Update to the Revised Report Geotechnical Investigation

Leucadia Mixed-Use 1900-1950 North Coast Highway, Encinitas, California

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NOVA Project 2019189
April 13, 2021

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Fenway Capital Advisors
674 Via de la Valle, Suite 310
Solana Beach, CA 92075

April 13, 2021
NOVA Project 2019189

Attention: Mike Jensen

Subject: Update to the Revised Report
Geotechnical Investigation
Leucadia Mixed-Use
1950 North Coast Highway, Encinitas, California

Dear Mr. Jensen:

NOVA Services, Inc. (NOVA) is pleased to present herewith the above-referenced report. The work reported herein was completed by NOVA for Fenway Capital Advisors in accordance with NOVA's proposal dated August 15, 2019, as authorized on September 6, 2019. A report of the findings of the geotechnical investigation was submitted in April 27, 2020 and revised in NOVA's report submitted September 10, 2020.

This update to the September 10, 2020 report has been requested due to changes in stormwater planning for the site that have come about during conversations with the City of Encinitas, as well as minor revisions to the bluff overlay zone area calculations. The only revisions to NOVA's September 10, 2020 report are located in Section 4.4 Hillside Inland Bluff Overlay Zone, Section 8.0 Stormwater Infiltration of this report, and attached in Appendix E Infiltration Records.

NOVA appreciates the opportunity to be of continued service on this project.

Sincerely,
NOVA Services, Inc.

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Report
Geotechnical Investigation

Leucadia Mixed-Use
1900-1950 North Coast Highway, Encinitas, California

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1.0 INTRODUCTION

1.1 Terms of Reference

This report presents the findings of the geotechnical investigation of project known to NOVA as the 'Leucadia Mixed Use,' proposed for development on three adjacent parcels (APN 216-041-20, APN 216-041-21, and APN 216-041-06), 1900-1950 North Coast Highway 101 in Encinitas (hereinafter, 'the site').

The work reported herein was completed by NOVA Services, Inc. (NOVA) for Fenway Capital Advisors in accordance with NOVA's proposal dated August 15, 2019, as authorized on September 6, 2019 and as modified by a Change Orders dated September 25, 2019, and March 3, 2020.

Figure 1-1 provides a graphic that depicts the site vicinity.

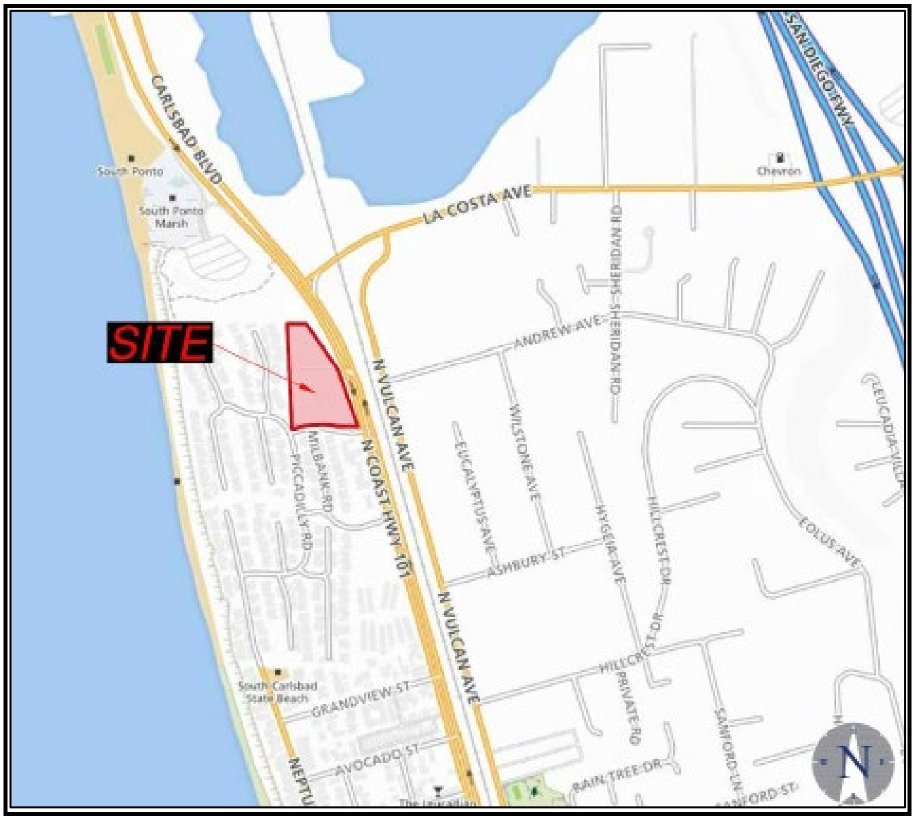


Figure 1-1. Site Vicinity Map

1.2 Update to the Revised Report

A report of the findings of the geotechnical investigation was submitted in April 27, 2020 and revised in NOVA's report submitted September 10, 2020. Review by the City of Encinitas based on revisions to the stormwater planning and slope analysis bluff overlay has required revisions to the geotechnical report. This report updates prior reporting to address the updated plans. The report revises and supersedes the September 10, 2020 report of the same title.

1.3 Objectives, Scope and Limitations of Work

1.3.1 Objectives

The objectives of the work reported herein are twofold, as described below.

1. Objective 1, Geotechnical. Characterize the subsurface in a manner sufficient to provide recommendations for geotechnical-related design, including foundations and related earthwork.

2. Objective 2, Infiltration. Assess the suitability of the site for development of permanent stormwater infiltration Best Management Practices ('BMPs').

1.3.2 Scope

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

1. Task 1, Background Review. Reviewed available background data regarding the site area, including geotechnical reports, historical aerial photography, topographic maps, geologic data, fault maps and reports, and preliminary development plans for the project. Structural information was not available at the time of this report.

2. Task 2, Subsurface Exploration. The subsurface exploration included seven subtasks:

    - *Subtask 2-1, Reconnaissance.* Prior to undertaking any invasive work, NOVA conducted a site reconnaissance, including layout of subsurface explorations used to determine subsurface conditions. Underground Service Alert was notified for utility mark-out services. NOVA contracted with a private utility locator to conduct additional underground utility locations.

    - *Subtask 2-2, Permitting and Coordination.* Borings were permitted in accordance with County of San Diego DEH requirements. Specialty subcontractors were retained to conduct the drilling, cone penetrometer soundings, and shear wave velocity survey. NOVA coordinated with the client regarding access for fieldwork.

    - *Subtask 2-3, Engineering Borings.* A specialty subcontractor drilled 8 hollow-stem auger borings. A NOVA geologist directed the drilling and conducted logging and sampling using ASTM methods.

    - *Subtask 2-4, Percolation Testing.* Two hollow stem auger borings were located in areas of possible Drainage Management Areas (DMAs). The borings were converted to wells and tested to determine percolation rates.

    - *Subtask 2-5, CPT Soundings.* Two cone penetration test soundings ('CPT', after ASTM D 5778) were completed by a specialty subcontractor.

    - *Subtask 2-6, Closure.* Borings were backfilled in accordance with County of San Diego DEH requirements.

- *Subtask 2-7, Seismic Shear Wave Survey*. A shear wave survey was conducted by a Professional Geophysicist. Findings of this survey provide the shear wave velocity in the top 30 meters of the site for site class determination, and for use in the site-specific ground motion hazard analysis.

3. Task 3, Laboratory Testing. Laboratory testing was conducted on representative samples of soils recovered from the engineering borings. The testing was focused toward determination of soil 'index' characteristics that can be correlated with soil mechanical characteristics (i.e., strength and compressibility).

4. Task 4, Engineering Evaluations. The findings of Tasks 1 through 3 were utilized to support geotechnical evaluations relevant to the planned development, addressing foundation design, earthwork, temporary shoring, pavements, and development of permanent stormwater infiltration BMPs.

5. Task 5, Reporting. This report presents the findings of the subsurface investigation and recommendations for foundation design and earthwork.

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 10 of this report addresses this consideration in more detail

This report addresses geotechnical 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 provides additional discussion regarding limitations and use of this report.

1.4 Understood Use of This Report

NOVA expects that the findings and recommendations provided herein will be utilized by the Design Team in making geotechnical-related design and construction decisions for the planned development.

1.5 Report Organization

The remainder of this report is organized as described below.

- Section 2 reviews the presently available project information.
- Section 3 describes the field investigation and laboratory testing.
- Section 4 describes the site conditions.
- Section 5 reviews geologic, soil and siting-related hazards common to this area of California, considering each for its potential to affect this site.
- Section 6 provides recommendations for foundation design and related earthwork.
- Section 7 provides guidance for design of temporary shoring.
- Section 8 provides guidance for development of permanent stormwater infiltration BMPs.
- Section 9 provides recommendations for pavement design.
- Section 10 addresses design phase review and construction observation.
- Section 11 lists the references used in preparation of the report.

Figures and tables that amplify discussion in the text are embedded at their point of reference. Plates providing larger scale view of certain figures are provided immediately following the text of the report. The report is supported by seven appendices.

- Appendix A provides important guidance regarding the use and limitations of the report.
- Appendix B provides logs of borings.
- Appendix C provides records of CPT soundings.
- Appendix D provides laboratory analytical results.
- Appendix E provides infiltration records.
- Appendix F provides the results of the seismic shear wave survey.
- Appendix G provides the results of a Site-Specific Ground Motion Hazard Analysis.

2.0 PROJECT INFORMATION

2.1 Location

The mixed-use multifamily and commercial development is proposed to be constructed on three adjacent parcels, APN 216-041-20 (1.94 acres), APN 216-041-21 (1.06 acres), and APN 216-041-06 (collectively hereinafter, 'the site') in Encinitas. The nominal addresses of the site are 1900-1950 North Coast Highway 101, Encinitas, California.

The triangular site is bounded to the north by a site currently being graded for a new hotel development, North Coast Highway 101 to the east, Moorgate Road to the south, and residential development to the west.

The location and limits of the site are depicted on Figure 2-1.

Figure 2-1. Site Location and Limits

2.2 Current Site Use

As may be seen by review of Figure 2-1, the site includes several existing small structures and some asphalt and concrete pavements. The southeast portion of the site includes buildings currently being used commercially, with small single-story buildings and parking. Until relatively recently, the northern portion of the project was the site of a restaurant with two appurtenant parking areas.

The following describes the parcels that comprise the site.

- APN 216-041-20. Parcel 20 to the north, is currently occupied by a closed restaurant with a large two-level parking lot on a graded pad. Elevations on this pad range from +58 feet mean sea level (msl) at its access point with Highway 101, to +94 feet msl at the property line on the west. The eastern edge of the lower parking lot has an approximately 20-foot high slope descending to Highway 101. Historic aerial photography indicates that Parcel 20 has been developed since at least 1980.

- APN 216-041-21. Parcel 21 to the south is vacant and open, with elevations ranging from +95 feet msl along the western property line, to +58 msl at its access point with Highway 101. Parcel 21 has been undeveloped since at least 1947.

- APN 216-041-06. Parcel 06 at the southeast corner of the site is currently occupied by a restaurant, two small commercial businesses, and at grade parking areas. This parcel is contiguous with Parcel 21 to the west, with a 12-foot high cut slope separating them. Average elevation for this parcel is +57 feet msl. Parcel 06 has been developed since at least 1947.

2.3 Historic Site Use

NOVA reviewed aerial photography dating to 1947 using the Historical Aerials website in order to better understand historic site usage. This review indicates that the site and the area around it was largely undeveloped until sometime after 1967. Aerial photography available for review over the period 1947 to 1967 indicates that the area was largely used for agriculture. The site area itself was undeveloped except for a few small buildings on Parcel 06.

Surrounding areas in the site vicinity became developed for residential use sometime over the period 1967 to 1980. Despite relatively intensive development in the area, the site itself remained largely undeveloped. There is no indication by review of historic aerial photography that the site was used for any purpose other than the light commercial use evident from the existing structures on site.

Figure 2-2 (following page) provides a 1967 aerial view of the site, the time immediately prior to development of the area around the site. Evident in this view is the past agricultural use of the site and site area.

Figure 2-2. 1967 Aerial View of the Site Vicinity

2.4 Planned Development

2.4.1 Architectural

NOVA's understanding of current planning for the development is based upon review of conceptual planning provided in Concept Plan 3, Fenway Hwy 101, Encinitas, Stephen Dalton Architects, 26 August 2019 (hereinafter, 'SDA 2019').

The mixed-use development will include four multi-family, multi-story residential structures with underground parking in the central and southwest portion of the site, six at-grade retail structures with one to two stories of residential units above them along the eastern portion of the site, and an at-grade boutique hotel in the northwest corner of the site.

The residential buildings will be constructed with two below-grade garage parking levels. The difference in existing surface ground elevation compared to the design elevations across the site will result in significant grading in order to develop the site. Grading plans are not currently available.

Figure 2-3 (following page) shows a sketch of the Conceptual Plan 3 (SDA 2019).

Figure 2-3. Concept Plan 3
(source: SDA 2019)

2.4.2 *Structural*

Design is only conceptual at this point, such that no structural drawings are available. The two levels of below-grade construction will be of reinforced concrete. The above grade elements of the development may be framed in wood, masonry and/or steel.

Figure 2-4 provides an elevation view of the development, indicating the extent of below-grade construction that is anticipated.

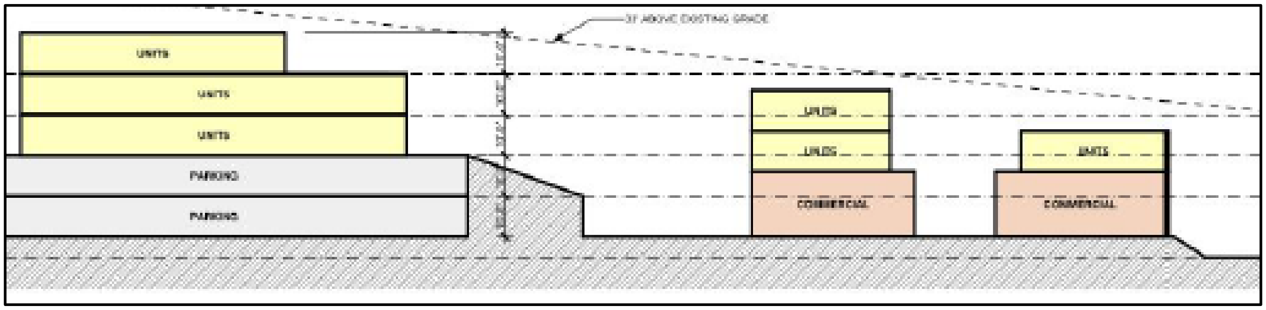


Figure 2-4. Elevation View, East-West Section
(source: SDA 2019, Section A-A)

Information regarding column loads is not yet available. The magnitude of these loads will vary primarily with column spacing. Based upon review of the architectural drawings, NOVA anticipates that interior columns will be loaded to about 800 kips (DL + LL), with exterior and corner columns loaded to about half that magnitude. It is unlikely that the structure will have foundations with net uplift.

2.4.3 Stormwater Infiltration BMPs

At the time of NOVA's field investigation, stormwater mitigation planning by the civil engineer had not been completed, so preliminary infiltration testing was performed. The subsurface exploration (see Section 3 and Section 4) indicates that the near-surface soils will behave as a relatively clean, sandy soil. As is discussed in Section 8, the design infiltration rate (I) ranges between 1.51 and 1.74 inches per hour, favorable for permanent stormwater BMPs. However, the *City of Encinitas BMP Design Manual* limits the use of permanent stormwater BMPs near slopes and coastal bluffs. Since the time of the field investigation, due to the adjacency of the site to the coastal bluffs to the west, the site has been designed with a 'no infiltration' condition.

2.4.4 Earthwork

The parking structure is developed to two levels beneath the residential structures as indicated on Figure 2-4, with excavations ranging between 15 feet and 32 feet below ground surface (bgs). SDA 2019 indicates the majority of the site is in cut, and will result in a significant amount of soil being removed from the property.

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

Virtually all of the excavation would be completed in the naturally occurring formational sandstone that underlie the site. The sandstone should largely be readily excavated utilizing conventional earthmoving equipment (i.e., larger backhoes, bulldozers, etc.). However, it should be expected that localized areas of higher cementation within the formation may require special excavation techniques. Such techniques could include the use of hoe-rams to loosen soils prior to excavation and ripping.

3.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

3.1 Overview

The subsurface exploration included eight engineering borings ('B-1' through 'B-8'); percolation testing at two locations ('P-1' and 'P-2'); 2 CPT soundings ('CPT-1' and 'CPT-2'); and a shear wave survey ('SW-1). Figure 3-1 provides a plan view of the site that indicates the locations of the separate elements of the subsurface exploration. Plate 1, provided immediately following the text of this report, reproduces this graphic in larger scale. Plate 2 provides geologic cross-sections, indicating subsurface conditions across the site.

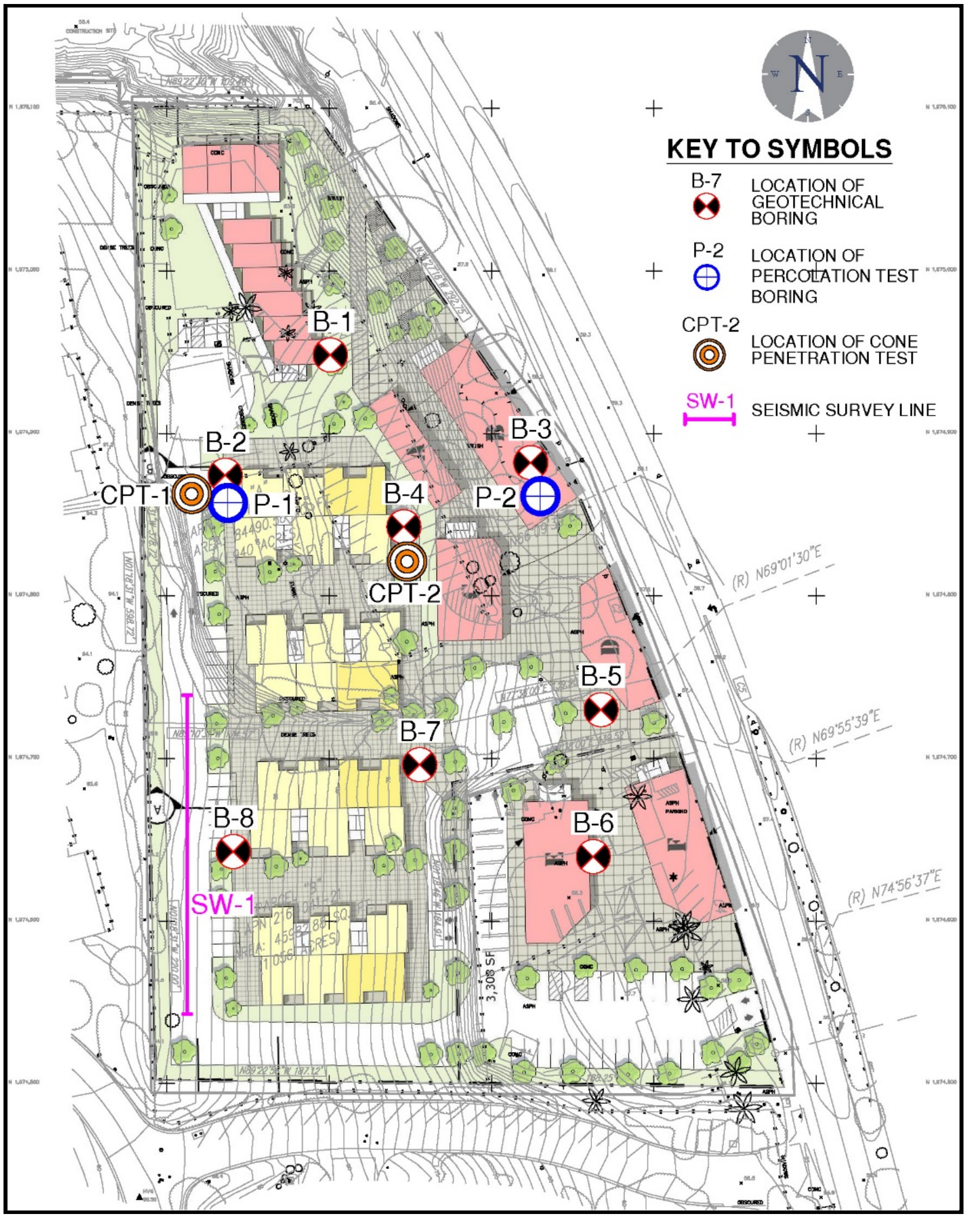


Figure 3-1. Location of the CPT Soundings, Engineering, and Percolation Test Borings

3.2 Engineering Borings

3.2.1 Drilling

Eight hollow-stem auger borings were drilled to depths between 21.5 feet and 56.5 feet below ground surface (bgs) on September 17 and 18, 2019. The borings were drilled at the locations depicted on Figure 3-1 under the surveillance of a NOVA geologist. Samples recovered from the borings were delivered to NOVA's materials laboratory for analysis.

The borings were advanced by a CME 95 truck-mounted drilling rig utilizing hollow-stem auger drilling equipment. Boring locations were determined in the field by the geologist. Figure 3-2 (following page) depicts drilling operations. Table 3-1 abstracts the indications of the borings.

Table 3-1. Abstract of the Engineering Borings

Boring Reference	Approx. Ground Surface Elevation (feet, msl)	Total Depth Below Ground Surface (feet)	Elevation at Completion (feet, msl) [1]	Depth to Formation (feet) [2]
B-1	+76.0	21.5	+54.5	3
B-2	+76.0	43.0	+33.0	0
B-3	+54.0	21.5	+32.5	5.5
B-4	+68.0	36.5	+31.5	0
B-5	+55.0	21.5	+33.5	0
B-6	+55.0	21.5	+33.5	2.5
B-7	+71.0	41.5	+29.5	0
B-8	+87.0	56.5	+30.5	0

Notes: 1. None of the borings encountered groundwater.
2. 'Formation' is sandstone of the Old Paralic Deposits (Qop).

3.2.2 Sampling

Both disturbed and relatively undisturbed samples were recovered from the borings. Sampling of the soils is described below.

1. The Modified California sampler ('ring sampler', after ASTM D 3550) 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. SPT blow counts for the final 12 inches of penetration comprise the SPT 'N' value, an index of soil consistency.

3. Bulk samples were recovered to provide composite samples for index testing.

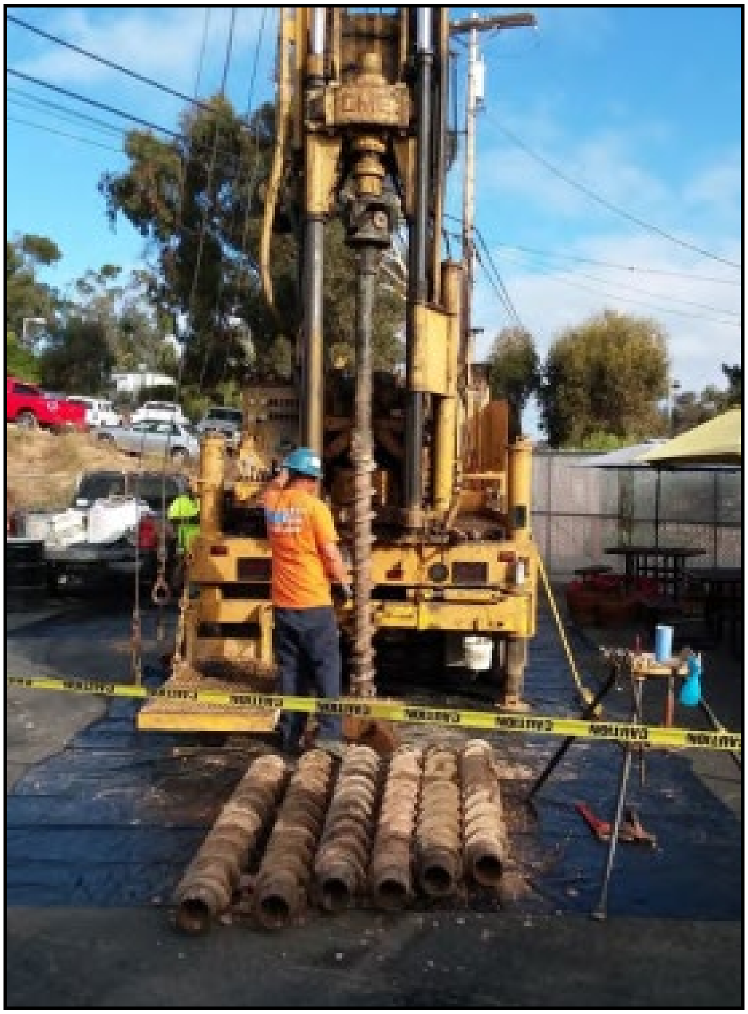


Figure 3-2. Drilling Operations, September 17, 2019

3.2.3 Logging

The 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 presented in Appendix B.

3.2.4 Closure

Upon completion, the borings were backfilled with soil cuttings. The area was cleaned and left as close to the original condition as practical. Where borings exceeded the depth of 21.5 feet bgs, the backfill was completed using bentonite in compliance with San Diego DEH well closure requirements.

3.3 CPT Soundings

Two (2) Cone Penetration Test soundings ('CPT', after ASTM 5778) were completed to depths of 50 feet and 38 feet bgs on October 10, 2019 at locations indicated on Figure 3-1.

Table 3-2 abstracts the CPT soundings.

Table 3-2. Abstract of the CPT Soundings

Sounding	Approx. Elevation (feet, msl)	Total Depth (feet)	Tip Elevation (feet, msl)
CPT-1	+82	50	+32
CPT-2	+68	38	+30

Both the CPT soundings and the borings indicate a relatively homogenous subsurface of fine to medium grained soils. Subsurface exploration using the cone penetration test (CPT) allows development of a continuous profile of the subsurface, useful for more detailed characterization of the soils.

Figure 3-3 (following page) reproduces the profile developed by CPT-1, from which it can be seen that sands dominate the subsurface. Both soundings correlate well with the borings.

3.4 Percolation Testing

3.4.1 General

The final location of the stormwater infiltration facilities, or BMPs, have not been designed at this time. NOVA directed the construction of two percolation test borings following the recommendations for percolation testing presented in the City of Encinitas BMP Design Manual, dated February 2016. The percolation test locations are shown on Figure 3-1 and designated as P-1 and P-2.

3.4.2 Drilling

The borings were drilled with a truck-mounted 8-inch hollow stem auger to depths of 5 feet 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.

3.4.3 Conversion to Percolation Well

Once the borings were drilled to the desired depth, the borings were converted to percolation test wells by placing an approximately 2-inch layer of ¾-inch gravel on the bottoms, then extending 3-inch diameter Schedule 40 perforated PVC pipe to the ground surface. The ¾-inch gravel was used to partially fill the annular space around the pipe below the existing finish grade to minimize the potential of caving.

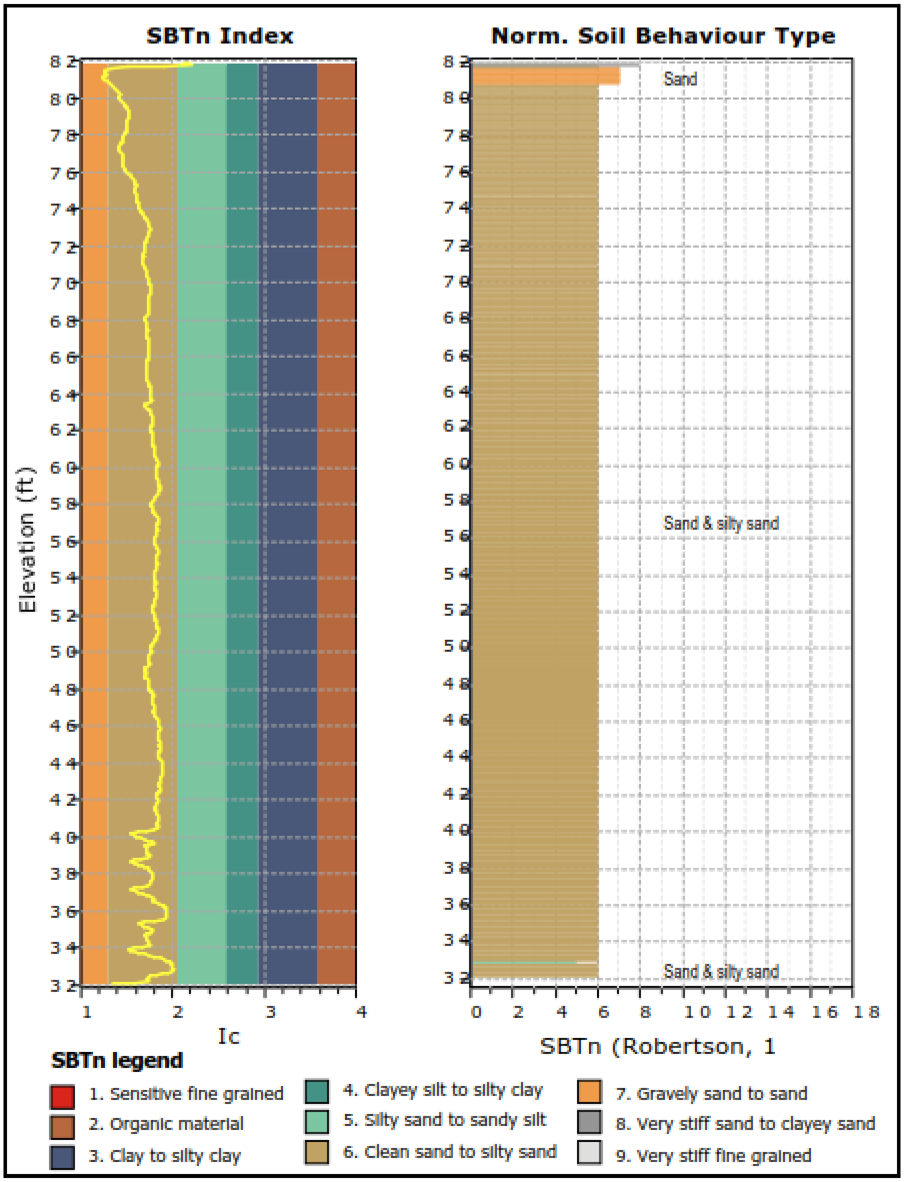


Figure 3-3. Subsurface Profile at CPT-1

3.4.4 *Percolation Testing*

The percolation test wells were pre-soaked by filling the wells with water to at least five times the well radius. The pre-soak water was observed to percolate more than 6-inches into the soil unit within 25 minutes. Therefore, the holes were filled with water to the ground surface elevation and testing commenced the following day, within a 26-hour window.

On the day of the percolation testing, a 25-minute test interval was used to determine if the fast test method (1-hour) or the slow test method (6-hours) should be used. When more than 6-inches of water percolated within the first 25 minutes in each test well, the fast test method was chosen per the City’s BMP Design Manual.

Water levels were recorded every 10 minutes for one hour. At the beginning of each 10-minute test interval, the water level was raised to the water level of previous test intervals in order to maintain a near-constant head during the hour-long test.

Table 3-3 abstracts the percolation test conditions and resulting percolation rate.

Table 3-3. Abstract of the Percolation Testing

Boring	Approximate Elevation (feet, msl)	Total Depth (feet)	Approximate Percolation Test Elevation (feet, msl)	Percolation Rate (in/hr)	Subsurface Unit Tested
P-1	+55	5.0	+50	58.32	Qop
P-2	+76	5.0	+71	77.04	Qop

Notes: 1. The referenced geologic unit is Old Paralic Deposits (Qop).
2. Percolation rate is not infiltration rate. Infiltration rates are discussed in Section 8.

The infiltration rates, which were calculated using the percolation rates, is discussed in detail in Section 8.

3.4.5 Closure

At the conclusion of the percolation testing, the PVC pipes were removed and the resulting holes were backfilled with soil cuttings.

3.5 Laboratory Testing

3.5.1 General

Soil samples recovered from the engineering borings 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 engineering properties.

The laboratory program included visual classifications of all soil samples as well as index testing in general accordance with ASTM standards. Records of the geotechnical laboratory testing are provided in Appendix D.

3.5.2 Maximum Density and Optimum Moisture

Two samples of the near-surface soil were tested to determine its moisture-density relationship after ASTM D1557 (the 'modified Proctor'). This testing indicated an optimum dry density (γ_{dry}) of γ_{dry} = 133.0 lb/ft^3 at an optimum moisture content (w) of w = 8.3 percent at B-1 and 133.8 lb/ft^3 at an optimum moisture content 7.3 percent at B-5.

3.5.3 Soil Gradation

The visual classifications were further evaluated by grain size testing. Table 3-4 (following page) provides an abstract of this testing.

Table 3-4. Abstract of the Soil Gradation Testing

Sample Reference		Percent Finer than the U.S. No 200 Sieve	Classification after ASTM D2488
Boring	Depth (feet)		
B-2	15	7	SP-SM
B-2	20	5	SP
B-3	5	13	SP-SM
B-4	5	21	SM
B-4	10	11	SP-SM
B-4	15	6	SP-SM
B-5	0	23	SM
B-5	5	29	SM
B-7	5	8	SP-SM
B-7	7.5	8	SP-SM
B-7	10	15	SM
B-7	35	4	SP
B-8	2.5	14	SP-SM
B-8	7.5	14	SP-SM
B-8	15	8	SP-SM
B-8	25	6	SP-SM
B-8	45	13	SP-SM
B-8	50	6	SP-SM

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

3.5.4 R-Value

The Resistance Value (R-value) test is a material stiffness test, demonstrating a material's resistance to deformation as a function of the ratio of transmitted lateral pressure to applied vertical pressure.

The purpose of this test is to determine the suitability of prospective subgrade soils and road aggregates for use in the pavement sections of roadways. The test is used by Caltrans for pavement design, replacing the California Bearing Ratio (CBR) test. A saturated cylindrical soil sample is placed in a Hveem Stabilometer device and then compressed. The stabilometer measures the horizontal pressure that is produced while the specimen is under compression.

A sample representative of soils from the near-surface was selected for this testing. Testing after ASTM D 2844 indicated an R-value of 35, a value characteristic of the sands that mantle the site.

3.5.5 Direct Shear

Two samples of the paralic deposits were tested in direct shear after ASTM D3080. The results of this testing are provided on Table 3-5 (following page).

Table 3-5. Abstract of the Direct Shear Testing

Sample Reference		Apparent Cohesion (c, psf)	Angle of Internal Friction (ϕ, degrees)
Boring	Depth (feet)		
B-7	7.5	323	25
B-8	10	307	38

Note: B-7 @ 27’ is a remolded sample, remolded at 8% moisture

3.5.6 *Chemical Testing*

Resistivity, sulfate content and chloride contents were determined to estimate the potential corrosivity of the soils. These chemical tests were performed on a representative sample of the near-surface soils by Clarkson Laboratory and Supply, Inc.

Table 3-6 abstracts the chemical testing. Indications of this testing are discussed in more detail in Section 6.3.

Table 3-6. Abstract of Chemical Testing

Sample Ref		pH	Resistivity (Ohm-cm)	Sulfates		Chlorides	
Boring	Depth (feet)			ppm	%	ppm	%
B-5	2 – 5	7.4	1200	64	0.006	170	0.004
B-8	2.5 – 7.5	7.4	10000	33	0.003	32	0.003

3.6 Shear Wave Velocity Analysis

3.6.1 *General*

A seismic shear wave survey was performed on March 7, 2020 by a Professional Geophysicist (PGP). The purpose of the survey was to assess the one-dimensional average shear wave velocity of the underlying site soils to a minimum depth of 100 feet bgs in order to classify the site in accordance with ASCE 7-16 Table 20.3-1.

The seismic survey of the site included one shear wave survey traverse, approximately 180 feet in length. The approximate location of the survey is shown on Figure 3-1 and Plate 1.

3.6.2 *Methodology*

Multi-channel analysis of surface waves (MASW) and microtremor array measurement (MAM) methods were used for the analysis. Combining results of both methods maximizes the depth and resolution of the data.

The MAM survey records vibrations from background and ambient noise. The ground vibrations were recorded using a 32-second record length at 2-milisecond sampling rate with 30 separate records obtained for quality control purposes.

Figure 3-4. Seismic Survey Line, View towards the South

A 24-channel Geometrics StrataVisor NZXP model signal-enhancement refraction seismograph was used in conjunction with 24 4.5-Hz geophones spaced at regular intervals. For the MASW survey, two seismic records were obtained by multiple hammer strikes of a 16-pound sledge hammer on steel plates positioned 25 feet from the end of each terminus of the seismic line. Vibrations were recorded using a one second record length at a sampling rate of 0.5 milliseconds.

After the field data was collected, the geophysicist combined the MASW and MAM survey results using specialized software specific to this purpose.

3.6.3 Findings (V100)

The weighted average for velocity in the upper 100 feet of the site (V100) was computed from ASCE 7-16 Equation 20.4-1. The seismic model indicates that the average shear wave velocity (weighted average) in the upper 100 feet is 1,077.6 ft/sec. This average velocity classifies the underlying soils as Site Class D, near the D/C boundary.

4.0 SITE CONDITIONS

4.1 Geologic Setting

4.1.1 Regional

The project area is located in the Coastal Plain of the Peninsular Range 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.

This area of the Province has undergone several episodes of marine inundation and subsequent marine regression (coastline changes) throughout the last 54 million years. These events have resulted in the deposition of a thick sequence of marine and nonmarine sedimentary rocks on the basement igneous rocks of the Southern California Batholith and metamorphic rocks.

The gradual emergence of the region from the sea occurred in Pleistocene time, and numerous wave-cut platforms, most of which were covered by relatively thin marine and nonmarine terrace deposits, formed as the sea receded from the land. Accelerated fluvial erosion during periods of heavy rainfall, along with the lowering of base sea level during Quaternary times, resulted in the rolling hills, mesas, and deeply incised canyons which characterize the landforms in western San Diego County.

The Coastal Plain increases in elevation from west to east across marine terrace surfaces uplifted during Pleistocene time. Sedimentary rocks consist of sandstones, siltstones, and claystones that were deposited during the Cretaceous, Tertiary, and Quaternary periods.

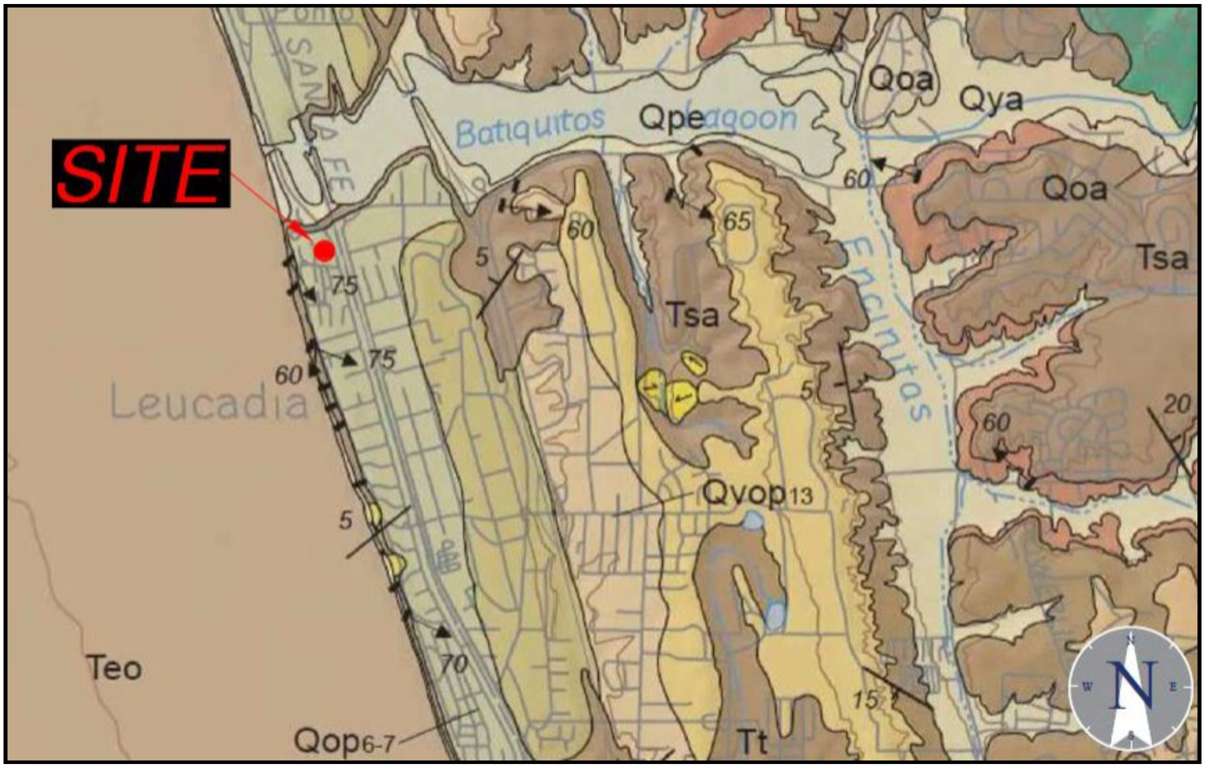
4.1.2 Site Vicinity

The subject property is sited atop a coastal terrace, that forms a coastal bluff west of the property, at the beach. The property is underlain by Pleistocene-aged old paralic deposits (Qop6-7). Differently numbered Qvop deposits (evident in Figure 4-1 on the following page) designate different geologic ages and elevations of abrasion platforms.

The old paralic deposits occur widely. They are found from the international border and extend northward beyond San Diego County. These deposits comprise the dominant near-surface geologic formation in much of coastal San Diego County. Paralic deposits provide relatively high-quality foundation support for civil development across the county.

Figure 4-1 (following page) reproduces geologic mapping of the site vicinity.

The paralic deposits generally consist of strandline, beach, estuarine and colluvial siltstones, sandstones and conglomerates. As encountered at this site, the paralic deposits consist of orange-brown, dry to damp, weakly cemented, weathered, friable, silty sandstone. This silty sandstone is underlain by a pale orange gray to grayish-white, dry to damp friable sandstone with trace silt. In some areas of existing improvements, the paralic deposits are overlain by a thin veneer of artificial fill (Qaf), to maximum depths of 5 feet below existing ground surface (bgs) in Boring B-3, but generally less than 2 feet.



KEY TO SYMBOLS

Qop6-7	OLD PARALIC DEPOSITS, UNITS 6-7	Qya	YOUNG ALLUVIAL FLOOD-PLAIN DEPOSITS	Qop2-4	OLD PARALIC DEPOSITS, 2-4
Tsa	SANTIAGO FORMATION	Qpe	PARALIC ESTUARINE DEPOSITS	Teo	UNDIVIDED EOCENE ROCKS IN OFFSHORE AREA

Figure 4-1. Geologic Mapping of the Site Vicinity

4.2 Surface, Subsurface, and Groundwater

4.2.1 Surface

The site includes several small structures and some asphalt and concrete pavements. The southeast portion of the site is used commercially, and until recently, a restaurant operated on the northern parcel. The restaurant is closed, but the structure and appurtenant utilities and parking lot remain. This northern portion of the site includes several mature trees surrounding the parking area. The undeveloped portions of the site are very lightly vegetated.

The western edge of the site is a north-south trending ridge with elevations between +88 feet and 95 feet msl. This ridge slopes to the east across Parcels 21 and 06 (the southern parcels) at a surface gradient of about 10%, descending about 30 feet to Highway 101. On Parcel 20, the parking lot areas have been graded into the slope, creating two benched areas, with slopes on the east and west.

Figure 4-2 (following page) depicts surface conditions looking west from Parcel 06 to 21.

Figure 4-2. Surface Conditions

4.2.2 Subsurface

The field exploration indicates the site is covered by a thin veneer of undocumented sandy fill in some locations within the areas of existing improvements, below which occurs sandstones associated with old paralic deposits. For the purposes of this report, the sequence of subsurface soils may be described as listed below.

1. Unit 1, Fill (Qaf). Undocumented fill is assumed to underlie the existing structures on the property. This fill was encountered within Borings B-1, B-3, and Boring B-6, extending to depths of 2 feet to 5 feet. The fill may be thicker in other areas of the site not explored by our borings. The fill is an orange-brown silty sand of medium dense consistency.

2. Unit 2, Old Paralic Deposits (Qop6-7). The entire site is underlain by marine terrace deposits. The upper 15 to 20 feet of this unit is slightly cemented, with the consistency of a dense silty fine to medium sand. At depth, cementation weakens as the unit grades to a dense, poorly-graded (‘well-sorted’) sandstone with only trace fines.

   These friable poorly-graded sandy zones may affect shoring and other construction. Figures 4-3 and 4-4 (following page) depict the cemented and friable sandstone lenses.

Plate 2, provided immediately following the text of this report, presents geologic cross-sections that indicate the occurrence of the above soil/rock units across the site.

4.2.3 Groundwater

As is indicated on Table 3-1, none of the borings encountered groundwater. Groundwater thus occurs below about El +10 feet msl, at least 48 feet below the finished floor of the lowest parking level (to be set at about El +58 feet msl).

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 (e.g., evident seeps, areas of water staining, eroded areas, etc.) at the time of NOVA’s field exploration that would suggest active problems with surface water.

Figure 4-3. Near Surface Unit 2 Paralic Sandstone

Figure 4-4. Unit 2 Paralic Deposits (Poorly-Graded Sands) at Depth

4.3 Mechanical Characteristics of the Subsurface Materials

4.3.1 Indications of the Borings and Soundings

As is discussed in Section 3, the borings and CPT soundings indicate the site is underlain by deep, sandy soils. *In situ* testing conducted for this project indicates that these sands are of generally dense consistency, with higher strength and low compressibility.

Figure 4-5 is a graphic that provides the variation of soil strength and compressibility with depth, reproducing data developed at CPT-1.

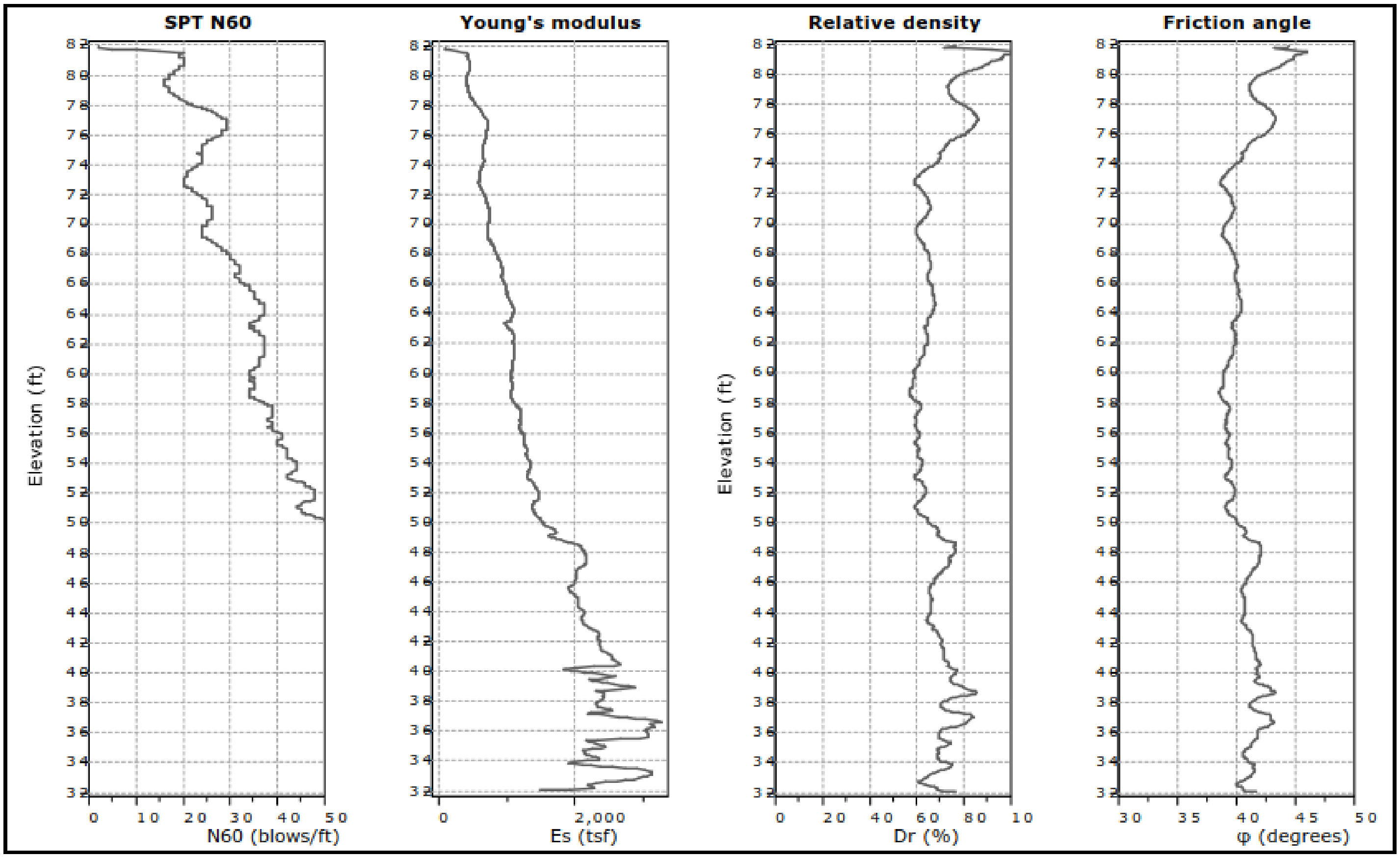


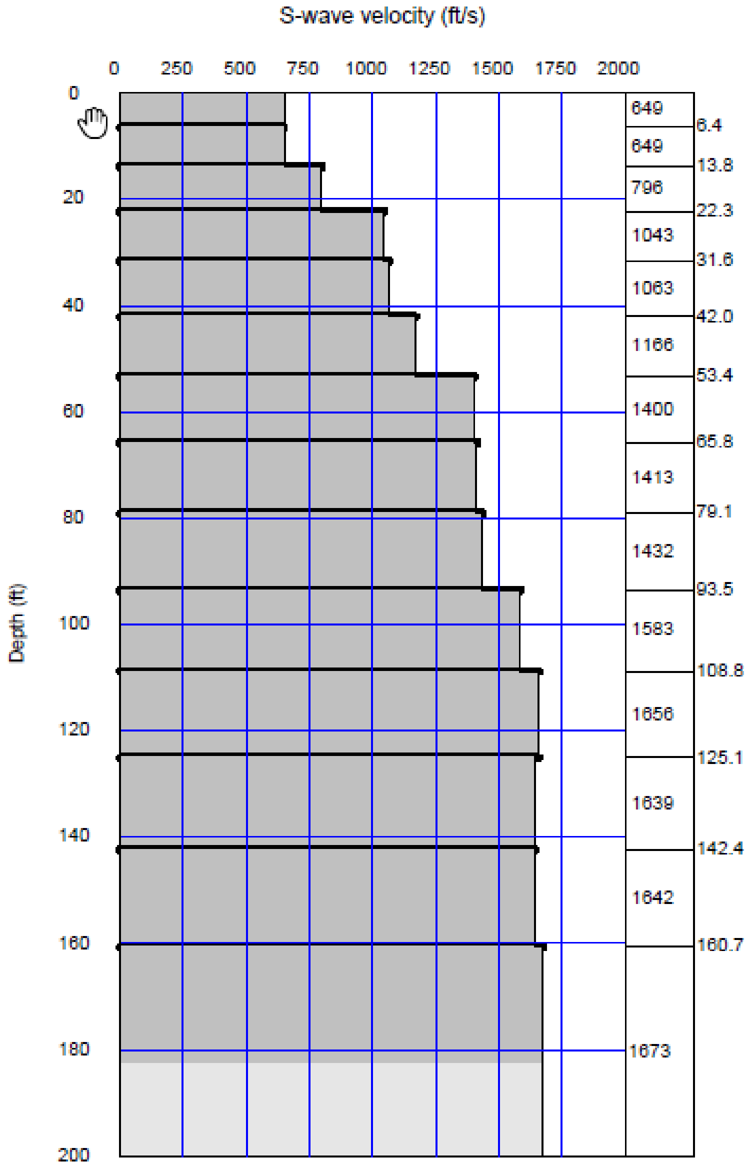
Figure 4-5. Strength and Compressibility of the Subsurface

4.3.2 Indications of Shear Wave Survey

As is discussed in Section 3, NOVA completed a seismic shear wave study along a single approximately 180-foot traverse (‘SW-1’) aligned as shown on Figure 3-1 and Plate 1.

Shear wave velocity measurements obtained from this survey correlate well with the data obtained from the borings. Analysis of the data indicates that the weighted average shear wave velocity in the upper 100 feet (‘V100’) beneath the limits of the planned building is V100 = 1,077.6 feet per second.

Figure 4-6 plots average shear wave velocities over different intervals of the subsurface to a depth of 200 feet below ground surface. Considering the weighted average of the shear wave velocity in the top 100 feet as determined along traverse SW-1, the site may be classified as Site Class D after ASCE 7-16, Table 20.3-1.



SHEAR-WAVE VELOCITY MODEL: Average Vs 100ft = 1,077.6 ft/sec

Site Classification (CBC-2019 1613A.2.2 & ASCE 7-16 Ch. 20)- "D" (Stiff Soil)

Figure 4-6. Seismic Shear Wave Velocity Profile

4.4 Hillside Inland Bluff Overlay Zone

4.4.1 Regulatory

In an effort to preserve natural coastal and inland bluffs within the City of Encinitas, Encinitas has adopted Municipal Code 30.34.030: Hillside/Inland Bluff Overlay Zone (HIBCZ) regulations.

The scope and principal requirements of the HIBCZ regulations are abstracted below.

- The HIBOZ regulations apply to all areas within the Special Study Overlay Zone where site-specific analysis indicates that 10% or more of the area of a parcel of land exceeds 25% gradient for natural slopes.

- Applicants for projects proposed within the HIBCZ must submit a slope analysis based upon a topographic map with contour intervals not exceeding two feet. This analysis must describe the slope categories in acres and also graphically depict the location of each category on the topographic map.

- Where development is proposed on slopes greater than 25% grade, slopes greater than 25% grade must be preserved in their natural state.

4.4.2 Existing Slope Gradients

Figure 4-7 (following page) shows a topographic map of the existing conditions of the site, indicating that 15.57% of the project has a slope greater than 25%. Figure 4-7 is reproduced in larger scale as Plate 3, provided immediately following the text of this report.

Slopes greater than 25% grade (i.e., of 4:1 (horizontal : vertical)), are indicated in Figure 4-7 in yellow and red. Slopes indicated in yellow have gradients of 25% to 40%, and slopes indicated in red have gradients of 40% or more.

4.4.3 Review of Historical Grading

General

NOVA reviewed aerial photography that depicts earthwork-related development of the site over the period 1947 to 1980. Additionally, earthwork initiated in 2019 that is associated with development of the adjacent Encinitas Beach Resort is also considered.

After performing background review of aerial photography available for this site, it is NOVA’s judgment that the slopes indicated on Figure 4-7 (Also included as Plate 3 of this report) are manufactured slopes that are the direct result of grading activities. The following subsections detail NOVA’s review.

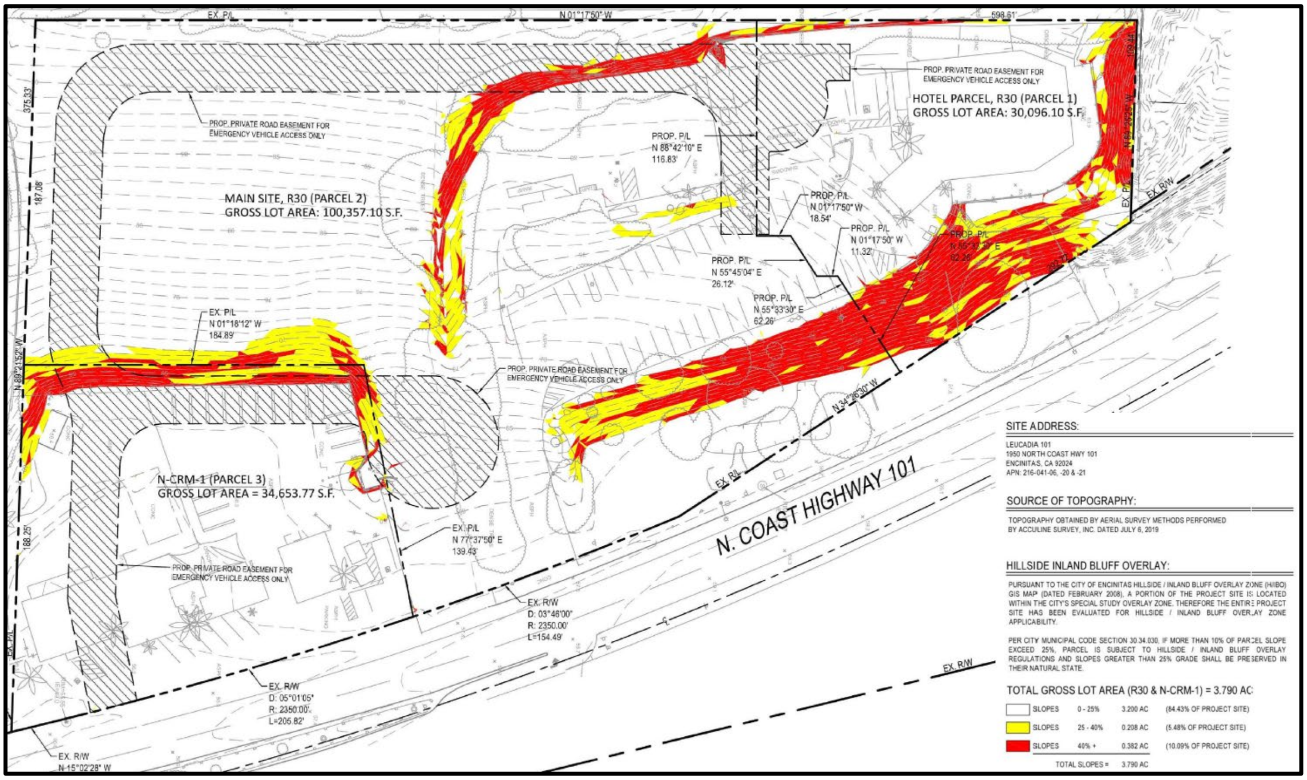


Figure 4-7. Site-Specific Slope Analysis
(source: PSLA 2021)

<u>Site Condition in 1947</u>

Figure 4-8 depicts the site in 1947. The property is mostly unimproved, natural terrain. The three buildings that exist today are shown in the southeast of the property.

Figure 4-8. 1947 Aerial Photo

Site Condition in 1964

Figure 4-9 shows the site condition in 1964. Evident in review of this figure is that the parking lot behind the buildings on the southeast corner of the site was expanded by cutting into the slope to the west.

Figure 4-9. 1964 Aerial Photo

In addition to the parking lot cuts, in the northern portion of the site adjacent to Highway 101, the photo indicates cutting of the lower lying area between the road and the ascending slope to the west. This area is the existing right-of-way with wet and dry utilities installed within it.

By cutting this area flat, a slope greater than 25% above Highway 101 was formed.

Site Condition in 1980

Figure 4-10 shows the site in 1980. Evident in this figure is the southeast parking lot graded to the present day configuration with cut slopes exceeding 25%. Also evident in the north, are two parking lots serving the restaurant in the northern portion of the site that were cut into the natural slope, creating manmade cut slopes steeper than 25% in order to accommodate parking.

Figure 4-10. 1980 Aerial Photo

Figure 4-10 may be compared to Figure 4-7 (and Plate 3), from which it can be seen that the graded slopes are those identified in Figure 4-7 as having grades exceeding 25%.

For further comparison, Figure 4-11a and Figure 4-11b (following page) juxtapose 1947 and 1980 photos for the purpose of presenting ‘before grading’ and ‘after grading’ site conditions. It appears clear that grading has altered the site over the period 1947 to 1980.

Figure 4-11a. 1947 Site Condition

Figure 4-11b. 1980 Site Condition

<u>2019 Grading</u>

In 2019, grading for the Encinitas Beach Resort required a large back cut which encroached into the north end of the subject site to accommodate the resort structures. Figure 4-12 presents this portion of the Encinitas Beach Resort Grading Plan next to the Slope Analysis figure. The red dotted line on the grading plan indicates the top of the back cut on the subject property. This back cut had been made by the time the topographic map was created for the Slope Analysis, and the area on the north end of the property exceeding 25% slope was made by this back cut. It is apparent by review of the 1947 aerial photo (Figure 4-11a), that this steep slope is not a natural feature.

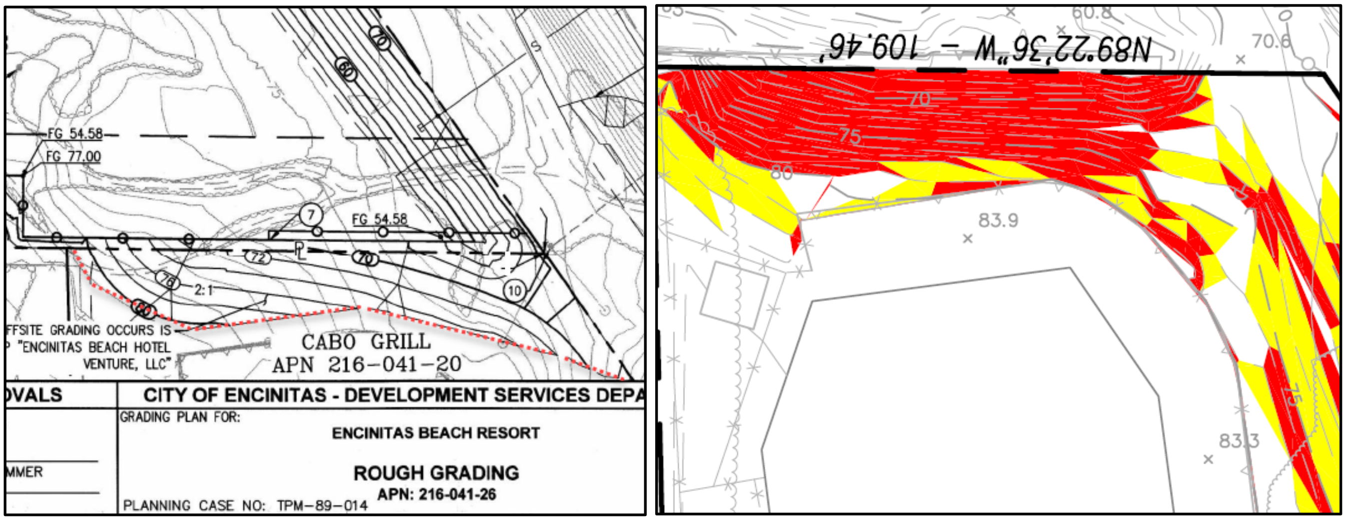


Figure 4-12. 2019 Back Cut for Encinitas Resort Construction

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.

The primary hazard identified by this review is that the site is at risk for moderate-to-severe ground shaking in response to a large-magnitude earthquake during the lifetime of the planned development. While strong ground motion could affect the site, there is little risk of liquefaction or related seismic phenomena. The expectation of strong ground motion is common to all civil works in this area of California.

The following subsections describe NOVA's review of soil and geologic hazards.

5.2 Geologic Hazards

5.2.1 Strong Ground Motion

The site is not located within a currently designated Alquist-Priolo Earthquake Zone (Hart and Bryant, 2007). No known active faults are mapped as crossing the site area. The nearest known active faulting is in the Rose Canyon fault system, located approximately 3.5 miles west of the site. This system has the potential to be a source of strong ground motion.

The seismicity of the site was evaluated utilizing a web-based analytical tool provided by the USGS. This evaluation shows the site may be subjected to a Magnitude 7 seismic event, with a corresponding risk-based Peak Ground Acceleration (PGA_M) of PGA_M ~ 0.57 g.

5.2.2 Fault Rupture

No evidence of faulting was observed during NOVA's geologic reconnaissance of the site. There are no known active faults underlying the property. The site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone). No active or potentially active faults are mapped within the City of Encinitas (Encinitas, 2018).

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.

Figure 5-1 (following page) maps faulting in the site vicinity.

Figure 5-1. Faulting in the Site Vicinity

5.2.3 Landslide

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

Geologic reconnaissance and review of aerial photography indicated no evidence of active or dormant landsliding. Clues to landslide hazards can also be obtained by review of mapping that depicts both historic landslides and landslide-prone topography. Figure 5-2 reproduces such mapping for the site area. The mapping indicates that the site is in an area judged to be 'generally susceptible' to landsliding.

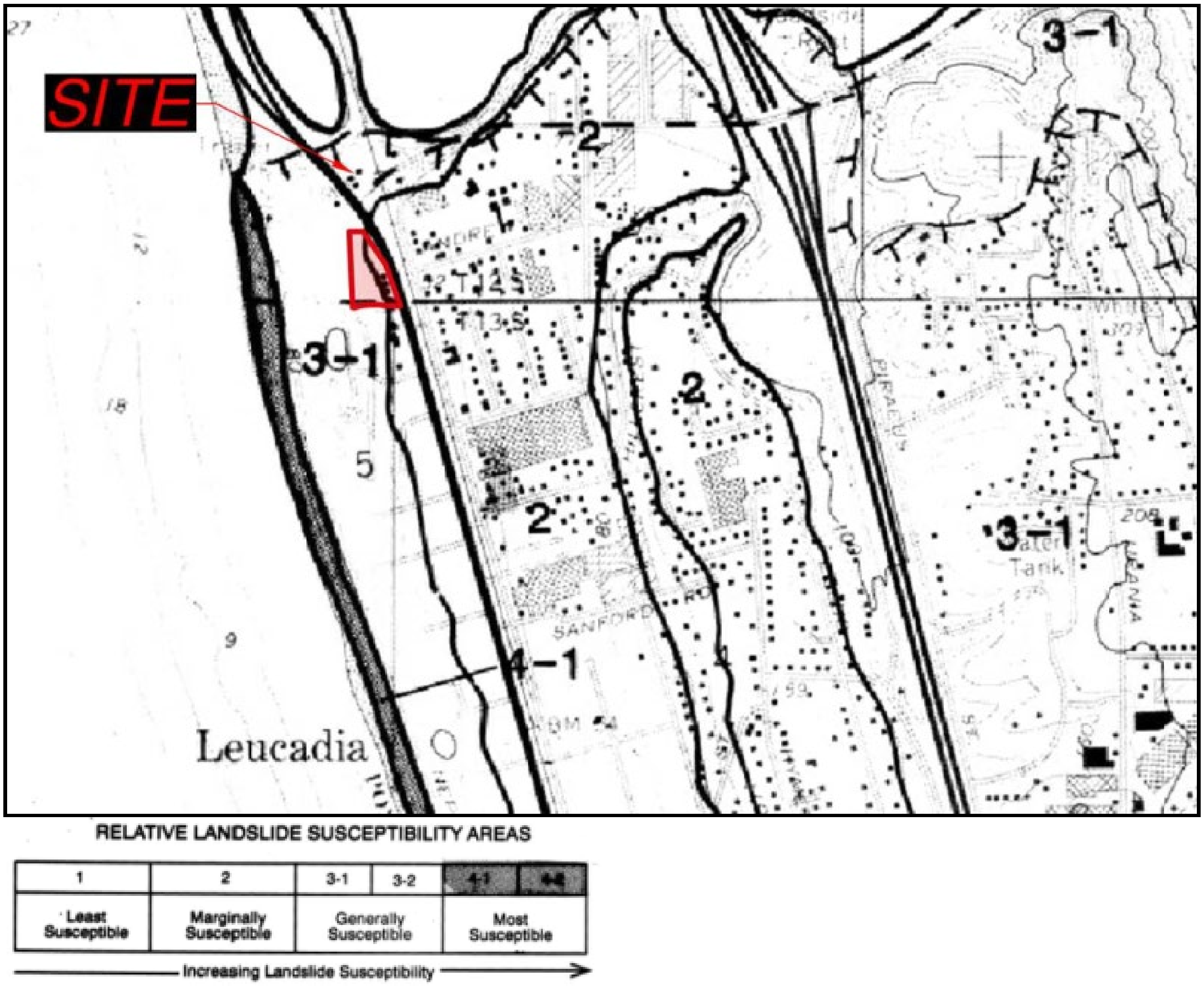


Figure 5-2. Landslide Susceptibility Mapping of the Site Area

In consideration of the shallow existing ground slopes and proposed grades at the project, NOVA considers the landslide hazard at the site to be ‘negligible’ for the site and the surrounding areas. The proposed development will not affect the landslide hazard characterization.

5.3 Soil Hazards

5.3.1 Embankment Stability

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

New slopes planned as part of the future site development will be stabilized with retaining walls. There is no concern regarding embankment stability at this site.

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 geologically 'younger' soils (i.e., soils of Holocene age), shallow water table (less than about 60 feet depth), and cohesionless (i.e., sandy and silty) soils of looser consistency. The seismic ground motions increase soil water pressures, decreasing grain-to-grain contact among the soil particles, which causes the soils to lose strength.

Resistance of a soil mass to liquefaction increases with increasing density, plasticity (associated with clay-sized particles), geologic age, cementation, and stress history. The cemented, dense (see Section 4) and geologically 'older' Unit 2 paralic deposits have no potential for liquefaction.

Seismically Induced Settlement

Apart from liquefaction, a strong seismic event can induce settlement within loose to moderately dense, unsaturated granular soils. The cemented, dense Unit 2 paralic deposits (with Vs > 1,000 feet/second) will not be prone to 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 clayey soil characterized by their ability to undergo significant volume changes (shrinking or swelling) due to variations in moisture content, the magnitude of which is related to both clay content and plasticity index. These volume changes can be damaging to structures. Nationally, the annual value of real estate damage caused by expansive soils is exceeded only by that caused by termites.

The predominately sandy soils of Units 1-2 are not potentially expansive.

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, dune sands, and loess (wind-blown sediment) deposits. These soils are characterized by low *in situ* density, low moisture contents, and relatively high unwetted 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.

The geologic age and depositional history of the Unit 1 and Unit 2 soils are such that these soils are not potentially hydro-collapsible.

5.3.5 Corrosive Soils

Chemical testing of the near-surface soils indicates the soils contain low concentrations of soluble sulfates and chlorides. These soils will not be corrosive to embedded concrete, but based on resistivity measurements, may be considered mildly to moderately corrosive to buried metals. Section 6 addresses this consideration in more detail.

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 right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

As building locations are finalized on the north end of the project, care should be taken that proposed structures do not load the retaining wall to the north. Section 6.5.4 addresses foundation setback requirements in more detail.

5.4.2 Flood

The project site is located within a FEMA-designated flood zone, designated as Flood “Zone X” Zone X is an “*area of minimal flood hazard*.” Figure 5-3 (following page) reproduces this mapping.

5.4.3 Tsunami

Tsunamis are seismic sea waves with a long wavelength (long compared to the ocean depth) generated by sudden movements of the ocean bottom during earthquakes, landslides, or volcanic activity. Review of published mapping of inundation potential indicates the site will not be affected.

Figure 5-4 (following page) maps potential tsunami inundation in the site area. As may be seen by review of this graphic, the site is not at risk for inundation by a tsunami.

Figure 5-3. Flood Mapping of the Site Area
(source: adapted from FEMA Map 06073C19037G, May 2012)

Figure 5-4. Tsunami Inundation Mapping of the Site Area
(source: CEMA, 2009)

6.0 EARTHWORK AND FOUNDATIONS

6.1 Overview

6.1.1 Review of Site Hazards

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

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

6.1.2 Site Suitability

Based upon the indications of the field and laboratory data developed for this investigation, as well as review of previously developed subsurface information, it is the opinion of NOVA that the site is suitable for development utilizing shallow foundations, 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 right-of-ways located adjacent to the site if the recommendations of this report are incorporated into project design.

6.1.3 Review and Surveillance

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

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

6.2 Seismic Design Parameters

6.2.1 Site Class

The shear wave testing described in Section 3 and Section 4 shows that the average shear wave velocity of the upper 100 feet of the subsurface (V_{100}) averages 1,077.6 feet per second. This average shear wave velocity meets the criterion for Site Class D per ASCE 7-16 (Table 20.3-1).

6.2.2 Seismic Design Parameters

Per Chapter 11 of ASCE 7-16, because the site is Site Class D, a site-specific risk-targeted maximum considered earthquake ground motion hazard analysis was performed in accordance with Chapter 21 of ASCE 7-16. Appendix G presents detailed documentation of the procedures and findings of this analysis. Table 6-1 presents the seismic design parameters resulting from this analysis.

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

Parameter	OSHPD and Table 11.4.6	Site Specific
Site Class	D	
Site Latitude (decimal degrees)	33.080933°N	
Site Longitude (decimal degrees)	-117.308435°W	
Site Coefficient, F_a	1.07	N/A
Site Coefficient, F_v	1.88	N/A
Mapped Short Period Spectral Acceleration, S_S	1.18 g	1.18
Mapped One-Second Period Spectral Acceleration, S_1	0.42 g	0.42
Short Period Spectral Acceleration Adjusted For Site Class, S_{MS}	1.26 g	1.38
One-Second Period Spectral Acceleration Adjusted For Site Class, S_{M1}	0.79 g	1.15
Design Short Period Spectral Acceleration, S_{DS}	0.84 g	0.92
Design One-Second Period Spectral Acceleration, S_{D1}	0.53 g	0.77

Source: *OSHPD Seismic Design Maps: https://www.seismicmaps.org/*

6.3 Corrosivity and Sulfates

6.3.1 Summary of Testing

Electrical resistivity, chloride content, and pH level are all indicators of the soil's tendency to corrode ferrous metals and/or attack embedded concrete. Levels of water-soluble sulfates correlate with the potential for a soil to attack embedded concrete. Chemical testing for these parameters were performed on representative samples. Records of this testing are provided in Appendix D. Table 6-2 summarizes the results.

Table 6-2. Summary of Corrosivity Testing

Parameter	Units	B-5 @ 2 – 5'	B-8 @ 2.5 – 7.5'
pH	standard unit	7.4	7.4
Resistivity	Ω-cm	1200	10000
Water-Soluble Chloride	ppm	170	32
Water Soluble Sulfate	ppm	64	33

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 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 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 moderately 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 on Table 6-2, the soil sample tested indicated water-soluble sulfate (SO_4) content of 170 parts per million ('ppm,' 0.017% by weight). With $SO_4 < 0.10$ percent by weight, the American Concrete Institute (ACI) 318-08 considers a soil to have no potential (S0) for sulfate attack. 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	$SO_4 < 0.10$	-	-	-
Moderate	S1	$0.10 \leq SO_4 < 0.20$	II	0.50	4,000
Severe	S2	$0.20 \leq SO_4 \leq 2.00$	V	0.45	4,500
Very severe	S3	$SO_4 > 2.0$	V + pozzolan	0.45	4,500

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

6.3.4 Limitations

Testing to determine several chemical parameters that indicate a potential for soils to be corrosive to construction materials are traditionally completed by the Geotechnical Engineer, comparing testing results with a variety of indices regarding corrosion potential. Like most geotechnical consultants, NOVA does not practice 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 on the known condition at the site and the conceptual project plans, NOVA expects that earthwork at the site will include (i) general clearing and grubbing measures with localized deeper excavation of tree roots in areas with mature trees; (ii) remedial grading in the locations of building pads on existing fill, building pads with cut/fill transition conditions, flatwork, and retaining wall footings; (iii) excavation and shoring of subterranean parking structure; (iv) construction of building and retaining wall footings; (v) retaining wall construction, and backfill; (vi) flatwork and pavement subgrade preparation, and (vii) the installation and backfill of wet and dry 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 Site Preparation

Prior to the start of earthwork, the site should be cleared of existing brush, trees and root systems, pavements, foundations and utilities from the existing site use. The deleterious materials should be disposed of in approved off-site locations.

At the outset of site work, the Contractor should establish construction Best Management Practices ('BMPs') to prevent erosion of graded/excavated areas until such time as permanent drainage and erosion control measures have been installed.

6.4.3 Excavation Characteristics

As is discussed in Section 2, Unit 2 paralic deposits will generally be readily excavated by earthwork equipment usual for developments of this nature. No special excavation techniques will be required for the majority of the sandstones encountered within the planed limits of excavation.

As is discussed in Section 4, the Unit 2 paralic deposits include friable poorly-graded sandy zones that may affect construction. The Contractor should anticipate that localized cohesionless sand layers in Unit 2 may be readily excavated, but difficult to control once exposed. Sloughing sand can occur because of this condition.

Construction complications resulting from sloughing and loss of ground occurred at the nearby Oceanside Beach Resort construction site. Slope failures, utility trench instability and related issues can occur in these clean (i.e., free of silt and clay-sized particles), uncemented sands.

6.4.4 Select Fill

General

All fill for structures or pavements should be Select Fill.

Materials

Select Fill should be a mineral soil free of organics and regulated constituents with the characteristics listed below.

- At least 40% by weight finer than ¼-inches in size.
- Maximum particle size of 3-inches.
- Expansion index (EI) of less than 30 (i.e., EI < 30, after ASTM D 4829).

Unit 1 and Unit 2 soils will conform to the above criteria.

Placement

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

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

Equipment employed for densification should be purpose-built. This will require the use of vibratory equipment to effect thorough compaction. Kneading and/or static compaction should not be accepted.

6.4.5 Remedial Grading

Buildings A, C, and the multi-family residential structure are anticipated to be founded entirely on competent formational materials, and are not anticipated to require remedial grading.

Undocumented Fill

As is discussed in Section 4, undocumented fill is assumed to underlie the existing structures on the property. This fill was encountered within Borings B-1, B-3, and Boring B-6, extending to depths of 2 feet to 5 feet. The fill may be thicker in other areas of the site not explored by our borings. The fill is an orange-brown silty sand of medium dense consistency.

Consistent with the policy of the City of Encinitas, all undocumented fill must be removed and replaced with Select Fill per Section 6.4.4.

Over-excavation

Structures and site retaining walls proposed in locations of existing undocumented fill, should be over-excavated to a minimum depth of 2 feet below the bottom of the proposed footings and on-grade slabs. Mixed-use Buildings B, D, E, and F are proposed in locations where existing fill or desiccated and disturbed formation was encountered and should be over-excavated such that the foundation will bear on a uniformly compacted fill blanket. Over-excavations should extend a minimum of 5 feet outside of the footprint of the structure.

Transition Conditions

Based on the conceptual design, the proposed hotel structure in the northern portion of the site has 2 feet of cut to grade on the northern edge of the structure and 6 feet of fill below the southern edge to achieve finish pad grade. Foundations for this structure should be entirely founded on engineered fill in order to mitigate potential effects of differential settlement resulting from spanning bedrock and fill.

The building should be over-excavated such that there is a minimum of three feet of engineered fill below the bottom of footings. In addition, any undocumented fill should be removed to contact with Unit 2 paralic deposits prior to placing new fill.

6.4.6 Processing Removal Bottoms

Once excavation is complete, the exposed surface should be inspected for areas of unusual softness, wetness or disturbance. Prior to placing new fill, the bottom of the excavations should be moisture conditioning to at least 3% above the optimum moisture content and compacted to a minimum of 90 percent relative compaction after ASTM D1557 (the ‘modified Proctor’).

6.4.7 Foundation Preparation

Soils loosened by excavation to the foundation level should be re-densified and verified by inspection by a representative of the GEOR that this preparation is adequate. The following general approach should be employed.

1. Step 1, Moisture Conditioning. The exposed soils should be moisture conditioned to 2% above optimum moisture content (with reference to ASTM D1557) to a depth of 12-inches.
2. Step 2, Densification. Moisture conditioned Unit 2 soils should be densified by compaction using vibratory compaction equipment, effecting thorough compaction to a minimum of 12- inches depth. Quality control testing should be undertaken to confirm that the upper 12- inches of foundation soil is densified to at least 90% relative compaction after ASTM D1557.
3. Step 3, Proof-Rolling of Slab Areas. The entire area of exposed, densified Unit 2 soil in slab areas should be proof-rolled by a heavy vehicle (for example, a loaded dump truck). Areas that appear soft/loose during the proof rolling process should be re-densified.

6.4.8 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.

Special care should be given to utility trenches excavated in the Unit 2 paralic deposits, where zones of loose, clean, and uncemented sand are likely to be encountered and may readily slough. Per OSHA guidelines, slopes created during temporary trenching for utilities should not be exposed for more than 24 hours. The maximum slope should not exceed 1.5:1 (H:V), and where possible, should be laid back on the order of 3:1. All excavations with vertical sides should be shielded or supported to a height at least 18 inches above the top of the vertical side.

Backfill for utility trenches is as important as the original subgrade preparation or engineered fill placed to support either a foundation or slab. Backfill for utility trenches must be placed to meet the project specifications for the engineered fill of this project. Unless otherwise specified, the backfill for the utility trenches should be placed in 4- to 6-inch loose lifts and compacted to a minimum of 90% 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.

6.4.9 Pavement Subgrades

The upper 12-inches of all pavement subgrades should be moisture conditioned to at least 2% above the optimum moisture content and replaced to at least 95% relative compaction after ASTM D1557 (the 'modified Proctor').

6.4.10 Flatwork

Prior to casting exterior flatwork, the upper 1 foot of subgrade soils should be removed and replaced to at least 90% relative compaction after ASTM D1557.

6.5 Shallow Foundations

6.5.1 General

Structures can be supported on shallow foundations embedded in either compacted fill or the Unit 2 sandstone provided the earthwork is completed as described in Section 6.4. The following subsections provide recommendations for shallow foundations. It is recommended that all foundation elements, including any grade beams, be reinforced top and bottom. The actual reinforcement should be designed by the Structural Engineer.

6.5.2 Shallow Foundations Supported on Compacted Fill

Minimum Dimensions and Reinforcing

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 30 inches wide, embedded at least 24 inches below surrounding grade.

Allowable Contact Stress

Continuous and isolated footings constructed as described in the preceding sections and supported on compacted fill may be designed using an allowable (net) contact stress of 2,500 pounds per square foot (psf). An allowable increase of 500 psf for each additional 12 inches in depth may be utilized, if desired.

In no case should the maximum allowable contact stress should be greater than 3,500 psf. The maximum bearing value applies to combined dead and sustained live loads (DL + LL). The allowable bearing pressure may be increased by one-third when considering transient live loads, including seismic and wind forces.

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. A frictional coefficient of 0.35 may be used. No reduction is necessary when combining frictional and passive resistance.

Settlement

Structure supported on shallow foundations as recommended above will settle on the order of 0.5 inch or less, with about 50% of this settlement occurring during the construction period.

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

6.5.3 Shallow Foundations Supported on Unit 2 Sandstone

The Unit 2 sandstones will provide high-capacity foundation support for shallow foundations. NOVA recommends use of conventional foundations, consisting of isolated and continuous footings, as described below.

Isolated Foundations

Isolated foundations for interior columns may be designed for an allowable contact stress of 4,500 psf for dead and commonly applied live loads (DL+LL). These foundation units should have a minimum width of 30 inches, embedded a minimum of 24 inches into sound Unit 2 sandstones. This bearing value may be increased by one-third for transient loads such as wind and seismic.

Continuous Foundations

Continuous foundations may be designed for an allowable contact stress of 4,000 psf for dead and commonly applied live loads (DL+LL). These footings must be a minimum of 24 inches in width and embedded a minimum of 24 inches into the Unit 2 sandstones.

This bearing value may be increased by one-third for transient loads such as wind and seismic.

Resistance to Lateral Loads

Lateral loads to shallow foundations cast ‘neat’ against Unit 2 sandstones may be resisted by passive earth pressure against the face of the footing, calculated as a fluid density of 300 psf per foot of depth, neglecting the upper 1 foot of soil below surrounding grade in this calculation. Additionally, a coefficient of friction of 0.35 between soil and the concrete base of the footing may be used with dead loads.

Settlement

Supported as recommended above, the structure will settle on the order of 0.5 inch or less. This movement will occur elastically, as dead load (DL) and permanent live loads (LL) are applied.

In usual circumstance, about 50% of this settlement will occur during the construction period. Angular distortion due to differential settlement of adjacent, unevenly loaded footings should be less than 1 inch in 40 feet (i.e., Δ/L less than 1:480).

6.5.4 Ground Supported Slabs

The ground level of the garage structures may employ conventional on-grade (ground-supported) slab designed using a modulus of subgrade reaction (k) of 120 pounds per cubic inch (i.e., k = 120 pci) for compacted fill and180 pci for Unit 2 sandstones.

The actual slab thickness and reinforcement should be designed by the Structural Engineer. NOVA recommends the slab 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 must be established to a depth of ¼ the slab thickness as depicted in Figure 6-1.

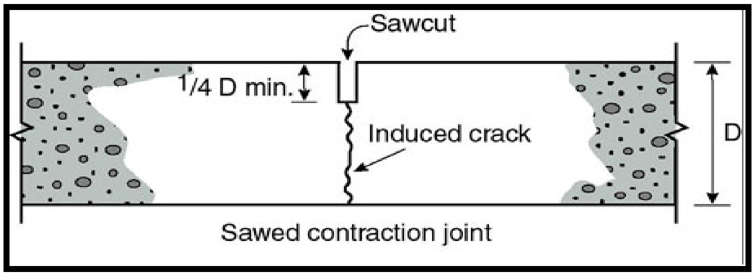


Figure 6-1. Sawed Contraction Joint

6.5.5 Setbacks From Slopes and Walls

The face of shallow foundations and slabs should be set back at least one third the height (H) of any slope from the face (‘daylight’) of that slope.

New structures may approach a permanent retaining wall at the north end of the site. Foundations should be located such that there is no new load placed on these walls.

The face of foundations adjacent to slopes must be located such that there is a minimum of 7 horizontal feet to daylight.

6.6 Capillary Break and Underslab Vapor Retarder

6.6.1 Capillary Break

NOVA recommends that the requirements for a capillary break (‘sand layer’) be determined in accordance with ACI Publication 302 “*Guide for Concrete Floor and Slab Construction*.” A "capillary break" may consist of a 4-inch thick layer of compacted, well-graded sand should be placed below the floor slab. This porous fill should be clean coarse sand or sound, durable gravel with not more than 5% 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 Retarder

Soil moisture vapor that penetrates ground-supported concrete slabs can result in damage to moisture-sensitive floors, some floor sealers, or sensitive equipment in direct contact with the floor. It is not the responsibility of the geotechnical consultant to provide recommendations for vapor retarders to address this concern. This responsibility usually falls to the Architect. Decisions regarding the appropriate vapor retarder are principally driven by the nature of the building space above the slab, floor coverings, anticipated penetrations, concerns for mold or soil gas, and a variety of other environmental, aesthetic, and materials factors known only to the Architect.

A variety of specialty polyethylene (polyolefin)-based vapor retarding products are available to retard moisture transmission into and through concrete slabs. This remainder of this section provides an overview of design and installation guidance, and considers the use of vapor retarders in the building construction in the San Diego area.

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

- ASTM E1745-97 (2009). Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs.
- ASTM E154-88 (2005). Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Walls, or as Ground Cover.
- ASTM E96-95 (2005). Standard Test Methods for Water Vapor Transmission of Materials.
- ASTM E1643-98 (2009). Standard Practice for Installation of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs.
- ACI 302.2R-06. Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials.

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

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

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

6.7 Control of Moisture Around Foundations

6.7.1 General

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

6.7.2 Erosion and Moisture Control During Construction

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

6.7.3 Design

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

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

6.8 Walls

6.8.1 Lateral Pressures

Lateral earth pressures to retaining walls are related to the type of backfill, drainage conditions, slope of the backfill surface, and the allowable rotation of the wall. It is expected that the garage walls will be unyielding, designed to resist 'at rest' soil loads. Table 6-5 provides recommendations for lateral soil for varying conditions of wall yield. Groundwater level will be well below the wall levels.

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. Surcharge loading should consider wall loads that may develop from adjacent roads and sidewalks. To account for such potential loads, a surcharge pressure of 75 psf can be applied uniformly over the wall to a depth of about 12 feet.

Table 6-5. Wall Lateral Loads from Soil

Condition	Equivalent Fluid Pressure (psf/foot)	
	Level Backfill	2:1 Backfill Sloping Upwards
Active	35	55
At Rest	55	80
Passive	350	350

6.8.2 Seismic Increment to Non-Yielding Garage Walls

The lateral seismic thrust acting on a non-yielding retaining walls greater than 6 feet in height should be estimated by the dynamic (seismic) thrust, ΔP_E. Dynamic thrust is approximated as:

$\Delta P_E = k_h H^2 \gamma$ 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 the unit weight of the backfill, about 120 lb/ft^3

The resultant dynamic thrust acts at a distance of 0.33H above the base of the wall.

6.8.3 Elevator Pits

The parking garage will include elevators. Elevators may require pits that extend below the lowest slab level. An elevator pit slab and related retaining wall footings will derive suitable support from the Unit 2 sandstones around them.

Design for the elevator pit walls should consider the circumstances and conditions described below.

1. Wall Yield. Proper function of elevator pits should not allow yielding of the elevator pit walls. As such, walls should be designed to resist ‘at rest’ lateral soil pressures and seismic pressures provided above, also allowing for any structural surcharge.

2. Construction. Design of the elevator pit walls should include consideration for surcharge conditions that will occur during and after construction.

6.9 Temporary Slopes

It is the sole responsibility of the Contractor to provide safe temporary slopes during construction. Special care should be given to temporary slopes excavated in the Unit 2 paralic deposits, where zones of loose, clean, and uncemented sand are likely to be encountered and may readily slough. Per OSHA guidelines for Type C soils, slopes created during temporary trenching for utilities should not be exposed for more than 24 hours. The maximum slope should not exceed 1.5:1 (H:V), and where possible, should be laid back on the order of 3:1. All excavations with vertical sides should be shielded or supported to a height at least 18 inches above the top of the vertical side.

7.0 TEMPORARY SHORING

7.1 General

It is the sole responsibility of the Contractor to provide an excavation that is safe, with deflections that do not damage nearby structures or utilities. To this end, the Contractor should retain a qualified Shoring Engineer for design of temporary shoring for larger excavations. The Shoring Engineer should be solely responsible for the design, utilizing the indications of subsurface conditions provided in the geotechnical reporting.

The following subsections provide guidance for the Owner, the Design Team, and the Shoring Engineer in development of these designs.

7.2 Design Conditions for Soldier Pile Wall

7.2.1 General

The Owner and the Design Team should consider that design for braced/retained excavation may address two broad conditions of wall loading as described below.

1. Condition 1, 'At Rest.' Design for the retaining wall should consider the use of 'at-rest' soil pressures at locations where wall deflections may affect potentially damaging settlement to utilities or structures. In such instances, design for walls should be designed to resist 'at rest' lateral soil pressures.

2. Condition 2, 'Active.' Design for the walls that are not located near sensitive structures or utilities should consider design to resist 'active' earth pressures. Based on review of the site area, it appears that this condition may be more appropriate for temporary walls that would be used for the planned development.

7.2.2 Method of Temporary Shoring

Designed to resist the Condition 1 'at rest' (i.e., 'K_o') earth pressures employs a rectangular wall pressure distribution that is more conservative than the Condition 2 loading. Figure 7-1 (following page) provides this load distribution, reproducing published guidance of relevance to this design circumstance.

Design for the 'at rest' wall pressure diagram depicted in Figure 7-1 using the parameters would yield:

$P\ (psf) = 0.45\ (K_o)\ (\gamma)\ (H)$ where,
$K_o = 1 - \sin\phi$ $\phi = 35°$, and $K_o = (1 - 0.57) = 0.43$
$\gamma = 120\ lb/ft^3$
H = wall height

$P = 0.45 \times 0.43 \times 120 \times H = 23H$

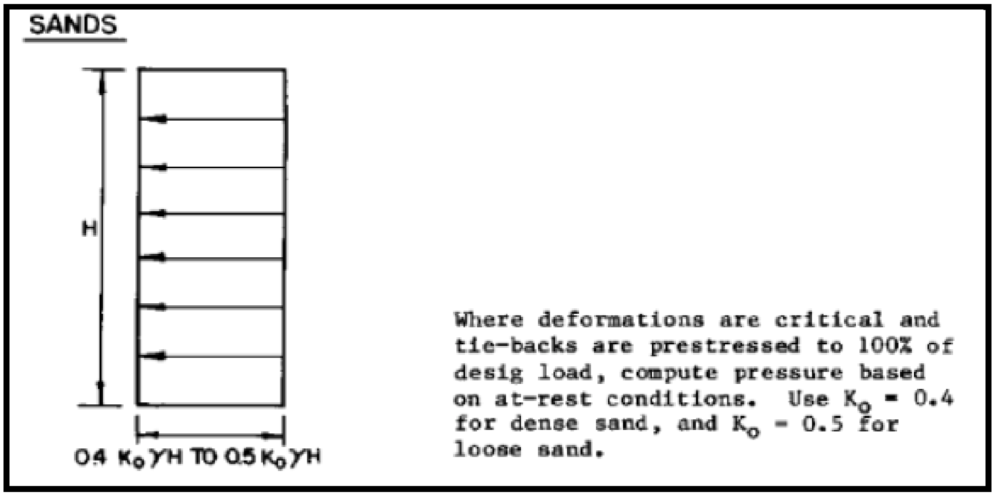


Figure 7-1. 'At Rest' Wall Pressure Distribution in Sands
(source: NAVFAC 1986)

The Shoring Engineer should also consider additional lateral pressure due to the surcharging effects of adjacent structures or traffic loads should be considered by the Shoring Engineer, as appropriate. These loads will act as a surcharge to the temporary wall.

7.2.3 Condition 2, 'Active'

Based on review of aerial photography of the area, it is the judgment of NOVA that the site is favorable for design for less conservative wall pressures than those driven by Condition 1. That is, there is no indication that the project bounds an area where wall deflections will immediately threaten structures or utilities.

As such, NOVA recommends that wall design be completed using active earth pressures as described by the trapezoidal active earth pressure distribution of Figure 7-2(a) (following page). The magnitude of the maximum trapezoidal pressure may be calculated as:

$P \text{ (psf)} = 0.65\,(K_a)\,(\gamma)\,(H)$ where,

$K_a = (1 - \sin\phi) / (1 + \sin\phi)$ $\phi = 35°$, $K_a = 0.27$

$\gamma = 120 \text{ lb/ft}^3$

H = wall height

For a variety of assumptions regarding γ, ϕ, and K_a, NOVA estimates that the maximum magnitude of lateral pressure for a tied back soldier beam and lagging wall system will normalize to be in the range 19H to 24H, where 'H' is the height of the wall in feet.

NOVA recommends employing the trapezoidal distribution of Figure 7-2(a), using 21H for determination of the maximum wall pressure.

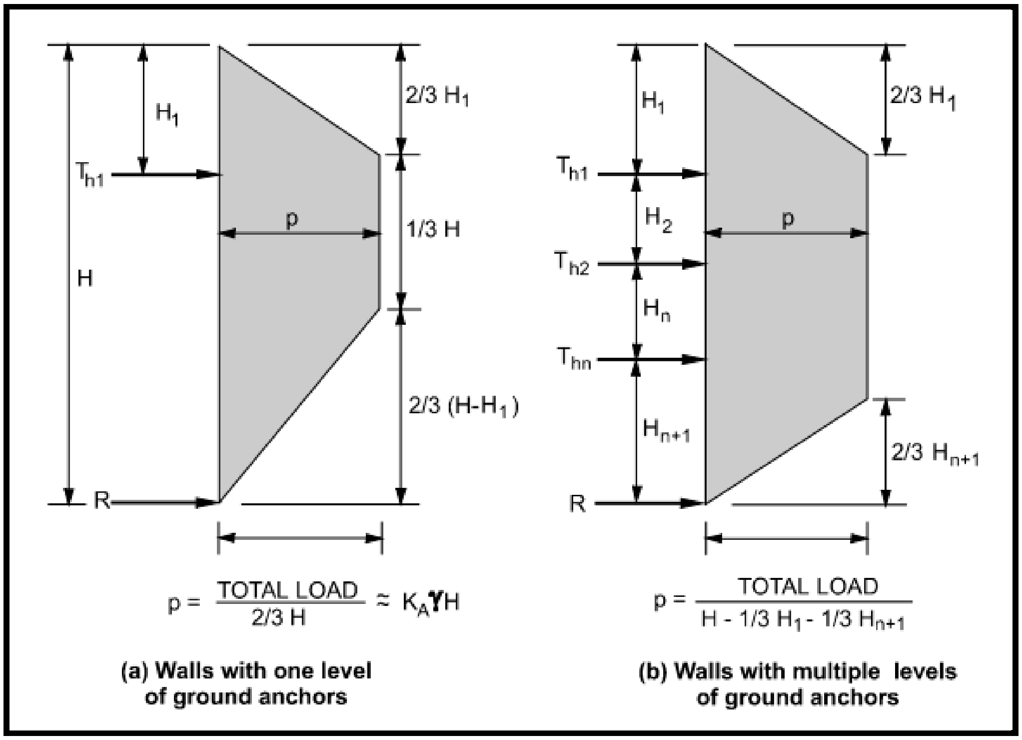


Figure 7-2. 'Active' Lateral Earth Pressures
(source: FHWA 1999)

It should be noted that the pressure distribution of Figure 7-2(a) are empirical, derived from experience. As may be seen by review of Figure 7-2(a), the recommendation for this pressure distribution follows guidance provided by FHWA 1999. It should be understood that other empirical pressure distributions may be preferred by others. However, it is NOVA's experience that the pressure distribution of Figure 6 works well to predict wall loads/anchor loads in this area.

7.2.4 Passive Resistance to Soldier Piles

It is expected that soldier beams will be set in pre-drilled holes and backfilled with lean concrete or a sand cement slurry with a compressive strength of at least 700 psf. Passive resistance to embedment of a temporary wall may be calculated using an 'equivalent fluid wall pressure' distribution, where the maximum equivalent fluid pressure (P) may be calculated as:

$P\ (\text{psf}) = (K_p)\ (\gamma)\ (D)$ where,

$K_p = (1 + \sin\phi) / (1 - \sin\phi)$ $\phi = 35°$, $K_p = 2.6$

$\gamma = 120\ \text{lb/ft}^3$

D = depth of wall embedment

$P = 3.6 \times 120 \times D = 440\ D$ (ultimate)

7.2.5 Rakers

If rakers (inclined struts) are employed for the taller portions of the excavation, these units will gain lateral resistance from either (i) temporary foundations or (ii) the central part of the basement level slab. In the latter case, the excavation would first be carried in full depth at its center, so that the basement level slab could be placed. Thereafter, the slab could provide resistance to rakers loads.

If temporary foundations are utilized to support the rakers inclined at 40° or steeper, mass concrete heel blocks, embedded a minimum of 3 feet below surrounding grade will provide ultimate passive resistance of 500 psf over the face of the heel block. Alternatively, a steel section may be embedded in a predrilled hole to provide lateral resistance similar to that described above for soldier piles.

7.3 Tieback Anchors

7.3.1 Failure Wedge

Design should assume that the failure wedge adjacent to the shoring is defined by a plane drawn at 29° from the vertical from the toe of the wall. Figure 7-3 depicts this wedge graphically.

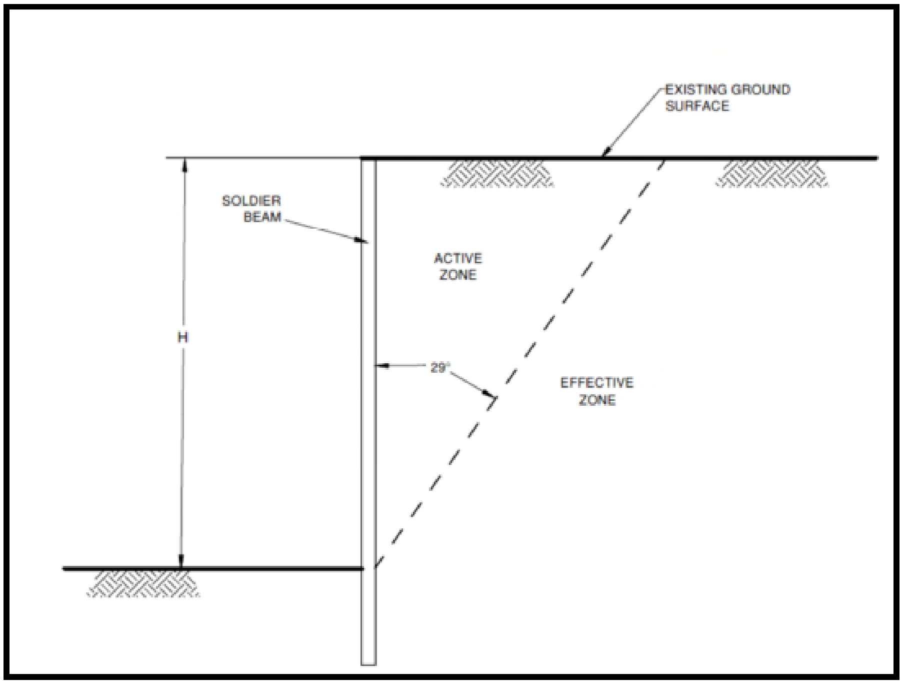


Figure 7-3. Recommended Effective Zone for Tieback Anchors

Tieback anchors should extend at least 20 feet beyond the failure wedge (i.e., the “bonded” zone) depicted in Figure 7-3. The intent of this provision is to provide global stability for the shored wall. The bonded length should commence at least 5 feet beyond the failure wedge).

7.3.2 Anchor Installation

The anchors may be installed at angles of 15° to 35° below the horizontal. The anchors should be filled with concrete placed by pumping from the tip of the anchor to the failure wedge (i.e., over the bonded zone). The portion of the anchor tendons outside of the bonded length should be sleeved in plastic (i.e., over the unbonded zone). If the anchor tendons are sleeved, it is acceptable to concrete the entire length of the anchor.

7.3.3 Bond Stress

It is usual that tiebacks are installed using 'post-grouted' construction. The Shoring Engineer should be solely responsible for determination of allowable bond stresses on pressure-concreted ('post-grouted') anchors. Based upon experience at similar sites, NOVA expects that an allowable bond stress of at least 3,500 psf should be achievable.

Only the resistance developed beyond the failure wedge should be used in resisting lateral loads. If the anchors are spaced at least 6 feet on center, no reduction in the capacity of the anchors need be considered due to group action. In no event should the anchors extend less than the minimum length beyond the potential failure wedge as given above.

As a tie-back anchor system is intended for temporary use, provisions should be made in the design to de-tension and abandon the tie-backs when the basement walls are able to support the lateral loads.

7.3.4 Performance Testing

Wall design should provide for (i) performance testing, (ii) proof testing, and (iii) creep testing of wall anchors. In this regard, it is recommended that guidance provided in FHWA 1999 be utilized. Guidance for proof testing for all anchors provides for loading to a single cycle and load hold at the test load. The guidance provides that loading be applied pre-provided in load increments of 0.25DL, 0.50DL, 1.00DL, 1.20DL and 1.33DL (the 'test load').

All of the production anchors should be tested to at least 130% of the design load; the total deflection during the tests should not exceed 1.5-inches. The rate of creep under the 130% test should not exceed 0.1-inch over a 15-minute period for the anchor to be approved for the design loading.

7.4 Miscellaneous Wall Design Considerations

End bearing for soldier piles will be negligible and should not be considered. As noted previously, it is expected that soldier beams will be set in pre-drilled holes and backfilled with lean concrete or a sand cement slurry with a compressive strength of at least 700 psf. The soil-pile bond will be on the order of 400 psf or greater.

The coefficient of friction (μ) between the wall and surrounding soils is μ = 0.35.

7.5 Construction

7.5.1 General

NOVA expects that a soldier beam and lagging wall can be designed with a single level of tiebacks. A wall retaining a 25-foot excavation with a tieback set at about 8 feet depth will develop individual tieback loads on the order of 130 kips.

Walls will be constructed by first setting the soldier beams. Thereafter, the pace of the excavation will be limited by the establishment of lagging. Excavation should not be advanced deeper than about 3 feet below the bottom of the lagging at any time. These gaps of up to 3 feet should only be allowed to stand for short periods of time in order to decrease the potential for sloughing/caving. Backfilling should be conducted when necessary between the back of the lagging and excavation sidewalls to reduce any sloughing in this zone.

The Contractor should also recognize that temporary wall construction at a nearby site with similar subsurface conditions was affected by sloughing sands. According to a conversation NOVA had with the general contractor constructing that site, the site proforma was calculated assuming 5-foot excavations for shoring; however, when the lower zone of friable sandstone was encountered, excavations were reduced from 5 feet to 1-foot in order to mitigate caving.

7.5.2 Expected Wall Movements

Actual wall movement and related ground settlement are related to a variety of factors, most significantly (i) subsurface conditions, including effective dewatering; and, (ii) workmanship in wall construction.

The formational sandstones are favorable for sound wall construction. The Geotechnical Engineer-of-Record (GEOR) should coordinate with the Shoring Engineer to ensure that good workmanship prevails throughout wall construction. The combination of workmanship and favorable subsurface conditions will result in good wall performance. Additionally, ground and wall movement monitoring should be employed to detect any unusual wall movement before the condition becomes problematic.

Design for a Condition 1 wall will limit wall movement, though the ‘at rest’ wall condition will not eliminate all wall movement. Because of the reliance on a wide variety of parameters, including workmanship, it is difficult to rigorously predict wall performance during the design stage. Expectations in this regard are primarily empirical. Figure 7-4 (following page) provides a published summary of project experiences.

A wall designed for Condition 1 might expect to have a horizontal movement (δ_{Hm}) of about 0.2% of the wall height (H). Assuming H ~25 feet, the design might anticipate δ_{Hm} ~ 0.5-inch.

Walls designed for Condition 2 will likely limit deflection of the top of the wall to about 1-inch or less. This wall movement will limit ground settlement immediately behind the shoring system to a similar amount or less. This movement should be imperceptible beyond a distance of about 20 feet from the wall.

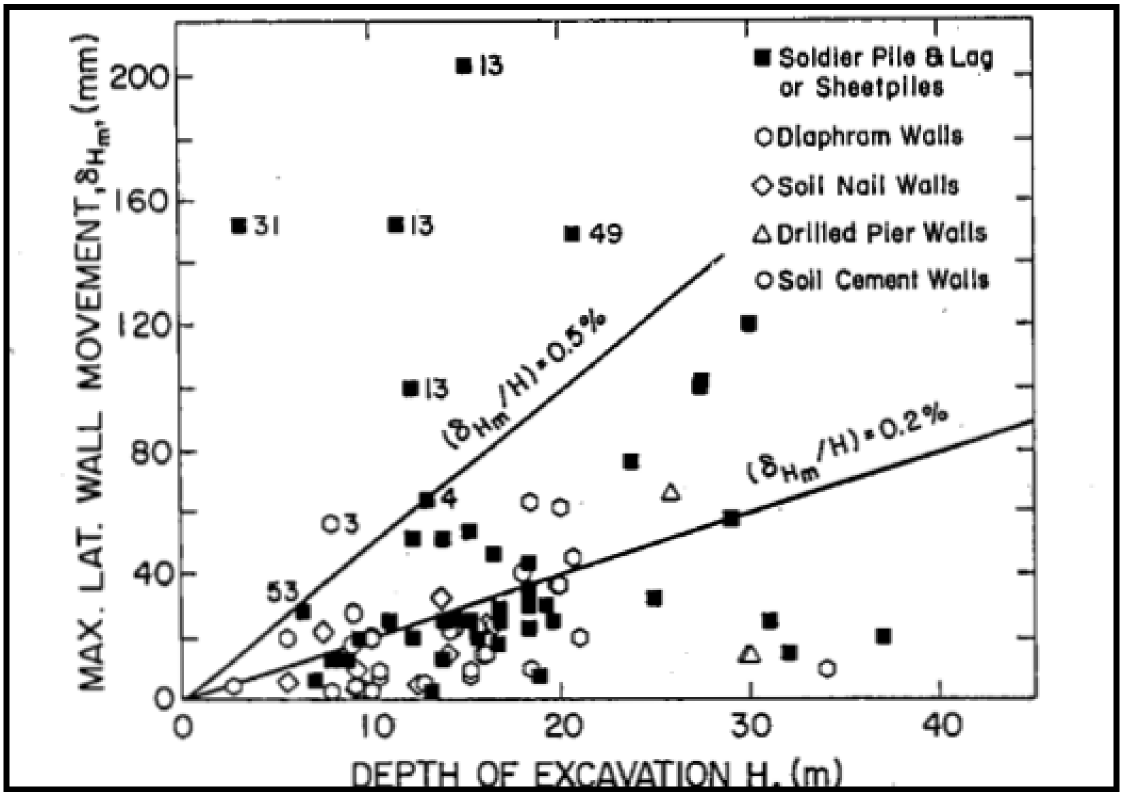


Figure 7-4. Observed Maximum Lateral Movements for In Situ Walls in Stiff Clays, Residual Soils and Sands

(source: Clough and O'Rourke 1990)

8.0 STORMWATER INFILTRATION

8.1 Overview

Based upon the indications of the field exploration and laboratory testing reported herein, NOVA has evaluated the site based on guidance contained in the City of Encinitas BMP Design Manual, (hereafter, 'the BMP Manual').

Section 3.4 provides a description of the field work undertaken to complete the testing. Figure 3-1 depicts the location 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. This section provides NOVA's assessment of the feasibility of stormwater infiltration BMPs utilizing the information developed by the field exploration described in Section 3.4, as well as other elements of the site assessment.

8.2 Infiltration Rate

The percolation rate of a soil profile is not the same as its infiltration rate ('I'). Therefore, when percolation rates are established by field testing, the measured/calculated field percolation rate is converted to an estimated infiltration rate utilizing the Porchet Method in accordance with guidance contained in the BMP Manual.

Table 8-1 provides a summary of the infiltration rate determined by the percolation testing. The measured infiltration rates range from I = 1.51 to I = 1.74 inches per hour using a preliminary factor of safety (F) of F = 2.

Table 8-1. Infiltration Rate Determined by Percolation Testing

Boring	Approximate Ground Elev. (feet, msl)	Depth of Test (feet)	Approximate Test Elev. (feet, msl)	Percolation Rate (inches/hour)	Infiltration Rate (inches/hour)	Design Infiltration Rate (in/hour, F=2*)
P-1	+55	+50	5.0	58.32	3.02	1.51
P-2	+76	+71	5.0	77.04	3.49	1.74

Note: 'F' indicates 'Factor of Safety'

8.3 Review of Geotechnical Feasibility Criteria

8.3.1 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 and Coastal Bluff 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.

8.3.2 Soil and Geologic Conditions

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

1. Unit 1, Fill (Qaf). Fill is assumed to underlie the structures on the property, and was encountered within Boring B-6 and Boring B-3, extending a depth of approximately 2 to 5 feet. The fill is an orange brown silty sand of medium dense consistency.

2. Unit 2, Old Paralic Deposits (Qop6-7). The entire site is underlain by marine terrace deposits. The upper 15 to 20 feet of this unit is slightly cemented, with the consistency of a dense silty fine to medium sand. At depth, cementation weakens as the unit grades down to dense, poorly-graded (‘well-sorted’) sandstone with only a trace amount of fines present.

8.3.3 Settlement and Volume Change

The Unit 1 and 2 soils are not prone to substantial swelling upon wetting and shrinkage upon drying. Introduction of water to this unit will likely not create damaging foundation movement.

8.3.4 Slope Stability and Coastal Bluff Stability

Infiltration of water has the potential to result in an increased risk of slope failure of existing slopes and coastal bluff zones. As such, BMPs are not suitable for any location on site.

8.3.5 Utilities

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

8.3.6 Groundwater Mounding

In consideration of the measured percolation rates, it is unlikely that groundwater mounding will occur if stormwater infiltration is attempted.

8.3.7 Retaining Walls and Foundations

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

8.3.8 Other Factors

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

8.4 Suitability of the Site for Stormwater Infiltration

It is NOVA's judgment that the site is not suitable for development of stormwater infiltration BMPs. This judgment is based upon consideration of the variety of factors detailed above; most significantly the risk of coastal bluff failure.

Appendix E provides completed forms related to stormwater infiltration.

9.0 PAVEMENTS

9.1 General

9.1.1 Control of Moisture

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 recommendations 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;
- compact utility trenches for landscaped areas to the same criteria as the pavement subgrade;
- 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 curbs and gutters; 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 12-inches below the base of the curb).

9.1.2 Planning for Preventive Maintenance

Preventative maintenance should be planned and provided for. Preventative maintenance activities are intended to slow the rate of pavement deterioration and to preserve the pavement investment. Preventative maintenance consists of both localized maintenance (e.g. crack sealing and patching) and global maintenance (e.g. surface sealing). Preventative maintenance is usually the first priority when implementing a planned pavement maintenance program and provides the highest return on investment for pavements.

9.2 Subgrade Preparation

9.2.1 Rough Grading

Grading for paved areas should be as described in Section 6.4, completely removing the Unit 1 undocumented fill, and replacing the soil to design grade with select fill. Any replacement filling should be done in lifts (i) not to exceed 10-inches thickness or (ii) the ability of the compaction equipment employed to densified through a complete lift, whichever is less.

Prior to constructing pavements, inspection should ensure that at least the upper 12” of subgrade soils are either (i) Unit 2 soil; or (ii) sandy Unit 1 fill densified to at least 95% relative compaction after ASTM D 1557, following moisture conditioning to at least 2% above the optimum moisture content.

9.2.2 Proof-Rolling

After the completion of compaction/densification, or excavation to the design subgrade level, areas to receive pavements should be proof-rolled. A loaded dump truck or similar should be used to aid in identifying localized soft or unsuitable material.

Any soft or unsuitable materials encountered during this proof-rolling should be removed, replaced with an approved backfill, and compacted. The Geotechnical Engineer can provide alternative options such as using geogrid and/or geotextile to stabilize the subgrade at the time of construction, if necessary.

9.2.3 Moisture Control

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.

9.2.4 Surveillance

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

9.3 Flexible Pavements

The structural design of flexible pavement depends primarily on anticipated traffic conditions, subgrade soils, and construction materials. Table 9-1 (following page) provides preliminary flexible pavement sections using an R-value of 35. This R-value was indicated by laboratory testing described in Section 3.

9.4 Rigid Pavements

9.4.1 General

Concrete pavement sections should be developed in the same manner as undertaken for all other slabs and pavements - removal of the Unit 1 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 a base course of 6 inches and a properly prepared subgrade support a wide range of traffic indices.

Where rigid pavements are used, the concrete should be obtained from an approved mix design with the minimum properties of Table 9-2 (following page).

Table 9-1. Preliminary Flexible Pavement Sections, R = 35

Area	Traffic Index	Asphalt Thickness (inches)	Base Thickness (inches)
Passenger Car Driveways	5.0	4	6
		4	6
Heavy Duty Driveways	6.0	3	7.5
		4	6

1. The above sections assume properly prepared subgrade consisting of at least 12-inches of subgrade compacted to a minimum of 95% relative compaction after ASTM D1557, with EI <30.

2. The aggregate base materials should be placed at a minimum of 95% relative compaction after ASTM D1557.

Table 9-2. Recommended Concrete Requirements for Pavements

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

9.4.2 Jointing and Reinforcement

Longitudinal and transverse joints should be provided as needed in concrete pavements for expansion/contraction and isolation. Sawed joints should be cut within 24-hours of concrete placement, and should be a minimum of 25% of slab thickness plus 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.

9.5 Grasscrete Pavements and Interlocking Pavers

9.5.1 Location

Vehicular grasscrete pavements are proposed at the northwest end of the site to support the fire lane. Figure 9-1 locates this feature. Interlocking pavers are proposed throughout the driveways of the site.

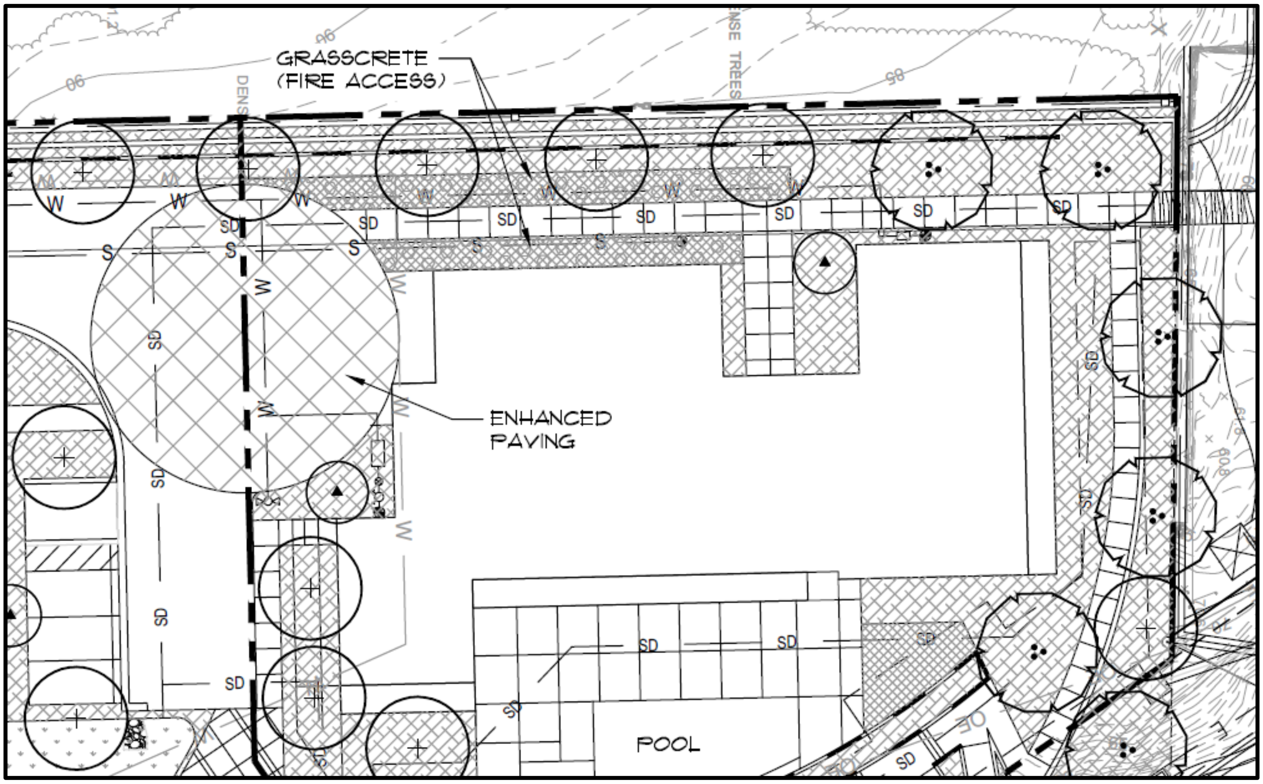


Figure 9-1. Location of Proposed Grasscrete Pavements
(source: SDA 2020)

9.5.2 General

The geotechnical recommendations for pavers provided herein have been developed in general conformance with (i) guidelines of the Interlocking Concrete Pavement Institute (ICPI), Technical Specification No. 4, February 2020; and, (ii) details for Grasspave2 by Invisible Structures Inc., January 2018.

Grasscrete pavement sections and interlocking concrete pavers should be developed in the same manner as undertaken for all other slabs and pavements - removal of the Unit 1 fill and replacement of that material in an engineered manner as described in Section 9.2.

9.5.3 Grasscrete Pavements

The structural design of porous grass pavers depends primarily on anticipated traffic conditions, subgrade soils, and construction materials.

Table 9-3 provides preliminary porous pavement sections for design purposes using an R-value of 35. An R-value test of the subgrade soils can be performed after the grading operations are complete in order to provide a final pavement section.

Table 9-3. Preliminary Recommendations for Porous Grass Pavers, R = 35

Area	Estimated Subgrade R-Value	Ring Thickness (in)	Base Course Thickness (in)
Fire Lanes	35	1*	18

Notes: (*) Assumed ring height

The grasscrete pavements should be underlain with at least 18-inches of Caltrans Class II base, moisture conditioned to slightly above the optimum moisture content and densified to at least 95 percent of the maximum dry density determined by ASTM D1557 (the 'modified Proctor').

9.5.4 Interlocking Concrete Pavers

Concrete paver units should be at least 80 millimeters (3 ⅛-inches) thick for vehicular concrete pavers. Interlocking concrete pavement can be constructed by placing the concrete paver units over a 1-inch bedding sand layer generally conforming to ASTM C-33 sand.

Bedding and Joint Sand Gradation

Table 9-4 summarizes bedding sand gradation recommendations and recommended joint sand gradation. The joint sand should comply with ASTM C144 with a maximum 100 percent passing the No. 16 sieves and no more than 5 percent passing the No. 200 sieve.

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

Table 9-4. Gradation of Sand for Paver Systems

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

Base and Subgrade

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

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

9.5.5 Control of Infiltration

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

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

Figure 9-2 depicts a design to control infiltrating surface water that reflects the above recommendations.

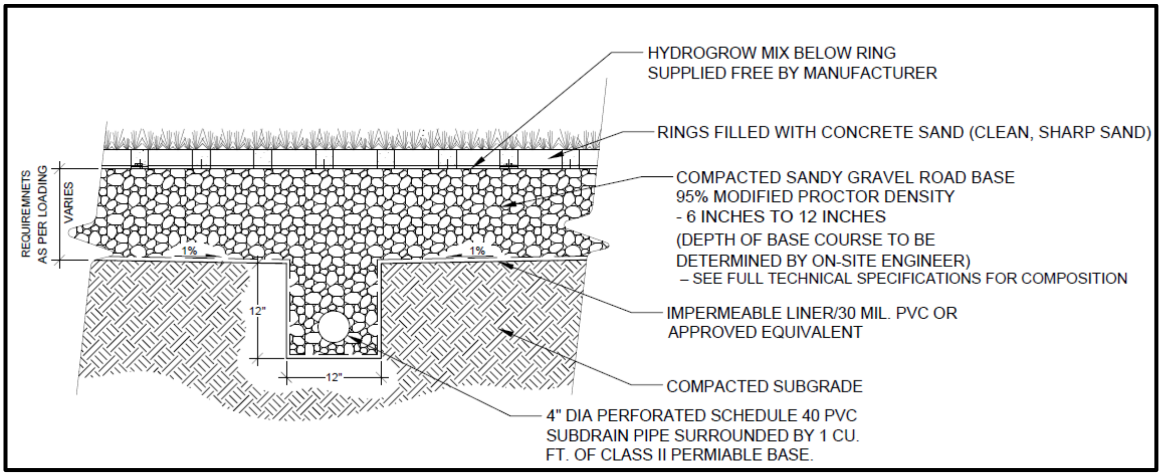


Figure 9-2. Design to Control Infiltration
(source adapted from IS 2018)

9.5.6 Installation

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

9.5.7 Edge Restraint

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

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

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

Based on the criteria presented above, concrete pavement should consist of a minimum of 7 inches of PCC placed over a 6-inch base course. The base and subgrade soil should be compacted to a dry density of at least 95 percent of the laboratory maximum dry density near to slightly above optimum moisture content. This pavement section is based on a minimum concrete compressive strength of approximately 3,250 psi (pounds per square inch).

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

9.5.8 Maintenance

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

10.0 GEOTECHNICAL REVIEW, OBSERVATION, AND TESTING

10.1 Overview

As is discussed in Section 1, the recommendations contained in this report are based upon a limited number of borings and reliance, tempered with judgment, upon the 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.

10.2 Design Phase Review

NOVA should be retained to provide review of final grading and foundation plans. This review is provided for in NOVA's proposal for this work.

10.3 Construction Observation and Testing

10.3.1 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 soil densification/compaction during grading.
- 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, Contractor, and/or Construction Manager to determine any additional inspection items required by the Architect/Engineer or the governing jurisdiction.

10.3.2 Continuous Soils Special Inspection

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

- Over-excavation for remedial grading, including scarification and re-compaction.
- Fill placement and compaction.
- Pavement subgrade preparation and base course compaction.

10.3.3 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.
- Building pad moisture conditioning.

10.3.4 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 compaction equipment appropriate to the full lift thickness of the type of soil being compacted. Successful compaction at this site will require proper moisture conditioning and the use of vibratory compaction equipment. Reliance on construction traffic (for example, loaders or dump trucks) to achieve compaction will not be approved.

11.0 REFERENCES

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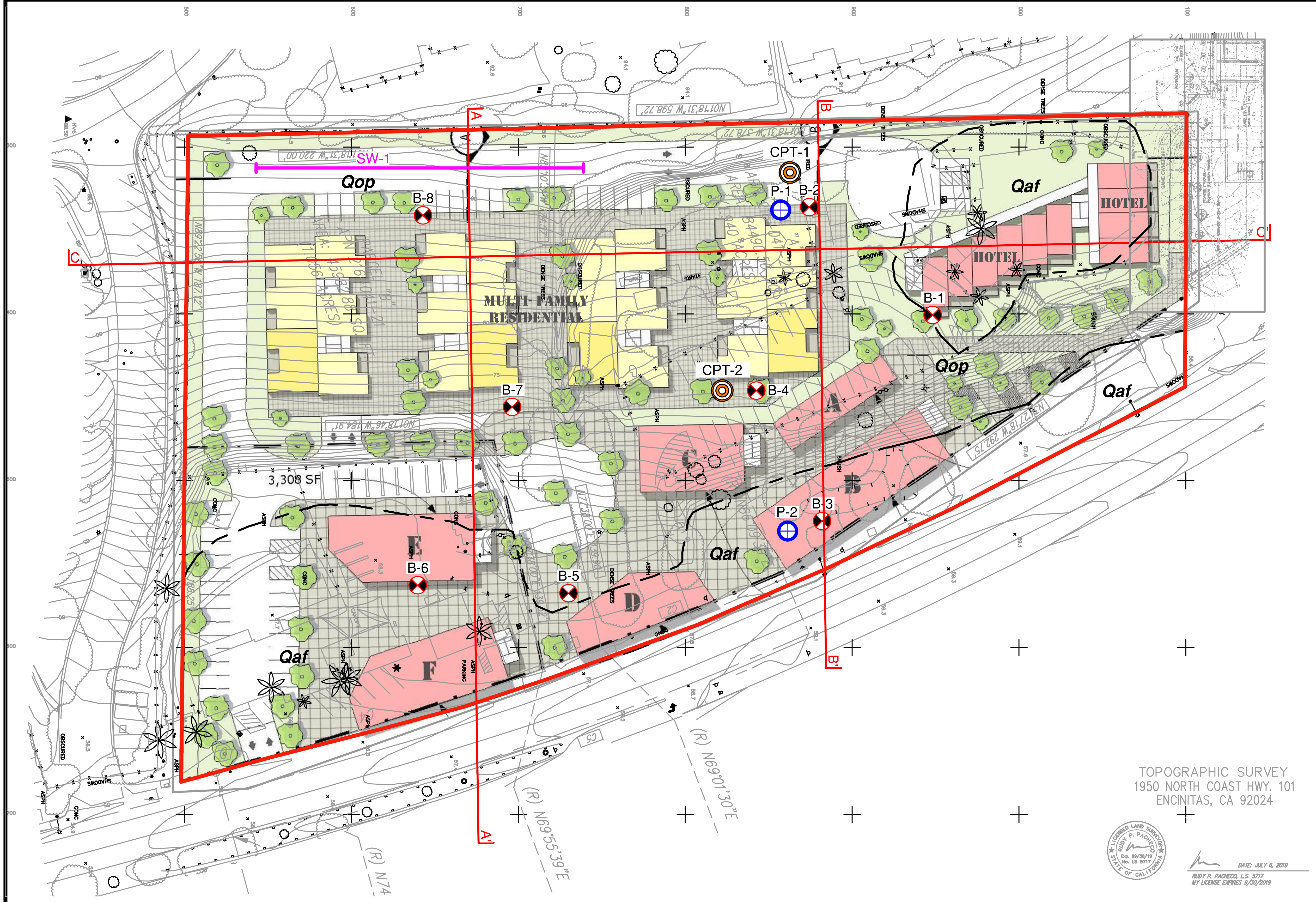
PLATES



KEY TO SYMBOLS

Qaf	FILL
Qop	OLD PARALIC DEPOSITS, UNDIVIDED
B-7	LOCATION OF GEOTECHNICAL BORING
P-2	LOCATION OF PERCOLATION TEST BORING
CPT-2	LOCATION OF CPT
C C'	CROSS SECTION
〰	LIMITS OF GRADING
╌	APPROXIMATE GEOLOGIC CONTACT
SW-1	SEISMIC SURVEY LINE

ACCULINE SURVEY, INC.
1919 GRAND AVE. SUITE 1G,
SAN DIEGO, CA 92109.
(858) 483-6665 FAX(858) 483-6056

DATE OF PLOT 07/06/2019
FILE NO. 2037-TOPO-19

RICARDO JORDAN
No. 68436
Exp. 9/30/19
CIVIL

CITY OF ENCINITAS - DEVELOPMENT SERVICES DEPARTMENT
ENCINITAS BEACH RESORT
APN 216-041-26
PLANNING CASE NO: TPM-89-014

DRAWING NO.
6394-G
SHEET 19A OF 41

NOVA
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA

858-292-7575
858-292-7570 (FAX)

WWW.USA-NOVA.COM

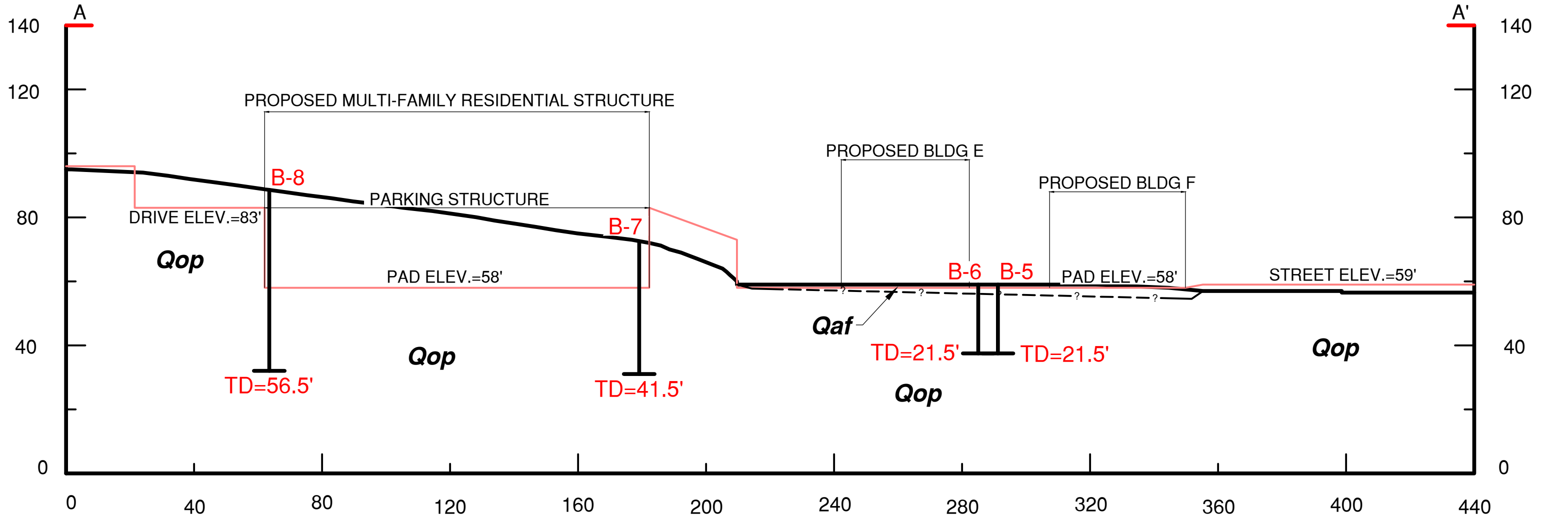
LEUCADIA MIXED-USE
1900-1950 NORTH COAST HWY 101
ENCINITAS, CA

PROJECT NO:	2019189
DATE:	APR 2021
DRAWN BY:	DTJ
REVIEWED BY:	MS

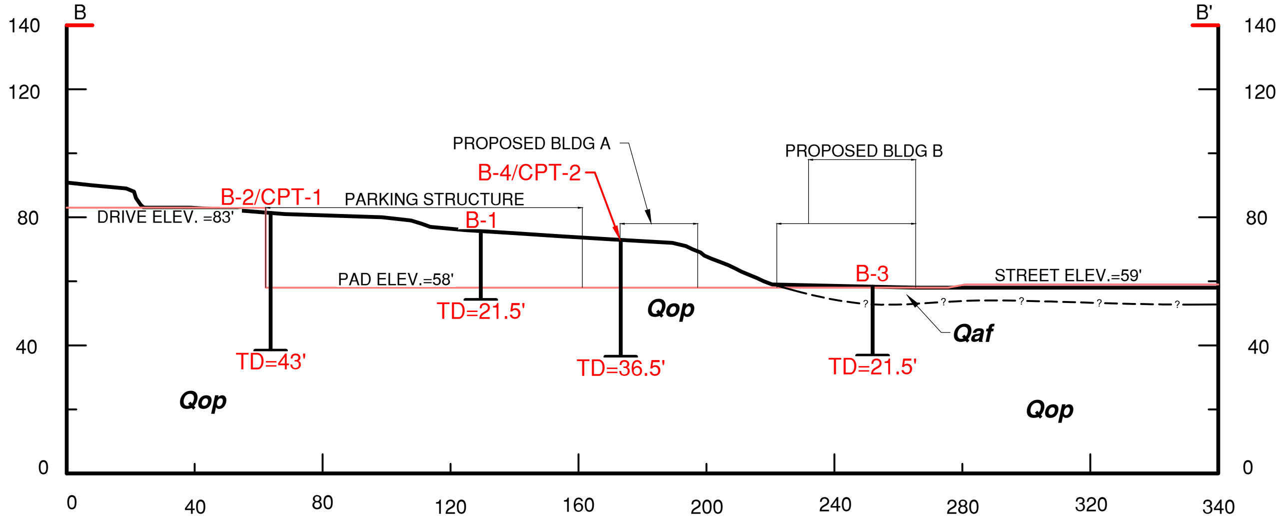
SUBSURFACE INVESTIGATION MAP

0 60' 120'

PLATE 1



SECTION AA'



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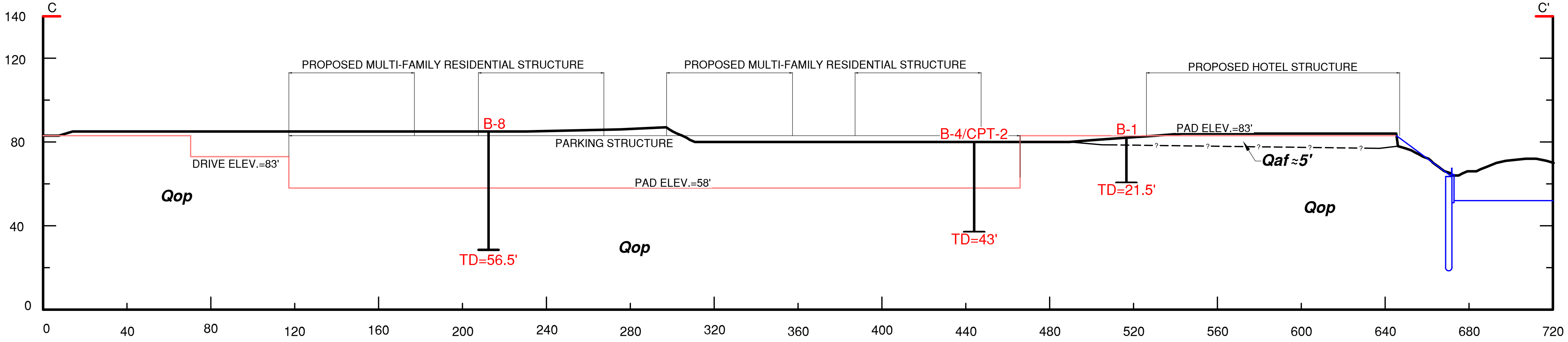


SECTION CC'

KEY TO SYMBOLS

Qaf	FILL
Qop	OLD PARALIC DEPOSITS, UNDIVIDED

EXISTING GRADE

GEOLOGIC CONTACT
(QUERIED WHERE INFERRED)

PROPOSED FINISH GRADE
(SDA 2019)

PROPOSED FINISH GRADE AND
RETAINING WALL/SOLDER BEAM
(HALE 2019)

NOVA
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA

858-292-7575
858-292-7570 (FAX)

WWW.USA-NOVA.COM

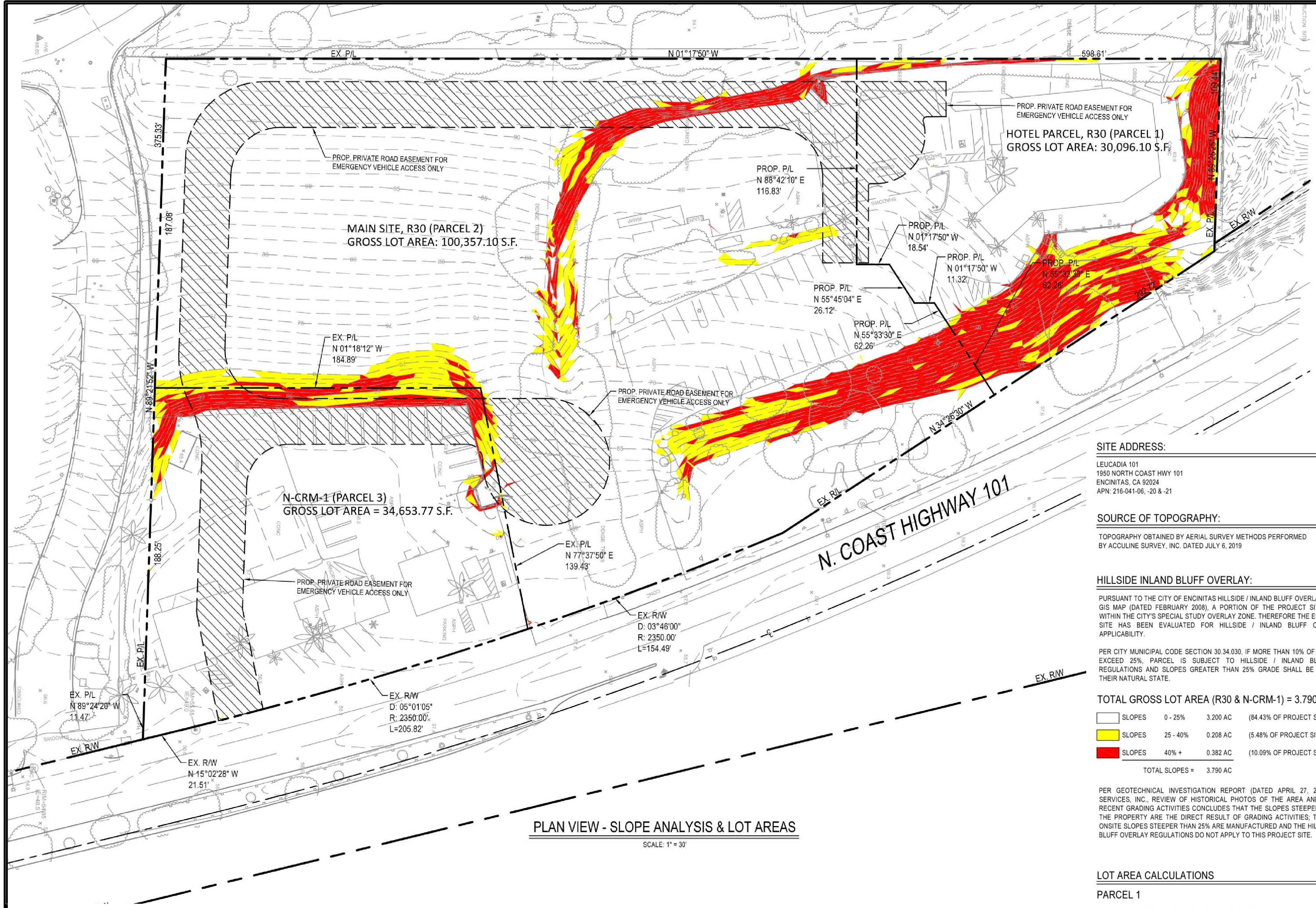
LEUCADIA MIXED-USE
1900-1950 NORTH COAST HWY 101
ENCINITAS, CA

PROJECT NO: 2019189
DATE: APR 2021
DRAWN BY: DTJ
REVIEWED BY: MS

SITE SECTIONS

0 60' 120'

PLATE 2



PLAN VIEW - SLOPE ANALYSIS & LOT AREAS

SCALE: 1" = 30'

SITE ADDRESS:

LEUCADIA 101
1950 NORTH COAST HWY 101
ENCINITAS, CA 92024
APN: 216-041-06, -20 & -21

SOURCE OF TOPOGRAPHY:

TOPOGRAPHY OBTAINED BY AERIAL SURVEY METHODS PERFORMED BY ACCULINE SURVEY, INC. DATED JULY 6, 2019

HILLSIDE INLAND BLUFF OVERLAY:

PURSUANT TO THE CITY OF ENCINITAS HILLSIDE / INLAND BLUFF OVERLAY ZONE (H/IBO) GIS MAP (DATED FEBRUARY 2008), A PORTION OF THE PROJECT SITE IS LOCATED WITHIN THE CITY'S SPECIAL STUDY OVERLAY ZONE. THEREFORE THE ENTIRE PROJECT SITE HAS BEEN EVALUATED FOR HILLSIDE / INLAND BLUFF OVERLAY ZONE APPLICABILITY.

PER CITY MUNICIPAL CODE SECTION 30.34.030, IF MORE THAN 10% OF PARCEL SLOPE EXCEED 25%, PARCEL IS SUBJECT TO HILLSIDE / INLAND BLUFF OVERLAY REGULATIONS AND SLOPES GREATER THAN 25% GRADE SHALL BE PRESERVED IN THEIR NATURAL STATE.

TOTAL GROSS LOT AREA (R30 & N-CRM-1) = 3.790 AC

☐	SLOPES	0 - 25%	3.200 AC	(84.43% OF PROJECT SITE)
🟨	SLOPES	25 - 40%	0.208 AC	(5.48% OF PROJECT SITE)
🟥	SLOPES	40% +	0.382 AC	(10.09% OF PROJECT SITE)
	TOTAL SLOPES =		3.790 AC	

PER GEOTECHNICAL INVESTIGATION REPORT (DATED APRIL 27, 2020) BY NOVA SERVICES, INC., REVIEW OF HISTORICAL PHOTOS OF THE AREA AND ANALYSIS OF RECENT GRADING ACTIVITIES CONCLUDES THAT THE SLOPES STEEPER THAN 25% ON THE PROPERTY ARE THE DIRECT RESULT OF GRADING ACTIVITIES; THEREFORE ALL ONSITE SLOPES STEEPER THAN 25% ARE MANUFACTURED AND THE HILLSIDE / INLAND BLUFF OVERLAY REGULATIONS DO NOT APPLY TO THIS PROJECT SITE.

LOT AREA CALCULATIONS

PARCEL 1

▨	GROSS LOT AREA (PARCEL 1)	= 30,096 SF
	LESS PRIVATE ROAD EASEMENT*	= (0) SF
	NET LOT AREA (PARCEL 1)	= 30,096 SF

PARCEL 2

▨	GROSS LOT AREA (PARCEL 2)	= 100,357 SF
	LESS PRIVATE ROAD EASEMENT*	= (0) SF
	NET LOT AREA (PARCEL 2)	= 100,357 SF

PARCEL 3

▨	GROSS LOT AREA (PARCEL 3)	= 34,654 SF
	LESS PRIVATE ROAD EASEMENT*	= (0) SF
	NET LOT AREA (PARCEL 3)	= 34,654 SF

*PRIVATE ROAD EASEMENT FOR EMERGENCY VEHICLE ACCESS PURPOSES DOES NOT SUBTRACT FROM GROSS LOT AREA

NOVA
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LEUCADIA MIXED-USE
1900-1950 NORTH COAST HWY 101
ENCINITAS, CA

PROJECT NO:	2019189
DATE:	APR 2021
DRAWN BY:	DTJ
REVIEWED BY:	MS

PLSA SLOPE ANALYSIS

0 60' 120'

PLATE 3

APPENDIX A
USE OF THE GEOTECHNICAL REPORT

Important Information About Your

Geotechnical Engineering Report

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

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

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

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

Read the Full Report

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

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

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

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

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

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

Rely, on Your ASFE-Member Geotechncial 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.

ASFE
The Best People on Earth

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

APPENDIX B
LOGS OF BORINGS

BORING LOG B-1

DATE EXCAVATED: SEPTEMBER 17, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 76 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; MEDIUM BROWN, DAMP, LOOSE, FINE TO MEDIUM GRAINED, TRACE ROOTS*	MD	
	▨		⧄	SM	38	**OLD PARALIC DEPOSITS (Qop):** *SILTY SANDSTONE; MEDIUM BROWN, DAMP, DENSE, FINE TO MEDIUM GRAINED*		
5	▨		■		45	*ORANGE BROWN, MEDIUM DENSE*		*4.6% 124.9pcf*
	▨		⧄		16			
10	▨	☒	■		65	*ORANGE BROWN WITH LIGHT BROWN MOTTLING, DENSE*		
15	▨		⧄		19	*ORANGE GRAY BROWN, MOIST, MEDIUM DENSE, MEDIUM GRAINED*		
20	▨		⧄		19			
						BORING TERMINATED AT 21.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
25								
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

NOVA

LOGGED BY:	GAN	DATE:	APR 2021
REVIEWED BY:	MS	PROJECT NO.:	2019189

APPENDIX B.1

BORING LOG B-2

DATE EXCAVATED: SEPTEMBER 18, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 76 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)	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		**OLD PARALIC DEPOSITS (Qop):** *SILTY SANDSTONE; ORANGE BROWN, DAMP, LOOSE, MEDIUM GRAINED*		
	☒	⧄		14	*MEDIUM DENSE*		
5		■		29			
		⧄		22			
			SP-SM		*SANDSTONE WITH TRACE SILT, GRAY ORANGE, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
10		■		36		DS	
15	☒	⧄		32	*PALE ORANGE GRAY, MOIST, DENSE*	SA	
20	☒	⧄	SP	27	*SANDSTONE; PALE ORANGE GRAY, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	SA	
25		⧄		38	*DENSE*		
30							

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY: GAN
DATE: APR 2021
REVIEWED BY: MS
PROJECT NO.: 2019189

NOVA

APPENDIX B.2

CONTINUED BORING LOG B-2

DATE EXCAVATED: SEPTEMBER 17, 2019
EQUIPMENT: CME 95
EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING
GPS COORD.: N/A
GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED
ELEVATION: ± 76 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

SOIL DESCRIPTION
SUMMARY OF SUBSURFACE CONDITIONS
(USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)

DEPTH (FT)	GRAPHIC LOG	BULK SAMPLE	CAL/SPT SAMPLE	SOIL CLASS. (USCS)	BLOWS PER 12-INCHES	SOIL DESCRIPTION	LABORATORY	REMARKS
30	▒	☒	▨	SP	58	**OLD PARALIC DEPOSITS (Qop):** *(CONTINUED) SANDSTONE; PALE ORANGE GRAY, MOIST, DENSE, FINE TO MEDIUM GRAINED*		
35	▒		⧄		31	*MEDIUM DENSE*		
40	▒	☒	▨	SP-SM	50/3"	*SANDSTONE WITH TRACE SILT; BROWN WITH BLACK MOTTLING, MOIST, VERY DENSE, FINE GRAINED*		
	▒		⧄		57	*DARK BROWN GRAY, FINE TO MEDIUM GRAINED*		
						BORING TERMINATED AT 43 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BENTONITE.		
45								
50								
55								
60								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
▨	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY: GAN
DATE: APR 2021
REVIEWED BY: MS
PROJECT NO.: 2019189

NOVA

APPENDIX B.3

BORING LOG B-3

DATE EXCAVATED: SEPTEMBER 17, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 54 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)	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; LIGHT BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE ROOTS*		
		■		63	*DENSE, SOME ROOTS, SOME GLASS FRAGMENTS* *SCATTERED GRAVEL*		*2.2% 112.5pcf*
5		⧄	SP-SM	24	**OLD PARALIC DEPOSITS (Qop):** *SANDSTONE WITH TRACE SILT; LIGHT GRAY ORANGE, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	SA	
		■		41			*1.4% 104.0pcf*
10		⧄		21			
15	☒	⧄	SP	32	*POORLY-GRADED SANDSTONE; LIGHT GRAY WHITE, DENSE, FINE TO MEDIUM GRAINED*		
20		⧄		44			
					BORING TERMINATED AT 21.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
25							
30							

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	——	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	– – –	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

NOVA

LOGGED BY:	GAN	DATE:	APR 2021
REVIEWED BY:	MS	PROJECT NO.:	2019189

APPENDIX B.4

BORING LOG B-4

DATE EXCAVATED: SEPTEMBER 18, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 68 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		**OLD PARALIC DEPOSITS (Qop):** *SILTY SANDSTONE; ORANGE BROWN, MOIST, LOOSE, FINE TO MEDIUM GRAINED*		
			■		22	*MEDIUM DENSE*		
5			⧄		30		SA	
			■	SP-SM	69	*SANDSTONE WITH TRACE SILT; ORANGE BROWN WITH GRAY MOTTLING, MOIST, DENSE, MEDIUM GRAINED*		
10		☒	⧄		18	*MEDIUM DENSE*	SA	
15		☒	■		47	*LIGHT ORANGE BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED*	SA	
20			⧄	SM	32	*SILTY SANDSTONE; LIGHT ORANGE BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED*		
25			⧄	SP-SM	36	*SANDSTONE WITH TRACE SILT; ORANGE TO GRAY ORANGE, DAMP TO MOIST, MEDIUM DENSE, MEDIUM DENSE, MEDIUM GRAINED*		
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY:	GAN	DATE:	APR 2021
REVIEWED BY:	MS	PROJECT NO.:	2019189

NOVA

APPENDIX B.5

CONTINUED BORING LOG B-4

DATE EXCAVATED: SEPTEMBER 18, 2019
EQUIPMENT: CME 95
EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING
GPS COORD.: N/A
GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED
ELEVATION: ± 68 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
30			■	SP-SM	57	**OLD PARALIC DEPOSITS (Qop):** *(CONTINUED) SANDSTONE WITH TRACE SILT; ORANGE GRAY, MOIST, DENSE, MEDIUM TO COARSE GRAINED*		
35		☒	■		50/5"	*VERY PALE ORANGE, DAMP, VERY DENSE, MEDIUM GRAINED*		
						BORING TERMINATED AT 36.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
40								
45								
50								
55								
60								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY:	GAN	DATE:	APR 2021
REVIEWED BY:	MS	PROJECT NO.:	2019189

NOVA

APPENDIX B.6

BORING LOG B-5

DATE EXCAVATED: SEPTEMBER 17, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 55 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		**OLD PARALIC DEPOSITS (Qop):** *SILTY SANDSTONE; ORANGE BROWN, MOIST, LOOSE, FINE TO MEDIUM GRAINED*	CR MD SA	
			▨		15			*2.3% 88pcf*
5			⧄		48	*DENSE*	SA	
			▨	SP-SM	61	*SANDSTONE WITH TRACE SILT; ORANGE BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED*		
10			⧄		31	*LIGHT ORANGE BROWN, MOIST, DENSE, MEDIUM GRAINED*		
15		☒	⧄		48	*INTERBEDDED GRAY ORANGE AND DARK GRAY SANDSTONE LENSES, MOIST, DENSE, FINE TO MEDIUM GRAINED*		
20			▨	SP	46	*SANDSTONE; LIGHT YELLOW WHITE, MOIST, DENSE, FINE TO MEDIUM GRAINED*		
						BORING TERMINATED AT 21.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
25								
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	——	GEOLOGIC CONTACT
▨	CAL. MOD. SAMPLE (ASTM D3550)	– – –	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY: GAN
DATE: APR 2021
REVIEWED BY: MS
PROJECT NO.: 2019189

NOVA

APPENDIX B.7

BORING LOG B-6

DATE EXCAVATED: SEPTEMBER 17, 2019
EQUIPMENT: CME 95
EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING
GPS COORD.: N/A
GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED
ELEVATION: ± 55 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; ORANGE BROWN, DAMP TO MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
		☒		SP-SM		**OLD PARALIC DEPOSITS (Qop):** *SANDSTONE WITH TRACE SILT; ORANGE BROWN, DAMP, MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
5			■		31			*5.5% 112.1pcf*
			⧄		16			
10			■		36			
			⧄		23			
15			■	SM	40	*SILTY SANDSTONE; ORANGE GRAY MOTTLED WITH ORANGE BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
20			⧄		52	*VERY DENSE, MEDIUM GRAINED*		
						BORING TERMINATED AT 21.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING.		
25								
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

NOVA

LOGGED BY:	GAN	DATE:	APR 2021
REVIEWED BY:	MS	PROJECT NO.:	2019189

BORING LOG B-7

DATE EXCAVATED: SEPTEMBER 18, 2019
EQUIPMENT: CME 95
EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING
GPS COORD.: N/A
GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED
ELEVATION: ± 71 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		**OLD PARALIC DEPOSITS (Qop):** *SANDSTONE WITH TRACE SILT; ORANGE BROWN, MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
			■		21	*MEDIUM DENSE*	DS	
5			⧄	SP-SM	14	*POORLY-GRADED SANDSTONE-SILTY SANDSTONE; LIGHT YELLOW ORANGE BROWN, DAMP TO MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	SA	
			■		71	*DENSE*	DS	
10			⧄	SM	28	*SILTY SANDSTONE; LIGHT ORANGE BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	SA	
15			■	SP	59	*SANDSTONE; ORANGE MEDIUM BROWN, MOIST, DENSE, MEDIUM GRAINED*		
20			⧄		34	*LIGHT ORANGE WITH ORANGE BROWN CROSS BEDDING, FINE GRAINED*		
25								
			■		65	*LIGHT ORANGE BROWN WITH ORANGE STAINING, DENSE*		
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY: GAN
DATE: APR 2021
REVIEWED BY: MS
PROJECT NO.: 2019189

NOVA

APPENDIX B.9

CONTINUED BORING LOG B-7

DATE EXCAVATED: SEPTEMBER 18, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 71 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
30	▨		⧄	SP	34	**OLD PARALIC DEPOSITS (Qop):** *(CONTINUED) SANDSTONE WITH TRACE SILT; LIGHT ORANGE BROWN, MOIST, DENSE, FINE GRAINED*		
35	▨	☒	■		50/6"	*6" ORANGE WHITE LENSE* *DARK ORANGE WITH BLACK SPECS, VERY DENSE*	SA	
40	▨		⧄		55	*LIGHT ORANGE WHITE, DRY, MEDIUM GRAINED*		
						BORING TERMINATED AT 41.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BENTONITE		
45								
50								
55								
60								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	——	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY: GAN

DATE: APR 2021

REVIEWED BY: MS

PROJECT NO.: 2019189

NOVA

APPENDIX B.10

BORING LOG B-8

DATE EXCAVATED: SEPTEMBER 18, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 87 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				SP-SM		**OLD PARALIC DEPOSITS (Qop):** *SANDSTONE WITH TRACE SILT; ORANGE BROWN, DAMP, LOOSE, FINE TO MEDIUM GRAINED*		
		☒	⧄		8	*LOOSE*	SA CR RV	
5				ML		*6-INCH SILTSTONE LENSE, MEDIUM BROWN*		
			■	SP-SM	20	*SANDSTONE WITH TRACE SILT, ORANGE BROWN, DAMP, MEDIUM DENSE, FINE TO MEDIUM GRAINED*		
			⧄		23		SA	
10			■		46	*DENSE*	DS	
15			⧄		19	*ORANGE BROWN MOTTLED WITH GRAY*	SA	
20			■		45	*POORLY-GRADED SANDSTONE, LIGHT ORANGE GRAY, DAMP, MEDIUM DENSE, MEDIUM GRAINED*		*DISTURBED SAMPLE*
25			⧄		34		SA	
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY: GAN

DATE: APR 2021

REVIEWED BY: MS

PROJECT NO.: 2019189

NOVA

APPENDIX B.11

CONTINUED BORING LOG B-8

DATE EXCAVATED: SEPTEMBER 18, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 87 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
30			■	SP-SM	75	**OLD PARALIC DEPOSITS (Qop):** *(CONTINUED) SANDSTONE WITH TRACE SILT; LIGHT ORANGE GRAY, DAMP, DENSE, MEDIUM GRAINED*		
35			⧄		33	*FINE GRAINED*		
40			■		50/2"	*VERY DENSE*		
45			⧄		31	*ORANGE BROWN, MOIST, DENSE, FINE TO MEDIUM GRAINED, DARK ORANGE, CROSS BEDDING*	SA	
50		☒	■		50/5"	*LIGHT ORANGE BROWN, DAMP, VERY DENSE, FINE TO MEDIUM GRAINED*	SA	
55			⧄	SW	42	*WELL-GRADED SANDSTONE; VERY PALE ORANGE, DAMP, DENSE, FINE TO VERY COARSE GRAINED*		
						BORING TERMINATED AT 56.5 FT. NO GROUNDWATER ENCOUNTERED. NO CAVING. BACKFILLED WITH BENTONITE.		
60								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	– – –	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY: GAN
DATE: APR 2021
REVIEWED BY: MS
PROJECT NO.: 2019189

NOVA

APPENDIX B.12

BORING LOG P-1

DATE EXCAVATED: SEPTEMBER 17, 2019

EQUIPMENT: CME 95

EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING

GPS COORD.: N/A

GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED

ELEVATION: ± 54 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; LIGHT BROWN, DRY, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, TRACE ROOTS* *DENSE, SOME ROOTS, SOME GLASS FRAGMENTS* *SCATTERED GRAVEL*		
5						*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION WELL.*		
10								
15								
20								
25								
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

LOGGED BY:	GAN	DATE:	APR 2021
REVIEWED BY:	MS	PROJECT NO.:	2019189

NOVA

APPENDIX B.13

BORING LOG P-2

DATE EXCAVATED: SEPTEMBER 18, 2019
EQUIPMENT: CME 95
EXCAVATION DESCRIPTION: 8-INCH DIAMETER AUGER BORING
GPS COORD.: N/A
GROUNDWATER DEPTH: GROUNDWATER NOT ENCOUNTERED
ELEVATION: ± 76 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		**OLD PARALIC DEPOSITS (Qop):** *SILTY SANDSTONE; ORANGE BROWN, DAMP, LOOSE, MEDIUM GRAINED* *MEDIUM DENSE*		
5						*BORING TERMINATED AT 5 FT AND CONVERTED TO A PERCOLATION WELL.*		
10								
15								
20								
25								
30								

KEY TO SYMBOLS

▼/▽	GROUNDWATER / STABILIZED	#	ERRONEOUS BLOW COUNT
☒	BULK SAMPLE	*	NO SAMPLE RECOVERY
⧄	SPT SAMPLE (ASTM D1586)	———	GEOLOGIC CONTACT
■	CAL. MOD. SAMPLE (ASTM D3550)	— — —	SOIL TYPE CHANGE

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

NOVA

LOGGED BY:	GAN	DATE:	APR 2021
REVIEWED BY:	MS	PROJECT NO.:	2019189

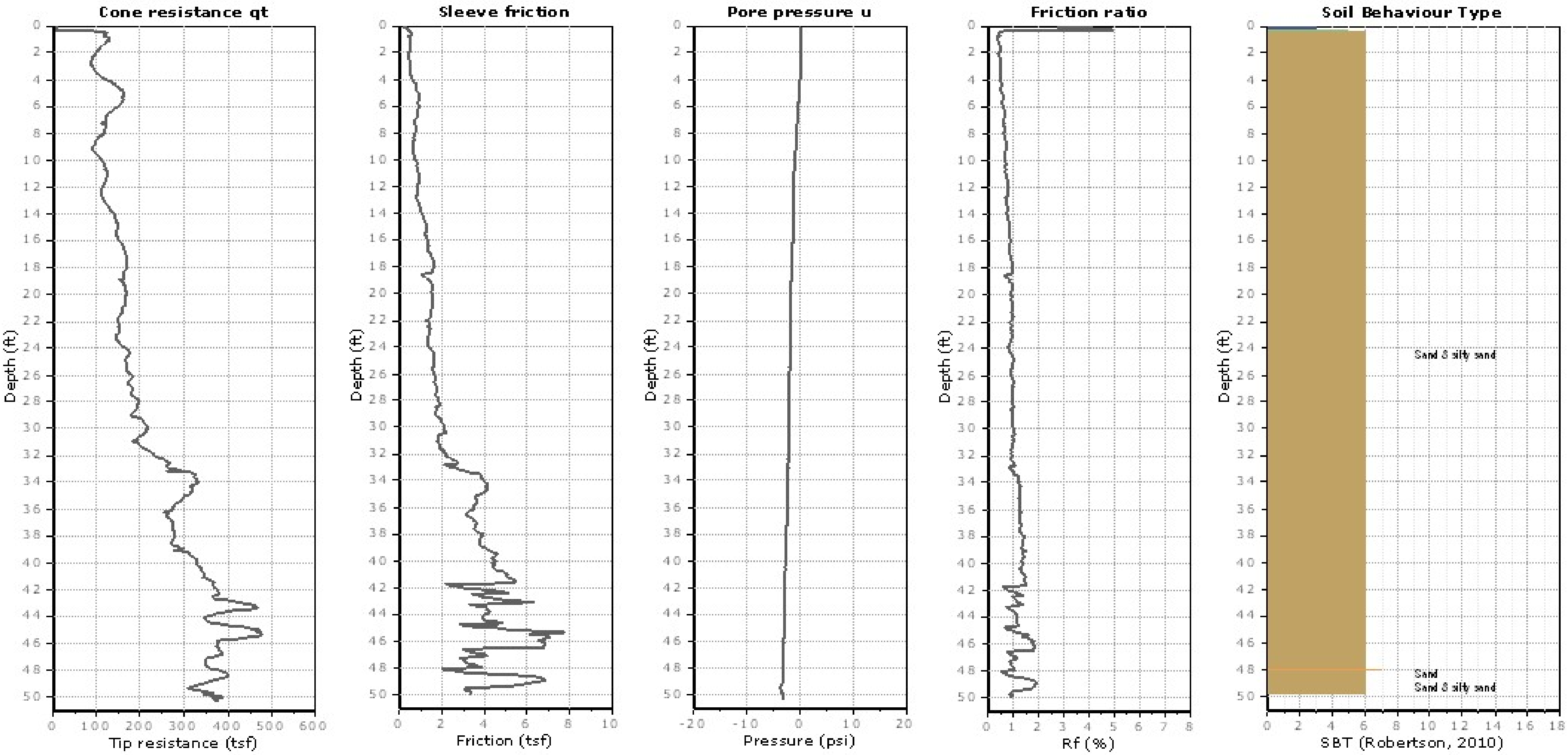
APPENDIX B.14

APPENDIX C
LOGS OF THE CPT SOUNDINGS

Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **NOVA Services / Leucadia Mixed Use**
Location: **Encinitas, CA**

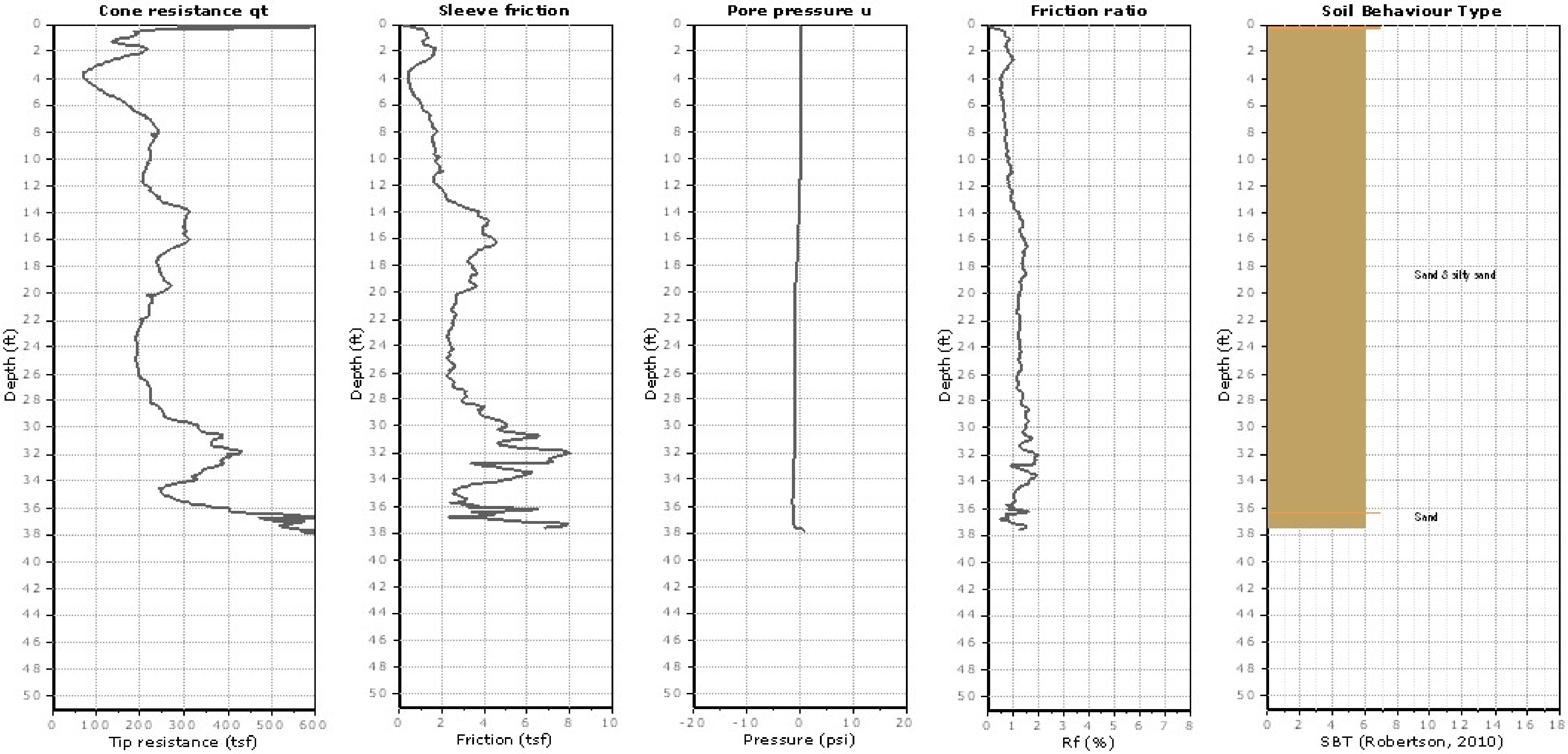
CPT-1
Total depth: 50.27 ft, Date: 10/10/2019



Kehoe Testing and Engineering
714-901-7270
steve@kehoetesting.com
www.kehoetesting.com

Project: **NOVA Services / Leucadia Mixed Use**
Location: **Encinitas, CA**

CPT-2
Total depth: 37.93 ft, Date: 10/10/2019



APPENDIX D
RECORDS OF LABORATORY TESTING

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.*
- **DENSITY OF SOIL IN PLACE (ASTM D2937):** *In-place moisture contents and dry densities were determined for representative soil samples. This information was an aid to classification and permitted recognition of variations in material consistency with depth. The dry unit weight is determined in pounds per cubic foot, and the in-place moisture content is determined as a percentage of the soil's dry weight. The results are summarized in the exploration logs presented 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.*
- **DIRECT SHEAR TEST (ASTM D3080):** *Direct shear tests were performed on remolded and relatively undisturbed samples in general accordance with ASTM D3080 to evaluate the shear stregth 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.*
- **R-VALUE (ASTM D2844):** *The resistance Value, or R-Value, for near-surface site soils were evaluated in general accordance with California Test (CT) 301 and ASTM D2844. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results.*
- **GRADATION ANALYSIS (ASTM C 136 and/or ASTM D422):** *Tests were performed on selected representative soil samples in general accordance with ASTM D422. The grain size distributions of selected samples were determined in accordance with ASTM C 136 and/or ASTM D422. The results of the tests are summarized on Appendix C.3 through Appendix C.20.*

LAB TEST SUMMARY

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	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 - 2.5	Medium Brown Silty Sand	133.0	8.3
B-5	0 - 2.5	Orange Brown Silty Sandstone	133.8	7.3

Resistance Value (Cal. Test Method 301 & ASTM D2844)

Sample Location	Sample Depth (ft.)	Soil Description	R-Value
B-8	2.5 - 7.5	Sandstone with Trace Silt	35

DENSITY OF SOIL IN PLACE (ASTM D2937)

Sample Location	Sample Depth (ft)	Soil Description	Moisture (%)	Dry Density (pcf)
B-1	5	Orange Brown Sandstone with Silt	4.6	124.9
B-3	2.5	Light Brown Silty Sandstone	2.2	112.5
B-3	7.5	Light Gray Orange Sandstone with Silt	1.4	104.0
B-5	2.5	Orange Brown Silty Sandstone	2.3	88.0
B-6	5	Orange Brown Sandstone with Silt	5.5	112.1

Direct Shear (ASTM D3080)

Sample Location	Depth (feet)	Soil Description	Friction Angle (degrees)	Apparent Cohesion (psf)
B-7	7.5	Light Yellow Orange Brown Sandstone with Silt	26	323
B-8	10	Orange Brown Sandstone with Silt	38	307

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

Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content		Chloride Content	
				(ppm)	(%)	(ppm)	(%)
B-5	2 - 5	7.4	1200	64	0.006	170	0.017
B-8	2.5 - 7.5	7.4	10000	33	0.003	32	0.003

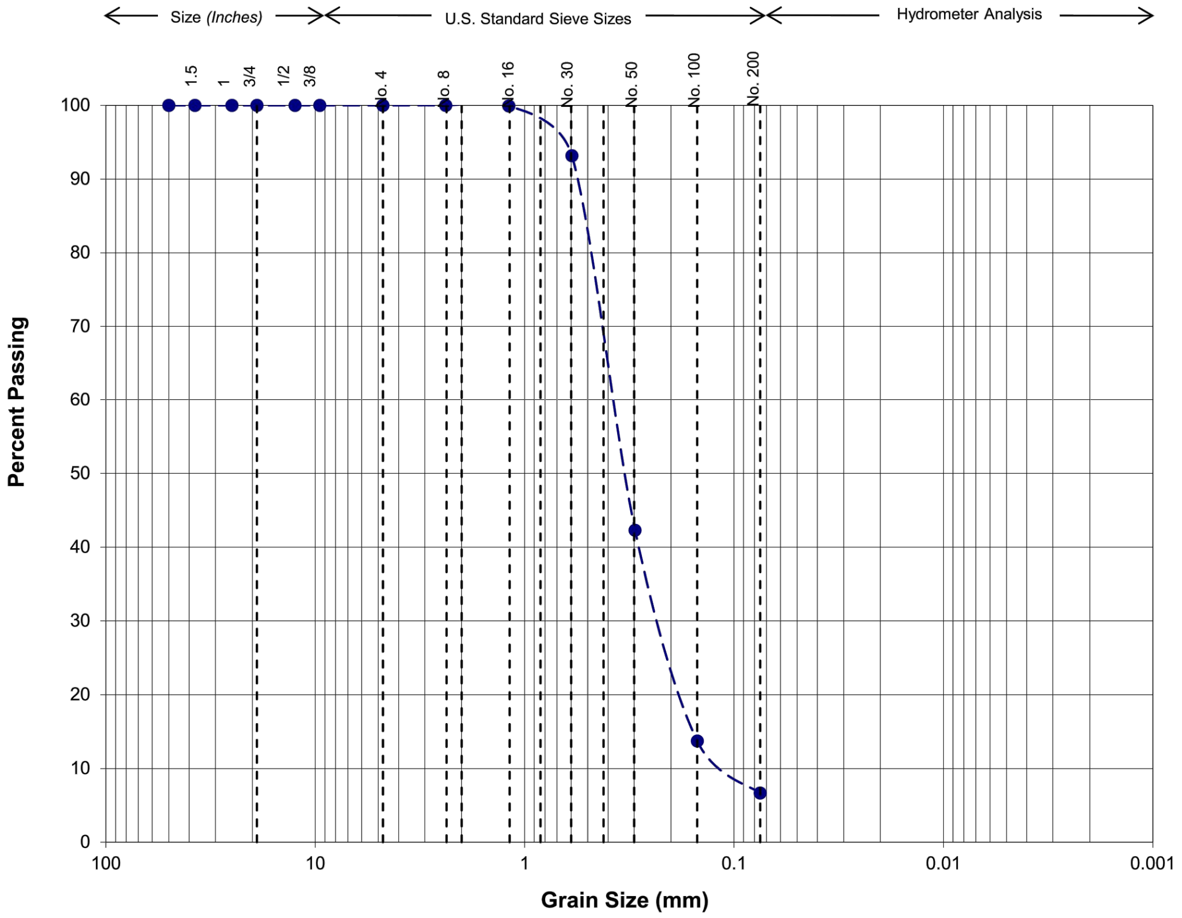
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

LAB TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.2



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B-2
Depth (ft):	15
USCS Soil Type:	SP-SM
Passing No. 200 (%):	7

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4373 VIEWRIDGE AVENUE, SUITE B
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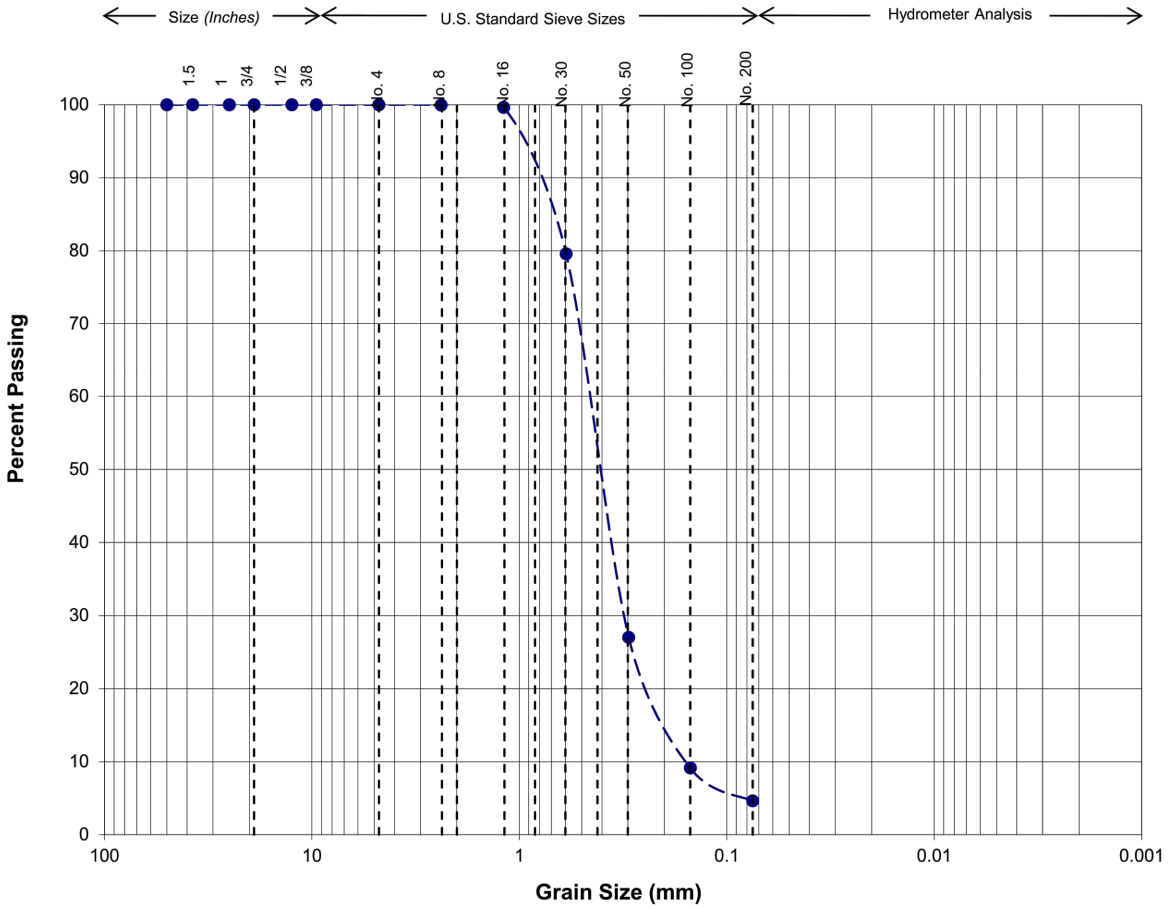
GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B-2
Depth (ft):	20
USCS Soil Type:	SP
Passing No. 200 (%):	5

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4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
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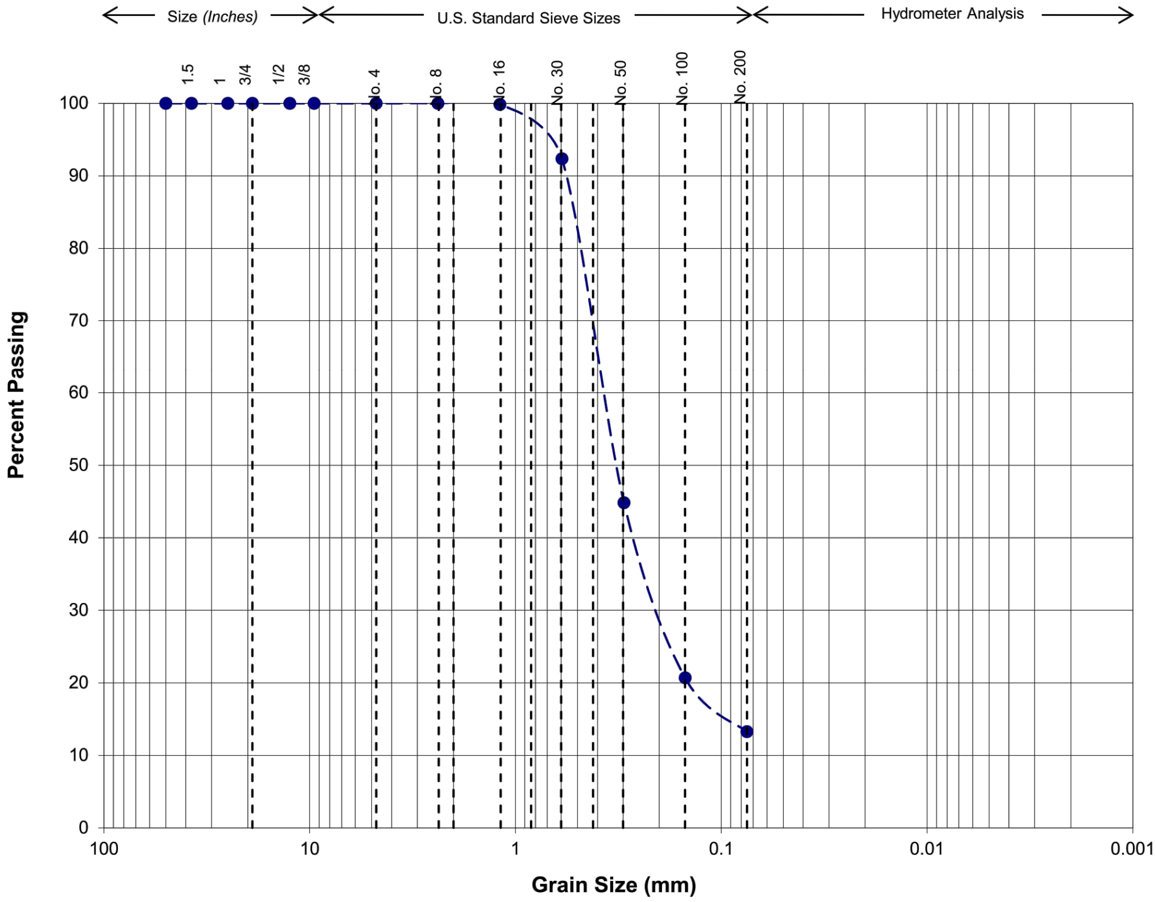
GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.4



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-3

Depth (ft): 5

USCS Soil Type: SP-SM

Passing No. 200 (%): 13

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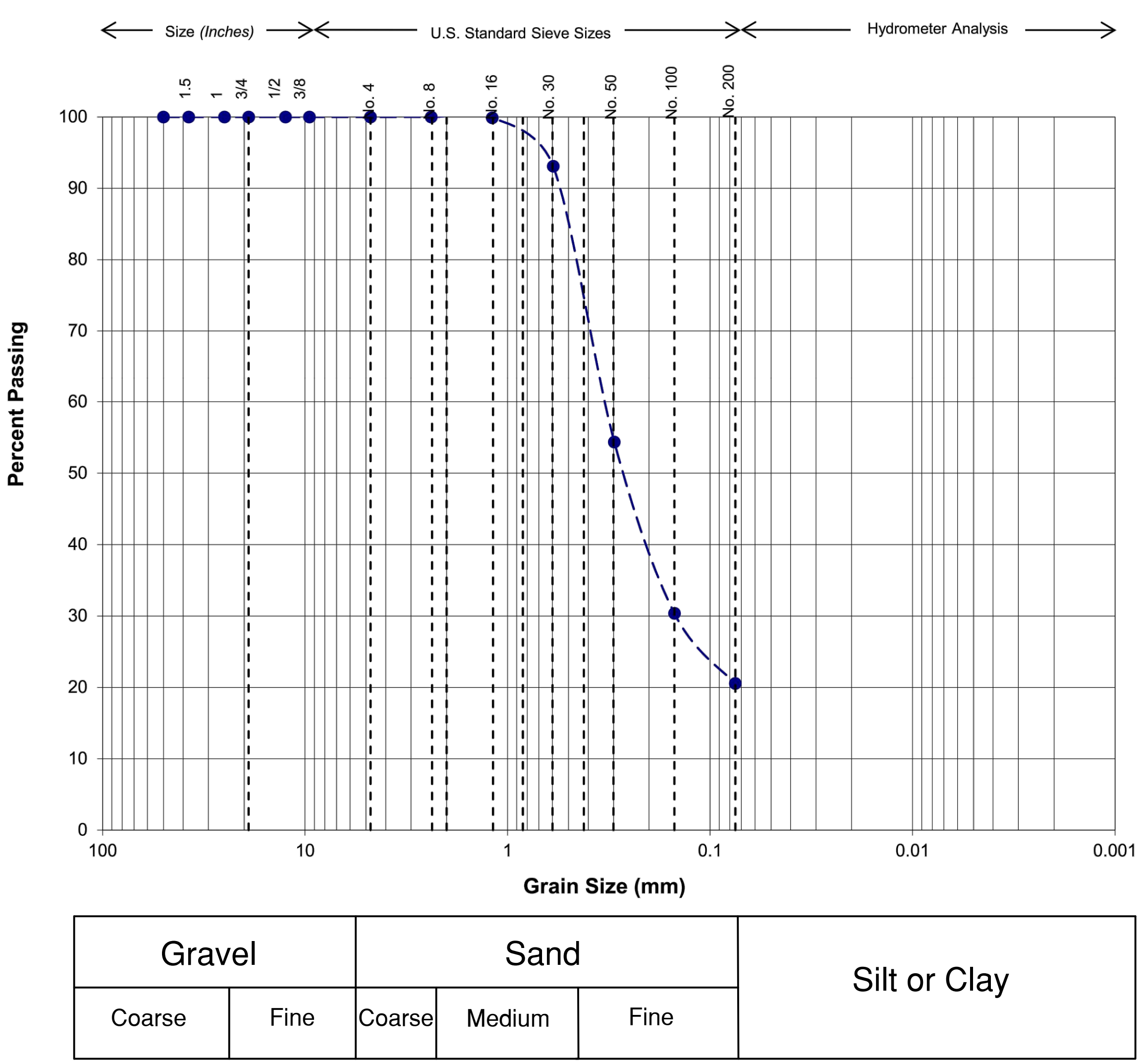
4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.5



Sample Location: B-4

Depth (ft): 5

USCS Soil Type: SM

Passing No. 200 (%): 21

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SAN DIEGO, CALIFORNIA
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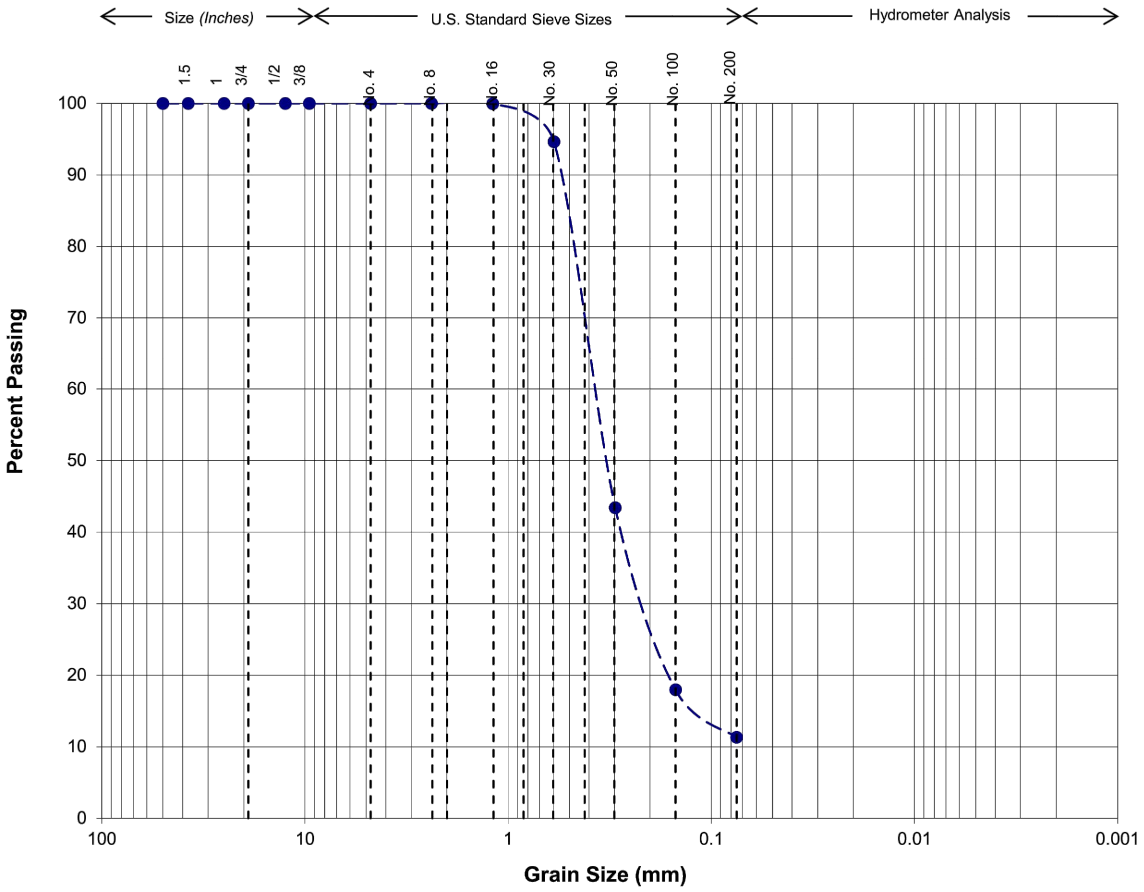
GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.6



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-4

Depth (ft): 10

USCS Soil Type: SP-SM

Passing No. 200 (%): 11

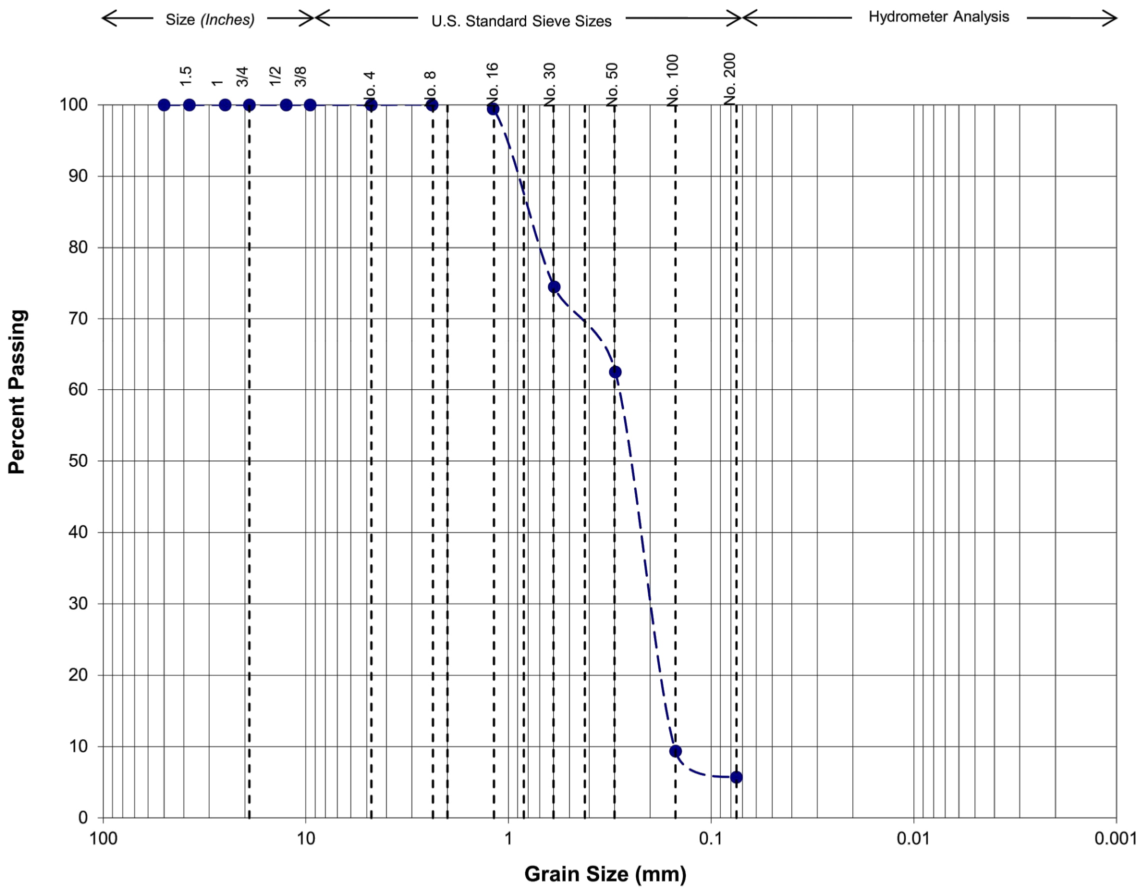
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.7



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-4

Depth (ft): 15

USCS Soil Type: SP-SM

Passing No. 200 (%): 6

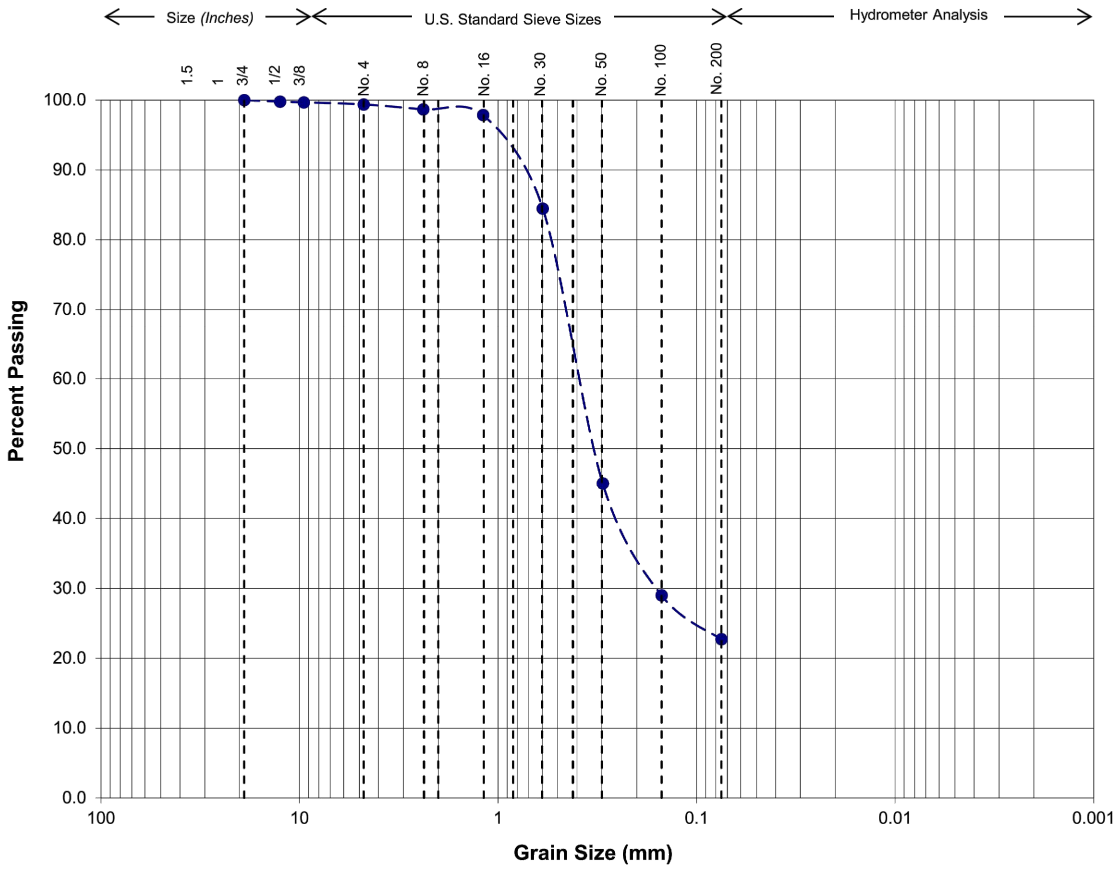
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.8



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B-5
Depth (ft):	0 - 2.5
USCS Soil Type:	SM
Passing No. 200 (%):	23

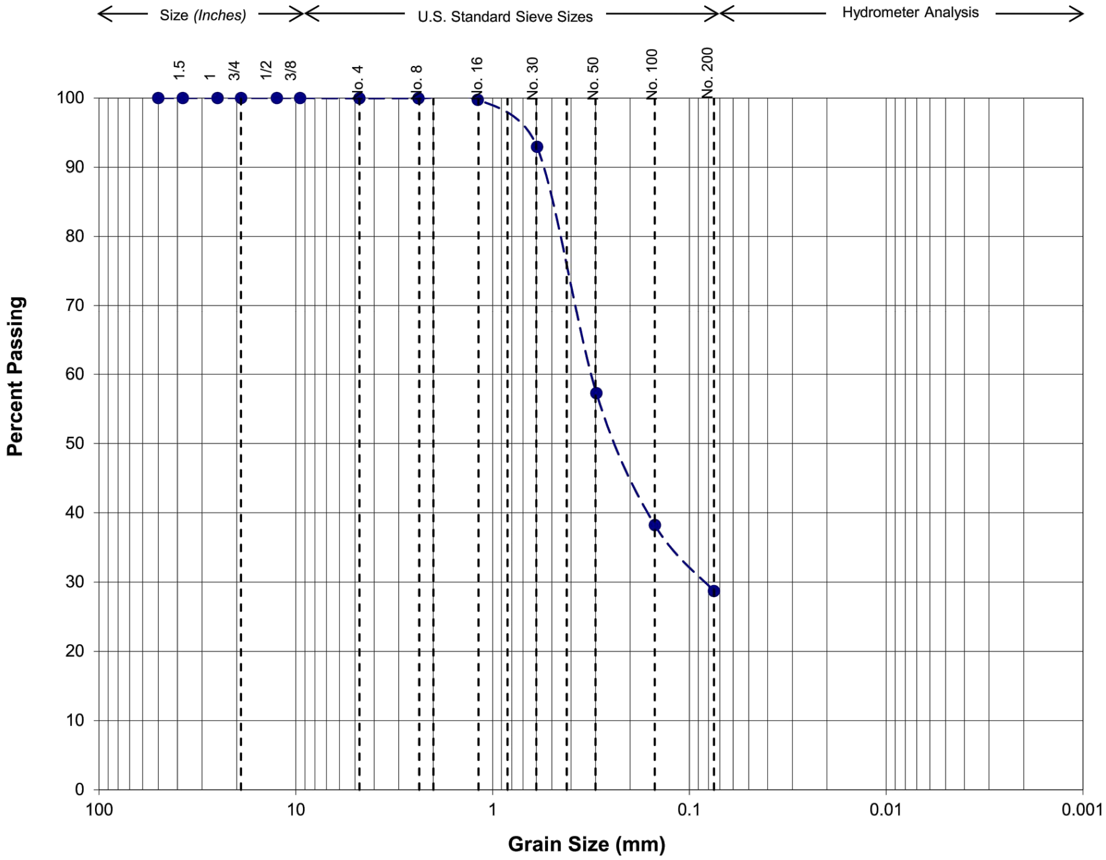
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP
DATE: APR 2021
PROJECT: 2019189
APPENDIX: D.9



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-5

Depth (ft): 5

USCS Soil Type: SM

Passing No. 200 (%): 29

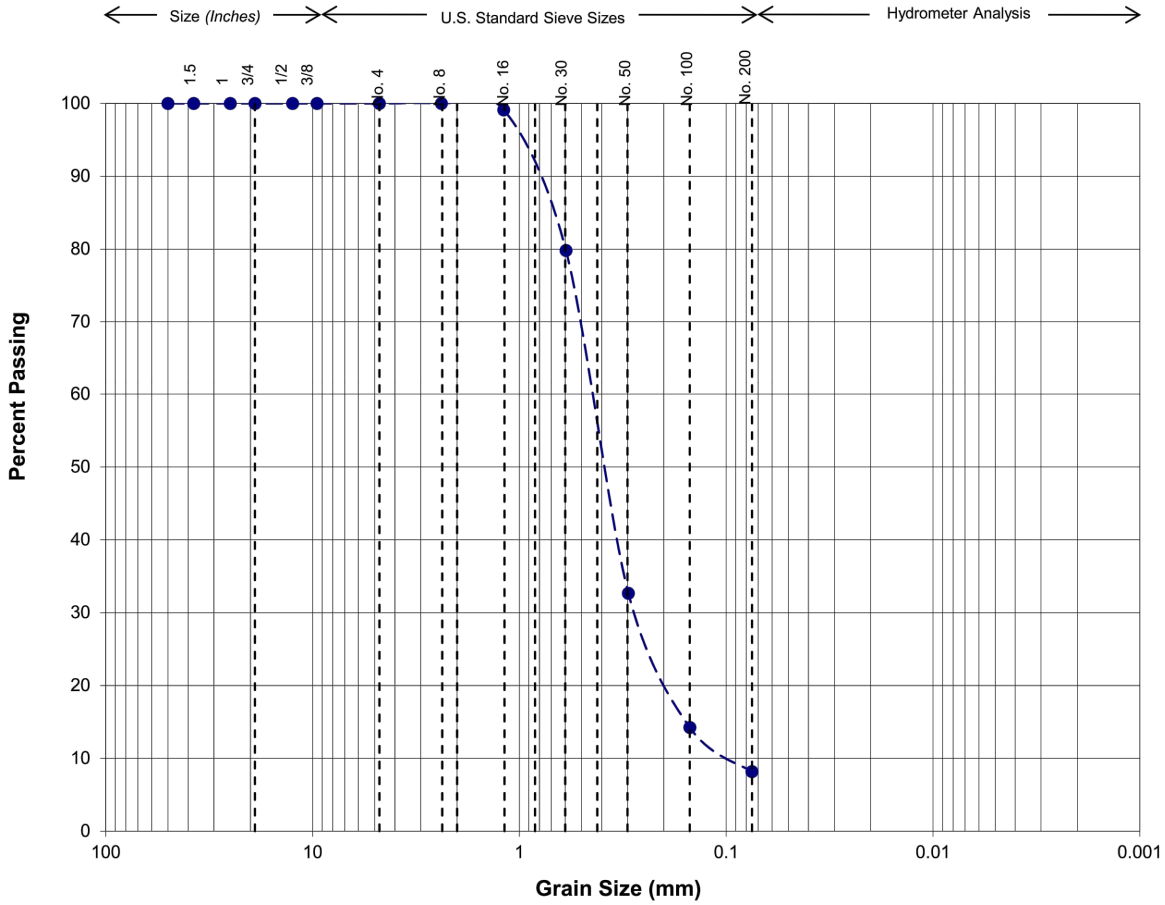
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.10



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-7

Depth (ft): 5

USCS Soil Type: SP-SM

Passing No. 200 (%): 8

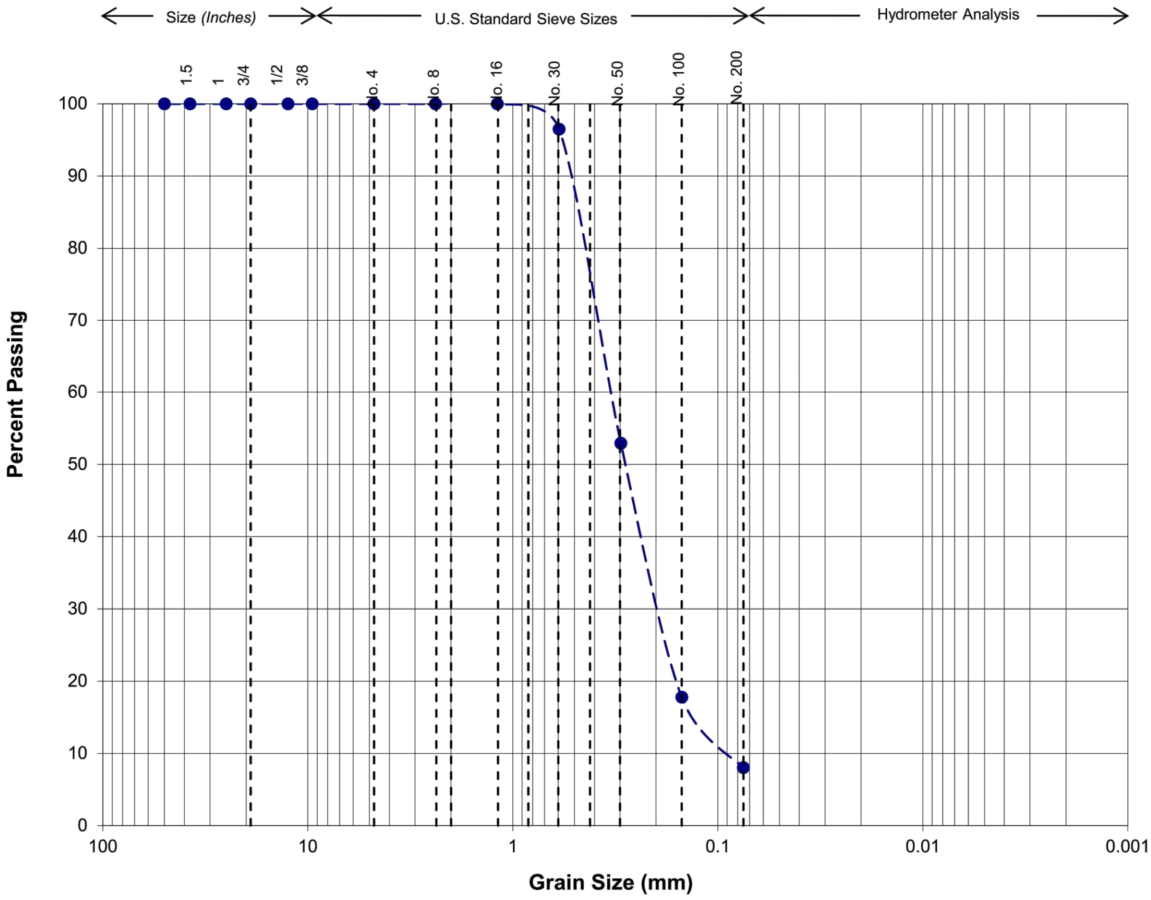
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.11



<table>
<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location: B-7

Depth (ft): 7.5

USCS Soil Type: SP-SM

Passing No. 200 (%): 8

NOVA

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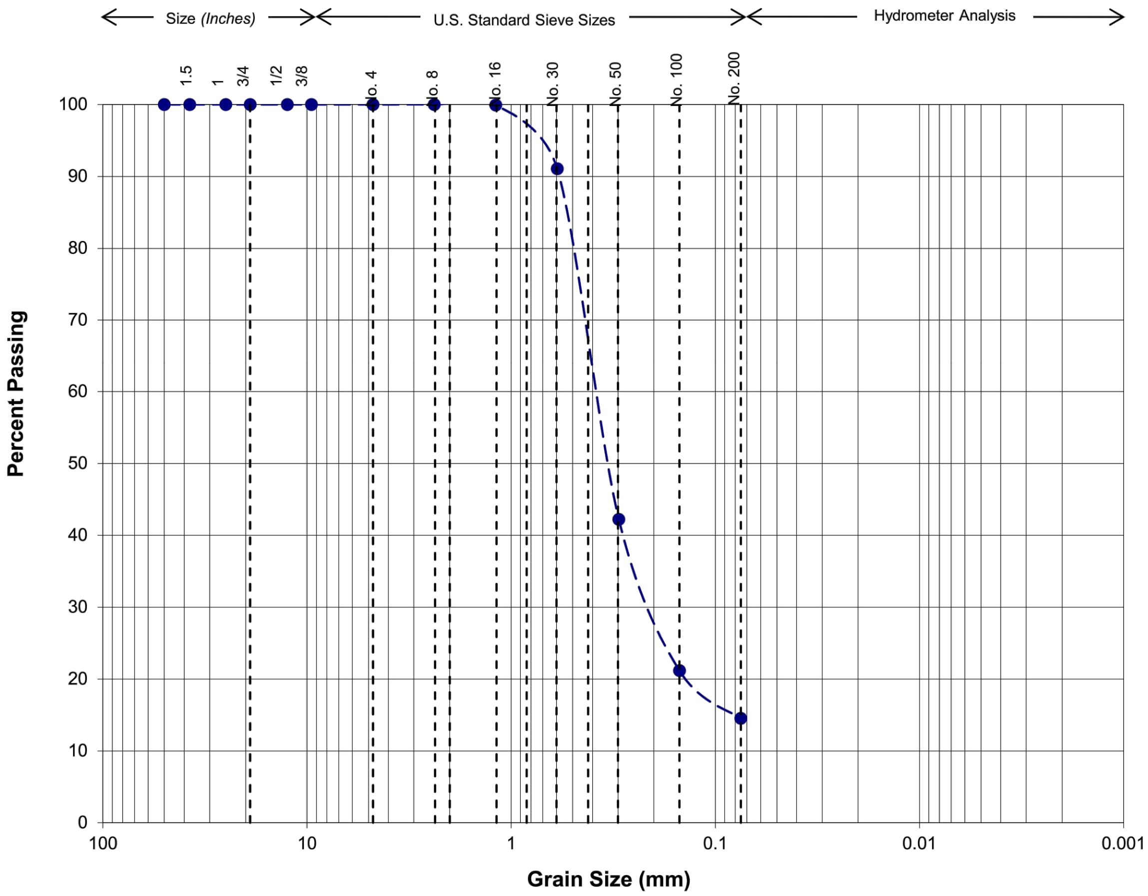
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.12



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-7

Depth (ft): 10

USCS Soil Type: SM

Passing No. 200 (%): 15

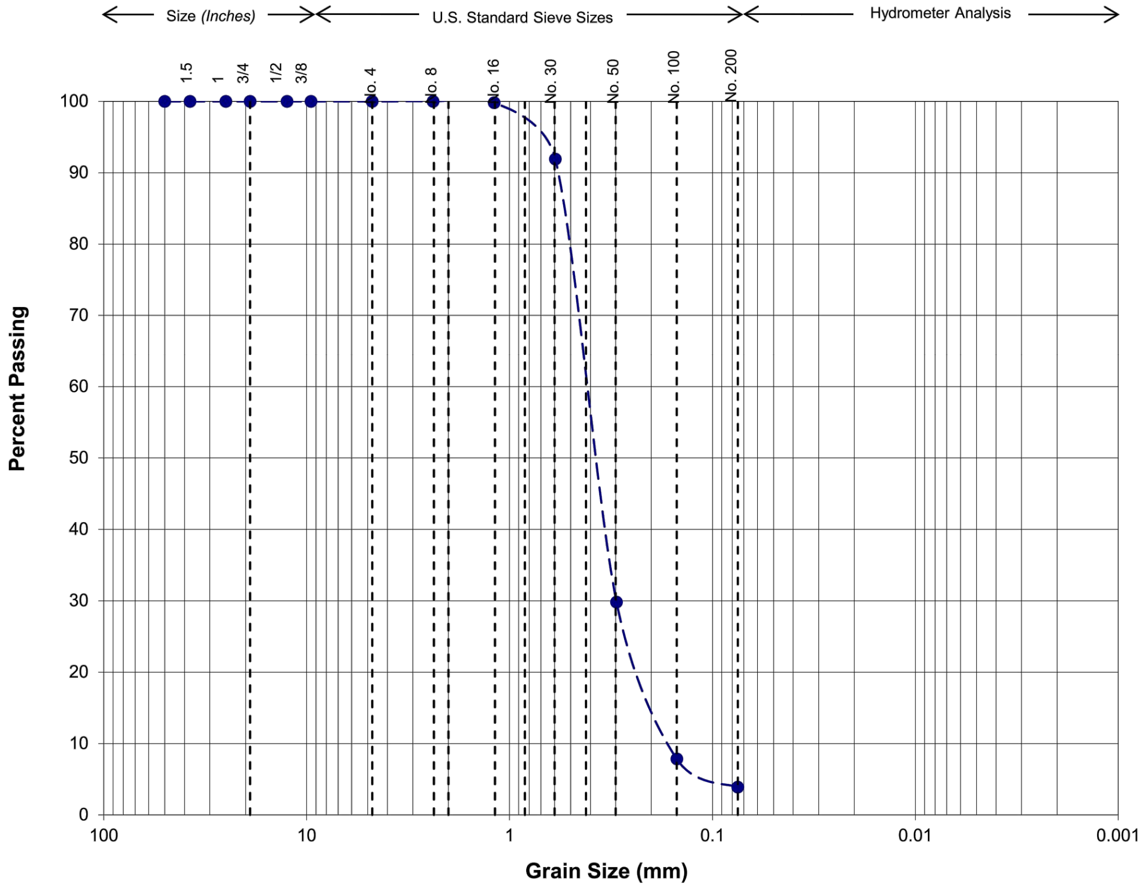
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.13



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-7

Depth (ft): 35

USCS Soil Type: SP

Passing No. 200 (%): 4

NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
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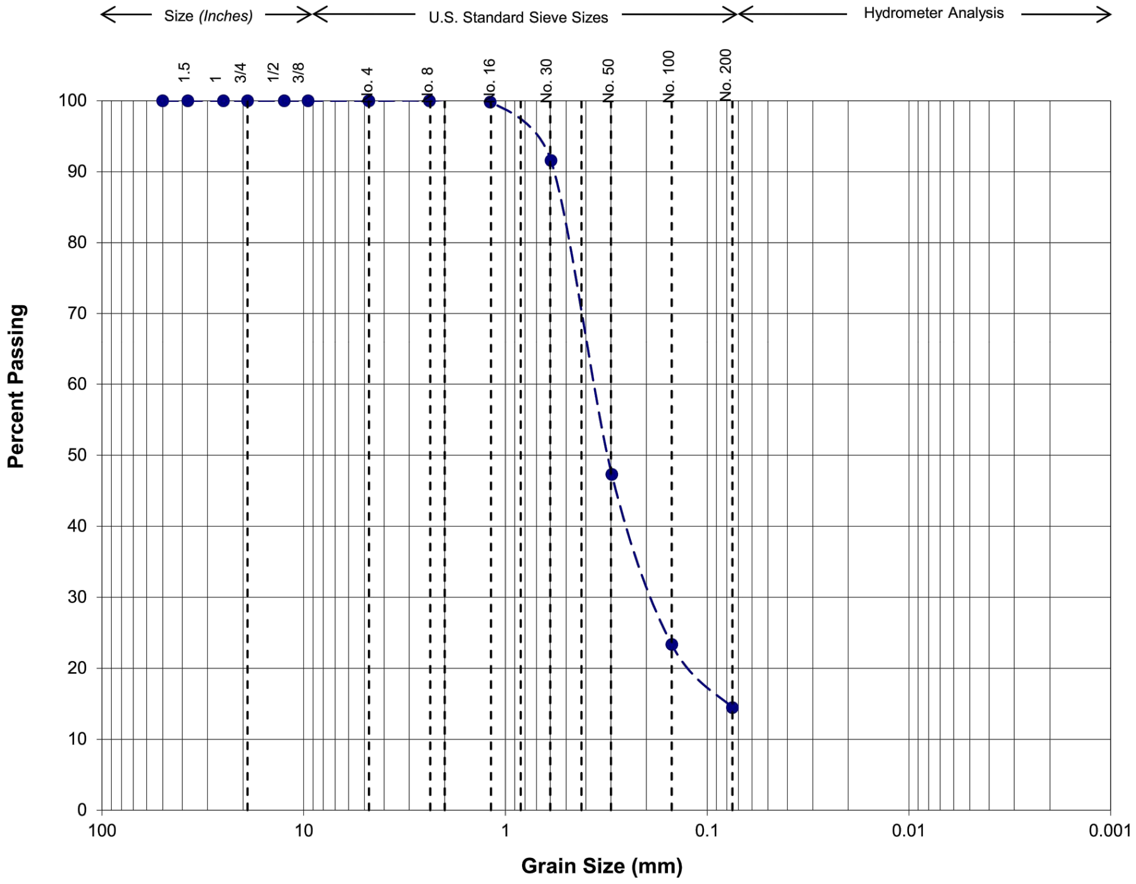
GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.14



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-8

Depth (ft): 2.5

USCS Soil Type: SP-SM

Passing No. 200 (%): 14

NOVA

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SAN DIEGO, CALIFORNIA
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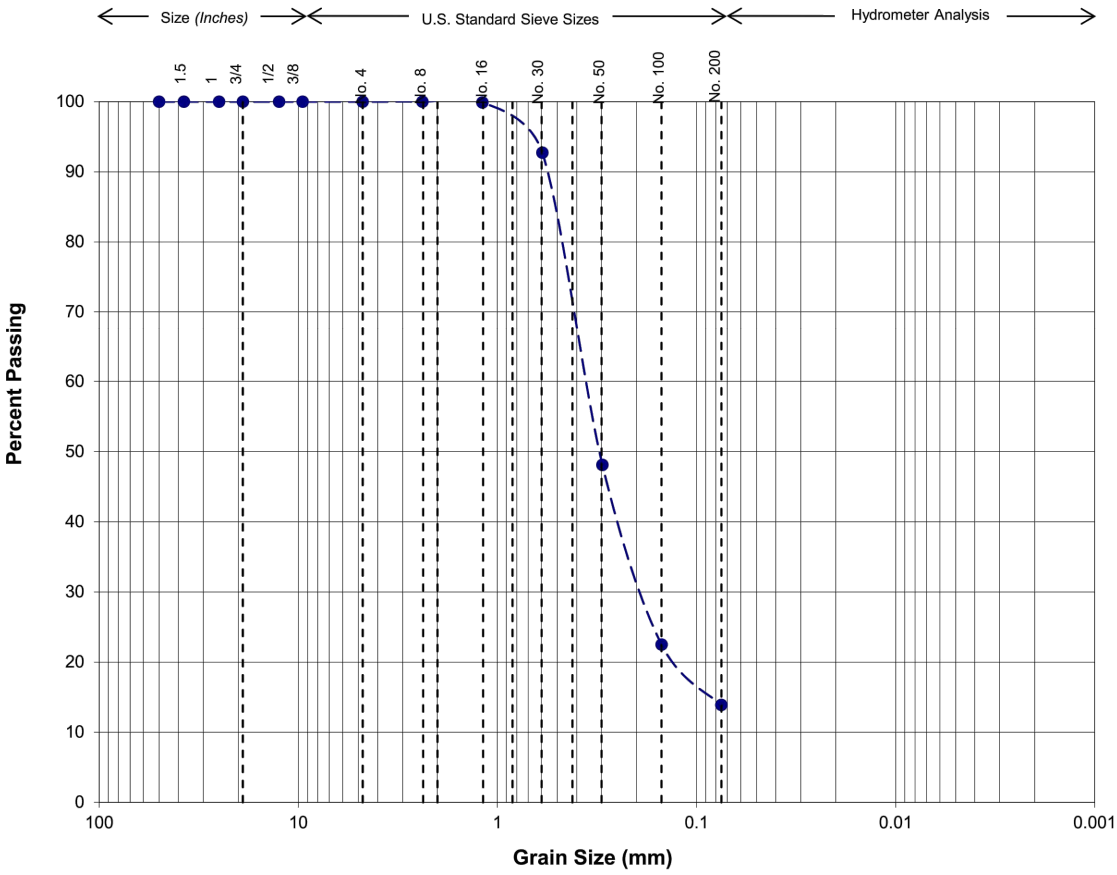
GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.15



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B-8
Depth (ft):	7.5
USCS Soil Type:	SP-SM
Passing No. 200 (%):	14

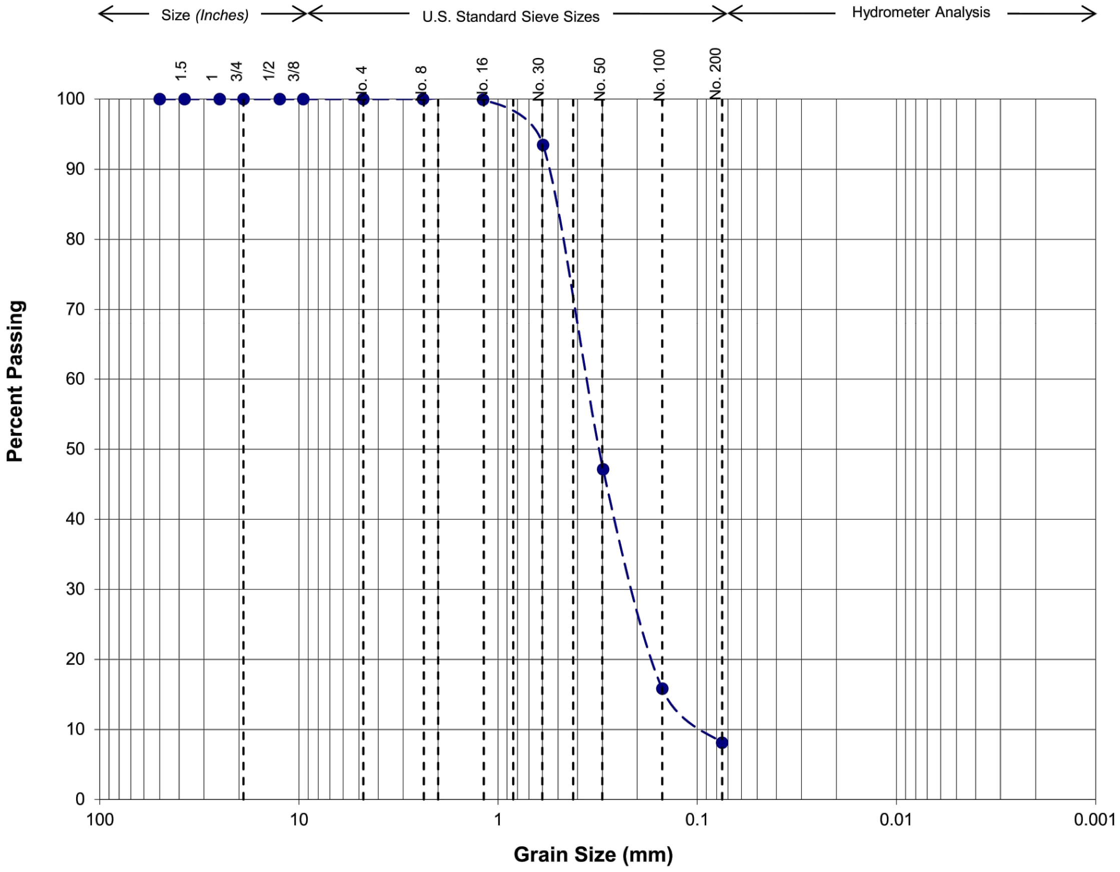
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.16



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-8

Depth (ft): 15

USCS Soil Type: SP-SM

Passing No. 200 (%): 8

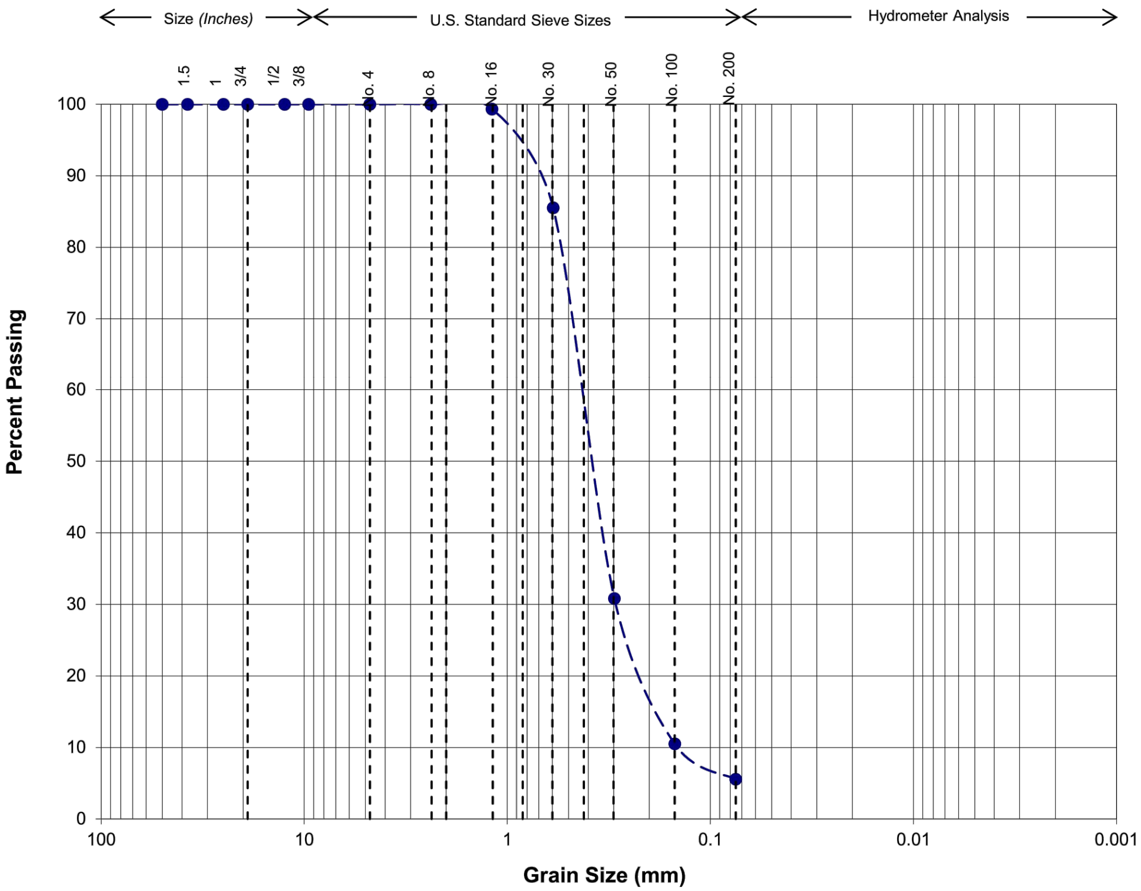
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.17



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-8

Depth (ft): 25

USCS Soil Type: SP-SM

Passing No. 200 (%): 6

NOVA

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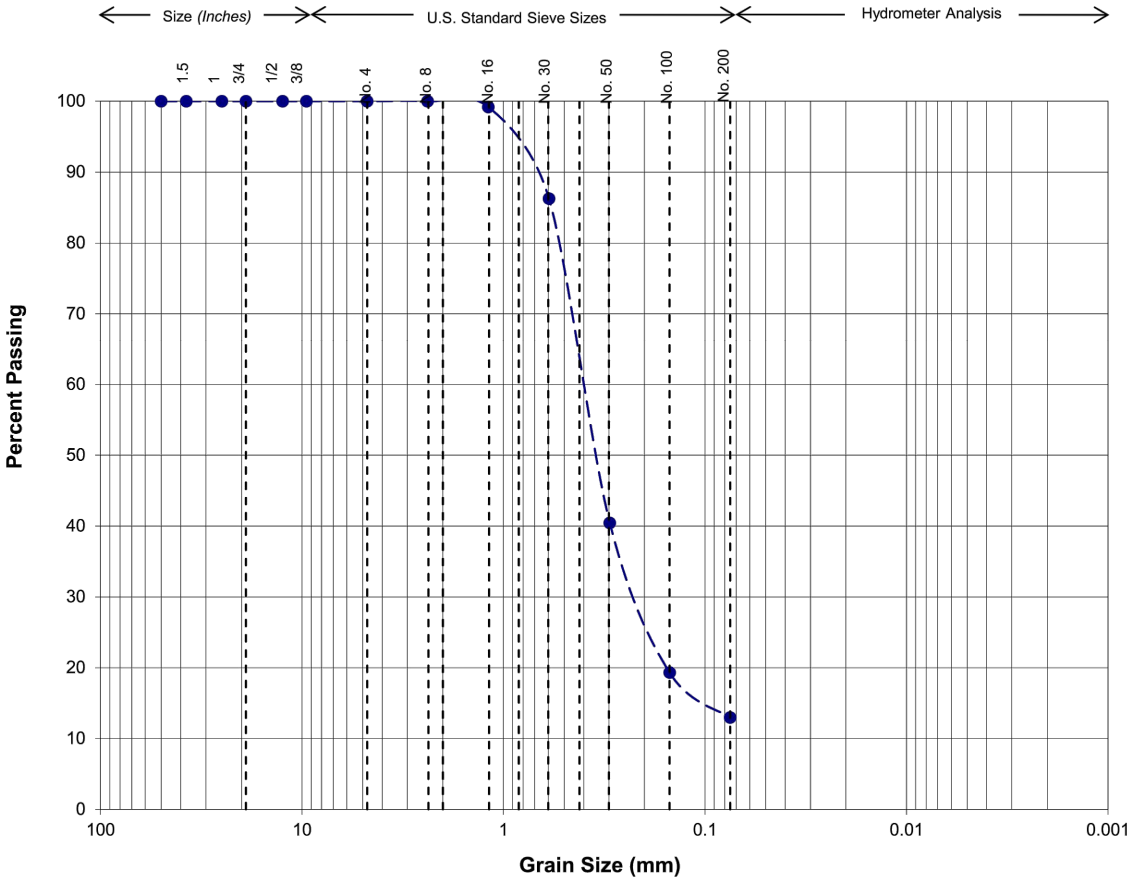
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.18



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-8

Depth (ft): 45

USCS Soil Type: SP-SM

Passing No. 200 (%): 13

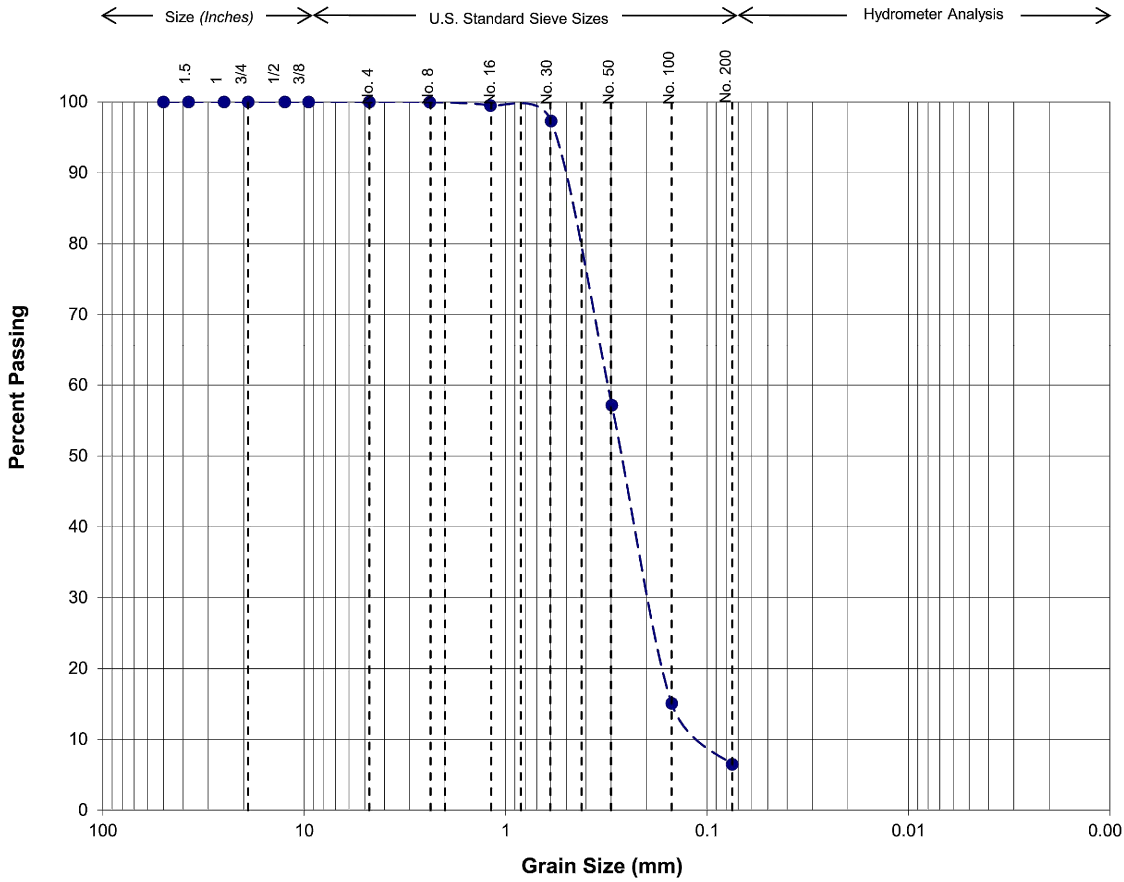
NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
PHONE: 858-292-7575 FAX: 858-292-7570

GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.19



<table>
<tr><td colspan="2">Gravel</td><td colspan="3">Sand</td><td rowspan="2">Silt or Clay</td></tr>
<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location:	B-8
Depth (ft):	50
USCS Soil Type:	SP-SM
Passing No. 200 (%):	6

NOVA

4373 VIEWRIDGE AVENUE, SUITE B
SAN DIEGO, CALIFORNIA
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GRADATION ANALYSIS TEST RESULTS

LEUCADIA MIXED USE
1950 N. COAST HWY 101
ENCINITAS, CALIFORNIA

BY: HAP	DATE: APR 2021	PROJECT: 2019189	APPENDIX: D.20

APPENDIX E
INFILTRATION RECORDS

Categorization of Infiltration Feasibility Condition	**Form I-8**

Part 1 - Full Infiltration Feasibility Screening Criteria

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 infiltration rate of the existing soils for locations P-1 and P-2, based on the on-site infiltration study was calculated to be greater than 0.5 inches per hour (1.51 inches per hour and 1.74 inches per hour for P-1 and P-2, respectively) after applying a minimum factor of safety (F) of F=2.

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.

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**

Infiltration of water has the potential to result in an increased risk of slope failure of existing slopes and nearby coastal bluff zones. As such, BMPs are not suitable for any location on site.

Form I-8 Page 2 of 4

<table>
<tr><th>Criteria</th><th>Screening Question</th><th>Yes</th><th>No</th></tr>
<tr><td>3</td><td>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, stormwater 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.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:
<i>Water contamination was not evaluated by NOVA Services.</i>
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>4</td><td>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.</td><td></td><td></td></tr>
<tr><td colspan="4">Provide basis:
<i>The potential for water balance was not evaluated by NOVA Services.</i>
Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</td></tr>
<tr><td>Part 1 Result *</td><td colspan="2">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</td><td><i>Proceed to Part 2</i></td></tr>
</table>

*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 Agency/Jurisdictions to substantiate findings

Form I-8 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:

The infiltration rate of the existing soils for locations P-1 and P-2, based on the on-site infiltration study was calculated to be greater than 0.5 inches per hour (1.51 inches per hour and 1.74 inches per hour for P-1 and P-2, respectively) after applying a minimum factor of safety (F) of F=2.

The soil and geologic conditions allow for infiltration but not without increasing the risk of geotechnical hazards.

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.

Criteria	Screening Question	Yes	No
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:

Infiltration of water has the potential to result in an increased risk of slope failure of existing slopes and nearby coastal bluff zones. As such, BMPs are not suitable for any location on site.

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.

Form I-8 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, stormwater 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.**	*No Infiltration*

*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 Agency/Jurisdictions to substantiate findings

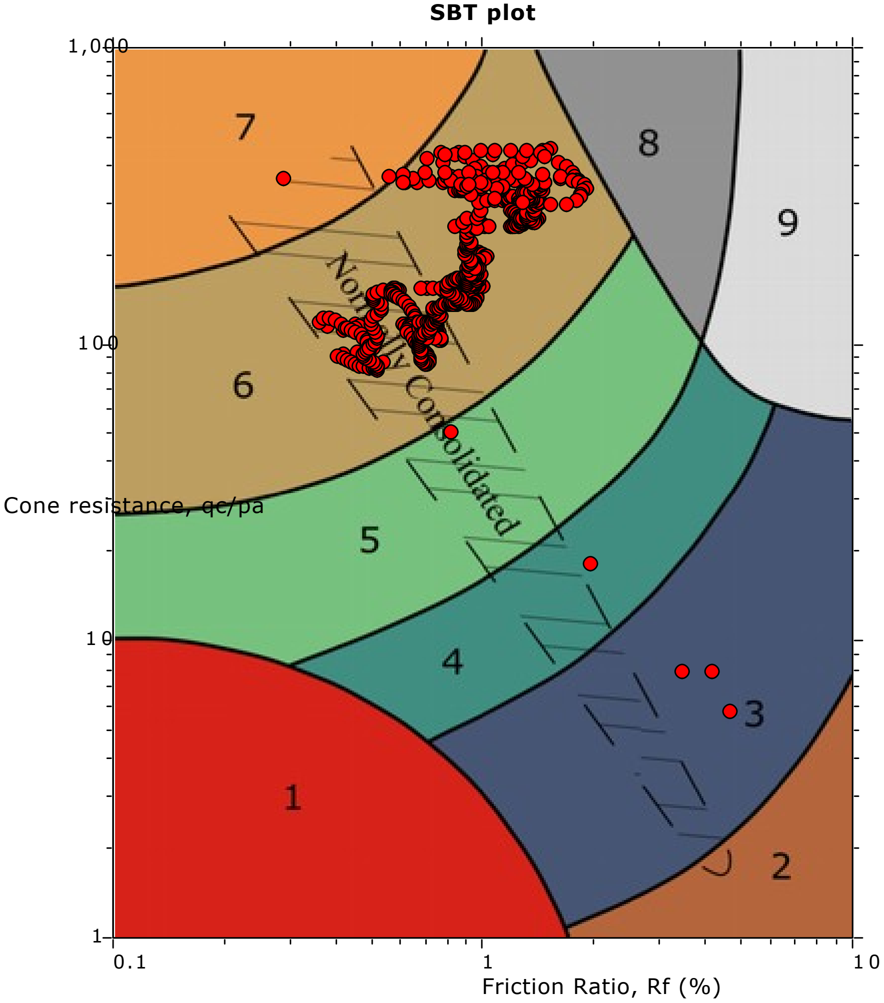
APPENDIX F
SEISMIC SHEAR WAVE SURVEY

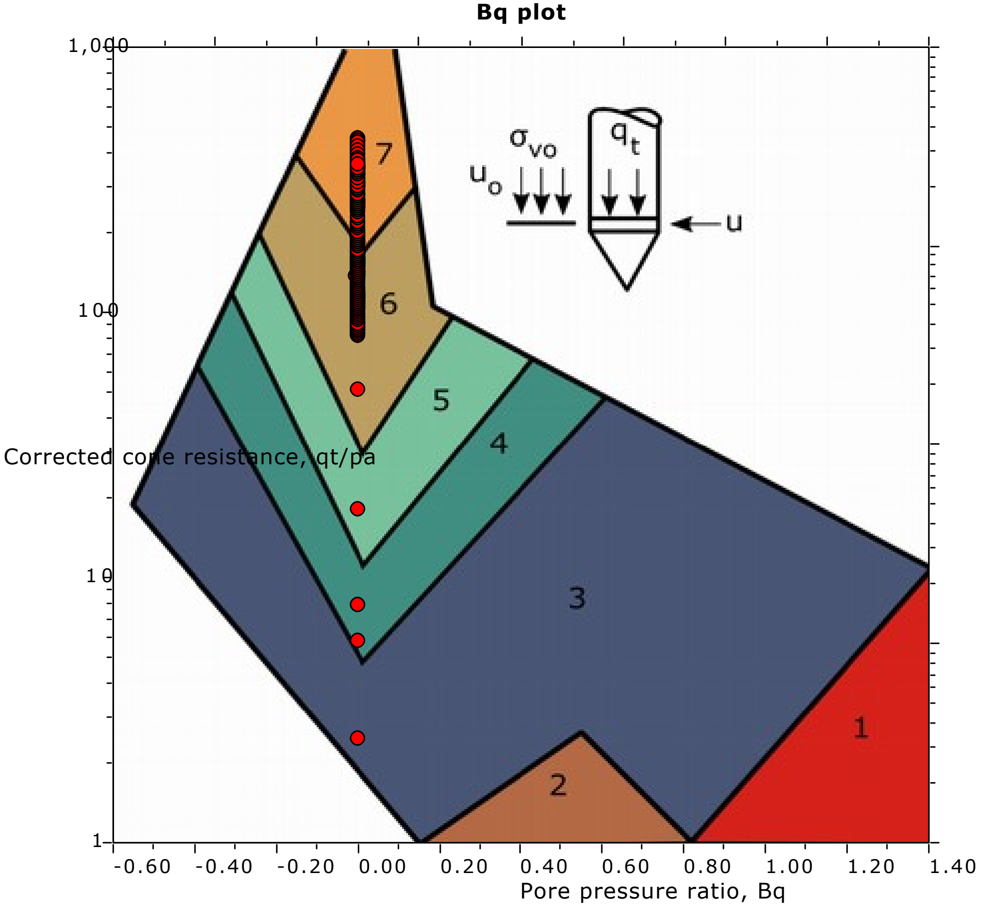
NOVA Services, Inc.
4373 Viewridge, Suite B
San Diego, CA 92123
858-292-7575

Project: **Leucadia Mixed Use**
Location: **Encinitas, CA**

CPT-1
Total depth: 50.27 ft, Date: 10/10/2019
Surface Elevation: 82.00 ft
Cone Type: Vertek
Cone Operator: Kehoe Testing & Engineering

SBT - Bq plots

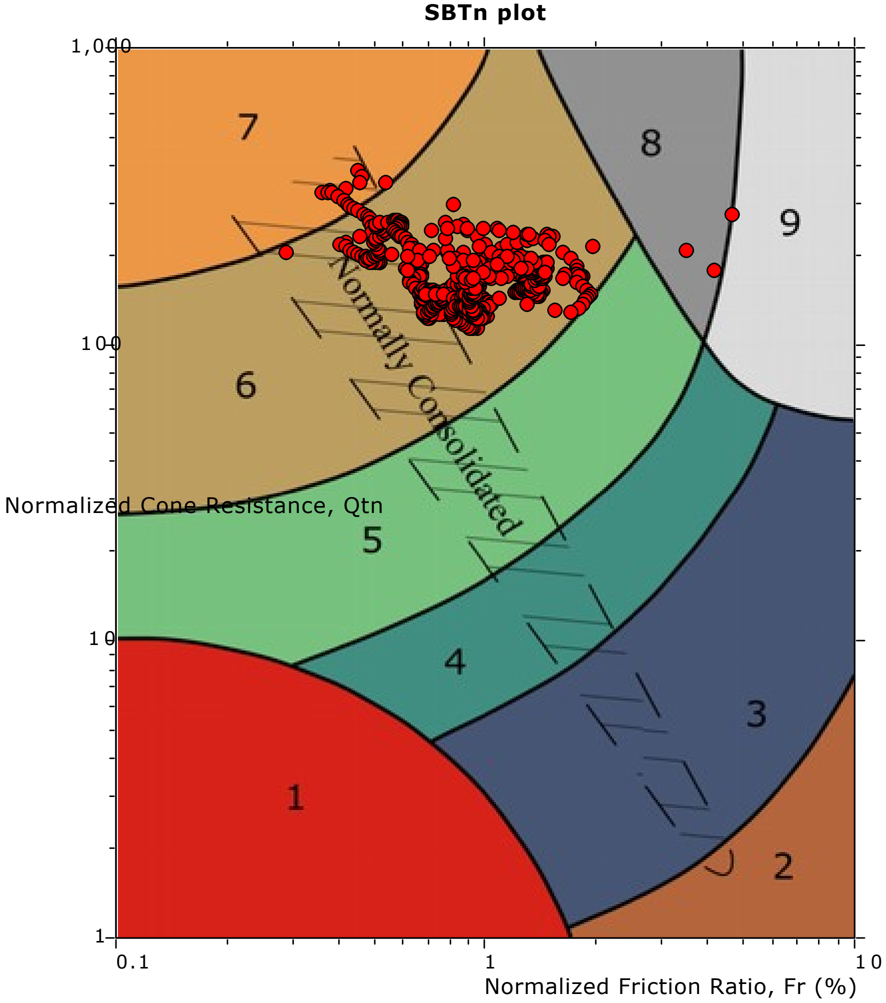


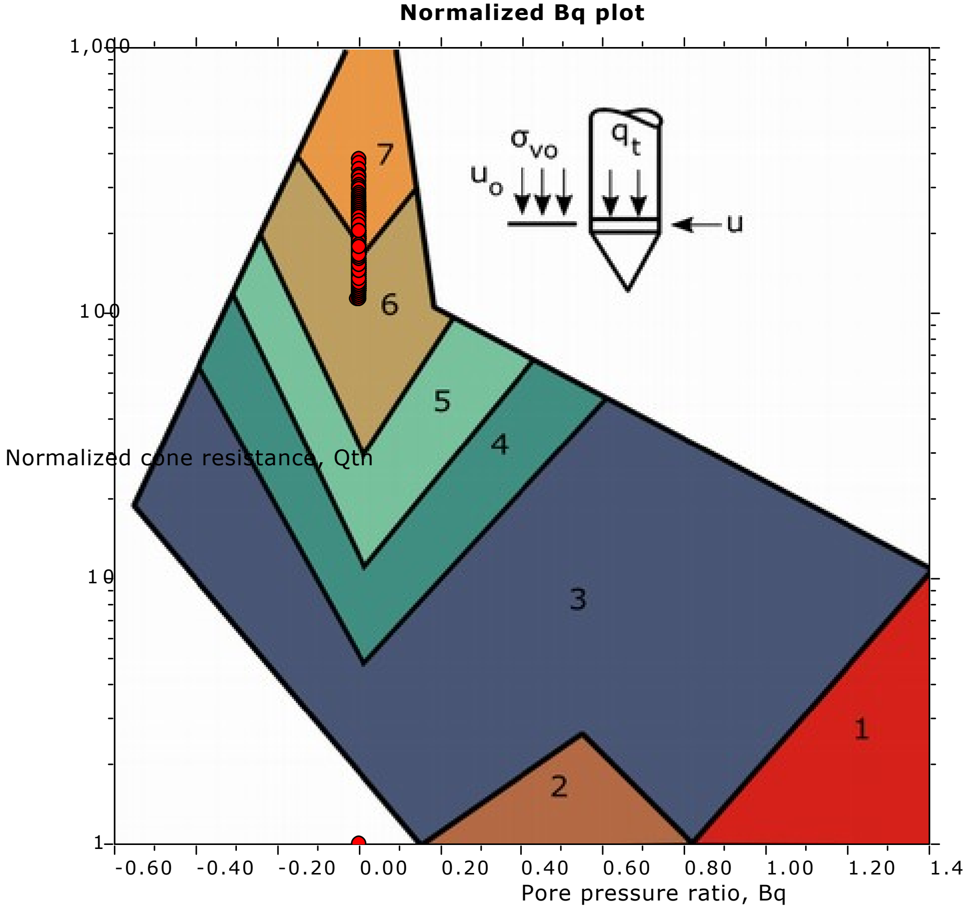


SBT legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

SBT - Bq plots (normalized)





SBTn legend

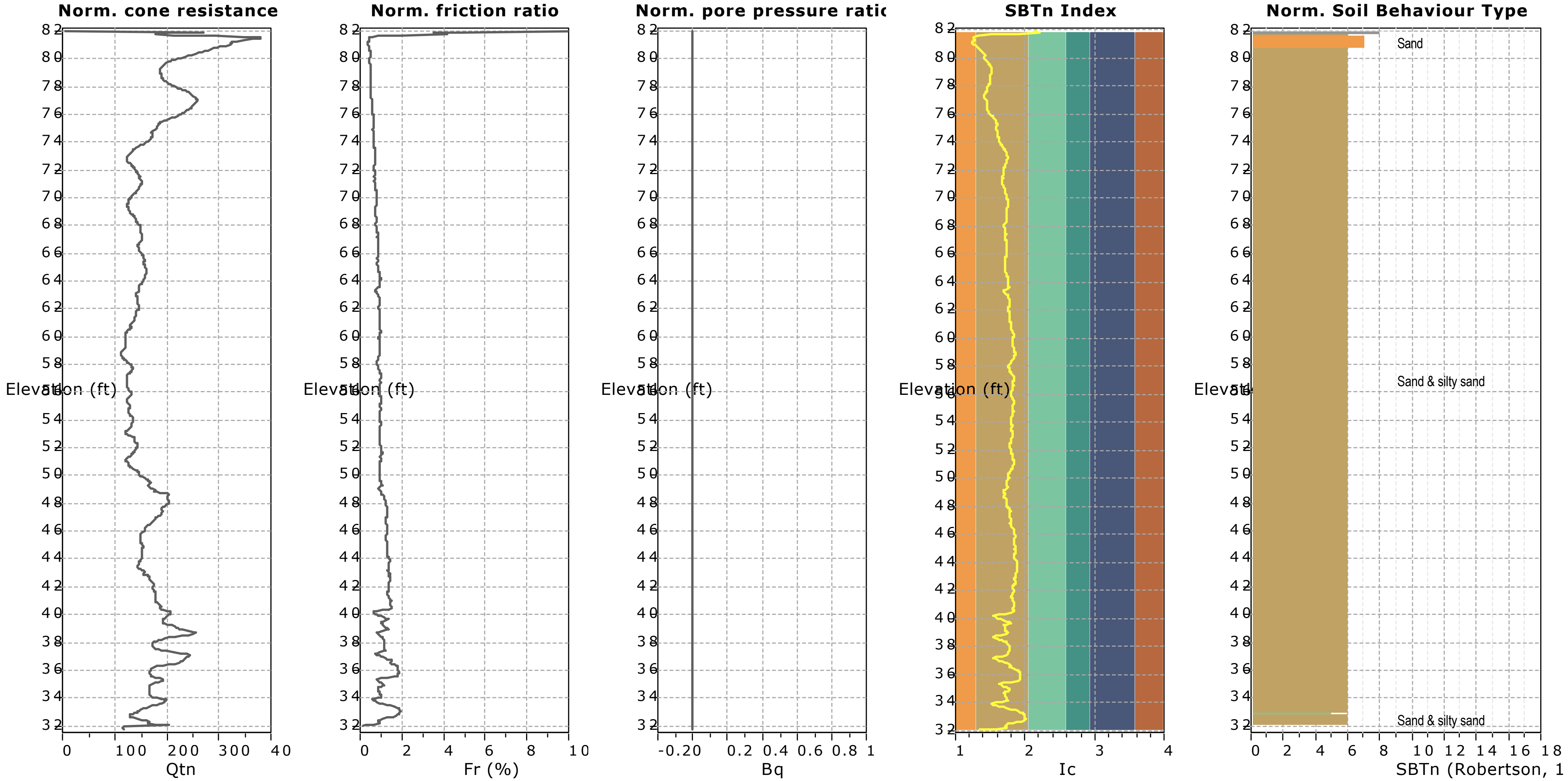
🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

NOVA Services, Inc.
4373 Viewridge, Suite B
San Diego, CA 92123
858-292-7575

Project: **Leucadia Mixed Use**
Location: **Encinitas, CA**

CPT-1

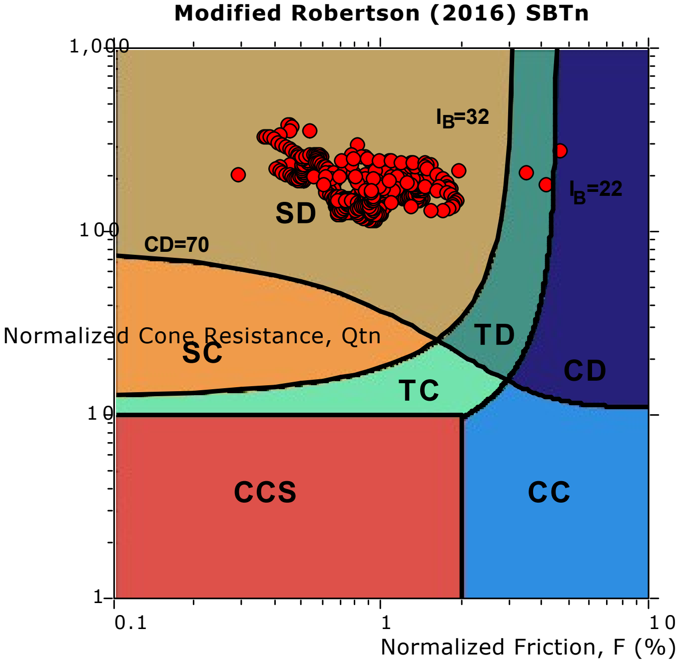
Total depth: 50.27 ft, Date: 10/10/2019
Surface Elevation: 82.00 ft
Cone Type: Vertek
Cone Operator: Kehoe Testing & Engineering

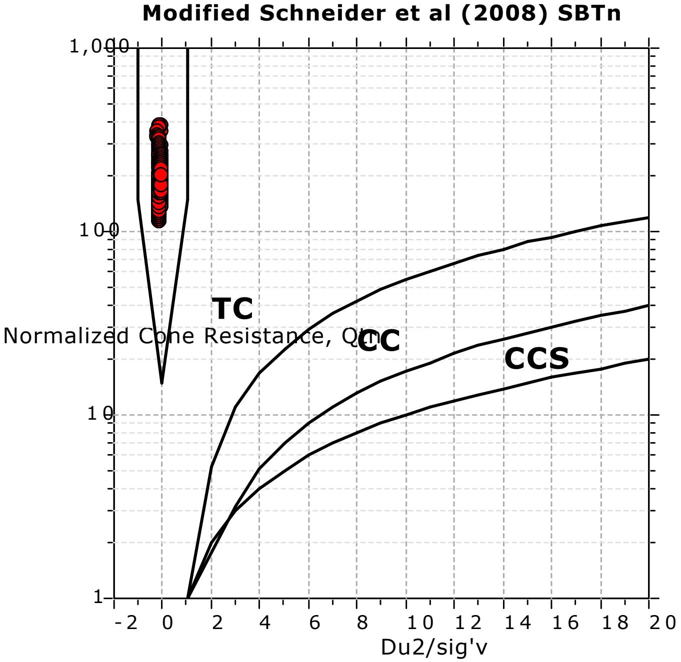


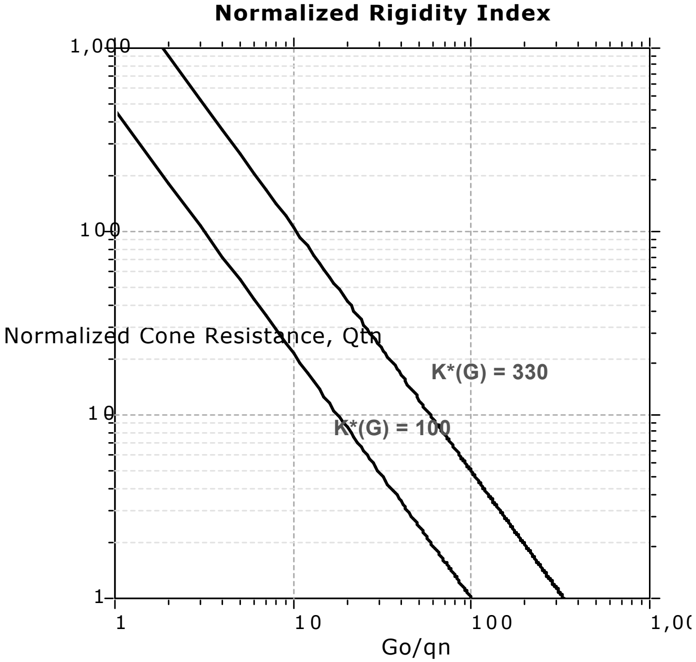
SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬜ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

Updated SBTn plots


Modified Robertson (2016) SBTn


Modified Schneider et al (2008) SBTn


Normalized Rigidity Index

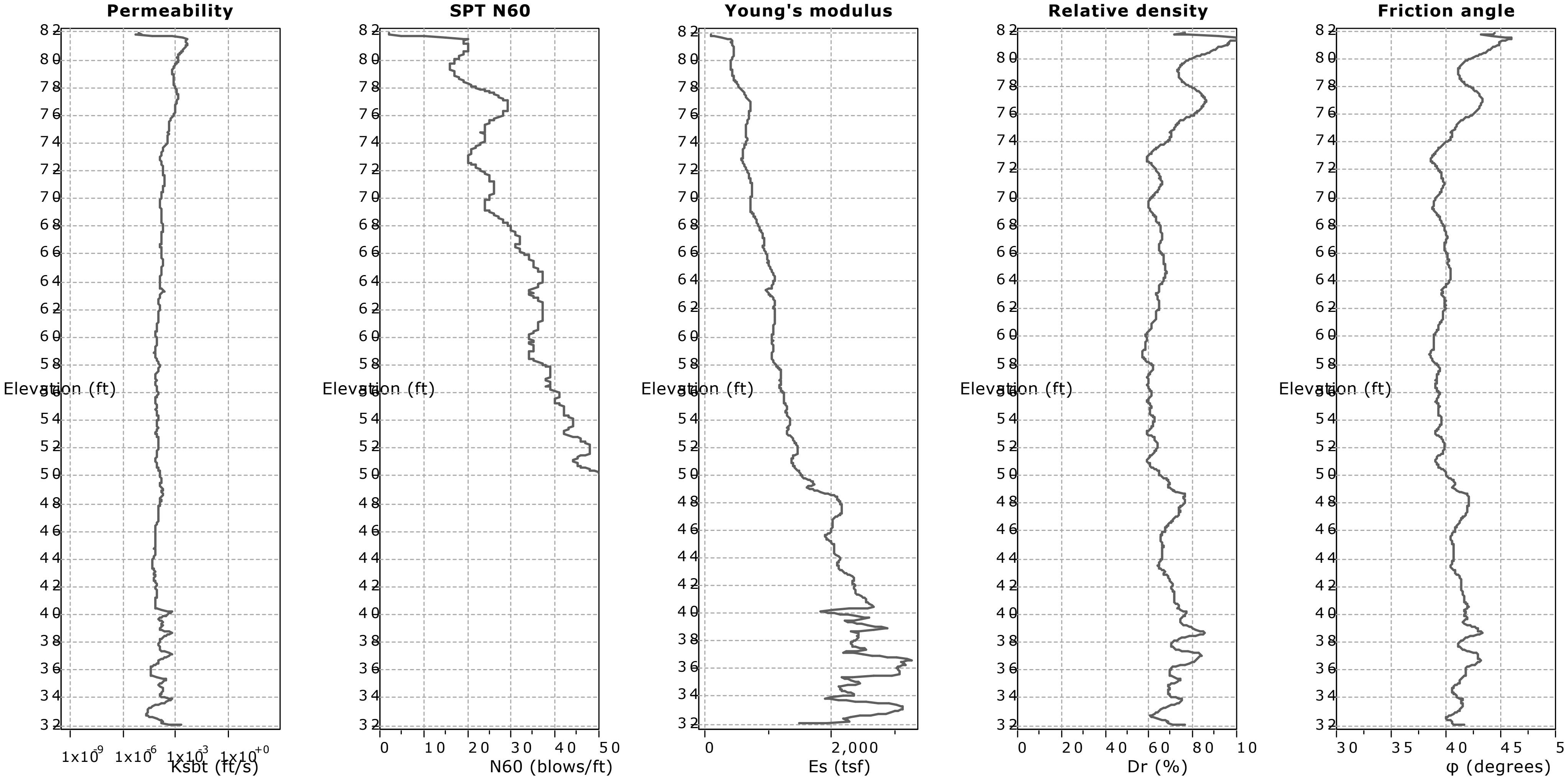
CCS:	Clay-like - Contractive - Sensitive
CC:	Clay-like - Contractive
CD:	Clay-like - Dilative
TC:	Transitional - Contractive
TD:	Transitional - Dilative
SC:	Sand-like - Contractive
SD:	Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

NOVA Services, Inc.
4373 Viewridge, Suite B
San Diego, CA 92123
858-292-7575

Project: **Leucadia Mixed Use**
Location: **Encinitas, CA**

CPT-1
Total depth: 50.27 ft, Date: 10/10/2019
Surface Elevation: 82.00 ft
Cone Type: Vertek
Cone Operator: Kehoe Testing & Engineering



Calculation parameters

Permeability: Based on SBT_n

SPT N_{60}: Based on I_c and q_t

Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

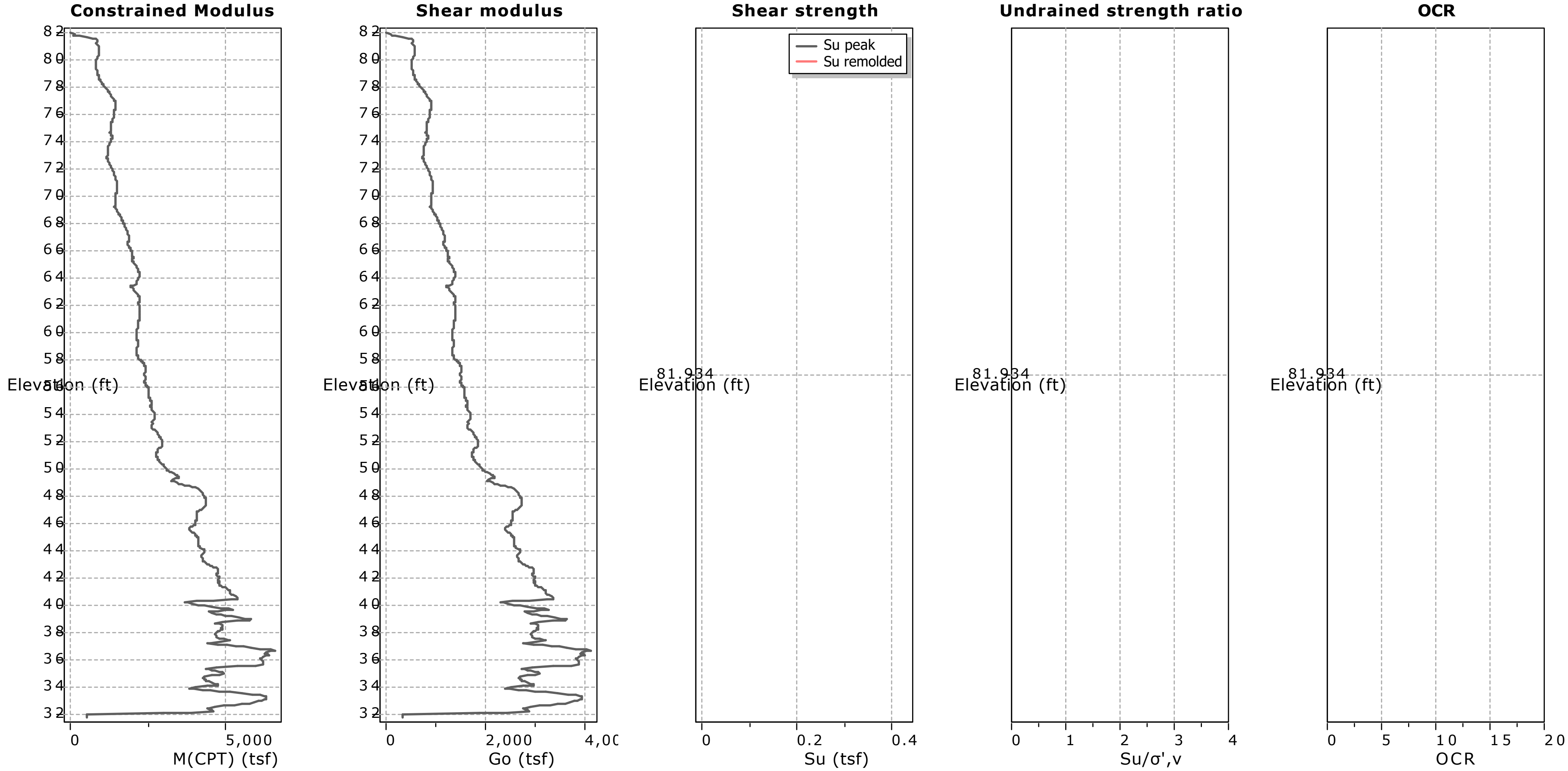
Relative density constant, C_{Dr}: 350.0

Phi: Based on Kulhawy & Mayne (1990)

—●— User defined estimation data

Project: **Leucadia Mixed Use**
Location: **Encinitas, CA**

CPT-1
Total depth: 50.27 ft, Date: 10/10/2019
Surface Elevation: 82.00 ft
Cone Type: Vertek
Cone Operator: Kehoe Testing & Engineering



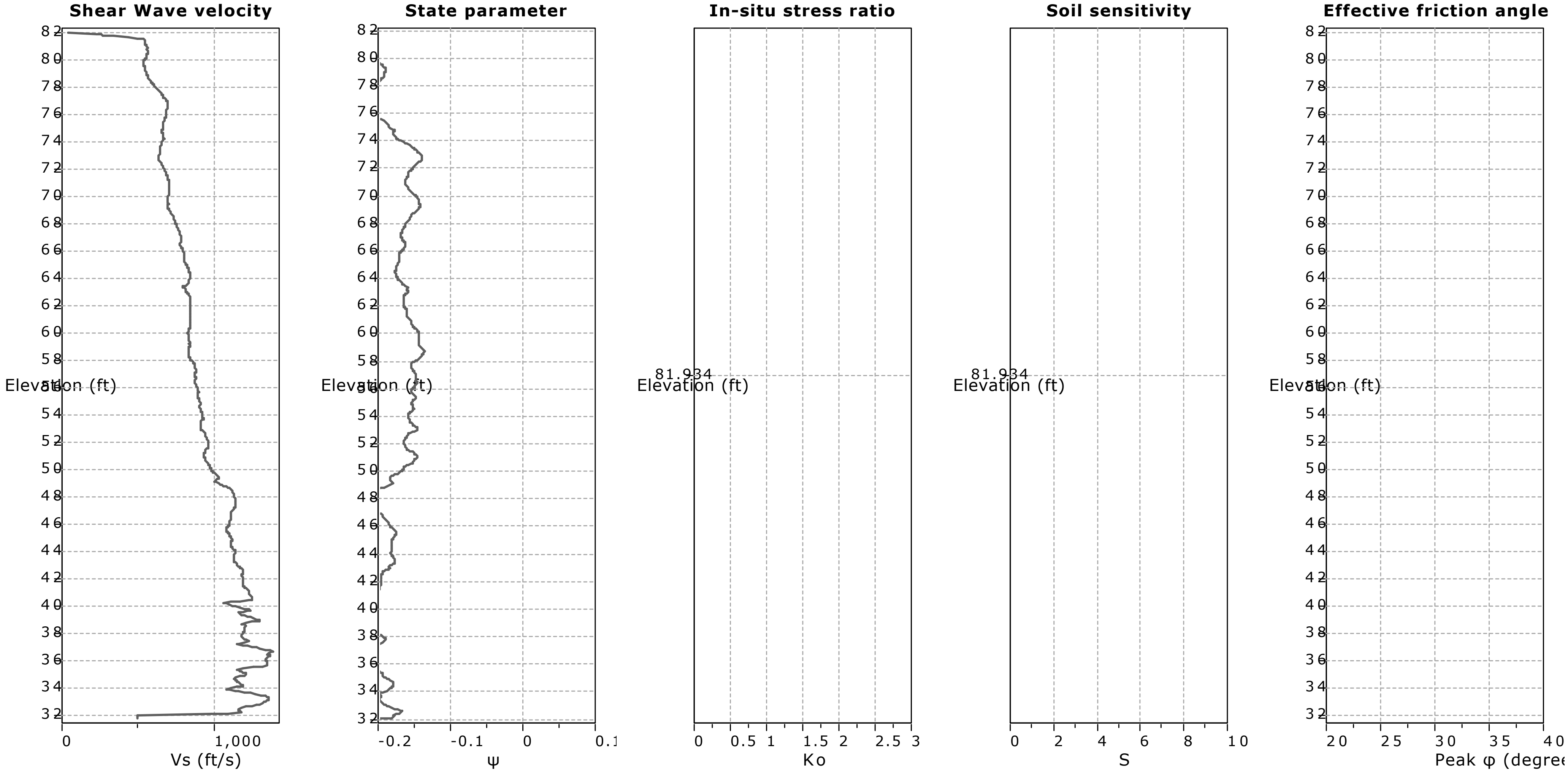
Calculation parameters

Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

OCR factor for clays, N_{kt}: 0.33
●— User defined estimation data
●— Flat Dilatometer Test data

Project: **Leucadia Mixed Use**
Location: **Encinitas, CA**

CPT-1
Total depth: 50.27 ft, Date: 10/10/2019
Surface Elevation: 82.00 ft
Cone Type: Vertek
Cone Operator: Kehoe Testing & Engineering



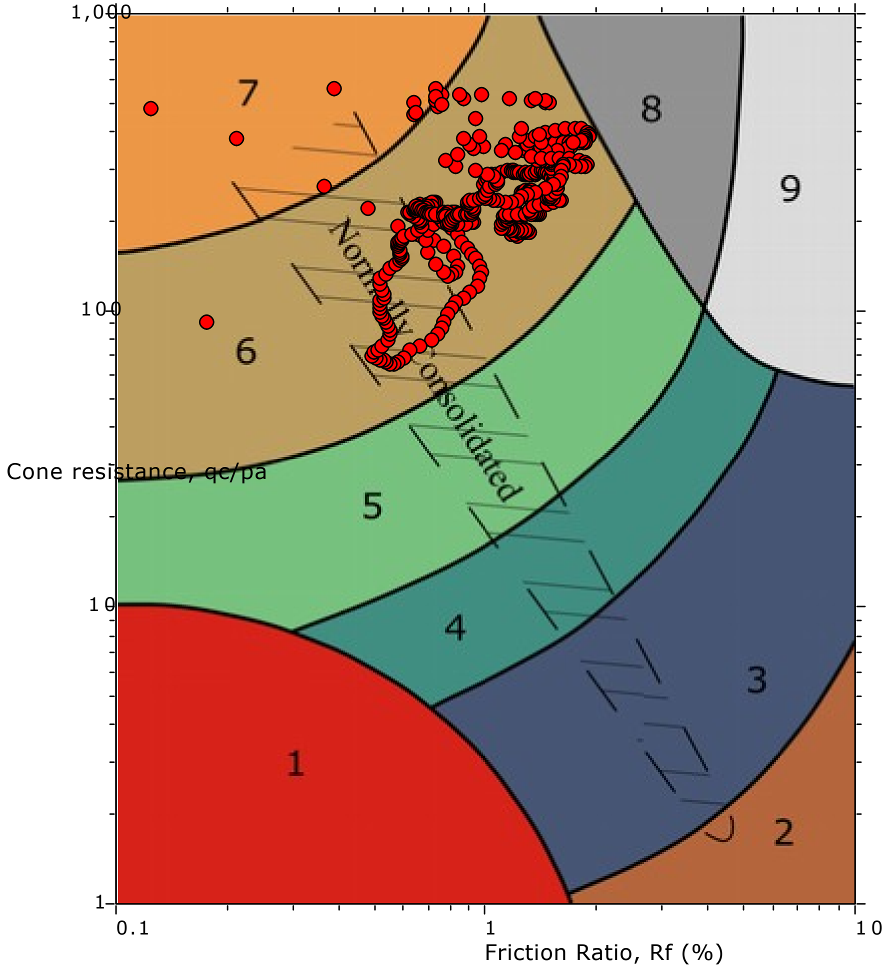
Calculation parameters

Soil Sensitivity factor, N_S: 7.00

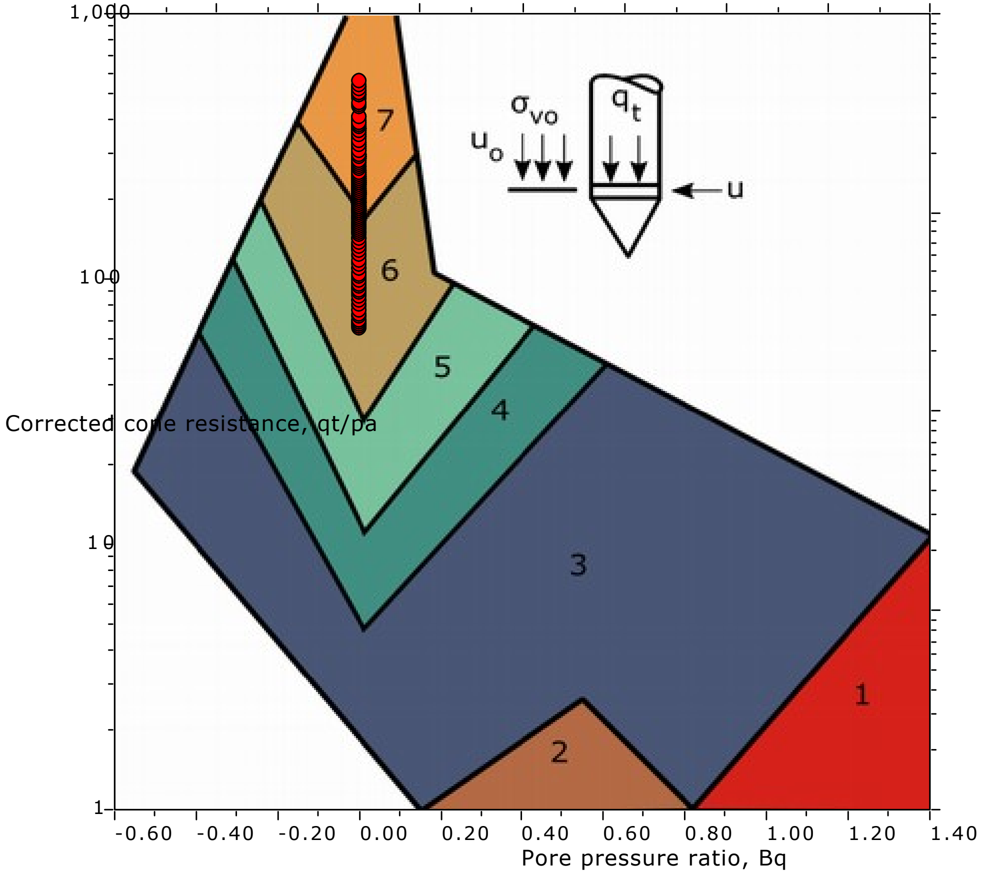
● User defined estimation data

SBT - Bq plots

SBT plot



Bq plot

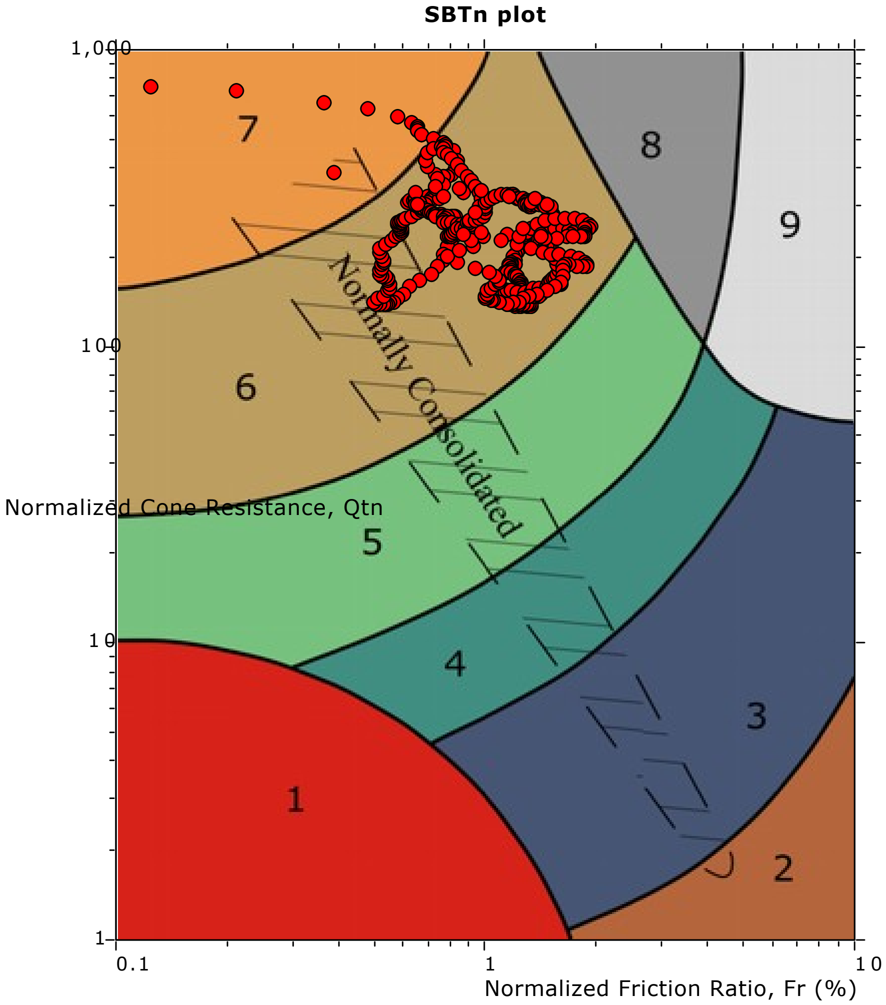


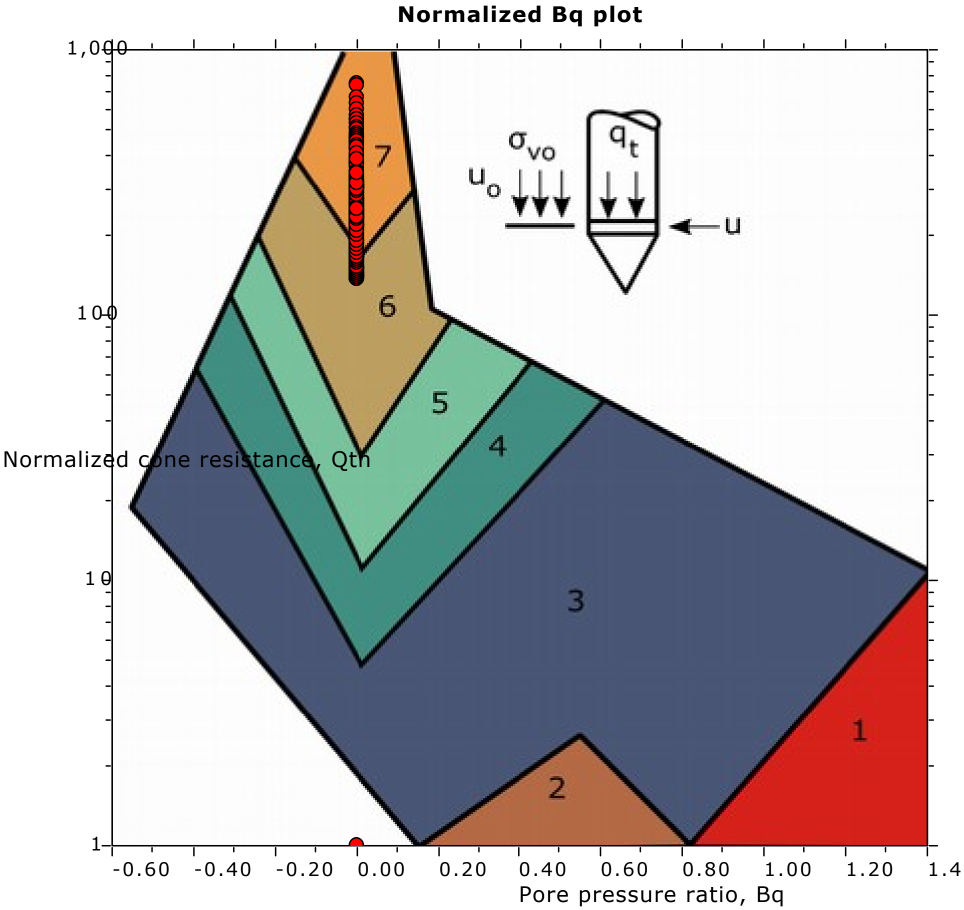
SBT legend

🟥 1. Sensitive fine grained	🟩 4. Clayey silt to silty clay	🟧 7. Gravely sand to sand
🟫 2. Organic material	🟩 5. Silty sand to sandy silt	⬛ 8. Very stiff sand to clayey sand
🟦 3. Clay to silty clay	🟫 6. Clean sand to silty sand	⬜ 9. Very stiff fine grained

CPT-2
Total depth: 37.93 ft, Date: 10/10/2019
Surface Elevation: 68.00 ft
Cone Type: Vertek
Cone Operator: Kehoe Testing & Engineering

SBT - Bq plots (normalized)





SBTn legend

🟥 1. Sensitive fine grained
🟫 2. Organic material
🟦 3. Clay to silty clay
🟩 4. Clayey silt to silty clay
🟩 5. Silty sand to sandy silt
🟫 6. Clean sand to silty sand
🟧 7. Gravely sand to sand
⬛ 8. Very stiff sand to clayey sand
⬜ 9. Very stiff fine grained

NOVA Services, Inc.
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San Diego, CA 92123
858-292-7575

Project: **Leucadia Mixed Use**
Location: **Encinitas, CA**

CPT-2

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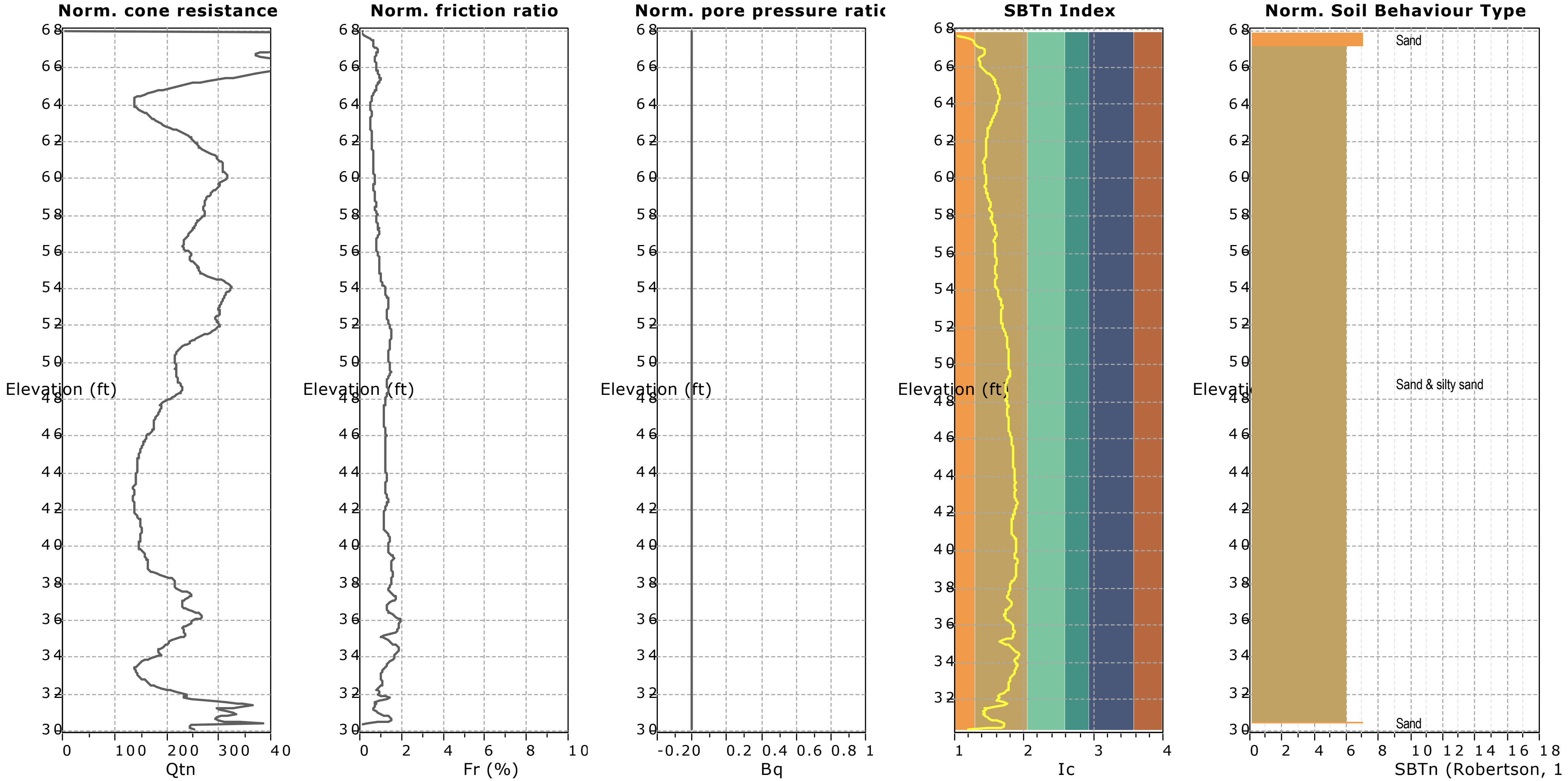
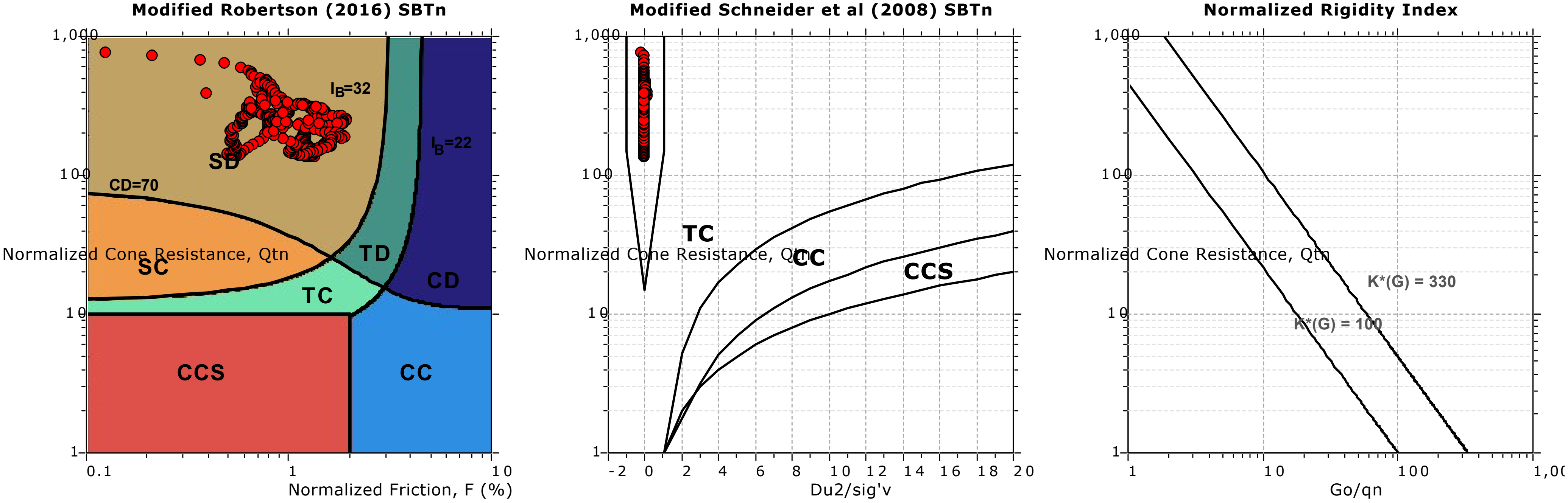
Surface Elevation: 68.00 ft
Cone Type: Vertek
Cone Operator: Kehoe Testing & Engineering



SBTn legend

- 🟥 1. Sensitive fine grained
- 🟫 2. Organic material
- 🟦 3. Clay to silty clay
- 🟩 4. Clayey silt to silty clay
- 🟩 5. Silty sand to sandy silt
- 🟨 6. Clean sand to silty sand
- 🟧 7. Gravely sand to sand
- ⬛ 8. Very stiff sand to clayey sand
- ⬜ 9. Very stiff fine grained

Updated SBTn plots



CCS: Clay-like - Contractive - Sensitive
CC: Clay-like - Contractive
CD: Clay-like - Dilative
TC: Transitional - Contractive
TD: Transitional - Dilative
SC: Sand-like - Contractive
SD: Sand-like - Dilative

K(G) > 330: Soils with significant microstructure (e.g. age/cementation)

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CPT-2
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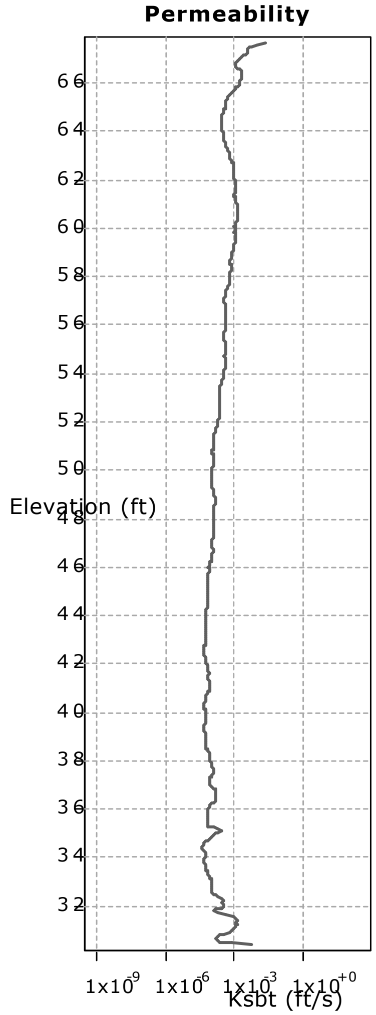

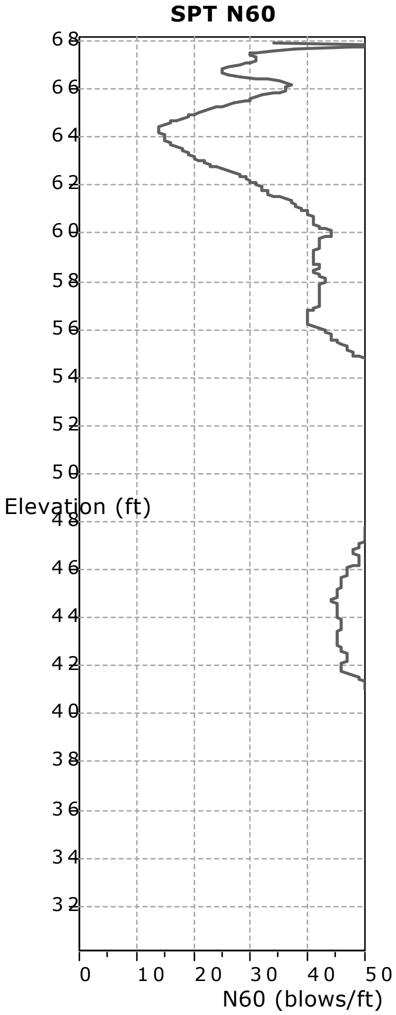

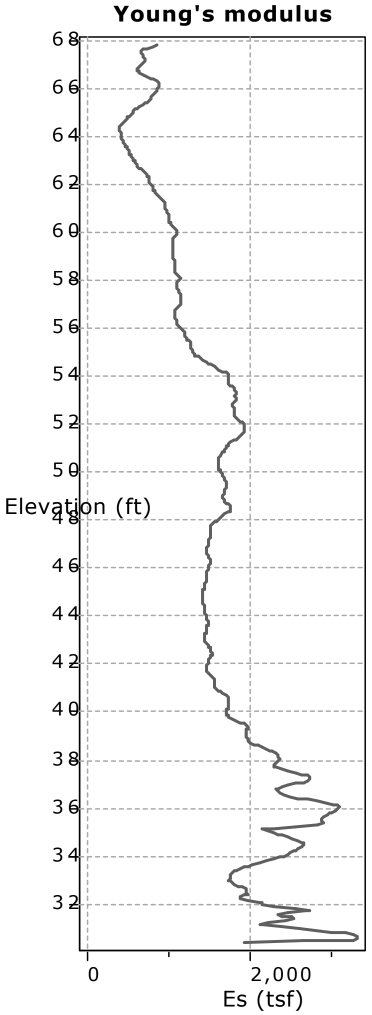

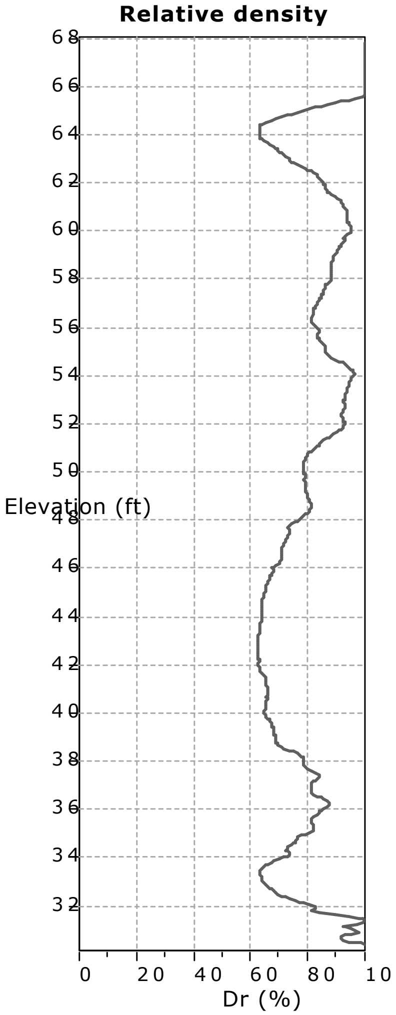

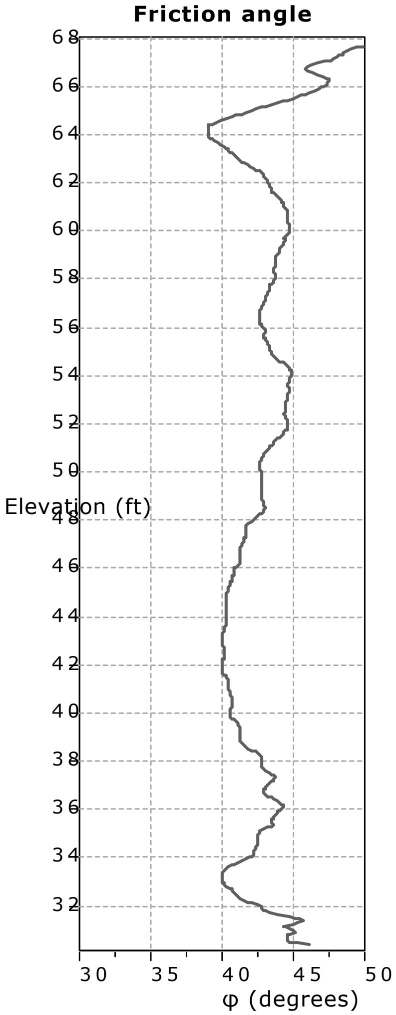

Calculation parameters

Permeability: Based on SBT_n
SPT N_{60}: Based on I_c and q_t
Young's modulus: Based on variable alpha using I_c (Robertson, 2009)

Relative density constant, C_{Dr}: 350.0
Phi: Based on Kulhawy & Mayne (1990)
—●— User defined estimation data

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CPT-2
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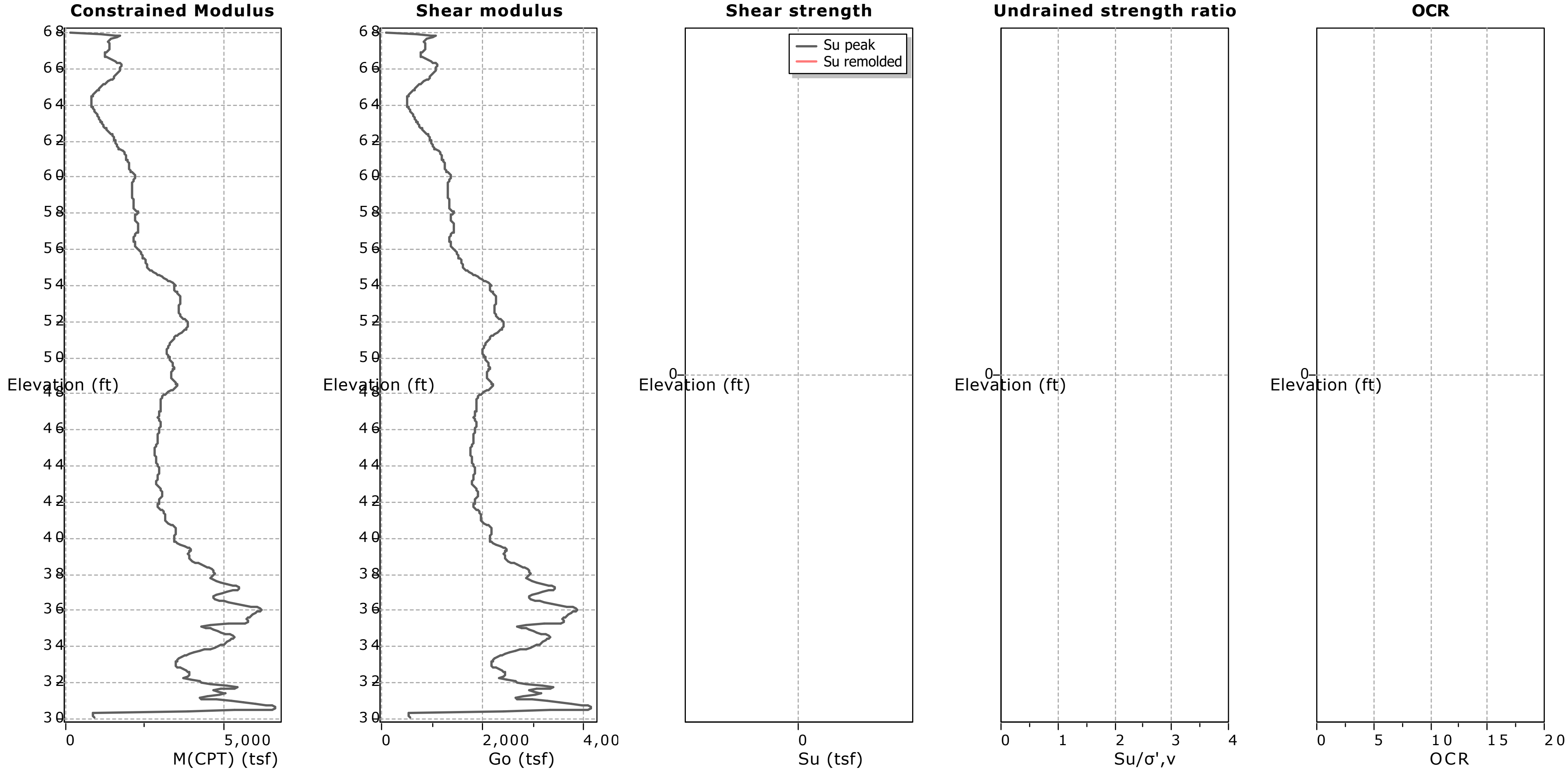

Calculation parameters

Constrained modulus: Based on variable alpha using I_c and Q_{tn} (Robertson, 2009)
Go: Based on variable alpha using I_c (Robertson, 2009)
Undrained shear strength cone factor for clays, N_{kt}: 14

OCR factor for clays, N_{kt}: 0.33
● User defined estimation data
● Flat Dilatometer Test data

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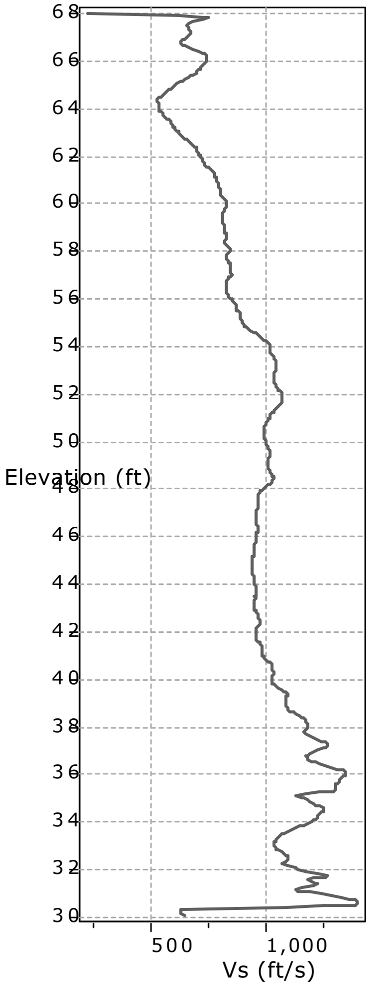
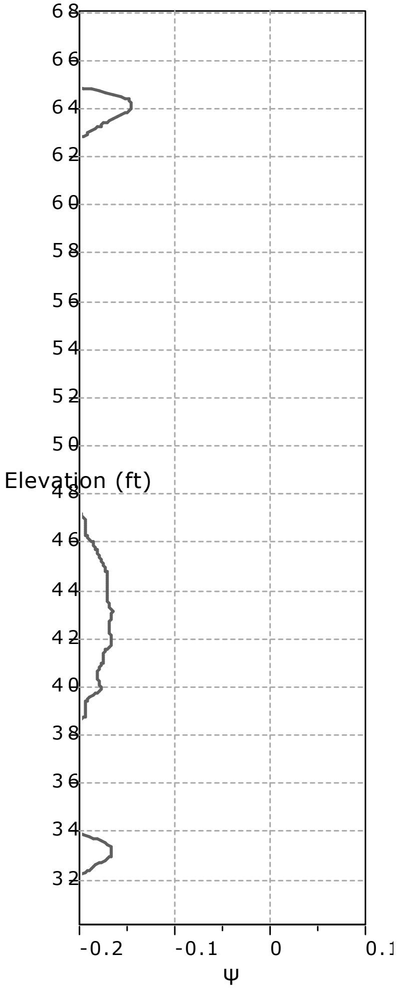
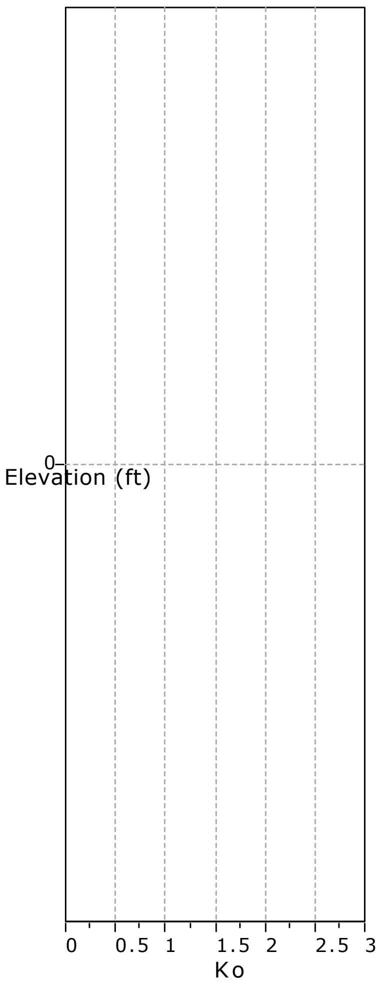
Shear Wave velocity



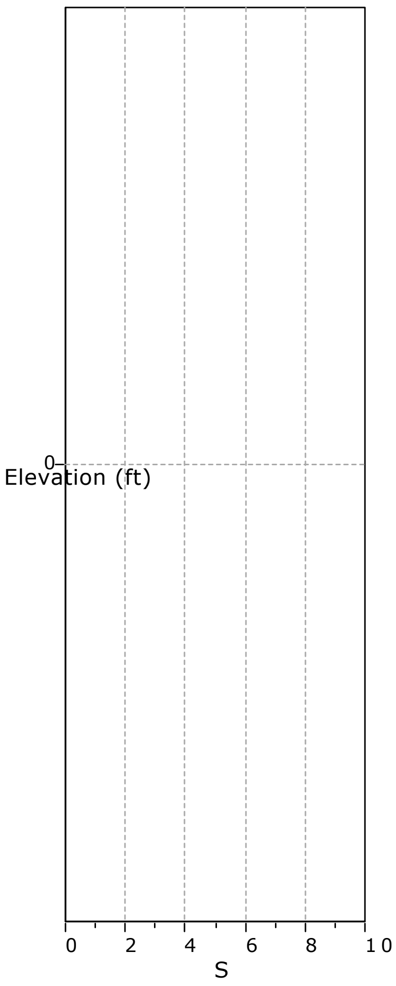
State parameter



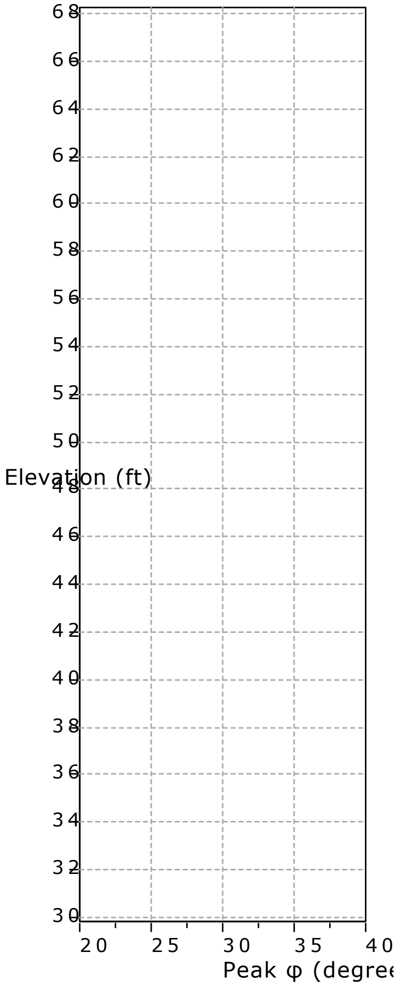
In-situ stress ratio



Soil sensitivity



Effective friction angle



Calculation parameters

Soil Sensitivity factor, N_S: 7.00
—●— User defined estimation data

Presented below is a list of formulas used for the estimation of various soil properties. The formulas are presented in SI unit system and assume that all components are expressed in the same units.

:: Unit Weight, g (kN/m³) ::

$$g = g_w \cdot \left(0.27 \cdot \log(R_f) + 0.36 \cdot \log(\frac{q_t}{p_a}) + 1.236 \right)$$

where g_w = water unit weight

:: Permeability, k (m/s) ::

$I_c < 3.27$ and $I_c > 1.00$ then $k = 10^{0.952-3.04 \cdot I_c}$

$I_c \leq 4.00$ and $I_c > 3.27$ then $k = 10^{-4.52-1.37 \cdot I_c}$

:: N_{SPT} (blows per 30 cm) ::

$$N_{60} = \left(\frac{q_c}{P_a} \right) \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

$$N_{1(60)} = Q_{tn} \cdot \frac{1}{10^{1.1268-0.2817 \cdot I_c}}$$

:: Young's Modulus, Es (MPa) ::

$$(q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.55 \cdot I_c + 1.68}$$

(applicable only to $I_c < I_{c\_cutoff}$)

:: Relative Density, Dr (%) ::

$$100 \cdot \sqrt{\frac{Q_{tn}}{k_{DR}}}$$

(applicable only to SBT_n: 5, 6, 7 and 8 or $I_c < I_{c\_cutoff}$)

:: State Parameter, ψ ::

$$\psi = 0.56 - 0.33 \cdot \log(Q_{tn,cs})$$

:: Peak drained friction angle, φ (°) ::

$$\varphi = 17.60 + 11 \cdot \log(Q_{tn})$$

(applicable only to SBT_n: 5, 6, 7 and 8)

:: 1-D constrained modulus, M (MPa) ::

If $I_c > 2.20$

$a = 14$ for $Q_{tn} > 14$

$a = Q_{tn}$ for $Q_{tn} \leq 14$

$M_{CPT} = a \cdot (q_t - \sigma_v)$

If $I_c \leq 2.20$

$M_{CPT} = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$

:: Small strain shear Modulus, Go (MPa) ::

$$G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.55 \cdot I_c + 1.68}$$

:: Shear Wave Velocity, Vs (m/s) ::

$$V_s = \left(\frac{G_0}{\rho} \right)^{0.50}$$

:: Undrained peak shear strength, Su (kPa) ::

$N_{kt} = 10.50 + 7 \cdot \log(F_r)$ or user defined

$$S_u = \frac{(q_t - \sigma_v)}{N_{kt}}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Remolded undrained shear strength, Su(rem) (kPa) ::

$S_{u(rem)} = f_s$ (applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Overconsolidation Ratio, OCR ::

$$k_{OCR} = \left[\frac{Q_{tn}^{0.20}}{0.25 \cdot (10.50 \cdot + 7 \cdot \log(F_r))} \right]^{1.25}$$ or user defined

$$OCR = k_{OCR} \cdot Q_{tn}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: In situ Stress Ratio, Ko ::

$$K_O = (1 - \sin \varphi') \cdot OCR^{\sin \varphi'}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Soil Sensitivity, S_t ::

$$S_t = \frac{N_S}{F_r}$$

(applicable only to SBT_n: 1, 2, 3, 4 and 9 or $I_c > I_{c\_cutoff}$)

:: Effective Stress Friction Angle, φ<sun>' (°) ::

$$\varphi' = 29.5° \cdot B_q^{0.121} \cdot (0.256 + 0.336 \cdot B_q + \log Q_t)$$

(applicable for $0.10<B_q<1.00$)

References

- Robertson, P.K., Cabal K.L., Guide to Cone Penetration Testing for Geotechnical Engineering, Gregg Drilling & Testing, Inc., 5th Edition, November 2012
- Robertson, P.K., Interpretation of Cone Penetration Tests - a unified approach., Can. Geotech. J. 46(11): 1337–1355 (2009)

APPENDIX G
SITE-SPECIFIC GROUND MOTION HAZARD ANALYSIS

SITE - SPECIFIC SEISMIC GROUND MOTION HAZARD ANALYSIS

Subject: Report
Site - Specific Ground Motion Hazard Analysis Leucadia Mixed-Use Development, 1950 North Coast Highway 101

References: *ASCE/SEI 7-16 Minimum Design Loads and Associated Criteria for Buildings and Other Structures*, American Society of Civil Engineers, ISBN 9780784414248, 2017.

Pacific Earthquake Engineering Research Center, *Weighted Average of 2014 NGA West-2 GMPEs,* Seyhan, E., April 14, 2015.

Kramer, S., *Geotechnical Earthquake Engineering,* ISBN 978-81-317-0718-0, Pearson Education, Inc. 1996.

Pacific Earthquake Engineering Research Center (PEER), NGA-West2 Excel File of Five Horizontal Ground Motion Prediction Equations.

USGS, *Unified Hazard Tool*, found at: https://earthquake.usgs.gov/hazards/interactive/.

USGS, *Risk-Targeted Ground Motion Calculator*, found at: https://earthquake.usgs.gov/designmaps/rtgm/.

USGS, *Quaternary Fault and Fold Database of the United States*, found at: https://earthquake.usgs.gov/hazards/qfaults/.

USGS, 2013, *Open-File Report 2013-1165*, *Uniform California Earthquake Rupture Forecast, Version 3 (UCERF3), Fault Section Data*

SEAOC/OSHPD, *Seismic Design Maps,* found at: https://seismicmaps.org/.

BACKGROUND

It has been known for some time[1] that seismic design parameters determined as described in ASCE 7-10 Chapter 11 have the potential to underestimate potential accelerations for structures founded on Site Classes D, E, and F. The recent code update of Sections 11 and 21 in ASCE 7-16 is intended to mitigate these shortcomings by requiring a Site-Specific Seismic Ground Motion Hazard Analysis (SGMHA). The SGMHA is used for quantitative estimations of ground motion characteristics at a specific site by incorporating several site-specific variables, principally distance from fault, site shear wave velocity, and fault geometry.

The SGMHA includes the following principal elements of analyses and evaluation:

- field determination of the site class of the subject site,
- Probabilistic Seismic Hazard Analysis (PSHA),
- Deterministic Seismic Hazard Analysis (DSHA), and
- determining design acceleration parameters using the resulting acceleration spectra.

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[1] For example, see Kircher, C. A., *New Site-Specific Ground Motion Requirements of ASCE 7-16,* Proceedings, SEAOC Convention, 2017

The PSHA allows the uncertainties in size, location, and rate of recurrence of earthquakes and the variation of ground motion characteristics to be considered in the seismic evaluation. A DSHA involves development of an evaluation of ground motion hazard at a site based on a scenario in which an earthquake of a specified size occurring at a specified location occurs. This procedure provides a framework for evaluating worst-case ground motions (Kramer 1996).

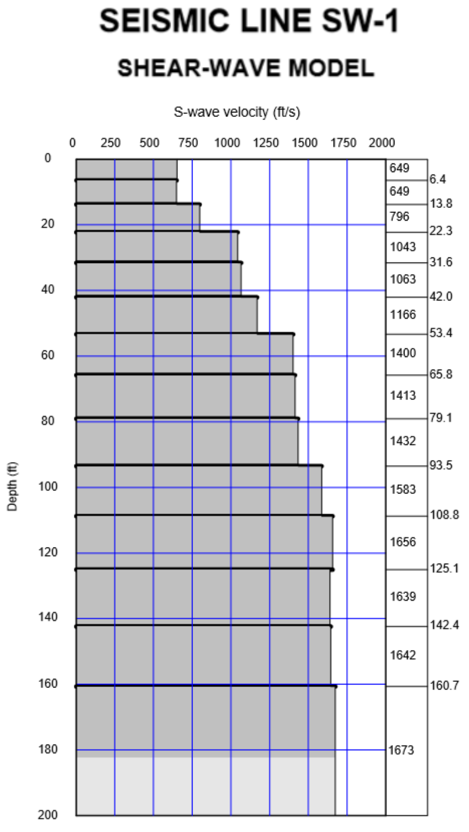
NOVA has completed a SGMHA for the subject site in accordance with CBC 2019 and ASCE 7- 16. This report provides a summary of NOVA's procedure and results of the SGMHA for the site.

PROCEDURE

Site Classification

A seismic shear wave survey of the subject site was completed on March 7, 2020. The objective of this survey was to determine the site class based on shear wave velocities of the upper 30 meters (~100 feet) of the underlying soils, referred to as either V_{s30} or V_{100}.

The measured shear wave velocities were found to average 1,077.6 feet/second within the soils/bedrock of the upper 100 feet of the site. Results of this analysis are shown in Figure 1. Using Table 20.3-1 from ASCE 7-16, the site bedrock is determined to be Site Class D.



SHEAR-WAVE VELOCITY MODEL: Average Vs 100ft = 1,077.6 ft/sec

Site Classification (CBC-2019 1613A.2.2 & ASCE 7-16 Ch. 20)- "D" (Stiff Soil)

Figure 1. Shear Wave Velocities for the Subject Site

Probabilistic Hazard Analysis

The Probabilistic Seismic Hazard Analysis (PSHA) was completed using tools provided by USGS for this purpose. Site-specific parameters including site location by latitude and longitude (33.080933°N, -117.308435°W) site class, and probability of an earthquake with 2% exceedance in 50 years were input into the Unified Hazard Tool. Peak Ground Acceleration was selected for Spectral Period and the default Time Horizon of 2,475 years was used. The earthquake fault dataset selected for the calculations performed by the tool was Dynamic: Conterminous U.S. 2014 (update) (v4.2.0) Edition. Calculations provided spectral acceleration values for periods between 0 and 5 seconds.

The computed values were then input into the USGS Risk-Targeted Ground Motion (RTGM) Calculator and recorded at periods shown in Table 1. Maximum Direction Scale Factors were then applied using coefficients specified in ASCE 7-16 Section 21.2 for varying periods. Maximum Direction RTGM was then calculated as the product of RTGM and the Maximum Direction Scale Factor as shown in Table 1.

Table 1. Probabilistic Seismic Hazard Analysis Values

Period (s)	Risk Targeted GM (g)	Max Dir Scale Factor	Max Direction RTGM (g)
0	0.538	1.1	0.592
0.1	0.925	1.1	1.018
0.2	1.254	1.1	1.379
0.3	1.366	1.125	1.537
0.5	1.274	1.175	1.497
0.75	1.015	1.2375	1.256
1	0.824	1.3	1.071
2	0.425	1.35	0.574
3	0.269	1.4	0.377
4	0.186	1.45	0.270
5	0.138	1.5	0.207

Figure 2 (following page) provides the probabilistic site response based on the method described above.

Deterministic Hazard Analysis

The Deterministic Seismic Hazard Analysis (DSHA) located the nearest active fault to the site using the USGS KML fault database overlain on Google Earth. Other active faults in the region were evaluated to ensure the correct controlling fault was used in the analysis. The nearest active fault was the Silver Strand section of the Rose Canyon Fault Zone at a distance of 6.42 km from the site.

The PEER NGA-West2 excel file with five models calculating horizontal ground motion was used in the DSHA. The file provides the weighted average of peak values and the response spectra of the NGA-West2 horizontal ground motion prediction equations. NOVA used four of the five available models in the evaluation. The following four were weighted at 25% contribution: Abrahamson et al., Boor et al., Campbell and Bozorgnia, and Chiou and Youngs.

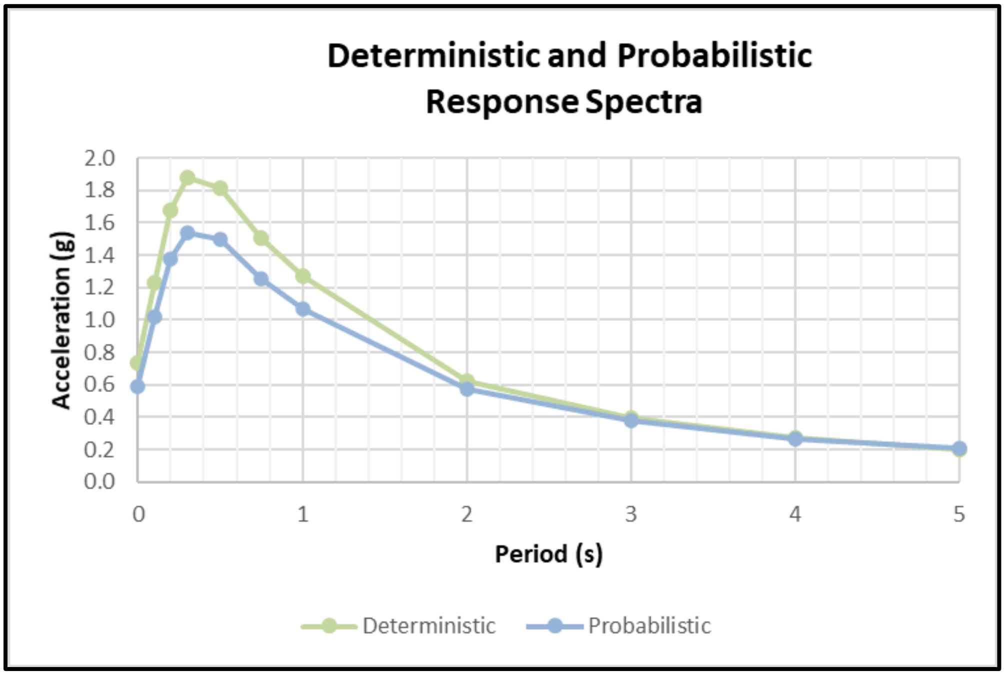


Figure 2. Probabilistic and Deterministic Seismic Accelerations

Fault-specific parameters inputted into this spreadsheet were retrieved from the USGS Fault Section Data Database (USGS 2013). Parameters specific to the Rose Canyon Fault included Lower Seismic Depth (7.7 km) and Dip Angle (90°), and Earthquake Magnitude. Earthquake magnitude was determined using deaggregation modeling provided by the USGS Unified Hazard Tool, which yielded a modal magnitude of 6.9.

Shear wave velocity in the upper 30 meters (V_{S30}), determined from the site seismic shear-wave survey, was also input into the model. The deterministic spectral response acceleration at each period was calculated as an 84th percentile 5% damped spectral response acceleration. These values were multiplied by the same Maximum Direction Scale Factors applied in the PSHA to produce the Maximum Direction Deterministic Spectral Accelerations. For simplicity of data presentation, the same periods were selected as those used in the PSHA.

The values for the deterministic accelerations are shown in Table 2. Figure 2 depicts the Deterministic and Probabilistic curves graphically. Per Section 21.2.3, the MCE_R is taken as the lesser of the spectral response accelerations from the PSHA and the DSHA; and therefore, the PSHA accelerations control for this site-specific analysis.

Table 2. Deterministic Seismic Hazard Analysis Values

Period (s)	84th Percentile 5% Dampening	Max Dir Scale Factor	Max Direction Deterministic Spectral Acceleration (g)
0	0.669	1.1	0.736
0.1	1.121	1.1	1.233
0.2	1.522	1.1	1.674
0.3	1.672	1.125	1.880
0.5	1.541	1.175	1.810
0.75	1.216	1.2375	1.505
1	0.975	1.3	1.267
2	0.458	1.35	0.619
3	0.284	1.4	0.398
4	0.189	1.45	0.274
5	0.132	1.5	0.198

Design Response Spectrum

Per ASCE 7-16 Section 21.3, the design spectral response acceleration at any period is calculated as 2/3 the MCE_R acceleration, or 2/3 of the probabilistic response spectra, in this case.

Per ASCE 7-16 Section 21.3, the spectral response calculated above, shall not be less than 80% of those determined in accordance with Section 11.4.6, where F_a is determined by Table 11.4-1, and F_v = 2.5 (based on Site Class D and S_1>0.2).

Figure 3 presents the design response spectrum and the 80% code-based design response spectrum, which confirms site-specific design accelerations exceed the 80% design response at all periods.

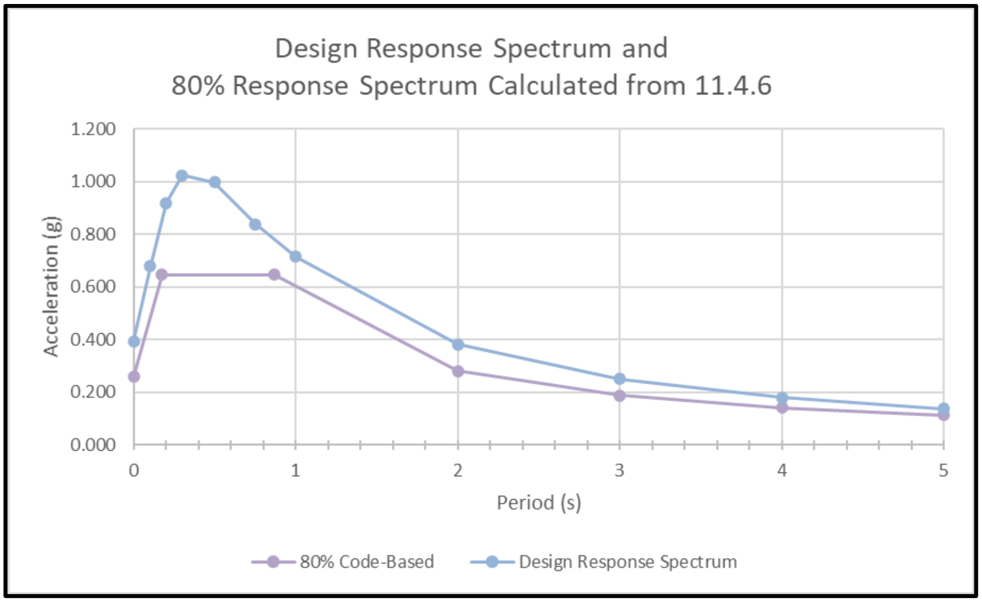


Figure 3. Design Response Spectrum and 80% Code- Based Design Response Spectrum

Design Acceleration Parameters

Following Section 21.4 of ASCE 7-16, S_{DS} was taken as 90% of the maximum spectral acceleration (S_a) from the PSHA over the periods 0.2s to 5s. S_{D1} was taken as the maximum product value of period and spectral acceleration for the period, calculated over the periods 1s through 5s. The parameters S_{MS} and S_{M1} were calculated as 1.5 times S_{DS} and S_{D1}, respectively.

The values calculated were confirmed not to be less than 80% of the values determined in accordance with Section 11.4.3 of ASCE 7-16 for S_{MS} and S_{M1} and Section 11.4.5 for S_{DS} and S_{D1}. The calculated values of S_{DS}, S_{D1}, S_{MS}, and S_{M1} are shown in Table 3.

Table 3. Site-Specific and Code-Based Design Acceleration Parameters

Parameter	Site Specific	OSHPD and Table 11.4.6
S_{MS}	1.38	1.26(1)
S_{M1}	1.15	0.79(2)
S_{DS}	0.92	0.84(3)
S_{D1}	0.77	0.53(4)
S_S	1.18	1.18(5)
S_1	0.42	0.42(5)
Fa	N/A	1.07(6)
Fv	N/A	1.88(6)

[1] Equation 11.4-1 [2] Equation 11.4-2 [3] Equation 11. 4-3
[4] Equation 11.4-4 [5] Source OSHPD Seismic Design Maps https://www.seismicmaps.org/
[6] Linear interpolation of Table 11.4.6

Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration

The probabilistic peak ground acceleration was determined according to Section 21.5.1 using the Risk Targeting Ground Motion Tool for the Unified Hazard Ground Motion at a period of 0s. This calculator presents the geometric mean peak ground acceleration with a 2% probability of exceedance within a 50-year period. The resulting acceleration is 0.59 g.

The deterministic peak ground acceleration was determined according to Section 21.5.2 and calculated as the largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region. PGA was calculated as the point in the DSHA where the period is equal to 0s, resulting in spectral acceleration of 0.67 g.

The site-specific peak ground acceleration (PGA_M) was taken as the lesser of the probabilistic and deterministic peak ground accelerations. In accordance with code, it was confirmed that PGA_M was not taken as less than 80% of PGA_M determined from Eq. 11.8-1.

Table 4. Calculated and Code Based MCE_G

Parameter	Calculated	OSHPD	80% OSHPD
MCE_G	0.59	0.58	0.46