project site should be compacted as follows:

- a. In pavement areas the upper 8 inches of subgrade, and all aggregate subbase and aggregate base, should be compacted to a minimum of 95% of its maximum dry density,
- b. In pavement areas all utility trench backfill should be compacted to 95% of its maximum dry density,
- c. All remaining soil on the project site should be compacted to a minimum of 90% of its maximum dry density.

16. The maximum dry density will be obtained from a laboratory compaction curve run in accordance with ASTM Procedure #D1557. This test will also establish the optimum moisture content of the material. Field density testing will be performed in accordance with ASTM Test #D6938 (nuclear method).

17. We recommend field density testing be performed in maximum 1 foot elevation differences. In general terms, we recommend at least one compaction test per 200 linear feet of utility trench or retaining wall backfill, and at least one compaction test per 500 square feet of building or structure area. This is a subjective value and may be changed by the geotechnical engineer based on a review of the final project layout and exposed field conditions.



18. Engineered fill placed on existing slopes that are steeper than 5:1 (H:V) should be keyed and benched into competent native material. Refer to the following Cut and Fill Slope section of this report for additional information on benches and keyways.

Cut and Fill Slopes

19. Fill slopes should be constructed with engineered fill meeting the minimum density requirements of this report and have a gradient no steeper than 3:1 (H:V). When the height of the fill slope is less than or equal to 4 feet, a minimum 8-foot-wide bench with a 2% negatively sloping bottom should be constructed at the base of the fill slope. Benches and keys will be designated in the field by a representative of Pacific Crest Engineering Inc.

20. Fill slopes should not exceed 4 feet in vertical height unless specifically reviewed by Pacific Crest Engineering Inc.

21. Permanent cut slopes shall not exceed a 3:1 (H:V) gradient. All cut slopes should not exceed a 4-foot vertical height unless specifically reviewed by a representative of Pacific Crest Engineering Inc.

22. The above slope gradients are based on the strength characteristics of the materials under conditions of normal moisture content that would result from rainfall falling directly on the slope, and do not take into account the additional activating forces applied by seepage from spring areas or subsurface groundwater. Therefore, in order to maintain stable slopes at the recommended gradients, it is important that any seepage forces and accompanying hydrostatic pressure (if encountered) be relieved by adequate drainage. Drainage facilities may include subdrains, gravel blankets, rock fill surface trenches or horizontally drilled drains. Configurations and type of drainage will be determined by a representative of Pacific Crest Engineering Inc. during the grading operations.

23. The above recommended gradients do not preclude periodic maintenance of the slopes, as minor sloughing and erosion may take place.

24. If a fill slope is to be placed above a cut slope, the toe of the fill slope should be set back at least 8 feet horizontally from the top of the cut slope. A lateral surface drain should be placed in the area between the cut and fill slopes.

25. All flatwork, pavement, curbs and gutters should be set back at least 3 feet horizontally from the crown of any slope. All foundations should be set back at least 10 feet horizontally from the top of cut/fill slope and bottom of a cut slope.

Soil Moisture and Weather Conditions

26. If earthwork activities are done during or soon after the rainy season, the on-site soils and other materials may be too wet in their existing condition to be used as engineered fill. These materials may require a diligent and active drying and/or mixing operation to reduce the moisture content to the levels required to obtain adequate compaction as an engineered fill. If the on-site soils or other materials are too dry, water may need to be added. In some cases, the time and effort to dry the on-site soil may be considered excessive, and the import of engineered fill material may be required.



Utility Trench Backfill

27. Utility trenches that are parallel to the sides of a structure should be placed so that they do not extend below a line sloping down and away at a 2:1 (horizontal to vertical) slope from the bottom outside edge of all footings.

28. Utility pipes should be designed and constructed so that the top of pipe is a minimum of 24 inches below the finish subgrade elevation of any road or pavement areas. Any pipes within the top 24 inches of finish subgrade should be concrete encased, per design by the project civil engineer.

29. For the purpose of this section of the report, backfill is defined as material placed in a trench starting one foot above the pipe, and bedding is all material placed in a trench below the backfill.

30. Unless concrete bedding is required around utility pipes, free-draining clean sand should be used as bedding. Sand bedding should be compacted to at least 95 percent relative compaction. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

31. Approved imported clean sand or native soil should be used as utility trench backfill. Backfill in trenches located under and adjacent to structural fill, foundations, concrete slabs and pavements should be placed in horizontal layers no more than 8 inches thick. This includes areas such as sidewalks, patios, and other hardscape areas. Each layer of trench backfill should be moisture conditioned and compacted to at least 95 percent relative compaction

32. All utility trenches beneath perimeter footing should be backfilled with controlled density fill (such as 2-sack sand\cement slurry) to help minimize potential moisture intrusion below interior floors. The length of the plug should be at least three times the width of the footing or grade beam at the structure perimeter, but not less than 36 inches. A representative from Pacific Crest Engineering Inc. should be contacted to observe the placement of slurry plugs. In addition, all utility pipes which penetrate through the footings, stemwalls or grade beams (below the exterior soil grade) should also be sealed water-tight, as determined by the project civil engineer or architect.

33. Utility trenches which carry "nested" conduits (stacked vertically) should be backfilled with a control density fill (such as 2-sack sand\cement slurry) to an elevation one foot above the nested conduit stack. The use of pea gravel or clean sand as backfill within a zone of nested conduits is not recommended.

34. A representative from our firm should be present to observe the bottom of all trench excavations, prior to placement of utility pipes and conduits. In addition, we should observe the condition of the trench prior to placement of sand bedding, and to observe compaction of the sand bedding, in addition to any backfill planned above the bedding zone.

35. Jetting of the trench backfill is not recommended as it may result in an unsatisfactory compaction.

36. Trenches must be shored as required by the local agency and the State of California Division of Industrial Safety construction safety orders.

Excavations and Shoring

37. It should be understood that on-site safety is the *sole responsibility* of the Contractor, and that the Contractor shall designate a *competent person* (as defined by CAL-OSHA) to monitor excavations prior to



the start of each work day, and throughout the work day as conditions change. The competent person designated by the Contractor shall determine if flatter slope gradients are more appropriate, or if shoring should be installed to protect workers in the vicinity of the slope excavation. Refer to Title 8, California Code of Regulations, Sections 1539-1543.

38. All excavations must meet the requirements of 29 CFR 1926.651 and 1926.652 or comparable OSHA approved state plan requirements.

39. The "top" of any temporary cut slope and excavations should be set-back at least ten feet (measured horizontally) from any nearby structure or property line. Any excavations which cannot meet this requirement will need to have a shoring system designed to support steeper gradients.

40. Temporary shoring is not currently anticipated for this project. Should these requirements change, please contact our office for additional recommendations.

FOUNDATIONS - SPREAD FOOTINGS

41. At the time we prepared this report, foundation and grading plans had not been completed. We request an opportunity to review these items during the design stages to determine if supplemental recommendations will be required.

42. Based upon the results of our investigation, we recommend that the proposed Restroom be founded on spread footing foundations bearing upon engineered fill as described in the preceding sections of this report. No isolated footings should be constructed.

43. All footings must be trenched at least 18 inches below lowest adjacent compacted pad grade.

44. No footings shall be constructed with the intent of placing engineered fill against the footing after the footing is poured and counting that engineered fill as part of the embedment depth of the footing.

45. Footings constructed to the criteria above may be designed using the following parameters: bearing capacities:

- a. Allowable bearing capacity = 1,000 psf for dead plus live loading with a 1/3rd increase for seismic or wind loading
- b. Frictional coefficient between foundations and underlying soil subgrade = 0.30
- c. Ultimate passive resistance = 250 pounds per cubic foot

46. Passive resistance between the sides of the footing and the adjacent soil is only applicable where concrete is placed neatly against undisturbed soil. Voids created by concrete forms should be backfilled with compacted engineered fill or concrete.

47. The upper 1 foot of soil should be ignored when calculating passive soil resistance.

48. In computing the pressures transmitted to the soil by the footings, the embedded weight of the footing may be neglected.

49. Provided our recommendations are followed, total and differential settlement due to applied dead and live loads is expected to be within tolerable limits.

50. No footing should be placed closer than 10 feet to the top of a fill slope nor base of a cut slope.



51. No footing shall be placed on slopes steeper than 4:1 (H:V). If the intent is to place the foundation on sloping ground which exceeds 4:1 (H:V), Pacific Crest Engineering Inc. should be contacted for an alternative pier and grade beam foundation design.

52. All grade beams, thickened slab edges and other foundation elements which impart structure loads to the soil should be considered "footings" and constructed according to the recommendations of this section, including required depths below lowest adjacent grade.

53. The footing excavations must be free of loose material prior to placing concrete. The footing excavations should be thoroughly saturated prior to placing concrete.

54. Footing excavations must be observed by a representative of Pacific Crest Engineering Inc. before placement of formwork, steel and concrete to verify bedding into proper material.

55. The footings should contain steel reinforcement as determined by the project civil or structural engineer in accordance with applicable CBC or ACI Standards.

FLATWORK

56. Subgrade sections beneath all flatwork should be comprised of non-expansive, engineered fill that has been prepared as recommended in the Earthwork section of this report.

57. All exterior slabs, sidewalks, curbs gutters, etc., should be a minimum of 5 inches thick, underlain by a minimum of 12 inches of Class 2 aggregate base and structurally independent of adjacent structural foundation systems. The minimum aggregate base section is in addition to the recommended subgrade sections described in the Earthwork section of this report (i.e. sidewalks should be underlain by 12 inches of Class 2 aggregate base and 12 inches of non-expansive engineered fill).

58. All curbs and gutters constructed on slopes steeper than 3:1 (H:V) should be embedded a minimum of 12 inches and set back a minimum of 3 feet horizontally from the crown of the adjacent slope.

59. Requirements for pre-wetting of the subgrade soils prior to the pouring of the slabs will depend on the specific soils and seasonal moisture conditions and will be determined by a representative of Pacific Crest Engineering Inc. at the time of construction. It is important that the subgrade soils be properly moisture conditioned at the time the concrete is poured. Subgrade moisture contents should not be allowed to exceed our moisture recommendations for effective compaction, and should be maintained until the slab is poured.

60. Final slab thickness, reinforcement, and doweling should be determined by the project civil or structural engineer. The use of welded wire mesh is not recommended for slab reinforcement.

PAVEMENT DESIGN

General

61. To have the selected pavement sections perform to their greatest efficiency, it is very important that the following items be considered:

- a. Properly scarify and moisture condition the upper 8 inches of the subgrade soil and compact it to a minimum of 95% of its maximum dry density, at a moisture content of 1 to 3% over the optimum moisture content for the soil. Expansive subgrade soils should be moisture conditioned



to between 3 to 5 percent above the laboratory optimum and compacted between 88% and 92% of its maximum dry density.

- b. Provide sufficient gradient to prevent ponding of water.
- c. Use only quality materials of the type and thickness (minimum) specified. All aggregate base must meet Caltrans Standard Specifications for Class 2 materials and be angular in shape. All Class 2 aggregate base should be ¾ inch maximum in aggregate size.
- d. Compact the base uniformly to a minimum of 95% of its maximum dry density.
- e. Maintenance should be performed on a routine basis.

62. Porous pavement systems which consist of porous paving blocks, asphaltic concrete or concrete are generally not recommended due to the potential for saturation of the subgrade soils and resulting increased potential for a shorter pavement life. These pavement systems should only be used with the understanding by the Owner of the increased potential for pavement cracking, rutting, potholes, etc. At a minimum, porous pavement systems should include a layer of Mirafi HP370 geotextile fabric placed on the subgrade soil beneath the porous paving section.

63. All curbs and gutters constructed on slopes steeper than 3:1 (H:V) should be embedded a minimum of 12 inches and set back a minimum of 3 feet horizontally from the crown of the adjacent slope.

Asphalt Concrete Pavement Sections

64. The soils that will comprise the pavement subgrade are predominantly clay, with an R-Value of approximately 6. We have used an "R" value of 6 for design of pavement sections provided below.

65. The following table provide flexible pavement design which is based on the 6th Edition of the Caltrans Highway Design Manual – Chapter 630 (last updated December 31, 2016). The design thickness layer was determined to the nearest 0.5 inch, and includes a factor of 0.20 feet added to the asphalt concrete and aggregate base thicknesses, as outlined in the design procedure. Based on this procedure, the following minimum pavement sections are recommended:

TABLE No. 6 - Recommended Pavement Sections

Material	Traffic Index		
	4½	5	6
Asphalt Concrete	2.5 inches	3.0 inches	3.5 inches
Class 2 Aggregate Base	9.0 inches	10.0 inches	12.5 inches
Non-Expansive Engineered Fill ¹	18.0 inches	18.0 inches	18.0 inches
<i>Total Section</i>	<i>29.5 inches</i>	<i>31.0 inches</i>	<i>34.0 inches</i>

Note 1: Non-expansive engineered fill section can be reduced to 12 inches for pedestrian pathway areas

66. Asphaltic concrete should only be placed during periods of fair weather when the free air temperature is within prescribed limits by Caltrans Specifications.



Rigid Pavement Sections

67. The vehicular Portland Cement Concrete (PCC) pavement recommendations as summarized below are based on design procedures outlined in the Portland Cement Association (PCA) design manual titled *"Thickness Design for Concrete Highway and Street Pavements"* (PCA, 1984) and supplemented by procedures by the American Concrete Pavement Association (ACPA) in their report titled *"Design of Concrete Pavement for Streets and Roads"* (ACPA, 2006).

68. As noted above, the soils that will comprise the pavement subgrade are predominantly clay with an R-Value of 6. This R-value generally correlates to modulus of subgrade reaction "k" values of 50 to 100 pci (PCA, 1984). We have conservatively assumed an "k" value of 75 pci for design of the PCC pavement sections provide below.

69. The design of PCC pavement is a function of the Average Daily Truck Traffic (ADTT), which is defined as the average truck traffic volume in both directions on a section of road over a 24-hour period. It is our understanding that ADTT values have not been tabulated for the subject project; therefore, we have provided PCC pavement sections for an assumed range of ADTT values. An allowable ADTT should be chosen that is greater than what is expected for development.

70. The following table provides minimum PCC thicknesses for a range of assumed ADTT values for PCC pavements with and without concrete curb and gutter.

TABLE No. 7 - PCC Pavement Sections

Allowable ADTT	Minimum PCC Thickness (in)	
	With Curb & Gutter	Without Curb & Gutter
3	5.0	5.5
10	5.5	6.5
20	5.5	6.5
50	6.0	7.0

71. PCC pavement should have a minimum compressive strength of 3500 psi.

72. The PCC pavement sections provided above should be underlain by a minimum of 12 inches of Class 2 aggregate base compacted to 95 percent of maximum dry density, as well as 18 inches of engineered fill compacted to 95 percent of maximum dry density. The engineered fill section may be reduced to 12 inches of engineered fill compacted to 95 percent of maximum dry density for pedestrian pathway areas.

73. Expansion/control joints and steel reinforcement should be determined by the project civil. As a minimum, we recommend that joint spacing be limited to a maximum of 2 feet in each direction for each inch of PCC thickness.

SURFACE DRAINAGE

74. Surface drainage will be an important aspect of managing the effects of expansive soils upon planned and existing improvements. Surface water drainage is the responsibility of the project civil engineer. The following should be considered by the civil engineer in design of the project.



75. The project should incorporate drainage improvements around planned and existing improvements in an effort to control soil moisture and minimize the shrink/swell effects of the underlying expansive soil. Landscaping plans should consider measures to maintain a consistent level of moderate soil moisture year-round. This typically includes semi-drought vegetation (as opposed to lawns or plants that need to be watered all summer).

76. Surface water must not be allowed to pond or be trapped adjacent to foundations, or on building pads and parking areas.

77. All roof eaves should be guttered, with the outlets from the downspouts provided with adequate capacity to carry the storm water away from structures to reduce the possibility of soil saturation and erosion. The connection should be in a closed conduit which discharges at an approved location away from structures and graded areas. Stormwater should not be conveyed onto permeable pavement surfaces.

78. Slope failures can occur where surface drainage is allowed to concentrate on unprotected slopes. Appropriate landscaping and surface drainage control around the project area is imperative in order to minimize the potential for shallow slope failures and erosion. Stormwater discharge locations should not be located at the top or on the face of any slope.

79. Final grades should be provided with positive gradient away from all foundation elements. Soil grades should slope away from foundations at least 5 percent for the first 10 feet. Impervious surfaces should slope away from foundations at least 2 percent for the first 10 feet. Concentrations of surface runoff should be handled by providing structures, such as paved or lined ditches, catch basins, etc.

80. Irrigation activities at the site should be done in a controlled and reasonable manner.

81. Following completion of the project we recommend that storm drainage provisions and performance of permanent erosion control measures be closely observed through the first season of significant rainfall, to determine if these systems are performing adequately and, if necessary, resolve any unforeseen issues.

82. The building and surface drainage facilities must not be altered, nor any filling or excavation work performed in the area without first consulting Pacific Crest Engineering Inc. Surface drainage improvements developed by the project civil engineer must be maintained by the property owner at all times, as improper drainage provisions can produce undesirable affects.

STORM WATER INFILTRATION

83. At the time we prepared this report, the project plans had not been completed and the infiltration locations and system details had not been finalized. We request an opportunity to review these plans during the design stages to determine if supplemental recommendations will be required.

84. Our infiltration test borings generally encountered clayey sand and sandy clay within the 2-foot test zone. The fines content (clay and silt fraction) within the infiltration zone ranged from 27% to 63%. These soil conditions facilitated Final Infiltration Rates (I_f) of 0.0 inches/hour, and Factored Infiltration Rates (k_f) of 0.0 inches/hour. Refer to the Findings and Analysis section above and Appendix C of this report for a complete summary of all infiltration data.

85. Infiltration rates tend to decrease as the percentage of fine grained soil increases. The Unified Soil Classification System defines fine grained soils as material with 50 percent or more passing the No. 200



sieve. Furthermore, fine grained soil can be divided into two sub-groups, silt and clay. The deviation between silt and clay is also dependent on the material's respective particle size, with silt being coarser grained than clay. Therefore, infiltration rates also tend to decrease as a soil transitions from silt to clay. A representative of Pacific Crest Engineering, Inc. should be present during the grading process to verify that the encountered soils are consistent with the conditions discussed in this report.

86. Infiltration of water adjacent to structures may saturate surficial soils, resulting in a reduction of shear strength. This reduction in shear strength may trigger or exacerbate differential settlement of the structure. Therefore, we recommend that infiltration systems situated upslope of structures should be setback a minimum 15 feet horizontally from structural foundation elements, while infiltration systems situated downslope of structures should be setback a minimum of 10 feet horizontally from structural foundations. Infiltration areas should also be set back a minimum of 5 feet from all concrete slabs-on-grade and flatwork. Stormwater features within setback should be lined to prevent infiltration.

87. Maintenance of the storm water drainage facilities will be critical in order to maintain the design infiltration rates. The storm water drainage facilities must be inspected and maintained on a routine basis. Repairs and upgrades, whenever necessary, must be made in a timely manner. We recommended that the owner inspect the drainage systems prior to each rainy season, following the first significant rain, and throughout each rainy season. The Civil and Geotechnical Engineers should be consulted if significant drainage problems occur so that the conditions can be observed, and supplemental recommendations can be provided, as necessary.

EROSION CONTROL

88. The surface soils are classified as having a moderate potential for erosion. Therefore, the finished ground surface should be planted with ground cover and continually maintained to minimize surface erosion. For specific and detailed recommendations regarding erosion control on and surrounding the project site, the project civil engineer or an erosion control specialist should be consulted.

89. The surfaces of all cut and fill slopes should be prepared and maintained to reduce erosion. This work, at a minimum, should include track rolling of the slope and effective planting. The protection of the slopes should be installed as soon as practicable so that a sufficient growth will be established prior to inclement weather conditions. It is vital that no slope be left standing through a winter season without the erosion control measures having been provided.

PLAN REVIEW

90. We respectfully request an opportunity to review the project plans and specifications during preparation and before bidding to verify that the recommendations of this report have been included and to provide additional recommendations, if needed. These plan review services are also typically required by the reviewing agency. Misinterpretation of our recommendations or omission of our requirements from the project plans and specifications may result in changes to the project design during the construction phase, with the potential for additional costs and delays in order to bring the project into conformance with the requirements outlined within this report. Services performed for review of the project plans and specifications are considered "post-report" services and billed on a "time and materials" fee basis in accordance with our latest Standard Fee Schedule.



VI. LIMITATIONS AND UNIFORMITY OF CONDITIONS

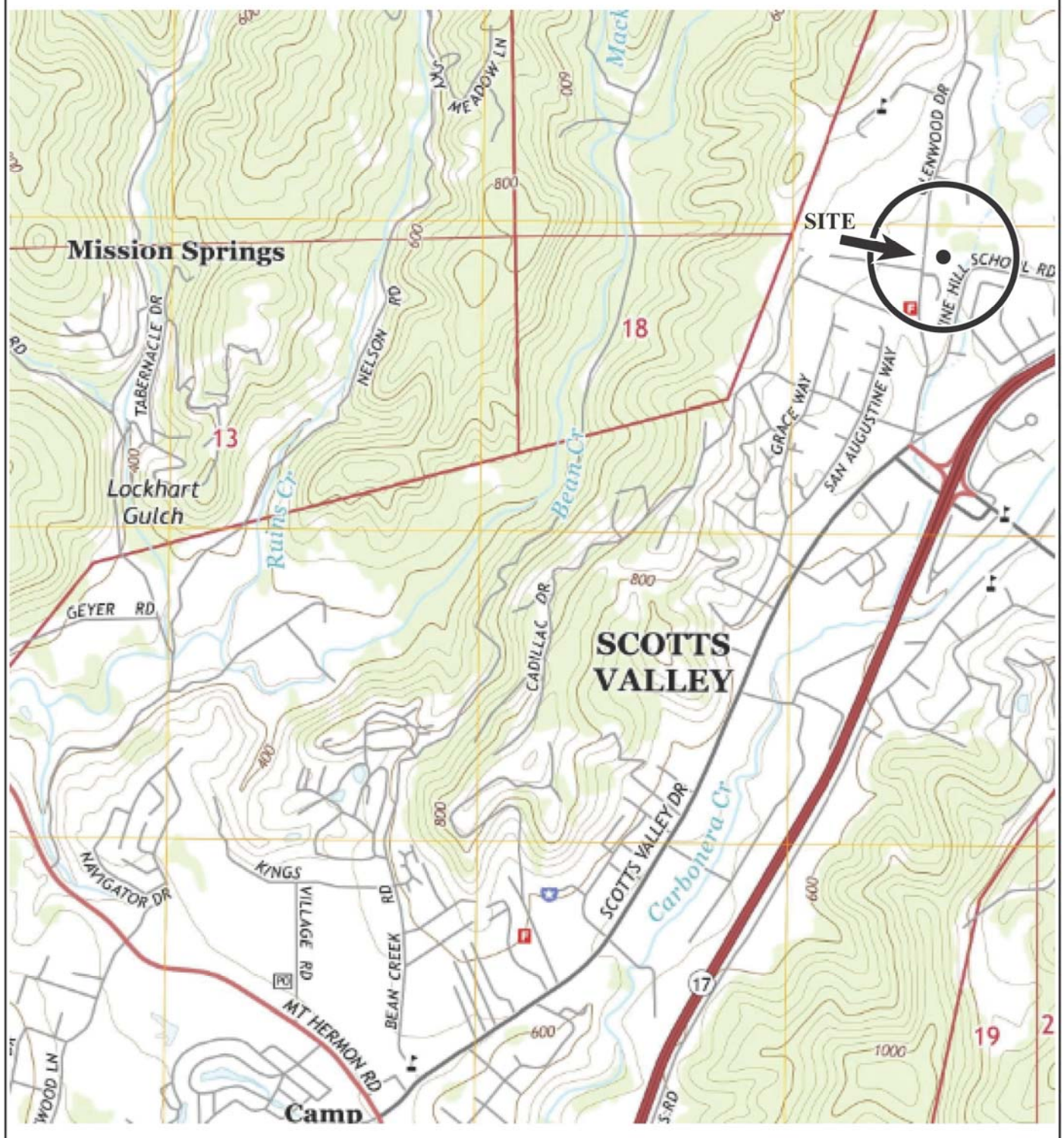
1. This Geotechnical Investigation was prepared specifically for the Verde Design and for the specific project and location described in the body of this report. This report and the recommendations included herein should be utilized for this specific project and location exclusively. This Geotechnical Investigation should not be applied to nor utilized on any other project or project site.
2. The recommendations of this report are based upon the assumption that the soil conditions do not deviate from those disclosed in the borings. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that planned at the time, our firm should be notified so that supplemental recommendations can be provided.
3. This report is issued with the understanding that it is the responsibility of the owner, or his representative, to ensure that the information and recommendations contained herein are called to the attention of the Architects and Engineers for the project and incorporated into the plans, and that the necessary steps are taken to ensure that the Contractors and Subcontractors carry out such recommendations in the field.
4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated, wholly or partially, by changes outside of our control. This report should therefore be reviewed in light of future planned construction and then current applicable codes. This report should not be considered valid after a period of two (2) years without our review.
5. This report was prepared upon your request for our services in accordance with currently accepted standards of professional geotechnical engineering practice. No warranty as to the contents of this report is intended, and none shall be inferred from the statements or opinions expressed.
6. The scope of our services mutually agreed upon for this project did not include any environmental assessment or study for the presence of hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site.



APPENDIX A

Regional Site Map
Site Map Showing Test Borings
Key to Soil Classification
Log of Test Borings
Atterberg Limits
Keyway & Bench Detail





0 1000 ft.

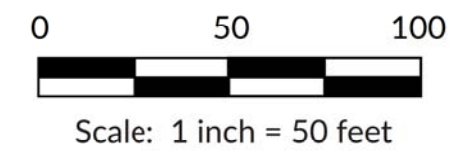


Base Map: Felton Quadrangle, California U.S
Geological Survey 7.5' Quadrangle, 2018,
scale 1:24,000



LEGEND

-  **B-1** Approximate location of infiltration test
-  **B-2** Approximate location of test boring



Base Map: Google Earth

KEY TO SOIL CLASSIFICATION - FINE GRAINED SOILS (FGS) **UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2487 (Modified)**

MAJOR DIVISIONS		SYMBOL	FINES	COARSENESS	SAND/GRAVEL	GROUP NAME
SILT AND CLAY	*LL < 35% Low Plasticity	CL Lean Clay PI > 7 Plots Above A Line	<30% plus No. 200	<15% plus No. 200		Lean Clay / Silt
				15-30% plus No. 200	% sand ≥ % gravel	Lean Clay with Sand / Silt with Sand
			≥30% plus No. 200	% sand ≥ % gravel	% sand < % gravel	Lean Clay with Gravel / Silt with Gravel
					< 15% gravel	Sandy Lean Clay / Sandy Silt
		ML Silt PI > 4 Plots Below A Line	≥30% plus No. 200	% sand < % gravel	≥ 15% gravel	Sandy Lean Clay with Gravel / Sandy Silt with Gravel
					< 15% sand	Gravelly Lean Clay / Gravelly Silt
			<30% plus No. 200	15-30% plus No. 200	≥ 15% sand	Gravelly Lean Clay with Sand / Gravelly Silt with Sand
						Silty Clay
		CL - ML 4 < PI < 7	<30% plus No. 200	15-30% plus No. 200	% sand ≥ % gravel	Silty Clay with Sand
					% sand < % gravel	Silty Clay with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Silty Clay
					≥15% gravel	Sandy Silty Clay with Gravel
					< 15% sand	Gravelly Silty Clay
					≥ 15% sand	Gravelly Silty Clay with Sand
	35% ≤ *LL < 50% Intermediate Plasticity	CI	<30% plus No. 200	15-30% plus No. 200		Clay
					% sand ≥ % gravel	Clay with Sand
			≥30% plus No. 200	% sand ≥ % gravel	% sand < % gravel	Clay with Gravel
					< 15% gravel	Sandy Clay
					≥ 15% gravel	Sandy Clay with Gravel
					< 15% sand	Gravelly Clay
	*LL > 50% High Plasticity	CH Fat Clay Plots Above A Line	<30% plus No. 200	15-30% plus No. 200	≥ 15% sand	Gravelly Clay with Sand
						Fat Clay or Elastic Silt
			≥30% plus No. 200	% sand ≥ % gravel	% sand ≥ % gravel	Fat Clay with Sand
					% sand < % gravel	Elastic Silt with Sand
		MH Elastic Silt Plots Below A Line	≥30% plus No. 200	% sand ≥ % gravel	Fat Clay with Gravel / Elastic Silt with Gravel	
					< 15% gravel	Sandy Fat Clay / Sandy Elastic Silt
					≥ 15% gravel	Sandy Fat Clay with Gravel / Sandy Elastic Silt with Gravel
					< 15% sand	Gravelly Fat Clay / Gravelly Elastic Silt
			% sand < % gravel	≥ 15% sand	Gravelly Fat Clay with Sand / Gravelly Elastic Silt with Sand	

* LL = Liquid Limit
 * PI = Plasticity Index

BORING LOG EXPLANATION

Depth, ft.	Sample	Sample Type	SOIL DESCRIPTION
1	1-1	3	Soil Sample Number L = 3" Outside Diameter M = 2.5" Outside Diameter T = 2" Outside Diameter ST = Shelby Tube B = Bag Sample 1, 2, 3 = Retained Samples = Retained Sample  ← Ground water elevation
2	L	2	
3		1	
4			
5			

MOISTURE

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp, but no visible water
WET	Visible free water, usually soil is below the water table

CONSISTENCY

DESCRIPTION	UNCONFINED SHEAR STRENGTH (KSF)	STANDARD PENETRATION (BLOWS/FOOT)
VERY SOFT	< 0.25	< 2
SOFT	0.25 - 0.5	2 - 4
FIRM	0.5 - 1.0	5 - 8
STIFF	1.0 - 2.0	9 - 15
VERY STIFF	2.0 - 4.0	16 - 30
HARD	> 4.0	> 30



Pacific Crest
ENGINEERING INC

Boring Log Explanation - FGS
 Al Shugart Dog Park
 Scotts Valley, California

Figure No. 3
 Project No. 25116
 Date: 12/23/25

KEY TO SOIL CLASSIFICATION - COARSE GRAINED SOILS
UNIFIED SOIL CLASSIFICATION SYSTEM - ASTM D2487 (Modified)

MAJOR DIVISIONS		FINES	GRADE/TYPE OF FINES	SYMBOL	GROUP NAME *
GRAVEL	More than 50% of coarse fraction is larger than No. 4 sieve size	<5%	$Cu \geq 4$ and $1 \leq Cc \leq 3$	GW	Well-Graded Gravel / Well-Graded Gravel with Sand
			$Cu < 4$ and/or $1 > Cc > 3$	GP	Poorly Graded Gravel / Poorly Graded Gravel with Sand
		5-12%	ML or MH	GW - GM	Well-Graded Gravel with Silt / Well- Graded Gravel with Silt and Sand
				GP - GM	Poorly Graded Gravel with Silt / Poorly Graded Gravel with Silt and Sand
			CL, CI or CH	GW - GC	Well-Graded Gravel with Clay / Well-Graded Gravel with Clay and Sand
				GP - GC	Poorly Graded Gravel with Clay / Poorly Graded Gravel with Clay and Sand
		>12%	ML or MH	GM	Silty Gravel / Silty Gravel with Sand
			CL, CI or CH	GC	Clayey Gravel / Clayey Gravel with Sand
			CL - ML	GC - GM	Silty, Clayey Gravel / Silty, Clayey Gravel with Sand
SAND	50% or more of coarse fraction is smaller than No. 4 sieve size	<5%	$Cu \geq 6$ and $1 \leq Cc \leq 3$	SW	Well-Graded Sand / Well-Graded Sand with Gravel
			$Cu < 6$ and/or $1 > Cc > 3$	SP	Poorly Graded Sand / Poorly Graded Sand with Gravel
		5-12%	ML or MH	SW - SM	Well-Graded Sand with Silt / Well- Graded Sand with Silt and Gravel
				SP - SM	Poorly Graded Sand with Silt / Poorly Graded Sand with Silt and Gravel
			CL, CI or CH	SW - SC	Well-Graded Sand with Clay / Well-Graded Sand with Clay and Gravel
				SP - SC	Poorly Graded Sand with Clay / Poorly Graded Sand with Clay and Gravel
		>12%	ML or MH	SM	Silty Sand / Silty Sand with Gravel
			CL, CI or CH	SC	Clayey Sand / Clayey Sand with Gravel
			CL - ML	SC - SM	Silty, Clayey Sand / Silty, Clayey Sand with Gravel

* The term "with sand" refers to materials containing 15% or greater sand particles within a gravel soil, while the term "with gravel" refers to materials containing 15% or greater gravel particles within a sand soil.

US STANDARD SIEVE SIZE:	3 inch	¾ inch	No. 4	No. 10	No. 40	No. 200	0.002 µm
	COARSE	FINE	COARSE	MEDIUM	FINE		
COBBLES AND BOULDERS	GRAVEL		SAND			SILT	CLAY

RELATIVE DENSITY

DESCRIPTION	STANDARD PENETRATION (BLOWS/FOOT)
VERY LOOSE	0 - 4
LOOSE	5 - 10
MEDIUM DENSE	11 - 30
DENSE	31 - 50
VERY DENSE	> 50

MOISTURE

DESCRIPTION	CRITERIA
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp, but no visible water
WET	Visible free water, usually soil is below the water table

LOGGED BY BA DATE DRILLED 11/03/25 BORING DIAMETER 6" SS BORING NO. 1

DRILL RIG Britton Track HAMMER TYPE Auto-Trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
1	1-1 Bulk		SANDY CLAY: Dark brown (10YR 3/3), low to medium plasticity, fine-grained sand, moist	CL								
2												
3												
4												
5	1-2 L		SANDY CLAY: Very dark grayish brown (10YR 3/2), low plasticity, trace fine-grained sand, moist, hard	CL	7							
6		2			8			17.0	100.5	63.5	18	0.3% Gravel
		1			14	14	4.0					36.2% Sand
7			Boring terminated at 6½ feet. No groundwater encountered.									63.5% Fines
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												
21												
22												
23												

LOGGED BY BA		DATE DRILLED 11/03/25		BORING DIAMETER 6" SS		BORING NO. 2						
DRILL RIG Britton Track			HAMMER TYPE Auto-Trip									
Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
1	2-1 Bulk		SANDY CLAY: Dark brown (10YR 3/3), low plasticity, very fine-grained sand, moist, very stiff	CL								
2	2-2 L	2 1	Moist, medium dense		9 9 10	12		15.3	89.7	56.5		2.6% Gravel 40.9% Sand 56.5% Fines
3	2-3 T				6 8 13	21		16.5			16	
5	2-4 L	2 1	Low to medium plasticity, fine-grained sand, moist, stiff		7 10 13	15		43.0	71.5	57.8		3.6% Gravel 38.6% Sand 57.8% Fines
7	2-5 T		GRAVELLY CLAY: Dark grayish brown (10YR 4/2), coarse subangular to angular gravel, trace fine-grained sand, trace oxidation patches, moist, very stiff	CL	5 7 12	19	2.5					
10	2-6 L	2 1	CLAYEY SAND: Dark yellowish brown (10YR 4/4), low to medium plasticity, trace gravel, moist, loose	SC	3 6 8	9		35.3	84.3	38.6		7.5% Gravel 53.9% Sand 38.6% Fines
12	2-7 T		WELL GRADED SAND WITH CLAY AND GRAVEL: Dark yellowish brown (10YR 4/4), low to medium plasticity, trace oxidation patches, trace gravel, moist to wet, medium dense	SW	3 3 10	13	1.75	34.1		22.7		36.4% Gravel 40.9% Sand 22.7% Fines
15	2-8 T		CLAYSTONE: Very dark gray (2.5Y 3/1), trace oxidation patches, high dry strength, dry, very dense									
16					50/3"	50/3"						
20	2-9 T		Color change to dark gray (2.5Y 4/1), medium to high plasticity when wet, trace fine-grained sand, dry, very dense					27.9		66.4		0.0% Gravel 33.6% Sand 66.4% Fines
21					50/3"	50/3"						
22			Boring terminated at 21½ feet. No groundwater encountered.									
23												



Pacific Crest
ENGINEERING INC

Log of Test Borings
Al Shugart Dog Park
Scotts Valley, CA

Figure No. 6
Project No. 25116
Date: 12/23/25

LOGGED BY		DATE DRILLED		BORING DIAMETER		BORING NO.						
BA		11/03/25		6" SS		3						
DRILL RIG			HAMMER TYPE									
Britton Track			Auto-Trip									
Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
1	3-1 Bulk		CLAY: Dark brown (10YR 3/3), medium to high plasticity, trace fine-grained sand, moist	CI								
2												
3												
4												
5	3-2 L		SILTY SAND: Very dark gray (10YR 3/1) with brownish yellow (10YR 6/6) patches, low plasticity, fine-grained sand, few gravel, moist, medium dense	SM	13							
6		2			18			13.5	103.5	27.6		4.8% Gravel
		1			20	25	4.5+					67.6% Sand 27.6% Fines
7			Boring terminated at 6½ feet. No groundwater encountered.									
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												
21												
22												
23												



Pacific Crest

ENGINEERING INC

Log of Test Borings

Al Shugart Dog Park

Scotts Valley, CA

Figure No. 7

Project No. 25116

Date: 12/23/25

LOGGED BY BA		DATE DRILLED 11/03/25		BORING DIAMETER 6" SS		BORING NO. 4						
DRILL RIG Britton Track		HAMMER TYPE Auto-Trip										
Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
1	4-1 Bulk		CLAYEY SAND: Very dark brown (10YR 2/2), medium to high plasticity, fine-grained sand, moist	SC								
2	4-2 L	2	Moist, loose		7							
3	4-3 T	1	Moist, medium dense		6	9		12.4	92.7	42.7	18	3.9% Gravel 53.4% Sand 42.7% Fines
4					8							
5	4-4 L	2	CLAYEY SAND: Very dark grayish brown (10YR 3/2), very fine- to fine-grained sand, fines exhibit medium plasticity, trace oxidation patches, moist, medium dense	SC	5	12	4.5+					
6	4-5 T	1			5							
7					5							
8			SAND: Brown (10YR 5/3), medium- to fine-grained sand, subangular to angular shaped, moist, medium dense	SP	7	12						
9												
10	4-6 L	2	CLAY WITH SAND: Black (10YR 2/1), medium to high plasticity, trace fine-grained sand, trace gravel, moist, stiff	CI	4							
11	4-7 T	1	Moist, stiff		8	14						
12					13							
13					2	9						
14					3							
15	4-8 T		WELL GRADED SAND WITH CLAY AND GRAVEL: Very dark gray (10YR 3/1), fine to coarse grained sand, well graded, some gravel, fines exhibit low to medium plasticity, wet, loose	SW	6	9						
16												
17												
18												
19												
20	4-9 T		Color change to dark grayish brown (10YR 4/2), wet, loose		3							
21					4			36.7		19.8		26.9% Gravel 53.3% Sand 19.8% Fines
22					6	10						
23												



Pacific Crest
ENGINEERING INC

Log of Test Borings
Al Shugart Dog Park
Scotts Valley, CA

Figure No. 8
Project No. 25116
Date: 12/23/25

LOGGED BY BA		DATE DRILLED 11/03/25		BORING DIAMETER 6" SS		BORING NO. 4						
DRILL RIG Britton Track		HAMMER TYPE Auto-Trip										
Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
24			WELL GRADED SAND WITH CLAY AND GRAVEL: Very dark gray (10YR 3/1), fine to coarse grained sand, well-graded, some gravel, fines exhibit low to medium plasticity, wet, loose	SW								
25	4-10	T	CLAYEY SAND: Very dark gray (10YR 3/1), high plasticity, trace gravel, no recovery, sampler advanced with little to no blow count (pushed), moist to wet, very loose	SC								
26					0	0	<0.5					
27												
28												
29												
30	4-11	T	Color change to gray (10YR 5/1), trace fine-grained sand, moist to wet, loose		1							
31					4			29.4		40.4		0.2% Gravel 59.4% Sand 40.4% Fines
32					6	10	<0.5					
33												
34												
35	4-12	T	CLAYSTONE: Dark gray (10YR 4/1), friable, trace very, massive, trace very fine-grained sand, dry to moist, very dense									
36					50/5"	50/5"						
37												
38												
39												
40	4-13	T	Dry to moist, hard									
41					50/6"	50/6"						
42			Boring terminated at 41½ feet. Groundwater encountered at 14 feet.									
43												
44												
45												
46												



Pacific Crest
ENGINEERING INC

Log of Test Borings
Al Shugart Dog Park
Scotts Valley, CA

Figure No. 9
Project No. 25116
Date: 12/23/25

LOGGED BY		DATE DRILLED		BORING DIAMETER		BORING NO.						
BA		11/03/25		6" SS		5						
DRILL RIG			HAMMER TYPE									
Britton Track			Auto-Trip									
Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
1			SANDY CLAY: Dark brown (10YR 3/3), low to medium plasticity, fine to medium grained sand, moist	CL								
2												
3												
4												
5	5-1		Sampled through root, moist, very stiff		6							1.2% Gravel 36.9% Sand 61.9% Fines
6	L	2			15			18.1	98.6	61.9	24	
		1			15	20						
7			Boring terminated at 6½ feet. No groundwater encountered									
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												
21												
22												
23												



Pacific Crest

ENGINEERING INC

Log of Test Borings

Al Shugart Dog Park

Scotts Valley, CA

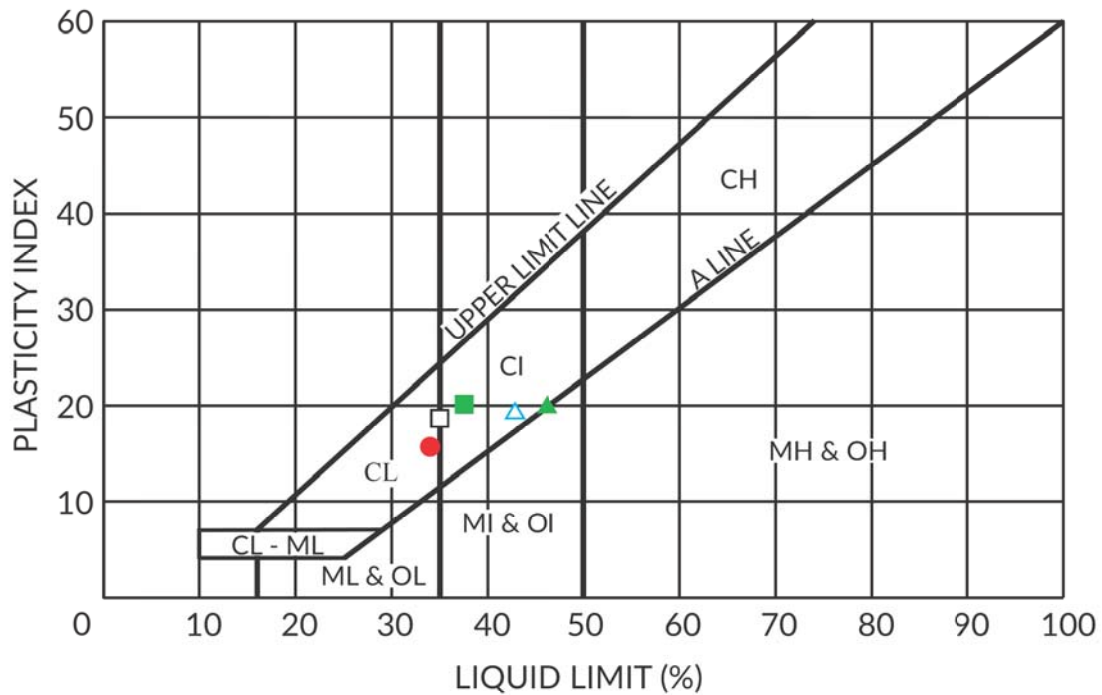
Figure No. 10

Project No. 25116

Date: 12/23/25

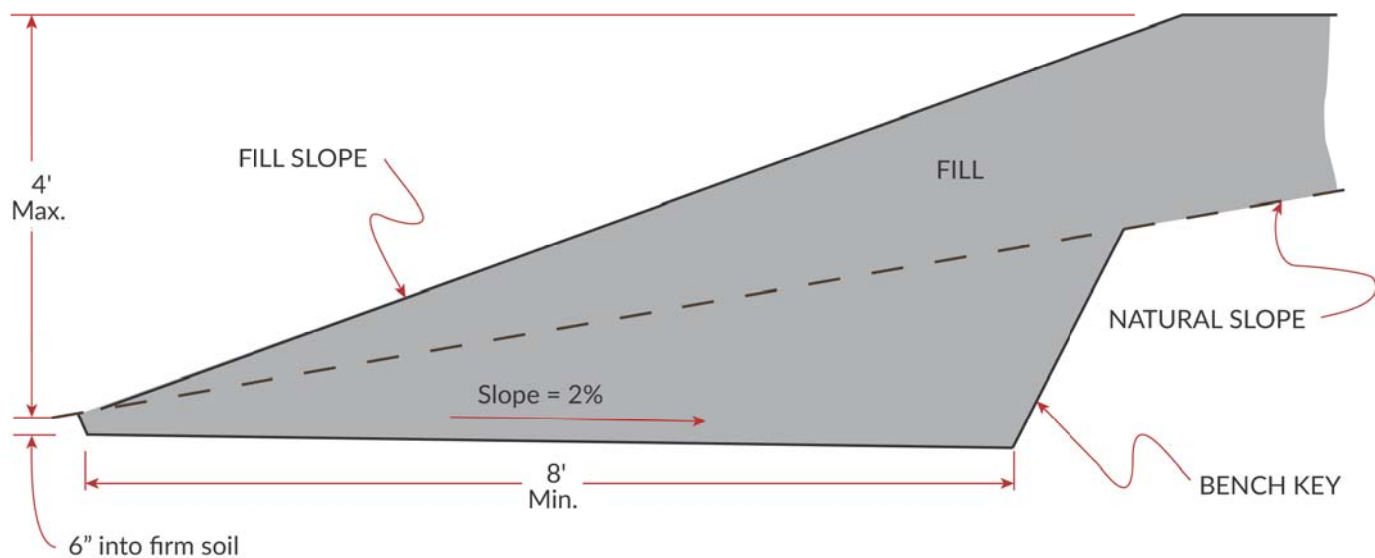
ATTERBERG LIMITS - ASTM D4318

PLASTICITY CHART



*This chart has been modified to include the intermediate classifications CI, MI and OI for clays and silts with liquid limits between 35 and 50.

SYMBOL	SAMPLE #	LL (%)	PL (%)	PI
●	1-2-1	34	16	18
□	2-3	35	19	16
■	4-2-1	38	20	18
▲	4-6-1	46	20	26
△	5-1-1	43	19	24



TYPICAL BENCH FILL - FILL SLOPES NO GREATER THAN 4 FEET
not to scale



APPENDIX B

Boring Logs from 2013 Report



LOGGED BY CLA DATE DRILLED 1/11/13 BORING DIAMETER 8" HS BORING NO. 1

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	1-1		Dark brown CLAY, relatively smooth texture, silty, trace very fine grained sand, low plasticity, very small mica flakes scattered throughout the sample, trace rootlets and trace sub-rounded shaped and weathered siltstone gravels up to 1/4 inch in diameter randomly distributed throughout the sample, damp, firm	CL				29.7	
2	L				7	9	99.9	22.1	
3	1-2		Sample grades to gray Silty SAND, very fine grained, poorly graded, very small mica flakes scattered throughout the sample, trace oxidation patches and moisture pockets randomly distributed throughout the sample, moist, loose	SM	6			18.9	
4	T								
5	1-3		Color change to mottled light brown and dark yellowish brown, trace binder, trace sub-rounded shaped mudstone gravels up to 1/4 inch in diameter randomly distributed throughout the sample, damp, medium dense		14		86.6	29.7	40.6% Passing #200 Sieve
6	L								
7									
8									
9									
10	1-4		Mottled light brown, brown, and dark yellowish brown Clayey SAND, very fine grained, poorly graded, slightly sticky, very small mica flakes scattered throughout the sample, trace sub-rounded shaped mudstone gravels randomly distributed throughout the sample, damp, medium dense	SC	17		92.1	29.7	Qu = 1003 psf
11	L						94.8	27.6	20.0% Passing #200 Sieve
12									
13									
14									
15	1-5		Color change to grayish brown, trace oxidation patches scattered throughout the sample, increased moisture content (moist to wet) and gravel lens at 16 feet, gravels are mudstone, sub-rounded to rounded shaped and up to 1/2 inch in diameter, damp to moist, medium dense		12		83.3	38.1	
16	L								
17									
18									
19									
20	1-6		 No Sample Recovered: Sent sampler back down the bore hole to retrieve disturbed sample. Color change to mottled grayish brown and dark yellowish brown, decrease in clay content, rounded mudstone gravels up to 2 inches in diameter, wet, loose		9		82.0	41.8	19.7% Passing #200 Sieve
21	L							40.2	
22			Driller noted increased drilling resistancy near 22 feet						
23			SANTA CRUZ MUDSTONE BEDROCK						
24									

 Pacific Crest Engineering Inc.
 444 Airport Blvd., Suite 106
 Watsonville, CA 95076

Log of Test Borings
 Shugart Park Pedestrian Bridge
 Scotts Valley, California

 Figure No. 4
 Project No. 1293
 Date: 3/7/13

LOGGED BY CLA DATE DRILLED 1/11/13 BORING DIAMETER 8" HS BORING NO. 1

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	1-7 L		SANTA CRUZ MUDSTONE BEDROCK; Described as gray Sandy SILT, very fine grained sand, small mica flakes scattered throughout the sample, the mudstone is fairly competent, massive, slightly mottled, damp, very hard Slow but steady, very hard drilling. Driller noted that the drill is "really standing up" on the mudstone.		50/4"			30.8	
26									
27	1-8 T		Trace pockets of increased moisture, likely from fractures within the mudstone, slightly weathered, damp to dry, very hard		50/6"			30.8	
28									
29			Boring terminated at 31 1/2 feet. Groundwater initially encountered at 20 feet. Measured at 20 feet upon removing the auger from the bore hole.						
30									
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

 Pacific Crest Engineering Inc.
 444 Airport Blvd., Suite 106
 Watsonville, CA 95076

Log of Test Borings
 Shugart Park Pedestrian Bridge
 Scotts Valley, California

 Figure No. 5
 Project No. 1293
 Date: 3/7/13

LOGGED BY CLA DATE DRILLED 1/11/13 BORING DIAMETER 8" HS BORING NO. 2

Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	2-1		Blackish brown CLAY, intermediate plasticity, trace very fine grained sand, small mica flakes scattered throughout the sample, trace weathered and competent rounded sandstone and mudstone gravels up to 1/2 inch in diameter	CI				29.3	
2					11	22	96.6	25.8	
3	2-2		randomly distributed throughout the sample, large sandstone gravel in the shoe, moist, stiff						
4			Sand content increases slightly with depth and grades to very fine-fine grained, damp, firm		6			22.2	
5	2-3								
6			Sample grades to Clayey SAND at 6 feet, fine to medium grained, sub-angular to sub-rounded shaped, poorly graded, trace yellow oxidation patches randomly distributed throughout the sample, sample grades to mottled blackish brown and dark reddish brown by 6 1/2 feet and grades to very fine grained Sandy CLAY, clay exhibits intermediate plasticity, damp, stiff	SC/CI	11	25	100.3	25.1	
7									
8									
9									
10	2-4		Gray grading to mottled gray and dark yellowish brown CLAY, intermediate plasticity, trace very fine to fine grained sand, small mica flakes scattered throughout the sample, trace rootlets randomly distributed throughout the sample, 2 inch diameter sub-angular shaped mudstone cobble near 11 1/2 feet, damp, firm	CI	7	25	84.7	36.8	
11								34.9	
12			Driller noted increased drilling resistancy near 12 1/2 feet						
13			SANTA CRUZ MUDSTONE BEDROCK; Described as mottled gray and dark yellowish brown SILT, trace very fine grained sand, very small mica flakes scattered throughout the sample, mudstone is massive, fairly competent, and slightly weathered, dry, very hard		50/3"				Direct Shear: C = 0 psf $\phi = 41.6^\circ$
14	2-5								
15									
16			Slow hard drilling to 20 feet, very hard and very slow drilling, but relatively smooth and steady						
17									
18									
19			Color change to gray, trace oxidation patches randomly distributed throughout the sample, the mudstone is "poker chipped" and slightly more friable than the previous sample, the last 6 inches were very hard and the sampler bounced off the mudstone while sampling, damp to dry, very hard		50/4.5"			28.1	
20	2-6								
21									
22			Boring terminated at 21 1/2 feet. Groundwater initially encountered at 20 feet and measured at 20 feet upon removing the auger from the bore hole.						
23									
24									

 Pacific Crest Engineering Inc.
 444 Airport Blvd., Suite 106
 Watsonville, CA 95076

Log of Test Borings
 Shugart Park Pedestrian Bridge
 Scotts Valley, California

 Figure No. 6
 Project No. 1293
 Date: 3/7/13

LOGGED BY <u>CLA</u> DATE DRILLED <u>1/31/13</u> BORING DIAMETER <u>4" SS</u> BORING NO. <u>3</u>									
Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	3-1		Blackish brown CLAY, very smooth texture, very small mica flakes scattered throughout the sample, trace rootlets randomly distributed throughout the sample, damp, stiff	CL					
2						14			
3	3-2		Mottled blackish brown, strong reddish brown, and yellowish brown, CLAY, smooth texture, trace very fine grained sand, very small mica flakes scattered throughout the sample, trace very weathered rounded sandstone gravels randomly distributed throughout the sample, damp, stiff	CL					
4						13			
5	3-3		Slightly silty, slight increase in gravel content, trace sub-rounded shaped, weathered, fine grained chert gravels randomly distributed throughout the sample, damp, very stiff						
6						16			
7									
8									
9									
10									
11	3-4		Color change to mottled dark brown, dark yellowish brown, and gray, slight increase in silt content, slight increase in gravel content, gravels are up to 1/2 inch in diameter, damp, very stiff						
12						19			
13									
14									
15									
16									
17									
18									
19									
20									
21			Driller noted a change in the drilling consistency near 22 feet, drilling became very hard						
22				SANTA CRUZ MUDSTONE BEDROCK					
23									
24									
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076			Log of Test Borings Shugart Park Pedestrian Bridge Scotts Valley, California			Figure No. 7 Project No. 1293 Date: 3/7/13			

LOGGED BY <u>CLA</u> DATE DRILLED <u>1/31/13</u> BORING DIAMETER <u>4" SS</u> BORING NO. <u>3</u>									
Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
25	3-5 T		SANTA CRUZ MUDSTONE BEDROCK; Described as gray Silty SAND, very fine grained, poorly graded, mudstone is weathered, massive, and friable, very small mica flakes scattered throughout the sample, slightly damp to dry, very dense		80/2"				
26									
27			Boring terminated at 24 1/2 feet. Groundwater initially encountered near 21 feet. Groundwater measured at 14 1/2 feet upon removing the auger from the bore hole.						
28									
29									
30									
31									
32									
33									
34									
35									
36									
37									
38									
39									
40									
41									
42									
43									
44									
45									
46									
47									
48									

Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076	Log of Test Borings Shugart Park Pedestrian Bridge Scotts Valley, California	Figure No. 8 Project No. 1293 Date: 3/7/13
---	---	--

LOGGED BY <u>CLA</u> DATE DRILLED <u>1/31/13</u> BORING DIAMETER <u>4" SS</u> BORING NO. <u>4</u>									
Depth (feet)	Sample No. and Type	Symbol	Soil Description	Unified Soil Classification	SPT "N" Value	Plasticity Index	Dry Density (pcf)	Moisture % of Dry Wt.	Misc. Lab Results
1	4-1		Blackish brown CLAY, silty, relatively smooth texture, very small mica flakes scattered throughout the sample, trace rootlets randomly distributed throughout the sample, trace rounded chert pebbles up to 1/4 inch in diameter, trace pockets of increased moisture, damp, stiff	CL	15				
2	L								
3	4-2		Interbedded dark brown Sandy CLAY and dark brown Clayey SAND, sand is very fine to fine grained and poorly graded, trace pockets of oxidized sub-angular shaped sandstone gravels, small mica flakes scattered throughout the sample, damp, stiff/loose	CL/SC	10				
4	T								
5	4-3		Mottled dark brown and dark yellowish brown Clayey SAND, slightly sticky, very fine to fine grained, poorly graded, very small mica flakes scattered throughout the sample, clay content decreases with depth, trace sub-angular shaped siltstone gravels up to 1/4 inch in diameter near 6 feet, damp to slightly moist, loose	SC	8				
6	L								
7									
8									
9									
10	4-4		Mottled gray and dark yellowish brown Sandy CLAY grading to Sandy SILT, fairly smooth texture, sand is very fine to fine grained, trace very fine to fine grained pockets of sand randomly distributed throughout the sample, very small mica flakes scattered throughout the sample, trace sub-rounded shaped siltstone gravels up to 1/4 inch in diameter randomly distributed throughout the sample, damp, very stiff	CL/ML	17				
11	L								
12									
13									
14									
15									
16									
17									
18			Driller noted a change in the drilling consistency near 19 feet, drilling became very hard and very slow						
19									
20	4-5		SANTA CRUZ MUDSTONE BEDROCK; Described as laminated dark yellowish brown and gray SILT, fairly smooth texture, very small mica flakes scattered throughout the sample, bedrock is weathered, massive, and friable, dry to slightly damp, very dense, (low blow count may be a result of slough. Advanced sampler additional 6 inches)		50/2"				
21	T								
22			Boring terminated at 22 feet. Groundwater initially encountered near 15 feet and measured at 7'2" feet upon removing the auger from the bore hole.						
23									
24									
Pacific Crest Engineering Inc. 444 Airport Blvd., Suite 106 Watsonville, CA 95076				Log of Test Borings Shugart Park Pedestrian Bridge Scotts Valley, California			Figure No. 8 Project No. 1293 Date: 3/7/13		

APPENDIX C

Infiltration Test Results



SHALLOW QUICK INFILTROMETER TEST

Native Soil Assessment for Small Infiltration Based Stormwater Control Measures

Test Information								
Test No.:	B-1	Test Date:	11/12/2025	Test By:	CA	Job No.:	25116	
Location of Test:		Closest to Glenwood Drive						
Soil Information								
% Gravel		% Sand		% Silt		% Clay		
USCS Description:		Lean Clay		USCS Classification:		CL		
Test Configuration & Constants								
Existing Surface Elevation (ft.)		0.0		Boring Depth from Top of Pipe (ft.)		7.10		
Bioswale Invert Elevation (ft.)		-		Diameter of Perforated Pipe (in.)		3.25		
Bottom of Boring Elevation (ft.)		-6.5		Diameter of Test Boring (in.)		6.0		
Boring Depth (ft.)		6.5		Cross-Section Area of Boring (in ²)		28.3		
Constant Head Infiltration Data								
Interval		Actual Time (hr:min)	Interval Time (min)	Water Head (in)	Initial Fill Volume (in ³)	Final Fill Volume (in ³)	Infiltration Volume (in ³)	
0	Start	9:52 AM	30	34.2	205.2	436.2	0.7	
	End	10:22 AM						
Infiltration Data								
Interval		Actual Time (hr:min)	Interval Time (min)	Flow Readings		Infiltration Volume (in ³)	Infiltration Rate (in/hr)	
				Water Elev. (in)	Change in Elev (in)			
1	Start	10:22 AM	30.00	51.00	0.24	1.4	0.01	
	End	10:52 AM		51.24				
2	Start	10:52 AM	30.00	51.24	0.24	1.4	0.01	
	End	11:22 AM		51.48				
3	Start	11:22 AM	30.00	51.48	0.00	0.0	0.00	
	End	11:52 AM		51.48				
4	Start	11:52 AM	30.00	51.48	0.24	1.4	0.01	
	End	12:22 PM		51.72				
5	Start	12:22 PM	30.00	51.72	0.24	1.4	0.01	
	End	12:52 PM		51.96				
6	Start	12:52 PM	30.00	51.96	0.12	0.7	0.01	
	End	1:22 PM		52.08				
7	Start	1:22 PM	30.00	52.08	0.00	0.0	0.00	
	End	1:52 PM		52.08				
8	Start	1:52 PM	30.00	52.08	0.00	0.0	0.00	
	End	2:22 PM		52.08				
Test Results								
Infiltration Rate, I _t (in/hr):			0.0		Factored Infiltration Rate, K _m (in/hr):		0.0	

SHALLOW QUICK INFILTROMETER TEST

Native Soil Assessment for Small Infiltration Based Stormwater Control Measures

Test Information							
Test No.:	B-3	Test Date:	11/12/2025	Test By:	CA	Job No.:	25116
Location of Test:		Closest to Glenwood Drive					
Soil Information							
% Gravel		% Sand		% Silt		% Clay	
USCS Description:		Lean Clay		USCS Classification:		CL	
Test Configuration & Constants							
Existing Surface Elevation (ft.)		0.0		Boring Depth from Top of Pipe (ft.)		7.00	
Bioswale Invert Elevation (ft.)		-		Diameter of Perforated Pipe (in.)		3.25	
Bottom of Boring Elevation (ft.)		-6.5		Diameter of Test Boring (in.)		6.0	
Boring Depth (ft.)		6.5		Cross-Section Area of Boring (in ²)		28.3	
Constant Head Infiltration Data							
Interval		Actual Time (hr:min)	Interval Time (min)	Water Head (in)	Initial Fill Volume (in ³)	Final Fill Volume (in ³)	Infiltration Volume (in ³)
0	Start	10:08 AM	30	36.0	216.0	909.0	2.0
	End	10:38 AM					
Infiltration Data							
Interval		Actual Time (hr:min)	Interval Time (min)	Flow Readings		Infiltration Volume (in ³)	Infiltration Rate (in/hr)
				Water Elev. (in)	Change in Elev (in)		
1	Start	10:38 AM	30.00	48.00	3.24	19.4	0.19
	End	11:08 AM		51.24			
2	Start	11:08 AM	30.00	51.24	0.96	5.8	0.05
	End	11:38 AM		52.20			
3	Start	11:38 AM	30.00	52.20	0.84	5.0	0.05
	End	12:08 PM		53.04			
4	Start	12:08 PM	30.00	53.04	0.96	5.8	0.05
	End	12:38 PM		54.00			
5	Start	12:38 PM	30.00	54.00	1.68	10.1	0.09
	End	1:08 PM		55.68			
6	Start	1:08 PM	30.00	55.68	1.20	7.2	0.06
	End	1:38 PM		56.88			
7	Start	1:38 PM	30.00	56.88	0.96	5.8	0.05
	End	2:08 PM		57.84			
8	Start	2:08 PM	30.00	57.84	0.72	4.3	0.04
	End	2:38 PM		58.56			
Test Results							
Infiltration Rate, I _t (in/hr):			0.0		Factored Infiltration Rate, K _m (in/hr):		0.0

SHALLOW QUICK INFILTROMETER TEST

Native Soil Assessment for Small Infiltration Based Stormwater Control Measures

Test Information							
Test No.:	B-5	Test Date:	11/12/2025	Test By:	CA	Job No.:	25116
Location of Test:		Closest to Glenwood Drive					
Soil Information							
% Gravel		% Sand		% Silt		% Clay	
USCS Description:		Lean Clay		USCS Classification:		CL	
Test Configuration & Constants							
Existing Surface Elevation (ft.)		0.0		Boring Depth from Top of Pipe (ft.)		7.00	
Bioswale Invert Elevation (ft.)		-		Diameter of Perforated Pipe (in.)		3.25	
Bottom of Boring Elevation (ft.)		-6.5		Diameter of Test Boring (in.)		6.0	
Boring Depth (ft.)		6.5		Cross-Section Area of Boring (in ²)		28.3	
Constant Head Infiltration Data							
Interval		Actual Time (hr:min)	Interval Time (min)	Water Head (in)	Initial Fill Volume (in ³)	Final Fill Volume (in ³)	Infiltration Volume (in ³)
0	Start	10:40 AM	30	24.0	144.0	185.8	0.2
	End	11:10 AM					
Infiltration Data							
Interval		Actual Time (hr:min)	Interval Time (min)	Flow Readings		Infiltration Volume (in ³)	Infiltration Rate (in/hr)
				Water Elev. (in)	Change in Elev (in)		
1	Start	11:10 AM	30.00	25.20	3.96	23.8	0.41
	End	11:40 AM		29.16			
2	Start	11:40 AM	30.00	29.16	1.08	6.5	0.10
	End	12:10 PM		30.24			
3	Start	12:10 PM	30.00	30.24	0.48	2.9	0.05
	End	12:40 PM		30.72			
4	Start	12:40 PM	30.00	30.72	0.48	2.9	0.04
	End	1:10 PM		31.20			
5	Start	1:10 PM	30.00	31.20	0.36	2.2	0.03
	End	1:40 PM		31.56			
6	Start	1:40 PM	30.00	31.56	0.24	1.4	0.02
	End	2:10 PM		31.80			
7	Start	2:10 PM	30.00	31.80	0.24	1.4	0.02
	End	2:40 PM		32.04			
8	Start	2:40 PM	30.00	32.04	0.12	0.7	0.01
	End	3:10 PM		32.16			
Test Results							
Infiltration Rate, I _t (in/hr):			0.0		Factored Infiltration Rate, K _m (in/hr):		0.0

APPENDIX D

Liquefaction Analysis



SPT BASED LIQUEFACTION ANALYSIS REPORT

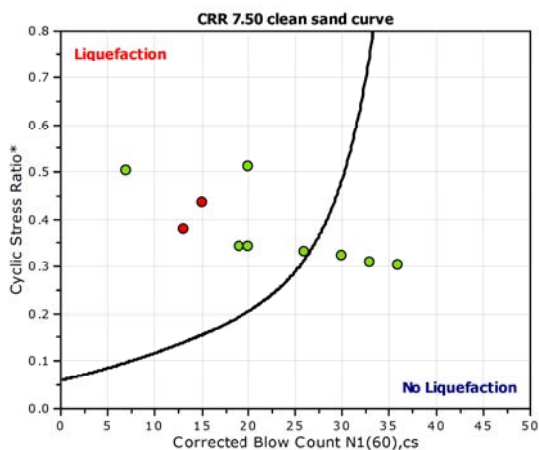
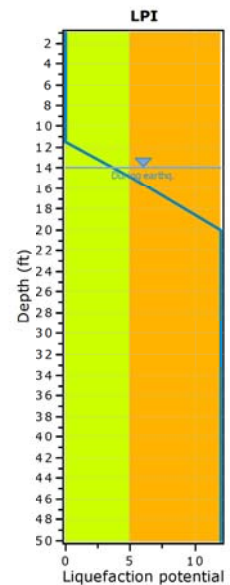
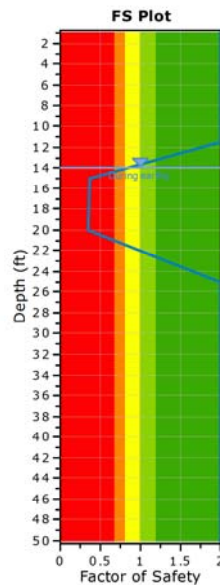
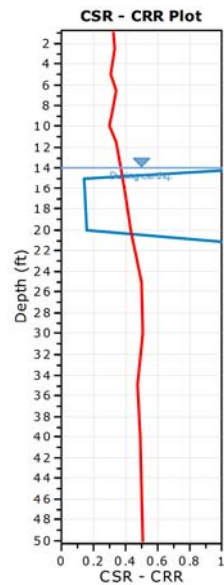
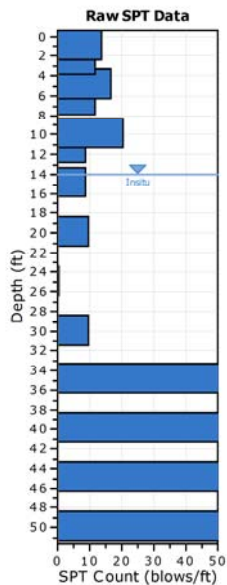
Project title :

SPT Name: B4 Shugart

Location :

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	14.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	14.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w :	7.00
Borehole diameter:	65mm to 115mm	Peak ground acceleration:	0.64 g
Rod length:	3.28 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.33		



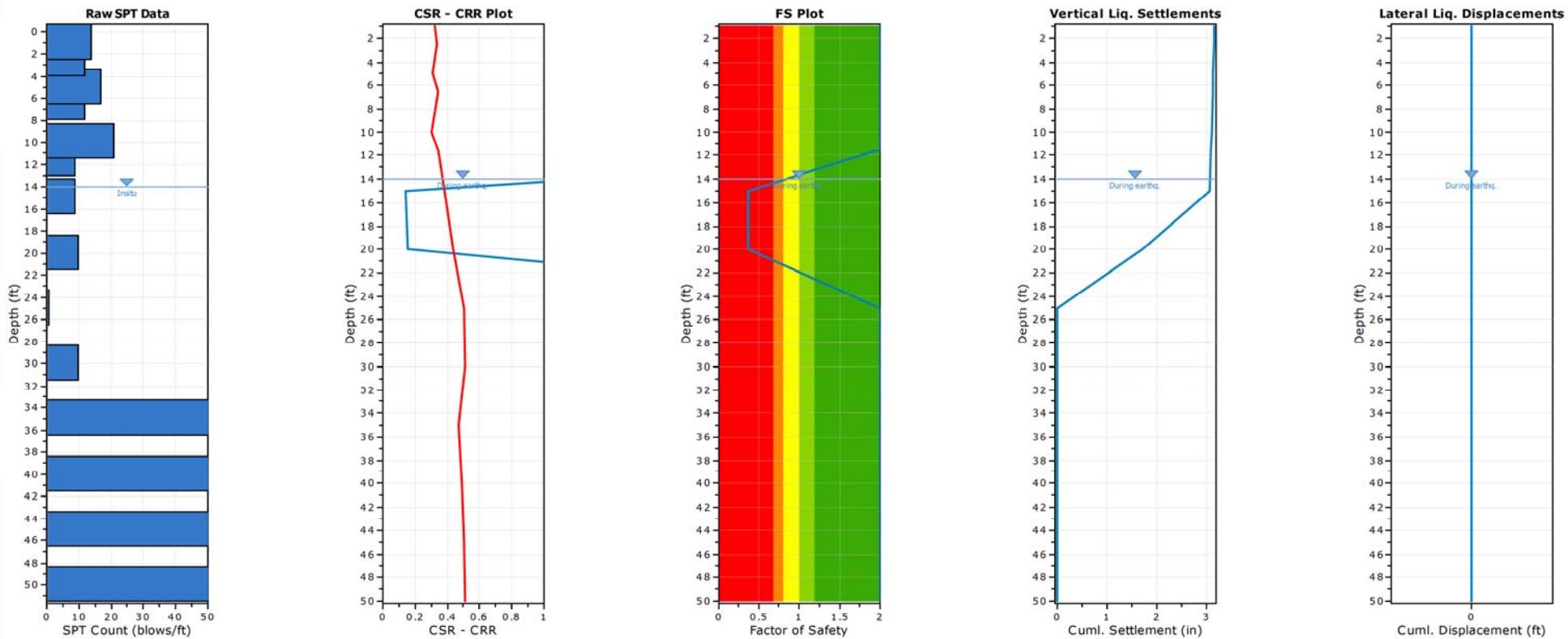
F.S. color scheme

- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

LPI color scheme

- Very high risk
- High risk
- Low risk

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
1.00	14	80.00	90.00	1.00	No
2.50	12	80.00	90.00	1.50	No
5.00	17	50.00	100.00	2.50	Yes
6.50	12	5.00	110.00	1.50	Yes
10.00	21	80.00	95.00	3.50	No
11.50	9	80.00	95.00	1.50	No
15.00	9	10.00	115.00	3.50	Yes
20.00	10	10.00	115.00	5.00	Yes
25.00	1	90.00	70.00	5.00	No
30.00	10	90.00	70.00	5.00	No
35.00	100	90.00	130.00	5.00	No
40.00	100	90.00	130.00	5.00	No
45.00	100	90.00	130.00	5.00	No
50.00	100	90.00	130.00	5.00	No

Abbreviations

Depth: Depth at which test was performed (ft)
 SPT Field Value: Number of blows per foot
 Fines Content: Fines content at test depth (%)
 Unit Weight: Unit weight at test depth (pcf)
 Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
 Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	CRR _{7.5}
1.00	14	90.00	0.04	0.00	0.04	0.34	1.70	1.33	1.00	0.75	1.00	24	80.00	5.54	30	4.000
2.50	12	90.00	0.11	0.00	0.11	0.36	1.70	1.33	1.00	0.75	1.00	20	80.00	5.54	26	4.000
5.00	17	100.00	0.24	0.00	0.24	0.32	1.61	1.33	1.00	0.75	1.00	27	50.00	5.61	33	4.000
6.50	12	110.00	0.32	0.00	0.32	0.44	1.69	1.33	1.00	0.75	1.00	20	5.00	0.00	20	4.000
10.00	21	95.00	0.49	0.00	0.49	0.31	1.28	1.33	1.00	0.85	1.00	30	80.00	5.54	36	4.000
11.50	9	95.00	0.56	0.00	0.56	0.43	1.32	1.33	1.00	0.85	1.00	13	80.00	5.54	19	4.000
15.00	9	115.00	0.76	0.03	0.73	0.50	1.21	1.33	1.00	0.85	1.00	12	10.00	1.15	13	0.140
20.00	10	115.00	1.05	0.19	0.86	0.48	1.11	1.33	1.00	0.95	1.00	14	10.00	1.15	15	0.156
25.00	1	70.00	1.22	0.34	0.88	0.57	1.11	1.33	1.00	0.95	1.00	1	90.00	5.51	7	4.000
30.00	10	70.00	1.40	0.50	0.90	0.44	1.08	1.33	1.00	1.00	1.00	14	90.00	5.51	20	4.000
35.00	100	130.00	1.72	0.66	1.07	0.26	1.00	1.33	1.00	1.00	1.00	133	90.00	5.51	139	4.000
40.00	100	130.00	2.05	0.81	1.23	0.26	0.96	1.33	1.00	1.00	1.00	128	90.00	5.51	134	4.000
45.00	100	130.00	2.37	0.97	1.40	0.26	0.93	1.33	1.00	1.00	1.00	123	90.00	5.51	129	4.000
50.00	100	130.00	2.70	1.12	1.57	0.26	0.90	1.33	1.00	1.00	1.00	120	90.00	5.51	126	4.000

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60CS}$	CRR _{7.5}
---------------	-----------------------	-------------------------	---------------------	----------------	-------------------------	---	-------	-------	-------	-------	-------	--------------	-----------	--------------------	----------------	--------------------

Abbreviations

σ_v : Total stress during SPT test (tsf)
 u_o : Water pore pressure during SPT test (tsf)
 σ'_{vo} : Effective overburden pressure during SPT test (tsf)
m: Stress exponent normalization factor
 C_N : Overburden correction factor
 C_E : Energy correction factor
 C_B : Borehole diameter correction factor
 C_R : Rod length correction factor
 C_S : Liner correction factor
 $N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
 $\Delta(N_1)_{60}$: Equivalent clean sand adjustment
 $N_{1(60)CS}$: Corrected $N_{1(60)}$ value for fines content
CRR_{7.5}: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF _{max}	$(N_1)_{60CS}$	MSF	CSR _{eq,M=7.5}	K_{sigma}	CSR*	FS
1.00	90.00	0.04	0.00	0.04	1.00	1.00	0.418	2.00	30	1.18	0.355	1.10	0.323	2.000 ●
2.50	90.00	0.11	0.00	0.11	1.00	1.00	0.416	1.77	26	1.14	0.366	1.10	0.333	2.000 ●
5.00	100.00	0.24	0.00	0.24	0.99	1.00	0.413	2.19	33	1.21	0.341	1.10	0.310	2.000 ●
6.50	110.00	0.32	0.00	0.32	0.99	1.00	0.410	1.49	20	1.09	0.378	1.10	0.343	2.000 ●
10.00	95.00	0.49	0.00	0.49	0.97	1.00	0.405	2.20	36	1.21	0.334	1.10	0.304	2.000 ●
11.50	95.00	0.56	0.00	0.56	0.97	1.00	0.403	1.45	19	1.08	0.373	1.08	0.344	2.000 ●
15.00	115.00	0.76	0.03	0.73	0.95	1.00	0.413	1.26	13	1.05	0.395	1.04	0.381	0.368 ●
20.00	115.00	1.05	0.19	0.86	0.93	1.00	0.471	1.32	15	1.06	0.446	1.02	0.436	0.358 ●
25.00	70.00	1.22	0.34	0.88	0.90	1.00	0.523	1.14	7	1.02	0.511	1.02	0.503	2.000 ●
30.00	70.00	1.40	0.50	0.90	0.88	1.00	0.569	1.49	20	1.09	0.523	1.02	0.512	2.000 ●
35.00	130.00	1.72	0.66	1.07	0.85	1.00	0.571	2.20	139	1.21	0.472	1.00	0.473	2.000 ●
40.00	130.00	2.05	0.81	1.23	0.82	1.00	0.567	2.20	134	1.21	0.468	0.95	0.490	2.000 ●
45.00	130.00	2.37	0.97	1.40	0.79	1.00	0.559	2.20	129	1.21	0.461	0.92	0.503	2.000 ●
50.00	130.00	2.70	1.12	1.57	0.77	1.00	0.547	2.20	126	1.21	0.452	0.88	0.511	2.000 ●

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
 $u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
 $\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
 r_d : Nonlinear shear mass factor
 α : Improvement factor due to stone columns
CSR: Cyclic Stress Ratio
MSF: Magnitude Scaling Factor
CSR_{eq,M=7.5}: CSR adjusted for M=7.5
 K_{sigma} : Effective overburden stress factor
CSR*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
1.00	2.000	0.00	9.85	1.50	0.00
2.50	2.000	0.00	9.62	1.50	0.00
5.00	2.000	0.00	9.24	2.50	0.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I _L
6.50	2.000	0.00	9.01	1.50	0.00
10.00	2.000	0.00	8.48	3.50	0.00
11.50	2.000	0.00	8.25	1.50	0.00
15.00	0.368	0.63	7.71	3.50	5.20
20.00	0.358	0.64	6.95	5.00	6.80
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 12.00I_L = 0.00 - No liquefactionI_L between 0.00 and 5 - Liquefaction not probableI_L between 5 and 15 - Liquefaction probableI_L > 15 - Liquefaction certain**:: Vertical settlements estimation for dry sands ::**

Depth (ft)	(N ₁) ₆₀	T _{av}	p	G _{max} (tsf)	a	b	γ	ε ₁₅	N _c	ε _{Nc} (%)	Δh (ft)	ΔS (in)
1.00	24	0.02	0.03	0.24	0.13	41138.88	0.00	0.00	10.85	0.02	1.00	0.004
2.50	20	0.05	0.08	0.36	0.13	23740.42	0.00	0.00	10.85	0.03	1.50	0.010
5.00	27	0.10	0.16	0.57	0.13	15162.88	0.00	0.00	10.85	0.02	2.50	0.012
6.50	20	0.13	0.21	0.56	0.14	12679.15	0.00	0.00	10.85	0.06	1.50	0.023
10.00	30	0.20	0.33	0.84	0.14	9864.25	0.00	0.00	10.85	0.02	3.50	0.018
11.50	13	0.22	0.37	0.73	0.15	9087.26	0.00	0.00	10.85	0.08	1.50	0.030

Cumulative settlement: 0.097**Abbreviations**T_{av}: Average cyclic shear stress

p: Average stress

G_{max}: Maximum shear modulus (tsf)

a, b: Shear strain formula variables

γ: Average shear strain

ε₁₅: Volumetric strain after 15 cyclesN_c: Number of cyclesε_{Nc}: Volumetric strain for number of cycles N_c (%)

Δh: Thickness of soil layer (in)

ΔS: Settlement of soil layer (in)

:: Vertical & Lateral displacements estimation for saturated sands ::

Depth (ft)	(N ₁) _{60cs}	γ _{lim} (%)	F _a	FS _{liq}	γ _{max} (%)	e _v (%)	dz (ft)	S _{v-1D} (in)	LDI (ft)
15.00	13	34.14	0.83	0.368	34.14	3.17	3.50	1.332	0.00
20.00	15	27.51	0.75	0.358	27.51	2.87	5.00	1.725	0.00
25.00	7	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
30.00	20	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
35.00	139	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
40.00	134	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00

:: Vertical & Lateral displacements estimation for saturated sands ::									
Depth (ft)	(N ₁) _{60cs}	γ _{lim} (%)	F _a	FS _{liq}	γ _{max} (%)	e _v (%)	dz (ft)	S _{v-1D} (in)	LDI (ft)
45.00	129	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
50.00	126	0.00	0.00	2.000	0.00	0.00	5.00	0.000	0.00
Cumulative settlements:								3.057	0.00

Abbreviations

- γ_{lim}: Limiting shear strain (%)
- F_a/N: Maximun shear strain factor
- γ_{max}: Maximum shear strain (%)
- e_v:: Post liquefaction volumetric strain (%)
- S_{v-1D}: Estimated vertical settlement (in)
- LDI: Estimated lateral displacement (ft)

References

- Ronald D. Andrus, Hossein Hayati, Nisha P. Mohanan, 2009. Correcting Liquefaction Resistance for Aged Sands Using Measured to Estimated Velocity Ratio, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 135, No. 6, June 1
- Boulanger, R.W. and Idriss, I. M., 2014. CPT AND SPT BASED LIQUEFACTION TRIGGERING PROCEDURES. DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING COLLEGE OF ENGINEERING UNIVERSITY OF CALIFORNIA AT DAVIS
- Dipl.-Ing. Heinz J. Priebe, Vibro Replacement to Prevent Earthquake Induced Liquefaction, *Proceedings of the Geotechnique-Colloquium at Darmstadt, Germany*, on March 19th, 1998 (also published in *Ground Engineering*, September 1998), Technical paper 12-57E
- Robertson, P.K. and Cabal, K.L., 2007, *Guide to Cone Penetration Testing for Geotechnical Engineering*. Available at no cost at <http://www.geologismiki.gr/>
- Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 127, October, pp 817-833
- Zhang, G., Robertson, P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, *Canadian Geotechnical Journal*, 39: pp 1168-1180
- Zhang, G., Robertson, P.K., Brachman, R., 2004, Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT, ASCE, *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 130, No. 8, 861-871
- Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, ASCE, *Journal of Geotechnical & Geoenvironmental Engineering*, Vol. 124, No. 4, 364-368
- R. Kayen, R. E. S. Moss, E. M. Thompson, R. B. Seed, K. O. Cetin, A. Der Kiureghian, Y. Tanaka, K. Tokimatsu, 2013. Shear-Wave Velocity-Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential, *Journal of Geotechnical and Geoenvironmental Engineering*, Vol. 139, No. 3, March 1