Appendix F

Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

SEASIDE FIRE STATION NO. 2
SEASIDE, CALIFORNIA

FOR
RRM DESIGN GROUP
SAN LUIS OBISPO, CALIFORNIA

Pacific Crest
ENGINEERING INC

CONSULTING GEOTECHNICAL ENGINEERS

2302-M232-E51
MARCH 2023
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Pacific Crest ENGINEERING INC

GEOTECHNICAL | ENVIRONMENTAL | CHEMICAL | MATERIAL TESTING | SPECIAL INSPECTIONS

March 10, 2023 Project No. 2302-M232-E51

Mr. Michael Scott, Principal
RRM Design Group
3765 South Higuera St. Suite 102
San Luis Obispo, CA 93401

Subject: **Geotechnical Investigation – Design Phase**
Seaside Fire Station No. 2
Northwest Corner of 1st Avenue and Giggling Road
APN 031-151-012
Seaside, California

Dear Mr. Scott,

In accordance with your authorization, we have performed a design-level geotechnical investigation for the proposed fire station located at the northwestern corner of Giggling Road and 1st Avenue in Seaside, 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.

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TABLE OF CONTENTS

I. INTRODUCTION **1**
PURPOSE AND SCOPE 1
PROJECT LOCATION 2
PROPOSED IMPROVEMENTS 2
II. INVESTIGATION METHODS **2**
TEST BORINGS 2
INFILTRATION TESTING 3
LABORATORY TESTING 4
III. FINDINGS AND ANALYSIS **4**
GEOLOGIC SETTING 4
SURFACE CONDITIONS 4
SUBSURFACE CONDITIONS 5
INFILTRATION TEST RESULTS 5
SOIL CORROSIVITY 6
FAULTING AND SEISMICITY 7
GEOTECHNICAL HAZARDS 8
IV. DISCUSSION AND CONCLUSIONS **10**
GENERAL 10
PRIMARY GEOTECHNICAL CONSIDERATIONS 10
V. RECOMMENDATIONS **11**
GENERAL EARTHWORK 11
FOUNDATIONS 15
SLAB-ON-GRADE CONSTRUCTION 17
RETAINING WALLS 18
STRUCTURAL PAVEMENT 19
SURFACE DRAINAGE 21
STORM WATER INFILTRATION 22
EROSION CONTROL 23
PLAN REVIEW 23
VI. LIMITATIONS AND UNIFORMITY OF CONDITIONS **23**
APPENDIX A **25**
Regional Site Map 26
Site Map Showing Test Borings 27
Key to Soil Classification 28

Log of Test Borings......30
Corrosivity Test Summary......49
Direct Shear Test Results......51
R-Value Test Results......52
Typical Retaining Wall Drain Detail......54
Surcharge Pressure Diagram......55

APPENDIX B......56

Infiltration Test Results......57

APPENDIX C......60

Seismically-Induced Settlement Calculations......61

GEOTECHNICAL INVESTIGATION REPORT
Seaside Fire Station No. 2
Seaside, California

I. INTRODUCTION

PURPOSE AND SCOPE

This report describes the geotechnical investigation and presents our conclusions and recommendations for the construction of the proposed fire station in Seaside, California. For purposes of this report, "site" refers to the undeveloped parcel located on the northwest corner of Giggling Road and 1st Street in Seaside, California.

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 and documents:

   - Geologic Map of the Monterey Peninsula and Vicinity, Monterey County, California, Dibblee Jr., 1999.
   - Geologic Map of Monterey County, California, Rosenberg, 2001.
   - Map Showing Relative Earthquake-Induced Landslide Susceptibility of Monterey County, California, Rosenberg, 2001.
   - Map Showing Liquefaction Susceptibility of Monterey County, California, Rosenberg, 2001.
   - Map Showing Relative Fault Hazards of Monterey County, California, Rosenberg, 2001.
   - U.S. Geological Survey (and the California Geologic Survey), 2018, Quaternary fault and fold database for the United States, accessed April 2021, from USGS web site: https://www.usgs.gov/natural-hazards/earthquake-hazards/hazards

3. Field exploration including the drilling and logging of eleven (11) geotechnical test borings and three (3) infiltration test 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. Preparation of this report documenting our investigation and presenting geotechnical recommendations for the design and construction of the project.

PROJECT LOCATION

The subject site is an undeveloped parcel located at the northwest corner of Giggling Drive and 1st Street in Seaside. Please refer to the Regional Site Map, Figure No. 1, in Appendix A for the general vicinity of the project site, which is approximately located by the following coordinates:

Latitude = 36.64463611 degrees
Longitude = -121.8138167 degrees

PROPOSED IMPROVEMENTS

Based on our review of preliminary plans and discussions with design team, it is our understanding that the project will include the construction of an approximately 12,544 ft^2 fire station, two parking lots, a multi-story training tower, storage/maintenance buildings, a trash enclosure structure, and fuel storage tanks. The site improvements will also include retaining walls along 1st Street, an approximately 1-acre fire truck driver training area, driveways, walking paths, and patio flatwork. The project will also include bioswales for storm water management, driveway access gates, and underground utilities associated with these improvements.

The vast majority of earthwork activities will be focused on the fire station, parking lots, and firetruck driver's training area. Except for the retaining walls along 1st Street, the excavations for the proposed improvements are expected to be 4 feet or less. Given the relatively shallow excavation depths, we do not expect the need for temporary shoring or extensive cut/fill slopes.

II. INVESTIGATION METHODS

TEST BORINGS

Eleven, 8-inch diameter test borings (Boring B1 through Boring B11) were drilled at the site on January 17 and 18, 2023. 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 hollow stem 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.

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 described 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.

Samples retrieved with the track mounted drill rig were obtained 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 with an automatic trip hammer. The field blow counts in 6-inch increments were obtained and are reported on the Boring Logs adjacent to each sample as well as the Standard Penetration Test data (SPT). 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. 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.

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

Three (3) infiltration test borings were advanced in the area of the proposed detention/retention basins (Borings P1, P2 and P3). The locations of the infiltration test borings are depicted on the site map included within Appendix A of this report. The infiltration test borings were advanced to depths of 5 to 7 feet below the existing ground surface elevation, as exact grades of the bottom of the bioswale(s) were unknown at the time of our investigation. The "Native Soil Assessment For Small Infiltration-Based Storm water Control Measures" test procedure was followed during the testing of these infiltration test borings.

All infiltration test holes were drilled using a track mounted drill rig equipped with 8-inch diameter augers. An engineer from Pacific Crest Engineering Inc. was present during the drilling operations to log the soil encountered and to verify the infiltration test depths. Approximately 1 to 2 inches of clean, ⅜-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 a flow meter, allowing the initial 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 5- to 20-minutes contingent on the rate of fall. 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 rate of fall 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 rate of fall 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_t) are calculated in (in^3/in^2)/hr. or (in/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 in Table 1 below. The complete infiltration test sheets are provided within Appendix B of this report.

<u>LABORATORY TESTING</u>

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.
- Direct Shear testing in accordance with ASTM D3080.
- R-Value testing in accordance with CTM 301.
- Corrosivity testing in accordance with CTM 643, 422, and 417.

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

<u>GEOLOGIC SETTING</u>

The surficial geology in the project site is mapped as Eolian Deposits (Rosenberg, 2001). These deposits are described as poorly graded silt and sand deposited as extensive coastal dune fields. These deposits are generally described as weakly to moderately consolidated and fine to medium grained. The native soils encountered within our test borings are generally consistent with this description.

<u>SURFACE CONDITIONS</u>

The subject site is situated near the northwestern corner of Giggling Road and 1st Street in Seaside, California. The site is gently flat but slopes up (4H:1V) on the eastern edge to meet the grade of 1st Street. The site is currently undeveloped but shows some signs of minor grading in the areas around 1st Street and Giggling Road as well as unpaved access roads that traverse the parcel. Overhead powerline poles are located along the southern edge of the parcel and run parallel to Giggling Road. The site is currently undeveloped, sparsely wooded and covered in native grasses, ice plant and shrubbery native to the Monterey Bay area.

<u>SUBSURFACE CONDITIONS</u>

Our subsurface exploration program included the advancement of eleven (11) test borings and three (3) infiltration test borings. All test borings were drilled as close to proposed improvements as possible, given the preliminary layout drawings that were available to us at the time. The remaining three (3) infiltration borings were located as close as possible to proposed bioswale areas. The exploratory borings extended from 16½ to 51½ feet below existing grades, while the infiltration test boings within the proposed bioswale areas extended from 5 to 7 feet below existing grades. The soil profiles and 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 upper surficial soils within the site were generally classified as silty sand or sand with silt and extended from about 9½ to 19½ below existing grades. These soils were generally poorly graded and fine to medium grained. At these depths, the soil was generally described as very loose to medium dense.

Underlying the surficial soils described above our borings encountered a layer of poorly graded sand that extended to the maximum explored depth of 51½. These sands were generally medium to fine grained. At these depths, the soil was generally described as medium dense to dense.

Groundwater was not encountered in our test borings and no evidence of shallow ground water was observed at the site.

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

<u>INFILTRATION TEST RESULTS</u>

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

Table No. 1 – Summary of Infiltration Test Results

Test No.	Test Depth (ft)	Soil Type within Test Zone	Soil Gradation			Infiltration Rate, I_t (in/hr.)	Factored Infiltration Rate, K_f (in/hr.)
			Gravel (%)	Sand (%)	Fines (%)		
P1	4.6 to 6.6	Silty Sand	0.0	85.6	14.4	0.6	0.4
P2	3.8 to 5.8	Silty Sand	0.0	84.0	16.0	1.4	0.9
P3	2.8 to 4.8	Silty Sand	0.0	80.9	19.1	0.3	0.1

Infiltration tests were performed in 8-inch diameter borings. In general, we expect tests performed in larger diameter borings to generate faster infiltration rates. This is due to a key assumption in the Porchet Method for calculating infiltration rates which assumes the horizontal and vertical hydraulic conductivities of a soil are equal. In reality, the vertical hydraulic conductivity of a soil is typically greater than the horizontal. Consequently, infiltration rates generally increase if the surface area at the base of the boring is increased.

This testing was conducted during a relatively wet winter following several years of regional drought. As a result, the current saturation levels of the in-situ soils may be higher than normal. Generally, infiltration rates tend to decrease as the relative saturation of the soil increases. Therefore, the infiltration rates as achieved during this site-specific investigation may increase or decrease depending on the relative saturation of the soils. As a result, we would recommend that the civil engineer apply a safety factor to the design values as a way to account for seasonal variations. Please note that the "Factored Infiltration Rate, K_f" provided above includes a factor of safety equal to two. The actual factor of safety should be determined by the project civil engineer.

SOIL CORROSIVITY

In order to address the corrosivity potential at the subject site, testing was performed on two (2) samples of the on-site soils likely to come in contact with concrete and buried metallic structures. The results are summarized as follows:

TABLE No. 2 - Corrosivity Test Summary

Sample	Approximate Sample Depth (ft)	Soil Resistivity	Chloride	Sulfate (water soluble)	pH
		Ohm-cm	mg/kg	mg/kg	
2-3-1	6.0	8177	4	13	6.9
8-1-1	2.0	13930	30	24	5.3

According to the Cal Trans Corrosion Guidelines, Version 3.2 (May 2021), a site may be considered corrosive to foundation elements if one or more of the following conditions exist:

- The soil resistivity is less than 1,100 ohm-cm
- Chloride concentration is greater than or equal to 500 mg/Kg (ppm)
- Sulfate concentration is greater than or equal to 1500 mg/Kg (ppm)
- The soil pH is 5.5 or less

Furthermore, According to Pacific Gas and Electric (PG&E) Electric & Gas Service Requirements (TD-7001M) 2020-2021, a site may be considered corrosive if one or more of the following conditions exist:

- The soil resistivity is less than 3,000 ohm-cm
- The soil pH is less than 4.5 or greater than 9

In comparing the test results to the threshold values, we have determined that the soils likely to be in contact with concrete and buried metallic structures are potentially corrosive. The corrosion potential for any imported select fill should also be tested for corrosivity. Please refer to Appendix A for a site plan that shows the corrosivity test boring locations (Figure 2), associated boring logs, and specific results of the corrosivity testing by the analytical laboratory (Figure 24 & 25).

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. 3. The fault distances are approximate distances based on the U.S. Geological Survey and California Geological Survey, Quaternary fault and fold database, accessed in April 2021 from the USGS website (https://www.usgs.gov/natural-hazards/earthquake-hazards/hazards) and overlaid onto Google Earth.

Table No. 3 - Distance to Significant Faults

Fault Name	Distance (miles)	Direction
Reliz	2	Northeast
Monterey Bay-Tularcitos	4½	Southwest
San Gregorio	12½	Southwest
Zayante-Vergeles	16½	Northeast
San Andreas	20½	Northeast
Sargant	24½	Northeast
Calaveras	26½	Northeast

Seismic Shaking and 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 2022 incorporating the ASCE 7-16 standard, and the project site location.

Table No. 4 - 2022 CBC Seismic Design Parameters [1, 2]

Seismic Design Parameter	ASCE 7-16 Value
Site Class	D
Spectral Acceleration for Short Periods	$S_s = 1.393g$
Spectral Acceleration for 1-second Period	$S_1 = 0.506g$
Short Period Site Coefficient	$F_a = 1.0$

Seismic Design Parameter	ASCE 7-16 Value
1-Second Period Site Coefficient	Fv = N/A[2]
MCE Spectral Response Acceleration for Short Period	S_{MS} = 1.393g
MCE Spectral Response Acceleration for 1-Second Period	S_{M1} = N/A[2]
Design Spectral Response Acceleration for Short Period	S_{DS} = 0.929g
Design Spectral Response Acceleration for 1-Second Period	S_{D1} = N/A[2]
Seismic Design Category [3]	D

Note 1: Design values have been obtained by using the ASCE Hazard Tool at https://asce7hazardtool.online

Note 2: Per Section 11.4.8 of ASCE 7-16, a ground motion hazard analysis may be required for Site Class D sites with S_1 greater than or equal to 0.2. The values provided in this table assume that the value of the seismic response coefficient Cs can be determined by the structural engineer based on the Exceptions as detailed in Section 11.4.8. This should be verified by the structural designer and Pacific Crest Engineering, Inc. should be contacted for revised parameters if these Exceptions are not applicable to the project.

Note 3: The Seismic Design Category assumes a structure with IV occupancy as defined by Table 1604.5 of the 2022 CBC. Pacific Crest Engineering Inc. should be contacted for revised seismic design parameters if the proposed structure has a different occupancy rating than that assumed.

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 damage to improvements and the need for post-earthquake repairs. It should be assumed that exterior improvements such as pavements or sidewalks may also 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 projects in the Seaside area include seismic shaking (discussed above), ground surface fault rupture, liquefaction, lateral spreading and landsliding. A qualitative 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 U.S. Geological Survey, Quaternary fault and fold database 2022, the project site is not underlain by any active or potentially active faults.

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 2 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 and Lateral Spreading

Based upon our review of the regional liquefaction maps (Rosenberg, 2001) the subject site and surrounding area lie within an area mapped as having a low potential for liquefaction.

Liquefaction tends to occur in loose, saturated fine-grained sands and coarse silt, or clays with low plasticity. We did not encounter groundwater during our field investigation. Consequently, it is our opinion that the potential for liquefaction to occur at the subject site may be considered low.

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

Landsliding

Based upon our review of the Map Showing Relative Earthquake-Induced Landsliding of Monterey County, California (Rosenberg, 2001), the subject site and surrounding area lie within an area mapped as having a low potential for earthquake-induced landsliding.

The subject site and immediate vicinity are relatively flat to gently sloping. Provided our recommendations are followed, it is our opinion that the potential for shallow landsliding to occur and adversely affect the proposed development may be considered low.

Slope failures can also 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.

Seismically Induced Settlement

Seismically induced settlement occurs as a result of the compression of intergranular void space during a seismic loading event. In order to assess this hazard, we have evaluated the potential for the upper 50 feet of soil column to settle under seismic “dynamic” loading.

The potential for seismically induced dry sand settlement was evaluated quantitatively for this project, based upon the data obtained from our exploratory test borings. Our analysis utilized the software program LiqSVs Version 1.2.1.6, which is based upon the most recent recommendations of the NCEER Workshop and the work of Pradel 1998. The program calculates the seismically induced settlement due to “dynamic” compaction of loose, dry sands above the design water table.

The following criteria were used in our analysis:

- Peak Ground Acceleration (PGA_M) value of 0.58g determined in accordance with section 1803A.5.12 of the California Building Code.
- Earthquake magnitude 7.1 occurring on the San Andreas Fault, as derived from a deaggregation tool available from the USGS website.
- Groundwater elevation greater than 50 feet below ground surface.

Using the above parameters and subsurface data obtained during the course of our investigation, we have estimated seismically induced settlement on the order of 1 to 2 inches. Please refer to Appendix C for full model parameters and results.

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, 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 earthwork 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. Loose and Compressible Soils Beneath Foundations and Concrete Slabs-On-Grade: Loose and compressible native soils of varying depth underlie the site. Foundations and concrete slabs-on-grade underlain by compressible material may be subject to settlement and distress. In order to reduce potential settlement and distress we recommend that soils underlying proposed foundations, concrete slabs and/or pavement sections be subexcavated as recommended below and recompacted as engineered fill. Detailed recommendations for earthwork, foundations, and concrete slabs-on-grade are presented in the following sections of this report.

b. *Seismically Induced Settlement:* The soils underlying the site have the potential for settlement during a strong seismic event. Calculated seismically induced settlements are on the order of 1 to

2 inches. Similar to our mitigation approach for loose surficial soils, this hazard may also be reduced by over excavating the loose surficial soils and bringing the building pad up to design grades with engineered fill. Detailed recommendations for earthwork, foundations, and concrete slabs-on-grade are presented in the following sections of this report.

c. *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 CBC and the recommendations of this report to minimize reaction to seismic shaking. Structures built in accordance with the latest edition of the California Building Code have an increased potential for experiencing relatively minor damage which should be repairable, however strong seismic shaking could result in damage to improvements and the need for post-earthquake repairs.

V. RECOMMENDATIONS

GENERAL EARTHWORK

Clearing and Stripping

1. The initial preparation of the site may consist of demolition of portions of any existing structures and their foundations and removal of debris. All foundation elements from existing structures must be completely removed from the building areas. 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 old structures and their foundations, 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 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 6 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 man-made fill, if encountered, 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:

   Exterior concrete slabs-on-grade/flatwork: 12 inches below design soil subgrade elevation.
   Structural pavement sections: 12 inches below design soil subgrade elevation
   Structural foundations/interior concrete slabs: 5 feet below design ground surface, or 3 feet below bottom of footing, whichever is greater.

   Following subexcavation to the recommended depths, the exposed subgrade soil should then be scarified 8 inches, moisture conditioned and compacted as outlined below.

8. Subexcavations should extend at least 5 feet horizontally beyond structural foundations and at least 3 feet horizontally beyond pavements and concrete flatwork.

9. Final depth of subexcavation should be determined by a representative of Pacific Crest Engineering Inc., in the field.

<u>Material for Engineered Fill</u>

10. 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, well graded, and contain sufficient binder to allow utility trenches to stand open,
    d. free of rocks in excess of 2 inches in size.

11. In addition to the above requirements, import fill should have a Plasticity Index between 4 and 12, and a minimum Resistance "R" Value of 30, and be 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.

<u>Engineered Fill Placement and Compaction</u>

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 36 inches of engineered fill beneath all structural foundations and interior concrete slabs, and 12 inches beneath exterior concrete slabs-on-grade and pavement subgrades. Recompacted sections should extend at least 5 feet horizontally beyond all foundations and 3 feet beyond the edges of exterior concrete slabs/flatwork and pavements.

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.

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

   a. In pavement areas the upper 12 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 engineered fill below structural foundations and interior concrete slabs should be compacted to a minimum of 95% of its maximum dry density.
   d. 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 2,000 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.

<u>Cut and Fill Slopes</u>

18. No permanent cut or fill slopes are currently proposed for this project. Should permanent cut or fill slopes be proposed, our office should be contacted for additional recommendations. In general, cut or fill slopes should conform to the recommendations of this section.

19. Fill slopes should be constructed with engineered fill meeting the minimum requirements of this report and have a gradient no steeper than 3:1 (horizontal to vertical).

20. Permanent cut slopes in soil shall not exceed a 4:1 (horizontal to vertical) gradient.

21. 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.

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

23. All flatwork should be set back at least 5 feet horizontally from the top of cut and fill slopes. All foundations should be set back at least 8 feet horizontally from the top of cut and fill slopes.

Soil Moisture and Weather Conditions

24. 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 aggregate base may be required.

Utility Trench Backfill

25. Utility trenches that are parallel to the sides of the building 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.

26. 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.

27. 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.

28. 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.

29. 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 water conditioned and compacted to at least 95 percent relative compaction

30. 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.

31. 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.

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

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

<u>Excavations and Shoring</u>

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

35. 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 the slope excavation 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.

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

37. 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 sidewall gradients.

<u>FOUNDATIONS</u>

38. At the time we prepared this report, the project plans had not been completed and the exact locations of the structures and foundation details had not been finalized. We request the opportunity to review these items during the final design stages to determine if supplemental recommendations will be required.

<u>Spread Footings</u>

39. Considering the current proposed building area, the soil characteristics including the potential for settlement, and the site preparation recommendations previously provided, it is our opinion that an appropriate foundation system to support proposed structures will consist of reinforced concrete spread footings constructed as an interconnected grid and embedded into engineered fill. This system could consist of continuous exterior footings, in conjunction with interior continuous footings or concrete slabs. The footings and slab should be tied together to form an interconnected foundation grid. Isolated footings are not recommended.

40. Building areas should be underlain by engineered fill that has been prepared as outlined in the Earthwork section of this report.

41. All footings must be trenched at least 24 inches below lowest adjacent compacted pad grade.

42. All footings should be excavated into engineered fill. 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.

43. Footings constructed to the criteria above may be designed using the following parameters:

   a. Allowable bearing capacity = 2,000 psf for dead plus live loading with a one-third (1/3) increase for seismic or wind loading
   b. Ultimate friction coefficient between foundations and underlying soil subgrade = 0.30
   c. Ultimate passive resistance = 300 pounds per cubic foot

44. Passive soil resistance and friction on the base of the footing may be used in combination with no reduction.

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

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

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

48. Footings located adjacent to utility trenches should be deepened so that the base of the foundation extends below an imaginary 1:1 plane that starts at the base of the trench/pad grade and extends upwards towards the footing.

49. No footing should be placed closer than 10 feet to the top of a fill slope nor 8 feet from the base of a cut slope.

50. 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**.

51. All grade beams, thickened slab edges and other foundation elements which impart structure loads to the soil (from dead, live, wind or seismic loads) should be considered “footings” and constructed according to the recommendations of this section, including required depths below lowest adjacent soil grade.

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

53. 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.

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

SLAB-ON-GRADE CONSTRUCTION

55. All concrete slabs should be underlain by non-expansive engineered fill conforming to the recommendations of this report. In addition to the recommendations presented below, design and construction of concrete slab-on-grade floors should also follow Section 4.505.2 of the 2022 California Green Building Standards Code, which includes installing a vapor retarder in direct contact with concrete and a mix design that addresses bleeding, shrinkage and curling.

56. All exterior non-structural slabs, patios, walkways, etc., should be a minimum of 4 inches in thickness and structurally independent of structural foundation system(s).

57. All interior concrete slabs-on-grade should be underlain by a minimum 6-inch-thick capillary break of ¾ inch clean crushed rock (no fines). It is recommended that neither Class II baserock nor sand be employed as the capillary break material.

58. Where floor coverings are anticipated or vapor transmission may be a problem, a vapor retarder/membrane should be placed between the capillary break layer and the floor slab in order to reduce the potential for moisture condensation under floor coverings. We recommend a high-quality vapor retarder at least 10 mil thick and puncture resistant (Stego Wrap or equivalent). The vapor retarder must meet the minimum specifications for ASTM E-1745, Standard Specification for Water Vapor Retarder. Please note that low density polyethylene film (such as Visqueen) may meet minimum current standards for permeability but not puncture resistance. Laps and seams should be overlapped at least six inches and properly sealed to provide a continuous layer beneath the entire slab that is free of holes, tears or gaps. Joints and penetrations should also be properly sealed.

59. Floor coverings should be installed on concrete slabs that have been constructed according to the guidelines outlined in ACI 302.2R and the recommendations of the flooring material manufacturer.

60. Currently, ACI 302-1R and Section 4.505.2 of the 2022 California Green Building Standards Code recommend that concrete slabs to receive moisture sensitive floor coverings be placed directly upon the vapor retarder, with no sand cushion. ACI states that vapor retarders are not effective in preventing residual moisture within the concrete slab from migrating to the surface. Including a low water-to-cement ratio (less than 0.50) and/or admixtures into the mix design are generally necessary to minimize water content, reduce soluble alkali content, and provide workability to the concrete. As noted in CIP 29 (*Concrete in Practice by the National Ready Mixed Concrete Association*), placing concrete directly on the vapor retarder can also create potential problems. If environmental conditions do not permit rapid drying of bleed water from the slab surface the excess bleeding can delay finishing operations (refer to CIP 13, 19 and 20). Most of these problems can be alleviated by using a concrete mix with a low water content, moderate cement factor, and well-graded aggregate with the largest possible size. With the increased occurrence of moisture related floor covering failures, minor cracking of floors placed on a vapor retarder and other problems discussed here are considered a more acceptable risk than failure of floor coverings, and these potential risks should be clearly understood by the Client and Project Owner.

61. If a sand layer is chosen as a cushion for slabs without floor coverings, it should consist of a clean sand. Clean sand is defined as 100 percent passing the #4 sieve, and less than 5 percent passing the #200 sieve.

62. 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.

63. Recommendations given above for the reduction of moisture transmission through the slab are general in nature and present good construction practice. Moisture protection measures for concrete slabs-on-grade should meet applicable ACI and ASTM standards. Pacific Crest Engineering Inc. are not waterproofing experts. For a more complete and specific discussion of moisture protection within the structure, a qualified waterproofing expert should be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. The waterproofing consultant should provide recommendations for mitigation of potential adverse impacts of moisture vapor transmission on various components of the structure as deemed appropriate.

64. 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.

<u>RETAINING WALLS</u>

65. Retaining walls with full drainage should be designed using the following criteria:

a. The following lateral earth pressure values should be used for design:

Table No. 5 - Active and At-Rest Earth Pressure Values

Maximum Backfill Slope (H:V)	Active Earth Pressure (psf/ft of depth)	At-Rest Earth Pressure (psf/ft of depth)
Level	45	65
4:1	55	80

b. Should the slope behind the retaining walls be other than shown above, supplemental design criteria will be provided for the active earth or at rest pressures for the particular slope angle.

c. Active earth pressure values may be used when walls are free to yield an amount sufficient to develop the active earth pressure condition (about ½% of height). The effect of wall rotation should be considered for areas behind the planned retaining wall (pavements, foundations, slabs, etc.). When walls are restrained at the top or to design for minimal wall rotation, at-rest earth pressure values should be used.

d. A resistance to lateral sliding coefficient of 0.30, and a passive lateral bearing pressure of 300 psf/foot may be assumed. One of these values should be reduced by one-third where both friction and passive resistance are utilized for sliding resistance.

e. Passive resistance should be neglected over the upper 12 inches of footing depth, or where there is less than 8 feet of horizontal distance from face of footing to face of slope.

f. For surcharge pressures due to live or dead loads which transmit a force to the wall, please refer to the attached Figure No. 30 included in Appendix A of this report.

g. If applicable, traffic surcharges on the retaining wall may be simulated by assuming that an additional 2 feet of soil (240 psf) exists on the inboard side of the wall.

h. Retaining wall foundations bearing upon native soil or engineered fill may be designed using an allowable bearing capacity of 1,750 psf.

i. If the structural designer wishes to include seismic forces in their design, the wall may be designed using the above active soil pressures plus a horizontal seismic force of $15H^2$ pounds per lineal foot (where H is the height of retained material). The resultant seismic force should be applied at a point 1/3rd above the base of the wall. This force has been estimated using the Mononobe-Okabe method of analysis as modified by Whitman (1990) and Lew and Sitar (2010). A reduced factor of safety for overturning and sliding may be used in seismic design as determined by the structural designer.

j. The above seismic forces should not be used in combination with at rest lateral soil pressures.

RETAINING WALL DRAINAGE

66. The above design criteria are based on fully drained conditions. Therefore, we recommend that permeable material meeting the State of California Standard Specification Section 68-2.02F, Class 1, Type A, be placed behind the wall, with a minimum width of 12 inches and extending for the full height of the wall to within 1 foot of the ground surface. The top of the permeable material should be covered with Mirafi 140N filter fabric or equivalent and then compacted native soil placed to the ground surface. A 4-inch diameter perforated rigid plastic drain pipe should be installed within 3 inches of the bottom of the permeable material and be discharge to a suitable, approved location. The perforations should be placed downward; oriented along the lower half of the pipe. Neither the pipe nor the permeable material should be wrapped in filter fabric. Please refer to the Typical Retaining Wall Drain Detail, Figure 29, in Appendix A for details.

STRUCTURAL PAVEMENT

Asphalt Concrete Pavement

67. The soils that will comprise the pavement subgrade will in all likelihood be the silty sands and sandy silts that predominate the surficial soils around the development area. The “R” Value results ranged from 66 to 70. We have conservatively assumed an “R” value of 50 for design of pavement sections provided below. This assumption should be verified during construction.

68. The table below provides a flexible pavement design based on the 6th Edition of the Caltrans Highway Design Manual – Chapter 630 (last updated July 1, 2020). Traffic Index (TI) values of 4½ to 6 are provided.

The project civil engineer should verify the required TI for this project. Our office should be contacted for supplemental recommendations for TI values that are not provided below.

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, R=78 min.	4.0 inches	4.0 inches	6.0 inches
Compacted Subgrade	8.0 inches	8.0 inches	8.0 inches
Total Section	*14.5 inches*	*15.0 inches*	*17.5 inches*

69. 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 2 to 4% over the optimum moisture content for the soil.

b. Provide sufficient gradient to prevent ponding of water.

c. Use only quality materials of the type and thickness (minimum) specified. All aggregate base and subbase 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 and subbase uniformly to a minimum of 95% of its maximum dry density.

e. Use ½ inch maximum, Type "A" medium graded asphaltic concrete. Place the asphaltic concrete only during periods of fair weather when the free air temperature is within prescribed limits by Cal Trans Specifications.

f. 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. 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. These pavement systems should only be used with the understanding by the Owner of the increased potential for pavement cracking, rutting, potholes, etc.

g. Maintenance should be undertaken on a routine basis.

Portland Cement Concrete Pavement

70. 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).

71. As noted above, the soils that will comprise the pavement subgrade will in all likelihood be the silty sands that predominate the surficial soils around the development area. The "R" Value results for these soils ranged from 66 to 70. These R-values generally correlate to modulus of subgrade reaction "k" values of 180 to 220 pci (PCA, 1984). We have conservatively assumed an "k" value of 200 pci for design of the PCC pavement sections provide below. This assumption should be verified during construction.

72. 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.

73. 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
23	5.5	6.5
190	6.0	7.0
1100	6.5	7.5

74. PCC pavement should have a minimum compressive strength of 4000 psi.

75. The PCC pavement sections provided above should be underlain by a minimum of 6 inches of Class 2 aggregate base and 12 inches of compacted subgrade, compacted to 95 percent relative compaction.

76. Expansion and control joints should be determined by the project civil or structural engineer. 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

77. 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.

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

79. 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.

80. 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.

81. 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.

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

83. 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.

84. 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.

<u>STORM WATER INFILTRATION</u>

85. 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.

86. It is our understanding that all stormwater will be conveyed to the proposed bioswales along Giggling Road. Our infiltration test borings within the proposed bioswale area generally encountered silty sand within the 2-foot test zone. The fines content (silt fraction) within the infiltration zone ranged from 14.4% to 19.1%. These soil conditions facilitated Final Infiltration Rates (I_t) from 0.3 to 1.4 inches/hour, and Factored Infiltration Rates (K_f) from 0.1 to 0.7 inches/hour. Refer to the Findings and Analysis section above and Appendix B of this report for a complete summary of infiltration data.

87. Infiltration rates tend to decrease as the percentage of fine grained soil increases. 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.

88. Infiltration of water adjacent to buildings 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 be setback a minimum of 15 feet horizontally from structural foundation elements. Infiltration areas should also be set back a minimum of 8 feet from all exterior concrete slabs-on-grade, flatwork and pavements. Stormwater features within setback limits should be lined to prevent infiltration.

89. 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 recommend 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

90. The surface soils are classified as having high 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.

91. 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

92. 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 RRM Design Group 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.

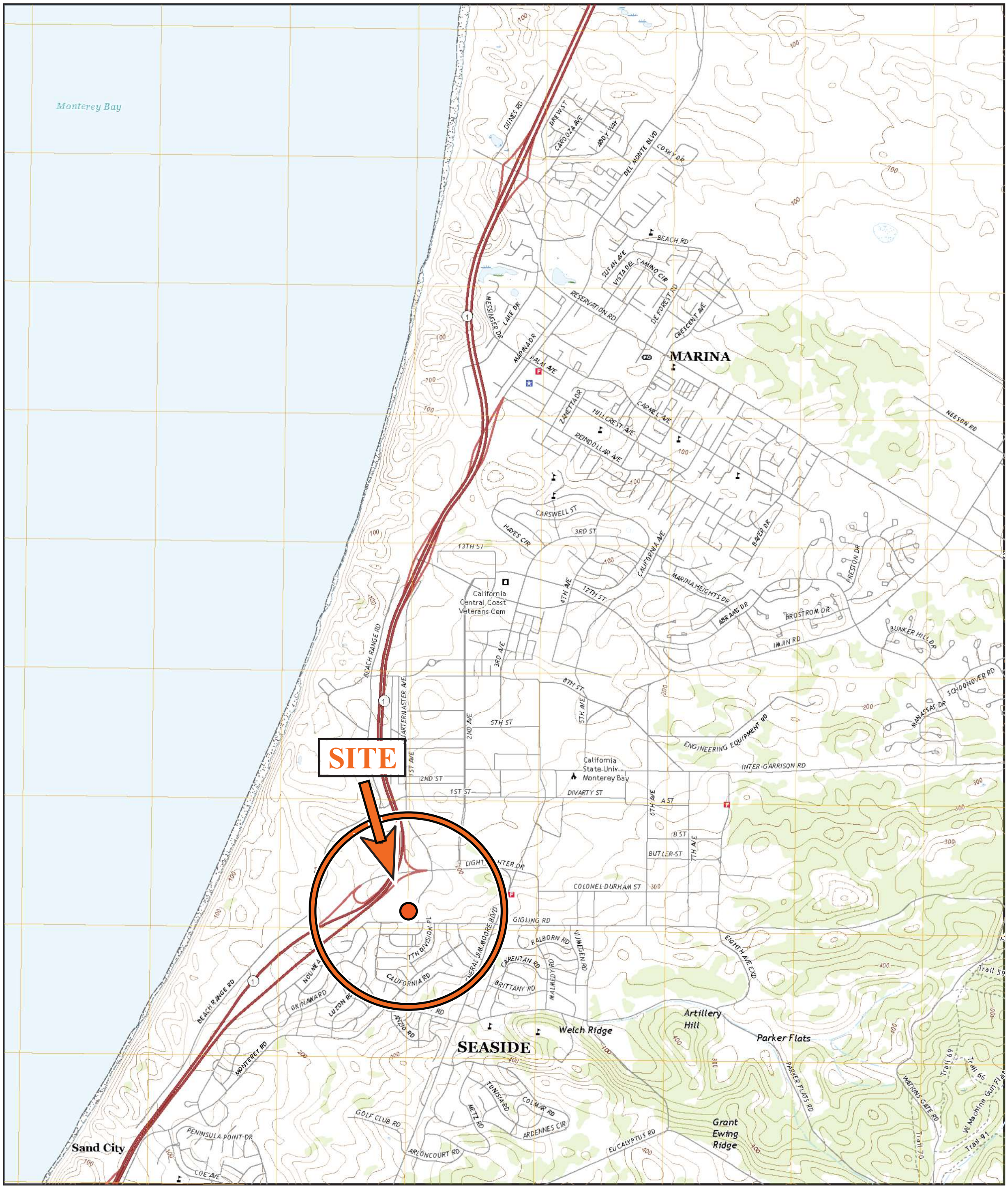
4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they are due to natural process or the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards 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
Corrosivity Test Summary
Direct Shear Test Results
R-Value Test Results
Typical Retaining Wall Drain Detail
Surcharge Pressure Diagram



0 4000 ft.

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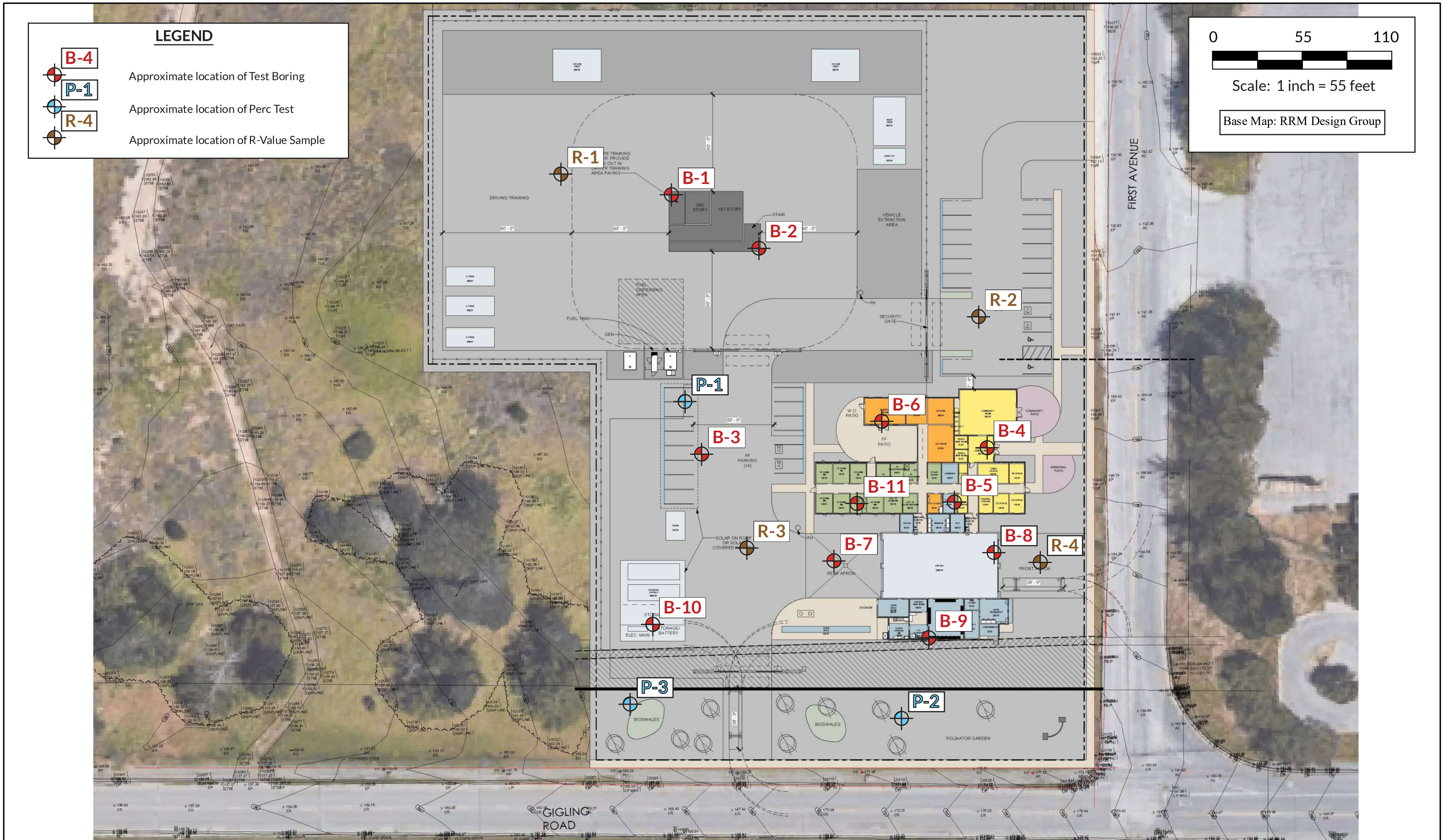
Base Map: © USGS Topographic Maps

Pacific Crest ENGINEERING INC	**Regional Site Map** Seaside Fire Station No. 2 Seaside, California	Figure No. 1 Project No. 2302 Date: 3/10/23



Site Map Showing Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 2
Project No. 2302
Date: 3/10/23

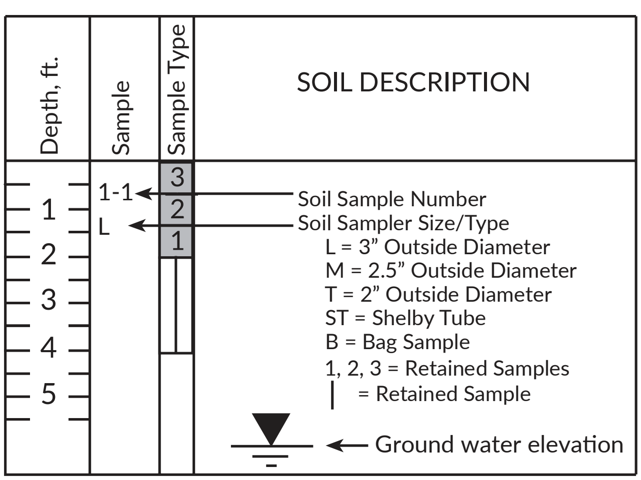
Pacific Crest
ENGINEERING INC

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 -OR- **ML** Silt PI > 4 Plots Below 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
					% sand < % gravel	Lean Clay with Gravel / Silt with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Lean Clay / Sandy Silt
					≥ 15% gravel	Sandy Lean Clay with Gravel / Sandy Silt with Gravel
				% sand < % gravel	< 15% sand	Gravelly Lean Clay / Gravelly Silt
					≥ 15% sand	Gravelly Lean Clay with Sand / Gravelly Silt with Sand
		CL - ML 4 < PI < 7	<30% plus No. 200	<15% plus No. 200		Silty Clay
				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
				% sand < % gravel	< 15% sand	Gravelly Silty Clay
					≥ 15% sand	Gravelly Silty Clay with Sand
	35% ≤ *LL < 50% Intermediate Plasticity	**CI**	<30% plus No. 200	<15% plus No. 200		Clay
				15-30% plus No. 200	% sand ≥ % gravel	Clay with Sand
					% sand < % gravel	Clay with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Clay
					≥ 15% gravel	Sandy Clay with Gravel
				% sand < % gravel	< 15% sand	Gravelly Clay
					≥ 15% sand	Gravelly Clay with Sand
	*LL > 50% High Plasticity	**CH** Fat Clay Plots Above A Line -OR- **MH** Elastic Silt Plots Below A Line	<30% plus No. 200	<15% plus No. 200		Fat Clay or Elastic Silt
				15-30% plus No. 200	% sand ≥ % gravel	Fat Clay with Sand Elastic Silt with Sand
					% sand < % gravel	Fat Clay with Gravel / Elastic Silt with Gravel
			≥30% plus No. 200	% sand ≥ % gravel	< 15% gravel	Sandy Fat Clay / Sandy Elastic Silt
					≥ 15% gravel	Sandy Fat Clay with Gravel / Sandy Elastic Silt with Gravel
				% sand < % gravel	< 15% sand	Gravelly Fat Clay / Gravelly Elastic Silt
					≥ 15% sand	Gravelly Fat Clay with Sand / Gravelly Elastic Silt with Sand

* LL = Liquid Limit
* PI = Plasticity Index

BORING LOG EXPLANATION



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** Seaside Firestation No. 2 Seaside, California	Figure No. 3 Project No. 2302 Date: 3/10/23

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 ≥ 4 and 1 ≤ Cc ≤ 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 ≥ 6 and 1 ≤ Cc ≤ 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

Pacific Crest ENGINEERING INC

Boring Log Explanation - CGS
Seaside Firestation No. 2
Seaside, California

Figure No. 4
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/17/23 | BORING DIAMETER 8" HS | BORING NO. B1

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Brown (7.5YR 4/3), fine to very fine grained, poorly graded, rootlets throughout, moist, loose	SM								
1–2.5	1-1 L	2, 1			3 5 6	6		8.9	101.4			
2.5–4	1-2 T		Decrease in rootlets, moist, loose		2 3 3	6		10.7		12.8		
4.5–5			**SAND WITH SILT:** Yellowish brown (10YR 5/8), medium grained to fine grained, poorly graded, moist, loose	SP -SM								
5–6.5	1-3 L	2, 1			5 7 10	9		15.7	102.8			
10–11.5	1-4 L	2, 1	Very pale brown (10YR 8/4), medium to fine grained, poorly graded, scattered mica flakes throughout, dry to damp, medium dense		3 10 14	13		3.4	96.3			
15–16.5	1-5 T		Very fine to fine grained, poorly graded, dry to damp, medium dense		5 10 16	26		3.1		5.6		
19.5–20			**SAND:** Brownish yellow (10YR 6/6), medium to fine grained, poorly graded, dry to damp, medium dense	SP								
20–21.5	1-6 L	2, 1			8 17 21	20		2.6	103.7			

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 5
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/17/23 | BORING DIAMETER 8" HS | BORING NO. B1

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
23–24			**SAND:** Brownish yellow (10YR 6/6), medium to fine grained, poorly graded, dry to damp, medium dense	SP								
25–26	1-7	T	Slight increase in mica flakes, slightly damp, dense		9 15 22	37		2.9		3.9		
30–31	1-8	T	Less mica flakes, damp to dry, dense		7 11 20	31		3.1		2.7		
35–36	1-9	L (2, 1)	Yellow (10YR 7/6), medium to fine grained, some very fine grains, trace mica, poorly graded, damp, dense (driller added water)		15 27 40	35		3.8	94.0			
40–41	1-10	T	Damp to moist, dense (driller added water)		11 15 24	39		3.5		2.7		
45–46	1-11	L (2, 1)	Moist, medium dense		13 25 34	30		4.3	93.8			

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 6
Project No. 2302
Date: 3/10/23

LOGGED BY JP   DATE DRILLED 1/17/23   BORING DIAMETER 8" HS   BORING NO. B1

DRILL RIG Britton - Track mounted CME 55   HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
			SAND: Yellow (10YR 7/6), medium to fine grained, some very fine grains, trace mica, poorly graded, moist (driller added water)	SP								
	1-12 T		Slight increase in mica flakes, slightly damp, dense		13 19 28	47		4.3		3.6		
			Boring terminated at 51½ feet. No groundwater encountered.									

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 7
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/17/23 | BORING DIAMETER 8" HS | BORING NO. B2

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Dark brown (7.5YR 3/3), fine to very fine grained, poorly graded, rootlets throughout, moist, loose	SM								
1–2.5	2-1 L	2, 1			2 5 5	5		9.4	104.7			
2.5–4	2-2 T		Trace rootlets, moist, loose		1 2 2	4		13.5				
5–6.5	2-3 L	2, 1	Reddish yellow (7.5YR 6/8) at 5½ feet, medium to fine grained, poorly graded, moist, medium dense (driller added water)		3 12 29	22		12.4	106.3			
9.5			**SAND:** Very pale brown (10YR 8/4), medium to fine grained, poorly graded, dry, loose	SP								
10–11.5	2-4 L	2, 1			4 9 11	10		3.1	86.3	3.4		
15–16.5	2-5 T		Medium to fine grained, poorly graded, clean sand, dry, medium dense		4 8 14	22		1.9				
20–21.5	2-6 L	1	Dry, medium dense		12 22 26	25		2.0	82.2			
21.5			Boring terminated at 21½ feet. No groundwater encountered.									

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 8
Project No. 2302
Date: 3/10/23

LOGGED BY JP	DATE DRILLED 1/17/23	BORING DIAMETER 8" HS	BORING NO. B3
DRILL RIG Britton - Track mounted CME 55	HAMMER TYPE Auto-trip		

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Dark brown (7.5YR 3/3), fine to very fine grained, poorly graded, rootlets throughout, moist, loose	SM								
1–2.5	3-1 L	2, 1			2, 1, 1	1		9.4	104.7			
2.5–4	3-2 T		Less rootlets, moist, loose		2, 2, 2	4		11.0				
5–6.5	3-3 L	2, 1	Reddish yellow (7.5YR 6/8), medium to fine grained, poorly graded, moist, loose		3, 5, 7	7		14.4	101.1			
9.5			**SAND:** Yellow (10YR 7/6), medium to fine grained, poorly graded, dry, medium dense	SP								
10–11.5	3-4 T				4, 7, 8	15		2.7		3.5		
15–16.5	3-5 L	2, 1	Dry, medium dense		6, 10, 16	14		1.7	88.3			
16.5			Boring terminated at 16½ feet. No groundwater encountered.									

Depth scale: 1–23 feet.

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 9
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/17/23 | BORING DIAMETER 8" HS | BORING NO. B4

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Black (7.5YR 2.5/1) and very dark brown (7.5YR 2.5/2), fine to very fine grained, poorly graded, abundant rootlets, moist, very loose	SM								
1–2.5	4-1 L	2, 1			1 2 2	2		9.9	94.7			
2.5–4	4-2 T		Moist, loose		1 2 3	5		11.2				
5–6.5	4-3 L	2, 1	Strong brown (7.5YR 4/6), medium to fine grained, poorly graded, silt exhibits low plasticity, moist, loose		4 5 9	7		12.4	105.6			
9.5			**SAND:** Yellow (10YR 7/6), medium to fine grained, poorly graded, damp to dry, loose	SP								
10–11.5	4-4 L	2, 1			3 6 12	9		2.5	82.7	2.2		
15–16.5	4-5 T		Damp to dry, medium dense		4 8 8	16		2.8				
20–21.5	4-6 L	2, 1	Dry, medium dense		7 13 20	17		3.2	96.6			
21.5			Boring terminated at 21½ feet. No groundwater encountered.									

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 10
Project No. 2302
Date: 3/10/23

LOGGED BY JP	DATE DRILLED 1/18/23	BORING DIAMETER 8" HS	BORING NO. B5
DRILL RIG Britton - Track mounted CME 55		HAMMER TYPE Auto-trip	

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Dark brown (7.5YR 3/4), fine to very fine grained poorly graded, rootlets throughout, moist, very loose	SM								
1–2.5	5-1 L	2, 1			1, 1, 2	1		11.8	94.2			
2.5–4	5-2 T		Brown, medium to fine grained, poorly graded, moist, very loose		1, 2, 1	3		12.3				
5–6.5	5-3 L	2, 1	Silt exhibits intermediate plasticity, wet, loose		2, 3, 6	5		11.8	105.2	19.3		
6.5–8	5-4 T		Wet, loose		4, 4, 6	10		13.7		16.0		
9.5			**SAND:** Brownish yellow, medium to fine grained, poorly graded, moist, medium dense	SP								
10–11.5	5-5 L	2, 1			4, 11, 14	13		2.4	96.9			
15–16.5	5-6 T		Very pale brown with brownish yellow, medium to fine grained, poorly graded, slightly damp, medium dense		4, 6, 8	14		3.0				
20–21.5	5-7 L	2, 1	Yellow (10YR 8/6), medium to fine grained, poorly graded, slightly damp to dry, medium dense		7, 15, 19	18		2.9	91.6			

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 11
Project No. 2302
Date: 3/10/23

LOGGED BY JP	DATE DRILLED 1/18/23	BORING DIAMETER 8" HS	BORING NO. B5
DRILL RIG Britton - Track mounted CME 55		HAMMER TYPE Auto-trip	

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
			SAND: Yellow (10YR 8/6), medium to fine grained, poorly graded, slightly damp to dry, medium dense	SP								
24												
25	5-8 L		Dry, medium dense		11							
		2			19							
26		1			30	21		2.3	91.5	3.2		
27			Boring terminated at 26½ feet. No groundwater encountered.									
28												
29												
30												
31												
32												
33												
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 12
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/17/23 | BORING DIAMETER 8" HS | BORING NO. B6

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
			SAND WITH SILT: Dark brown (7.5YR 3/3), fine to very fine grained, poorly graded, rootlests throughout, moist, loose	SP-SM								
1	6-1 L	2, 1			4, 4, 5	5		9.5	101.7			
2												
3	6-2 T		Brown (7.5YR 4/4), some rootlets, moist, loose		1, 1, 2	3		12.7				
4												
5	6-3 L	2, 1	Brownish yellow (10YR 6/6), medium to very fine grained, poorly graded, moist, loose		3, 6, 9	8		13.1	103.0	11.8		
6												
7												
8												
9												
10			**SAND:** Yellow (10YR 7/8), medium to fine grained, poorly graded, dry, medium dense	SP								
	6-4 L	2, 1			6, 9, 12	11		3.7	97.6			
11												
12												
13												
14												
15	6-5 T		Very pale brown (10YR 7/4), medium to fine grained, poorly graded, dry, medium dense		6, 12, 16	28		1.9				
16												
17												
18												
19												
20	6-6 L	2, 1	Coarse to very fine grained, poorly graded, damp to dry, medium dense		10, 22, 26	25		2.7	98.3	1.9		
21												
22												
23												

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 13
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/17/23 | BORING DIAMETER 8" HS | BORING NO. B6

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
24			**SAND:** Very pale brown (10YR 7/4), medium to fine grained, poorly graded, damp to dry, medium dense	SP								
25	6-7 L		Yellow (10YR 7/6), mediium to fine grained, poorly graded, damp to dry, medium dense		8							
		2			17							
26		1			24	22		2.7	97.1			
27			Boring terminated at 26½ feet. No groundwater encountered.									
28												
29												
30												
31												
32												
33												
34												
35												
36												
37												
38												
39												
40												
41												
42												
43												
44												
45												
46												

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 14
Project No. 2302
Date: 3/10/23

LOGGED BY JP	DATE DRILLED 1/18/23	BORING DIAMETER 8” HS	BORING NO. B7
DRILL RIG Britton - Track mounted CME 55		HAMMER TYPE Auto-trip	

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
			SILTY SAND: Brown (7.5YR 4/2), fine to very fine grained, poorly graded, rootlets throughout, moist, very loose	SM								
	7-1 L	2 1			1 1 1	1		10.4	96.0			
	7-2 T		Brown (7.5YR 5/4), medium to fine grained, poorly graded, dry, very loose		1 1 2	3		11.0				
	7-3 L	2 1	Lack of rootlets, moist, loose		2 5 5	5		12.4	104.4	16.7		
			SAND: Yellow (10YR 7/6), medium to fine grained, poorly graded, dry, medium dense	SP								
	7-4 T				7 13 13	26		2.7				
	7-5 L		No sample recovered, medium dense		9 21 24	23						
	7-6 L	1	Dry, medium dense		14 25 31	29		1.3	97.4			
	7-7 T		Dry, medium dense		7 15 23	38		1.3				
			Boring terminated at 23 feet. No groundwater.									

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 15
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/18/23 | BORING DIAMETER 8" HS | BORING NO. B8

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–2	8-1	L (2, 1)	**SILTY SAND:** Dark brown (7.5YR 3/4), fine to very fine grained, poorly graded, rootlets throughout, moist, very loose	SM	1 2 2	2		10.8	95.1			
2.5–4	8-2	T	Brown (7.5YR 4/4), some rootlets, moist, very loose		1 1 3	4		10.1				
5–6.5	8-3	L (2, 1)	Slightly less silt, silt exhibits low plasticity, some rootlets, moist, very loose		1 2 6	4		12.2	100.3	15.6		
6.5–8	8-4	T	**SAND:** Brownish yellow (10YR 6/6), medium to fine grained, poorly graded, slightly damp to dry, medium dense	SP	9 11 14	25		4.1				
10–11.5	8-5	L (2, 1)	Yellowish brown (10YR 5/8), slightly damp to dry, medium dense		14 19 20	21		2.5	97.2			
15–16.5	8-6	T	Very pale brown (10YR 7/4), dry, medium dense		5 10 13	26		1.4				
20–21.5	8-7	L (2, 1)	Slightly moist, medium dense		7 15 36	26		1.6	96.0			
21.5			Boring terminated at 21½ feet. No groundwater encountered.									

Pacific Crest
ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 16
Project No. 2302
Date: 3/10/23

LOGGED BY	DATE DRILLED	BORING DIAMETER	BORING NO.
JP	1/18/23	8" HS	B9

DRILL RIG: Britton - Track mounted CME 55 HAMMER TYPE: Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Dark brown (7.5YR 3/3), fine to very fine grained, poorly graded, wood pieces, rootlets throughout, moist, very loose	SM								
1–2.5	9-1 L	2, 1			1 1 2	1		10.6	97.7			
2.5–4	9-2 T		Decrease in rootlets, moist, very loose		1 1 1	2		10.0				
5–6.5	9-3 L	2, 1	Brown, moist to wet, very loose		1 3 5	4		3.6	95.6	17.2		
6.5–8	9-4 T		Very pale brown (10YR 8/3), sand is medium to fine grained and poorly graded, dry, medium dense		8 12 15	27		3.1		15.5		
9.5			**SAND:** Yellowish brown (10YR 5/8), grades to yellow (10YR 7/6), medium to very fine grained, trace coarse grains, poorly graded, dry, medium dense	SP								
10–11.5	9-5 L	2, 1			16 26 31	19		3.6	100.7			
15–16.5	9-6 T		Very pale brown (10YR 7/4), medium to fine grained, poorly graded, dry, medium dense		4 7 9	16		1.5				
20–21.5	9-7 L	2, 1	Dry, medium dense		10 21 30	26		1.5	93.4	2.7		

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 17
Project No. 2302
Date: 3/10/23

LOGGED BY JP	DATE DRILLED 1/18/23	BORING DIAMETER 8" HS	BORING NO. B9
DRILL RIG Britton - Track mounted CME 55	HAMMER TYPE Auto-trip		

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
			SAND: Very pale brown (10YR 7/4), medium to fine grained, poorly graded, dry, medium dense	SP								
25	9-8 T		Dry, dense		6 13 21	33		1.4				
			Boring terminated at 26½ feet. No groundwater encountered.									
30												
35												
40												
45												

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 18
Project No. 2302
Date: 3/10/23

LOGGED BY MJM　DATE DRILLED 1/18/23　BORING DIAMETER 8" HS　BORING NO. B10

DRILL RIG Britton - Track mounted CME 55　HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1.5	10-1 L	1	**SILTY SAND:** Brown (7.5YR 4/3), fine to very fine grained, poorly graded, trace rootlets throughout, moist, very loose	SM	1 2 1	1		8.5				
2.5–4	10-2 T		**SAND:** Yellowish brown (10YR 5/6), medium to fine grained, poorly graded, trace rootlets throughout, moist, very loose	SP	1 1 2	3		11.9				
5–6.5	10-3 L	1	Lack of rootlets, moist, loose		2 5 6	6		13.1	104.9			
10–11.5	10-4 L	2, 1	Yellow (10YR 7/6), dry, medium dense		8 14 16	16		2.6	97.3	3.0		
15–16.5	10-5 T		Dry, medium dense		5 7 9	16		3.2				
20–21.5	10-6 L	2, 1	Dry, medium dense		7 17 17	18		2.8	96.6			
			Boring terminated at 21½ feet. No groundwater encountered.									

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 19
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/18/23 | BORING DIAMETER 8" HS | BORING NO. B11

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Dark yellowish brown (10YR 3/4), fine to very fine grained, poorly graded, rootlets throughout, moist, loose	SM								
1	11-1 L	2, 1			2							
2					6							
					7	7		8.3	104.9			
3	11-2 T		Moist, loose		2							
					3							
4					3	6		11.9				
5	11-3 L	2, 1	Yellowish brown (10YR 5/8), moist, loose		4							
6					7							
					9	8		12.5	105.0	12.7		
7												
8												
9												
10	11-4 L	2, 1	**SAND:** Very pale brown (10YR 7/4), medium to fine grained, poorly graded, dry, medium dense	SP	6							
11					11							
					16	14		1.5	94.8			
12												
13												
14												
15	11-5 T		Slightly damp to dry, medium dense		5							
16					7							
					9	16		1.5				
17												
18												
19												
20	11-6 L	2, 1	Yellow (10YR 7/6), medium to fine grained, poorly graded, damp to dry, medium dense		7							
21					11							
					18	21		1.8	88.3	2.9		
22			Boring terminated at 21½ feet. No groundwater encountered.									
23												

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 20
Project No. 2302
Date: 3/10/23

LOGGED BY JP | DATE DRILLED 1/17/23 | BORING DIAMETER 8" HS | BORING NO. P1

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
0–1			**SILTY SAND:** Dark brown (7.5YR 3/3), fine to very fine grained, poorly graded, trace rootlets throughout, moist, loose	SM								
1–2.5	P1-1 L	2, 1			2 4 4	4		9.4	100.5			
2.5–4	P1-2 T		Moist, loose		2 2 2	4		12.3				
4.5			**SAND:** Yellowish brown (10YR 5/4), medium to fine grained, poorly graded, slightly damp to dry, medium dense	SP								
5–6.5	P1-3 L	2, 1			3 7 20	14		4.9	99.9	14.4		
7			Boring terminated at 7 feet. No groundwater encountered.									

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 21
Project No. 2302
Date: 3/10/23

Pacific Crest ENGINEERING INC

LOGGED BY JP | DATE DRILLED 1/18/23 | BORING DIAMETER 8" HS | BORING NO. P2

DRILL RIG Britton - Track mounted CME 55 | HAMMER TYPE Auto-trip

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
			SILTY SAND: Dark brown (7.5YR 3/4), fine to very fine grained, poorly graded, trace rootlets throughout, moist	SM								
	P2-1 B							10.2				
	P2-2 B		Moist					10.0				
	P2-3 B		Moist					9.3		16.0		
			Boring terminated at 5 feet. No groundwater encountered.									

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 22
Project No. 2302
Date: 3/10/23

LOGGED BY JP	DATE DRILLED 1/18/23	BORING DIAMETER 8" HS	BORING NO. P3
DRILL RIG Britton - Track mounted CME 55		HAMMER TYPE Auto-trip	

Depth (feet)	Sample	Sample Type	Soil Description	USCS	Field Blow Counts	SPT "N60" Value	Pocket Pen. (tsf)	Moisture Content (%)	Dry Density (pcf)	% Passing #200	Plasticity Index	Additional Lab Results
			SILTY SAND: Dark brown (7.5YR 3/3), fine to very fine grained, poorly graded, trace rootlets throughout, moist	SM								
1	P3-1 B							10.4				
2												
3			**SAND:** Brown (7.5YR 4/3), medium to fine grained, poorly graded, moist to wet	SP								
4												
5	P3-2 B							13.7		19.1		
6			Boring terminated at 6 feet. No groundwater encountered.									
7												
8												
9												
10												
11												
12												
13												
14												
15												
16												
17												
18												
19												
20												
21												
22												
23												

Pacific Crest ENGINEERING INC

Log of Test Borings
Seaside Fire Station No. 2
Seaside, California

Figure No. 23
Project No. 2302
Date: 3/10/23

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL #	416-694	Date:	2/3/2023	Tested By:	PJ	Checked:	PJ
Client:	Pacific Crest Engineering	Project:	Seaside firestation #2			Proj. No:	2302
Remarks:							

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	Soil Visual Description
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
-	2-3-1	-	-	8177	-	4	13	0.0013	6.9	-	7.2	Yellowish Red Clayey SAND

Resistivity	Ohm-cm
Very Corrosive	0-1000
Corrosive	1,000-2,000
Fairly Corrosive	2,000-5,000
Mildly Corrosive	5,000-10,000
Negligible	>10,000

Chloride Concentration	mg/kg
Severe	>1,500
Positive	300-1,500
Negligible	0-300

Sulfate Concentration	mg/kg
Severe	>5,000
Considerable	2,000-5,000
Positive	1,000-2,000
Negligible	0-1,000

pH	
Potential for acid attack on concrete and steel	<5.5

Pacific Crest ENGINEERING INC

Corrosivity Test Summary
Seaside Fire Station No. 2
Seaside, California

Figure No. 24
Project No. 2302
Date: 3/10/22

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL #	416-693	Date:	2/9/2023	Tested By:	PJ	Checked:	PJ
Client:	Pacific Crest Engineering	Project:	Seaside firestation #2			Proj. No:	2302
Remarks:							

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate		pH	ORP	Moisture	
Boring	Sample, No.	Depth, ft.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	At Test	Soil Visual Description
						Dry Wt.	Dry Wt.	Dry Wt.		mv	%	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod	Cal 417-mod.	Cal 643	SM 2580B	ASTM D2216	
-	8-1-1	-	-	13930	-	30	24	0.0024	5.3	-	0.5	Brown SAND w/ Silt, trace organics

Resistivity	Ohm-cm
Very Corrosive	0-1000
Corrosive	1,000-2,000
Fairly Corrosive	2,000-5,000
Mildly Corrosive	5,000-10,000
Negligible	>10,000

Chloride Concentration	mg/kg
Severe	>1,500
Positive	300-1,500
Negligible	0-300

Sulfate Concentration	mg/kg
Severe	>5,000
Considerable	2,000-5,000
Positive	1,000-2,000
Negligible	0-1,000

pH	
Potential for acid attack on concrete and steel	<5.5

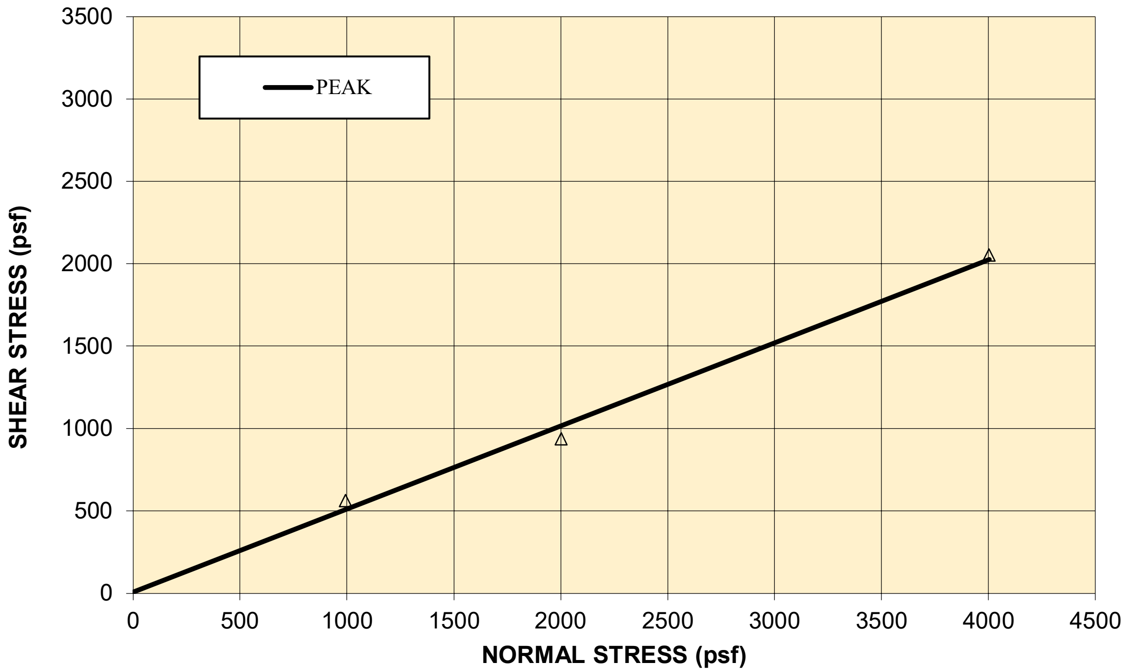
Pacific Crest ENGINEERING INC

Corrosivity Test Summary
Seaside Fire Station No. 2
Seaside, California

Figure No. 25
Project No. 2203
Date: 3/10/22

DIRECT SHEAR TEST - ASTM D3080

Direct Shear Test for Soils Under Consolidated Drained Conditions



SAMPLE:	7-1-1	USCS:	SM
SOIL TYPE:	SILTY SAND		

	ϕ	C (psf)
PEAK	27	0

Initial Sample Data:

Sample:	A	B	C
Sample Diameter (in):	2.42	2.42	2.42
Initial Sample Height (in):	1.000	1.000	1.000
Wet Density (pcf):	108.6	101.0	105.5
Moisture (%):	10.4%	10.4%	10.4%
Dry Density (pcf):	98.4	91.5	95.6
Void Ratio:	0.71	0.84	0.76
% Saturation:	39.4%	33.4%	36.8%

Sample Data At Test:

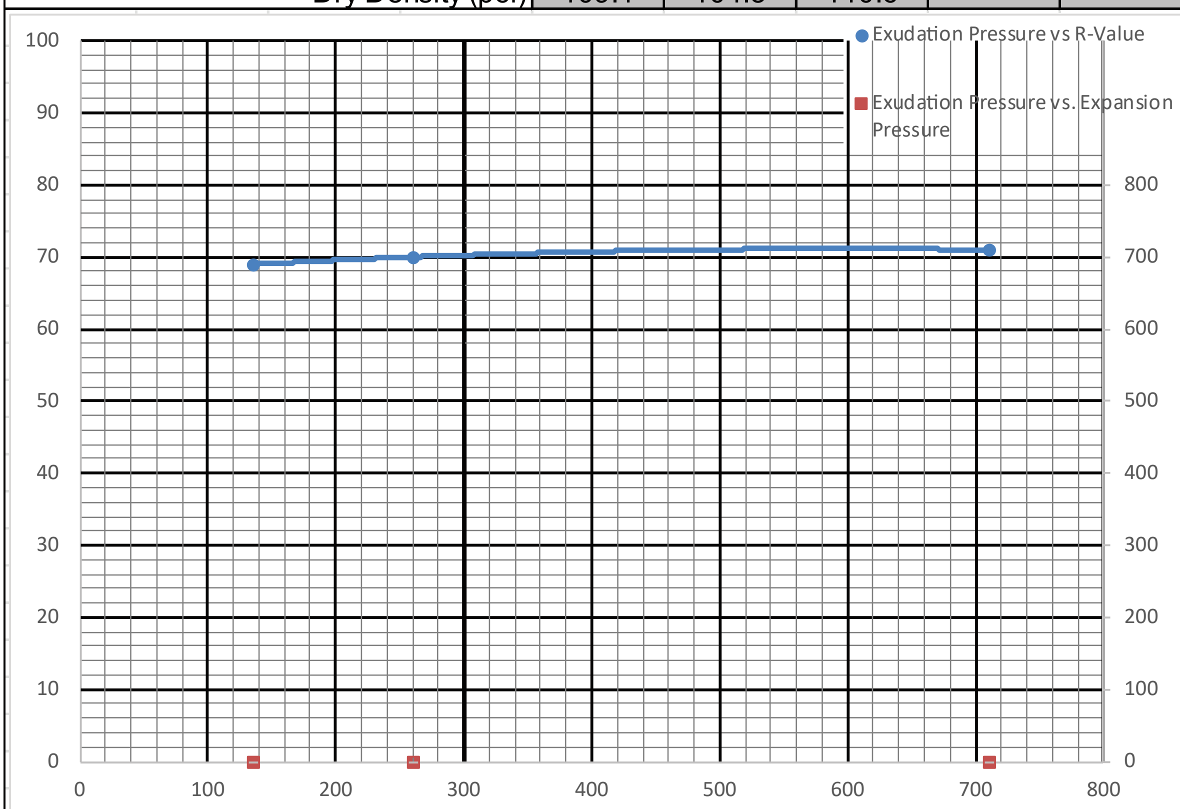
Normal Stress (psf):	994	2002	4003
Sample Height at Test (in):	0.961	0.917	0.902
Wet Density (pcf):	121.7	123.9	128.7
Moisture (%):	21.5%	25.5%	24.3%
Dry Density (pcf):	100.2	98.7	103.5
Void Ratio:	0.69	0.71	0.63
% Saturation:	85.0%	97.2%	104.3%
Strain Rate (in/min):	0.0020	0.0020	0.0020
Peak Shear Stress (psf):	562	936	2052

R-Value
CTM 301

COOPER TESTING LABORATORY

CTL Job No.:	416-695	Boring:	R-1	Reduced By:	RU
Client:	Pacific Crest Engineering	Sample:		Checked By:	PJ
Project Number:	2302	Depth:		Date:	2/17/2023
Project Name:	Seaside Firestation #2			**R-Value**	**70**
Soil Description:	Dark Brown Silty SAND				
Remarks:				**Expansion Pressure**	**0**

Specimen Designation	A	B	C	D	E
Compactor Foot Pressure (psi)	350	50	100		
Exudation Pressure (psi)	711	136	260		
Exudation Load (lbf)	8935	1709	3267		
Height After Compaction (in)	2.33	2.38	2.25		
Expansion Pressure (psf)	0	0	0		
Stabilometer @ 2000	28	28	27		
Turns Displacement	4.24	4.84	4.24		
R-value	74	71	74		
Corrected R-Value	71	69	70		
Moisture Content (%)	11.8	14.1	12.8		
Wet Density (pcf)	122.0	119.1	124.8		
Dry Density (pcf)	109.1	104.3	110.6		



Pacific Crest ENGINEERING INC

R-Value Test Results
Seaside Fire Station No. 2
Seaside, California

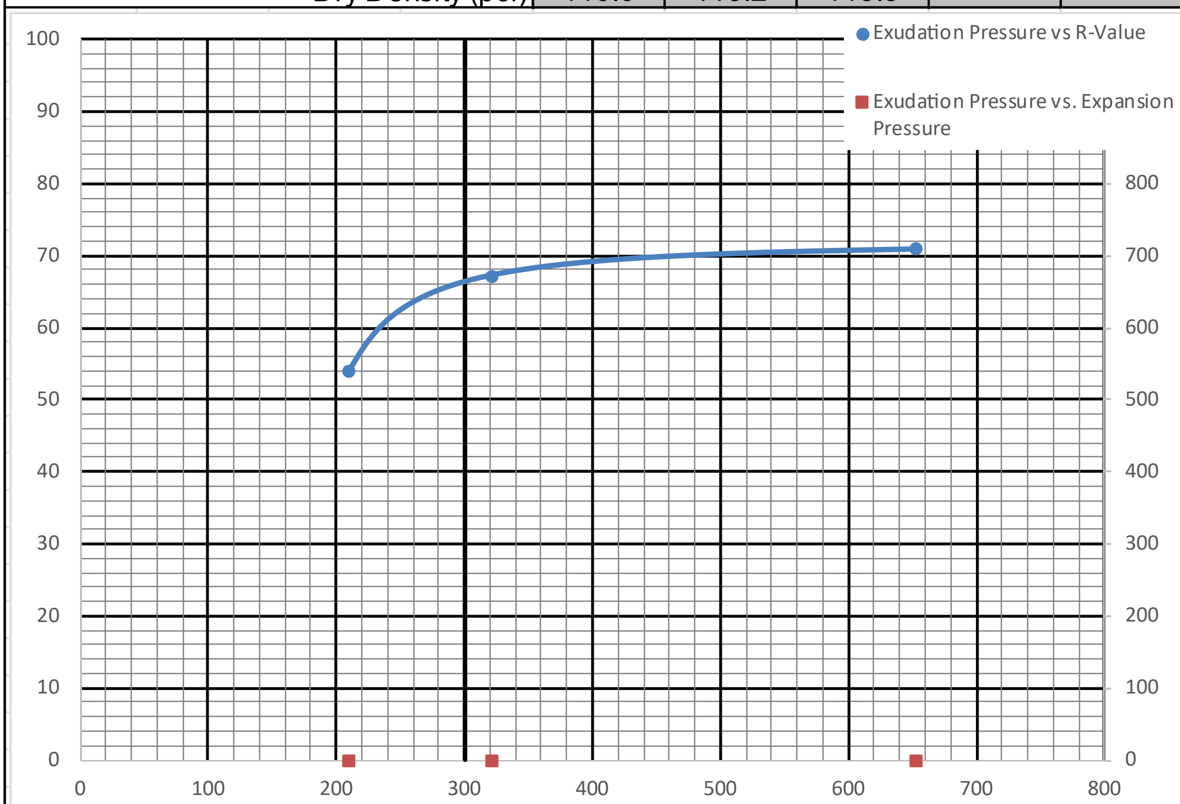
Figure No. 27
Project No. 2203
Date: 3/10/23

COOPER TESTING LABORATORY

R-Value
CTM 301

CTL Job No.:	416-696	Boring:		Reduced By:	RU
Client:	Pacific Crest Engineering Inc.	Sample:	2	Checked By:	PJ
Project Number:	2302	Depth:		Date:	2/21/2023
Project Name:	Seaside Firestation #2			**R-Value**	**66**
Soil Description:	Dark Brown Silty SAND				
Remarks:				**Expansion Pressure**	**0**

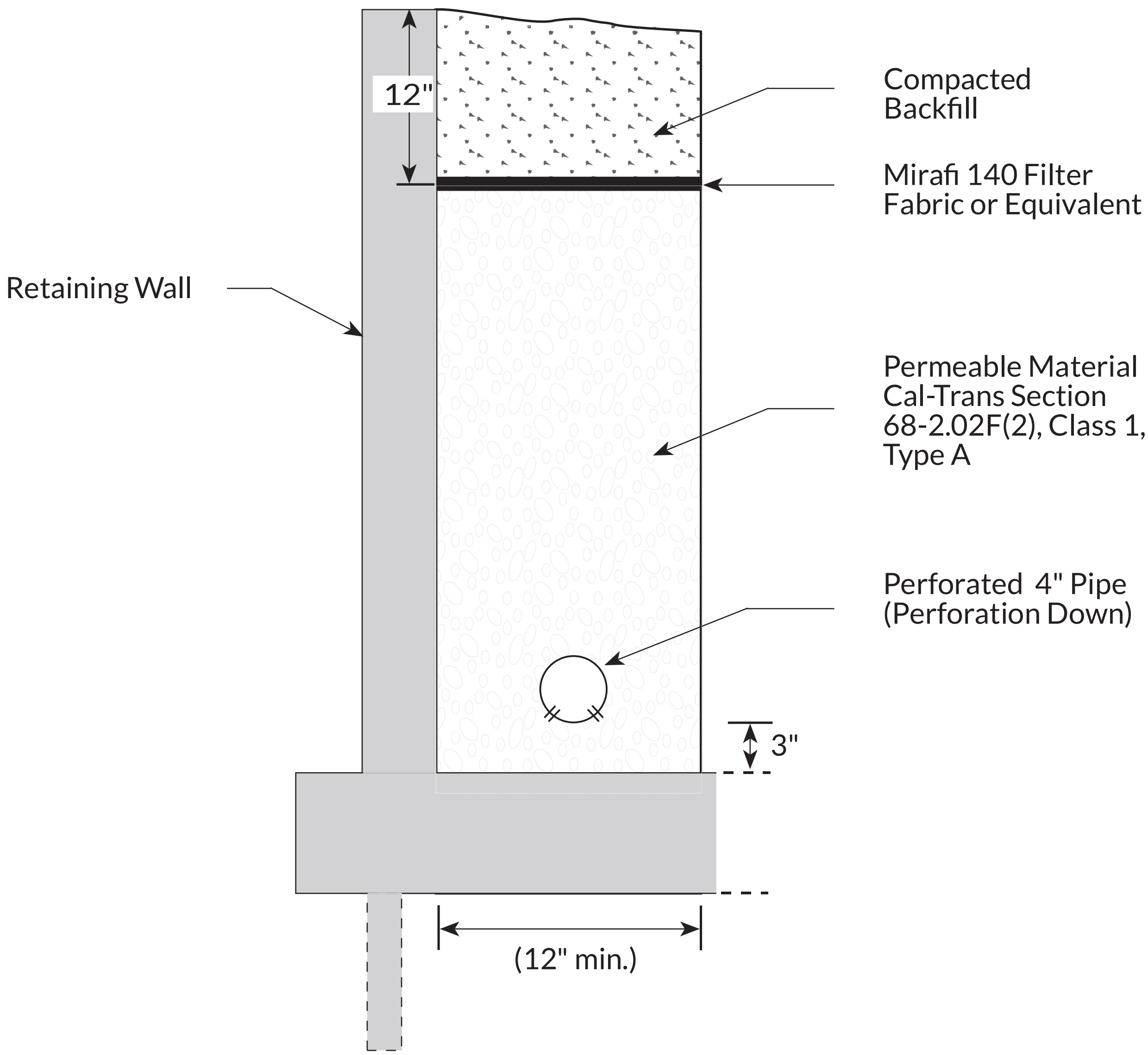
Specimen Designation	A	B	C	D	E
Compactor Foot Pressure (psi)	200	50	350		
Exudation Pressure (psi)	321	210	653		
Exudation Load (lbf)	4034	2639	8206		
Height After Compaction (in)	2.34	2.40	2.40		
Expansion Pressure (psf)	0	0	0		
Stabilometer @ 2000	30	46	28		
Turns Displacement	4.54	4.78	4.38		
R-value	70	56	73		
Corrected R-Value	67	54	71		
Moisture Content (%)	10.3	12.3	9.1		
Wet Density (pcf)	127.9	129.3	125.8		
Dry Density (pcf)	116.0	115.2	115.3		



Pacific Crest ENGINEERING INC

R-Value Test Results
Seaside Fire Station No. 2
Seaside, California

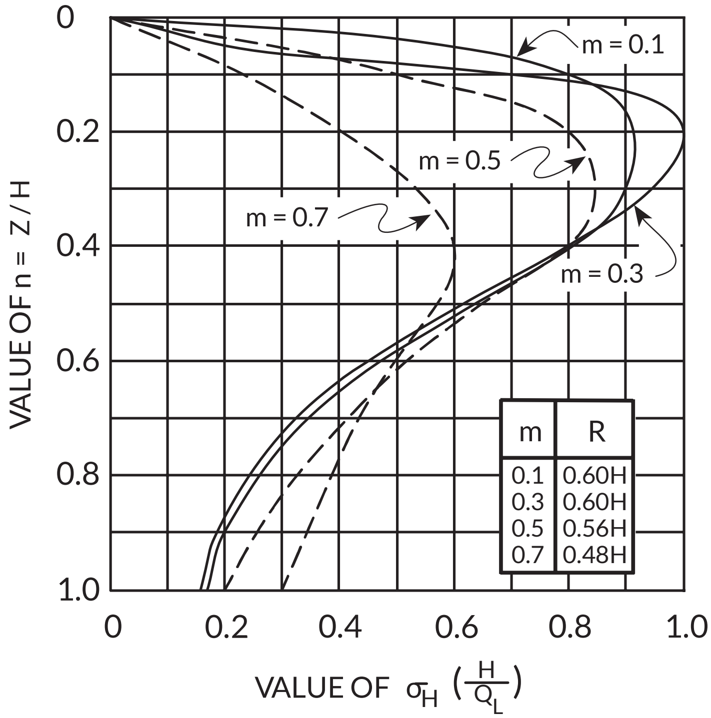
Figure No. 28
Project No. 2203
Date: 3/10/23



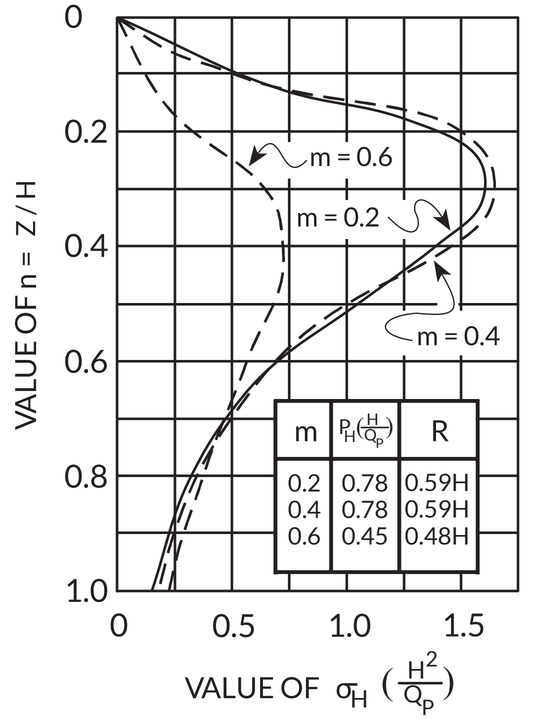
Not to Scale

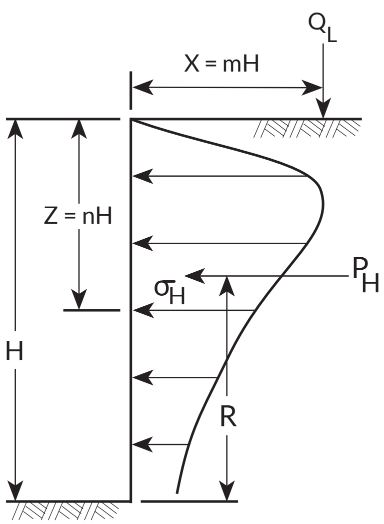
Pacific Crest ENGINEERING INC	**Typical Retaining Wall Drain Detail** Seaside Fire Station No. 2 Seaside, California	Figure No. 29 Project No. 2302 Date: 3/10/23

LINE LOAD



POINT LOAD





FOR m ≦ 0.4:

$$\sigma_H \left(\frac{H}{Q_L}\right) = \frac{0.20\,n}{(0.16 + n^2)^2}$$

$$P_H = 0.55 Q_L$$

FOR m > 0.4:

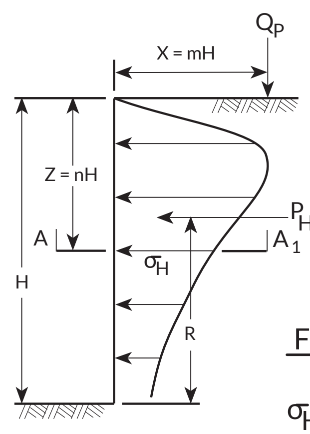
$$\sigma_H \left(\frac{H}{Q_L}\right) = \frac{1.28\,m^2 n}{(m^2+n^2)^2}$$

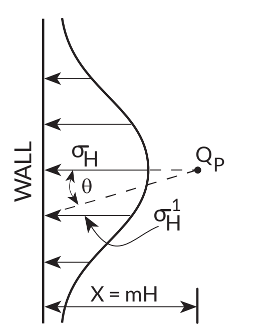
RESULTANT $P_H = \dfrac{0.64 Q_L}{(m^2+1)}$

PRESSURES FROM LINE LOAD Q_L

(BOUSSINESQ EQUATION MODIFIED BY EXPERMENT)

REFERENCE: Design Manual
NAVFAC DM-7.02
Figure 11
Page 7.2-74





SECTION A-A_1

FOR m ≦ 0.4:

$$\sigma_H \left(\frac{H^2}{Q_P}\right) = \frac{0.28\,n^2}{(0.16 + n^2)^3}$$

FOR m > 0.4:

$$\sigma_H \left(\frac{H^2}{Q_P}\right) = \frac{1.77\,m^2 n^2}{(m^2+n^2)^3}$$

$$\sigma_H^1 = \sigma_H \cos^2(1.1\,q)$$

PRESSURES FROM POINT LOAD Q_P

(BOUSSINESQ EQUATION
MODIFIED

Pacific Crest ENGINEERING INC	**Surcharge Pressure Diagram** Seaside Fire Station No. 2 Seaside, California	Figure No. 30 Project No. 2203 Date: 3/10/23

APPENDIX B

Infiltration Test Results

SHALLOW QUICK INFILTROMETER TEST
Native Soil Assessment for Small Infiltration Based Stormwater Control Measures

<table>
<tr><th colspan="8">Test Information</th></tr>
<tr><td>Test No.:</td><td>P-1</td><td>Test Date:</td><td>2/1/2023</td><td>Test By:</td><td>JP</td><td>Job No.:</td><td>2302</td></tr>
<tr><td colspan="2">Location of Test:</td><td colspan="6">P-1, NW area of site near proposed training tower</td></tr>
<tr><th colspan="8">Soil Information</th></tr>
<tr><td>% Gravel</td><td>0.0</td><td>% Sand</td><td>85.6</td><td>% Silt</td><td>14.4</td><td>% Clay</td><td>-</td></tr>
<tr><td colspan="2">USCS Description:</td><td colspan="2">Silty Sand</td><td colspan="2">USCS Classification:</td><td colspan="2">SM</td></tr>
<tr><th colspan="8">Test Configuration & Constants</th></tr>
<tr><td colspan="3">Existing Surface Elevation (ft.)</td><td>170.0</td><td colspan="3">Boring Depth from Top of Pipe (ft.)</td><td>7.6</td></tr>
<tr><td colspan="3">Bioswale Invert Elevation (ft.)</td><td>-</td><td colspan="3">Diameter of Perforated Pipe (in.)</td><td>3.75</td></tr>
<tr><td colspan="3">Bottom of Boring Elevation (ft.)</td><td>163.4</td><td colspan="3">Diameter of Test Boring (in.)</td><td>8.0</td></tr>
<tr><td colspan="3">Boring Depth (ft.)</td><td>6.6</td><td colspan="3">Cross-Section Area of Boring (in^2)</td><td>50.2</td></tr>
<tr><th colspan="8">Constant Head Infiltration Data</th></tr>
<tr><td colspan="2">Interval</td><td>Actual Time (hr:min)</td><td>Interval Time (min)</td><td>Water Head (in)</td><td>Initial Fill Volume (in^3)</td><td>Final Fill Volume (in^3)</td><td>Infiltration Volume (in^3)</td></tr>
<tr><td rowspan="2">0</td><td>Start</td><td>12:36 PM</td><td rowspan="2">30</td><td rowspan="2">24.0</td><td rowspan="2">254.1</td><td rowspan="2">981.8</td><td rowspan="2">2.2</td></tr>
<tr><td>End</td><td>1:06 PM</td></tr>
<tr><th colspan="8">Infiltration Data</th></tr>
<tr><td colspan="2" rowspan="2">Interval</td><td rowspan="2">Actual Time (hr:min)</td><td rowspan="2">Interval Time (min)</td><td colspan="2">Flow Readings</td><td rowspan="2">Infiltration Volume (in^3)</td><td rowspan="2">Infiltration Rate (in/hr)</td></tr>
<tr><td>Water Elev. (in)</td><td>Change in Elev (in)</td></tr>
<tr><td rowspan="14">1</td><td>Start</td><td>1:06 PM</td><td rowspan="2">10.00</td><td>24.00</td><td rowspan="2">1.75</td><td rowspan="2">14.0</td><td rowspan="2">0.84</td></tr>
<tr><td>End</td><td>1:16 PM</td><td>22.25</td></tr>
<tr><td>Start</td><td>1:16 PM</td><td rowspan="2">10.00</td><td>22.25</td><td rowspan="2">2.00</td><td rowspan="2">16.0</td><td rowspan="2">1.03</td></tr>
<tr><td>End</td><td>1:26 PM</td><td>20.25</td></tr>
<tr><td>Start</td><td>1:26 PM</td><td rowspan="2">10.00</td><td>20.25</td><td rowspan="2">2.00</td><td rowspan="2">16.0</td><td rowspan="2">1.13</td></tr>
<tr><td>End</td><td>1:37 PM</td><td>18.25</td></tr>
<tr><td>Start</td><td>1:37 PM</td><td rowspan="2">15.00</td><td>18.25</td><td rowspan="2">1.75</td><td rowspan="2">14.0</td><td rowspan="2">0.72</td></tr>
<tr><td>End</td><td>1:52 PM</td><td>16.50</td></tr>
<tr><td>Start</td><td>1:52 PM</td><td rowspan="2">15.00</td><td>16.50</td><td rowspan="2">1.75</td><td rowspan="2">14.0</td><td rowspan="2">0.79</td></tr>
<tr><td>End</td><td>2:07 PM</td><td>14.75</td></tr>
<tr><td>Start</td><td>2:07 PM</td><td rowspan="2">30.00</td><td>14.75</td><td rowspan="2">3.25</td><td rowspan="2">26.0</td><td rowspan="2">0.86</td></tr>
<tr><td>End</td><td>2:37 PM</td><td>11.50</td></tr>
<tr><td>Start</td><td>2:37 PM</td><td rowspan="2">30.00</td><td>11.50</td><td rowspan="2">2.25</td><td rowspan="2">18.0</td><td rowspan="2">0.73</td></tr>
<tr><td>End</td><td>3:07 PM</td><td>9.25</td></tr>
<tr><td rowspan="2"></td><td></td><td></td><td rowspan="2"></td><td></td><td rowspan="2"></td><td rowspan="2"></td><td rowspan="2"></td></tr>
<tr><td></td><td></td><td></td></tr>
<tr><th colspan="8">Test Results</th></tr>
<tr><td colspan="3">Infiltration Rate, I_t (in/hr):</td><td>0.8</td><td colspan="3">Factored Infiltration Rate, K_m (in/hr):</td><td>0.4</td></tr>
</table>

SHALLOW QUICK INFILTROMETER TEST

Native Soil Assessment for Small Infiltration Based Stormwater Control Measures

Test Information							
Test No.:	P-2	*Test Date:*	2/2/2023	*Test By:*	JP	*Job No.:*	2302
Location of Test:		P-2, South-center area of site near proposed bioswale					

Soil Information							
% Gravel	0.0	*% Sand*	84.0	*% Silt*	16.0	*% Clay*	-
USCS Description:		Silty Sand		*USCS Classification:*		SM	

Test Configuration & Constants			
Existing Surface Elevation (ft.)	173.0	*Boring Depth from Top of Pipe (ft.)*	6.92
Bioswale Invert Elevation (ft.)	-	*Diameter of Perforated Pipe (in.)*	3.75
Bottom of Boring Elevation (ft.)	167.3	*Diameter of Test Boring (in.)*	8.0
Boring Depth (ft.)	5.8	*Cross-Section Area of Boring (in^2)*	50.2

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*	8:50 AM	30	24.0	288.8	1443.8	3.5
	End	9:20 AM					

Infiltration Data

Interval		*Actual Time (hr:min)*	*Interval Time (min)*	*Flow Readings* *Water Elev. (in)*	*Flow Readings* *Change in Elev (in)*	*Infiltration Volume (in^3)*	*Infiltration Rate (in/hr)*
1	*Start*	9:20 AM	10.00	24.00	5.75	46.0	2.98
	End	9:30 AM		18.25			
	Start	9:30 AM	10.00	18.25	5.75	46.0	3.97
	End	9:40 AM		12.50			
	Start	9:40 AM	10.00	12.50	4.00	32.0	3.84
	End	9:50 AM		8.50			
	Start	9:50 AM	10.00	8.50	3.75	30.0	5.22
	End	10:00 AM		4.75			
	Start	10:00 AM	20.00	4.75	6.75	54.0	2.63
2	*End*	10:20 AM		22.00			
	Start	10:20 AM	20.00	22.00	7.50	60.0	2.22
	End	10:40 AM		14.50			
	Start	10:40 AM	20.00	14.50	7.75	62.0	3.68
	End	11:00 AM		6.75			
	Start	11:00 AM	20.00	6.75	7.25	58.0	2.54
	End	11:20 AM		23.50			

Test Results			
Infiltration Rate, I_t (in/hr):	1.9	Factored Infiltration Rate, K_m (in/hr):	0.9

SHALLOW QUICK INFILTROMETER TEST

Native Soil Assessment for Small Infiltration Based Stormwater Control Measures

Test Information							
Test No.:	P-3	*Test Date:*	2/2/2023	*Test By:*	JP	*Job No.:*	22141
Location of Test:		P-3, SW of site near proposed bioswale					
Soil Information							
% Gravel	0.0	*% Sand*	80.9	*% Silt*	19.1	*% Clay*	-
USCS Description:		Silty Sand		*USCS Classification:*		SM	
Test Configuration & Constants							
Existing Surface Elevation (ft.)			165.0	*Boring Depth from Top of Pipe (ft.)*			6.0
Bioswale Invert Elevation (ft.)			-	*Diameter of Perforated Pipe (in.)*			3.75
Bottom of Boring Elevation (ft.)			160.2	*Diameter of Test Boring (in.)*			8.0
Boring Depth (ft.)			4.8	*Cross-Section Area of Boring (in^2)*			50.2

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*	9:24 AM	30	24.5	277.2	981.8	2.1
	End	9:54 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*	9:54 AM	10.00	24.00	2.00	16.0	0.96
	End	10:04 AM		22.00			
	Start	10:04 AM	10.00	22.00	2.00	16.0	1.04
	End	10:14 AM		20.00			
	Start	10:14 AM	10.00	20.00	0.75	6.0	0.42
	End	10:24 AM		19.25			
	Start	10:24 AM	30.00	19.25	1.75	14.0	0.34
	End	10:54 AM		17.50			
	Start	10:54 AM	30.00	17.50	2.00	16.0	0.43
	End	11:24 AM		15.50			
	Start	11:24 AM	30.00	15.50	2.00	16.0	0.48
	End	11:54 AM		13.50			

Test Results

Infiltration Rate, I_t (in/hr):	0.3	Factored Infiltration Rate, K_m (in/hr):	0.1

APPENDIX C

Seismically Induced Settlement Calculations

GEOLOGISMIKI
Geotechnical Software

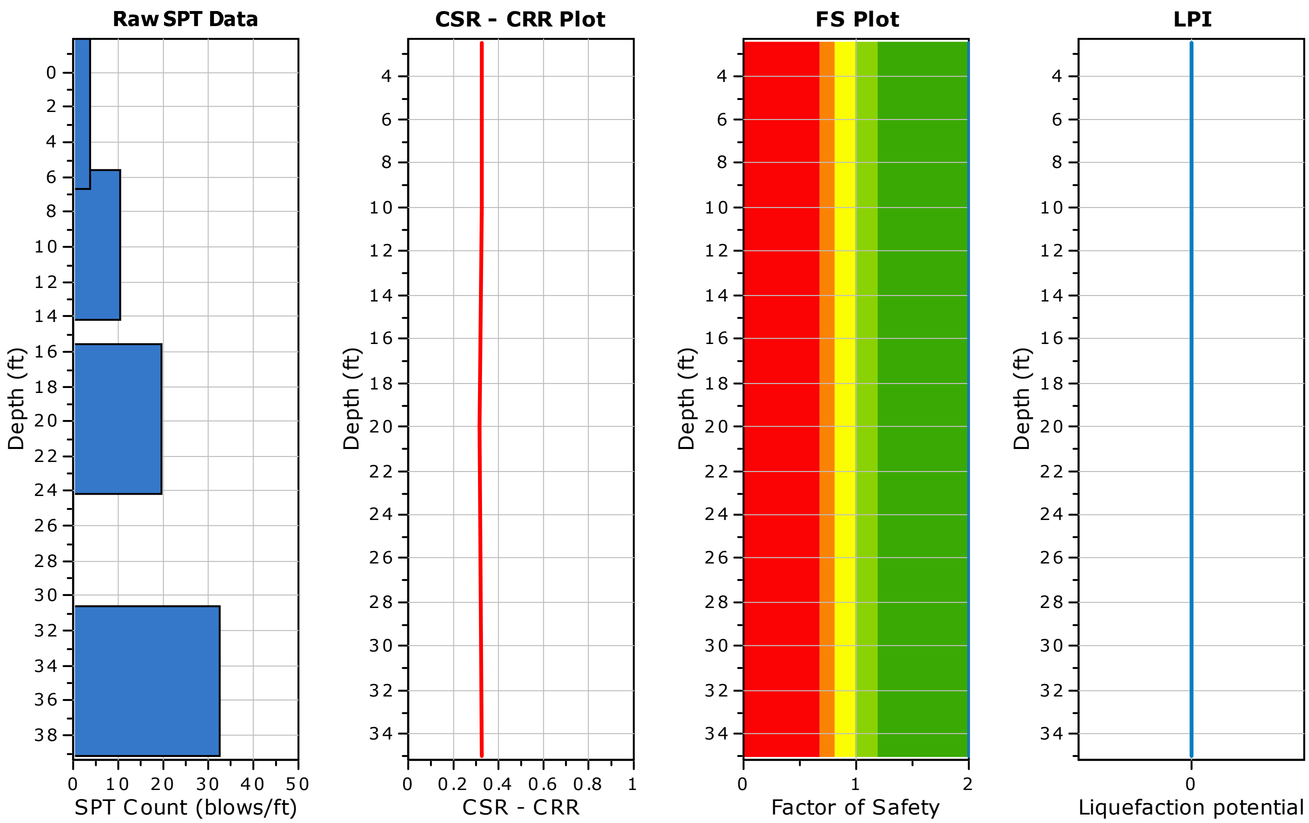
SPT BASED LIQUEFACTION ANALYSIS REPORT

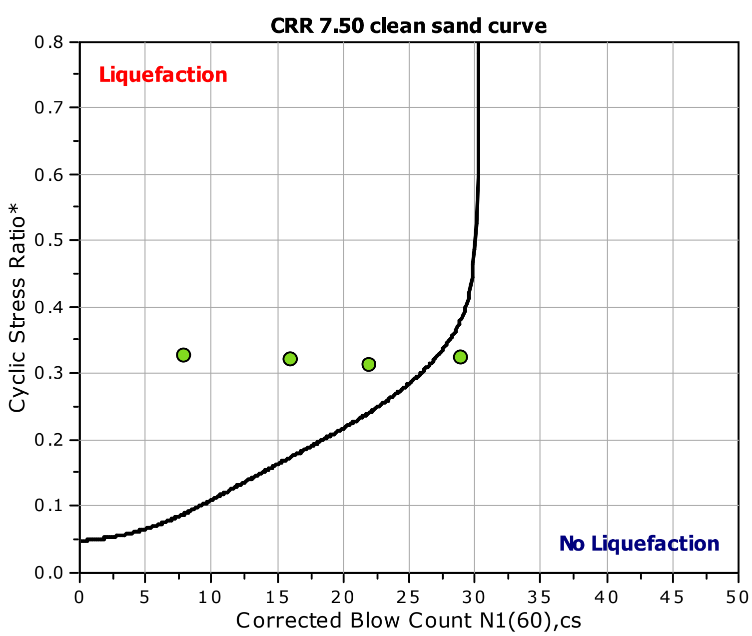
Project title : Seaside Fire Station No. 2 **SPT Name: Site Model**

Location : Seaside, California

:: Input parameters and analysis properties ::

Analysis method:	NCEER 1998	G.W.T. (in-situ):	70.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	70.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	7.10
Borehole diameter:	200mm	Peak ground acceleration:	0.58 g
Rod length:	3.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





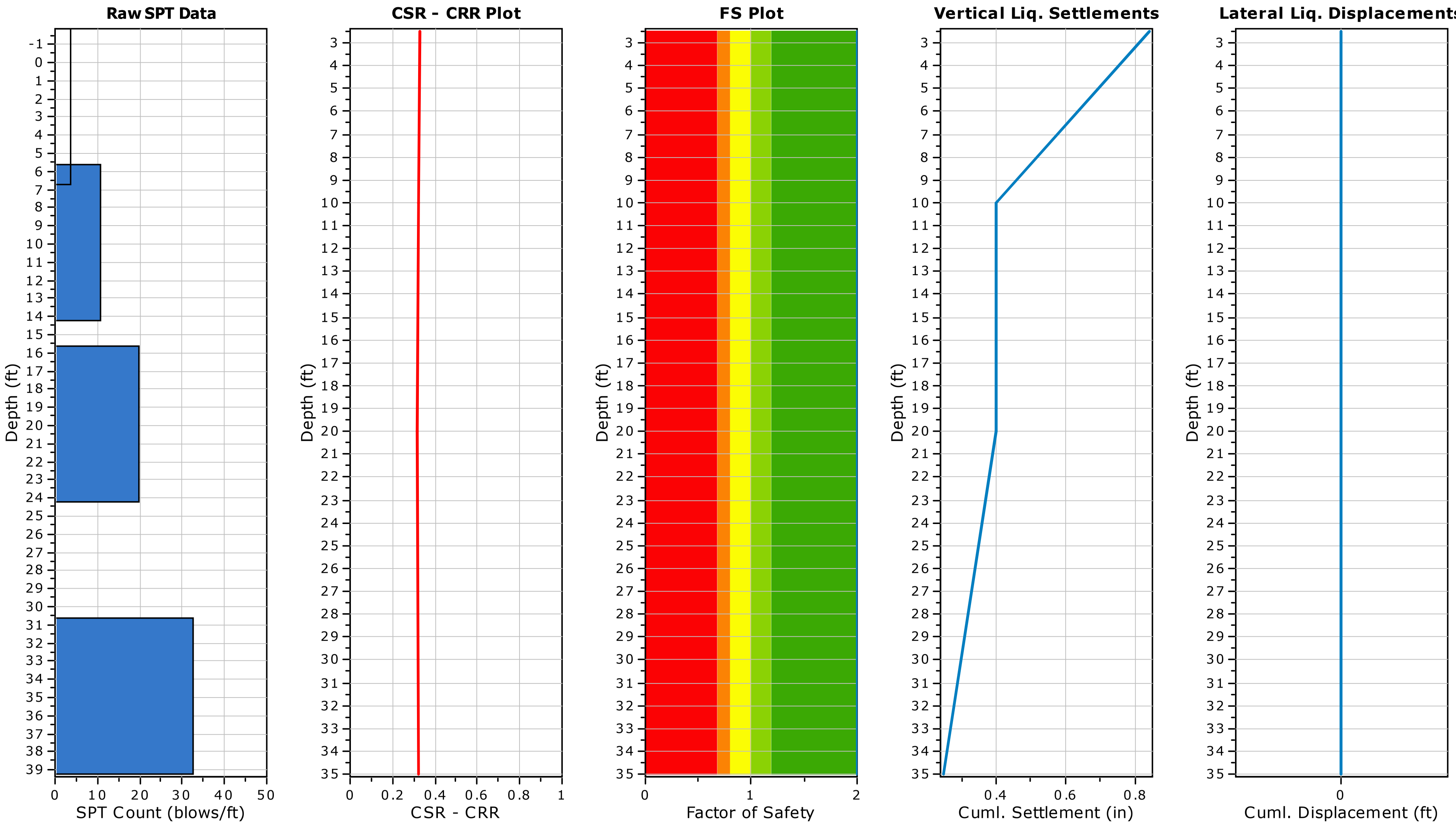
F.S. color scheme

- 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
2.50	4	13.00	111.00	5.00	Yes
10.00	11	15.00	113.00	10.00	No
20.00	20	3.00	97.00	10.00	Yes
35.00	33	3.00	97.00	25.00	Yes

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)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
2.50	4	111.00	0.14	0.00	0.14	1.70	1.00	1.15	0.75	1.00	6	13.00	1.89	1.04	8	4.000
10.00	11	113.00	0.56	0.00	0.56	1.37	1.00	1.15	0.75	1.00	13	15.00	2.50	1.05	16	4.000
20.00	20	97.00	1.05	0.00	1.05	1.01	1.00	1.15	0.95	1.00	22	3.00	0.00	1.00	22	4.000
35.00	33	97.00	1.77	0.00	1.77	0.77	1.00	1.15	1.00	1.00	29	3.00	0.00	1.00	29	4.000

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)
C_N: Overburden corretion 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
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ 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	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
2.50	111.00	0.14	0.00	0.14	1.00	1.00	0.376	1.15	0.327	1.00	0.327	2.000
10.00	113.00	0.56	0.00	0.56	0.98	1.00	0.369	1.15	0.321	1.00	0.321	2.000
20.00	97.00	1.05	0.00	1.05	0.96	1.00	0.361	1.15	0.314	1.00	0.314	2.000
35.00	97.00	1.77	0.00	1.77	0.89	1.00	0.336	1.15	0.292	0.90	0.324	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 (adjusted for improvement)
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
2.50	2.000	0.00	9.62	7.50	0.00
10.00	2.000	0.00	8.48	7.50	0.00
20.00	2.000	0.00	6.95	10.00	0.00
35.00	2.000	0.00	4.67	15.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
2.50	6	0.05	0.09	272.58	0.13	20933.42	0.00	0.00	11.65	0.37	5.00	0.440
10.00	13	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	10.00	0.000
20.00	22	0.38	0.70	1049.29	0.16	6224.26	0.00	0.00	11.65	0.06	10.00	0.151
35.00	29	0.60	1.19	1497.65	0.19	4535.87	0.00	0.00	11.65	0.04	25.00	0.249

Cumulative settlemetns: 0.839

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
α, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
2.50	6	34.29	0.00	5.00	0.000	0.00
10.00	13	50.48	0.00	10.00	0.000	0.00
20.00	22	65.67	0.00	10.00	0.000	0.00
35.00	29	75.39	0.00	25.00	0.000	0.00

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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