

Report Geotechnical Investigation

Hiser Property 7737 Mission Gorge Road, Santee, California



Cornerstone Communities
4365 Executive Drive, Suite 600
San Diego, CA 92121



NOVA Project 2020035
April 13, 2020



4373 Viewridge Avenue, Suite B
San Diego, California 92123
858.292.7575

944 Calle Amanecer, Suite F
San Clemente, CA 92673
949.388.7710

www.usa-nova.com



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Cornerstone Communities
4365 Executive Drive, Suite 600
San Diego, CA 92121

April 13, 2020
NOVA Project 2020035

Attention: Jack Robson, Vice President

Subject: Report
Geotechnical Investigation
Hiser Property
7737 Mission Gorge Road, Santee, California

Dear Mr. Robson:

NOVA Services, Inc. (NOVA) is pleased to present herewith the above-referenced report. The work reported therein was completed by NOVA for Cornerstone Communities in accordance with the scope of work identified in NOVA's proposal dated February 13, 2020, as authorized on February 13, 2020.

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

Sincerely,
NOVA Services, Inc.

Wail Mokhtar
Senior Project Manager

Melissa Stayner, PG, CEG
Senior Engineering Geologist



John F. O'Brien, PE, GE
Principal Geotechnical Engineer



Hillary A. Price
Project Geologist

REPORT GEOTECHNICAL INVESTIGATION

Hiser Property
7737 Mission Gorge Road, Santee, California

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1.0 INTRODUCTION

1.1 Terms of Reference

The work reported herein was completed by NOVA Services, Inc. (NOVA) for Cornerstone Communities in accordance with NOVA's proposal dated February 13 2020, as authorized on February 13, 2020.

The report provides the findings of a geotechnical investigation for a multi-family residential development known to NOVA as 'Hiser Property.' The subject development will be located at APNs 386-300-13 and -29, located behind (south of) the property at 7737 Mission Gorge Road in Santee, CA (hereinafter, 'the site'). Figure 1-1 provides a graphic that depicts the site vicinity.



Figure 1-1. Vicinity Map

1.2 Geotechnical Work by Others

This site and the planned development thereon have been the object of a prior geotechnical study by GeoSoils, Inc. (reference, *Geotechnical Feasibility Review, Hiser Property, APNs 386-300-13, -29, and -31, 7737 Mission Gorge Road, City of Santee, San Diego County, California*, GeoSoils, Inc., W.O. 7678-A-SC, 9 Sept 2019, hereinafter 'GSI 2019').

The work reported herein utilizes the findings of the engineering borings and seismic survey lines completed by GSI for the subsurface exploration. The recommendations provided herein supersede those provided in GSI 2019.

1.3 Objectives, Scope, and Limitations of This Work

1.3.1 Objectives

The work reported herein utilizes the findings of subsurface characterization by others and additional subsurface characterization by NOVA to accomplish the twofold objectives listed below.

1. Objective 1, Geotechnical. Provide recommendations for geotechnical-related design, including foundations and related earthwork.
2. Objective 2, Stormwater. Provide design phase recommendations for development of permanent stormwater infiltration Best Management Practices ('stormwater BMPs').

1.3.2 Scope

In order to accomplish the above objectives, NOVA undertook the task-based scope of work described below.

- Task 1, Background Review. Reviewed readily available background data regarding the site area, including prior geotechnical reporting (GSI 2019), geologic data, fault maps, and a conceptual civil engineering plan.
- Task 2, Subsurface Exploration. A subsurface exploration included the subtasks listed below.
 - *Subtask 2-1, Reconnaissance and Utility Clearance*. Prior to undertaking any invasive work, NOVA conducted a site reconnaissance and layout of engineering borings and percolation wells. Underground Service Alert and a utility location contractor were notified for underground utility mark-out services.
 - *Subtask 2-2, Subcontracting*. NOVA retained specialty contractors to conduct engineering borings and trench excavations.
 - *Subtask 2-3, Engineering Borings*. NOVA completed six (6) hollow-stem engineering borings including in areas to receive stormwater BMPs to depths between 10.5 to 20.5 feet bgs (below ground surface). A NOVA geologist directed the drilling and conducted logging and sampling using ASTM methods.
 - *Subtask 2-4, Trenches*. Seven (7) exploratory trenches were excavated to 10 feet in depth. The trenches were excavated by a backhoe, for the evaluating of the fill quality and percentage of oversize material.
 - *Subtask 2-5, Percolation Testing*. NOVA conducted design-phase percolation testing to develop infiltration rates for stormwater BMPs. Four (4) percolation test wells (2 well pairs) were constructed in prospective BMP areas following the guidance for such work provided in the City of Santee BMP Design Manual.

- Task 3, Laboratory Testing. Laboratory testing was completed to address index soil characteristics, utilizing these data for both soil classification and correlation with soil mechanical characteristics (i.e., strength and compressibility). Additional specialty testing was conducted to address the potential that the soils may be corrosive to embedded metals or at risk for sulfate attack to embedded concrete.
- Task 4, Engineering Evaluations. Utilizing the data developed by the preceding tasks, NOVA completed geotechnical and stormwater infiltration-focused engineering evaluations.
- Task 5, Reporting. Preparation of this report presenting NOVA's findings and recommendations completes the scope of work described in NOVA's proposal.

1.3.3 Limitations

Assessment of the subsurface in geological and geotechnical engineering is characterized by uncertainty. Opinions relating to environmental, geologic, and geotechnical conditions are based on limited data, such that actual conditions may vary from those encountered at the times and locations where the data are obtained, despite the use of due professional care. The judgments provided in this report are based upon NOVA's understanding of the planned construction, its experience with similar work, and its judgments regarding subsurface conditions indicated by the methods of subsurface exploration described in the report.

Conditions exposed by construction may vary from those disclosed by the borings. NOVA should be retained for design review and for surveillance to observe subsurface conditions revealed during construction. NOVA cannot assume responsibility for the recommendations of this report if NOVA does not perform construction observation. Section 9 of this report addresses this consideration in more detail.

This report addresses geotechnical and stormwater considerations only. The report does not provide any environmental assessment or investigation of the presence or absence of hazardous or toxic materials in the soil, soil gas, 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.

1.4 Understood Use of This Report

NOVA expects that the findings and recommendations provided herein will be utilized by Cornerstone Communities and its Design Team in decision-making regarding geotechnical-related design and construction.

1.5 Report Organization

The remainder of this report is organized as abstracted below.

- Section 2 reviews available project information.
- Section 3 describes the field exploration and laboratory testing.

- Section 4 describes the site physical setting, including the geologic setting and subsurface conditions.
- Section 5 reviews geologic and soil hazards common to the San Diego County region, considering each for its potential to affect the site.
- Section 6 provides preliminary recommendations for earthwork, ground improvement and foundation design.
- Section 7 addresses development of stormwater infiltration BMPs.
- Section 8 provides recommendations for pavement design.
- Section 9 provides recommendations for geotechnical observation during construction.
- Section 10 lists the principal references utilized in the development of the report.

Figures and tables intended to amplify the discussions in the text are embedded therein. Plate 1 provided immediately following the text of the report, depicts the locations of borings, trenches, and the seismic lines. Plate 2 through Plate 4 provide cross-sections of the subsurface.

The report is supported by five appendices.

- Appendix A provides guidance regarding the use and limitations of this report.
- Appendix B presents logs of the engineering borings.
- Appendix C provides records of the seismic refraction testing reported in GSI 2019.
- Appendix D records of geotechnical laboratory testing.
- Appendix E provides infiltration feasibility documents.

2.0 PROJECT INFORMATION

2.1 Site Description

2.1.1 Location

The approximately 7.1-acre site consists of APNs 386-300-13 and -29, located behind (south of) the property at 7737 Mission Gorge Road in Santee. The irregularly shaped site includes contiguous APNs 386-300-13, -29. The site is bounded on the north by a non-operational bottled water facility, on the west by a trailer park, on the south by single family residential development, and on the east by a multifamily development.

Figure 2-1 depicts the location and approximate limits of the site on a recent aerial view.



Figure 2-1. Site Location and Limits

2.1.2 Site Use

Current

The site has scattered mobile residential structures, with a series of paved roads, gravel roads, parking areas, concrete masonry unit (CMU) retaining walls, utility improvements, and several large trees.

Historic

Review of aerial photography dating to 1953 indicates that the site has been used for mobile residential housing since at least 1953.

2.2 Planned Development

2.2.1 General

Based upon discussions with Cornerstone Communities and Rick Engineering, NOVA understands that this property will be developed for condominium construction.

Development will also include several retaining walls that will adapt the existing sloping landform to that required for the proposed development. The walls will maximize area for building pads, interior drives, a pocket park, wet and dry utilities, two stormwater BMPs, and a stormwater vault.

The Conceptual Site Plan provided by Rick Engineering has been used as the base map for our geotechnical investigation and subsequent report figures (reference, *Conceptual Site Plan*, Rick Engineering, undated, provided to NOVA in February 2020; hereinafter, 'RE 2020').

2.2.2 Structural

No structural information is available. However, based upon experience with similar structures, NOVA expects that the buildings will be wood-framed, set atop a ground supported slab.

NOVA anticipates that interior column loads to isolated interior footings might be on the order of 50 kips (DL + LL) or less, with wall loads on the order of 1.5 kips per lineal foot.

2.2.3 Potential for Earthwork

NOVA's understanding of the potential for earthwork across the site is based upon review of grading depicted in a Conceptual Site Plan by Rick Engineering.

Cuts will extend to a maximum of about 23 feet in depth, with the thickest fills ranging to about 15 feet in height.

Development of the site is expected to include demolition of existing retaining walls, old foundations, utilities and pavements. Construction will include several conventional retaining walls and at least one mechanically stabilized earth (MSE) retaining wall. These walls will be constructed to adapt the sloping ground to a profile suitable for development of building pads for the various residential structures. Retaining walls will be developed to varying heights ranging to a maximum of about 17 feet for the proposed MSE wall on the southern property boundary.

Standard over-excavations are recommended below the buildings and retaining walls that are located on undocumented fill and on buildings that span between cut and fill transitions. Detailed earthwork recommendations are presented in Section 6.

Section 4.3 addresses this consideration in more detail. Section 6.4 provides guidance for planning for earthwork.

3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Overview

GSI 2019 reports the findings of six (6) engineering borings ('B-1' through 'B-6') and five (5) seismic refraction lines ('ST-1' through 'ST-5'). NOVA completed a supplemental field exploration on February 27-28, 2020. That work included six (6) engineering borings ('B-1' through 'B-6'), four percolation test borings ('P-1' and 'P-4'), and seven (7) exploratory trenches ('T-1' through 'T-7').

Figure 3-1 provides a plan view of the locations of subsurface explorations. Plate 1, provided immediately following the text of this report, presents the subsurface exploration map in larger scale.

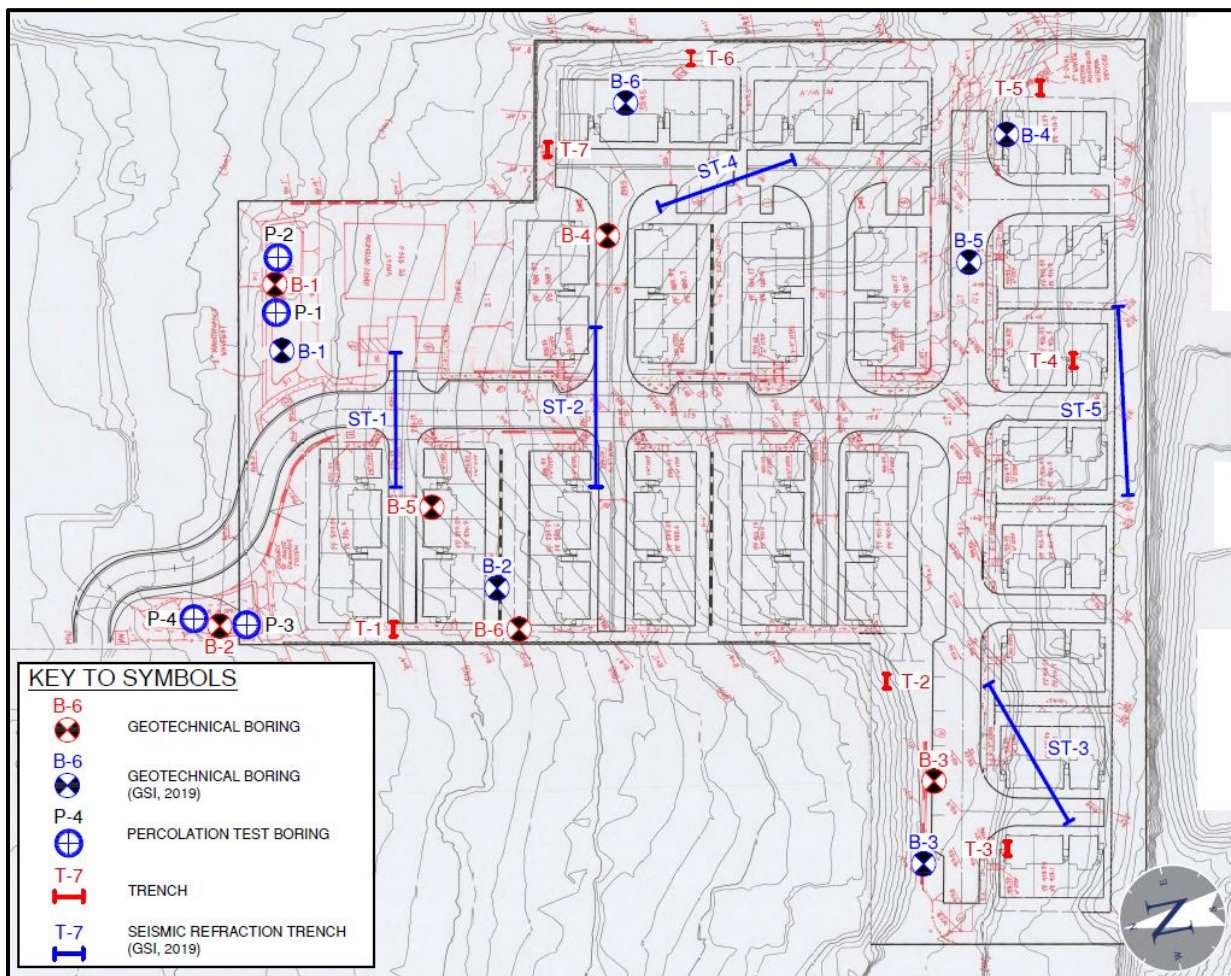


Figure 3-1. Locations of the Subsurface Explorations

3.2 NOVA Engineering Borings

3.2.1 Drilling

A NOVA geologist directed drilling and sampling of six engineering borings, referenced as 'B-1' through 'B-6', drilled to depths between 10.5 feet and 20.5 feet below ground surface (bgs) on February 27, 2020.

The borings were advanced by a track-mounted limited-access drilling rig utilizing hollow-stem auger drilling techniques. Figure 3-2 (following pages) depicts the drilling operations. Table 3-1 provides an abstract of the engineering borings.

Table 3-1. Abstract of the NOVA Engineering Borings

Boring Reference	Approx. Ground Surface Elev. (feet, msl) ¹	Total Depth Below Ground Surface (feet)	Elevation at Completion (feet, msl) ¹	Approx. Depth to Formation (feet) ²	Approx. Depth to Groundwater (feet)
B-1	+364	15.5	+348.5	3.5	Not encountered
B-2	+360	15.5	+344.5	4	Not encountered
B-3	+413	20.5	+392.5	3.5	Not encountered
B-4	+388	10.5	+377.5	2.5	Not encountered
B-5	+376	10.5	+365.5	4	Not encountered
B-6	+376	10.5	+365.5	2	Not encountered

Note 1: elevations are approximate and should be reviewed

Note 2: the referenced formational geologic unit is Cretaceous Tonalite (Kt)

3.2.2 Logging and Sampling

The geologist directed sampling and maintained a log of the subsurface materials that were encountered. Disturbed samples were recovered from the borings. Sampling of soils is described below.

1. The Modified California sampler ('ring sampler', after ASTM D3550) was driven using a 140-pound hammer falling for 30 inches with a total penetration of 18 inches, recording blow counts for each 6 inches of penetration.
2. The Standard Penetration Test sampler ('SPT', after ASTM D1586) was driven in the same manner as the ring sampler, recording blow counts in the same fashion. SPT blow counts for the final 12 inches of penetration comprise the SPT 'N' value, an index of soil strength and compressibility.
3. Bulk samples were recovered from the near subsurface.

The NOVA geologist maintained a log of all sampling, as well as a depiction of the subsurface materials based on the indications of the samples and observation of the drilling itself. The recovered samples were transferred to NOVA's geotechnical laboratory for visual inspection and laboratory testing. Records of the engineering borings are provided in Appendix B.



Figure 3-2. Drilling Operations, Location B-4, February 2020

3.2.3 Closure

Upon completion, the borings were each backfilled with soil cuttings to match the existing surfacing. The areas of the borings were restored to pre-drilling conditions to the degree practical.

3.3 NOVA Exploratory Trenches

Seven exploratory test trenches were excavated by a tracked excavator to depths between 4 and 10 feet bgs on February 27, 2020 at the locations indicated on Figure 3-1. Table 3-2 abstracts the exploratory test trenches. Figure 3-3 (following page) depicts the test trench excavation operations.

Table 3-2. Abstract of the NOVA Exploratory Trenches

Boring Reference	Approx. Ground Surface Elev. ¹ (feet, msl)	Total Depth Below Ground Surface (feet)	Elev. ¹ at Completion (feet, msl)	Depth to Bedrock ² (feet)
T-1	+376	10	+366	8
T-2	+412	7	+405	5
T-3	+424	6	+418	5
T-4	+423	4	+419	4
T-5	+410	6	+404	6
T-6	+384	6.5	+377.5	5.5
T-7	+381	4	+377	3

Note: 1. elevations are approximate

Note 2. the referenced geologic unit is weathered Cretaceous Tonalite (Kt)



Figure 3-3. Excavation Operations, Location T-3, February 2020

The NOVA geologist maintained a log of all sampling, as well as a description of the subsurface materials encountered within the trench. The recovered samples were transferred to NOVA's geotechnical laboratory for visual inspection and laboratory testing. Records of the trenches are presented in Appendix B.

3.3.1 Closure

Upon completion, the trenches were each backfilled with soil cuttings to match the elevation of the surface. The areas of the trenches were restored to pre-excavation conditions to the degree practical.

3.4 GSI Subsurface Exploration

3.4.1 GSI Engineering Borings

GSI 2019 reports the findings of a series of six (6) engineering borings completed to support the objectives of that work.

Table 3-3 (following page) is an abstract of the engineering borings.

Table 3-3. Abstract of the GSI Engineering Borings

Boring Reference	Ground Surface Elevation (feet, msl) ¹	Total Depth Below Ground Surface (feet)	Elevation at Completion (feet, msl) ¹	Approx. Depth to Formation (feet) ²	Approx. Depth to Groundwater (feet)
B-1	+364	19.5	+344.5	2.5	Not encountered
B-2	+376	19.5	+356.5	3.5	Not encountered
B-3	+414	19.5	+395.5	12.0	Not encountered
B-4	+416	16.5	+399.5	10.0	Not encountered
B-5	+414	19.5	+394.5	8.0	Not encountered
B-6	+388	15.5	+672.5	5.0	Not encountered

Note 1: elevations are approximate

Note 2: the referenced geologic unit is weathered Cretaceous Tonalite (Kt)

3.4.2 GSI Seismic Refraction Survey

GSI 2019 reports the findings of five (5) seismic refraction lines or ‘traverses.’ The alignments of seismic traverses are shown on Figure 3-1. Seismic refraction surveys are conducted to measure the seismic compression velocity (‘P-wave’) through the subsurface, whereby approximating the subsurface stratigraphy. The measurements of seismic velocity are also used to estimate the degree of rippability, providing a semi-quantitative assessment of this potential. The velocity-depth profiles developed in GSI 2019 are reproduced in Appendix C.

Consistent with the indications of the borings and the test pits, the layer boundaries indicated by the seismic velocities tend to mimic the surface topography. For the purposes of this report, the traverses indicate the sequence of subsurface materials that is abstracted below.

1. Fill/Colluvium. A near surface layer of undifferentiated fill/colluvium (or very highly weathered bedrock), mantles the site to a depth on the order of about ±1 to ±5 feet. The average velocity of this material is about ±1,580 fps, typical for such near surface material.
2. Transition Soil/Rock. Below a depth of about 1.5 feet and extending to about 5 feet depth, seismic velocities average about ±3,800 fps, with some variability. These velocities reflect weathered, weak and highly fractured bedrock.
3. Rippable Rock. Below about 5 feet depth and extending to approximately 22 to 40 feet below existing surface grades, the site is underlain by fractured, relatively sound rock. Seismic velocities range to about 6,000 fps in this unit.
4. Sound Rock. Below Unit 3, average seismic velocity of Layer 3 exceeds 6,000 fps, ranging to as high as about 12,300 fps. At depths where velocities are greater than about 6,000 fps, rock loosening by ripping is difficult and blasting usually is required.

Figure 3-4 (following page) provides reproduces the assessment of rippability provided in GSI 2019.

SEISMIC LINE NO.	GENERAL RIPPABILITY (ASSUMING A D9L DOZER OR CAT 235 HOE, OR EQUIVALENT)
ST-1	Rippable and trenchable to depths of ± 2 to ± 4 feet. Marginally trenchable to Non-trenchable, but rippable between 2 to 4 feet and 22 to 24 feet below existing grades (B.E.G.). Non-rippable below 22 to 24 feet B.E.G., localized blasting and/or rock breaking may not be precluded in areas of locally hard outcrops, or "floaters" at depth.
ST-2	Rippable and trenchable to depths of ± 2 to ± 3 feet. Marginally trenchable to non-trenchable, but rippable between 2 to 3 feet and 28 to 30 feet below existing grades. Non-rippable below 28 to 30 feet. Localized blasting and/or rock breaking may not be precluded in areas of locally hard outcrops, or "floaters" at depth.
ST-3	Rippable and trenchable to depths of ± 2 to 3 feet. Marginally trenchable to Non-trenchable, but rippable between 2 to 3 feet, and 34 to 40 feet below existing grades. Non-rippable below 34 to 40 feet. Localized blasting and/or rock breaking may not be precluded in areas of locally hard outcrops, or "floaters" at depth.
ST-4	Rippable and trenchable to depths of ± 1 to 3 feet. Non-trenchable but rippable between 1 to 3 feet and 28 to 30 feet below existing grades. Non-rippable below 28 to 30 feet. Localized blasting and/or rock breaking may not be precluded in areas of locally hard outcrops, or "floaters" at depth.
ST-5	Rippable and trenchable to depths of ± 1 to ± 3 feet. Non-trenchable but rippable between 1 to 3 feet, to about 33 to 40 feet below existing surface grades. Non-rippable below 33 to 40 feet. Localized blasting and/or rock breaking may not be precluded in areas of locally hard outcrops, or "floaters" at depth. Surface outcrops noted in vicinity.

Figure 3-4. Rippability as Indicated by GSI 2019 Seismic Refraction Survey
(source: GSI 2019)

3.5 Design Phase Percolation Testing

3.5.1 General

NOVA directed the excavation and construction of four (4) percolation test wells, 2 at each proposed basin, following the recommendations for percolation testing presented in the City of Santee BMP Design Manual, February 2016 edition (hereinafter 'the BMP Manual).

The percolation test locations are shown on Figure 3-1.

3.5.2 Drilling

The borings for the wells were each drilled with an 8-inch hollow stem auger to depths between 3 and 5 feet below ground surface (bgs). Field measurements were taken to confirm that the borings were excavated to approximately 8 inches in diameter. The borings were logged by a NOVA geologist, who observed and recorded exposed soil cuttings and the boring conditions. Exploratory engineering borings were drilled within 50 feet of each proposed basin, extending to at least 10 feet below the lowest elevation of the base of the proposed BMP, to evaluate the soil strata and demonstrate adequate separation from groundwater.

3.5.3 Conversion to Percolation Wells

Once the borings were drilled to the desired depths, the borings were converted to percolation test wells by placing an approximately 2-inch layer of $\frac{3}{4}$ -inch gravel on the bottom, then

extending 3-inch diameter Schedule 40 perforated PVC pipe to the ground surface. The $\frac{3}{4}$ -inch gravel was used to partially fill the annular space around the perforated pipe below the existing finish grade to minimize the potential of soil caving.

3.5.4 Percolation Testing

The percolation test wells were pre-soaked by filling the holes with water to the ground surface level and testing commenced within a 26-hour window. On the day of testing, two 25-minute trials were conducted in each well.

In 'P-1', the pre-soak water percolated at least 6 inches within the 25-minute trials. In test wells 'P-2', 'P-3', and 'P-4', the pre-soak water did not percolate at least 6 inches into the soil unit within 25 minutes.

Based on the results of the trials, water levels for percolation well 'P-1' were recorded every 10 minutes for 1 hour. For 'P-2', 'P-3', and 'P-4', water levels were recorded every 30 minutes for six hours. At the beginning of each test interval, the water level was raised to approximately the same level as the previous tests, in order to maintain a near-constant head during all test periods. Table 3-4 abstracts the indications of the percolation testing. Note that percolation rates are not the same as infiltration rates. Infiltration rates are discussed in Section 7.

Table 3-4. Abstract of the Percolation Testing

Boring Reference	Approximate Elevation (feet, msl)	Total Depth (feet)	Approximate Percolation Test Elevation (feet, msl)	Percolation Rate (inches/hour) ²	Subsurface Unit Tested ¹
P-1	+360	5	+355	40.32	Kt
P-2	+360	3	+357	5.04	Kt
P-3	+366	5	+361	0.24	Kt
P-4	+366	3	+363	3.84	Kt

Note 1: The referenced geologic unit is Tonalite Rock (Kt).

Note 2: Section 7 addresses infiltration rates (I) determined from percolation rates.

3.6 Laboratory Testing

3.6.1 General

Soil samples recovered from the engineering borings and exploratory trenches were transferred to NOVA's geotechnical laboratory where a geotechnical engineer reviewed the soil samples and the field logs. Representative soil samples were selected and tested in NOVA's materials laboratory to check visual classifications and to determine pertinent index characteristics. Records of the testing are provided in Appendix D.

3.6.2 Maximum Density and Optimum Moisture

Two (2) samples of the near-surface soil and two (2) samples of the weathered tonalite were each tested to determine its moisture-density relationship after ASTM D1557 (the 'modified Proctor'). Table 3-5 provides a summary of this testing.

Table 3-5. Abstract of the Maximum Dry Density and Optimum Moisture Content Testing

Sample Reference		Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Subsurface Unit Tested ¹
Boring	Depth (feet)			
B-1	0 - 5	134.0	8.0	Qaf
B-1	5 - 10	135.0	8.1	Kt
B-3	5 - 7	139.0	5.9	Kt
T-4	0 - 4	131.6	8.2	Qaf

Note 1: The referenced soil units are Fill (Qaf) and Tonalite Rock (Kt).

3.6.3 Gradation

The visual classifications were further evaluated by determinations of grain size distribution. Table 3-6 provides a summary of this testing.

Table 3-6. Abstract of the Soil Gradation Testing

Sample Reference		Percent Finer than the U.S. No 200 Sieve	Classification after ASTM D 2488
Boring	Depth (feet)		
B-1	0 - 5	42	SM
B-1	5-10	25	SM
B-2	5	24	SM
B-2	10	44	SM
B-3	5-7	32	SM
T-4	0-4	18	SM

Note: 'Passing #200' is percent by weight passing the U.S. # 200 sieve (0.074 mm), after ASTM D6913.

3.6.4 Chemical Testing

Resistivity, sulfate content, and chloride content testing of a representative sample of the near-surface soils was used to address the potential for the soils to corrode unprotected metals and the potential for sulfate attack to embedded concrete. Table 3-7 (following page) abstracts the chemical testing. Indications of this testing are discussed in more detail in Section 6.3.

Table 3-7. Abstract of Chemical Testing

Sample Ref		pH	Resistivity (Ω -cm)	Sulfates		Chlorides	
Boring	Depth (feet)			ppm	%	ppm	%
B-1	5 – 10	7.8	4,100	30	0.003	32	0.003
T-4	0 - 4	6.6	3,700	48	0.005	44	0.004

3.6.5 Expansion Potential

A representative sample of the underlying soils was tested to determine expansion index (EI), after ASTM D4829. The sample at B-1 from 0 to 5 feet indicated EI = 0, characteristic of a soil with very low expansion potential. This consideration is discussed in more detail in Section 5.3.

3.6.6 Direct Shear

The mechanical characteristics (strength and compressibility) of the soils were tested as described below. Once sample of the weathered tonalite was tested in direct shear after ASTM D3080. The results of this testing are provided on Table 3-8.

Table 3-8. Abstract of the Direct Shear Testing

Sample Reference		Peak		Ultimate	
Boring	Depth (feet)	Cohesion (Psf)	Friction angle (Degrees)	Cohesion (Psf)	Friction angle (Degrees)
B-3 (remolded)	5 – 7	175	32.4	135	32.2

Note: referenced sample is remolded at 8% moisture

3.6.7 Consolidation Testing

One (1) ring sample was tested in one dimensional consolidation after ASTM D2435. This testing was modeled to emulate conditions, saturating the soil at slightly above the existing effective overburden stress.

1. Recompression. The samples were loaded to at or above the existing effective overburden stress.
2. Inundation. The samples were inundated, recording the soil response to wetting.
3. Continued Loading. Following stabilization after inundation, the samples were loaded to above 10,000 psf at a Load Increment Ratio (LIR) of 1, recording continued soil compression.
4. Rebound. The samples were unloaded and the rebound recorded.

The results of this testing are provided on Table 3-9 (following page).

Table 3-9. Abstract of Consolidation Testing

Sample Ref		As Sampled		Strain on Saturation		Soil Classification
Boring	Depth (feet)	Natural Moisture (%)	Dry Unit Weight	Saturation Pressure	Strain on Saturation	
B-3	7.5	5.2	123.5	2,200	-2%	SM

3.7 GSI Laboratory Testing

3.7.1 General

The lab results from soil samples recovered from the GSI 2019 report engineering borings are included in the following subsections.

3.7.2 Maximum Density and Optimum Moisture

One composite sample of the near-surface soil was tested to determine its moisture-density relationship after ASTM D1557 (the 'modified Proctor'). This testing indicated an optimum dry density (γ_{dry}) of $\gamma_{dry} = 136.3 \text{ lb/ft}^3$ at an optimum moisture content (w) of $w = 7.1\%$ for a composite sample at B-3.

3.7.3 Gradation

The visual classifications were further evaluated by determinations of grain size distribution of a composite sample from B-2 and B-3 from 2.5 to 10 feet. Table 3-10 summarizes of this testing.

Table 3-10. Abstract of the GSI Soil Gradation Testing

Sample Reference		Percent Finer than the U.S. No 200 Sieve	Classification after ASTM D 2488
Boring	Depth (feet)		
B-2/B-3	2.5 - 10	31	SM

200 sieve (0.074 mm), after ASTM D6913.

3.7.4 Expansion Potential

A representative sample of the underlying soils was tested to determine expansion index (EI), after ASTM D4829. The composite sample from B-2 and B-3 from 2.5 to 10 feet indicated $EI < 20$, characteristic of a soil with Very Low expansion potential. This consideration is discussed in more detail in Section 5.3

3.7.1 Direct Shear

Two samples of weathered tonalite were tested in direct shear after ASTM D3080. Table 3-11 (following page) provides a summary of this testing.

Table 3-11. Abstract of the GSI Direct Shear Testing

Sample Location		Primary		Residual	
Boring	Depth (feet)	Cohesion (Psf)	Friction angle (Degrees)	Cohesion (Psf)	Friction angle (Degrees)
B-3 (remolded)	5 – 7	242	39.2	106	35.9
B-6 (undisturbed)	0 - 5	98	42.2	98	34.9

4.0 SITE CONDITIONS

4.1 Geologic Setting

4.1.1 Regional

The area of San Diego County where the project is located, is within the Coastal Foothills physiographic province of the Peninsular Ranges Geomorphic Province. This 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.

In general, the province consists of rugged mountains underlain mostly by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The source rocks for soils in the Foothills section varies from granite to gabbro and generally consists of tonalite, granodiorite and gabbro. Young and old granitic alluvium occurs in broad basins, on alluvial fans, and in narrow drainage ways.

4.1.2 Site Specific

The site is underlain (from ground surface downward) by undifferentiated fill/colluvium, and Cretaceous-aged tonalite (Kt), a type bedrock very similar to granite with a slightly different mineral composition. The undifferentiated fill/colluvium thicknesses encountered during the NOVA and GSI field investigations range from about 2 feet in the north up to 15.5 feet in the southwest. For simplicity, within this report, the fill and colluvial soils will be undifferentiated and referred to as fill (Qaf).

The tonalite bedrock is dense. The subsurface exploration indicates that the upper several feet of the bedrock are weathered and fractured, with the rock becoming denser and less fractured with depth.

Figure 4-1 (following page) reproduces mapping of the surface geologic units in the site area.

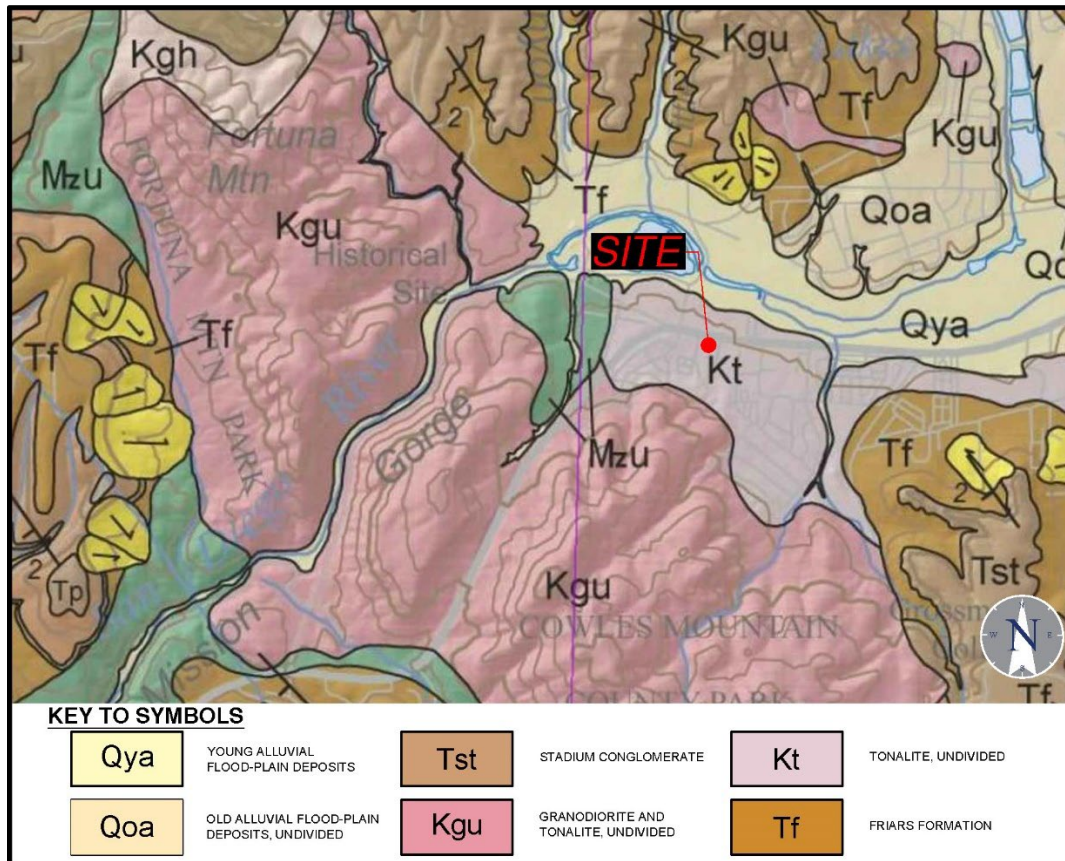


Figure 4-1. Surface Geologic Units in the Site Vicinity

4.2 Surface, Subsurface, and Groundwater

4.2.1 Surface

The site is currently occupied by existing residential structures, mobile homes, pavement areas, CMU retaining walls, and dense coverage of trees. Figures 4-2a through 4-2d (following pages) depict surface conditions at the time of NOVA's work.

The site slopes from the old water bottling facility on the northern boundary upward in elevation to the southern boundary. The site ranges in elevation from approximately +360 feet mean sea level (msl) near Mission Gorge Road to approximately +430 feet msl in the southwest corner. This ± 70 -foot elevation differential occurs over about 575 feet, a surface gradient of $\sim 12\%$.



Figure 4-2a. View of Southern Portion of Site, Looking East, February 2020



Figure 4-2b. View of Southeast Portion of Site, Looking South, February 2020



Figure 4-2c. View of Southwest Portion of Site, Looking West, February 2020



Figure 4-2d. View of Middle Portion of Site, Looking North, February 2020

4.2.2 Subsurface

For the purposes of this report, the subsurface may be generalized to occur as the sequence of soil and rock described below.

1. Unit 1, Fill/Colluvium. The site is covered by a veneer of undifferentiated fill/colluvium that ranges from 2 to 15.5 feet in thickness. The undifferentiated fill/colluvium is comprised of brown silty sands of loose to medium dense consistency. Locally, the undifferentiated fill/colluvium includes cobble-size to boulder-size pieces of granite. Because no records exist regarding the placement of this soil, the undifferentiated fill/colluvium is 'undocumented,' and at risk for wide variations in quality. For simplicity, within this report, the fill and colluvial soils will be undifferentiated and referred to as fill (Qaf).

Figure 4-3 (following page) depicts this layer within trench T-1.

Unit 2, Tonalite. Cretaceous-aged tonalite rock (Kt) forms the basement rock for the entire site. And increases in density with depth as the unit becomes less weathered and fractured. See Section 3.4 for description of density with depth.

The upper several feet of the naturally occurring subsurface is generally weathered tonalite, decomposed to the consistency of a medium to coarse, light brown to gray silty sand. Though dominated by sandy soils, this unit may include zones of localized extremely hard-core stones or "floaters", as well as gravelly zones. Core stones were not encountered during the investigation, but are characteristically encountered in tonalite bedrock.

Figure 4-4 (following page) depicts the weathered tonalite in the near subsurface.



Figure 4-3. Unit 1 Fill, T-1, 2/27/20



Figure 4-4. Unit 2 Weathered Tonalite, 2/27/20

4.2.3 Groundwater

Groundwater was not encountered in geotechnical borings that were advanced to 20.5 feet below ground surface, and is not anticipated to be a constraint to development.

4.2.4 Surface Water

No surface water was evident on the site at the time of NOVA's work. There was no visual evidence at the time of NOVA's field exploration of seeps, springs, or surface erosion that would suggest active problems with surface water.

4.3 Planned Site Grading

Plate 2 and Plate 3 (provided immediately following the text of this report) provide parallel south to north cross-sections through the planned development. Cross section locations are shown on Figure 3-1 and Plate 1.

As may be seen by review of these Plates, construction of the planned development will generally involve excavation of soils near the higher, southern portion of the site and placement of new fill across the lower northern portions of the site.

Cuts will extend to a maximum of about 23 feet in depth, with the thickest fills ranging to about 15 feet in height.

Foundations for new structures will be placed in both Unit 2 tonalite and new engineered fill. Several structures will transition across these two units. Grading to mitigate differential settlement caused by transition conditions are discussed in Section 6.4.

5.0 REVIEW OF GEOLOGIC, SOIL AND SITING HAZARDS

5.1 Overview

This section provides a review of geologic, soil and siting-related hazards common to this region of California, considering each for its potential to affect the planned development. Two hazards were identified by this review, as are described below.

1. Strong Ground Motion. The site is at risk for moderate-to-severe ground shaking in response to large-magnitude earthquakes generated during the lifetime of future 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.
2. Difficult Excavation. Sound granitic rock underlies the entire site. Locally, this rock occurs within the near subsurface, within depths planned for excavation for building pads and infrastructure. In areas within the southern portion of the site, where there are significant cuts, the sound rock may be difficult to loosen for removal, necessitating breaking and/or blasting, thus adding cost and time to the planned earthwork.

In addition, floaters, or localized non-rippable zones of tonalite may be encountered within areas identified by the field investigations as being 'trenchable' or 'rippable'. These generally require special breaking techniques.

Breaking and blasting often result in oversized material which will require offsite disposal.

The following subsections review the geologic, soil and siting hazards considered for this site.

5.2 Geologic Hazards

5.2.1 Strong Ground Motion

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion is considered significant during the design life of the proposed structure. Major known active faults in the region consist generally of in echelon, northwest striking, right-lateral, strike-slip faults. These include the San Andreas, Elsinore, and San Jacinto faults located northeast of the site, and the San Clemente, San Diego Trough, and Agua Blanca-Coronado Bank faults located to the west of the site.

The tectonic setting of the metropolitan San Diego area includes major north and northwest striking fault zones, the most prominent and active of which is the Rose Canyon fault zone (RCFZ). The RCFZ, is estimated to have the potential to generate earthquakes up to $MW = 6.9$.

A web-based tool provided by the American Society of Civil Engineers (ASCE) was used to estimate a corresponding risk-based Peak Ground Acceleration (PGA_M) of $PGA_M \sim 0.412$ g.

5.2.2 Fault Rupture

No evidence of faulting was observed during NOVA's geologic reconnaissance of the site. Because of the lack of known active faults on the site, the potential for surface rupture at the site is considered 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.

The nearest active fault is the San Diego section of the Newport Inglewood Rose Canyon fault zone, which is approximately 10.4 miles west of the site. Faults are considered active if there is known movement on the fault within the Holocene Period (11,700 years).

Figure 5-1 (following page) reproduces published mapping of active faulting in the site vicinity.

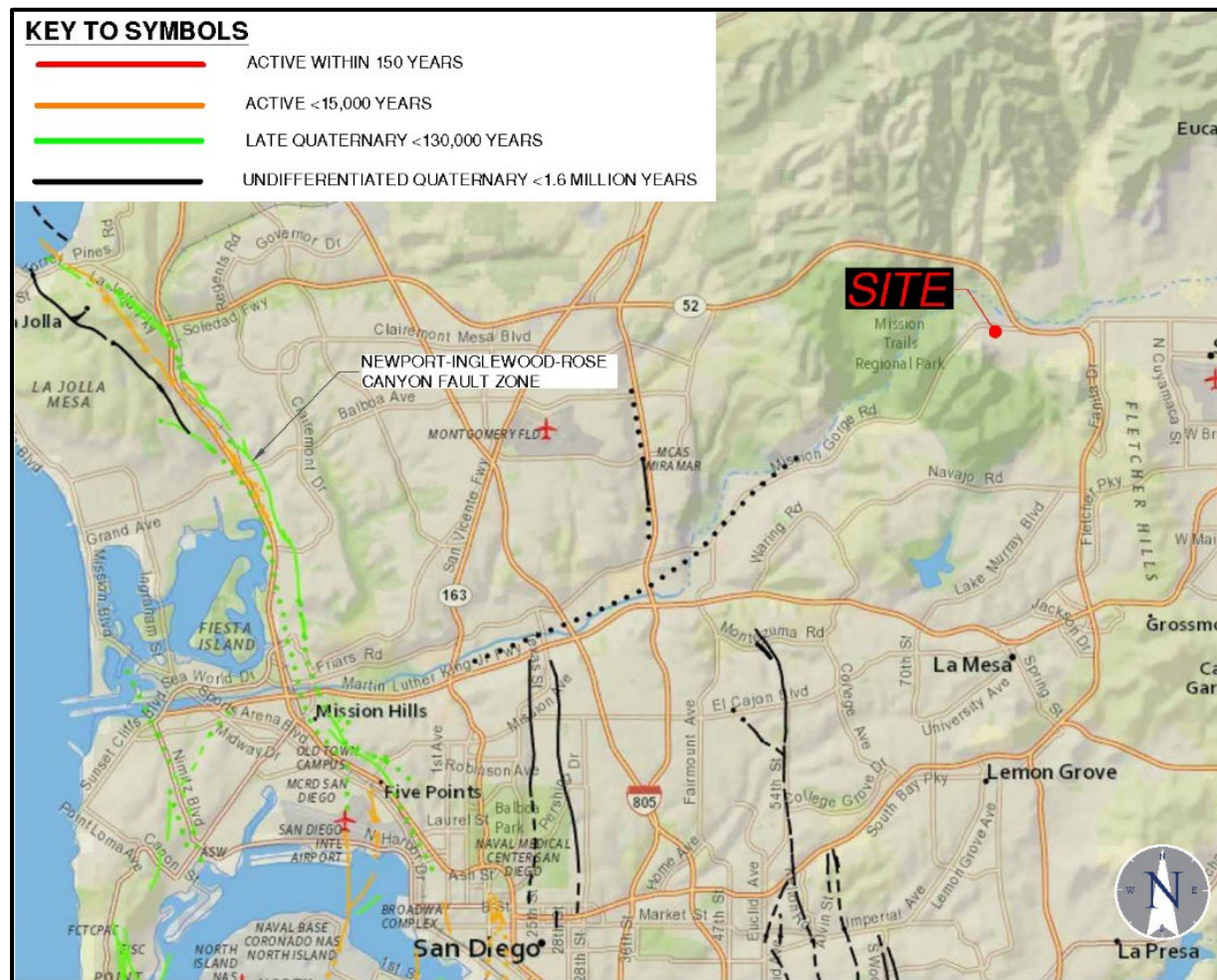


Figure 5-1. Faulting in the Site Vicinity

5.2.3 Ground Lurching

Seismically induced ground lurching occurs when soil or rock masses move at right angles to a cliff or steep slope in response to seismic waves. Structures built on these masses can experience significant lateral and vertical deformations if ground lurching occurs.

The phenomenon is usually associated with soft, unconsolidated soils with low cohesion adjacent to a slope. The granitic subsurface of this site is not at risk for ground lurching.

5.2.4 Landslide

As used herein, 'landslide' is intended to describe downslope displacement of a mass of rock, soil, and/or debris by sliding, flowing, or falling. Such mass earth movements may be greater than about 10 feet thick and larger than 300 feet across. Landslides typically can include cohesive block glides and disrupted slumps that are formed by translation or rotation of slope materials along one or more slip surfaces. These mass displacements can also include similarly large, 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, landsliding has also been precipitated by earthwork and grading, destabilizing slopes by cutting and/or filling on existing adverse geologic structure.

Geologic reconnaissance and review of aerial photography indicated no evidence of active or dormant landsliding. Further clues to landslide hazards can also be obtained by review of mapping that depicts both historic landslides and landslide-prone topography. Figure 5-2 (following page) reproduces such mapping for the site area. The mapping indicates that the site is in an area judged to be 'generally susceptible (3-1)' to landsliding, though the Santee General Plan Safety Element identifies the site to be in an area designated as "least susceptible to landslides".

In consideration of the existing site topography, geologic reconnaissance and review of published information, NOVA considers the site to be at low risk for landsliding in its current and proposed configuration.

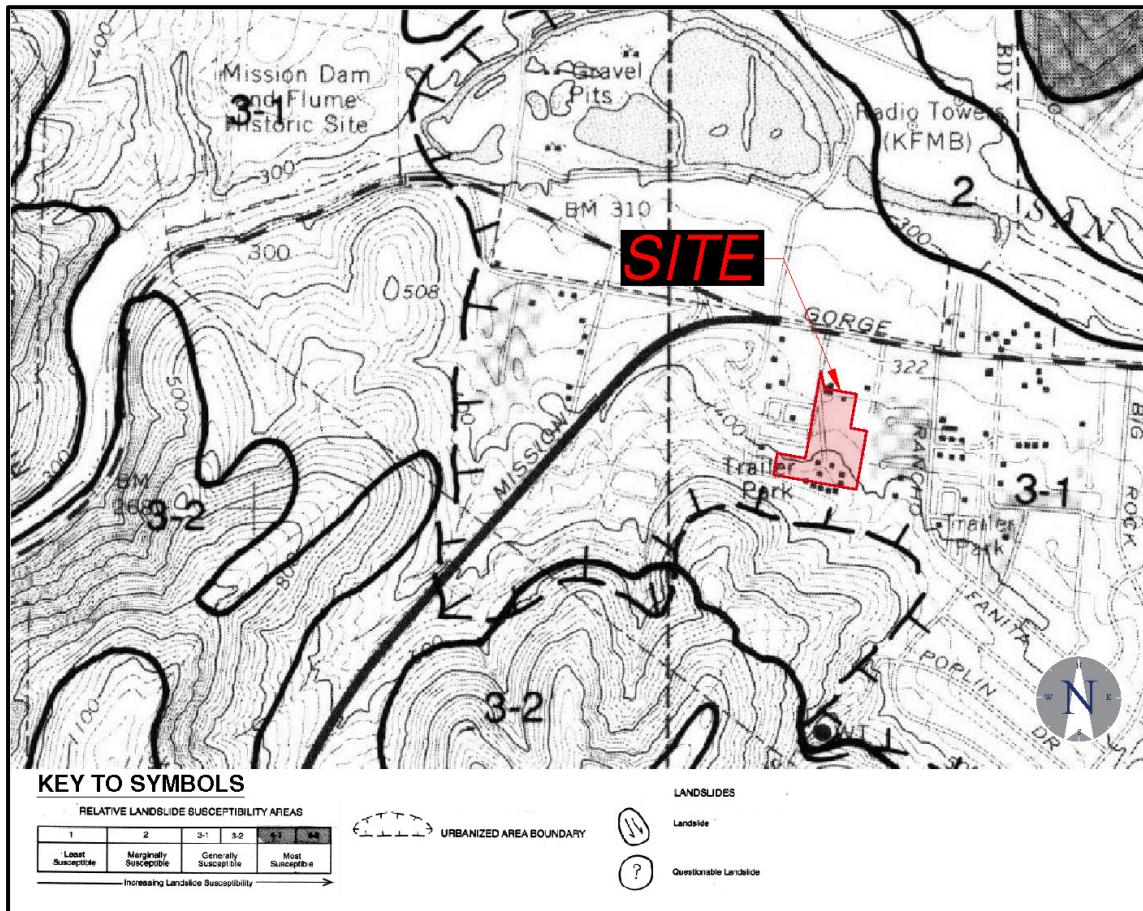


Figure 5-2. Landslide Risk in the Site Area
(source: adapted from Tan 1995)

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 as described above, embankment stability can include smaller scale slope failures such as erosion-related washouts and more subtle, less evident processes such as slope 'creep.'

The groundform of the predominantly granitic subsurface of this site is not at risk for generalized embankment instability. As is discussed in Section 2, development of the site is expected to include construction of several conventional retaining walls and at least one mechanically stabilized earth (MSE) retaining wall. These walls will be constructed to adapt the sloping ground to a profile suitable for development of building pads for the various residential structures. Retaining walls will be developed to varying heights ranging to a maximum of about 17 feet for the proposed MSE wall on the southern property boundary.

Based upon the indications of the subsurface explorations reported herein, and as is discussed in more detail in Section 6, MSE walls will be readily adapted to this site. Each wall will be

evaluated for internal and global stability, with documentation provided for each evaluation. MSE walls will all be developed with a factor of safety (FS) against global instability of $FS \geq 1.5$.

5.3.2 Seismic

Liquefaction

'Liquefaction' refers to the loss of soil strength during a seismic event. The phenomenon is observed in areas that include a shallow water table and coarse-grained (i.e., 'sandy') soils of loose to medium dense consistency. The ground motions increase soil water pressures, decreasing grain-to-grain contact among the soil particles, causing the soil mass to lose strength. Liquefaction resistance increases with increasing soil density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history.

The geologically older, dense soil and rock at this site are not at risk of liquefaction.

Seismically Induced Settlement

Ground shaking associated with a seismic event can affect settlement of unsaturated soils. However, this phenomenon is only observed in cohesionless (sandy) soils.

The Unit 2 weathered tonalite has no risk of seismic settlement.

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 and unconstrained, free to move along sloping ground.

Due to the absence of a potential for liquefaction and relatively flat surrounding topography, there is no potential for lateral spreading.

5.3.3 Expansive Soil

Expansive soils are characterized by their ability to undergo significant volume changes (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from precipitation, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors and may result in unacceptable settlement or heave of structures or concrete slabs supported on grade.

A representative sample of the underlying soils were tested to determine expansion index (EI), after ASTM D4829. The sample at B-1 from 0 to 5 feet indicated $E < 20$, characteristic of a soil with Very Low expansion potential after ASTM D4829.

5.3.4 Hydro-Collapsible Soils

Hydro-collapsible soils are common in the arid climates of the western United States in specific depositional environments- principally, in areas of young alluvial fans, debris flow sediments, and loess (wind-blown sediment) deposits. These soils are characterized by low *in situ* density, low moisture contents, and relatively high unwatered strength. The soil grains of hydro-collapsible

soils were initially deposited in a loose state (i.e., high initial 'void ratio') and thereafter lightly bonded by water sensitive binding agents (e.g., clay particles, low-grade cementation, etc.).

While relatively strong in a dry state, the introduction of water into these soils causes the binding agents to fail. Destruction of the bonds/binding causes relatively rapid densification and volume loss (collapse) of the soil. This change is manifested at the ground surface as subsidence or settlement. Ground settlements from the wetting can be damaging to structures and civil works. Human activities that can facilitate soil collapse include irrigation, water impoundment, changes to the natural drainage, disposal of wastewater, etc.

The consistency and geologic age of the underlying tonalite is such that these units are not potentially hydro-collapsible.

5.3.5 Near Surface Rock

As is discussed in Section 3 and Section 4, the sound granitic rock underlies the entire site. Locally, this rock occurs within the near subsurface, within depths planned for excavation for building pads and infrastructure. The sound rock may be difficult to loosen for removal, adding cost and time to the planned earthwork.

5.3.6 Corrosive Soil

Concentrations of water-soluble sulfates are an index of the potential for sulfate attack to embedded concrete. Electrical resistivity, chloride content, and pH level are all indicators of the soil's tendency to corrode ferrous metals.

Laboratory testing indicates that the onsite soils will not be corrosive to embedded concrete and ferrous metals. This consideration is discussed in more detail in Section 6.

5.4 Siting Hazards

5.4.1 Effect on Adjacent Properties

The proposed project will not affect the structural integrity of adjacent properties or existing public improvements and street rights-of-way located adjacent to the site if the recommendations of this report are incorporated into project design.

5.4.2 Flood

The site is not located within a FEMA-designated flood zone. Flood map No. 06073C1880G (May 16, 2012) designates the site as "Zone X," an area of minimal annual flood hazard. Figure 5-3 reproduces flood mapping by FEMA of the site area.

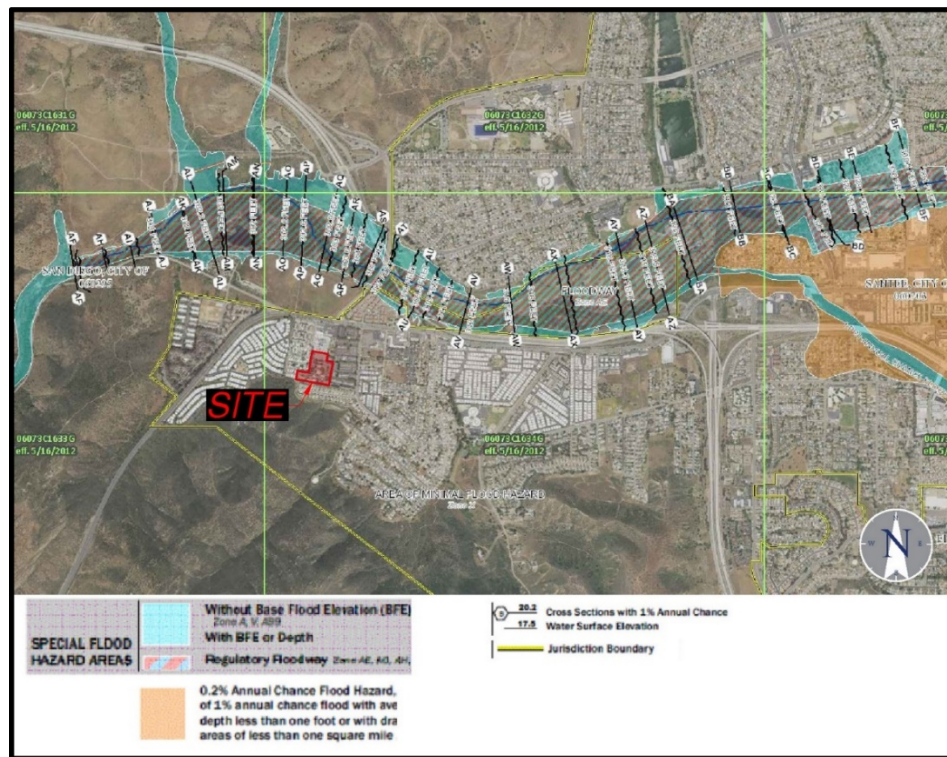


Figure 5-3. Flood Mapping of the Site Area
(source: adapted from FEMA Panel 06073C1880G, effective 5/16/2012)

5.4.3 Tsunami and Seiche

Tsunami describes a series of fast-moving, long-period ocean waves caused by earthquakes or volcanic eruptions. The altitude and distance of the site from the ocean preclude this threat.

Seiches are standing waves that develop in an enclosed or partially enclosed body of water such as lakes, reservoirs, and harbors and inlets. The site is not located near a body of water that could generate a seiche.

6.0 EARTHWORK AND FOUNDATIONS

6.1 Overview

6.1.1 Review of Site Hazards

Section 5 provides review of soil and geologic hazards common to development of civil works in the project area. Two hazards were identified by that review, as are described below.

1. Strong Ground Motion. The site is at risk for moderate-to-severe ground shaking in response to large-magnitude earthquakes generated during the lifetime of future 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.
2. Difficult Excavation. Sound granitic rock underlies the entire site. Locally, this rock occurs within depths planned for excavation for building pads and infrastructure. The sound rock may be difficult to mechanically loosen for removal, adding cost and time to the planned earthwork.

This section provides design guidance for development of the site, including guidance for design and construction of foundations, retaining walls, roadways and utilities. Section 6.2 provides seismic parameters for design of structures. Section 6.4 provides guidance for earthwork, including planning to address difficult-to-excavate rock that may occur in the near subsurface.

6.1.2 Site Suitability

Based upon the indications of the field and laboratory data developed for this investigation, as well as review of previously developed subsurface information, it is the judgment of NOVA that the site is suitable for proposed development, provided the geotechnical recommendations described herein are followed.

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

There are several proposed retaining walls located directly on project property lines. As such, excavations/backcuts for the footings will likely extend into neighboring properties.

6.1.3 Review and Surveillance

It is intended that the recommendations provided herein be sufficient to develop the project in general accordance with the 2019 California Building Code (CBC) requirements.

NOVA should review the grading plans, foundation plans, 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.

Section 9 design addresses review and construction surveillance in more detail.

6.2 Seismic Design Parameters

6.2.1 Site Class

The Site Class has been determined from ASCE 7, Table 20.3-1. The depth of soil information available for this site is limited. However, as is well-established by the geologic literature and by experience at numerous nearby sites, the stiffer soil and rock units that occur in the near-surface extend to depths greater than 100 feet, such that the site may be classified as Site Class C per ASCE 7-16 (Table 20.3-1).

6.2.2 Seismic Design Parameters

Table 6-1 provides seismic design parameters in accordance with ASCE 7-16.

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

Parameter	Site Class C
Site Latitude, degrees	32.834178
Site Longitude, degrees	-117.028233
Mapped Short Period Spectral Acceleration, S_S	0.798
Mapped One-Second Period Spectral Acceleration, S_1	0.291
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	0.958
One-Second Period Spectral Acceleration Adjusted For Site, S_{M1}	0.437
Design Short Period Spectral Acceleration, S_{DS}	0.639
Design One-Second Period Spectral Acceleration, S_{D1}	0.291
Peak Ground Acceleration, PGA	0.343
Peak Ground Acceleration Adjusted For Site, PGA_M	0.412

6.3 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. Levels of water-soluble sulfates are correlated with the potential for sulfate attack to concrete. These chemical tests were performed on representative samples of the near-surface soils. The results of the testing are tabulated in Table 6-2 (following page).

Table 6-2. Abstract of Chemical Testing

Sample Ref		pH	Resistivity (Ω -cm)	Sulfates		Chlorides	
Boring	Depth (feet)			ppm	%	ppm	%
B-1	5 – 10	7.8	4,100	30	0.003	32	0.003
T-4	0 - 4	6.6	3,700	48	0.005	44	0.004

6.3.2 Metals

Caltrans considers a soil to be corrosive 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 (0.2%) or greater, or
- the pH is 5.5 or less.

Based on the Caltrans criteria, the on-site soils would not be considered 'corrosive' to buried metals. Appendix D provides records of the chemical testing that include estimates of the life expectancy of buried metal culverts of varying gauge.

In addition to the above parameters, the risk of soil corrosivity affecting buried metals is considered by determination of electrical resistivity (ρ). Soil resistivity may be used to express the corrosivity of soil only in unsaturated soils. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of DC electrical current from the metal into the soil. As the resistivity of the soil decreases, the corrosivity generally increases. A common qualitative correlation (cited in Romanoff 1989, NACE 2007) between soil resistivity and corrosivity to ferrous metals is tabulated below.

Table 6-3. Soil Resistivity and Corrosion Potential

Minimum Soil Resistivity (Ω -cm)	Qualitative Corrosion Potential
0 to 2,000	Severe
2,000 to 10,000	Moderate
10,000 to 30,000	Mild
Over 30,000	Not Likely

Despite the relatively benign environment for corrosivity indicated by pH and water-soluble chlorides, the resistivity testing suggests that design should consider that the soils may be corrosive to embedded ferrous metals.

Typical recommendations for mitigation of such corrosion potential in embedded ferrous metals include:

- a high-quality protective coating such as an 18-mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar;
- electrical isolation from above grade ferrous metals and other dissimilar metals by means of dielectric fittings in utilities and exposed metal structures breaking grade; and,

- steel and wire reinforcement within concrete having contact with the site soils should have at least 2-inches of concrete cover.

If extremely sensitive ferrous metals are expected to be placed in contact with the site soils, it may be desirable to consult a corrosion specialist regarding choosing the construction materials and/or protection design for the objects of concern.

6.3.3 Sulfate Attack

As shown in Table 6-2, the soil samples tested indicated water-soluble sulfate (SO_4) content of 30 parts per million ('ppm,' 0.003% by weight) for the Unit 1 Fill and 48 parts per million ('ppm,' 0.005% by weight) for the Unit 2 tonalite. With $\text{SO}_4 < 0.10\%$ by weight, the American Concrete Institute (ACI) 318-08 considers the soil to have negligible potential (S0) potential for sulfate attack to embedded concrete.

Table 6-4 reproduces the Exposure Categories considered by ACI.

Table 6-4. Exposure Categories and Requirements for Water-Soluble Sulfates

Exposure Category	Class	Water-Soluble Sulfate (SO_4) In Soil	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 of the 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 in the field of corrosion protection, since this is not specifically a geotechnical issue. Should you require more information, a specialty corrosion consultant should be retained to address these issues.

6.4 Earthwork

6.4.1 General

Based upon the design concept that is currently understood, NOVA expects that earthwork will include (i) grading to create the new building pads, (ii) excavations for foundations for walls and structures, and (iii) excavation/backfill for 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 Excavation Characteristics, Planning and Contracting

As is discussed in Section 3, Section 4, and Section 5, tonalite bedrock underlies the entire site. Locally, this rock occurs within depths planned for excavation for building pads and infrastructure. Where this rock is unweathered, sound rock may be difficult to mechanically loosen for removal, adding cost and time to the planned earthwork.

The discussion in this section is intended to address potential difficulties with this earthwork.

Fill and Weathered Tonalite. The upper interval of the subsurface is fill and material derived from in place weathering of the parent tonalite. The interval is characterized by dense sandy soils. However, this interval could include zones of sound rock that will require special excavation and breaking methods. NOVA expects that the vast majority of this interval will be able to be excavated using heavy earthmoving equipment.

In addition, floaters, or localized non-rippable zones of tonalite may be encountered within areas identified by the field investigations as being ‘trenchable’ or ‘rippable’. These generally require special breaking techniques.

It should be noted that NOVA and GSI encountered a significant amount of material over 6-inch in maximum dimension in the fill and weathered tonalite that will require removal prior to using this material for engineered fill as discussed in Section 6.4.3.

Unweathered Tonalite. As the weathering decreases with depth, discontinuities (joints and fractures) will diminish in frequency. The compressive strength of unfractured tonalite can be similar to that of a low-grade concrete. The ability of ripping to mechanically loosen this interval will be dependent upon the frequency of fractures and joints. Based on results of the seismic survey, it is expected that the upper perhaps 15 feet of this unit will generally be rippable, though localized areas of sound, unfractured rock may require breaking techniques or blasting. Breaking and blasting often result in oversized material, not suitable for use as fill, and will require special handling or offsite disposal.

As is discussed in Section 3.4, the velocity data obtained from the seismic traverses reported in GSI 2019 suggest that most of this unit that is within the limits of the planned excavations will be rippable. Care should be taken in use of these data. While measurements of seismic velocity provide a comforting, semi-quantitative estimates of the potential for rippability, it should be understood that numerous variables are associated with actual rippability. Potential contractors for this work should be screened for experience with rock of this nature.

Contracting for Earthwork

The complexity of the residual/weathered soil profile - in particular, the dramatic differential in strength between Unit 1 soils and the Unit 2 rock- complicates planning for earthwork. It is NOVA’s experience that questions regarding ‘*What is rock?*’ often drive cost overruns and related post-construction litigation in earthwork projects in soil profiles such as occur at this site.

As is noted above, despite a well-developed base of subsurface information, pre-determination of actual rippability is difficult. Potential contractors for this work should be

screened to limit bidding to those with experience excavating rock and subsurface conditions similar to the site.

NOVA recommends that planning for earthwork for this project consider establishing contractual criteria for 'rock' prior to initiating excavation. For example, "rock" may be defined based upon the nature of the work and the excavation equipment, as are driven by subsurface conditions. A common recommendation/specification is cited below.

1. General Excavations. Rock is any material which cannot be loosened with (i) a single tooth ripper drawn by a crawler tractor having a drawbar pull rated at not less than 56,000 pounds (for example, a Caterpillar D8, or equivalent), or (ii) a front-end loader with a minimum bucket breakout force of 25,600 pounds (for example, a Caterpillar 977 or equivalent).
2. Trench Excavation. Rock is any material which cannot be excavated with a backhoe having a bucket curling force rated at not less than 33,000 pounds and a bucket equipped with excavation teeth (for example, a Caterpillar 225B or equivalent).

6.4.3 Select Fill

Materials

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

- at least 40% by weight finer than ¼-inches in size and,
- maximum particle size of 5 inches, and
- expansion index (EI) less than 30 (i.e., $EI < 30$, after ASTM D 4829).

Most of the Unit 1 fill and the weathered portions of the Unit 2 tonalite will conform to the above criteria; however, within the near-surface of both units, tonalite cobbles exceeding 5 inches was observed and documented. In some areas screening operations may be useful to help remove oversized material. In addition, backfill for MSE walls may require specific particle size specifications as recommended in Section 6.8.1 and by the wall designer.

Placement

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

The fill must be densified using specialty compaction equipment and methods appropriate to the soil type. Vibratory compaction should be used for dominantly sandy soils. Kneading and static compaction should be used for dominantly finer-grained soils.

Fill should be placed in loose lifts no thicker than the ability of the vibratory 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 used in constrained spaces (e.g., walk-behind compactors used in utility trenches) will be limited to about 4 inches or less.

6.4.4 Site Preparation

Establish Erosion and Sedimentation Control

Construction-related erosion and sedimentation must be controlled in accordance with Best Management Practices, as well as current City and State requirements. These controls should be established at the outset of site disturbance.

Demolition and Clearing

The site will be required to be cleared prior to construction, including demolition of the existing concrete slabs, walls and related pavements. Disposal of all demolition debris should be in accordance with local, state, and Federal regulations. Any existing utilities that are not planned to be reused should be removed, along with any unsuitable backfill materials, and capped at the property lines, or rerouted around the property and reconnected.

Underground utilities within the proposed structure footprint should be grouted in place or removed. Clearing, including the removal of any abandoned utilities, should be extended a minimum of 5 feet beyond the building and pavement limits. Care should be exercised during site grading so as to not damage any utilities that are to remain.

Following demolition operations, the exposed ground surface should be cleared of any soft or unsuitable materials or debris. Where possible, the stripping of the proposed building areas should be extended at least 5 feet beyond the planned limits of the proposed new structure (including pavements).

6.4.5 Grading for Buildings

Within the conceptual grading plan, there are three conditions that will occur for the structures: (i) structures founded entirely on bedrock cut; (ii) structures founded entirely on engineered fill; and (iii) structures with transition conditions, meaning foundations spanning cut and fill. The following subsections address earthwork procedures for each condition.

In all cases, the excavation bottoms should be approved by a NOVA geologist and elevations should be documented.

Buildings Founded on Unit 2 Tonalite

In areas in which foundations are bearing entirely on the tonalite bedrock, foundations may be excavated directly into the bedrock.

Buildings Founded on Engineered Fill

Within building areas, and extending to at least 5 feet outside building limits, the Unit 1 undocumented fill should be removed to the level of Unit 2. The soil surface loosened and exposed by this excavation should be scarified and re-densified to 90% relative compaction after ASTM D1557. Thereafter, excavations may be backfilled with

Select Fill (see Section 6.4.2). All building foundations should be underlain by a minimum of two feet of engineered fill.

Buildings with a Cut/Fill Transitions

Where building foundations are supported partially on engineered fill and partially on Unit 2 tonalite, all areas of the ground supported slabs and structure foundations should be over excavated as needed, such that they are underlain by a minimum of three feet of Select Fill.

Where this circumstance occurs, grading should be completed using the steps described below.

1. *Step 1, Over-Excavate.* Over-excavate the Unit 2 rock to a depth of 3 feet below the pad and bottom of footing level, extending this excavation at least 5 feet laterally outside the building limits.
2. *Step 2, Backfill.* Backfill the over excavated area to the base of the ground-level slab with Select Fill.

6.4.6 Over Excavation of Utilities

Utility improvement plans are not available at the time of this report; however, within the southern portion of the site, cuts up to approximately 25 feet bgs are anticipated for utility installation.

According to seismic survey findings presented in GSI 2019, at this depth, the tonalite bedrock is considered rippable but not trenchable. Over excavation of utilities during rough grading operations may be considered in areas where the deepest utility exceeds 3 feet into the tonalite bedrock, where the rock reportedly transitions from trenchable to rippable.

6.4.7 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. No utility should be aligned beneath footings within a projected 1H:1V limit from the edge of the footing to the base of the utility trench.

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% relative compaction after ASTM D 1557 (the 'modified Proctor') at soil moisture +2% 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% relative compaction with respect to the Modified Proctor.

Compaction testing should be performed for every 20 cubic yards of backfill placed or each lift within 30 linear feet of trench, whichever is less.

Backfill of utility trenches should not be placed with water standing in the trench. If granular material is used for the backfill, the material should have a gradation that will filter protect the backfill material from the adjacent soils. If this gradation is not available, a geosynthetic non-woven filter fabric should be used to reduce the potential for the migration of fines into the backfill material.

6.4.8 Southwest Fill Slope

In the southwestern portion of the site, there is an existing fill slope descending from the project down to the trailer park property to the north. Findings from the field investigation indicate that this 5 to 15-foot fill varies in condition from very loose to medium dense.

A 5-foot retaining wall is proposed near the top of the fill slope, which supports the east-west trending street that serves the southernmost condominiums. Based on the existing fill condition and proposed site configuration, existing fill within this slope should be removed. The new toe of slope and fill slope should be stabilized by construction of a keyway that is a minimum of 10 feet wide, excavated a minimum of 2 feet into competent Unit 2 bedrock, tipped into the slope a minimum of 5 degrees, and filled with Select Fill Material.

As the fill slope is brought up, it should be benched into the existing material on the south side of the excavation. Compaction of fill slopes should be performed by back-rolling with a sheepsfoot compactor at vertical intervals of four feet or less as the fill is being placed, and track-walking the face of the slope when the slope is completed. Alternatively, the fill slopes may be overfilled by at least 3 feet and then cut back to the compacted core at the design line and grade.

6.4.9 Flatwork

Prior to casting exterior flatwork, the upper 1-foot of subgrade soils should be removed and replaced with "Select" fill, moisture conditioned and recompacted, as recommended in Section 6.4.5.

After excavation and prior to replacement, the exposed soils (bottom of excavation) should be examined by the Geotechnical Engineer-of-Record (GEOR) to identify any localized loose, yielding or otherwise unsuitable materials. Prior to the placement of replacement fill, the soils loosened by excavation should be moisture conditioned to at least 2% over optimum moisture content to a depth of 12 inches, then recompacted to at least 90% relative compaction after ASTM D-1557.

Concrete slabs for pedestrian traffic or landscaping should be at least 4 inches thick.

6.5 Shallow Foundations

6.5.1 General

Structures can be supported on shallow foundations embedded in either engineered fill or Unit 2 tonalite as described in Section 6.4.

The following subsections provide recommendations for shallow foundations. It is recommended that all footings, including any grade beams, be reinforced top and bottom. The actual reinforcement should be designed by the Structural Engineer.

6.5.2 Minimum Dimensions

Continuous footings should be at least 24 inches wide and have a minimum embedment of 24 inches below lowest adjacent grade. Isolated square or rectangular footings should be a minimum of 36 inches in smallest dimension, embedded at least 24 inches below surrounding grade.

Footings proposed adjacent to the top of the slopes will need a minimum horizontal setback of 8 feet between the outer lower edge of the footing to the adjacent slope face.

6.5.3 Allowable Contact Stress

Continuous and isolated footings constructed as described above may be designed using an allowable (net) contact stress of 4,000 pounds per square foot (psf).

The bearing values apply to combined dead and sustained live loads (DL + LL). The allowable bearing values may be increased by one-third when considering transient loads such as seismic and wind.

6.5.4 Lateral Resistance

Resistance to lateral loads will be provided by a combination of (i) friction between the soils and foundation interface, and (ii) passive pressure acting against the vertical portion of the footings. Passive pressure may be calculated at 250 psf per foot of depth. An interface frictional coefficient of 0.35 may be used. No reduction is necessary when combining frictional and passive resistance.

6.5.5 Settlement

Structures supported on shallow foundations as recommended above will settle on the order of 0.5-inch or less, with about 70% of this settlement occurring during the construction period.

Angular distortion due to differential settlement of adjacent, unevenly loaded footings will be less than 1-inch in 40 feet (i.e., Δ/L less than 1:480).

6.5.6 Ground Supported Slabs

The ground level of garage structures may employ conventional on-grade (ground-supported) slabs designed using a modulus of subgrade reaction (k) of 100 pounds per cubic inch (i.e., $k = 100 \text{ pci}$).

The actual slab thickness and reinforcement should be designed by the Structural Engineer. NOVA recommends slabs be a minimum 5 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 at least $\frac{1}{4}$ the slab thickness, as depicted in Figure 6-1 (following page).

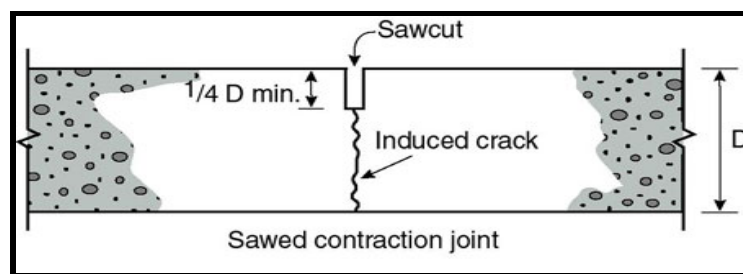


Figure 6-1. Sawn Contraction Joint

6.6 Moisture Barrier Beneath Slabs

6.6.1 Capillary Break

NOVA recommends that ground supported slabs include a capillary break. 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% coarser than the 1-inch sieve or more than 10% finer than the No. 4 sieve, such as AASHTO Coarse Aggregate No. 57.

6.6.2 Vapor Barrier

Ground-supported slabs beneath moisture sensitive equipment or enclosures should include a vapor membrane. Membranes set below floor slabs should be rugged enough to withstand construction. If a vapor barrier is desired, a minimum 15-mil polyethylene membrane should be placed over the porous fill to preclude floor dampness.

NOVA recommends that a minimum 15-mil low permeance vapor membrane be used. For example, Carlisle-CCW produces the Blackline 400® underslab, vapor and air barrier, a 15-mil low-density polyethylene (LDPE) rated at 0.012 perms after ASTM E 96.

6.6.3 Limitations of This Recommendation

Recommendations for moisture barriers are traditionally included with geotechnical foundation recommendations, though these requirements are primarily the responsibility of the Structural Engineer or Architect. If there is particular concern regarding moisture-sensitive materials or equipment to be placed above the slab-on-grade, a qualified person (for example, such as the flooring subcontractor and/or Structural Engineer) should be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. NOVA does not practice in the field of moisture vapor transmission since this is not specifically a geotechnical issue.

6.7 Conventional Retaining Walls

6.7.1 Shallow Foundations

Conventionally designed and reinforced concrete retaining walls should be developed on ground prepared in accordance with criteria provided in Section 6.4. Continuous shallow foundations may be designed in accordance with the criteria provided in Section 6.5. Retaining wall foundations should not be permitted to span cut and fill transitions. Foundations should bear entirely on formational soils or entirely on engineered fill.

If footings bear on fill, the footings should be over-excavated to a minimum of 2 feet below the bottom of footing and 3 feet laterally beyond the footing footprint. The bottoms of the excavation should be scarified to a minimum depth of 12 inches, and moisture conditioned per specifications in Section 6.4.

Construction of conventional retaining walls on the property lines as indicated on the conceptual grading plan, will likely result in excavations and back cuts that will extend beyond the subject property into neighboring properties. The degree of encroachment upon the neighboring properties will be directly correlated to the depth of remedial removals required for the retaining wall foundation.

6.7.2 Lateral Earth Pressures

Lateral earth pressures to walls are related to the type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. The groundwater level will be well below wall levels.

Table 6-6 provides recommendations for lateral soil and groundwater wall loading to below-grade walls with level backfill for varying conditions of wall yield.

Table 6-6. Wall Lateral Loads from Soil

Condition	Equivalent Fluid Pressure (psf/foot)	
	Level Backfill	2:1 Backfill Sloping Upwards
Active	35	65
At Rest	35	80
Passive	350	330

If footings or other surcharge loads are located a short distance outside the wall, these influences should be added to the lateral stress considered in the design of the wall.

6.7.3 Seismic

Non-Yielding Walls

Lateral seismic thrust acting on non-yielding below-grade walls can be estimated by the dynamic (seismic) thrust, ΔP_E . Dynamic thrust is approximated as:

$$\Delta P_E = k_h H^2 \gamma \quad \text{where,}$$

k_h , pseudostatic horizontal earthquake coefficient, equal to $S_{DS}/2.5$

H is the height of the wall in feet from the footing to the point of fixity

γ is equal to the unit weight of the backfill material, in pcf (about 125 pcf)

Cantilevered Walls

Walls less than 6 feet in height need not include a seismic load.

Cantilevered walls taller than 6 feet should consider an incremental lateral seismic thrust, ΔP_E , expressed as:

$$\Delta P_E = 0.4 k_h H^2 \gamma \quad \text{where,}$$

ΔP_E is the incremental seismic thrust

k_h is the pseudostatic horizontal earthquake coefficient, is equal to $S_{DS}/2.5$

H is the height of the wall in feet from the footing

γ is equal to the unit weight of the backfill material, in pcf (about 125 pcf)

The resultant dynamic thrust acts at a distance of $0.3H$ above the base of the wall.

6.7.4 Foundation Uplift

A soil unit weight of 125 pcf may be assumed for calculating the weight of soil over the wall footing in design of cantilevered retaining walls.

6.7.5 Wall Drainage

The recommended equivalent fluid pressures provided in the preceding subsection assume that constantly functioning drainage systems are installed between walls and soil backfill to prevent the uncontrolled buildup of hydrostatic pressures and lateral stresses in excess of those stated. Figure 6-2 (following page) depicts a conceptual wall design in this regard.

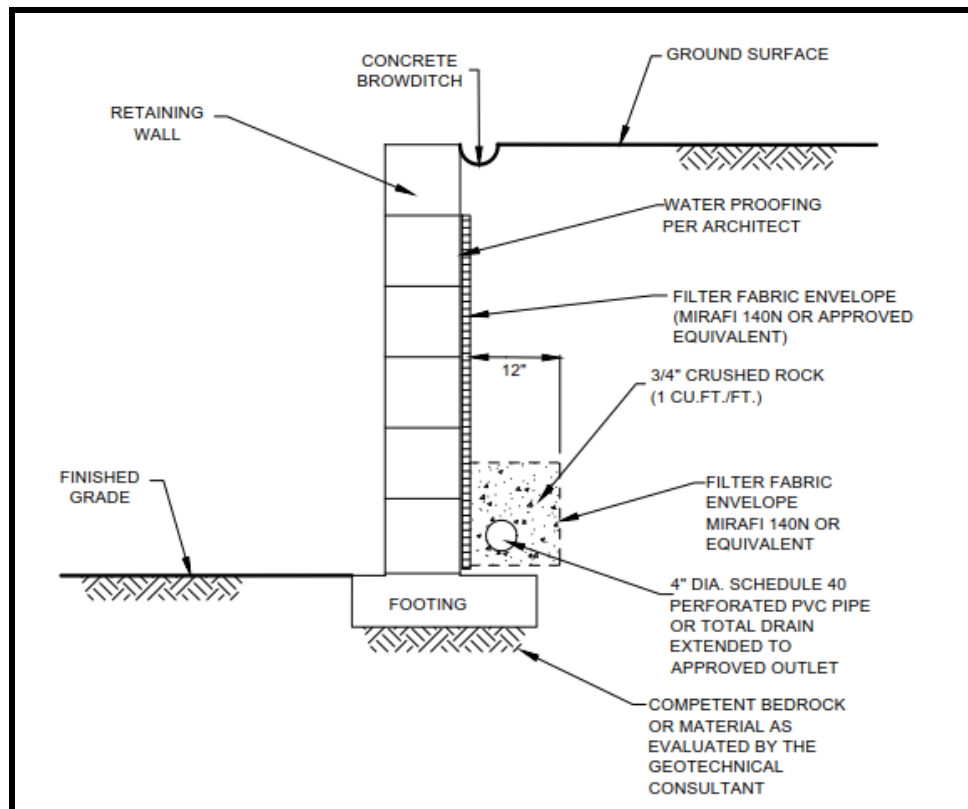


Figure 6-2. Conceptual Design for Wall Drainage

Design for wall drainage may include the use of pre-engineered wall drainage panels or a properly compacted granular free-draining backfill material ($EI < 30$). The use of drainage openings through the base of the wall (weep holes) is not recommended where the seepage could be a nuisance or otherwise adversely affect the ground adjacent to the base of the wall.

Numerous alternatives are available for collection of water behind retaining walls. The intent of Figure 6-2 is to depict the concepts described in this subsection.

6.8 MSE Walls

6.8.1 Select Granular Wall Backfill

The selection of the design parameters is up to the wall designer's judgment. Backfill in the reinforced zone should be confirmed by the wall designer, and in NOVA's opinion comprised of a select granular soil that meets the parameters listed below:

- at least 40% of the material less than 1/4-inch in size,
- a maximum particle size of 4 inches, and
- an expansion index (EI) of less than 20 (as determined by ASTM D 4829).

All fill/backfill placed as part of the MSE retaining wall system should be compacted to at least 90% relative compaction determined in accordance with ASTM D1557.

6.8.2 Backfill Strength Parameters

Table 6-7 provides minimum geotechnical parameters for the design of the MSE retaining walls. NOVA expects that a variety of select granular soils will meet these parameters.

Table 6-7. Soil Strength Parameters for MSE Retaining Walls

Parameter	Reinforced Zone	Retained	Foundation
Internal Friction Angle,	32	32	32
Cohesion, psf	250	250	250
Wet Unit Weight, pcf	125	125	125

6.8.3 Wall Backfill Material Requirements

The selection of the design parameters is up to the wall designer's judgment. Prior to placing the wall fill/backfill soil, the selected material should be sampled and tested to verify conformance with the minimum soil strength design parameters presented on Table 6-7. Imported fill that meet the design parameters set by the wall designer may be required. Limits of Backfill

All fill/backfill placed as part of the MSE retaining wall system should be compacted to at least 90% relative compaction determined in accordance with ASTM D1557. If the wall is not designed to withstand compaction equipment within a few feet of the face of the wall, the grid length embedment should be increased to account for the difference.

6.8.4 MSE Wall Foundations

Retaining walls should be developed on ground prepared in accordance with criteria provided in Section 6.4. Continuous shallow foundations or bearing pads may be designed in accordance with the criteria provided in Section 6.5.

If footings bear on fill, the footings should be over-excavated to a minimum of 2 feet below the bottom of footing and 3 feet laterally beyond the footing footprint. The bottoms of the excavation should be scarified to a minimum depth of 12 inches, and moisture conditioned per specifications in Section 6.4.

6.8.5 MSE Wall Drainage

The above recommendations assume a wall drainage panel or a properly compacted granular free-draining backfill material. The wall designer should indicate if the wall will require use of filter fabric.

6.8.6 Limitations

Improvements within the reinforced and retained zones may experience movement as the geosynthetic material elongates, developing full tensile resistance. The amount of movement is a function of the height of the wall and the type of geosynthetic grid. The estimated movement should be calculated by the MSE wall contractor and wall designer. The structural engineer should determine if the planned improvements can withstand the expected movement.

6.8.7 Design Review

The plans for the MSE retaining walls should be submitted to NOVA to verify the design parameters included herein are incorporated and reflected on the project plans. The intent of NOVA's review of the planned documents will be to verify the plans are consistent with provided geotechnical recommendations and to determine if additional recommendations are necessary. Responsibility for wall design will remain with the wall designer.

6.9 Temporary Slopes

Temporary slopes may be required for excavations during grading. All temporary excavations should comply with local safety ordinances. The safety of all excavations is solely the responsibility of the Contractor and should be evaluated during construction as the excavation progresses.

The faces of unbraced temporary slopes should be inspected daily by the Contractor's Competent Person before personnel are allowed to enter the excavation. Any zones of potential instability, sloughing or rattling should be brought to the attention of the Geotechnical Engineer-of-Record (GEOR) and corrective action implemented before personnel began working in the excavation.

Excavated soil should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. The GEOR should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during wet weather, berms are recommended along the tops of slope to prevent stormwater run on from affecting the exposed slopes.

7.0 STORMWATER INFILTRATION

7.1 Overview

Proposed permanent stormwater infiltration Best Management Practices ('stormwater BMPs') are basins located at the northeastern and northwestern corners of the project.

NOVA has evaluated the site as abstracted below after guidance contained in the City of Santee BMP Design Manual (hereinafter, 'the BMP Manual').

Section 3.5 provides a description of the fieldwork undertaken to complete the testing.

Figure 3-1 (and Plate 1) depict the locations of the testing. This section provides the results of that testing and related recommendations for management of stormwater in conformance with the BMP Manual.

As is well-established by the BMP Manual, the feasibility of stormwater infiltration is principally dependent on geotechnical and hydrogeologic conditions at the project site. In consideration of the infiltration rates observed during testing, NOVA concludes that each BMP basin allow for partial infiltration, provided care is taken in the placement of future BMPs, such that factors listed in Section 7.3.1 will not adversely impact the future structures.

7.2 Infiltration Rate

The percolation rate of a soil profile is not the same as its infiltration rate ('I'). Therefore, the field percolation rate was converted to an estimated infiltration rate utilizing the Porchet Method in accordance with guidance contained in the BMP Manual. Table 7-1 provides a summary of the infiltration rates determined by the percolation testing.

Table 7-1. Infiltration Rate Determined by Percolation Testing

BMP Location	Boring	Approximate Ground Elevation (feet, msl)	Depth of Test (feet)	Approximate Test Elevation (feet, msl)	Infiltration Rate (inches/hour)	Design Infiltration Rate (in/hour, F=2*)
Northeastern Basin	P-1	+360	5	+355	2.23	1.11
	P-2	+360	3	+357	0.34	0.17
Northwestern Basin	P-3	+366	5	+361	0.01	0.01
	P-4	+366	3	+363	0.25	0.13

Note: 'F' indicates 'Factor of Safety'

As may be seen by review of Table 7-1, a factor of safety (F) is applied to the infiltration rate (I) determined by the percolation testing. This factor of safety, at least $F = 2$ in local practice, considers the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time.

For the northeastern BMP basin, the calculated infiltration rates of P-1 (1.11 inches per hour) and P-2 (0.17 inches per hour) were averaged, resulting in the estimated infiltration rate of 0.64 inches per hour. With consideration of the variability of the infiltration rates, it is NOVA's opinion that the estimated reliable infiltration rate below the proposed facility is less than 0.5 inches per hour (partial infiltration).

For the northwestern BMP basin, the calculated infiltration rates of P-3 (0.01 inches per hour) and P-4 (0.13 inches per hour) were averaged, resulting in the estimated infiltration rate of 0.07 inches per hour (partial infiltration).

7.3 Review of Geotechnical Feasibility Criteria

7.3.1 Overview

Section C.2 of Appendix C of the BMP Manual provides seven factors that should be considered by the project geotechnical professional while assessing the feasibility of infiltration related to geotechnical conditions. These factors are listed below.

- C.2.1 Soil and Geologic Conditions
- C.2.2 Settlement and Volume Change
- C.2.3 Slope Stability
- C.2.4 Utility Considerations
- C.2.5 Groundwater Mounding
- C.2.6 Retaining Walls and Foundations
- C.2.7 Other Factors

The above geotechnical feasibility criteria are reviewed in the following subsections.

7.3.2 Soil and Geologic Conditions

The soil borings and percolation test borings completed for this assessment disclose the sequence of soil units in the near subsurface described below.

1. Unit 1, Fill/Colluvium. The site is covered by a veneer of fill and colluvium that ranges from 2 to 15.5 feet in thickness. The fill is comprised of brown silty sands of loose to medium dense consistency. Locally, the fill includes cobble-size to boulder-size pieces of granite. Because no records exist regarding the placement of this fill, the fill is 'undocumented,' and at risk for wide variations in quality.
2. Unit 2, Tonalite. The upper several feet of the naturally occurring subsurface are characteristically weathered granitic rock, decomposed to the consistency of a medium to coarse, light brown to gray silty sand. This residual soil is derived from the parent tonalite (Kt). Though dominated by sandy soils, this unit may include zones of localized extremely hard rock or "floaters", as well as gravelly zones.

Cretaceous-aged granitic tonalite (Kt) forms the basement rock for the entire site. The unit is characteristically very dense rock with the strength of a low-grade concrete as encountered below Unit 1.

7.3.3 Settlement and Volume Change

Settlement and volume change due to stormwater infiltration is not a concern with: (i) low expansive soils, (ii) no potential for liquefaction, and (iii) no potential for hydro collapse.

7.3.4 Slope Stability

BMPs should not be sited within 50 feet of an existing slope.

7.3.5 Utilities

Stormwater infiltration BMPs should not be sited within 10 feet of underground utilities.

7.3.6 Groundwater Mounding

Stormwater infiltration can result in damaging ground water mounding during wet periods. Based on the depth to groundwater and partial infiltration rates, groundwater mounding is not a high risk.

7.3.7 Retaining Walls and Foundations

Stormwater infiltration BMPs should not be sited within 10 feet from retaining walls and foundations.

7.3.8 Other

NOVA does not know of other factors that could affect implementation of stormwater infiltration BMPs.

7.4 Design Recommendation for Infiltration

In consideration of the foregoing, it is the judgment of NOVA that the site is suitable for partial infiltration of the stormwater BMP basins at the northeastern and northwestern corners of the site within the constraints cited above.

8.0 PAVEMENTS

8.1 Design Basis

The structural design of pavement sections depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. 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 Civil Engineer prior to final design.

8.2 Drainage and Moisture Control

Similar to the requirements for control of moisture beneath floor slabs and flatwork, control of surface drainage is important to the design and construction of pavements for this site.

Moisture must be controlled around and beneath pavements. Moreover, 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 and weakened over a long period of time.

The following should be considered to limit the amount of excess moisture which can reach the subgrade soils:

- maintain surface gradients at a minimum 2% grade away from the pavements;
- seal all landscaped areas in or adjacent to pavements to minimize or prevent moisture migration to subgrade soils;
- planters should not be located next to pavements (otherwise, subdrains should be used to drain the planter to appropriate outlets);
- place compacted backfill against the exterior side of curb and gutter; 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).

8.3 Preventative 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.

A plan for preventative maintenance should be comprised of both localized maintenance (e.g., crack sealing and patching) and global maintenance (e.g., surface sealing).

8.4 Subgrade Preparation

8.4.1 Grading

Preparation of subgrade for paved areas should include removal of existing pavements, and removal of the upper 2 feet of existing subgrade. The subgrade soils may be stockpiled for use as engineered fill replacement. Excavation bottoms should be examined to identify any localized soft, yielding or otherwise unsuitable materials by a field technician from NOVA.

The bottom of the excavation should be loosened to a depth of 12 inches, moisture conditioned, and densified in accordance with Section 6.4. prior to placing engineered fill. The upper 12 inches of subgrade should be moisture conditioned to approximately 3% above the optimum moisture content, and compacted to at least 95% relative compaction after ASTM D 1557.

8.4.2 Proof-Rolling

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.

8.4.3 Timely Base Course Construction

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.

8.5 Flexible Pavements

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. Table 8-1 provides preliminary flexible pavement sections using an assumed R-value of 25. The recommended pavement sections are for planning purposes only. Additional R-value testing should be performed on actual subgrade soils at the design subgrade levels to confirm the final pavement section for the project.

Table 8-1. Preliminary Recommendations for Flexible Pavements, R = 25

Area	Subgrade R-Value	Traffic Index	Asphalt Thickness (in)	Base Course Thickness (in)
Auto Parking	25	5	3.0	7.0
Roadways/Fire Lane	25	6	3.0	10.0

The above sections assume properly prepared subgrade consisting of at least 12 inches of subgrade densified to a minimum of 95% relative compaction at about 2% above the optimum moisture content.

The aggregate base course should also be placed at a minimum of 95% relative compaction. Construction materials (asphalt and aggregate base) should conform to the current Standard Specifications for Public Works Construction ('Green Book').

8.6 Rigid Pavements

8.6.1 General

Concrete pavement sections should be developed in the same manner as undertaken for all other slabs and pavements: removal of the upper 2 feet of the Unit 1 fill and replacement of that material in an engineered manner as described in Section 9.2.

Concrete pavement sections consisting of 7 inches of Portland cement concrete over 6 inches of base. The concrete pavement section should be supported by compacted subgrade that meets select fill requirements. It is further recommended that in areas where heavy traffic or point loads are anticipated, including areas in front of trash enclosures, the slab be reinforced with at least No. 4 bars placed at 18 inches on center each way.

Where rigid pavements are used, the concrete should be obtained from an approved mix design with the minimum properties of Table 8-2.

Table 8-2. 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

8.6.2 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 ¼-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% at the joints and tapered to regular thickness in 5 feet.

9.0 CONSTRUCTION REVIEW, OBSERVATION, AND TESTING

9.1 Overview

As is discussed in Section 1, the recommendations contained in this report are based upon a limited number of borings and an assumption of general continuity of subsurface conditions between borings.

The recommendations provided in both NOVA's proposal for this work and this report assume that NOVA will be retained to provide consultation and review during the design phase, to interpret this report during construction, and to provide construction monitoring in the form of testing and observation.

9.2 Design Phase Review

The recommendations of this report are based upon NOVA's current understanding and assumptions regarding planning for project development.

As is provided for in its proposal for this work, NOVA should review the final design. Such review is important for both (i) conformance with the recommendations provided herein, and (ii) consistency with NOVA's understanding of the planned development.

9.3 Construction Observation and Testing

9.3.1 General

Special inspections should be provided per Section 1705 of the California Building Code. The soils special inspector should be a representative of NOVA as the Geotechnical Engineer-of-Record (GEOR).

NOVA should be retained to provide construction-related services abstracted below.

- Surveillance during site preparation, grading, and foundation excavation.
- Inspection of the ground improvement described in Section 6.
- Soil special inspection during grading.

A program of quality control should be developed prior to the beginning of earthwork. It is the responsibility of the Owner, the Contractor and/or the Construction Manager to determine any additional inspection items required by the Architect/Engineer or the governing jurisdiction.

9.3.2 Continuous Soils Special Inspection

The earthwork operations listed below should be the object of continuous soils special inspection.

- Site grading, including scarification and engineered fill placement.
- Ground preparation as described in Section 6.
- Pavement subgrade preparation and base course compaction.

9.3.3 Periodic Soils Special Inspection

The earthwork operations listed below should be the object of periodic soils special inspection, subject to approval by the Building Official.

- Site preparation and removal of existing development features.
- Placement and compaction of utility trench backfill.
- Observation of foundation excavations.

9.3.4 Testing During Inspections

A preconstruction conference among representatives of the Owner, Contractor and/or Construction Manager, and Geotechnical Engineer is recommended to discuss the planned construction procedures and quality control requirements.

The locations and frequencies of compaction test should be determined by the geotechnical engineer at the time of construction. Test locations and frequencies may be subject to modification by the geotechnical engineer based upon soil and moisture conditions encountered, the size and type of compaction equipment used by the Contractor, the general trend of compaction test results, and other factors.

Of particular concern to NOVA during earthwork operations will be good practices in moisture conditioning, loose soil placement, and soil compaction. In particular, NOVA will be vigilant with regard to the use of compaction equipment appropriate to the full lift thickness of the type of soil being compacted. Reliance on construction traffic (for example, loaders or dump trucks) to achieve compaction will not be approved.

10.0 REFERENCES

10.1 Site Specific

Geotechnical Feasibility Review, Hiser Property, APNs 386-300-13, -29, and -31, 7737 Mission Gorge Road, City of Santee, San Diego County, California, GeoSoils, Inc., W.O. 7678-A-SC, 9 Sept 2019.

Conceptual Site Plan, Rick Engineering (RE), undated, provided to NOVA in February 2020.

10.2 Design

American Concrete Inst., 2002, Building Code Requirements for Structural Concrete, ACI 318-02.

American Concrete Inst., 2015, Guide to Concrete Floor and Slab Construction, ACI 302.1R-15.

ASCE, Minimum Design Load for Buildings and Other Structures, ASCE 7-16.

APWA, 2015 Standard Specifications for Public Works Construction ('Greenbook')

California Code of Regulations, Title 24, 2019 California Building Standards Code.

California Department of Transportation (Caltrans), 2003, *Corrosion Guidelines*, Version 1.0, available at <http://www.dot.ca.gov/hq/esc/ttsb/corrosion/pdf/2012-11-19-Corrosion-Guidelines.pdf>.

City of San Diego, 2011, *Guidelines for Geotechnical Reports*, Dev. Services Department.

City of Santee, *BMP Design Manual*, San Marcos, February 2016 Edition.

City of Santee, *General Plan, Safety Element*, Santee, 2000-2020, adopted August 27, 2003.

10.3 Site Setting

Jennings, C. W. and Bryant, W. A., 2010, *Fault Activity Map of California*, California Geological Survey, Geologic Data Map No. 6.

Kennedy, M.P. and Tan, S.S., 1975, *Geology of the San Diego Metropolitan Area, California*, California Division of Mines and Geology, Bulletin 200.

Norris, R. M. and Webb, R. W., 1990, Geology of California, Second Edition: John Wiley & Sons, Inc.

United States Federal Emergency Management Agency (FEMA), 2012, Flood Insurance Rate Map (FIRM), Flood Map 06073C1880G, Revised May 16, 2012.

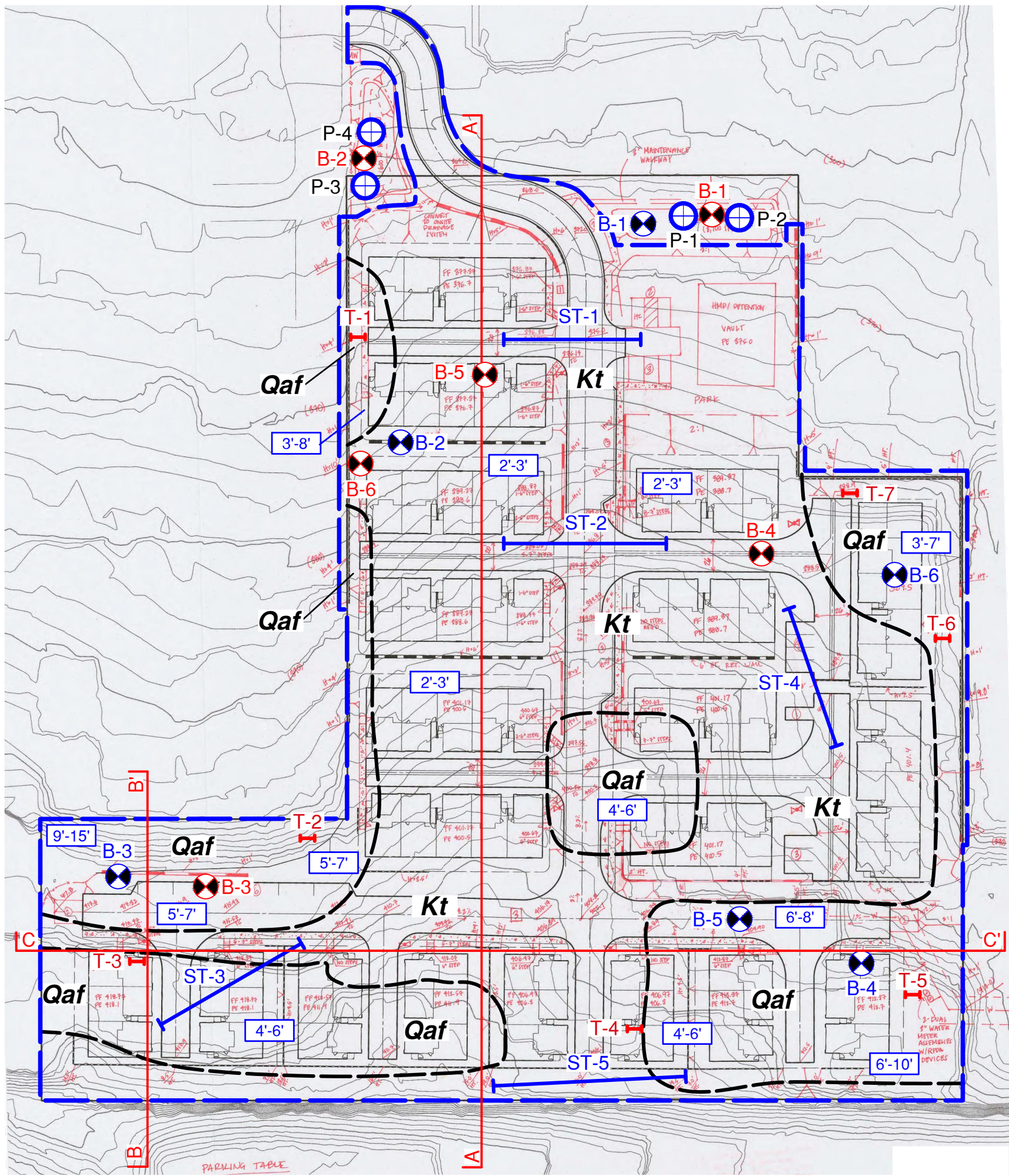
California Department of Water Resources, Water Data Library: found at <http://www.water.ca.gov/waterdatalibrary/>

California Division of Mines and Geology (CDMG), 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California*, Special Publication 117A.

Tan, S.S., 1995, *Landslide Hazards in San Diego County, California*, California Division of Mines and Geology.

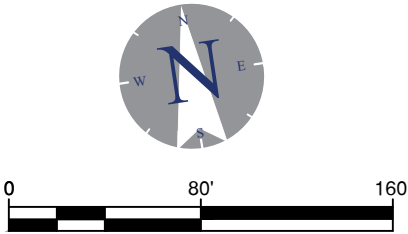
United States Department of Agriculture, Natural Resources Conservation Service, *Custom Soil Resource Report for San Diego County Area, California, August 15, 2017*

PLATES



KEY TO SYMBOLS

- Qaf** FILL
- Kt** TONALITE
- B-6** GEOTECHNICAL BORING (NOVA)
- B-6** GEOTECHNICAL BORING (GSI, 2019)
- P-4** PERCOLATION TEST BORING (NOVA)
- T-7** TRENCH (NOVA)
- ST-7** SEISMIC REFRACTION SURVEY (GSI, 2019)
- C-C'** GEOLOGIC CROSS-SECTION
- ~** LIMITS OF ANTICIPATED REMEDIAL GRADING
- 6'-10'** ANTICIPATED DEPTH OF REMEDIAL REMOVALS
- ~** APPROXIMATE LIMITS OF FILL/COLLUVIUM >3' IN THICKNESS



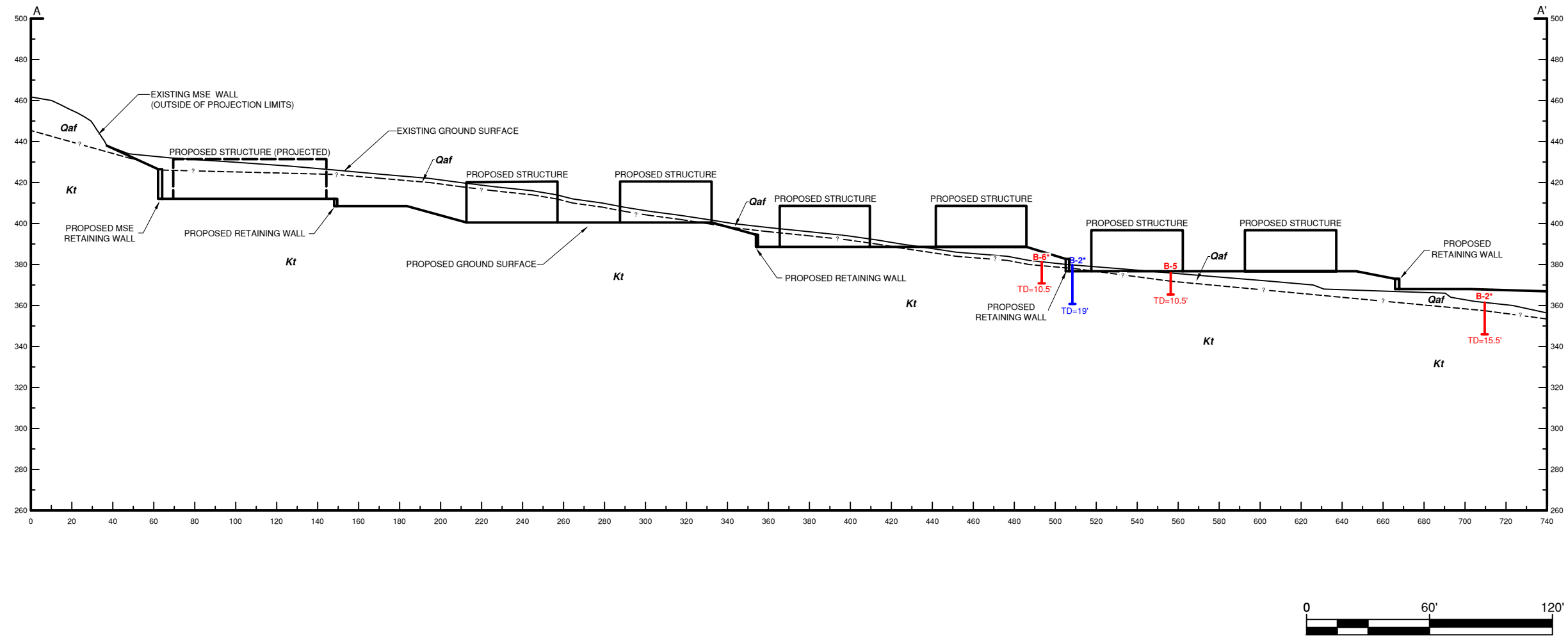
**HISER SANTEE
CORNERSTONE COMMUNITIES**
7377 MISSION GORGE RD
SANTEE, CA

PROJECT NO.:	2020035
DATE:	APRIL 2020
DRAWN BY:	DTJ
REVIEWED BY:	MS
SCALE:	1"=80'
DRAWING TITLE:	

SUBSURFACE
INVESTIGATION MAP

HISER SANTEE
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SANTEE, CA

PROJECT NO.:	2020035
DATE:	APRIL 2020
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KEY TO SYMBOLS

- Qaf**

FILL
- Kt**

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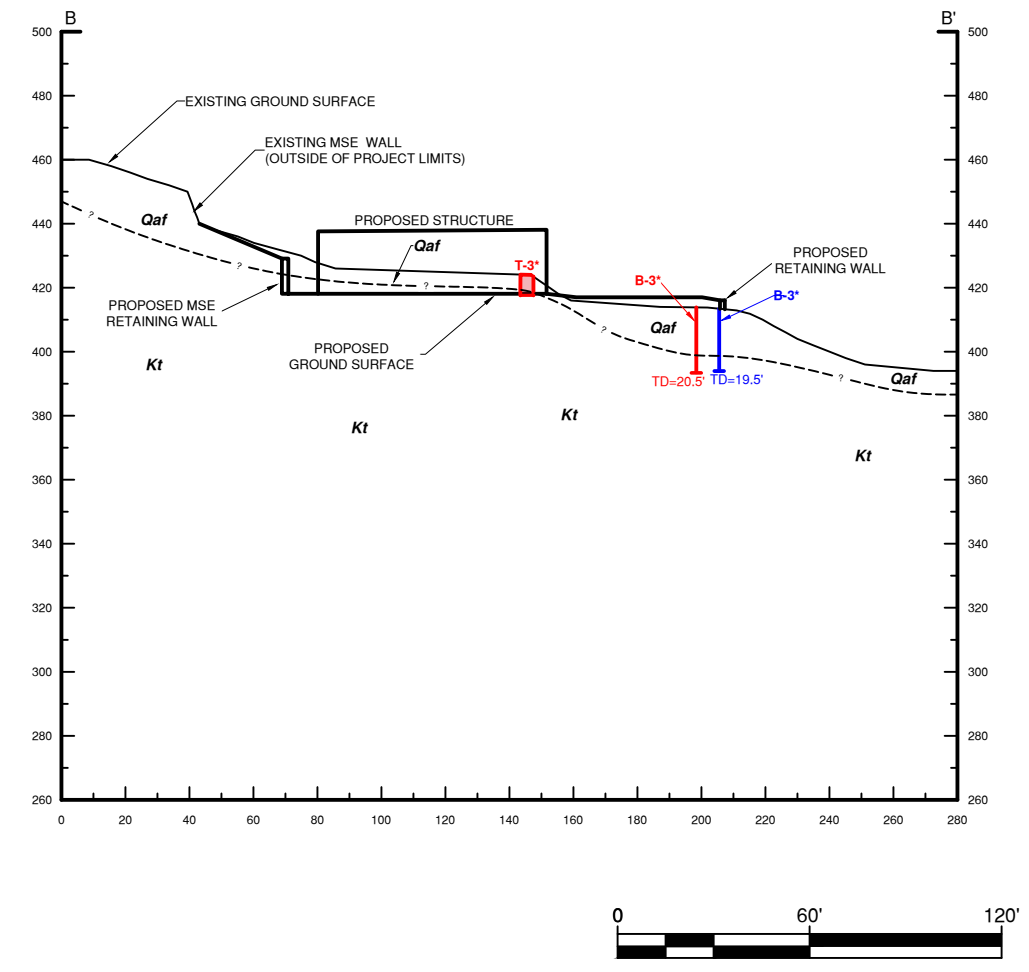
GEOTECHNICAL BORING (NOVA)
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GEOTECHNICAL BORING (GSI, 2019)
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GEOLOGIC CONTACT, QUERIED WHERE INFERRED

HISER SANTEE
CORNERSTONE COMMUNITIES
7377 MISSION GORGE RD
SANTEE, CA

PROJECT NO.:	2020035
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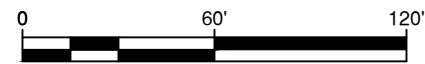
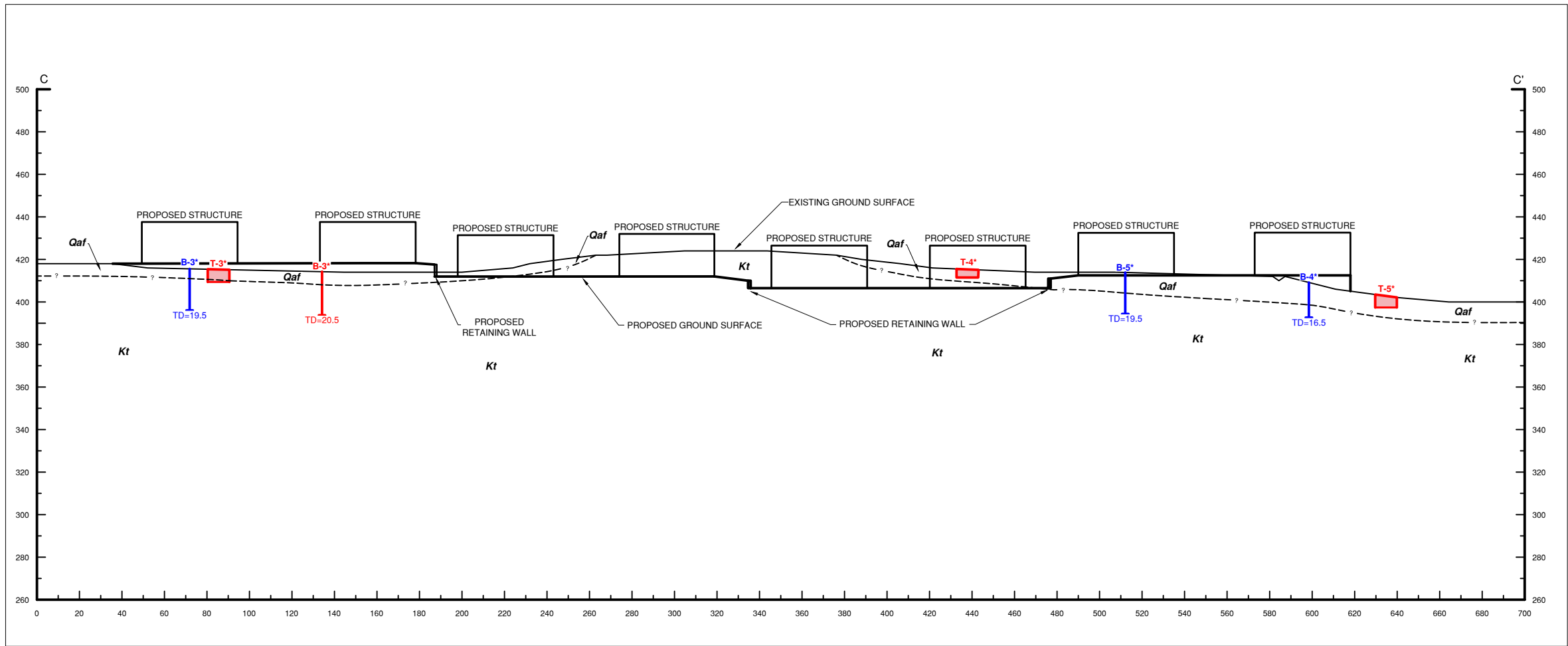


KEY TO SYMBOLS

Qaf	FILL	B-3	GEOTECHNICAL BORING (NOVA) *PROJECTED	B-3	GEOTECHNICAL BORING (GSI, 2019) *PROJECTED
Kt	TONALITE	T-3	TRENCH (NOVA) *PROJECTED	T-3	TRENCH (NOVA) *PROJECTED
			GEOLOGIC CONTACT, QUERIED WHERE INFERRED		

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7377 MISSION GORGE RD
SANTEE, CA

PROJECT NO.:	2020035
DATE:	APRIL 2020
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KEY TO SYMBOLS

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GEOTECHNICAL BORING (NOVA)
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GEOTECHNICAL BORING (GSI, 2019)
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TRENCH (NOVA)
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GEOLOGIC CONTACT, QUERIED WHERE INFERRED

APPENDIX A

USE OF 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.



8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

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APPENDIX B

LOGS OF BORINGS

NOVA 2020

BORING LOG B-1

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 364 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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 CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0			SM	54	FILL (Qaf): SILTY SAND; BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE COARSE SAND GRAINS	MD EI SA	VERY LOW
5			SM	50/6"	TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, VERY DENSE, FINE TO COARSE GRAINED	CR MD SA	
10				50/1"	VERY DENSE		
15				50/2"			
20					BORING TERMINATED AT 15.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
25							
30							

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.1

LOGGED BY: AR DATE: APRIL 2020

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GEOTECHNICAL
MATERIALS
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BORING LOG B-2

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 360 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE COARSE SAND GRAINS		
5				SM	50/3"	TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, DENSE TO VERY DENSE, FINE TO COARSE GRAINED GRAY	SA	
10					50/3"		SA	
15					50/2"			
20						BORING TERMINATED AT 15.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.2

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San Clemente, CA 92673
P: 949.388.7710

BORING LOG B-3

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 413 FT MSL

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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; BROWN WITH REDDISH BROWN MOTTLING, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED COARSE SAND GRAINS		
31								
5					18		MD DS SA CN	
					6	LOOSE		
10					4	VERY LOOSE		
					47	DENSE		
15				SM	50/5"	TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, DENSE TO VERY DENSE, FINE TO COARSE GRAINED		
20					50/2"			
						BORING TERMINATED AT 20.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.3

LOGGED BY: AR DATE: APRIL 2020

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San Clemente, CA 92673
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BORING LOG B-4

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 388 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; GRAY BROWN TO REDDISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED COARSE SAND GRAINS		
5				SM	50/5"	TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, VERY DENSE, FINE TO COARSE GRAINED		
5					50/3			
10					50/0"			
10.5						BORING TERMINATED AT 10.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
15								
20								
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.4

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REVIEWED BY: MS PROJECT NO.: 2020035

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BORING LOG B-5

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 376 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE GRAVEL		
					69	VERY DENSE		
5				SM	50/2"	TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, VERY DENSE, FINE TO COARSE GRAINED		
10					50/2"			
						BORING TERMINATED AT 10.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
15								
20								
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.5

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BORING LOG B-6

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 376 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE GRAVEL		
5				SM	50/4**	TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, VERY DENSE, FINE TO COARSE GRAINED		
10				SM	50/4**			
15								
20								
25								
30								
						BORING TERMINATED AT 10.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.6

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PERCOLATION BORING LOG P-1

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 364 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL		
5				SM		TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, VERY DENSE, FINE TO COARSE GRAINED		
5						BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION WELL.		
10								
15								
20								
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.7

LOGGED BY: AR DATE: APRIL 2020

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San Clemente, CA 92673
P: 949.388.7710

PERCOLATION BORING LOG P-2

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 364 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL REDDISH BROWN		
5						BORING TERMINATED AT 3 FT AND CONVERTED TO A PERCOLATION WELL.		
10								
15								
20								
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.8

LOGGED BY: AR DATE: APRIL 2020

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PERCOLATION BORING LOG P-3

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 360 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0		X		SM		FILL (Qaf): SILTY SAND; BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE CLAY		
				SM		TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, DENSE TO VERY DENSE, FINE TO COARSE GRAINED		
5						BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION WELL.		
10								
15								
20								
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.9

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PERCOLATION BORING LOG P-4

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 360 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LABORATORY	REMARKS
0				SM		FILL (Qaf): SILTY SAND; BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE COARSE SAND GRAINS		
				SM		TONALITE ROCK (Kt): SILTY SAND; LIGHT BROWN, DRY TO DAMP, DENSE TO VERY DENSE, FINE TO COARSE GRAINED BORING TERMINATED AT 15.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
5								
10								
15								
20								
25								
30								

KEY TO SYMBOLS

	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOWCOUNT
	BULK SAMPLE	*	NO SAMPLE RECOVERY
	SPT SAMPLE (ASTM D1586)	—	GEOLOGIC CONTACT
	CAL. MOD. SAMPLE (ASTM D3550)	- - -	SOIL TYPE CHANGE

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.10

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TRENCH LOG T-1

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CAT 303.5C CR EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 376 FT MSL

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				SM		FILL (Qaf): SILTY SAND; BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, ASPHALT CONCRETE FRAGMENTS ON SURFACE, SCATTERED ROOTS 1 FT THICK LAYER CONTAINING 1-3" ASPHALT CONCRETE FRAGMENTS LOOSE TO MEDIUM DENSE ORANGE BROWN		
5				SM		TONALITE ROCK (Kt): SILTY SAND; LIGHT ORANGE BROWN, DAMP TO MOIST, VERY DENSE, MEDIUM TO COARSE GRAINED, WEATHERED, TONALITE CLASTS 1-5"		
10						TRENCH TERMINATED AT 10 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
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

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.11

LOGGED BY: GAN DATE: APRIL 2020

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TRENCH LOG T-2

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CAT 303.5C CR EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 412 FT MSL

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		X		SM		FILL (Qaf): SILTY SAND WITH SOME CLAY; REDDISH BROWN, MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED, GRAVEL ON SURFACE LIGHT GRAYISH BROWN, MOIST, LOOSE TO MEDIUM DENSE, NO CLAY REDDISH BROWN LIGHT GRAYISH BROWN WITH REDDISH BROWN		
5		X		SM		TONALITE ROCK (Kt): SILTY SAND; LIGHT ORANGE-GRAYISH BROWN, MOIST, MEDIUM TO VERY DENSE, MEDIUM TO COARSE GRAINED, WEATHERED, ABUNDANT TONALITE CLASTS 4-6"		
15						TRENCH TERMINATED AT 7 FT. 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

HISER PROPERTY

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APPENDIX B.12

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TRENCH LOG T-3

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CAT 303.5C CR EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 424 FT MSL

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				SM		FILL (Qaf): SILTY SAND WITH GRAVEL; ORANGE TO LIGHT GRAYISH ORANGE, MOIST, MEDIUM DENSE, MEDIUM TO COARSE GRAINED, ABUNDANT GRAVELS 3", ABUNDANT COBBLES 6"		
5				SM		TONALITE ROCK (Kt): SILTY SAND; LIGHT ORANGE, DAMP, MEDIUM DENSE, MEDIUM TO COARSE GRAINED, GRAVEL, WEATHERED, FREQUENT GRANITIC CLASTS 4-6"		
15						TRENCH TERMINATED AT 6 FT. 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

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.13

LOGGED BY: GAN DATE: APRIL 2020

REVIEWED BY: MS PROJECT NO.: 2020035



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TRENCH LOG T-4

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CAT 303.5C CR EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 423 FT MSL

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						FILL (Qaf): SILTY SAND WITH GRAVEL; ORANGE BROWN, DAMP TO MOIST, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, SOME GRAVELS 3", SOME COBBLES 8"	CR MD SA	
5						TRENCH TERMINATED AT 4 FT ON TONALITE ROCK (Kt). 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

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APPENDIX B.14

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TRENCH LOG T-5

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CAT 303.5C CR EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 410 FT MSL

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				SM		2" OF ASPHALT CONCRETE		
						FILL (Qaf): SILTY SAND WITH GRAVEL; ORANGE BROWN, DAMP-MOIST, LOOSE TO MEDIUM DENSE, FINE TO COARSE GRAINED, SOME GRAVELS 3", SOME COBBLES 8"		
						SOME COBBLES 4"		
5						VERY DENSE		
						TRENCH TERMINATED AT 6 FT ON TONALITE ROCK (Kt). 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

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7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.15

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TRENCH LOG T-6

DATE EXCAVATED: FEBRUARY 27, 2020 **EQUIPMENT:** CAT 303.5C CR EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH **GPS COORD.:** N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED **ELEVATION:** ± 384 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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				SM		FILL (Qat): SILTY SAND; REDDISH BROWN, DAMP TO MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED		
5				SM		DENSE, SOME GRAVELS ≤ 3", SOME COBBLES ≤ 8"		
				SM		TONALITE ROCK (Kt): SILTY SAND; ORANGE TO REDDISH BROWN, MOIST, VERY DENSE, MEDIUM TO COARSE GRAINED, WEATHERED		
10						TRENCH TERMINATED AT 6.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
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

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.16

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TRENCH LOG T-7

DATE EXCAVATED: FEBRUARY 27, 2020 EQUIPMENT: CAT 303.5C CR EXCAVATOR

EXCAVATION DESCRIPTION: TRENCH GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED ELEVATION: ± 381 FT MSL

LAB TEST ABBREVIATIONS

CR CORROSION
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				SM		FILL (Qat): SILTY SAND WITH GRAVEL; REDDISH BROWN, DAMP-MOIST, MEDIUM DENSE, FINE TO COARSE GRAINED, SOME BRICKS		
				SM-SC		SILTY-CLAYEY SAND; REDDISH BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED, SOME GRAVELS 3"		
				SM		TONALITE ROCK (Kt): SILTY SAND; ORANGE TO REDDISH BROWN, MOIST, VERY DENSE, MEDIUM TO COARSE GRAINED, WEATHERED, TONALITE CLASTS <3"		
5						TRENCH TERMINATED AT 4 FT. 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

HISER PROPERTY

7737 MISSION GORGE RD
SANTEE, CALIFORNIA

APPENDIX B.17

LOGGED BY: GAN DATE: APRIL 2020

REVIEWED BY: MS PROJECT NO.: 2020035



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BORING LOG

Project: HISEN PROPERTY
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W.O. 7678-A

Boring B-1 Sheet 1 of 6

Date Excavated 8-18-19

Sample Method:
HOLLOW STEM AUGER
CAI SAMPLER
STANDARD PENETRATION-SPT

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
	Bulk	Undisturbed	Blows/ft.				
				SM			20' <u>UNDOCUMENTED FILL</u> : SILTY SAND, W/ GRAVEL, GRAY BROWN, dry, LOOSE.
				SM			
				SM			
5			21				21' <u>Colluvium</u> : SILTY SAND, REDDISH BROWN, DRY MEDIUM DENSE; POROUS.
10			50/4"				2 1/2' <u>BEDROCK</u> : GRANITIC ROCK BREAKING TO SILTY SAND UPON EXCAVATION, BROWN, SLIGHTLY MOIST, DENSE
							25' AS PER 2 1/2' "DISINTEGRATED GRANITE" or "DG"
15			50/4"				10' becomes GRAYISH BROWN, VERY DENSE.
20			50/2"				
25							
30							

TOTAL DEPTH = 19.2'
NO GROUNDWATER
BACK FILLED

Plate B-1

GeoSoils, Inc.

BORING LOG

W.O. 7678 A

Project: HISER PROPERTY,
SANTEE

Boring B-2 Sheet 2 of 6

Date Excavated 8-18-19

Sample Method: Hollow Stem
AUGER,
C-CAL SAMPLER
SPT-STANDARD PENETRATION-SPT

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
	Bulk	Undisturbed	Blows/ft.				
				SM			00' <u>UNDOCUMENTED FILL</u> : SILTY SAND, W/ GRAVEL, AND GRAVEL TO COBBLE SIZE ASPHALT CHUNKS.
	C		16	SM	104.5	2.6	
5	SPT		21	SM			01' <u>COLLUVIUM</u> : SILTY SAND, REDDISH BROWN, DRY, LOOSE.
							02 1/2' AS PER 1', LOOSE TO MEDIUM DENSE.
10	SPT		50 1/4"				03 1/2' <u>BEDROCK</u> : GRANITIC ROCK BREAKING TO SILTY SAND "DG" UPON EXCAVATION, BROWN, DRY, MEDIUM DENSE; "DG" TEXTURE.
							08' AS PER 3 1/2', BECOMES DENSE
15	SPT		50 1/4"				010' AS PER 8'
							015' AS PER 10'
20	SPT		50 3/4"				019' AS PER 15'
							TOTAL DEPTH = 19.2'
							NO GROUNDWATER
							BACK FILLED
30							

Plate B-2

GeoSoils, Inc.

BORING LOG

W.O. 7678A

Project: HISER PROPERTY
SANTEE

Boring B-3 Sheet 3 of 6

Date Excavated 8-18-19

Sample Method: HOLLOW STEM
AUGER W/ CAL SAMPLER
"C" ON STANDARD
PENETRATION TEST "SPT"

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
	Bulk	Undisturbed	Blows/ft.				
0				SM			00' <u>UNDOCUMENTED FILL</u> : SILTY SAND, W/ SCATTERED GRAVEL TO COBBLE SIZE ROCK AND ASPHALT, FEW BOULDER SIZE ROCK FRAGMENTS, GRAYISH BROWN, DRY, LOOSE TO MEDIUM DENSE
5			11				
10			12	SM			09' <u>CLAY</u> : SILTY SAND, REDDISH BROWN, MOIST, LOOSE TO MEDIUM DENSE; SLIGHTLY POROUS.
15			29/50 1/2	SM/SC	117.1	12.5	12' <u>BEDROCK</u> : GRANITIC ROCK BREAKING TO SILTY/CLAYEY SAND UPON EXCAVATION, MOTTLED REDDISH BROWN AND DARK GRAYISH BROWN, MOIST, DENSE.
20			50/44				15' <u>GRANITIC BEDROCK</u> : BREAKING TO SILTY SAND "DG" UPON EXCAVATION. SLIGHTLY MOIST, DENSE.
25							17' <u>DIFFICULT DRILLING</u> .
30							TOTAL DEPTH = 19 1/2' NO GROUNDWATER BACK FILLED

Plate B-3

GeoSoils, Inc.

BORING LOG

Project: HISER PROJECT 4
SANTEE

W.O. 7678A

Boring B-4 Sheet 4 of 6

Date Excavated 8-18-19

Sample Method: HOLLOW
STEM AUGER w/ CAL
SAMPLER [C] and
STANDARD PENETRATION
TEST - SPT

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
	Bulk	Undis- turbed	Blows/ft.				
				AC			0' ASPHALTIC CONCRETE "AC," 4" THICK
				SM			0.33' UNDOCUMENTED FILL: SILTY SAND, REDDISH BROWN, SLIGHTLY MOIST, LOOSE TO MEDIUM DENSE
5		SPT	16				
				SM			0.8' COLLUVIUM: SILTY SAND, REDDISH BROWN, SLIGHTLY MOIST, LOOSE
10		C	50/6"	SM	115	11.1	
							0.10' GRANITIC BEDROCK: GRANITIC ROCK BREAKING TO SILTY SAND UPON EXCAVATION - "DG," - BROWN TO YELLOWISH BROWN, SLIGHTLY MOIST, DENSE.
15		SPT	75				
20							TOTAL DEPTH = 16 1/2 FEET NO GROUNDWATER BACK FILLED.
25							
30							

Plate B-4

GeoSoils, Inc.

BORING LOG

W.O. 7678A

Project: HISEL PROPERTY
SANT EE

Boring B-5 Sheet 5 of 6

Date Excavated 8-18-19

Sample Method: HOLLOW STEM
AUGER, COI SAMPLER
"C" AND STANDARD
PENETRATION TEST - SPT

Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
	Bulk	Undisturbed	Blows/ft.				
				SM			00' ASPHALTIC CONCRETE "AC," 3 inches.
							00.25' UNDOCUMENTED FILL: SILTY SAND, BROWN, DRY TO SLIGHTLY MOIST, LOOSE.
5				SM			06' COLLUVIUM: SILTY SAND, REDDISH BROWN, MOIST, LOOSE.
10				SM			08' BEDROCK: GRANITIC ROCK BREAKING TO SILTY SAND "DG" UPON EXCAVATION, MOTTLED OLIVE BROWN TO BROWN, SLIGHTLY MOIST, DENSE.
15							013' BECOMES GRAYISH BROWN, and DENSE TO VERY DENSE; DIFFICULT DRILLING.
20							TOTAL DEPTH = 19 1/2'
25							NO GROUNDWATER
30							BACK FILLED

Plate B-5

GeoSoils, Inc.

BORING LOG

W.O. 7678A

Project: HISER PROPERTY
SANTAE

Boring B-6 Sheet 6 of 6

Date Excavated 8-18-19

Sample Method: Hollow Stem
Auger, CPT-Sampler [C]
STANDARD PENETRATION TEST
- SPT -

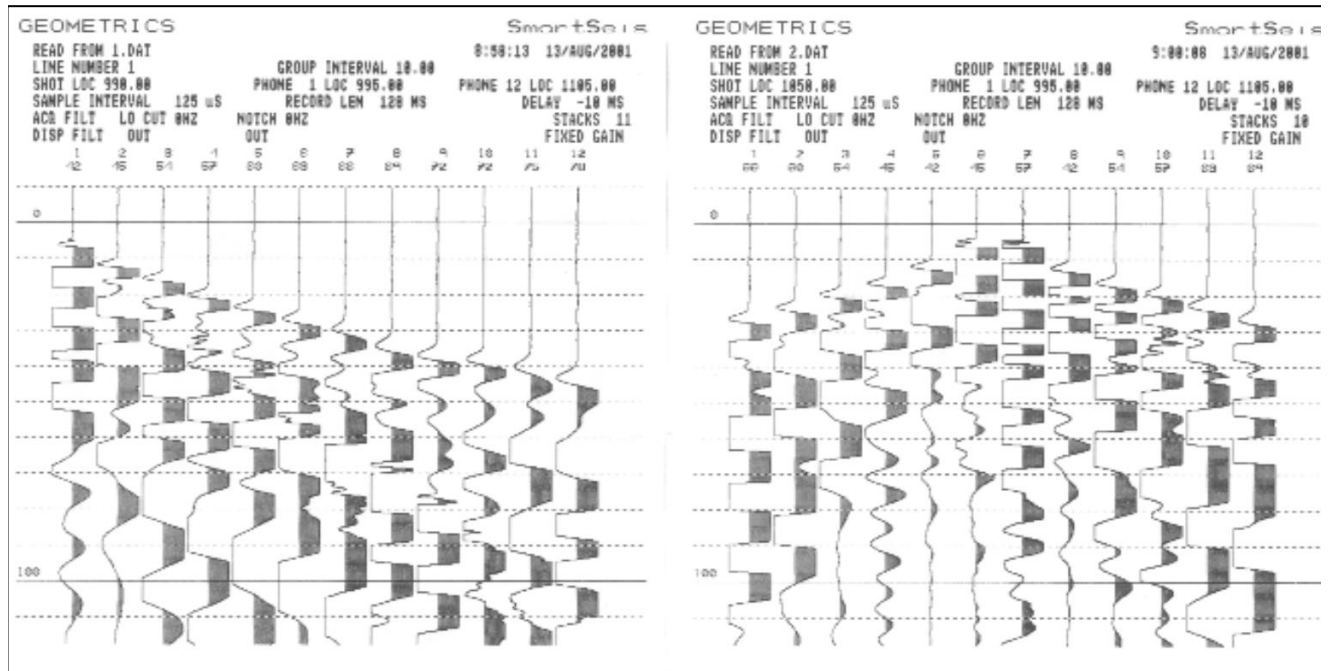
Depth (ft.)	Sample			USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
	Bulk	Undisturbed	Blows/ft.				
				SM			00' <u>UNDOCUMENTED FILL</u> : SILTY SAND, DARK BROWN, DRY, LOOSE, MANY GRAVEL SIZE ASPHALT FRAGMENTS.
				SM			02' <u>Colluvium</u> : SILTY SAND, BROWN SLIGHTLY MOIST, LOOSE.
5		C	24 50/100	SM SP	114.9	4.4	05' <u>BEDROCK</u> : GRANITIC ROCK BREAKING TO SILTY SAND AND SAND "DG" UPON EXCAVATION.
10			SPT 50/100				10' AS PER 5' BROWN, SLIGHTLY MOIST, DENSE.
15			SPT 50/100				15' AS PER 10'
20							TOTAL DEPTH = 15.33' NO GROUNDWATER BACKFILLED
25							
30							

Plate B-6

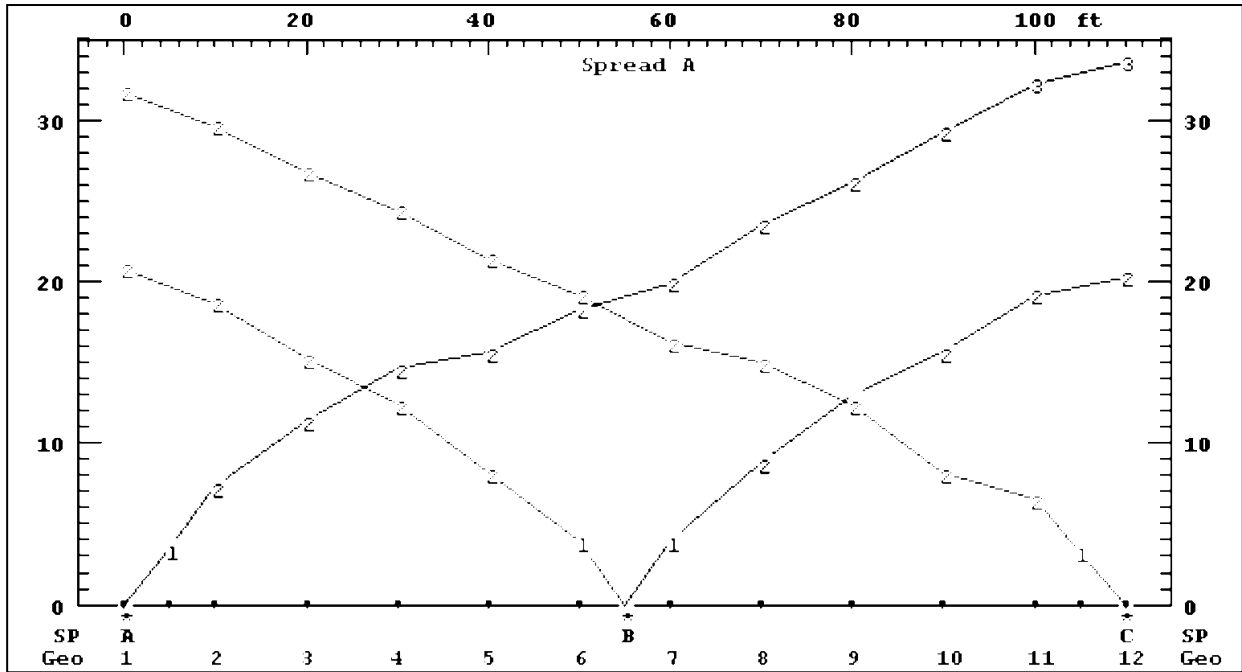
APPENDIX C

GSI SEISMIC REFRACTION SURVEY

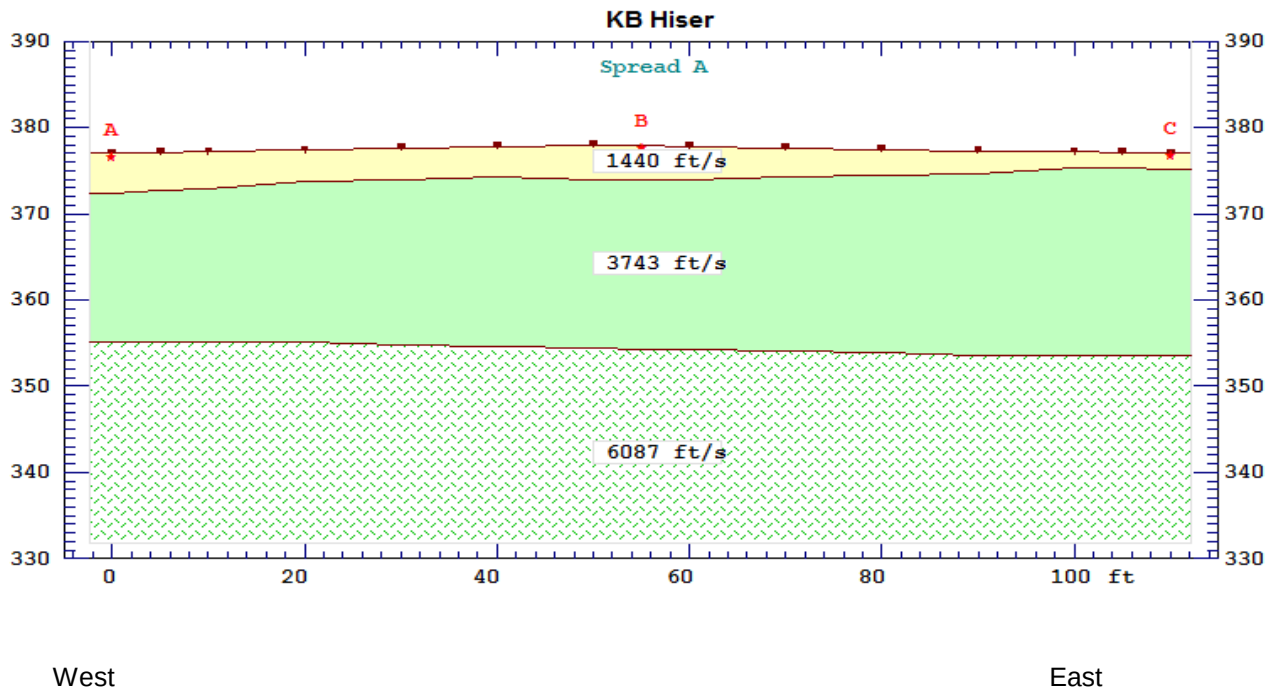
Example
Raw Seismic Data



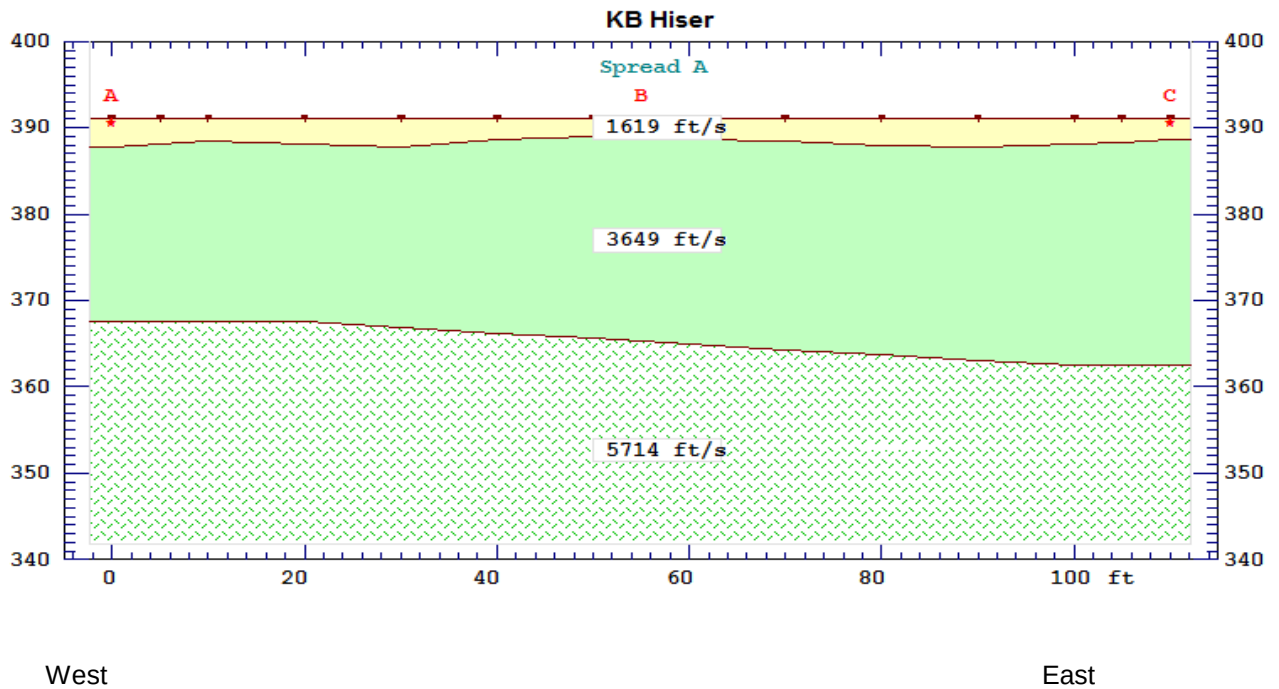
Example Seismic Line



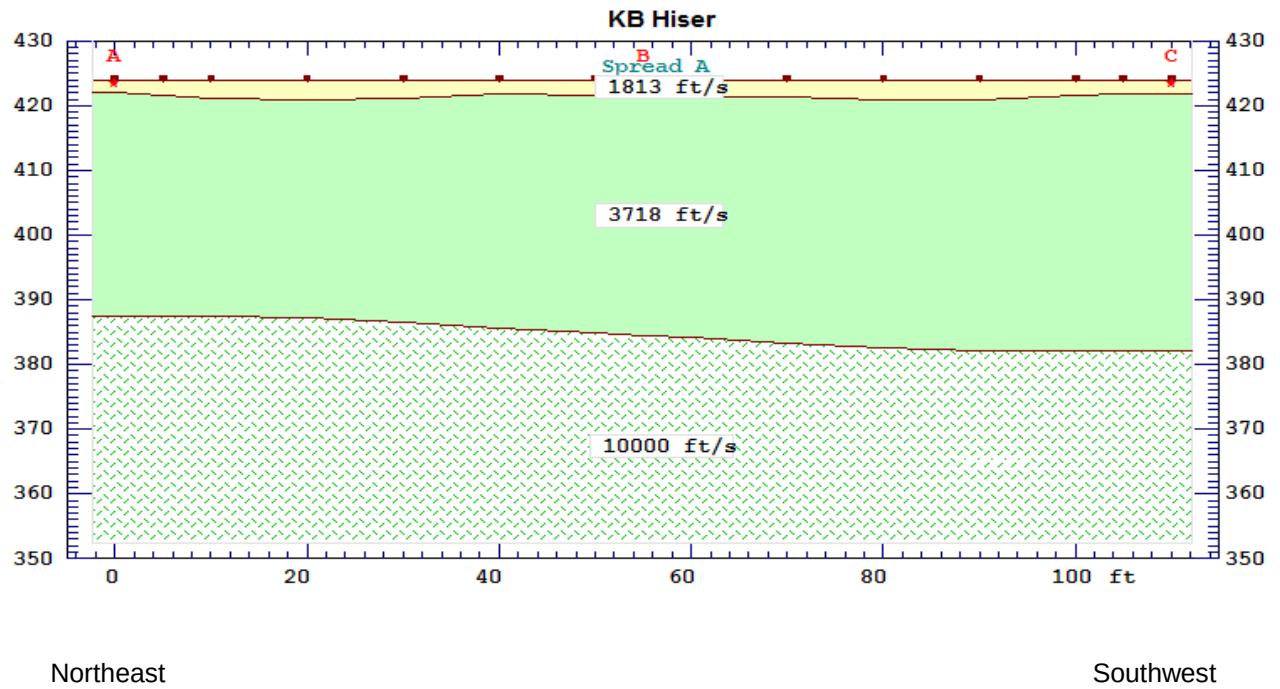
Seismic Line ST-1



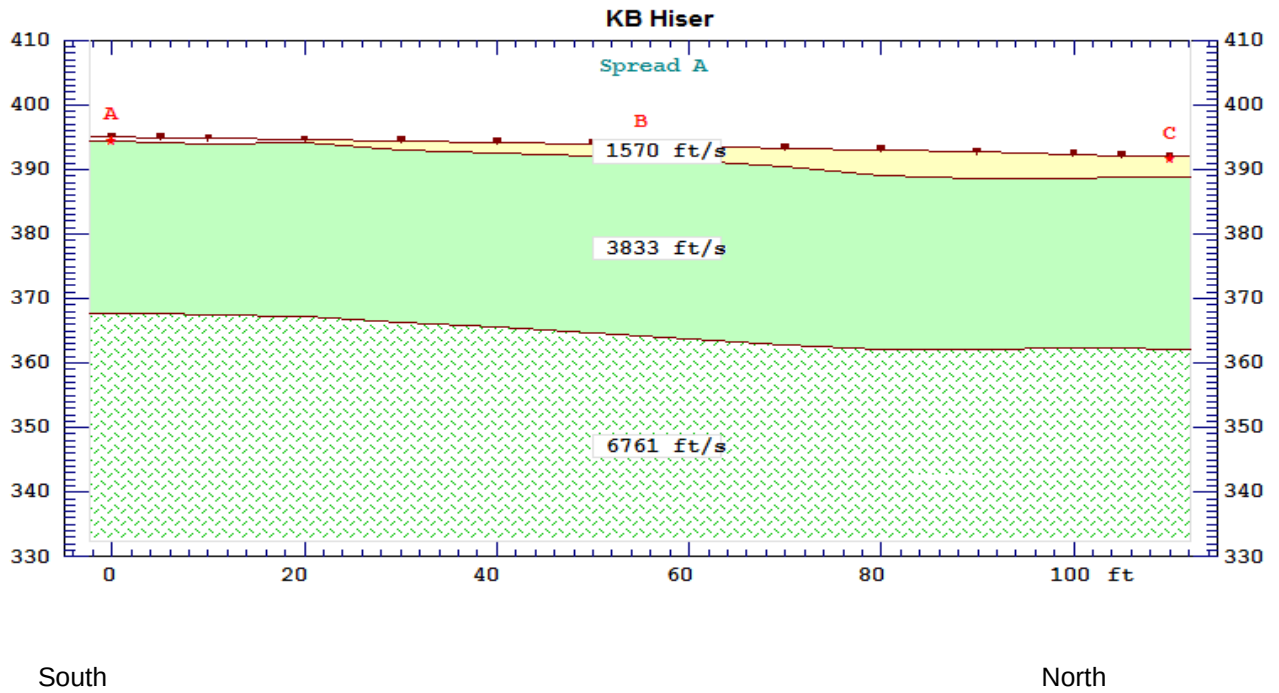
Seismic Line ST-2



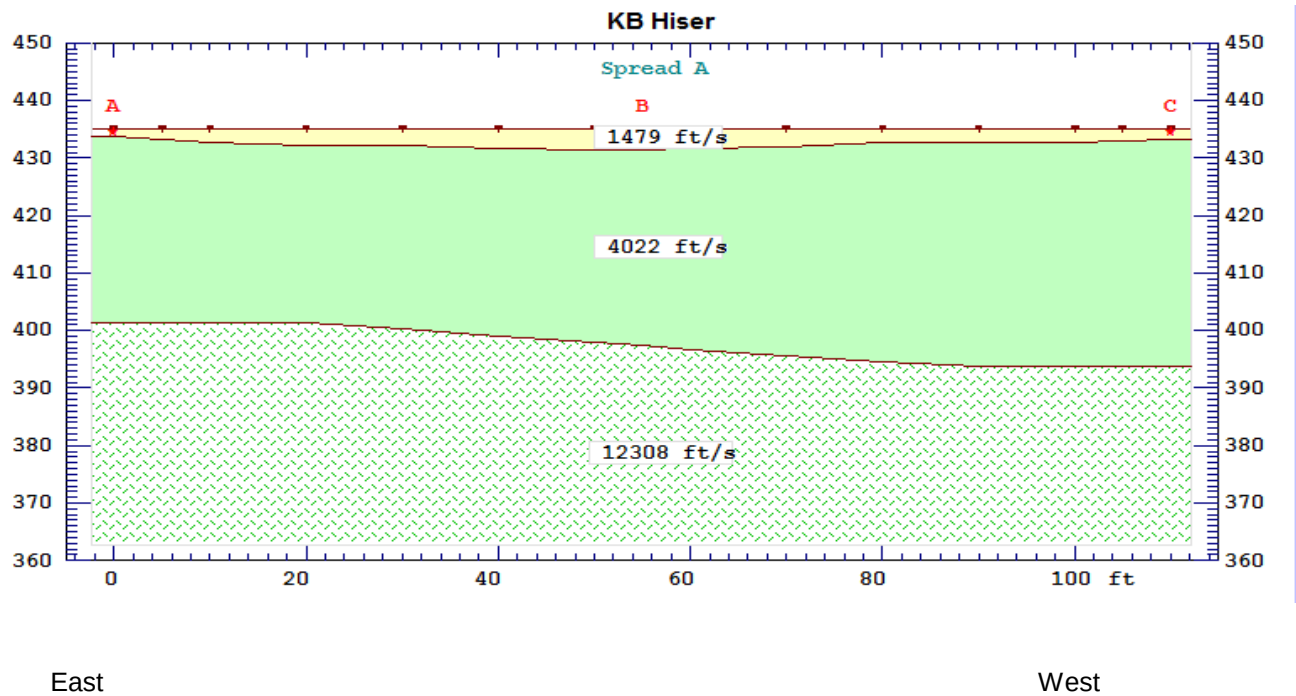
Seismic Line ST-3



Seismic Line ST-4



Seismic Line ST-5



APPENDIX D

RECORDS OF LABORATORY TESTING



NOVA 2020

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.
- **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.
- **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.
- **DIRECT SHEAR (ASTM D3080):** Direct shear tests were performed on remolded and relatively undisturbed samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions.
- **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 D.3 through Appendix D.8.
- **CONSOLIDATION PROPERTIES (ASTM D2435):** Tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are summarized on Appendix D.9.



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LAB TEST SUMMARY

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7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.1

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-1	0 - 5	Brown Silty Sand	134.0	8.0
B-1	5 - 10	Light Brown Silty Sand	135.0	8.1
B-3	5 - 7	Red Brown Silty Sand	139.0	5.9
T-4	0 - 4	Orange Brown Silty Sand	131.6	8.2

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft)	Expansion Index	Expansion Potential
B-1	0 - 5	0	Very Low

Direct Shear (ASTM D3080)

Sample Location	Depth (ft)	Soil Description	Friction Angle (degrees)	Apparent Cohesion (psf)
B-3	5 - 7	Reddish Brown Silty Sand	32	135

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 (%)
B-1	5 - 10	7.8	4100	30	0.003	32	0.003
T-4	0 - 4	6.6	3700	48	0.005	43	0.004



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LAB TEST RESULTS

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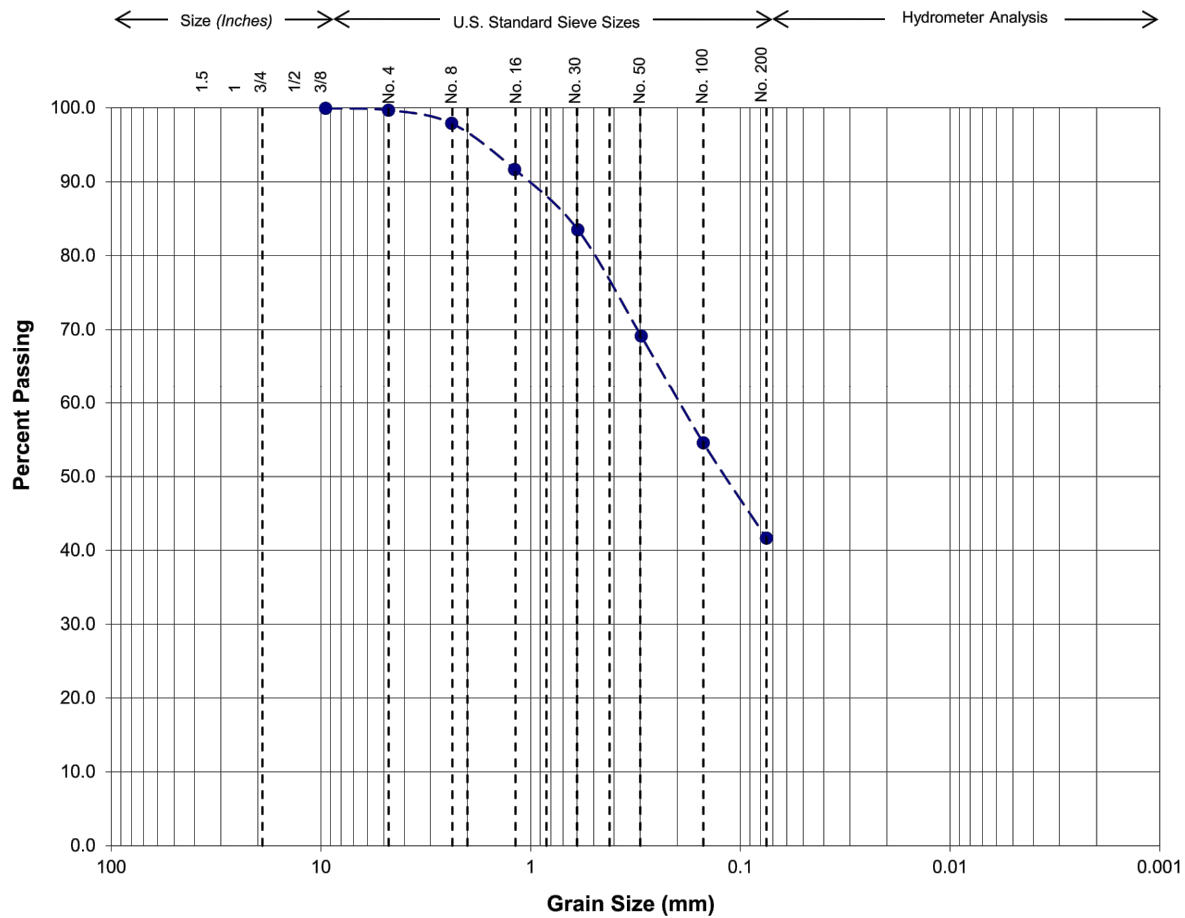
7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.2



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-1

Depth (ft): 0 - 5

USCS Soil Type: SM

Passing No. 200 (%): 42



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GRADATION ANALYSIS TEST RESULTS

HISER PROPERTY

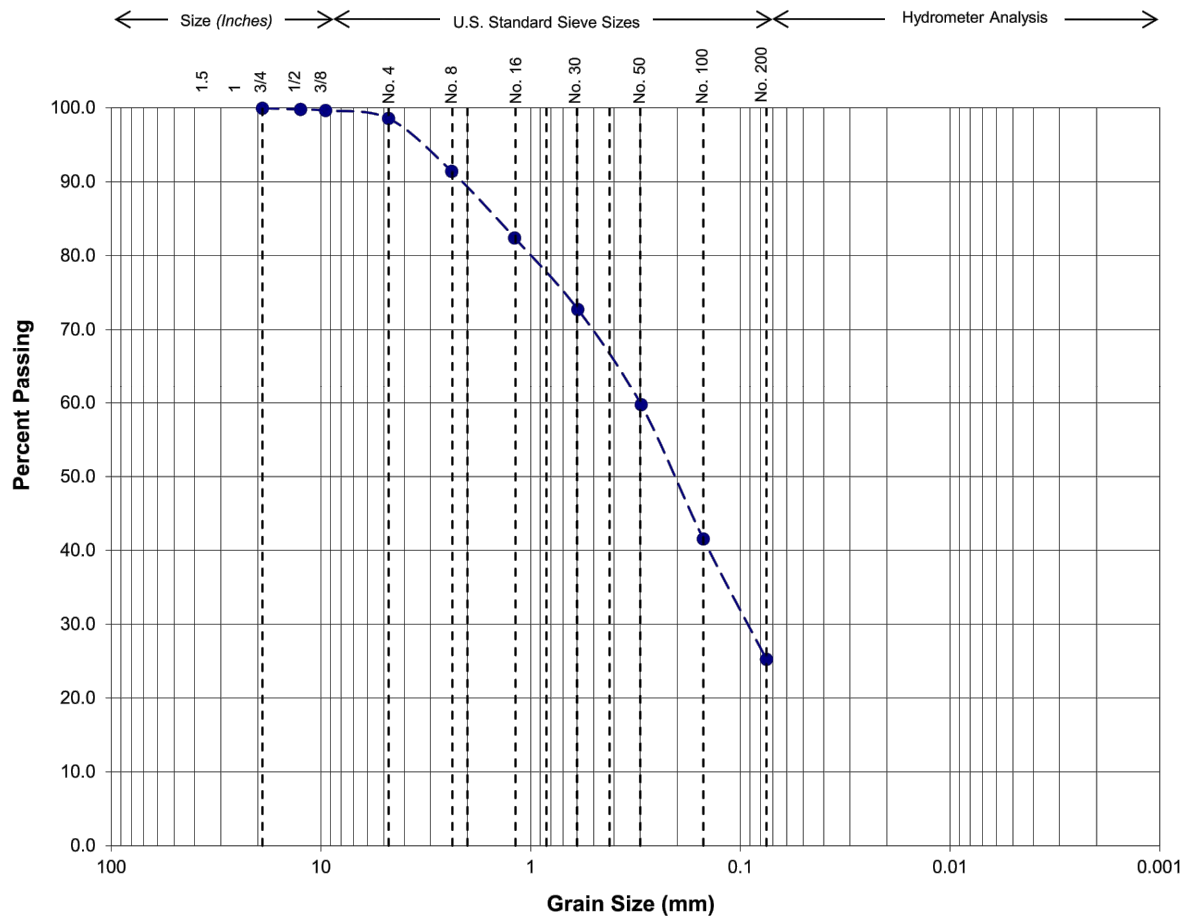
7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-1

Depth (ft): 5 - 10

USCS Soil Type: SM

Passing No. 200 (%): 25



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GRADATION ANALYSIS TEST RESULTS

HISER PROPERTY

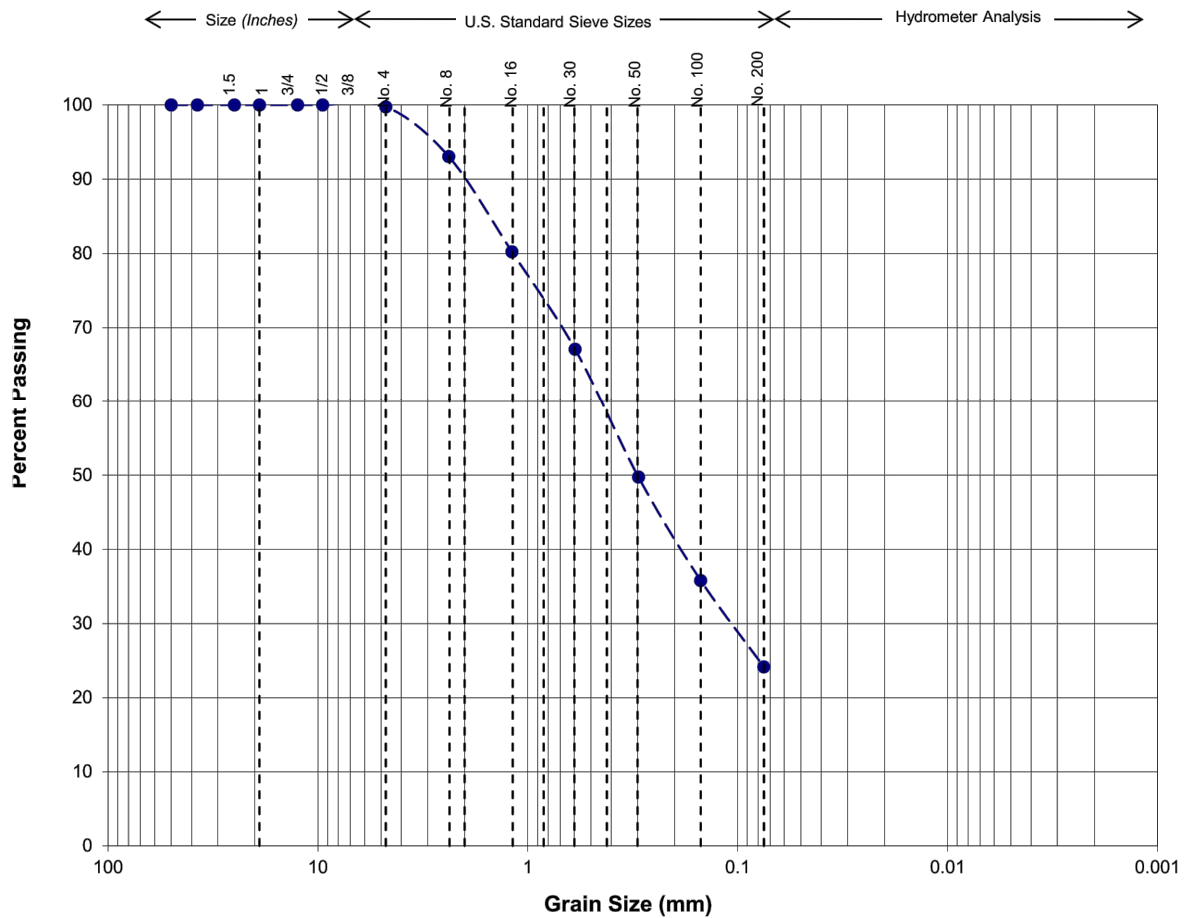
7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.4



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-2

Depth (ft): 5

USCS Soil Type: SM

Passing No. 200 (%): 24



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GRADATION ANALYSIS TEST RESULTS

HISER PROPERTY

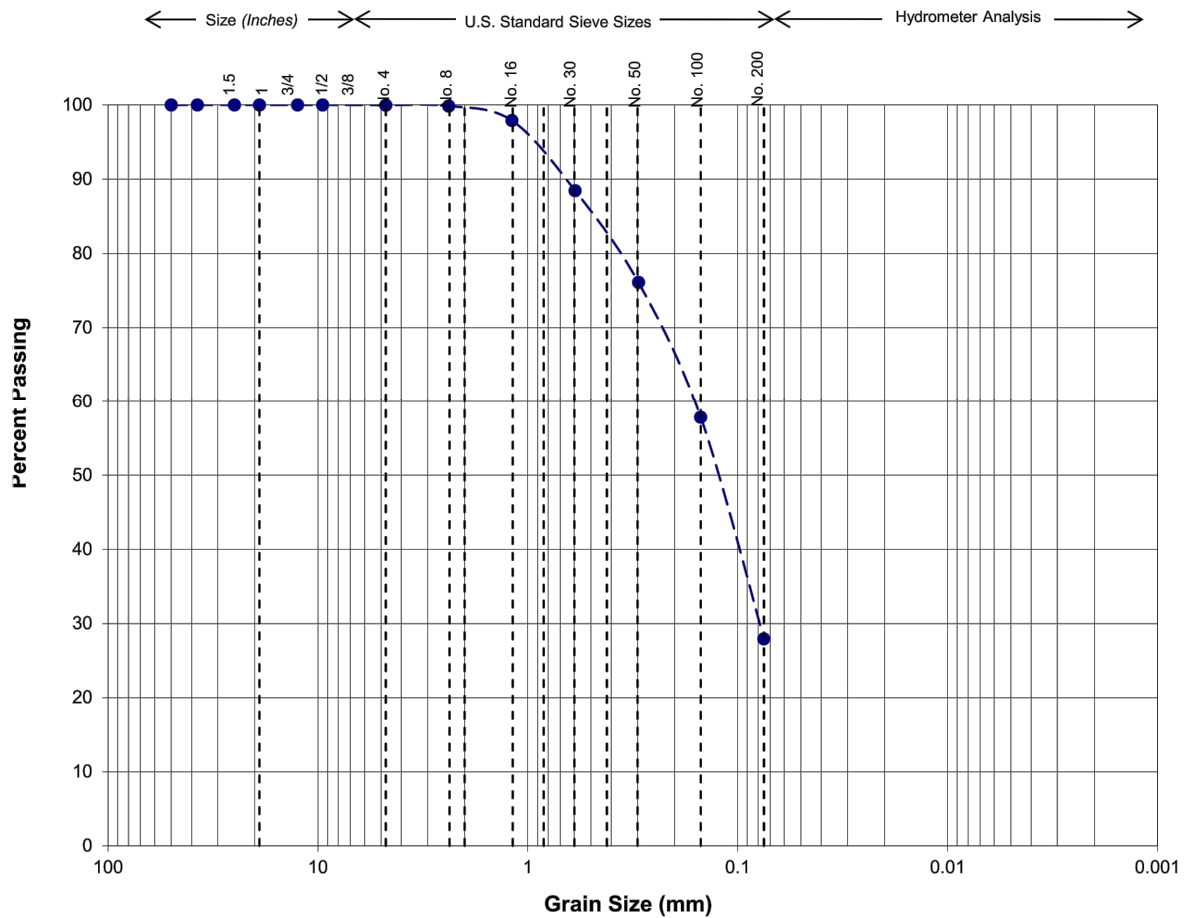
7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.5



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-2

Depth (ft): 10

USCS Soil Type: SM

Passing No. 200 (%): 28



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GRADATION ANALYSIS TEST RESULTS

HISER PROPERTY

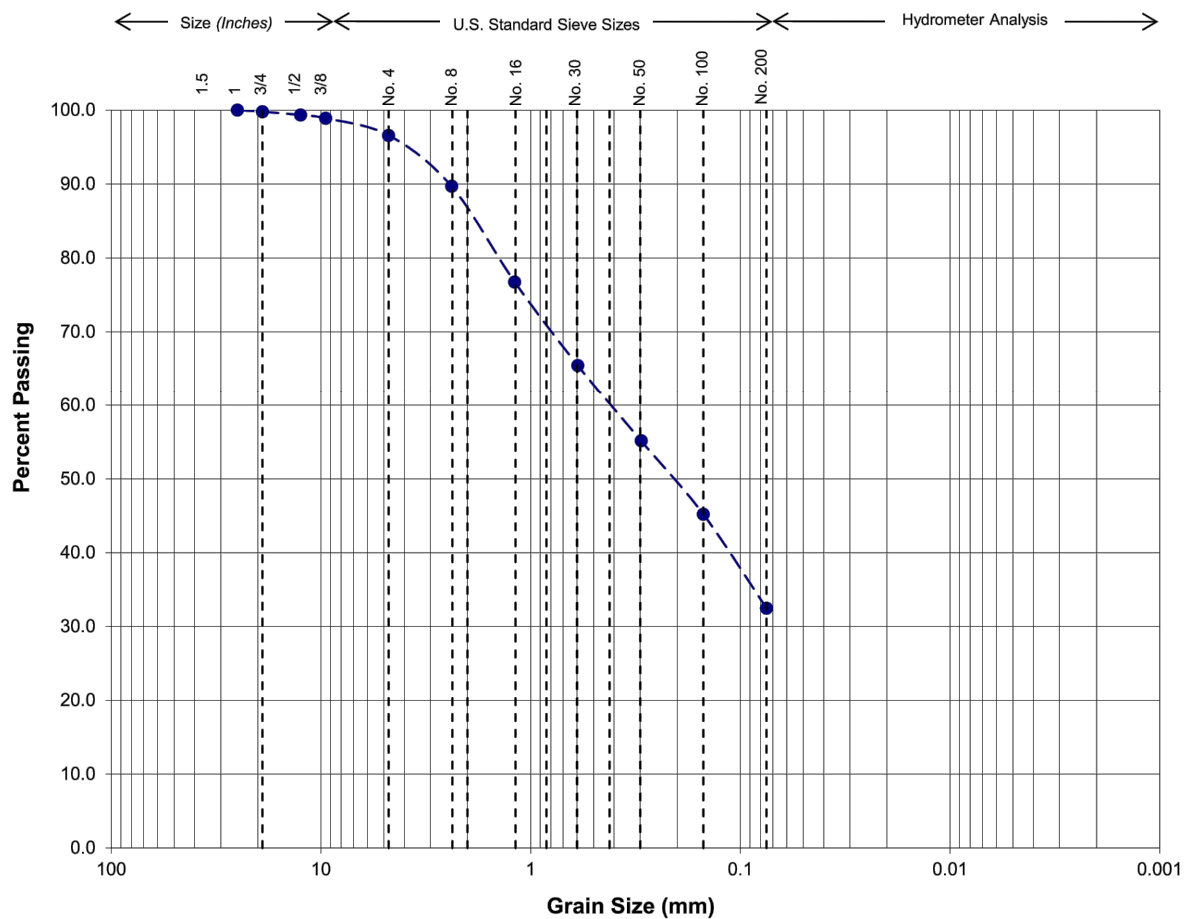
7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.6



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-3

Depth (ft): 5 - 7

USCS Soil Type: SM

Passing No. 200 (%): 32



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GRADATION ANALYSIS TEST RESULTS

HISER PROPERTY

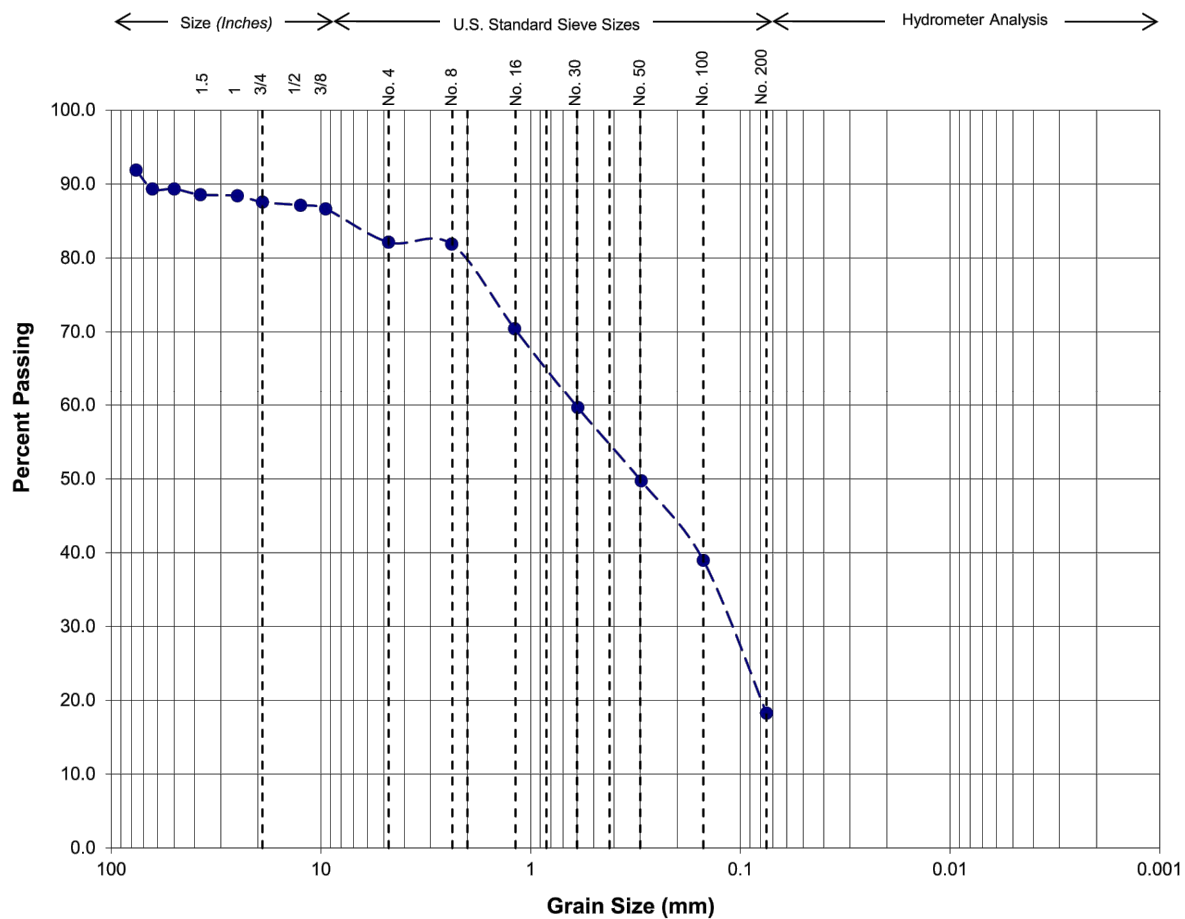
7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.7



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: T-4

Depth (ft): 0 - 4

USCS Soil Type: SM

Passing No. 200 (%): 18



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GRADATION ANALYSIS TEST RESULTS

HISER PROPERTY

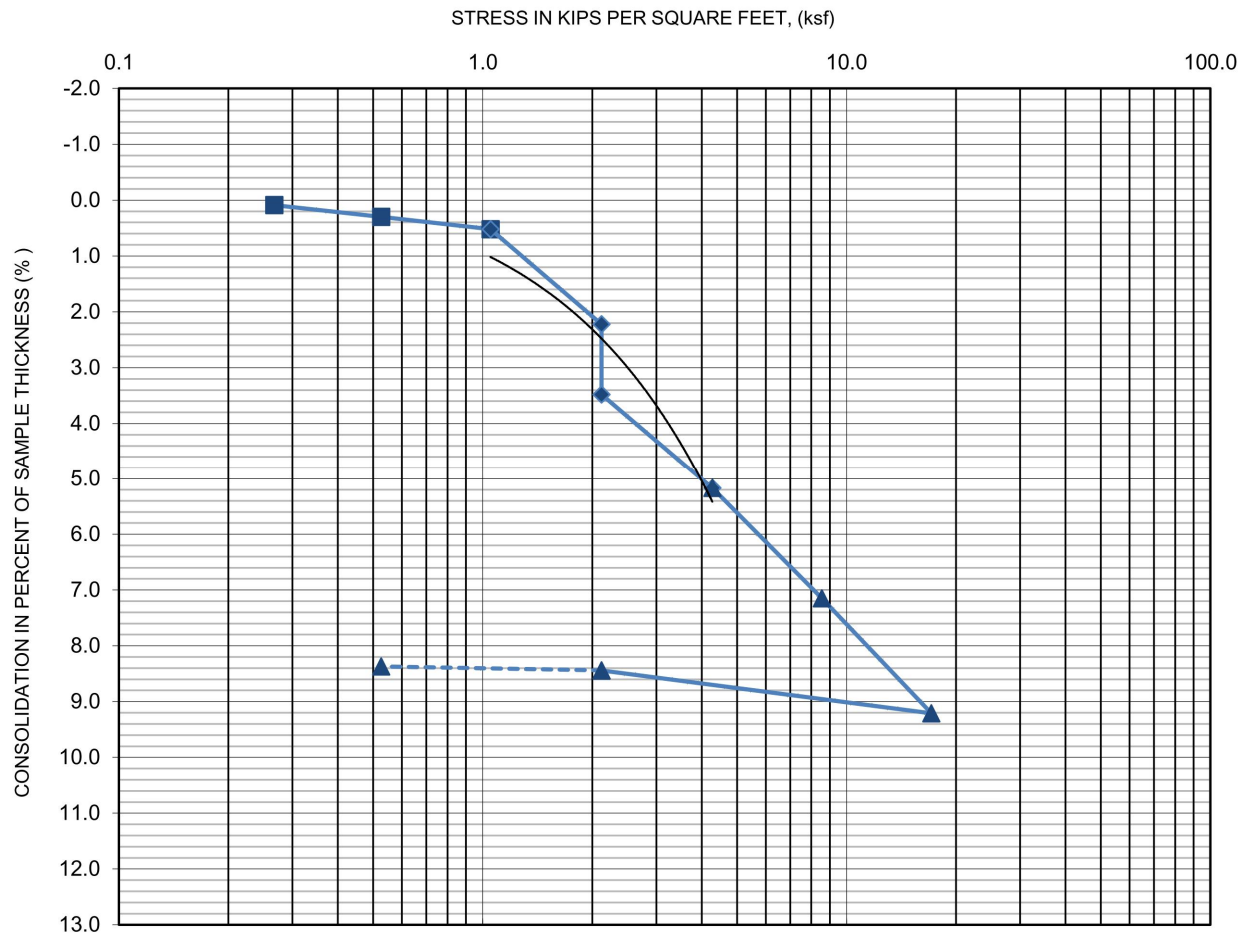
7737 MISSION GORGE ROAD
SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.8



—■— Loading Prior to H2O

—▲— Loading After H2O

- - -▲- - Rebound

Sample Location: B-3

Depth (ft): 7.5

USCS Soil Type: SM



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CONSOLIDATION TEST RESULTS

HISER PROPERTY

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SANTEE, CALIFORNIA

BY: CLS

DATE: APRIL 2020

PROJECT: 2020035

APPENDIX: D.9

GSI 2019

APPENDIX D

LABORATORY TESTING

Laboratory tests were performed on representative samples of site earth materials collected during our subsurface exploration in order to evaluate their physical characteristics. Test procedures used and results obtained are presented below.

Classification

Soils were visually classified with respect to the Unified Soil Classification System (U.S.C.S.) in general accordance with ASTM D 2487 and D 2488. The soil classifications of the onsite soils are provided on the Boring Logs in Appendix B.

Moisture Density Relations

The field moisture contents and field dry densities of selected, relatively undisturbed soil samples were previously evaluated in the laboratory, in general accordance with ASTM D 2216 and ASTM D 2937. The results of these tests are shown on Boring Logs in Appendix B.

Maximum Dry Density

The laboratory maximum dry density and optimum moisture content for a major soil type, encountered onsite, was evaluated in general accordance with test method ASTM D 1557. The following table presents the results:

LOCATION, SOIL TYPE	MAXIMUM DENSITY (PCF)	MOISTURE CONTENT (PERCENT)
B-3 Composite, Brown, Silty Sand	136.3	7.1

Particle-Size Analysis

A particle-size evaluation was performed on representative, soil samples in general accordance with ASTM D 422-63. The grain-size distribution curve is presented in Appendix D. The testing was utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The results of the particle-size evaluation indicate that the tested soils are a silty sand (1% gravel, 67.8% sand, and 31.2% fines) for a composite sample obtained from Borings B-2 and B-3 at a depth of 2½ to 10 feet.

Expansion Index

Tests were performed on representative soil samples collected during GSI's 2018 field exploration, in general accordance with ASTM D 4829. Test results and the soil's expansion potential are presented in the following table.

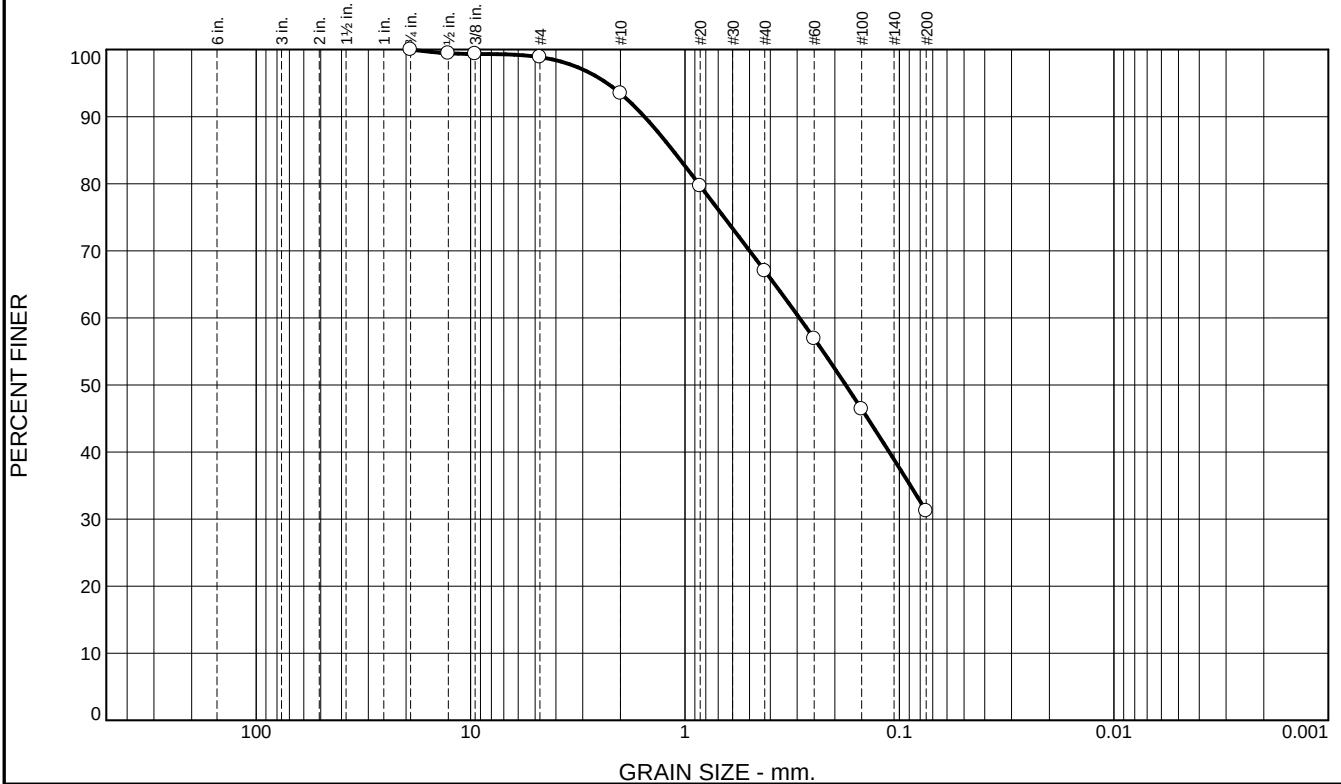
SAMPLE LOCATION	DESCRIPTION	EXPANSION INDEX	EXPANSION POTENTIAL
B-2/B-3 Composite 2½ to 10 feet	Silty Sand	<20	Very Low

Direct Shear

Shear testing was performed on a representative, undisturbed sample of site soil in general accordance with ASTM Test Method D 3080 in a Direct Shear Machine of the strain control type. The shear test results are presented in the following table:

SAMPLE LOCATION AND DEPTH (FT)	PRIMARY		RESIDUAL	
	COHESION (PSF)	FRICTION ANGLE (DEGREES)	COHESION (PSF)	FRICTION ANGLE (DEGREES)
B-3@ 5-7 (Remolded)	242	39.2	106	35.9
B-6 @ 5' (undisturbed)	98	42.2	98	34.9

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	1.1	5.4	26.5	35.8	31.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
.75	100.0		
.5	99.4		
.375	99.3		
#4	98.9		
#10	93.5		
#20	79.7		
#40	67.0		
#60	56.9		
#100	46.4		
#200	31.2		

* (no specification provided)

Source of Sample: B-3
Sample Number: B2/B3

Date: 8-27-19



Client: KB Home
Project: Hiser

Project No: 7678-A-SC

Plate

Material Description

Reddish Brown Silty Sand

Atterberg Limits

PL=

LL=

PI=

Coefficients

D₉₀= 1.5461
D₅₀= 0.1780
D₁₀=

D₈₅= 1.1412
D₃₀=
C_u=

D₆₀= 0.2930
D₁₅=
C_c=

Classification

USCS= SM

AASHTO=

Remarks

Tested By: TR

Checked By: TR

APPENDIX E

INFILTRATION FEASIBILITY

DOCUMENTS

Worksheet 0-1: Categorization of Infiltration Feasibility Condition

Feburary 2016

Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	Yes	No
3	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: Water contamination was not evaluated by NOVA Services. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: The potential for water balance was not evaluated by NOVA Services. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If all answers to rows 1 - 4 are “ Yes ” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration If any answer from row 1-4 is “ No ”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2		Proceed to Part 2

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Based on the on site infiltration testing, soil/geologic conditions below the north eastern proposed BMP facility allow for infiltration in an appreciable rate and volume.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
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Provide basis:

C2.1 A geologic investigation was performed at the subject site. Infiltration testing was conducted in under the guidance contained in Appendix D of the BMP Manual. (See NOVA 2020).

C2.2 Settlement and volume change due to water infiltration is not a concern with low expansive soils and no potential for liquefaction or hydro collapse.

C2.3 BMPs are to be sited a minimum of 50 feet away from any slope.

C2.4 BMPs are to be sited a minimum of 10 feet away from all underground utilities.

C2.5 Stormwater infiltration can result in damaging ground water mounding during wet periods. Based on the depth to groundwater and partial infiltration rate, groundwater mounding is not a high risk.

C2.6 Infiltration has the potential to increase lateral pressure and reduce soil strength which can impact foundations and retaining walls. BMPs are to be sited a minimum of 10 feet away from any foundations or retaining walls.

C2.7 Other Factors: The calculated infiltration rates of P-1 (1.11 inches per hour) and P-2 (0.17 inches per hour) were averaged, resulting in the estimated infiltration rate of 0.64 inches per hour. With consideration of the variability of the infiltration rates, it is NOVA's opinion that the estimated reliable infiltration rate below the proposed facility is less than 0.5 inches per hour. As such, partial infiltration is feasible at the north eastern proposed facility within the constraints cited above.

Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	Yes	No
7	Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: Water contamination was not evaluated by NOVA Services.			
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
8	Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: The potential for water balance was not evaluated by NOVA Services.			
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration. If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.		Partial Infiltration

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet 0-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		Worksheet C.4-1	
<u>Part 1 - Full Infiltration Feasibility Screening Criteria</u> Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?			
Criteria	Screening Question	Yes	No
1	Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X
Provide basis: The estimated infiltration rate of the existing soils below the north western proposed BMP facility (tests P-3 & P-4), based on the on-site infiltration study was calculated to be less than 0.5 inches per hour (0.07 inches per hour) after applying a minimum factor of safety (F) of F=2. The calculated infiltration rates of P-3 (0.01 inches per hour) and P-4 (0.13 inches per hour) were averaged, resulting in the estimated infiltration rate of 0.07 inches per hour. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability. (See geotechnical investigation report (NOVA 2020))			
2	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
Provide basis: No. See Criterion 1. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			

Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	Yes	No
3	Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: Water contamination was not evaluated by NOVA Services. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
4	Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: The potential for water balance was not evaluated by NOVA Services. Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.			
Part 1 Result*	If all answers to rows 1 - 4 are “ Yes ” a full infiltration design is potentially feasible. The feasibility screening category is Full Infiltration If any answer from row 1-4 is “ No ”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2		Proceed to Part 2

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 3 of 4

Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria

Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?

Criteria	Screening Question	Yes	No
5	Do soil and geologic conditions allow for infiltration in any appreciable rate or volume? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.	X	

Provide basis:

Based on the on site infiltration testing, soil/geologic conditions below the north western proposed BMP facility allow for infiltration in an appreciable rate and volume.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

6	Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.	X	
---	---	----------	--

Provide basis:

C2.1 A geologic investigation was performed at the subject site. Infiltration testing was conducted in under the guidance contained in Appendix D of the BMP Manual. (See NOVA 2020).
 C2.2 Settlement and volume change due to water infiltration is not a concern with low expansive soils and no potential for liquefaction or hydro collapse.
 C2.3 BMPs are to be sited a minimum of 50 feet away from any slope.
 C2.4 BMPs are to be sited a minimum of 10 feet away from all underground utilities.
 C2.5 Stormwater infiltration can result in damaging ground water mounding during wet periods. Based on the depth to groundwater and partial infiltration rate, groundwater mounding is not a high risk.
 C2.6 Infiltration has the potential to increase lateral pressure and reduce soil strength which can impact foundations and retaining walls. BMPs are to be sited a minimum of 10 feet away from any foundations or retaining walls.
 C2.7 Other Factors: Partial infiltration is feasible at the north western proposed facility within the constraints cited above.

Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	Yes	No
7	Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: Water contamination was not evaluated by NOVA Services.			
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
8	Can infiltration be allowed without violating downstream water rights? The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.		
Provide basis: The potential for water balance was not evaluated by NOVA Services.			
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.			
Part 2 Result*	If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is Partial Infiltration. If any answer from row 5-8 is no, then infiltration of any volume is considered to be infeasible within the drainage area. The feasibility screening category is No Infiltration.		Partial Infiltration

*To be completed using gathered site information and best professional judgment considering the definition of MEP in the Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings

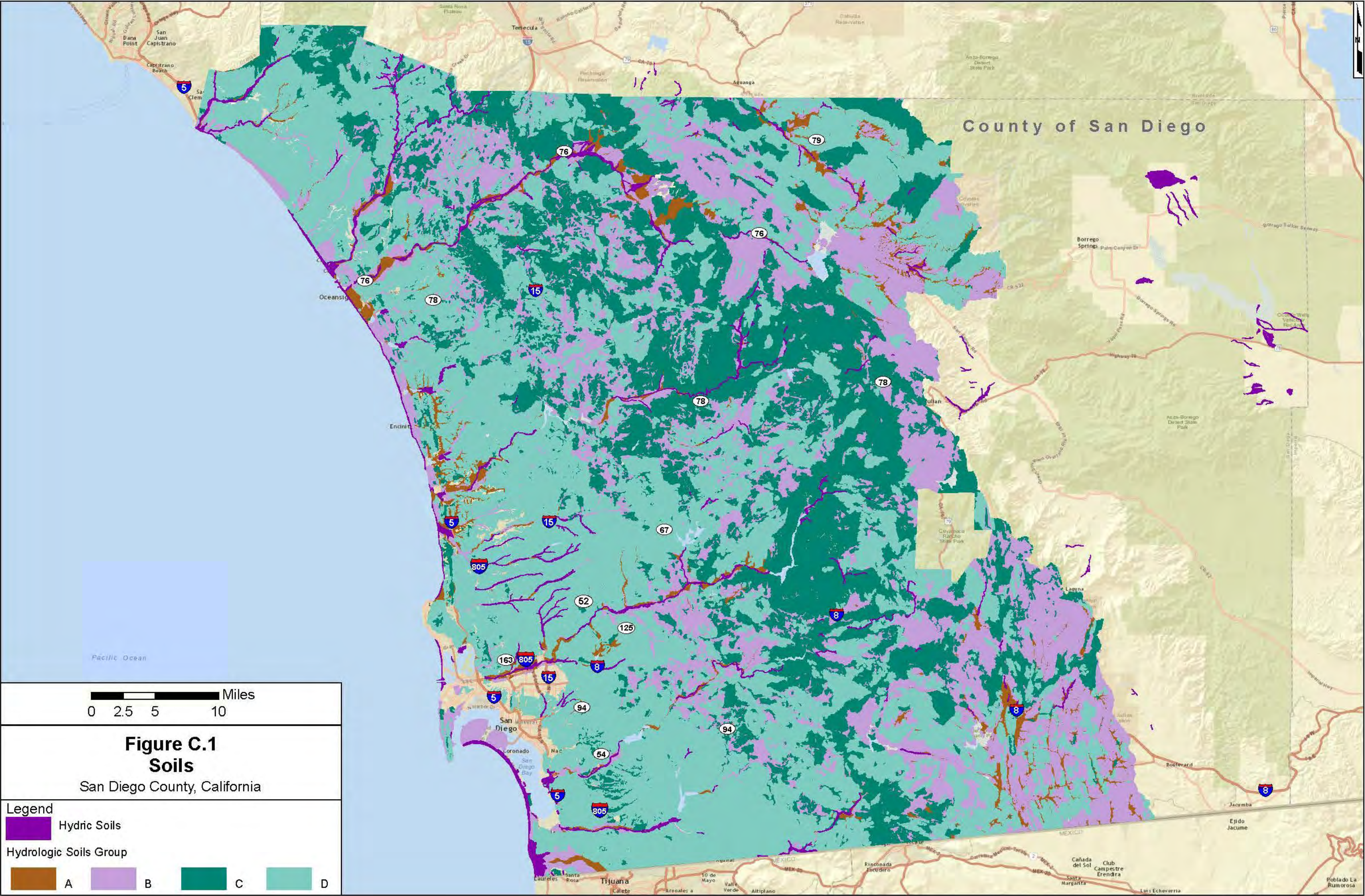
C.5 Feasibility Screening Exhibits

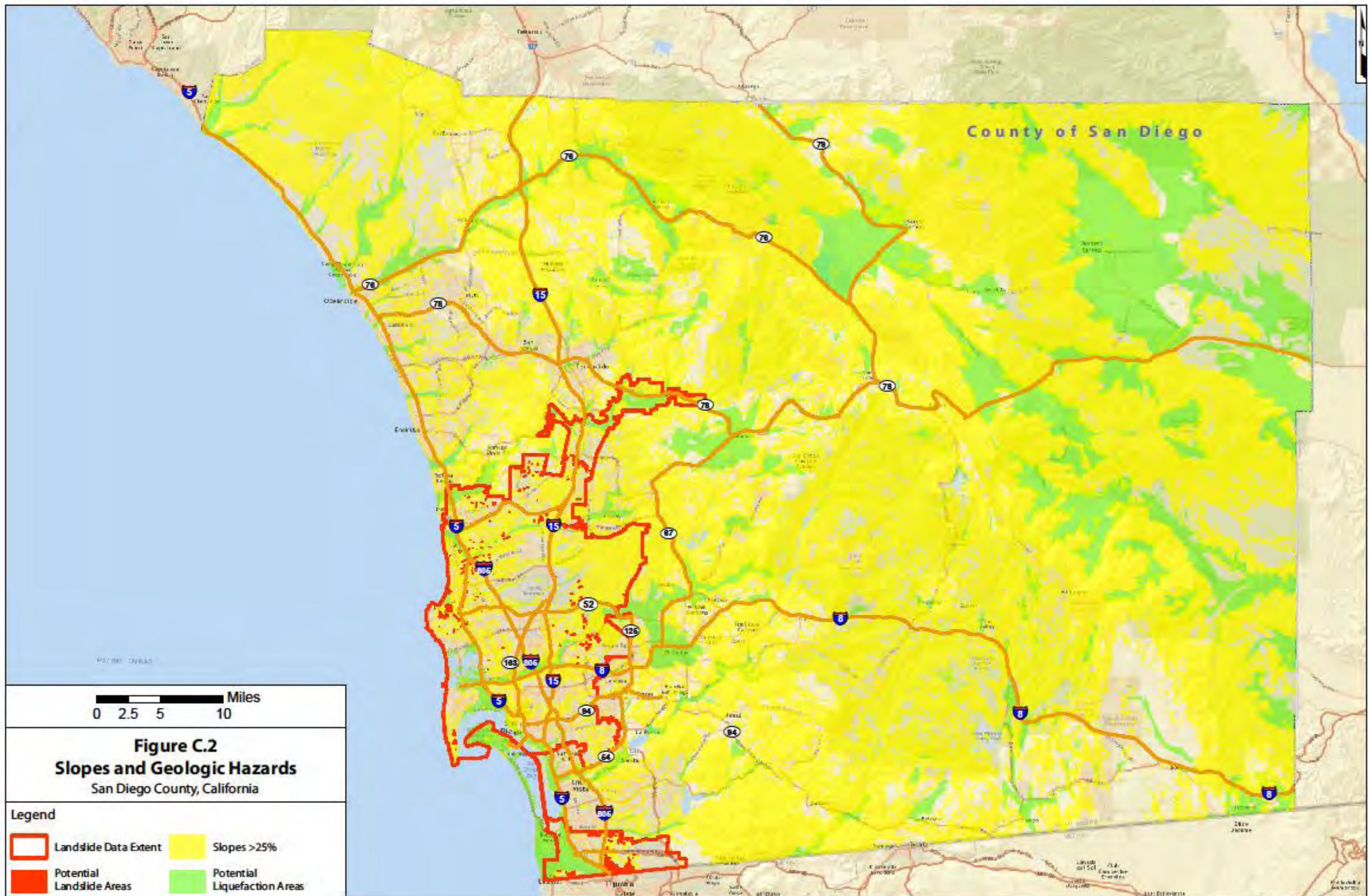
Table C.5-1 lists the feasibility screening exhibits that were generated using readily available GIS data sets to assist the project applicant to screen the project site for feasibility.

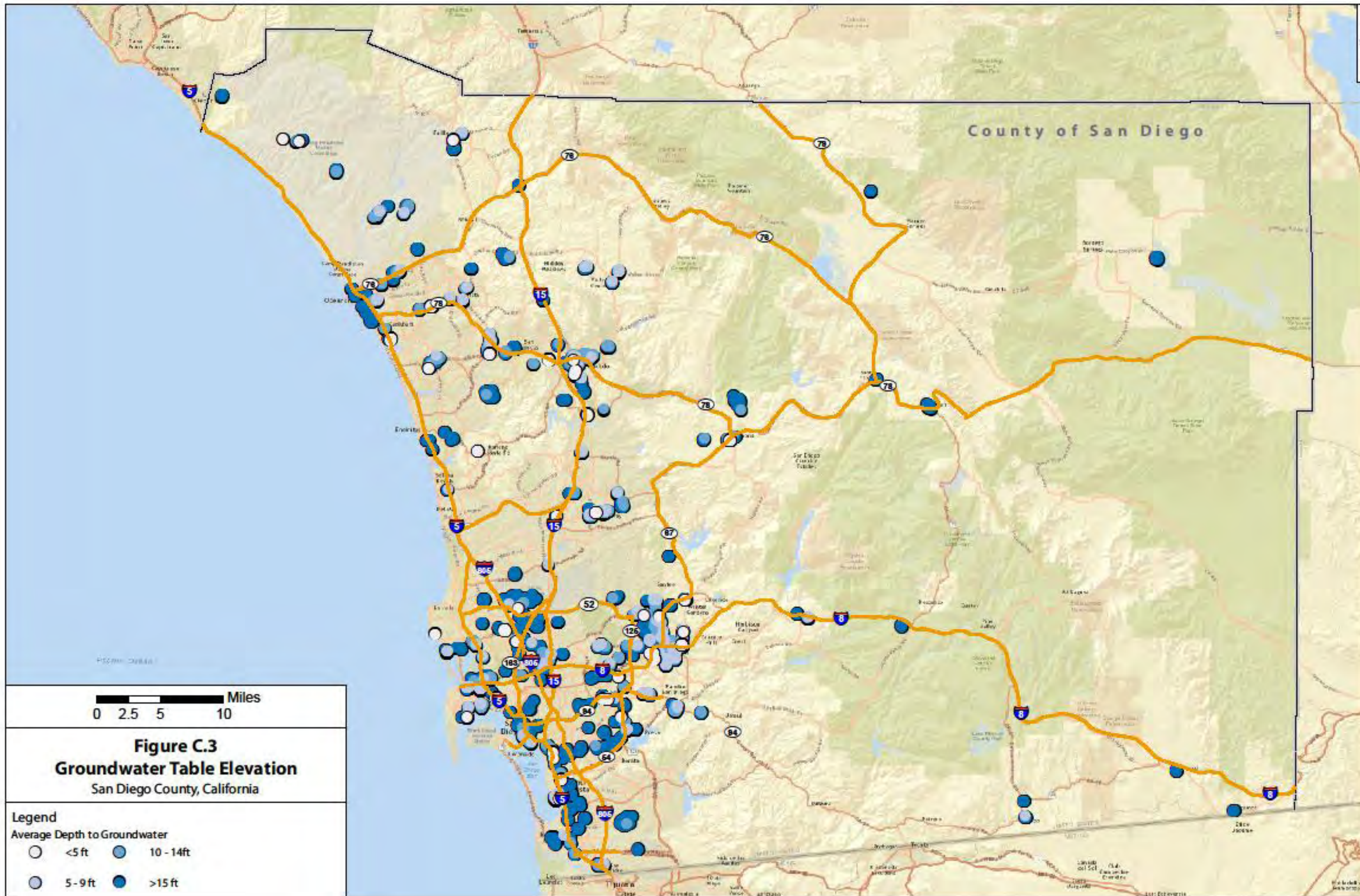
Table 0-1: Feasibility Screening Exhibits

Figures	Layer	Intent/Rationale	Data Sources
C.1 Soils	Hydrologic Soil Group – A, B, C, D	Hydrologic Soil Group will aid in determining areas of potential infiltration	SanGIS http://www.sangis.org/
	Hydric Soils	Hydric soils will indicate layers of intermittent saturation that may function like a D soil and should be avoided for infiltration	USDA Web Soil Survey. Hydric soils, (ratings of 100) were classified as hydric. http://websoilsurvey.sc.egov.usda.gov/App/HomePage.htm
C.2: Slopes and Geologic Hazards	Slopes >25%	BMPs are hard to construct on slopes >25% and can potentially cause slope instability	SanGIS http://www.sangis.org/
	Liquefaction Potential	BMPs (particularly infiltration BMPs) must not be sited in areas with high potential for liquefaction or landslides to minimize earthquake/landslide risks	SanGIS http://www.sangis.org/
	Landslide Potential		SanGIS Geologic Hazards layer. Subset of polygons with hazard codes related to landslides was selected. This data is limited to the City of San Diego Boundary. http://www.sangis.org/
C.3: Groundwater Table Elevations	Groundwater Depths	Infiltration BMPs will need to be sited in areas with adequate distance (>10 ft) from the groundwater table	GeoTracker. Data downloaded for San Diego county from 2014 and 2013. In cases where there were multiple measurements made at the same well, the average was taken over that year. http://geotracker.waterboards.ca.gov/data_download_by_county.asp
C.4: Contaminated Sites	Contaminated soils and/or groundwater sites	Infiltration must be limited in areas of contaminated soil/groundwater	GeoTracker. Data downloaded for San Diego county and limited to active cleanup sites http://geotracker.waterboards.ca.gov/

Appendix C: Geotechnical and Groundwater Investigation Requirements







0 2.5 5 10 Miles

Figure C.3
Groundwater Table Elevation
San Diego County, California

Legend

Average Depth to Groundwater

- | | | | |
|---|----------|---|------------|
| ○ | <5 ft | ● | 10 - 14 ft |
| ○ | 5 - 9 ft | ● | >15 ft |

