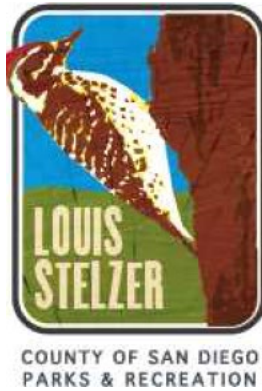


**REPORT
GEOTECHNICAL INVESTIGATION**



Proposed Ranger Station
Louis A. Stelzer County Park, Lakeside, CA

PREPARED FOR



MW Peltz & Associates
143 S. Cedros Avenue, Suite B104
Solana Beach, CA 92075

PREPARED BY



NOVA

NOVA Services, Inc.
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NOVA Project No. 2019133
January 22, 2020



GEOTECHNICAL ■ MATERIALS ■ SPECIAL INSPECTIONS
SBE ■ SLBE ■ SCOOP

MW Peltz & Associates, Inc.
143 S. Cedros Avenue, Suite B104
Solana Beach, California 92075

January 22, 2020
NOVA Project 2019133

Attention: Mr. Michael Peltz
Subject: Report
Geotechnical Investigation
Proposed Ranger Station
Louis A. Stelzer County Park, Lakeside, CA


Dear Mr. Peltz:

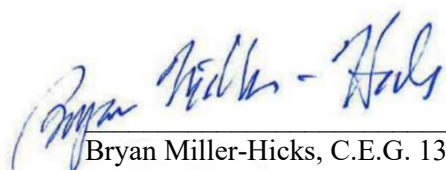
NOVA Services, Inc. (NOVA) is pleased to present herewith the above-referenced report. The work reported herein was completed by NOVA Services, Inc. (NOVA) for MW Peltz & Associates, Inc. (MWPA), in accordance with NOVA's proposal dated January 9, 2019, as authorized on June 11, 2019.

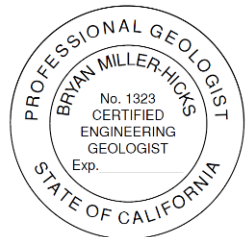
NOVA appreciates the opportunity to be of service to MWPA. Should you have any questions regarding this report or other matters, please do not hesitate to call.

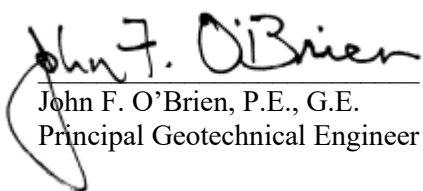
Sincerely,

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REPORT GEOTECHNICAL INVESTIGATION

Proposed Ranger Station
Louis A. Stelzer County Park, Lakeside, CA

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1.0 INTRODUCTION

1.1 Terms of Reference

This report presents the findings of a geotechnical investigation for the replacement and development of a new Ranger Station building at Louis A. Stelzer County Park in Lakeside. The 310-acre park is nominally located at 11470 Wildcat Canyon Road, about 4 miles northeast of Lakeside.

The work reported herein was completed by NOVA Services, Inc. (NOVA) for MW Peltz & Associates Inc. (MWPA), in accordance with the scope of work detailed in NOVA's proposal dated January 9, 2019 as authorized on June 11, 2019.

Figure 1-1 depicts the site vicinity.

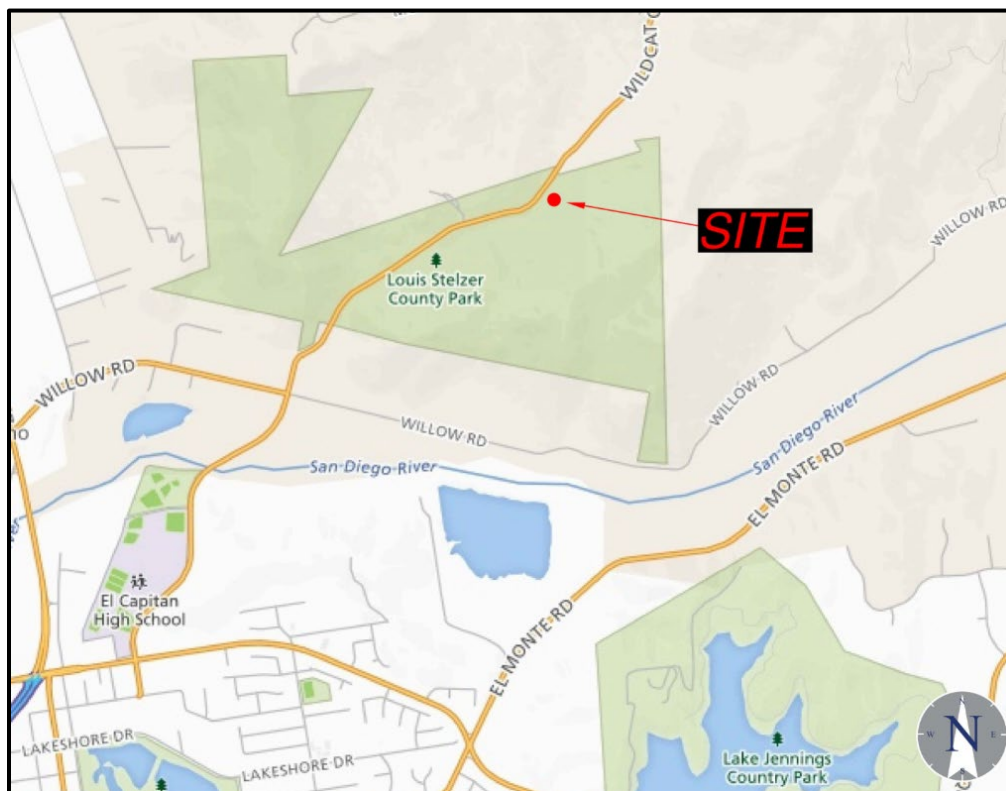


Figure 1-1. Vicinity Map

1.2 Objective, Scope, and Limitations of This Work

1.2.1 Objective

The objective of the work proposed herein is to characterize the subsurface conditions at the location of the proposed replacement building in a manner sufficient to develop recommendations for geotechnical-related development, including foundations and earthwork.

1.2.2 Scope

In order to accomplish the above objectives, NOVA undertook the scope of services as described below.

1. Task 1, Background Review. Reviewed background data, including geotechnical reports, published geologic reports, fault maps, and preliminary planning for the project.
2. Task 2, Subsurface Exploration. The subsurface exploration included the subtasks listed below.
 - *Subtask 2-1, Reconnaissance.* Prior to undertaking any invasive work, NOVA conducted a site reconnaissance, including layout of the test trenches used to explore the subsurface conditions. Underground Service Alert and a utility location contractor were notified for underground utility mark-out services.
 - *Subtask 2-2, Coordination.* NOVA retained a specialty subcontractor to conduct excavation of the test trenches and coordinated access for fieldwork.
 - *Subtask 2-3, Test Trenches.* A NOVA geologist directed the excavation of two (2) test trenches within the location of the proposed building. The trenches extended to dense bedrock at depths of 6 to 7.5 feet below ground surface (bgs), encountering practical backhoe excavation refusal at that point.
 - *Subtask 2-3, Closure.* The trenches were backfilled with the excavated cuttings on completion and the ground in the area restored to the degree practical.
3. Task 3, Laboratory Testing. Laboratory testing of bulk samples was completed. Testing addressed soil gradation, plasticity, moisture content and density, strength, and corrosivity (testing for sulfate and chloride concentrations, pH and resistivity).
4. Task 4, Engineering Evaluations. Conducted engineering evaluations, utilizing field and laboratory information obtained during the preceding tasks.
5. Task 5, Reporting. Preparation of this report presenting NOVA's findings and preliminary geotechnical recommendations completes the scope of work described in NOVA's proposal.

1.2.3 Limitations

The construction recommendations included in this report are not final. These recommendations are developed by NOVA using judgment and opinion and based upon the limited information available from the test trenches. NOVA can finalize its recommendations only by observing actual subsurface conditions revealed during construction. NOVA cannot assume responsibility or liability for the report's recommendations if NOVA does not perform construction observation.

This report does not address any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, groundwater, or surface water within or beyond the site.

Appendix A to this report provides important additional guidance regarding the use and limitations of this report. This information should be reviewed by all users of the report.

1.3 Organization of This Report

The remainder of this report is organized as described below.

- Section 2 reviews the presently available project information.
- Section 3 describes the subsurface exploration and laboratory testing.
- Section 4 describes the geologic, surface and subsurface conditions.
- Section 5 reviews geologic, soil and siting-related hazards common to the area.
- Section 6 provides recommendations for earthwork and foundation design.
- Section 7 provides recommendations for development of pavements.
- Section 8 lists the primary references used in development of this report.

Figures are embedded within the report to amplify the text. A plate depicting the location of NOVA's site work is provided in larger scale following the text of the report.

The report is supported by three appendices. Appendix A provides guidance regarding the use and limitations of this report. Appendix B presents logs of the engineering test trenches. Appendix C provides records of laboratory testing.

2.0 PROJECT INFORMATION

2.1 Location

Louis A. Stelzer County Park is located at 11470 Wildcat Canyon Road, about 4 miles northeast of Lakeside, California. The park is bounded by residential properties to the north, Wildcat Canyon Road to the west, and vacant, steep hillside topography to the east and south. The redevelopment will be to the southeast of the parking lot for the park.

Figure 2-1 provides a current aerial view showing the approximate limits of the proposed improvements.



Figure 2-1. Current Aerial View of Site Location and Limits
(source: adapted from Google Earth 2019)

2.2 Site Description

The park comprises 310 total acres of oak woods and coastal sage scrub. The park includes a parking lot, a ranger station/visitor center, restrooms, playgrounds, a horseshoe pit, picnic facilities, trails, and barbecues. Paved pathways and bridges also are features of the park.

An asphalt paved parking lot, oriented parallel to Wildcat Canyon Road, is set at about elevation +730 feet mean sea level (msl) to about +737 feet msl. The existing Ranger Station/Visitor Center, to be replaced, is on the southeast side of the parking lot. The Ranger Station is a one-story wooden structure.

2.3 Planned Improvements

2.3.1 General

NOVA's understanding of the planned Ranger Station replacement is based upon review of conceptual plans provided by MWPA. That documentation is listed below.

- MWPA 2018a, *Concept Alternative A, Stelzer Park*, M.W. Peltz & Associates, August 15, 2018.
- MWPA 2018b, *Concept Floor Plan, Stelzer Park*, M.W. Peltz & Associates, undated.

MWPA 2018a indicates that the existing ranger station/visitor center will be demolished and replaced with a new 1,000 sf +/- ranger station building. Figure 2-2 presents the Conceptual Plan for improvements to the Ranger Station/Visitor Center.



Figure 2-2. Conceptual Plan for Improvements to the Ranger Station
(source: MPWA 2018a)

2.3.2 Structural

No structural related design information is available at this time. However, it is expected that the replacement ranger station/visitor center will be no more than two levels in height, imposing relatively light loads to foundations.

2.3.3 Potential for Earthwork

Civil design drawings are not yet developed. However, based upon review of MWPA 2018a and MWPA 2018b, it appears that redevelopment of the site will be adapted to the existing ground form, such that the potential for earthwork will be limited to that required for development of foundations and below-grade utilities.

3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Subsurface Exploration

Two (2) exploratory test trenches ('T-1' and 'T-2') were excavated under the surveillance of a NOVA geologist on June 20, 2019. The geologist located the trenches and directed sampling. Samples recovered from the trenches were delivered to NOVA's materials laboratory for analysis.

Figure 3-1 depicts the approximate locations of the test trenches.



Figure 3-1. Test Trench Locations

3.2 Test Trenches

3.2.1 General

Test trenches T-1 and T-2 were excavated with a track-mounted backhoe with a bucket equipped with rock teeth, to depths of 7.5 and 6 feet bgs, respectively. The trenches extended to backhoe refusal on dense Granitic bedrock.

Table 3-1 (following page) provides an abstract of the trenches. Logs of the test trenches are presented in Appendix B.

Table 3-1. Abstract of the Exploratory Test Trenches

Test Trench Reference	Approx. Elevation (feet, msl)	Total Depth (feet)	Approx. Depth to Residual Soil (feet)	Approx. Depth to Formation (feet)
T-1	+725	7.5	3	7.5
T-2	+723	6	2	6

*Note: Elevations are approximated

Figure 3-2 depicts backhoe operations on June 20, 2019.



Figure 3-2. Backhoe Excavation, June 20, 2019

3.2.2 Closure

On completion, the trenches were backfilled with soil cuttings that were lightly tamped with the bucket. The areas where the trenches were excavated were cleaned and left as close to the original condition as practical.

3.3 Laboratory Testing

3.3.1 General

Soil samples were returned to the NOVA geotechnical laboratory following the field investigation. An experienced geotechnical engineer reviewed the exploratory test trench logs and classified the soil samples on the basis of texture and plasticity in accordance with the Unified Soil Classification System, after ASTM D 2488. The group symbols for each soil type are indicated in parentheses following the soil descriptions on the boring log. The stratification lines designating the interfaces between earth materials on the trench logs are approximate.

Representative soil samples were selected for testing in NOVA's materials laboratory to check visual classifications and to determine pertinent engineering properties. The laboratory testing program included index and chemical testing on selected soil samples. Testing was performed in general accordance with ASTM standards.

Records of the geotechnical laboratory testing are provided in Appendix C.

3.3.2 Chemical Testing

Laboratory testing was performed to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. Electrical resistivity, chloride content, and pH level are all indicators of the soil's tendency to corrode ferrous metals. Concentrations of water-soluble sulfates are an index of the potential for sulfates in soils to react with and damage concrete.

4.0 SITE CONDITIONS

4.1 Geologic Setting

4.1.1 Regional

The project area is located in the eastern San Diego County portion of the Peninsular Ranges Geomorphic Province. The geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California. The province varies in width from approximately 30 to 100 miles. The eastern San Diego County portion of the Geomorphic Province consists of rugged mountains and valleys underlain mostly by Jurassic volcanic and sedimentary rocks and Cretaceous igneous rocks of the southern California batholith.

4.1.2 Site

The site is underlain by a few feet of fill and colluvium/alluvium over granitic rocks of Cretaceous Age. Colluvial and alluvial soils in the area are typically silty sand with small amounts of clay. The granitic bedrock is typically dense to very dense and provides excellent bearing capacity for structures. Figure 4-1 reproduces published geologic mapping of the site vicinity.

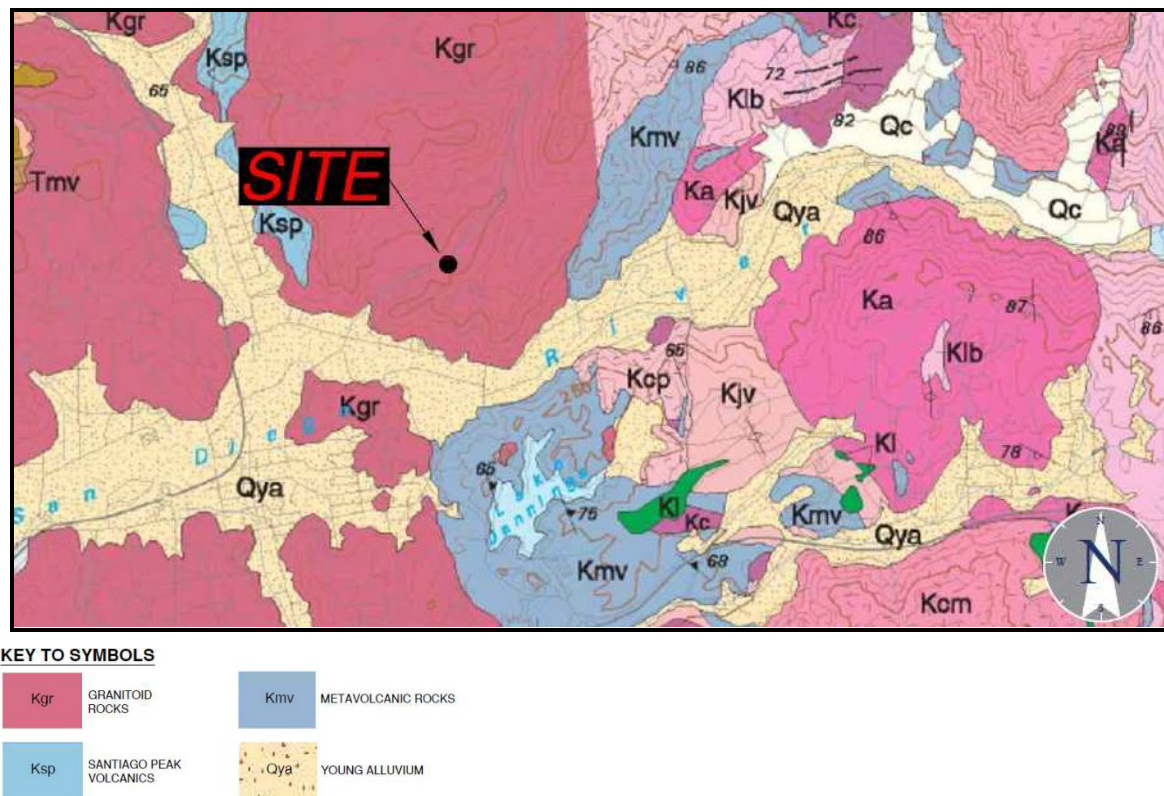


Figure 4-1. Geologic Mapping of the Site Vicinity

4.2 Site Conditions

4.2.1 Surface

The project site is currently occupied by a single-story Ranger Station/Visitor Center with a small shed to the northeast, and associated paved pathways. Large oak trees and dense vegetation surround the structures. Figure 4-2 depicts surface conditions at the time of NOVA's subsurface exploration.



Figure 4-2. Site View, Ranger Station, June 2019

4.2.2 Subsurface

The subsurface disclosed by the test trenches may be generalized to occur as described below.

1. Unit 1, Fill/Colluvium. The site is covered by a mantle of undifferentiated fill/colluvial soils approximately 2 to 3 feet in thickness. Unit 1 is comprised of clayey sand of loose to medium dense consistency, with numerous roots and vegetative matter in the upper two feet.
2. Unit 2, Residual Soil. Residual soil formed from in-place weathering of the granitic rock (Kgr) lies beneath Unit 1. Approximately 4 feet in thickness, this unit is comprised of very stiff/dense reddish-brown sandy clay and clayey sand. Some granitic rock boulders were encountered near the bottom of this unit. Figure 4-3 (following page) depicts a NOVA geologist determining the consistency of this unit by hand probing.
3. Unit 3, Granitic Rock. Granitic bedrock (Kgr) underlies the entire area at approximately 6 to 7.5 feet bgs. Trenching met refusal on this unit where encountered. Figure 4-4 (following page) depicts a granitic rock boulder indicative of this unit.

4.2.3 Groundwater

Groundwater was not encountered in either test trench. It is expected that groundwater first occurs at depths greater than 15 feet below ground at the location of the replacement ranger station/visitor center.



Figure 4-3. Unit 2 Residual Soil, In-Place Probing



Figure 4-4. Granitic Rock Boulder

5.0 REVIEW OF GEOLOGIC, SOIL AND SITING HAZARDS

5.1 Overview

This section provides review of geologic, soil and siting-related hazards that commonly affect civil works in this region. The primary hazard identified by this review is the likelihood of moderate-to-severe ground shaking in response to local moderate or more distant large-magnitude earthquakes during the life of the planned structures. While there is no risk of liquefaction or related phenomena, the occurrence of a major seismic event will affect the site during the life of the planned improvements.

5.2 Geologic Hazards

5.2.1 Strong Ground Motion

The site is about 21 miles east of the Newport-Inglewood-Rose Canyon Fault Zone, which is capable of generating earthquakes with $M > 7$. Figure 5-1 depicts this setting. The Peak Ground Acceleration (PGA) for the site was evaluated utilizing a web-based tool supported by ASCE, estimating the PGA at $\sim 0.34 g$.



Figure 5-1. Faulting in the Site Area

5.2.2 Faulting and Fault Rupture

The site is not located within a currently designated Alquist-Priolo Earthquake Zone. No known active faults are mapped on the site area. No active faults are known to underlie the site. As may be seen by review of Figure 5-1, mapped active faulting is located well to the west of Louis A. Stelzer County Park.

Because of the lack of known active faults that transect the site, the potential for surface rupture at the site is considered very low. Shallow ground rupture due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

5.2.3 Landslide

As used herein, ‘landslide’ describes downslope displacement of a mass of rock, soil, and/or debris by sliding, flowing, or falling. Such mass earth movements are greater than about 10 feet thick and larger than 300 feet across. Landslides typically include cohesive block glides and disrupted slumps that are formed by translation or rotation of the slope materials along one or more slip surfaces. These mass displacements can also include similarly larger-scale, but more narrowly confined modes of mass wasting such as rock topples, ‘mud flows’ and ‘debris flows’.

The causes of classic landslides start with a preexisting condition- characteristically, a plane of weak soil or rock- inherent within the rock or soil mass. Thereafter, movement may be precipitated by earthquakes, wet weather, and changes to the structure or loading conditions on a slope (e.g., by erosion, cutting, filling, release of water from broken pipes, etc.). Rainfall is the most common trigger for landslide events. In the San Diego area, landslide has been also been precipitated by a larger-scale earthwork, by destabilizing slopes by the cutting and/or filling on existing adverse geologic structure.

In assessment of this hazard, NOVA conducted a geologic reconnaissance of the site and reviewed aerial photography for indications of landslide instability at the site. This review indicated no evidence of active or dormant landsliding. Clues to the landslide hazard for an area can also be obtained by review of mapping that depicts both historic landslides and landslide-prone geology/topography. Review of this mapping indicates that the site is in an area judged ‘generally susceptible’ to landsliding.

In consideration of the indications of the geologic reconnaissance, review of published mapping, and review of aerial photography, NOVA considers the landslide hazard at the site to be ‘low’ for the site and the surrounding area in their current condition.

5.3 Soil Hazards

5.3.1 Embankment Stability

As used herein, ‘embankment stability’ is intended to mean the safety of localized natural or man-made embankments against failure. Unlike landslides described above, embankment stability can include smaller scale slope failures such as erosion-related washouts and more subtle, less evident processes such as soil ‘creep’.

No new slopes are planned as part of the future site development. Existing slopes in and around the area of the existing ranger station will not be modified by new construction, are well stabilized, and have been stable for 20 years. There is no concern regarding embankment stability at this site.

5.3.2 Liquefaction

General

‘Liquefaction’ refers to the loss of soil strength during a seismic event. The phenomenon is observed in areas that include geologically ‘younger’ soils (i.e., soils of Holocene age), shallow water table (less than about 60 feet depth), and cohesionless (i.e., sandy and silty) soils of looser consistency. The seismic ground motions increase soil water pressures, decreasing grain-to-grain

contact among the soil particles, which causes the soils to lose strength. Resistance of a soil mass to liquefaction increases with increasing density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history.

Neither the Unit 1 fill/colluvium, clayey Unit 2 residual soil, nor the very dense and geologically 'older' Unit 3 granitic bedrock have potential for liquefaction.

Lateral Spreading

Lateral spreading is a phenomenon in which large blocks of intact, non-liquefied soil move downslope on a liquefied soil layer. Lateral spreading is often a regional event. For lateral spreading to occur, a liquefiable soil zone must be laterally continuous, unconstrained laterally, and free to move along sloping ground. It is likely that some lateral spreading would be associated with major seismic events.

Seismically Induced Settlement

During a strong seismic event, seismically induced settlement (by densification) can occur within loose to moderately dense, unsaturated granular soils, a mechanism apart from liquefaction. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

The limited thickness of the unsaturated Unit 1 will limit any such settlement to immeasurably small levels (i.e., less than about 0.2 inch).

5.3.3 Expansive Soil

Expansive soils are clayey soils characterized by their ability to undergo significant volume changes (shrinking or swelling) due to variations in moisture content, the magnitude of which is related to both clay content and plasticity index. These volume changes can be damaging to structures. Nationally, the annual value of real estate damage caused by expansive soils is exceeded only by that caused by termites.

As is discussed in Section 3, the Unit 1 and Unit 2 soils have been characterized by testing to determine Expansion Index ('EI', after ASTM D 4829). Originally developed in Orange County in the 1960s, EI has been adopted by the California Building Code ('CBC', Section 1803.5.3) for characterization of expansive soils. The listing below tabulates qualitative descriptors of expansion potential based upon EI.

Table 5-1. Qualitative Descriptors of Expansion Potential Based Upon EI

Expansion Index ('EI'), ASTM D 4829	Expansion Potential, ASTM D 4829	Expansion Classification, 2016 CBC
0 to 20	Very Low	Non-Expansive
21 to 50	Low	Expansive
51 to 90	Medium	
91 to 130	High	
>130	Very high	

As is reported in Appendix C, the clayey Unit 1 fill and the Unit 2 residual soils are of Very Low to Low expansion potential, with Expansion Indexes ranging from of EI = 10 to EI = 30.

5.3.4 Hydro-Collapsible Soils

Hydro-collapsible soils are common in the western United States in specific depositional environments (principally, in arid areas of young alluvial fans, debris flow sediments, and loess (wind-blown sediment)) deposits. Such soils are characterized by low *in situ* density, low moisture contents, and relatively high unwetted strength. While relatively strong in a dry state, the introduction of water to these soils causes the binding agents to fail. Destruction of the bonds/binding causes relatively rapid densification and volume loss (collapse), a change manifested by subsidence or settlement.

The depositional history and observed condition of the subsoils in the vicinity of the ranger station indicate no potential for hydro-collapsible soils.

5.3.5 Corrosive Soils and Sulfate Attack

The tested soils have low levels of soluble sulfates, such that these soils will not be a risk for attack to embedded concrete foundations. The tested soils indicated resistivity that suggests the soils will be corrosive to exposed ferrous metals. Section 6 discusses the above considerations in more detail.

5.4 Siting Hazards

5.4.1 Flooding

The site is located within a FEMA-designated flood zone, designated as Flood “Zone X” (FEMA, 2012). Zone X depicts an area of minimal flood hazard. Figure 5-2 depicts flood mapping of the site area.



Figure 5-2. Flood Mapping of the Site Area

5.4.2 Tsunami

Tsunami (‘harbor wave’) describes a series of fast-moving, long-period ocean waves caused by earthquakes or volcanic eruptions. Tsunami can be the source of extensive coastal flooding. Based on the site’s location inland and high elevation, tsunamis will not affect the site.

5.4.3 Seiche

Seiches are standing waves that develop in an enclosed or partially enclosed body of water such as lakes or reservoirs. Harbors or inlets can also develop seiches. Most commonly caused by wind and atmospheric pressure changes, seiches can be effected by seismic events and tsunamis. There is no body of water near the site that could generate a seiche.

6.0 EARTHWORK AND FOUNDATIONS

6.1 Overview

6.1.1 Review of Site Hazards

Section 5 provides a review of soil, geologic and siting hazards common to development of civil works in the project area. The primary hazard identified by that review is that the site is at risk for moderate-to-severe ground shaking in response to a large-magnitude earthquake during the lifetime of the planned development. While strong ground motion could affect the site, there is no risk of liquefaction or related seismic phenomena. The expectation of strong ground motion is common to all civil works in this area of California.

Section 6.2 addresses design parameters to adapt structures to seismic shaking.

6.1.2 Site Suitability

Based upon the indications of the field and laboratory data developed for this investigation, it is the judgment of NOVA that the site is suitable for development of the replacement ranger station and visitor center utilizing shallow foundations established on compacted fill.

Development as presently envisioned will not affect the structural integrity of adjacent properties or existing public improvements and street right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

6.1.3 Review and Surveillance

The subsections following provide geotechnical recommendations for the planned development as it is now understood. It is intended that these recommendations provide sufficient geotechnical information to develop the project in general accordance with the 2019 California Building Code (CBC) requirements.

NOVA should be given the opportunity to review the grading plan, foundation plan, and geotechnical-related specifications as they become available to confirm that the recommendations presented in this report have been incorporated into the plans prepared for the project. All earthwork related to site and foundation preparation should be completed under the observation of NOVA.

6.2 Seismic Design Parameters

6.2.1 Site Class

The site-specific data used to determine the Site Class typically includes borings drilled to refusal materials to determine Standard Penetration resistances (N-values). Though the depth of soil information available for this site is limited, the site is known to be underlain by a variety of dense granitics to great depth, such that the site is classified as Site Class C per ASCE 7 (Table 20.3-1).

6.2.2 Seismic Design Parameters

Table 6-1 (following page) provides seismic design parameters for the site in accordance with 2019 CBC.

Table 6-1. Seismic Design Parameters, ASCE 7-16

Parameter	Value
Site Soil Class	C
Site Latitude (decimal degrees)	32.883473
Site Longitude (decimal degrees)	-116.896683
Site Coefficient, F_a	1.2
Site Coefficient, F_v	1.5
Mapped Short Period Spectral Acceleration, S_s	0.788 g
Mapped One-Second Period Spectral Acceleration, S_1	0.288 g
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	0.945 g
One-Second Period Spectral Acceleration Adjusted For Site Class, S_{M1}	0.432 g
Design Short Period Spectral Acceleration, S_{DS}	0.635 g
Design One-Second Period Spectral Acceleration, S_{D1}	0.288 g

Source: *ASCE 7-16 Hazard Tool*, found at: <https://asce7hazardtool.online/>

6.3 Metals Corrosivity and Sulfates

6.3.1 General

Electrical resistivity, chloride content, and pH level are all indicators of the soil's tendency to corrode ferrous metals. Concentrations of water-soluble sulfates are an index of the potential for sulfates in soils to react with and damage concrete. Chemical tests were performed on a representative soil sample, the results of which are tabulated on Table 6-2. Records of this testing are provided in Appendix C.

Table 6-2. Summary of Corrosivity Testing of the Near Surface Soil

Parameter	Units	Value
pH	standard unit	5.5
Resistivity	Ohm-cm	5,700
Water-Soluble Chloride	ppm	21
Water Soluble Sulfate	ppm	36

6.3.2 Metals

Caltrans considers a site to be 'corrosive' to unprotected embedded metals if one or more of the following conditions exist for representative soil and/or water samples taken at the site:

- chloride concentration is 500 parts per million (ppm) or greater;
- sulfate concentration is 2,000 ppm or greater;
- the pH is 5.5 or less;
- minimum resistivity is less than 2,000 ohm-cm.

Based on the Caltrans criteria, the on-site soils are considered ‘corrosive’ to unprotected metals embedded in the site soil.

6.3.3 Sulfates

As shown on Table 6-1, the soil sample tested indicated water-soluble sulfate (SO_4) content of 36 parts per million (‘ppm,’ 0.0036% by weight). With $\text{SO}_4 < 0.10$ percent by weight, the American Concrete Institute (ACI) 318-08 considers a soil to have no potential for sulfate attack to concrete (Class S0).

Table 6-3 shows the Exposure Categories considered by ACI.

Table 6-3. Exposure Categories and Requirements for Water-Soluble Sulfates

Exposure Category	Class	Water-Soluble Sulfate (SO_4) In Soil (percent by weight)	Cement Type (ASTM C150)	Max Water-Cement Ratio	Min. f'_c (psi)
Not Applicable	S0	$\text{SO}_4 < 0.10$	-	-	-
Moderate	S1	$0.10 \leq \text{SO}_4 < 0.20$	II	0.50	4,000
Severe	S2	$0.20 \leq \text{SO}_4 \leq 2.00$	V	0.45	4,500
Very severe	S3	$\text{SO}_4 > 2.0$	V + pozzolan	0.45	4,500

Adapted from: ACI 318-08, Building Code Requirements for Structural Concrete

6.3.4 Limitations

Testing to determine several chemical parameters that indicate a potential for soils to be corrosive to construction materials are traditionally completed by the Geotechnical Engineer, comparing testing results with a variety of indices regarding corrosion potential. Like most geotechnical consultants, NOVA does not practice corrosion engineering. Should you require more information, a specialty corrosion consultant should be retained to address these conditions.

6.4 Earthwork

6.4.1 General

As is noted in Section 2, no structural related design information is available at this time. However, based on the known condition of the site and the design concept that is currently considered, NOVA expects that earthwork will be limited to excavations for foundations and utilities.

Earthwork should be performed in accordance with Section 300 of the most recent approved edition of the “*Standard Specifications for Public Works Construction*” and “*Regional Supplement Amendments*.”

6.4.2 Select Fill

Materials

All structural fill should be Select Fill, a mineral soil free of organics and environmentally regulated constituents, with the characteristics listed below:

1. at least 40 percent by weight finer than ¼-inches in size;
2. maximum particle size of 4 inches; and,
3. expansion index (EI) of less than 50 (i.e., $\text{EI} < 50$, after ASTM D 4829).

Much of the Unit 1 will meet the above criteria. Any imported soils will need to be sampled and tested by NOVA to confirm suitability as Select Fill prior to use. NOVA should be afforded at least 72 hours to perform testing on all imported soils.

Compaction

Select Fill should be compacted to a minimum of 90 percent relative compaction after ASTM D1557 (the 'modified Proctor') following moisture conditioning to at least 2% above the optimum moisture content.

Select Fill should be placed in loose lifts no thicker than the ability of the compaction equipment to thoroughly densify the lift. For most self-propelled construction equipment, this will limit loose lifts to on the order of 10-inches or less. Lift thickness for hand-operated equipment (tamper, walked behind compactors, etc.) will be limited to on the order of 4 inches or less.

6.4.3 Site Preparation

Prior to the start of earthwork, the site should be cleared of structures, utilities, vegetation and related root systems, and existing pavement. The deleterious materials should be disposed of in approved off-site locations.

Any existing utilities which are to be abandoned should either be (i) excavated and the trenches backfilled; or, (ii) the lines completely filled with sand-cement slurry.

6.4.4 Foundation Preparation

Ground Supported Slab

The ground supported slab of the structure may be supported at grade following removal of the upper 24 inches of Unit 1 fill/colluvium beneath the finished floor level and backfilling of this removal to finish pad grade with Select Fill that conforms with Section 6.4.2.

The excavation for the on-grade slab will expose stiff, clayey Unit 2 residual soils that are sensitive to construction disturbance and to moisture. Once excavation is complete, the exposed surface should be inspected for areas of unusual softness, wetness, or disturbance. Prior to placing new fill, the bottom of the excavations should be moisture conditioning to at least 3% above the optimum moisture content and compacted to a minimum of 90 percent relative compaction after ASTM D1557 (the 'modified Proctor').

Isolated and Continuous Foundations

Isolated and continuous foundations should be supported on either Unit 2 residual soils or Select Fill. The upper surface of ground loosened by excavation should be redensified to at least 90% relative compaction after ASTM D 1557.

6.4.5 Stabilization of Subgrade Disturbed by Construction

The excavations for the on-grade slab and foundations will expose stiff, clayey Unit 2 soils that are sensitive to construction disturbance and to moisture. Once excavation is complete, the exposed surface should be inspected for areas of unusual softness, wetness or disturbance. Prior to placing new fill, the bottom of the excavations should be moisture conditioning to at least 3% above the optimum moisture

content and compacted to a minimum of 90 percent relative compaction after ASTM D1557 (the 'modified Proctor').

In the event compaction of the clay cannot be achieved, the bottom of removals can be stabilized by the use of at least 6 inches of Caltrans Class II base material or ¾-inch crushed rock. A representative of NOVA should approve the bottoms prior to placement of backfills.

6.4.6 Trenching and Backfilling for Utilities

Excavation for utility trenches must be performed in conformance with OSHA regulations contained in 29 CFR Part 1926.

Utility trench excavations have the potential to degrade the properties of the adjacent soils. Utility trench walls that are allowed to move laterally will reduce the bearing capacity and increase settlement of adjacent footings and overlying slabs.

Backfill for utility trenches is as important as the original subgrade preparation or engineered fill placed to support either a foundation or slab. Backfill for utility trenches must be placed to meet the project specifications for the engineered fill of this project. Unless otherwise specified, the backfill for the utility trenches should be placed in 4 to 6-inch loose lifts and compacted to a minimum of 90 percent relative compaction after ASTM D 1557 (the 'modified Proctor') at soil moisture +2 percent of the optimum moisture content. Up to 4 inches of bedding material placed directly under the pipes or conduits placed in the utility trench can be compacted to 90 percent relative compaction with respect to the Modified Proctor.

6.5 Foundations

6.5.1 Conventionally Reinforced Concrete Slab

The structure may employ conventional on-grade (ground-supported) slab designed to bear on the Unit 2 residual soil following subgrade preparation as described in Section 6.3 and using a modulus of subgrade reaction (k) of 90 pounds per cubic inch (i.e., $k = 90 \text{ pci}$).

The actual slab thickness and reinforcement should be designed by the Structural Engineer. NOVA recommends the slab be a minimum 6 inches thick, reinforced by at least #3 bars placed at 16 inches on center each way within the middle third of the slabs by supporting the steel on chairs or concrete blocks ("dobies").

Minor cracking of concrete after curing due to drying and shrinkage is normal. Cracking is aggravated by a variety of factors, including high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due during curing. The use of low-slump concrete or low water/cement ratios can reduce the potential for shrinkage cracking.

To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or 'weakened plane' joints at frequent intervals. Joints should be laid out to form approximately square panels and never exceeding a length to width ratio of 1.5 to 1. Proper joint spacing and depth are essential to effective control of random cracking. Joints are commonly spaced at distances equal to 24 to 30 times the slab thickness. Joint spacing that is greater than 15 feet should include the use of load transfer devices (dowels or diamond plates). Contraction/control joints should be established to a depth of ¼ the slab thickness as depicted in Figure 6-1.

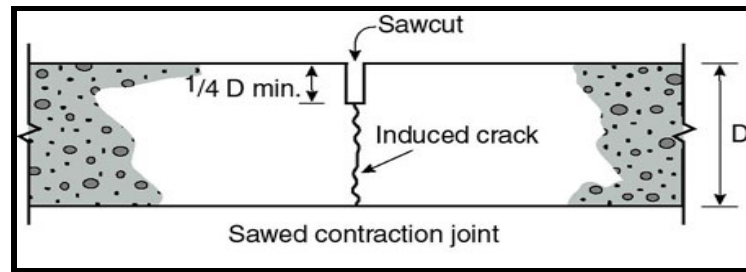


Figure 6-1. Sawed Contraction Joint

6.5.2 Conventional Shallow Foundations

Conventional foundations, consisting of isolated and continuous footings founded on ground prepared as described in Section 6.4 may be employed as described below.

Bearing Unit

Isolated and continuous footings should bear on either Unit 2 residual soil or on Select Fill developed as described in Section 6.4.4.

Isolated Foundations

Foundations may be designed for an allowable contact stress of 2,000 psf. This value may be increased by one-third for transient loads such as wind and seismic. These foundations should have a minimum width of 18 inches, embedded at least 24 inches below surrounding grade. This bearing value may be increased by one-third for transient loads such as wind and seismic.

Continuous Foundations

Continuous foundations may be designed for an allowable contact stress of 1,800 psf. These foundation units should have a minimum width of 18 inches, embedded a minimum of 24 inches below surrounding grade. This bearing value may be increased by one-third for transient loads such as wind and seismic.

Resistance to Lateral Loads

Lateral loads to shallow foundations cast 'neat' against compacted fill, Unit 1 or Unit 2 may be resisted by passive earth pressure against the face of the footing, calculated as a fluid density of 250 psf per foot of depth, neglecting the upper 1 foot of soil below surrounding grade in this calculation. Additionally, a coefficient of friction of 0.35 between soil and the concrete base of the footing may be used with dead loads.

Settlement

Supported as recommended above, the structure will settle on the order of 0.5 inch. This movement will occur elastically, as dead load (DL) and permanent live loads (LL) are applied. In usual circumstance, about 70% of this settlement will occur during the construction period. Angular distortion due to differential settlement of adjacent, unevenly loaded footings should be less than 1 inch in 40 feet (i.e., Δ/L less than 1:480).

6.6 Capillary Break and Underslab Vapor Retarder

6.6.1 Capillary Break

NOVA recommends that the requirements for a capillary break ('sand layer') be determined in accordance with ACI Publication 302 "*Guide for Concrete Floor and Slab Construction.*" A "capillary break" may consist of a 4-inch thick layer of compacted, well-graded sand should be placed below the floor slab.

This porous fill should be clean coarse sand or sound, durable gravel with not more than 5 percent coarser than the 1-inch sieve or more than 10 percent finer than the No. 4 sieve, such as AASHTO Coarse Aggregate No. 57.

6.6.2 Vapor Retarder

Design Responsibility

Soil moisture vapor that penetrates ground-supported concrete slabs can result in damage to moisture-sensitive floor covering, some floor sealers, or sensitive equipment in direct contact with the floor.

It is not the responsibility of the geotechnical consultant to provide recommendations for vapor retarders to address this concern. This responsibility usually falls to the Architect. Decisions regarding the appropriate vapor retarder are principally driven by the nature of the building space above the slab, floor coverings, anticipated penetrations, concerns for mold or soil gas, and a variety of other environmental, aesthetic and materials factors known only to the Architect.

Design Guidance

A variety of specialty polyethylene (polyolefin)-based vapor retarding products are available to retard moisture transmission into and through concrete slabs.

Guidance to support selection of vapor retarders and to address the issue of moisture transmission into and through concrete slabs is provided in a variety of publications by the American Society for Testing and Materials (ASTM) and the American Concrete Institute (ACI). A partial listing of those publications is provided below.

- ASTM E1745-97 (2009). *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*
- ASTM E154-88 (2005). *Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Walls, or as Ground Cover*
- ASTM E96-95 (2005). *Standard Test Methods for Water Vapor Transmission of Materials*
- ASTM E1643-98 (2009). *Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs*

- ACI 302.2R-06. *Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials*

Vapor retarders employed for ground supported slabs in the San Diego are commonly specified as minimum 10 mil polyolefin plastic that conforms to the requirements of ASTM E1745 as a Class A vapor retarder (i.e., a maximum vapor permeance of 0.1 perms, minimum 45 lb/in tensile strength and 2,200 grams puncture resistance). Among the commercial products that meet this requirement are the series of Yellow Guard® vapor retarders vended by Poly-America, L.P.; the Perminator® products by W. R. Meadows; and, Stego® Wrap products by Stego Industries, LLC.

The person responsible for design of the vapor barrier should consult with product vendors to ensure selection of the vapor retarder that best meets the project requirements. For example, concrete slabs with particularly sensitive floor coverings may require lower permeance or other performance-related factors are specified by the ASTM E1745 class rating.

Quality Assurance

The performance of vapor retarders is particularly sensitive to the quality of installation. Installation should be performed in accordance with the vendor's recommendations under full-time Quality Assurance (QA) surveillance.

6.7 Control of Moisture Around Foundations

6.7.1 General

Design for the structure should include care to control accumulations of moisture around and below foundations. Such design will require coordination from among the Design Team; at a minimum to include the Architect, the Civil Engineer, and the Landscape Architect.

6.7.2 Erosion and Moisture Control During Construction

Surface water should be controlled during construction, via berms, gravel/sandbags, silt fences, straw wattles, siltation basins, positive surface grades, or other methods to avoid damage to the finished work or adjoining properties. The Contractor should take measures to prevent erosion of graded areas until such time as permanent drainage and erosion control measures have been installed. After grading, all excavated surfaces should exhibit positive drainage and eliminate areas where water might pond.

6.7.3 Design

Civil, structural, architectural and landscaping design for the areas around foundations should be undertaken with a view to the maintenance of an environment that encourages drainage away from below-grade walls. Roof and surface drainage, landscaping, and utility connections should be designed to limit the potential for mounding of water near subterranean walls. In particular, rainfall to roofs should be collected in gutters and discharged away from foundations.

Proper surface drainage will be required to minimize the potential of water seeking the level of the garage walls and pavements. In areas where sidewalks or paving do not immediately adjoin the structure, protective slopes should be provided with a minimum grade (away from the structure) of approximately 3 percent for at least 5 feet. A minimum gradient of 1 percent is recommended in hardscape areas.

6.8 Flatwork

Prior to casting exterior flatwork, at least the one foot of Unit 1 soils should be removed and replaced with Select Fill, moisture conditioned and recompact, as described in Section 6.4.2. Concrete slabs for pedestrian traffic or landscaping should be at least four (4) inches thick. Most of the Unit 1 soil will conform to the criteria for Select Fill.

7.0 PAVEMENTS

7.1 Overview

7.1.1 General

The structural design of pavement sections depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. For the purposes of the preliminary evaluation provided in this section, NOVA has assumed a Traffic Index (TI) of 5.0 for passenger car parking, and 6.0 for the driveways. These traffic indices should be confirmed by the project civil engineer prior to final design.

7.1.2 Design to Limit Infiltration

The surface grades of pavements and related design features to limit infiltration should conform with the concepts discussed in Section 6.

An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the base course, softening of the subgrade and other problems related to the deterioration of the pavement can be expected.

Furthermore, good drainage should minimize the risk of the subgrade materials becoming saturated over a long period of time. The following recommendations should be considered to limit the amount of excess moisture, which can reach the subgrade soils:

- site grading at a minimum 2% grade away from the pavements;
- compaction of any utility trenches for landscaped areas to the same criteria as the pavement subgrade;
- sealing all landscaped areas in or adjacent to pavements to minimize or prevent moisture migration to subgrade soils near pavements; and,
- concrete curbs bordering landscaped areas should have a deepened edge to provide a cutoff for moisture flow beneath pavements (generally, the edge of the curb can be extended an additional twelve inches below the base of the curb).

7.1.3 Maintenance

Preventative maintenance should be planned and provided for. Preventative maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment.

Preventative maintenance consists of both localized maintenance (e.g. crack sealing and patching) and global maintenance (e.g. surface sealing). Preventative maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements.

7.1.4 Review and Surveillance

The Geotechnical Engineer-of-Record should review the planning and design for pavement to confirm that the recommendations presented in this report have been incorporated into the plans prepared for the project. The preparation of subgrades for roadways should be observed on a full-time basis by a representative of the Geotechnical Engineer-of-Record.

7.2 Pavement Subgrade Preparation

Remedial grading for paved areas should be as described in Section 6.4.3, removing the upper 12 inches of the Unit 1, compacting the bottom of the removals to at least 90% relative compaction after ASTM D 1557 (the ‘modified Proctor’). The removed soils should be replaced with “Select” fill and densified to at least 95% relative compaction after ASTM D 1557 (the ‘modified Proctor’).

After the completion of compaction/densification, areas to receive pavements should be proof-rolled. A loaded dump truck or similar should be used to aid in identifying localized soft or unsuitable material. Any soft or unsuitable materials encountered during this proof-rolling should be removed, replaced with an approved backfill, and compacted. The Geotechnical Engineer can provide alternative options such as using geogrid and/or geotextile to stabilize the subgrade at the time of construction, if necessary.

Construction should be managed such that preparation of the subgrade immediately precedes placement of the base course. Proper drainage of the paved areas should be provided to reduce moisture infiltration to the subgrade.

The preparation of roadway and parking area subgrades should be observed on a full-time basis by a representative of NOVA to confirm that any unsuitable materials have been removed and that the subgrade is suitable for support of the proposed driveways and parking areas, after ASTM D1557.

7.3 Flexible Pavements

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. Table 7-1 provides preliminary flexible pavement sections using an assumed R-value of 20.

Table 7-1. Preliminary Pavement Sections, R = 20

Area	Subgrade R-Value	Traffic Index	Asphalt Thickness (in)	Base Course Thickness (in)
Auto Parking	20	5	4.0	6.0
Roadways/Fire Lane	20	6	4.0	8.5

1. The above sections assume properly prepared subgrade consisting of at least 12 inches of subgrade compacted to a minimum of 90% relative compaction after ASTM D1557, with EI <50.
2. The aggregate base materials should be placed at a minimum of 95% relative compaction after ASTM D1557.

7.4 Rigid Pavements

7.4.1 General

Concrete pavement sections should be developed in the same manner as undertaken for all other slabs and pavements: removal of the Unit 1 and replacement of that material in an engineered manner as described in Section 7.2.

Concrete pavement sections consisting of 7 inches of Portland cement concrete over a base course of 6 inches and a properly prepared subgrade support a wide range of traffic indices.

Where rigid pavements are used, the concrete should be obtained from an approved mix design with the minimum properties of Table 7-2.

Table 7-2. Recommended Concrete Requirements

Property	Recommended Requirement
Compressive Strength @ 28 days	3,250 psi minimum
Strength Requirements	ASTM C94
Minimum Cement Content	5.5 sacks/cu. yd.
Cement Type	Type I Portland
Concrete Aggregate	ASTM C33 and CalTrans Section 703
Aggregate Size	1-inch maximum
Maximum Water Content	0.50 lb/lb of cement
Maximum Allowable Slump	4 inches

7.4.2 Jointing and Reinforcement

Longitudinal and transverse joints should be provided as needed in concrete pavements for expansion/contraction and isolation. Sawed joints should be cut within 24-hours of concrete placement, and should be a minimum of 25% of slab thickness plus 1/4 inch. All joints should be sealed to prevent entry of foreign material and doweled where necessary for load transfer.

Load transfer devices, such as dowels or keys are recommended at joints in the paving to reduce possible offsets. Where dowels cannot be used at joints accessible to wheel loads, pavement thickness should be increased by 25 percent at the joints and tapered to regular thickness in 5 feet.

7.5 Interlocking Concrete Pavers

7.5.1 Overview

The recommendations for interlocking concrete pavers provided herein have been developed in general conformance with *Structural Design of Interlocking Concrete Pavement for Roads and Parking Lots* Interlocking Concrete Pavement Institute (ICPI), Technical Specification No. 4, May 2011.

7.5.2 General

Concrete paver units should be at least 80 millimeters (3 1/8-inches) thick for vehicular concrete pavers. Interlocking concrete pavement can be constructed by placing the concrete paver units over a 1-inch bedding sand layer generally conforming to ASTM C-33 sand.

7.5.3 Bedding and Joint Sand Gradation

Table 7-3 summarizes bedding sand gradation recommendations and recommended joint sand gradation. The joint sand should comply with ASTM C144 with a maximum 100 percent passing the No. 16 sieves and no more than 5 percent passing the No. 200 sieve.

Bedding sand may be used as joint sand; however, additional effort may be required due to its coarser gradation.

Table 7-3. Gradation of Sand for Paver Systems

Sieve Size	Percent Passing	
	Bedding Sand	Joint Sand
3/8 – inch	100	-
No. 4	95 - 100	100
No. 8	80 - 100	95 - 100
No. 16	50 - 85	70 - 100
No. 30	25 - 60	40 - 75
No. 50	5 - 30	20 - 40
No. 100	0 - 10	10 - 25
No. 200	0 - 1	0 - 5

7.5.4 Base and Subgrade

The bedding sand should be underlain with at least 10-inches of Class II base compacted to at least 95 percent of the maximum dry density at or slightly above optimum moisture content as determined by ASTM D 1557.

The upper 12 inches of the subgrade soil should be scarified; moisture conditioned as necessary, and compacted to a dry density of at least 95 percent of the laboratory maximum dry density at or slightly above optimum moisture content as determined by ASTM D1557.

7.5.5 Control of Infiltration

An impermeable liner (e.g., 30-mil PVC or equivalent) should be placed surrounding the pavers to prevent soil subgrade saturation and lateral water migration. The liner should extend up to the top of the aggregate base layer and adhered to the edge restraint.

Water retained by the liner can be collected by a subdrain. The lined subgrade soils should be sloped at least one percent towards the subdrain. A 4-inch diameter, Schedule 40, perforated PVC pipe encapsulated with Caltrans Class II permeable base (or equivalent) should be suitable as a subdrain. This piping should connect to solid PVC pipe to convey the stormwater to a suitable outlet structure, i.e. area drain or storm drain structure.

Figure 7-1 depicts a design to control infiltrating surface water that reflects the above recommendations.

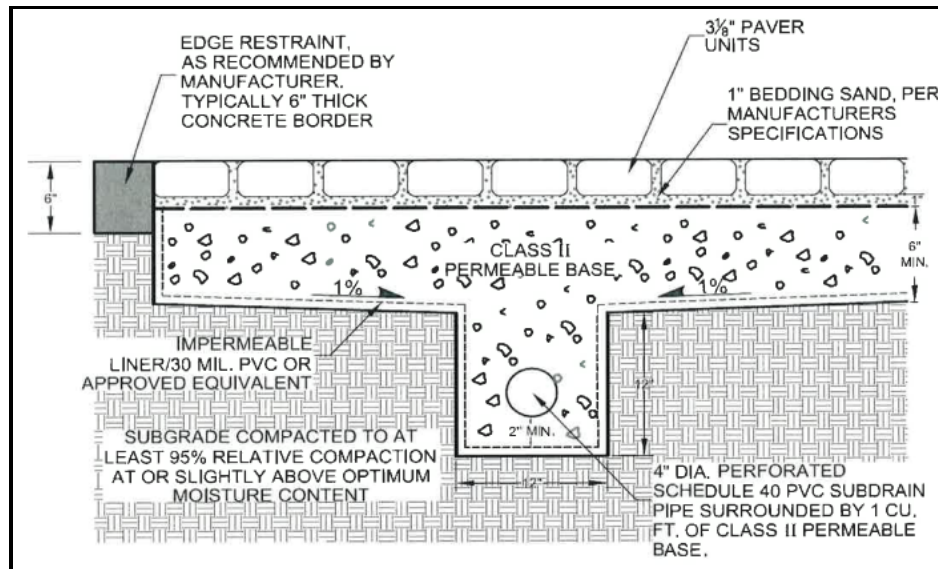


Figure 7-1. Design to Control Infiltration

7.5.6 Installation

Concrete paver installation should be performed in accordance with the manufacturer's and ICPI guidelines. Stable edge restraints such as concrete edge bands and curbs are essential to maintain horizontal interlock while the paver units are subjected to repeated vehicular loads.

7.5.7 Edge Restraint

The edge restraint may consist of a concrete pavement section. Other edge restraint recommendations can be found in the ICPI technical guidelines.

A concrete edge restraint pavement section may be designed in general conformance with the procedure recommended by the American Concrete Institute report ACI 330R-08 *Guide for Design and Construction of Concrete Parking Lots* using the following parameters:

Modulus of subgrade reaction, $k = 100$ pci
Modulus of rupture for concrete, $MR = 500$ psi
Traffic Category = B
Average daily truck traffic, ADTT (assumed) = 30

Based on the criteria presented above, concrete pavement should consist of a minimum of 6 inches of PCC placed over subgrade soil compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,200 psi (pounds per square inch).

No reinforcing steel will be necessary within the concrete for geotechnical purposes.

7.5.8 Maintenance

A maintenance schedule consisting of inspecting the pavement sections should be established. Periodic removal, replacement, and re-leveling of individual pavers may be required.

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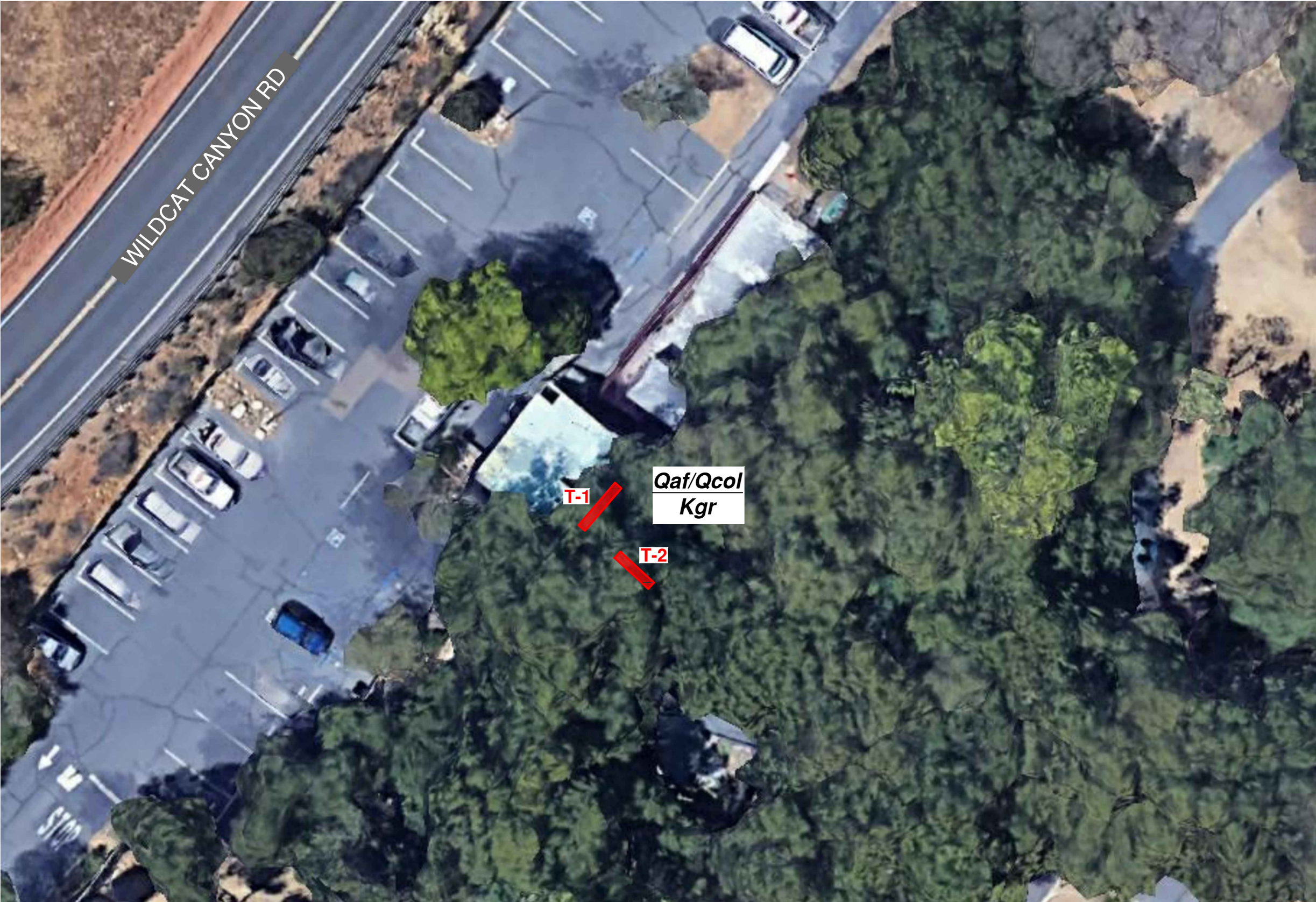
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United States Geological Survey and California Geological Survey, 2011, Quaternary Fault and Fold database for the United States, found at: <http://earthquake.usgs.gov/regional/qfaults/>.

PLATE 1





(SOURCE: GOOGLE EARTH 2019)

KEY TO SYMBOLS

<i>Qaf/Qcol</i>	UNDIFFERENTIATED FILL/COLLUVIUM	T-2	TRENCH LOCATION
<i>Kgr</i>	GRANITICS		



NOVA
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA

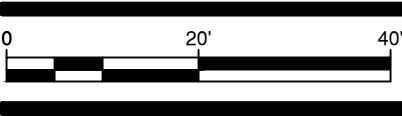
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STELZER RANGER STATION
LOUIS A. STELZER COUNTY PARK
LAKESIDE, CALIFORNIA

PROJECT NO: 2019133
DATE: JUNE 2019
DRAWN BY: DTW
REVIEWED BY: WM

SUBSURFACE
INVESTIGATION MAP



APPENDIX A
USE OF THE GEOTECHNICAL REPORT



Important Information About Your Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

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

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

Read the Full Report

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

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

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

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

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

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

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

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

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that

have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

Rely, on Your ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.



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APPENDIX B
LOGS OF EXPLORATORY TRENCHES



TRENCH LOG T-1

DATE EXCAVATED: JUNE 20, 2019 EQUIPMENT: EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 725 FT MSL
OBTAINED FROM GOOGLE EARTH

LAB TEST ABBREVIATIONS

CR CORROSIVITY
MD MAXIMUM DENSITY
DS DIRECT SHEAR
EI EXPANSION INDEX
AL ATTERBERG LIMITS
SA SIEVE ANALYSIS
RV RESISTANCE VALUE
CN CONSOLIDATION
SE SAND EQUIVALENT

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SC		FILL/COLLUVIUM, UNDIFFERENTIATED (Q_{at}/Q_{yc}): CLAYEY SAND; BROWN, DRY TO DAMP, LOOSE TO MEDIUM DENSE, SCATTERED GRAVEL, ABUNDANT ROOTS DAMP MEDIUM DENSE MEDIUM DENSE TO DENSE	CR MD EI SA	10 VERY LOW
5				CL		RESIDUAL SOIL: SANDY CLAY; RED BROWN, DAMP, VERY STIFF TO HARD, TRACE GRAVEL, FEW ROOTS, SLIGHTLY POROUS	EI SA	28 LOW 30 LOW
				SC		CLAYEY SAND; RED BROWN, DAMP, VERY DENSE, FINE TO COARSE GRAINED, GRANITIC FRAGMENTS, SCATTERED GRANITIC BOULDERS ≤ 14"		
15						TRENCH TERMINATED AT 7.5 FT DUE TO REFUSAL ON VERY DENSE GRANITIC ROCK (K _{gr}). NO GROUNDWATER ENCOUNTERED. NO CAVING.		

KEY TO SYMBOLS

	GROUNDWATER	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	DCP TEST (ASTM D6951)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

RANGER STATION

LOUIS A. STELZER COUNTY PARK
LAKESIDE, CALIFORNIA

LOGGED BY: DEM DATE: JUL 2019

REVIEWED BY: BMH PROJECT NO.: 2019133



APPENDIX B.1

TRENCH LOG T-2

DATE EXCAVATED: JUNE 20, 2019 EQUIPMENT: EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 723 FT MSL
OBTAINED FROM GOOGLE EARTH

LAB TEST ABBREVIATIONS

CR CORROSIVITY
MD MAXIMUM DENSITY
DS DIRECT SHEAR
EI EXPANSION INDEX
AL ATTERBERG LIMITS
SA SIEVE ANALYSIS
RV RESISTANCE VALUE
CN CONSOLIDATION
SE SAND EQUIVALENT

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	DCP TEST	SOIL CLASS. (USCS)	DCPT IN/25 BLOWS	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SC		FILL/COLLUVIUM, UNDIFFERENTIATED (Qat/Qyc): CLAYEY SAND; GRAY BROWN, DRY, LOOSE, SCATTERED GRAVEL BROWN, MEDIUM DENSE		
				CL		RESIDUAL SOIL: SANDY CLAY; RED BROWN, DAMP, VERY STIFF TO HARD, TRACE GRAVEL, FEW ROOTS, SLIGHTLY POROUS		
5				SC		CLAYEY SAND; RED BROWN, DAMP, VERY DENSE, FINE TO COARSE GRAINED, GRANITIC FRAGMENTS, SCATTERED GRANITIC BOULDERS ≤ 14"		
						TRENCH TERMINATED AT 7.5 FT DUE TO REFUSAL ON VERY DENSE GRANITIC ROCK (Kgr). NO GROUNDWATER ENCOUNTERED. NO CAVING.		
10								
15								

KEY TO SYMBOLS

	GROUNDWATER	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	DCP TEST (ASTM D6951)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

RANGER STATION

LOUIS A. STELZER COUNTY PARK
LAKESIDE, CALIFORNIA

LOGGED BY: DEM DATE: JUL 2019

REVIEWED BY: BMH PROJECT NO.: 2019133



APPENDIX B.2

APPENDIX C
LABORATORY ANALYTICAL RESULTS



Laboratory tests were performed in accordance with the generally accepted American Society for Testing and Materials (ASTM) test methods or suggested procedures. Brief descriptions of the tests performed are presented below:

- **CLASSIFICATION:** Field classifications were verified in the laboratory by visual examination. The final soil classifications are in accordance with the Unified Soils Classification System and are presented on the exploration logs in Appendix B.
- **ATTERBERG LIMITS (ASTM D 4318):** Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System.
- **EXPANSION INDEX (ASTM D4829):** The expansion index of selected materials was evaluated in general accordance with ASTM D4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours.
- **MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557 METHOD A,B,C):** The maximum dry density and optimum moisture content of typical soils were determined in the laboratory in accordance with ASTM Standard Test D1557, Method A, Method B, Method C.
- **MOISTURE CONTENT (ASTM D2216):** Tests were performed on selected representative soil samples to evaluate the water (moisture) content by mass of soil, rock, and similar materials where the reduction in mass by drying is due to loss of water. Test sample is dried in an oven at a temperature of $110^{\circ} \pm 5^{\circ}\text{C}$ to a constant mass. The loss of mass due to drying is considered to be water. The water (moisture) content were determined in general accordance with ASTM D2216
- **CORROSIVITY TEST (CAL. TEST METHOD 417, 422, 643):** Soil PH, and minimum resistivity tests were performed on a representative soil sample in general accordance with test method CT 643. The sulfate and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively.
- **GRADATION ANALYSIS (ASTM C 136 and/or ASTM D422):** Tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain size distributions of selected samples were determined in accordance with ASTM C 136 and/or ASTM D422. The results of the tests are summarized on Appendix C.3 through Appendix C.4.



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LAB TEST SUMMARY

RANGER STATION

LOUIS A. STELZER COUNTY PARK

LAKEVILLE, CALIFORNIA

BY: HP

DATE: JULY 2019

PROJECT: 2019133

APPENDIX: C.1

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft.)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
T-1	0' - 1.5'	Brown Clayey Sand	126.5	10.9

Moisture Content Test (ASTM D2216)

Sample Location	Sample Depth (ft.)	Soil Description	Moisture (%)
T-1	3'-4'	Red Brown Sandy Clay	8.8

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft.)	Expansion Index	Expansion Potential
T-1	0' - 1.5'	10	Very Low
T-1	3' - 4'	28	Low
T-1	4' - 3'	30	Low

Atterberg Limits (ASTM D4318)

Sample Location	Sample Depth (ft.)	Liquid Limit, LL	Plastic Limit, PL	Plasticity Index, PI	USCS (% Finer than No. 40)
T-1	3' - 4'	26	14	13	CL

Corrosivity (Cal. Test Method 417,422,643)

Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content (ppm)	Sulfate Content (%)	Chloride Content (ppm)	Chloride Content (%)
T-1	0' - 1.5'	5.5	5700	36	0.004	21	0.002



4373 VIEWRIDGE AVENUE, SUITE B

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LAB TEST RESULTS

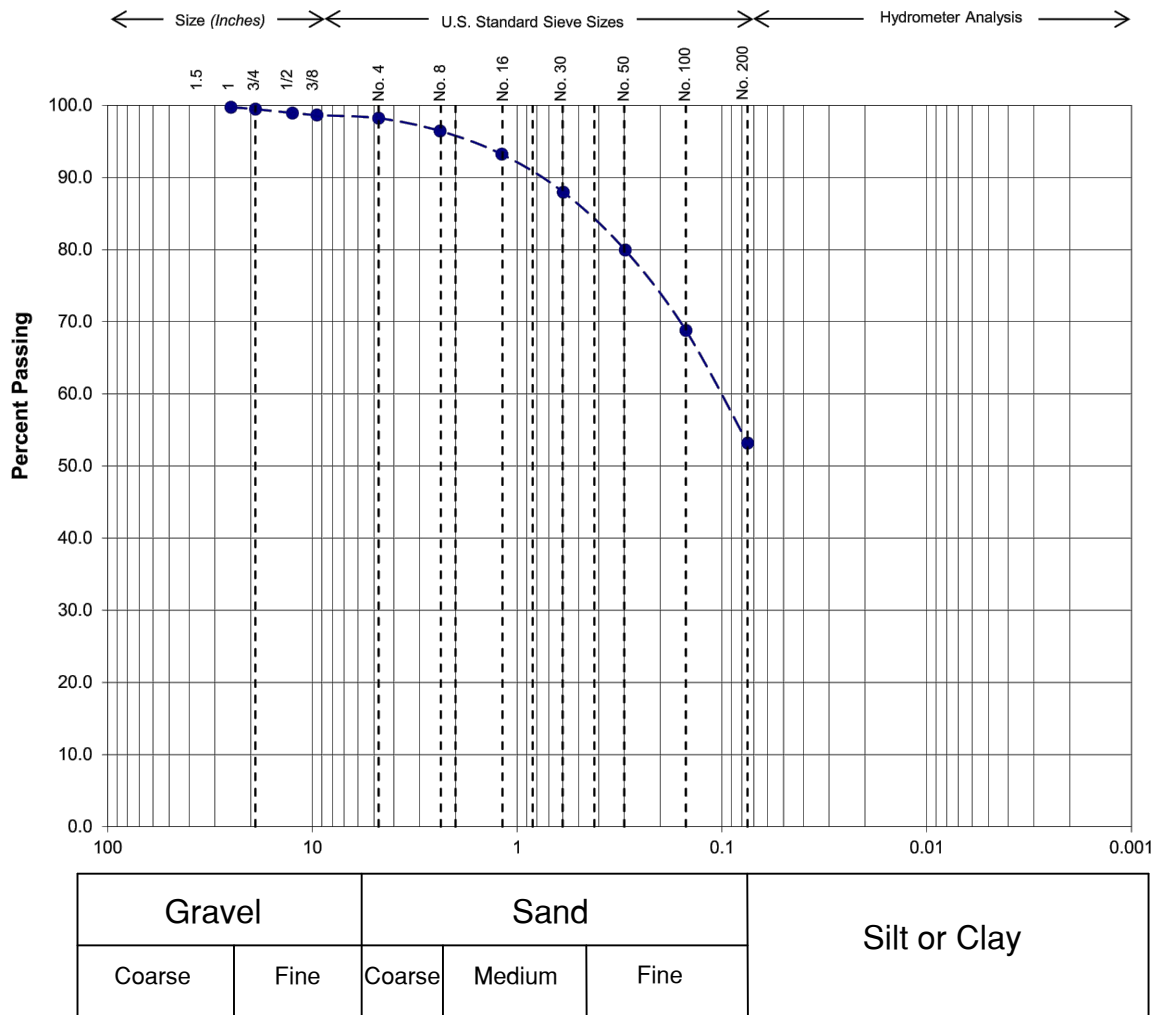
RANGER STATION
LOUIS A. STELZER COUNTY PARK
LAKESIDE, CALIFORNIA

BY: HP

DATE: JULY 2019

PROJECT: 2019133

APPENDIX: C.2



Sample Location: T-1

Depth (ft): 3' - 4'

USCS Soil Type: CL

Passing No. 200 (%): 53



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GRADATION ANALYSIS TEST RESULTS

RANGER STATION

LOUIS A. STELZER COUNTY PARK
LAKESIDE, CALIFORNIA

BY: HP

DATE: JULY 2019

PROJECT: 2019133

APPENDIX: C.4