11.5 Geotechnical Investigation

GEOTECHNICAL INVESTIGATION

Proposed NCTD Oceanside Transit Center Redevelopment

Oceanside, CA

Toll Brothers Apartment Living
23422 Mill Creek Drive, Suite 105
Laguna Hills, CA 92653

Toll Brothers
APARTMENT LIVING

NOVA Project No. 2021010
August 11, 2021

NOVA Services

4373 Viewridge Avenue
Suite B
San Diego, California 92123
858.292.7575

944 Calle Amanecer
Suite F
San Clemente, CA 92673
949.388.7710

www.usa-nova.com

Gary Hildabrand
Toll Brothers Apartment Living
23422 Mill Creek Drive, Suite 105
Laguna Hills, CA 92653

August 11, 2021
NOVA Project No. 2021010

Subject: Geotechnical Investigation
Proposed North County Transit District (NCTD) Oceanside Transit Center Redevelopment
Oceanside, CA

Dear Mr. Hildabrand:

NOVA Services, Inc. (NOVA) is pleased to present this report describing the geotechnical investigation and storm water infiltration testing performed for the Oceanside Transit Center redevelopment project. NOVA conducted the geotechnical investigation in general conformance with the scope of work presented in the proposal dated December 17, 2020 as authorized on February 16, 2021 and change order request dated June 17, 2020 as authorized on July 6, 2021.

This site is considered geotechnically suitable for the proposed development provided the recommendations within this report are followed.

NOVA appreciates the opportunity to be of service to Toll Brothers Apartment Living on this project. If you have any questions regarding this report, please do not hesitate to contact the undersigned at 858.292.7575 x 413.

Sincerely,
NOVA Services, Inc.

REGISTERED PROFESSIONAL ENGINEER
THOMAS B. CANADY
No. 50057
Exp. 6/30/2023
CIVIL
STATE OF CALIFORNIA

Tom Canady, PE
Principal Engineer

CERTIFIED ENGINEERING GEOLOGIST
MELISSA STAYNER
2707
STATE OF CALIFORNIA

Melissa Stayner, PG, CEG
Senior Engineering Geologist

Allen Rekani, GIT
Staff Geologist

GEOTECHNICAL INVESTIGATION

Proposed NCTD Oceanside Transit Center Redevelopment
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TABLE OF CONTENTS

List of Plates

List of Appendices

List of Figures

List of Figures (continued)

List of Tables

1. INTRODUCTION

This report presents the results of the geotechnical investigation NOVA performed for the Oceanside Transit Center redevelopment project. At this time, plans indicate that the project will consist of construction of a new mixed-use development which will include a new NCTD transit center and headquarters, residential apartment buildings, retail space, public plazas and parks, a parking structure, and a luxury hotel. The purpose of this work is to provide conclusions and recommendations regarding the geotechnical aspects of the project. Figure 1-1 presents the site vicinity map and Figure 1-2 (following page) presents the site location map.

Figure 1-1. Site Vicinity Map

Figure 1-2. Site Location Map

2. SCOPE OF WORK

2.1. Field Investigation

2.1.1 Overview

NOVA's field investigation consisted of drilling six (6) borings (B-1 through B-6) to depths up to about 18½ feet below ground surface (bgs) using a truck-mounted drill rig as well as a track-mounted limited access drill rig equipped with a hollow stem auger. Two (2) percolation borings were drilled (P-1 and P-2) to evaluate feasibility of stormwater infiltration.

Difficult drilling conditions and auger refusal on the indurated San Onofre Breccia was encountered in all of the geotechnical borings. As such, seismic refraction surveys were performed at four locations (S-1 through S-4) in order to inform depth to the breccia and the rippability of the breccia during excavations of the subsurface parking lots.

A shear wave velocity survey was conducted for characterization of Site Class based on the shear wave velocity in the upper 100 feet of the site soils and rock.

Figure 2-1 presents the approximate locations of the geotechnical borings, percolation test wells, seismic refraction surveys, and shear wave traverse.

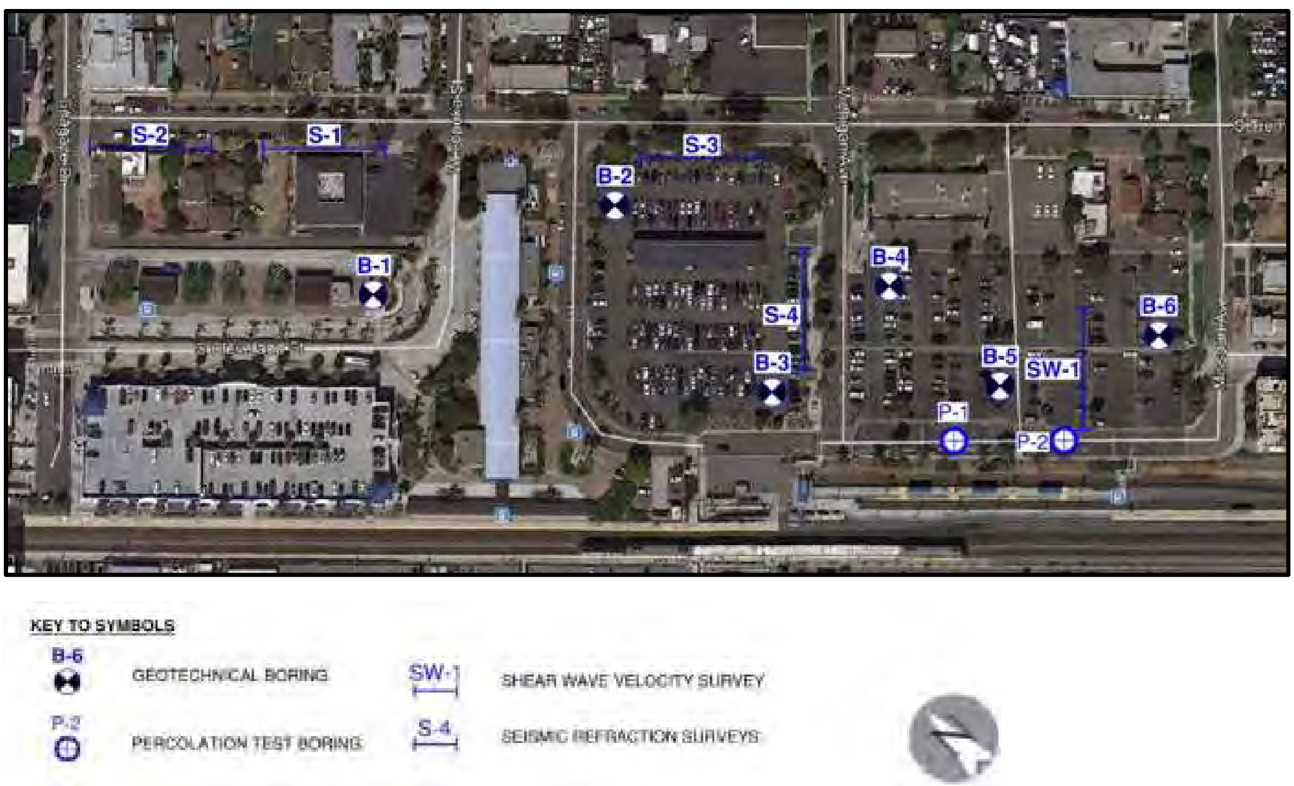

Figure 2-1. Locations of Subsurface Explorations

2.1.2 Geotechnical Borings

A NOVA geologist logged the borings and collected samples of the materials encountered for laboratory testing. Relatively undisturbed samples were obtained using a modified California (CAL) sampler, a ring-lined split tube sampler with a 3-inch outer diameter and 2½-inch inner diameter. Standard Penetration Tests (SPT) were performed in the borings using a 2-inch outer diameter and 1⅜-inch inner diameter split tube sampler. The CAL and SPT samplers were driven using an automatic hammer with a calibrated Energy Transfer Ratio (ETR) of 73.9% for the truck-mounted drill rig and 80.8% for the track-mounted limited access drill rig

The number of blows needed to drive the sampler the final 12 inches of an 18-inch drive is noted on the logs. The field blow counts, N, were corrected to a standard hammer (cathead and rope) with a 60% ETR. The corrected blow counts are noted on the boring logs as N_{60}. Disturbed bulk samples were obtained from the SPT sampler and the drill cuttings. Logs of the borings are presented in Appendix B. Soils are classified according to the Unified Soil Classification System.

2.1.3 Seismic Refraction

Seismic refraction relies on measurements of the travel times of seismic waves traveling through and refracting from subsurface layers with contrasting densities. Seismic waves are introduced to the subsurface by striking a steel plate at the surface with a heavy hammer, and measuring seismic wave reception from each geophone. 'Geophones' spaced at regular intervals intercept the seismic waves travelling through and reflecting off of the subsurface media. The data is then processed and interpreted using seismic interpretation software.

As employed in this instance, the objective of the testing was to identify the seismic wave velocities of the subsurface materials to both (i) identify the contact between the less dense Quaternary Old Paralic Deposits and underlying San Onofre Breccia, and (ii) characterize the requirements for excavation of the breccia.

Seismic refraction surveys measure the seismic compression velocity ('P-wave') and estimate the subsurface stratigraphy. The measurements of seismic velocity from these surveys can be used to estimate the relative difficulty of excavating rock and rock-like materials, also referred to as 'rippability'. Rippability is a semi-quantitative assessment of the potential for mechanical loosening of rock so that the rock may be excavated. Findings of the survey are discussed further in Section 4 of this report, and the tomographic and generalized stratigraphic profiles are included in Appendix D.

2.1.4 Shear Wave Velocity

A seismic shear wave survey was performed on April 8, 2021 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 included one (1) seismic shear wave survey traverse, approximately 160 feet in length. The approximate alignment of the survey is shown on Plate 1, and the results of the survey are presented in Appendix D.

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. 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 (2) 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. 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.

After the field data was collected, the geophysicist combined the MASW and MAM survey results using specialized software specific to this purpose. The weighted average velocity in the upper 100 feet of the site (V100) was computed after 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 1,606 feet/second. This average velocity classifies the underlying soils as Site Class C.

2.1.5 Borehole Percolation Testing

Borehole percolation testing was performed in accordance with the test method described in the City of Oceanside BMP Design Manual (Oceanside 2016). The procedure is discussed in Section 8 of this report and infiltration worksheets are presented in Appendix E.

2.2. Laboratory Testing

NOVA tested selected samples to evaluate soil classification and engineering properties and develop geotechnical conclusions and recommendations. The laboratory tests consisted of particle-size distribution, maximum density, in-place density, expansion index, direct shear, R-value, and corrosivity. The results of the laboratory tests and brief explanations of the test procedures are presented in Appendix C.

2.3. Analysis and Report Preparation

The results of the field and laboratory testing were evaluated to develop conclusions and recommendations regarding the geotechnical aspects of the proposed construction. This report presents NOVA's findings, conclusions, and recommendations.

3. SITE AND PROJECT DESCRIPTION

3.1. Site Description

The site consists of about nine acres developed over three blocks located between Seagaze Drive on the north, Missouri Avenue on the south, South Tremont Street on the east, and Cleveland Avenue on the west. The site currently supports the Oceanside Transit Center. Existing structures include an office building, a bus station with an associated restaurant, and asphalt-surfaced parking lots. The site slopes gently from northeast to southwest. Elevations across the nine-acre site range from about +63 feet mean sea level (msl) at the northeast point of the northernmost parking lot to about +43 feet msl in the southwest corner of the site. Generally speaking, Tremont Street on the east of the site is 10 feet higher than Cleveland Street on the west.

Review of historic aerial photography of the site vicinity indicates the current Transit Center structures and improvements have been in place in current configuration since 1997. Development of the Transit Center appears to have begun in 1983, with additional improvements developed over the intervening years. The earliest aerial photography available, from 1938, indicates the site area was lightly developed, supporting open land and small residential structures. Residential and commercial development increased steadily from 1938 until the current Transit Center was developed.

3.2. Proposed Construction

Plans are still conceptual at this time. Based on review of the referenced conceptual site plan (Stantec, 2021), NOVA understands the project will consist of construction of a new mixed-use community development which will include a new North County Transit District headquarters, new Oceanside transit center roads and stations, residential apartment buildings, retail space, public plazas, a parking structure, and a luxury hotel. One to two levels of below grade parking is proposed below the majority of the structures. Because Tremont Street is higher than Cleveland Street, there is one-level of subterranean parking proposed below the level of Cleveland Street, and two levels below Tremont Street. The figures presented on the following pages summarize the conceptual development. Figure 3-1 (following page) presents an overview of the blocks being redeveloped. Figure 3-2 (second page following) presents the proposed configuration, and Figure 3-3 (second page following) presents the site in cross section.

The residential buildings on the southernmost two blocks will include about 550 units and below-grade parking with about 1,120 spaces. New structures at the site will rise up to six levels above the existing grade of Cleveland Street. The project will include pavements for parking and drive aisles and stormwater Best Management Practices (BMP) facilities. BMP locations were not identified at the time of this report. NOVA assumes that stormwater BMP facilities will be constructed away from building foundations, retaining walls, and underground utilities.

Figure 3-1. Area of Redevelopment (north is to the left of page)
(Source: Stantec, 2021)

Figure 3-2. Proposed Configuration (north is to the left of page)
(Source: Stantec, 2021)

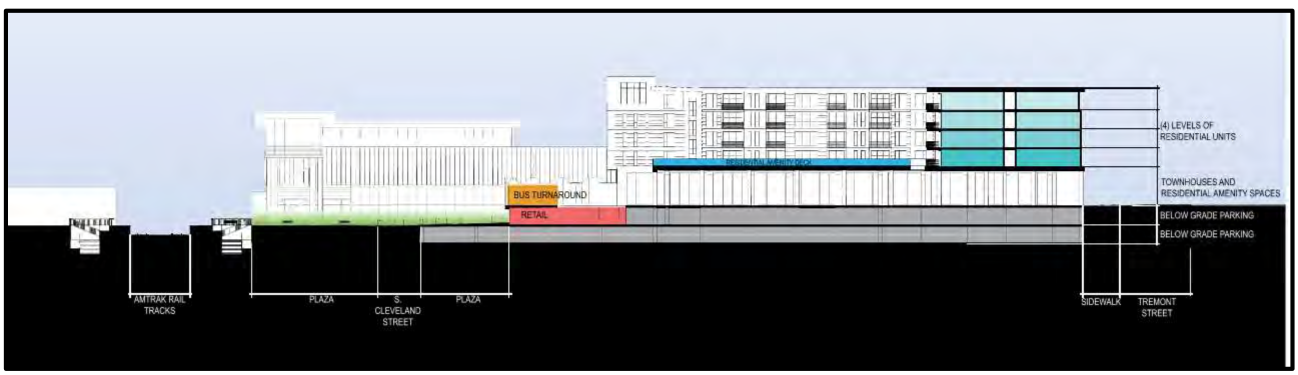

Figure 3-3. Cross-Section (Cleveland St. on left of page and Tremont St. on right)
(Source: Stantec, 2021)

3.3. Anticipated Earthwork

While no grading plans or finish grade elevations are available at this time, conceptual plans indicate the subterranean levels will be founded about 10 feet below Cleveland Street and 20 feet below Tremont Street, requiring excavations ranging from 10 to 20 feet below the existing ground surface. Limited remedial grading is anticipated in areas of new streets, bus lanes, and concrete flatwork.

4. GEOLOGY AND SUBSURFACE CONDITIONS

4.1. Regional Geology

The site is located within the Peninsular Ranges Geomorphic Province of California, which stretches from the Los Angeles basin to the tip of Baja California in Mexico. This province is characterized as a series of northwest-trending mountain ranges separated by subparallel fault zones, and a coastal plain of subdued landforms. The mountain ranges are underlain primarily by Mesozoic metamorphic rocks that were intruded by plutonic rocks of the southern California batholith, while the coastal plain is underlain by subsequently deposited marine and nonmarine sedimentary formations. Figure 4-1 presents the regional geology in the vicinity of the site.

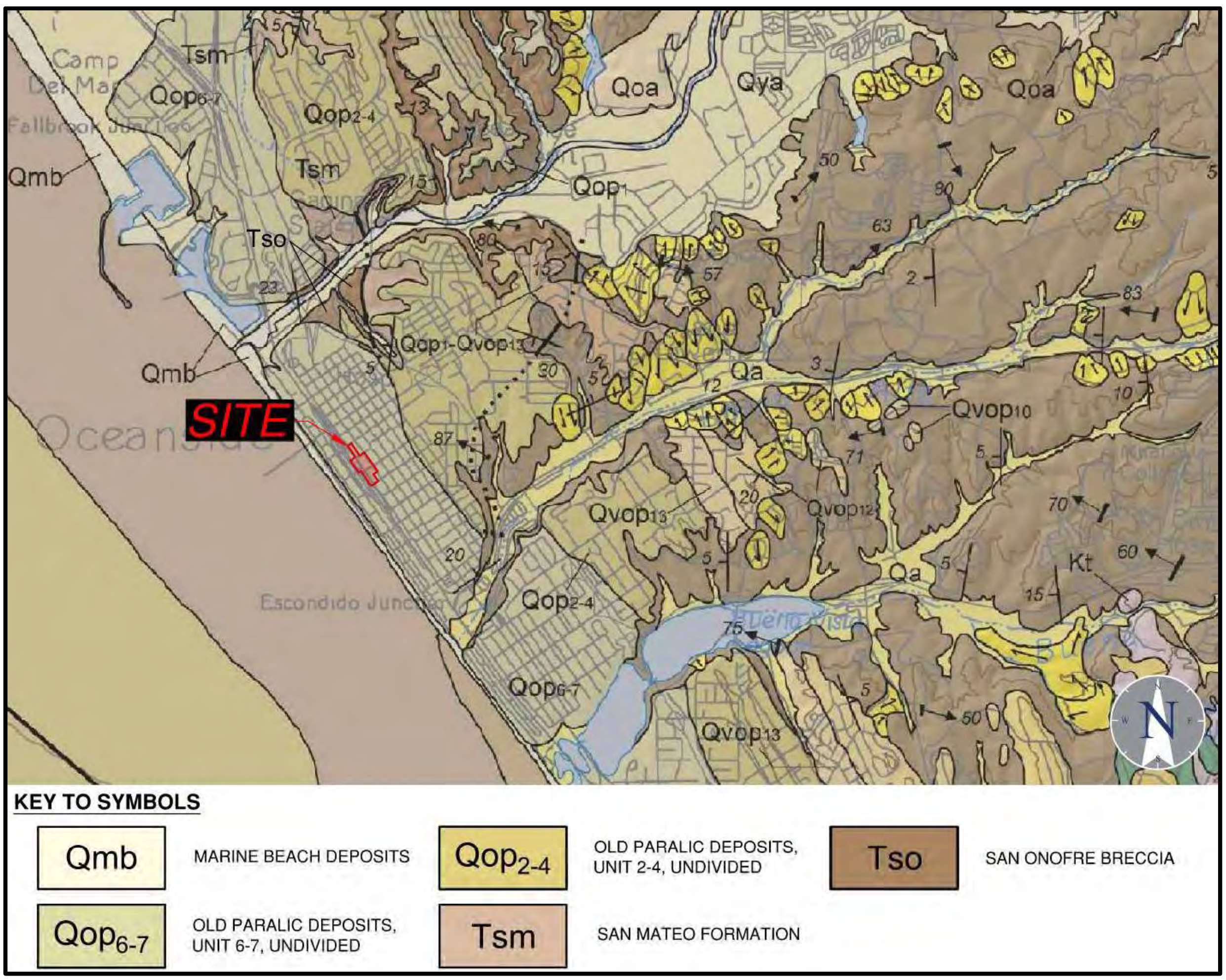

Figure 4-1. Regional Geology Map

4.2. Site-Specific Geology

The site is located within the Coastal Plain portion of the province, and is underlain by Quaternary Old Paralic Deposits, which overlie Tertiary-aged San Onofre Breccia. Plate 1 following the text of the report provides the geology of the site, with Plate 2 presenting cross-sections of the geologic units below the proposed development.

Quaternary Old Paralic Deposits (Qop): Old Paralic Deposits were encountered from the surface, extending to depths ranging from about 6½ feet at the northern east corner of Block 2 parking lot to 16½ feet at the southwestern end of the same parking lot. These deposits consisted reddish brown to orange and yellow brown, medium dense to dense silty sand with some lenses having scattered gravel. Figure 4-2 presents a photograph of the Old Paralic Deposits.

Figure 4-2. Old Paralic Deposits, 4/8/21

Tertiary San Onofre Breccia (Tso): The site is underlain at depth by a pale olive brown to gray orange brown clayey to silty sandstone lens of the San Onofre Breccia with abundant gravel and cobble, very dense in consistency. This Eocene unit is observed to be cemented, with drill auger refusal encountered within the upper 6 inches to 3 feet of the unit. This unit will require considerably more effort to excavate than the overlying Old Paralic Deposits, and is expected to include clasts of oversized material (rock larger than

6 inches in maximum dimension), that should be screened out in order to reuse the material as engineered fill. Figure 4-3 presents a photograph of the San Onofre Breccia.

Figure 4-3. San Onofre Breccia, 4/8/21

4.3. Discussion of Geologic Conditions

Based on the subsurface investigation and seismic refraction lines, the contact between the Old Paralic Deposits and the San Onofre Breccia slopes from the eastern portion of the site, near Tremont Street, down toward the western portion of the project near Cleveland Street. It also appears to slope downward from the northern portion of the project, gently downward to the south.

As may be seen from the cross-sections presented on Plate 2, along the eastern margin of the site where the removals are anticipated to be the deepest (approximately 20 feet) to accommodate two levels of parking, the breccia is the most shallow, ranging from approximately 6 feet below the existing surface to 15 feet below the existing surface. This will require excavations of approximately 5 to 14 feet into the cemented breccia. These excavations along the eastern margin of the project are anticipated to be founded in San Onofre Breccia.

In areas along the western margin of the site, the breccia contact is anticipated to be deeper, on the order of 15 to 18 feet below existing ground surface. Along this margin removals are anticipated to be 10 feet in depth to accommodate one-story of underground parking. Excavation along this margin is anticipated to be founded in Quaternary Old Paralic Deposits.

4.3.1 Rippability of San Onofre Breccia

Figures 4-4 and 4-5 present the tomographic profiles of Seismic Refraction survey S-3 and S-4. All records of the seismic refraction testing are provided in Appendix D.

A seismic velocity determined by seismic refraction (V_p) on the order of 3,800 fps is the upper limit of 'trenchable' rock (for utility installation), and 6,000 fps is generally considered the upper limit of 'rippable' rock with larger conventional earthmoving equipment. Figure 4-6 (following page) correlates rippability by a CAT 8R bulldozer with seismic wave velocity.

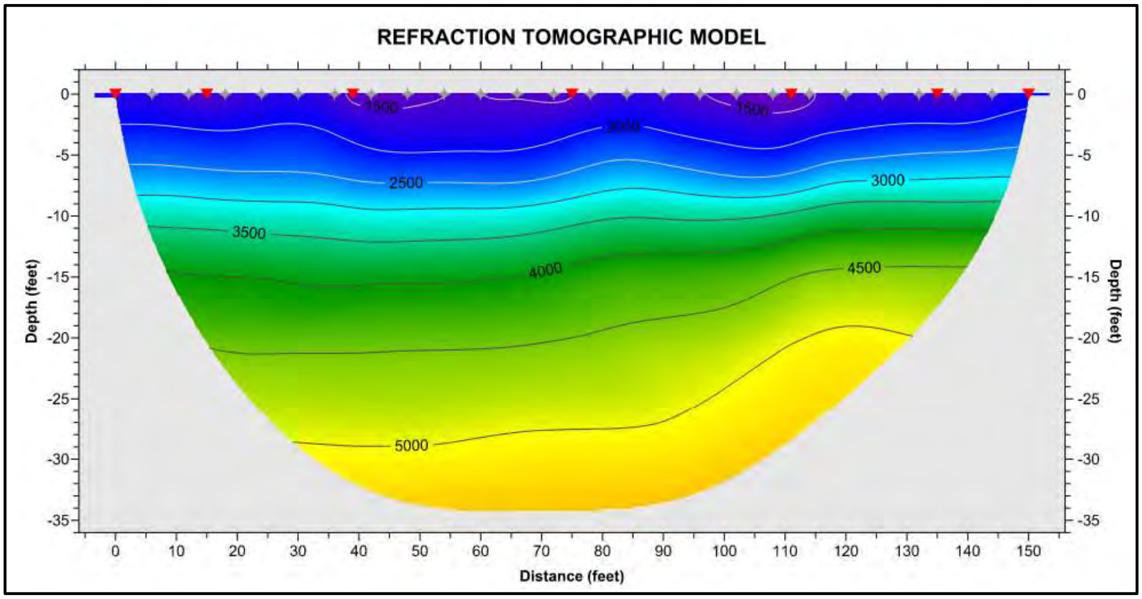

Figure 4-4 Subsurface Profile Along Seismic Line S-3

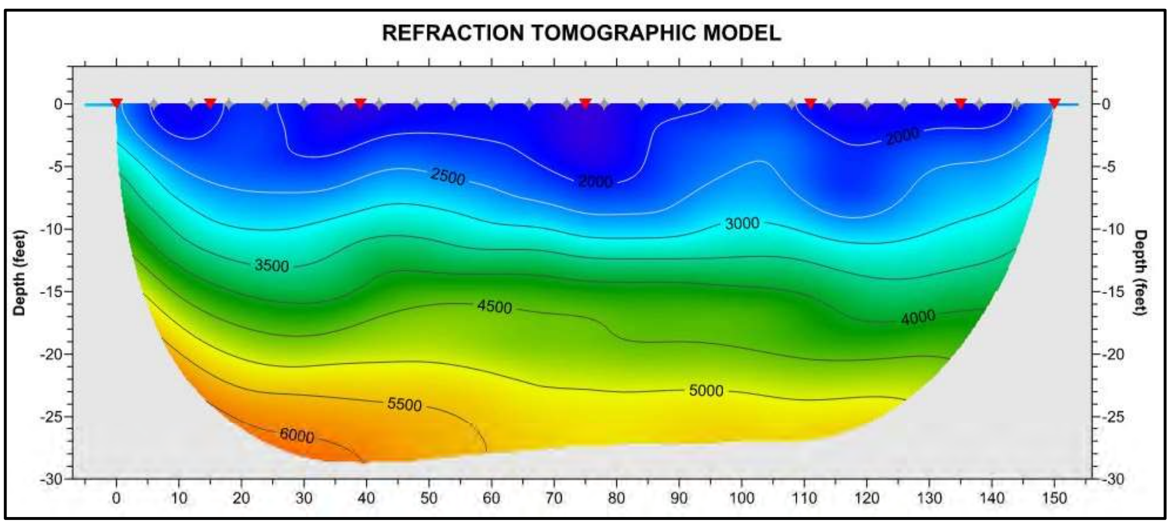

Figure 4-5 Subsurface Profile Along Seismic Line S-4

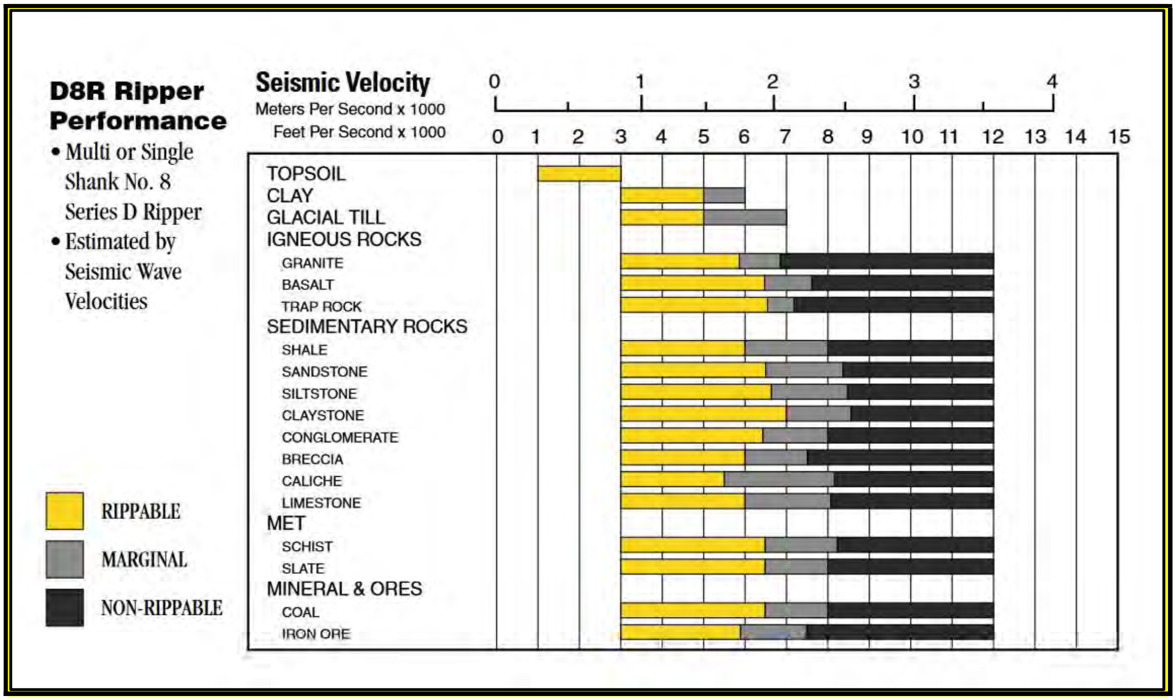

Figure 4-6. D8R Seismic Velocity Rippability Chart
(source: Caterpillar Performance Handbook, 2000)

The refraction surveys indicate that the San Onofre Breccia is generally rippable to the depths currently anticipated, though the velocities are variable, and localized areas of marginally rippable cemented material may be encountered during the deeper excavations. In some areas, particularly along the eastern portions of the site, trenching for utilities will likely require extra effort. Please note, use of charts such as Figure 3-8 should be undertaken with caution. As noted by the Caterpillar Performance Handbook:

> *Charts of ripper performance estimated by seismic wave velocities have been developed from field tests conducted in a variety of materials. Considering the extreme variations among materials and even among rocks of a specific classification, the charge must be recognized as being at best only one indicator of rippability.*
>
> *Ripping is still more art than science, and much will depend on operator skill and experience.*

4.4. Groundwater

Groundwater was not encountered in any of the borings. Perched groundwater commonly occurs where permeable material overlies less permeable materials, often after periods of rainfall. Groundwater levels may fluctuate in the future due to rainfall, irrigation, broken pipes, or changes in site drainage. Because groundwater rise or seepage is difficult to predict, such conditions are typically mitigated if and when they occur.

5. GEOLOGIC HAZARDS

5.1. Faulting and Surface Rupture

5.1.1 Strong Ground Motion

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

The tectonic setting of the metropolitan San Diego area includes major north and northwest striking fault zones, the most prominent and active of which is the Newport-Inglewood-Rose Canyon fault zone (NIRC). The NIRC includes offshore faulting from Newport Beach southeastward past Oceanside, until it returns back onshore at La Jolla Cove.

The seismicity of the site was evaluated utilizing the Seismic Design Maps web-based analytical tool provided by Structural Engineers Association of California (SEA) and California's Office of Statewide Health Planning and Development (OSHPD). The USGS Unified Hazard Tool indicates the site may be subjected to a Magnitude 6.9 seismic event, with a corresponding site-adjusted Peak Ground Acceleration (PGAM) of 0.568g.

5.1.2 Faulting in the Site Vicinity

Earthquake Fault Zones (formerly known as special study zones) have been established along known active faults in California in accordance with the Alquist-Priolo Earthquake Fault Zoning Act. The site is not located in an Alquist-Priolo Earthquake Fault Zone. No active surface faults are mapped across the site. The nearest active fault is the Oceanside section of the Newport-Inglewood-Rose Canyon fault zone (NIRCFZ), located approximately 4.6 miles to the southwest. Evidence of active faulting was not observed at the site during the time of our field evaluation. The probability of fault rupture is considered very low.

Figure 5-1 (following page) shows the locations of known faults in the general site area. Active faults are presented in orange and potentially active, or undifferentiated Quaternary faults are presented in black.

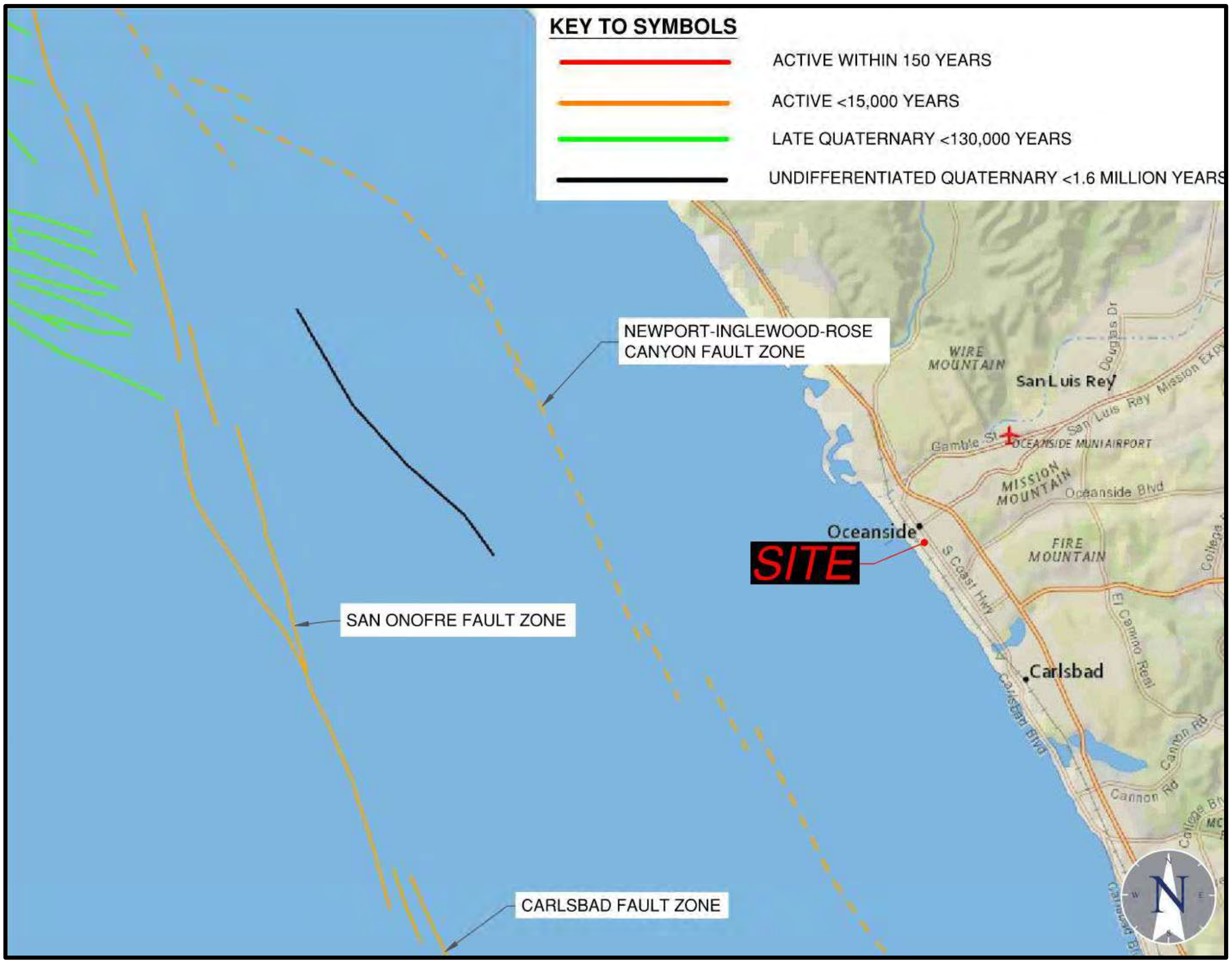

Figure 5-1. Faulting in the Site Vicinity
(Source: USGS U.S. Quaternary Faults)

5.2. CBC Seismic Design Parameters

A geologic hazard likely to affect the project is ground shaking as a result of movement along an active fault zone in the vicinity of the subject site. The site coefficients and maximum considered earthquake (MCE_R) spectral response acceleration parameters in accordance with the 2019 CBC and ASCE 7-16 are presented in Table 5-1 (following page).

Table 5-1. ASCE 7-16 Mapped Site Coefficients

Site Coordinates	
Latitude: 33.19181928	Longitude: -117.37829463
Site Coefficients and Spectral Response Acceleration Parameters	**Value**
Site Class	C
Site Coefficients, F_a	1.2
Site Coefficients, F_v	1.5
Mapped Spectral Response Acceleration at Short Period, S_s	1.072g
Mapped Spectral Response Acceleration at 1-Second Period, S_1	0.391g
Mapped Design Spectral Acceleration at Short Period, S_{DS}	0.858g
Design Spectral Acceleration at 1-Second Period, S_{D1}	0.391g
Site Peak Ground Acceleration, PGA_M	0.568g

5.3. Landslides and Slope Stability

Evidence of landslides, deep-seated landslides, or slope instabilities was not observed at the time of NOVA's field evaluation. Additionally, there are no mapped landslides in the vicinity of the project site. The potential for landslides or slope instabilities to occur at the site is considered very low.

5.4. Liquefaction and Dynamic Settlement

Liquification occurs when loose, saturated, generally fine sands and silts are subjected to strong ground shaking. The soils lose shear strength and become liquid, resulting in large total and differential ground surface settlements, as well as possible lateral spreading during an earthquake. Due to the lack of shallow groundwater and given the relatively dense nature of the materials beneath the site, the potential for liquefaction and dynamic settlement to occur is considered very low.

5.5. Flooding, Tsunamis, and Seiches

The site is mapped within an area of minimal flood hazard (FEMA, 2019). The site is not located within a mapped area on the State of California Tsunami Inundation Maps (Cal EMA, 2009); therefore, damage due to tsunamis is considered low. Seiches are periodic oscillations in large bodies of water such as lakes, harbors, bays, or reservoirs. The site is not located adjacent to any lakes or confined bodies of water; therefore, the potential for a seiche to affect the site is considered negligible.

5.6. Subsidence

The site is not located in an area of known subsidence associated with fluid withdrawal (groundwater or petroleum); therefore, the potential for subsidence due to the extraction of fluids is considered negligible.

5.7. Hydro-Consolidation

Hydro-consolidation can occur in recently deposited sediments (less than 10,000 years old) that were deposited in a semi-arid environment. Examples of such sediments are eolian sands, alluvial fan deposits, and mudflow sediments deposited during flash floods. The pore spaces between the particle grains can re-adjust when inundated by groundwater, causing the material to consolidate. The relatively dense materials underlying the site are not considered susceptible to hydro-consolidation.

6. CONCLUSIONS

Based on the results of NOVA's investigation, we consider the proposed construction feasible from a geotechnical standpoint provided the recommendations contained in this report are followed. Geotechnical conditions exist that should be addressed prior to construction. Geotechnical design and construction considerations include the following.

- There are no known active faults underlying the site. The primary seismic hazard at the site is the potential for moderate to severe ground shaking in response to large-magnitude earthquakes generated during the lifetime of the proposed construction. The risk of strong ground motion is common to all construction in southern California and is typically mitigated through building design in accordance with the CBC. While strong ground motion could affect the site, the risk of liquefaction is considered negligible.
- The site is underlain by Old Paralic Deposits and San Onofre Breccia. The Old Paralic Deposits and San Onofre Breccia are considered suitable for support of structural or fill loads. However, the stiffness, strength, and bearing properties of the two materials are different, and foundations for individual structures should not be underlain by transitions from Old Paralic Deposits to San Onofre Breccia.
- NOVA anticipates that the on-site materials have expansion indexes of 50 or less, or very low to low expansion potential.
- In general, excavations at the site are anticipated to be achievable using standard heavy earthmoving equipment in good-working order with experienced operators. However, localized zones of cemented San Onofre Breccia may require extra excavation effort or may encounter excavation refusal. These excavations may also generate a significant amount of oversized rock that will require extra effort to crush or export from the site.
- The proposed buildings can be supported on shallow spread footings with bottom levels bearing entirely on Old Paralic Deposits or entirely on breccia. For the at-grade NCTD offices building, the footings are anticipated to generally be underlain by Old Paralic Deposits. For the hotel, residential, and transit center buildings, the basement level footings are anticipated to generally be underlain by breccia, with minor amounts of old paralic deposits between the bottom of footings and breccia on the western portions of the buildings. Foundation recommendations are provided in the following section of this report.
- Groundwater was not encountered in the borings. Perched groundwater commonly occurs where permeable material overlies less permeable materials, often after periods of rainfall. Groundwater and perched groundwater are not expected to pose any constraints to development.
- The infiltration feasibility condition category is "Full Infiltration." Infiltration is discussed further in Section 8.

7. RECOMMENDATIONS

The remainder of this report presents recommendations regarding earthwork construction as well as preliminary geotechnical recommendations for the design of the proposed improvements. These recommendations are based on empirical and analytical methods typical of the standard-of-practice in southern California. If these recommendations appear not to address a specific feature of the project, please contact NOVA for additions or revisions to the recommendations. The recommendations presented herein may need to be updated once final plans are developed.

7.1. Earthwork

Grading and earthwork should be conducted in accordance with the CBC and the recommendations of this report. The following recommendations are provided regarding specific aspects of the proposed earthwork construction. These recommendations should be considered subject to revision based on field conditions observed by a NOVA field representative during grading.

7.1.1 Site Preparation

Site preparation should begin with the removal of existing improvements, vegetation, and debris. Subsurface improvements that are to be abandoned should be removed, and the resulting excavations should be backfilled and compacted in accordance with the recommendations of this report. Pipeline abandonment can consist of capping or rerouting at the project perimeter and removal within the project perimeter. If appropriate, abandoned pipelines can be filled with grout or slurry as recommended by and observed by the geotechnical consultant.

7.1.2 Building Areas

<u>NCTD Offices Building</u>

NOVA encountered shallow Old Paralic Deposits in the area of the proposed at-grade NCTD offices building. Accordingly, we recommend that this building be supported on spread footings with bottom levels bearing entirely on Paralic Deposits. If isolated fills are encountered beneath footings, 3-sack sand/cement slurry should be placed between the bottom of footing and the underlying Old Paralic Deposits.

Beneath the proposed building pad, any existing fill should be excavated to expose competent Paralic Deposits. During removal of existing structures and improvements, NOVA anticipates that portions of the near surface soils will be disturbed, and therefore some removal and recompaction effort will need to be performed. The excavated Paralic Deposits can be processed and reused for compacted fill.

The building should not be underlain by cut/fill transitions. If a cut/fill transition occurs beneath the building slab, the cut portion should be over-excavated to a depth of 2 feet below finished pad grade to provide a relatively uniform thickness of compacted fill beneath the entire building slab. Horizontally, the over-excavation should extend at least

5 feet outside the planned perimeter building foundations or up to existing improvements, whichever is less.

NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended. The resulting excavation should then be filled to the finished pad grade with compacted fill having an expansion index of 50 or less.

<u>Hotel, Residential, and Transit Center Buildings</u>

We anticipate that basement level footings for the hotel, parking garage, residential, and transit center buildings will generally be underlain by San Onofre Breccia. If fill or Old Paralic Deposits are encountered beneath footings, these materials should be removed to competent breccia, and 3-sack sand/cement slurry should be placed in the excavation between the bottom of footing and the underlying breccia. NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended.

7.1.3 Remedial Grading – Pedestrian Hardscape

Beneath proposed hardscape areas, the on-site soils should be excavated to a depth of at least 2 feet below planned subgrade elevation. Horizontally, excavations should extend at least 2 feet outside the planned hardscape or up to existing improvements, whichever is less. If competent formational materials are exposed, excavation need not be performed. If competent formational sands are exposed, excavation need not be performed.

NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended. The resulting surface should then be scarified to a depth of 6 to 8 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. If competent formational materials are exposed, scarification and recompaction need not be performed. The excavation should be filled with compacted fill having an expansion index of 50 or less.

7.1.4 Remedial Grading – Vehicular Pavements

Beneath proposed vehicular pavement areas, the existing soils should be excavated to a depth of at least 1 foot below planned subgrade elevation. Horizontally, excavations should extend at least at least 2 feet outside the planned pavement or up to existing improvements, whichever is less. NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended. The resulting surface should then be scarified to a depth of 6 to 8 inches, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. If competent formational materials are exposed, scarification and recompaction need not be performed. The excavation should be filled with material suitable for reuse as compacted fill.

7.1.5 Remedial Grading – Site Walls and Retaining Walls

Beneath proposed site walls and retaining walls not connected to buildings, any existing fill should be excavated to a depth of at least 2 feet below bottom of footing. Horizontally, the excavations should extend at least 2 feet outside the planned hardscape, wall footing, or up to existing improvements, whichever is less. If competent formational sands are exposed, excavation need not be performed. NOVA should observe the conditions exposed in the bottom of excavations to evaluate whether additional excavation is recommended. Any required fill should have an expansion index of 50 or less.

7.1.6 Expansive Soil

The on-site soils tested have expansion indexes ranging from 1 to 3, classified as very low expansion potential. To reduce the potential for expansive heave, the top 2 feet of material beneath building footings, concrete slabs-on-grade, hardscape, and site and retaining wall footings should have an expansion index of 50 or less. Horizontally, the soils having an expansion index of 50 or less should extend at least 5 feet outside the planned perimeter building foundations, at least 2 feet outside site/retaining wall footings and hardscape, or up to existing improvements, whichever is less. NOVA anticipates that the on-site silty sand will meet the expansion index criteria.

7.1.7 Compacted Fill

Fill and backfill should be placed in 6- to 8-inch-thick loose lifts, moisture conditioned to near optimum moisture content, and compacted to at least 90% relative compaction. The maximum density and optimum moisture content for the evaluation of relative compaction should be determined in accordance with ASTM D1557. Utility trench backfill beneath structures, pavements, and hardscape should be compacted to at least 90% relative compaction. The top 12 inches of subgrade beneath pavements should be compacted to at least 95% relative compaction.

7.1.8 Imported Soil

Imported soil should consist of predominately granular soil, and free of organic matter and rocks greater than 6 inches. Imported soil should be observed and, if appropriate, tested by NOVA prior to transport to the site to evaluate suitability for the intended use.

7.1.9 Subgrade Stabilization

Excavation bottoms should be firm and unyielding prior to placing fill. In areas of saturated or yielding subgrade, a reinforcing geogrid such as Tensar® Triax® TX-5 or equivalent can be placed on the excavation bottom, and then at least 12 inches of aggregate base placed and compacted. Once the surface of the aggregate base is firm enough to achieve compaction, then the remaining excavation should be filled to finished pad grade with suitable material.

7.1.10 Excavation Characteristics

It is anticipated that excavations within the Old Paralic Deposits can be achieved with conventional earthwork equipment in good working order. Should excavations extend into the underlying highly indurated San Onofre Breccia, difficult excavation should be anticipated. Gravel, cobbles, and potentially boulders should also be anticipated.

7.1.11 Oversized Material

Excavations may generate some oversized material, particularly excavations extending into the San Onofre Breccia. Oversized material is defined as rocks or cemented clasts greater than 6 inches in largest dimension. Oversized material should be broken down to no greater than 6 inches in largest dimension for use in fill, used as landscape material, or disposed of off-site.

7.1.12 Temporary Excavations

Temporary excavations 3 feet deep or less can be made vertically. Deeper temporary excavations in fill or Quaternary Old Paralic Deposits should be laid back no steeper than 1:1 (horizontal:vertical). Deeper temporary excavations in San Onofre Breccia should be laid back no steeper than ¾:1 (h:v).

The faces of temporary slopes should be inspected daily by the contractor's Competent Person before personnel are allowed to enter the excavation. Any zones of potential instability, sloughing, or raveling should be brought to the attention of the engineer and corrective action implemented before personnel begin working in the excavation. Excavated soils should not be stockpiled behind temporary excavations within a distance equal to the depth of the excavation. NOVA should be notified if other surcharge loads are anticipated so that lateral load criteria can be developed for the specific situation. If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to prevent runoff water from entering the excavation and eroding the slope faces.

Slopes steeper than those described above will require shoring. Additionally, temporary excavations that extend below a plane inclined at 1½:1 (h:v) downward from the outside bottom edge of existing structures or improvements will require shoring. Soldier piles and lagging, internally braced shoring, or trench boxes could be used. If trench boxes are used, the soil immediately adjacent to the trench box is not directly supported. Ground surface deformations immediately adjacent to the pit or trench could be greater where trench boxes are used compared to other methods of shoring.

7.1.13 Temporary Shoring

For design of cantilevered shoring with level backfill, an active earth pressure equal to a fluid weighing 35 pounds per cubic foot (pcf) can be used. For design of tied-back shoring with level backfill, a rectangular earth pressure distribution with a maximum pressure of 23H pounds per square foot (psf), where H is the height of shoring in feet, can be used. Alternatively, a trapezoidal pressure distribution with a maximum pressure of 28H psf at 0.1H down from the top of shoring and 0.2H up from the base of shoring can be used. The surcharge loads from traffic and construction equipment adjacent to the shored excavation can be modeled by assuming an additional 2 feet of soil behind the shoring.

For design of soldier piles embedded in formational materials, an allowable passive pressure of 350 psf per foot of embedment over three times the pile diameter or the spacing of the piles, whichever is less, up to a maximum of 7,500 psf can be used. Soldier piles should be spaced at least three pile diameters, center to center.

For design of tie-backs, a friction angle of 34 degrees, a cohesion of 200 psf and an average frictional resistance of 600 psf can be used for the portion of anchor embedded in formational materials. Only the frictional resistance developed beyond the active wedge will be effective in resisting lateral loads. It can be assumed that the active wedge adjacent to the shoring wall is defined by a plane drawn at 35 degrees from vertical through the bottom of the excavation. Anchor capacities should be proof-tested during construction. Where satisfactory tests are not achieved, the anchor diameter and/or length should be increased until satisfactory test results are obtained.

Continuous lagging will be required throughout. The soldier piles and tie-back anchors should be designed for the full-anticipated lateral pressure; however, the pressure on the lagging will be less due to arching in the soils. For design of lagging, the earth pressure but can be limited to a maximum value of 400 psf.

7.1.14 Slopes

Permanent slopes should be constructed no steeper than 2:1 (h:v). Faces of fill slopes should be compacted either by rolling with a sheepsfoot roller or other suitable equipment, or by overfilling and cutting back to design grade. Fills should be benched into sloping ground inclined steeper than 5:1 (h:v). In NOVA's opinion, slopes constructed no steeper than 2:1 (h:v) will possess an adequate factor of safety. An engineering geologist should observe cut slopes during grading to ascertain that no unforeseen adverse geologic conditions are encountered that require revised recommendations. Slopes are susceptible to surficial slope failure and erosion. Water should not be allowed to flow over the top of slope. Additionally, slopes should be planted with vegetation that will reduce the potential for erosion.

7.1.15 Surface Drainage

Final surface grades around structures should be designed to collect and direct surface water away from structures, including retaining walls, and toward appropriate drainage facilities. The ground around the structure should be graded so that surface water flows rapidly away from the structure without ponding. In general, we recommend that the ground adjacent to the structure slope away at a gradient of at least 2%. Densely vegetated areas where runoff can be impaired should have a minimum gradient of at least 5% within the first 5 feet from the structure. Roof gutters with downspouts that discharge directly into a closed drainage system are recommended on structures. Drainage patterns established at the time of fine grading should be maintained throughout the life of the proposed structures. Site irrigation should be limited to the minimum necessary to sustain landscape growth. Should excessive irrigation, impaired drainage, or unusually high rainfall occur, saturated zones of perched groundwater can develop.

7.1.16 Grading Plan Review

NOVA should review the grading plans and earthwork specifications to ascertain whether the intent of the recommendations contained in this report have been implemented, and that no revised recommendations are needed due to changes in the development scheme.

7.2. Foundations

The foundation recommendations provided herein are considered generally consistent with methods typically used in southern California. Other alternatives may be available. NOVA's recommendations are only minimum criteria based on geotechnical factors and should not be considered a structural design, or to preclude more restrictive criteria of governing agencies or by the structural engineer. The design of the foundation system should be performed by the project structural engineer, incorporating the geotechnical parameters described herein and the requirements of applicable building codes.

The proposed buildings can be supported on shallow spread footings with bottom levels bearing entirely on old paralic deposits or entirely on breccia. Site walls and retaining walls not connected to buildings can be supported on shallow spread footings with bottom levels bearing on compacted fill or old paralic deposits. Shade structures, covered walkways and other pole-type structures can be supported on cast-in-drilled hole (CIDH) concrete piles.

7.2.1 Spread Footings

Footings should extend at least 24 inches below lowest adjacent finished grade. A minimum width of 12 inches is recommended for continuous footings and 24 inches for isolated or wall footings. An allowable bearing capacity of 2,500 psf can be used for footings supported on compacted fill or Old Paralic Deposits. An allowable bearing capacity of 5,000 psf can be used for footings supported on competent San Onofre Breccia. The allowable bearing capacity can be increased by 500 psf for each foot of depth below the minimum and 250 psf for each foot of width beyond the minimum up to a maximum of 5,000 psf on compacted fill or old paralic deposits or 7,500 psf on competent breccia. The bearing value can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. Footings located adjacent to or within slopes should be extended to a depth such that a minimum horizontal distance of 10 feet exists between the lower outside footing edge and the face of the slope.

Lateral loads will be resisted by friction between the bottoms of footings and passive pressure on the faces of footings and other structural elements below grade. An allowable coefficient of friction of 0.35 can be used. An allowable passive pressure of 350 psf per foot of depth below the ground surface can be used for level ground conditions. The allowable passive pressure should be reduced for sloping ground conditions. The passive pressure can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

7.2.2 CIDH Piles

CIDH piles should be spaced at least three pile diameters, center to center, and be embedded in compacted fill and/or formational materials. The axial downward capacity of piles can be obtained from skin friction and end bearing. An allowable downward skin friction of 300 psf and an allowable end bearing of 5,000 psf can be used. If end bearing is used, the bottom of drilled holes should be cleaned of loose soil prior to placing concrete. The axial uplift capacity of piles can be obtained from skin friction and the weight of the pile. An allowable uplift skin friction of 100 psf can be used.

Lateral loads can be resisted by passive pressure on the piles. An allowable passive pressure of 350 psf per foot of embedment acting on twice the pile diameter up to a maximum of 5,000 psf can be used, based on a lateral deflection up to ½-inch at the ground surface and level ground conditions. The uplift and passive pressure values can be increased by ⅓ when considering the total of all loads, including wind or seismic forces. The upper 1 foot of soil should not be relied on for passive support unless the ground is covered with pavements or slabs.

7.2.3 Settlement Characteristics

Total foundation settlements are estimated to be less than 1-inch. Differential settlements between adjacent columns and across continuous footings are estimated to be less than ¾-inch over a distance of 40 feet. Settlements should be completed shortly after structural loads are applied.

7.2.4 Foundation Plan Review

NOVA should review the foundation plans to ascertain that the intent of the recommendations in this report has been implemented and that revised recommendations are not necessary as a result of changes after this report was completed.

7.2.5 Foundation Excavation Observations

A representative from NOVA should observe the foundation excavations prior to forming or placing reinforcing steel.

7.3. Interior Slabs-On-Grade

Interior concrete slabs-on-grade should be underlain by at least 2 feet of material with an expansion index of 50 or less. We recommend that conventional concrete slabs-on-grade floors be at least 5 inches thick and reinforced with at least No. 4 bars at 18 inches on center each way. To reduce the potential for excessive cracking, concrete slabs-on-grade should be provided with construction or 'weakened plane' joints at frequent intervals. The project structural engineer should design on-grade building slabs and joint spacing.

Moisture protection should be installed beneath slabs where moisture sensitive floor coverings will be used. The project architect should review the tolerable moisture transmission rate of the proposed floor covering and specify an appropriate moisture protection system. Typically, a plastic vapor barrier is used. Minimum 15-mil plastic is recommended. The plastic should comply with ASTM E1745. The vapor barrier installation should comply with ASTM E1643. The slab can be placed directly on the vapor barrier.

7.4. Hardscape

Hardscape should be underlain by at least 2 feet of material with an expansion index of 50 or less. Exterior concrete slabs should be at least 4 inches thick and reinforced with at least No. 3 bars at 18 inches on center each way. Slabs should be provided with weakened plane joints. Joints should be placed in accordance with the American Concrete Institute (ACI) guidelines. The project architect should select the final joint patterns. A 1-inch maximum size aggregate mix is recommended for concrete for exterior slabs. The corrosion potential of on-site soils with respect to reinforced concrete will need to be taken into account in concrete mix design. Coarse and fine aggregate in concrete should conform to the “Greenbook” Standard Specifications for Public Works Construction.

7.5. Conventional Retaining Walls

Conventional retaining walls can be supported on spread footings. The recommendations for spread footings provided in the foundation section of this report are also applicable to conventional retaining walls.

The active earth pressure for the design of unrestrained retaining walls with level backfill can be taken as equivalent to the pressure of a fluid weighing 35 pcf. The at-rest earth pressure for the design of restrained retaining wall with level backfill can be taken as equivalent to the pressure of a fluid weighing 55 pcf. These values assume a granular and drained backfill condition. Higher lateral earth pressures would apply if walls retain clay soils. An additional 20 pcf should be added to these values for walls with 2:1 (h:v) sloping backfill. An increase in earth pressure equivalent to an additional 2 feet of retained soil can be used to account for surcharge loads from light traffic. The above values do not include a factor of safety. Appropriate factors of safety should be incorporated into the design. If any other surcharge loads are anticipated, NOVA should be contacted for the necessary increase in soil pressure.

The seismic earth pressure can be taken as equivalent to the pressure of a fluid pressure weighing 21 pcf. This value is for level backfill and does not include a factor of safety. Appropriate factors of safety should be incorporated into the design. This pressure is in addition to the un-factored, active earth pressure. The total equivalent fluid pressure can be modeled as a triangular pressure distribution with the resultant acting at a height of H/3 up from the base of the wall, where H is the retained height of the wall. The passive pressure and bearing capacity can be increased by ⅓ in determining the seismic stability of the wall.

Retaining walls should be provided with a backdrain to reduce the accumulation of hydrostatic pressures or be designed to resist hydrostatic pressures. Backdrains can consist of a 2-foot-wide zone of ¾-inch crushed rock. The crushed rock should be separated from the adjacent soils using a non-woven filter fabric, such as Mirafi 140N or equivalent. A perforated pipe should be installed at the base of the backdrain and sloped to discharge to a suitable storm drain facility, or weep holes should be provided. As an alternative, a geocomposite drainage system such as Miradrain 6000 or equivalent placed behind the wall and connected to a suitable storm drain facility can be used. The project architect should provide dampproofing/waterproofing specifications and details. Figure 7-1 (following page) presents typical conventional retaining wall backdrain details. Note that the guidance provided on Figure 7-1 is conceptual. A variety of options are available to drain retaining walls.

Wall backfill should consist of granular, free-draining material having an expansion index of 20 or less. The backfill zone is defined by a 1:1 plane projected upward from the heel of the wall. Expansive or clayey soil should not be used. Additionally, backfill within 3 feet from the back of the wall should not contain rocks greater than 3 inches in dimension. Backfill should be compacted to at least 90% relative compaction. Backfill should not be placed until walls have achieved adequate structural strength. Compaction of wall backfill will be necessary to minimize settlement of the backfill and overlying settlement sensitive improvements. However, some settlement should still be anticipated. Provisions should be made for some settlement of concrete slabs and pavements supported on backfill. Additionally, any utilities supported on backfill should be designed to tolerate differential settlement.

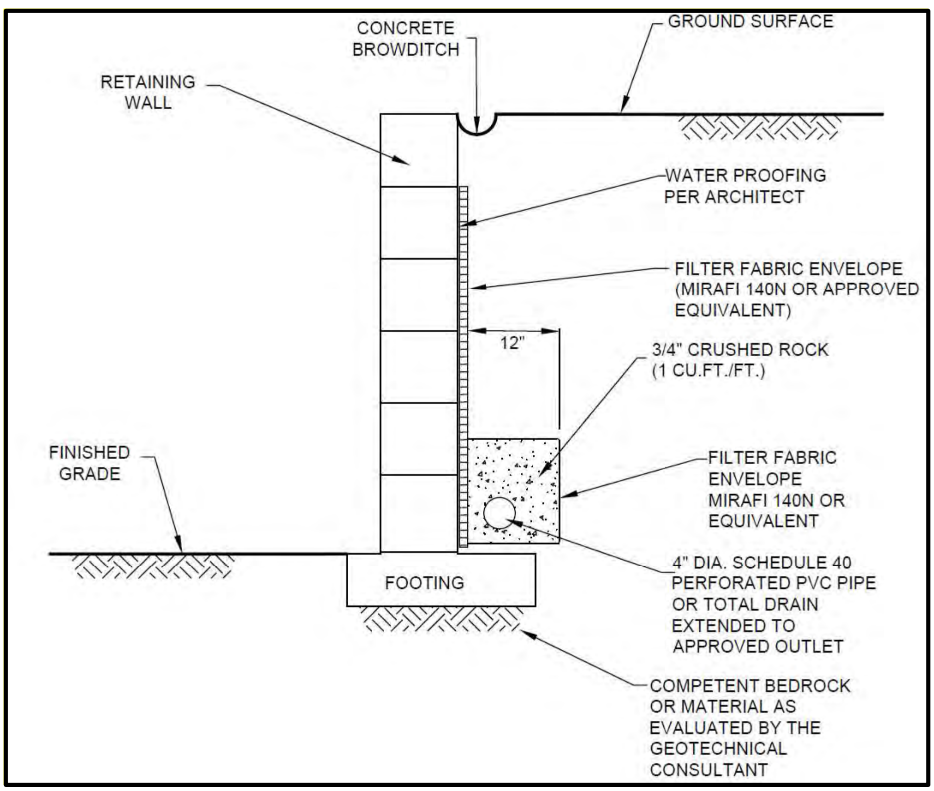

Figure 7-1. Typical Conventional Retaining Wall Backdrain Details

7.6. Pipelines

For level ground conditions, a passive earth pressure of 350 psf per foot of depth below the lowest adjacent final grade can be used to compute allowable thrust block resistance. A value of 150 psf per foot should be used below groundwater level, if encountered.

A modulus of soil reaction (E') of 1,500 psi can be used to evaluate the deflection of buried flexible pipelines. This value assumes that granular bedding material is placed adjacent to the pipe and is compacted to at least 90% relative compaction.

Pipe bedding as specified in the "Greenbook" Standard Specifications for Public Works Construction can be used. Bedding material should consist of clean sand having a sand equivalent not less than 20 and should extend to at least 12 inches above the top of pipe. Alternative materials meeting the intent of the bedding specifications are also acceptable. Samples of materials proposed for use as bedding should be provided to the engineer for inspection and testing before the material is imported for use on the project. The on-site materials are not expected to meet "Greenbook" bedding specifications. The pipe bedding material should be placed over the full width of the trench. After placement of the pipe, the bedding should be brought up uniformly on both sides of the pipe to reduce the potential for unbalanced loads. No voids or uncompacted areas should be left beneath the pipe haunches. Ponding or jetting the pipe bedding should not be allowed.

Where pipeline inclinations exceed 15%, cutoff walls are recommended in trench excavations. Additionally, we do not recommend that open graded rock be used for pipe bedding or backfill because of the potential for piping erosion. The recommended bedding is clean sand having a sand equivalent not less than 20 or 2-sack sand/cement slurry. If sand/cement slurry is used for pipe bedding to at least 1 foot over the top of the pipe, cutoff walls are not considered necessary. The need for cutoff walls should be further evaluated by the project civil engineer designing the pipeline.

7.7. Corrosivity

Representative samples of the on-site soils were tested to evaluate corrosion potential. The test results are presented in Appendix C. The project design engineer can use the sulfate results in conjunction with ACI 318 to specify the water/cement ratio, compressive strength, and cementitious material types for concrete exposed to soil. A corrosion engineer should be contacted to provide specific corrosion control recommendations.

7.8. Pavement Section Recommendations

The pavement support characteristics of the soils encountered during NOVA's investigation are considered low to medium. An R-value of 24 was assumed for design of preliminary pavement sections. The actual R-value of the subgrade soils should be determined after grading, and the final pavement sections should be provided. Based on an R-value of 24, the following preliminary pavement structural sections are provided for the assumed Traffic Indexes on Table 7-1.

Table 7-1. AC and PCC Pavement Sections

Traffic Type	Traffic Index	Asphalt Concrete (inches)	Portland Cement Concrete (inches)
Parking Stalls	4.5	3 AC / 5 AB	6 PCC / 6 AB
Driveways	6.0	4 AC / 8 AB	7 PCC / 6 AB
Bus/Fire Lanes	7.5	5 AC / 11 AB	7 PCC / 6 AB

AC: Asphalt Concrete
AB: Aggregate Base
PCC: Portland Cement Concrete

Subgrade preparation should be performed immediately prior to placement of the pavement section. The upper 12 inches of subgrade should be scarified, moisture conditioned to near optimum moisture content, and compacted to at least 95% relative compaction. All soft or yielding areas should be stabilized or removed and replaced with compacted fill or aggregate base. Aggregate base and asphalt concrete should conform to the Caltrans Standard Specifications or the “Greenbook” and should be compacted to at least 95% relative compaction. Aggregate base should have an R-value of not less than 78. All materials and methods of construction should conform to good engineering practices and the minimum local standards.

8. INFILTRATION FEASIBILITY

Final stormwater infiltration Best Management Practices ('stormwater BMP') locations were not identified at the time of the investigation; however, NOVA coordinated with the project engineer to provide infiltration testing in the areas most likely to have BMPs.

Two (2) percolation test borings (P-1 and P-2) were constructed following the recommendations for percolation testing presented in the City of Oceanside BMP Design Manual (hereinafter, 'the BMP Manual').

The percolation borings were drilled with a truck-mounted 8-inch hollow stem auger to a depth of approximately 10 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.

Once the borings were drilled to the desired depth, the borings were converted to percolation test borings by placing an approximately 2-inch layer of ¾-inch gravel on the bottom, 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 perforated pipe below existing finish grade to minimize the potential of soil caving.

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

In the percolation borings, the pre-soak water percolated at least 6 inches into the soil unit within 25 minutes. Based on the results of the trials, water levels were recorded every 10 minutes for 1-hour. At the beginning of each test interval, the water level was raised to approximately the same level as the previous tests, in order to maintain a near-constant head during all test periods.

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

Table 8-1. Infiltration Rate Test Results

Test Location	Test Depth (feet)	Material at Test Depth	Tested Infiltration Rate (inch/hour) FS=2
P-1	10	Old Paralic Deposits: Silty Sandstone	1.16
P-2	10	Old Paralic Deposits: Silty Sandstone	1.10

Note 1: elevations are approximate and should be reviewed
Note 2: 'FS' indicates 'Factor of Safety'

As may be seen by review of Table 9-1, a factor of safety (FS) is applied to the calculated infiltration rate (I) determined by the percolation testing. This factor of safety, at least FS = 2 in local practice, considers the nature and variability of subsurface materials, as well as the natural tendency of infiltration structures to become less efficient with time.

Full infiltration BMPs may be considered with infiltration rates greater than 0.5 inches per hour.

Appendix D presents Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions. Based on the measured infiltration rates, it is NOVA's judgment that the site is suitable for full infiltration within the Quaternary Old Paralic Deposits near the percolation test wells, such that factors listed in Section C.2 of Appendix C of the BMP Manual will not adversely impact the future and neighboring structures. Additionally, BMP facilities should be kept at least 10 feet from structural foundations.

9. CLOSURE

NOVA should review project plans and specifications prior to bidding and construction to check that the intent of the recommendations in this report has been incorporated. Observations and tests should be performed during construction. If the conditions encountered during construction differ from those anticipated based on the subsurface exploration program, the presence of personnel from NOVA during construction will enable an evaluation of the exposed conditions and modifications of the recommendations in this report or development of additional recommendations in a timely manner.

NOVA should be advised of changes in the project scope so that the recommendations contained in this report can be evaluated with respect to the revised plans. Changes in recommendations will be verified in writing. The findings in this report are valid as of the date of this report. Changes in the condition of the site can, however, occur with the passage of time, whether they are due to natural processes or work on this or adjacent areas. In addition, changes in the standards of practice and government regulations can occur. Thus, the findings in this report may be invalidated wholly or in part by changes beyond NOVA's control. This report should not be relied upon after a period of two years without a review by NOVA verifying the suitability of the conclusions and recommendations to site conditions at that time.

In the performance of professional services, NOVA exercises the level of care and skill ordinarily exercised by members of the geotechnical profession currently practicing under similar conditions and in the same locality. The client recognizes that subsurface conditions may vary from those encountered at the boring locations and that the data, interpretations, and recommendations reported herein are based solely on the information obtained by NOVA. NOVA will be responsible for those data, interpretations, and recommendations, but shall not be responsible for interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty whatsoever, express or implied, is made or intended in connection with the work performed or to be performed by us, or by our proposal for consulting or other services, or by our furnishing of oral or written reports or findings.

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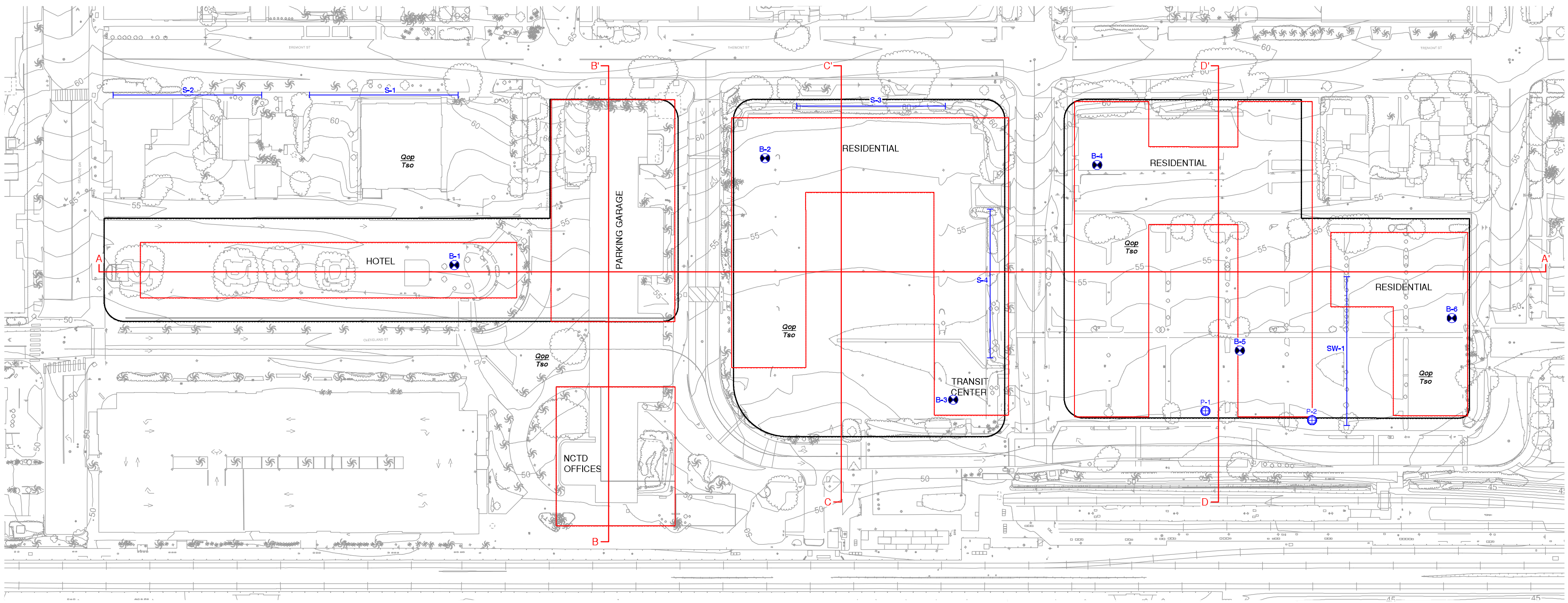
PLATES



KEY TO SYMBOLS

Qop	OLD PARALIC DEPOSITS
Tso	SAN ONOFRE BRECCIA
	LIMITS OF PROPOSED STRUCTURES BELOW GRADE
	LIMITS OF PROPOSED STRUCTURES ABOVE GRADE
B-6	GEOTECHNICAL BORING
P-2	PERCOLATION TEST BORING
SW-1	SHEAR WAVE VELOCITY SURVEY
S-4	SEISMIC REFRACTION SURVEYS
D D'	GEOLOGIC CROSS-SECTION

0 50' 100'

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P: 858.292.7575

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PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT

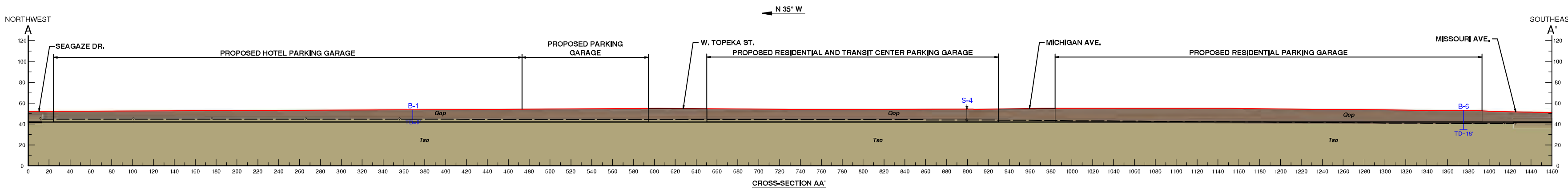
OCEANSIDE TRANSIT CENTER
OCEANSIDE, CA 92054

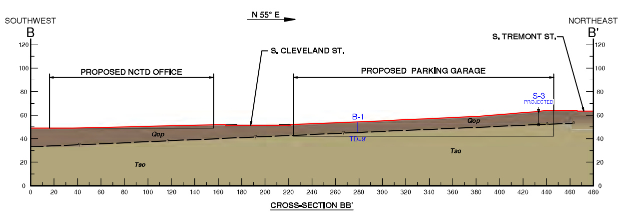
PROJECT NO.:	2021010
DATE:	AUGUST 2021
DRAWN BY:	DTJ
REVIEWED BY:	MS
SCALE:	1"=50'

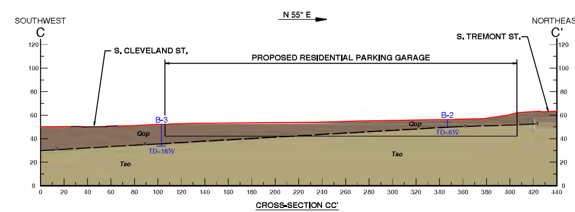
DRAWING TITLE:

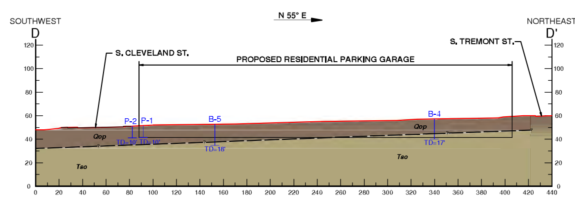
SUBSURFACE INVESTIGATION MAP

PLATE NO. 1 OF 2









KEY TO SYMBOLS

Qop	OLD PARALIC DEPOSITS
Tso	SAN ONOFRE BRECCIA
B-6	GEOTECHNICAL BORING
P-2	PERCOLATION TEST BORING
	EXISTING GRADE
	PROPOSED GRADE

0 50' 100'

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OCEANSIDE, CA 92054

PROJECT NO.:	2021010
DATE:	AUGUST 2021
DRAWN BY:	DTJ
REVIEWED BY:	MS
SCALE:	1"=50'
DRAWING TITLE:	

CROSS-SECTION AA' BB' CC' AND DD'

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

APPENDIX B
BORING LOGS

MAJOR DIVISIONS				TYPICAL NAMES
COARSE-GRAINED SOILS MORE THAN HALF IS COARSER THAN NO. 200 SIEVE	GRAVEL MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVEL WITH LESS THAN 15% FINES	GW	WELL-GRADED GRAVEL WITH OR WITHOUT SAND
			GP	POORLY GRADED GRAVEL WITH OR WITHOUT SAND
		GRAVEL WITH 15% OR MORE FINES	GM	SILTY GRAVEL WITH OR WITHOUT SAND
			GC	CLAYEY GRAVEL WITH OR WITHOUT SAND
	SAND MORE THAN HALF COARSE FRACTION IS FINER THAN NO. 4 SIEVE SIZE	CLEAN SAND WITH LESS THAN 15% FINES	SW	WELL-GRADED SAND WITH OR WITHOUT GRAVEL
			SP	POORLY GRADED SAND WITH OR WITHOUT GRAVEL
		SAND WITH 15% OR MORE FINES	SM	SILTY SAND WITH OR WITHOUT GRAVEL
			SC	CLAYEY SAND WITH OR WITHOUT GRAVEL
FINE-GRAINED SOILS MORE THAN HALF IS FINER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50% OR LESS		ML	SILT WITH OR WITHOUT SAND OR GRAVEL
			CL	LEAN CLAY WITH OR WITHOUT SAND OR GRAVEL
			OL	ORGANIC SILT OR CLAY OF LOW TO MEDIUM PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50%		MH	ELASTIC SILT WITH OR WITHOUT SAND OR GRAVEL
			CH	FAT CLAY WITH OR WITHOUT SAND OR GRAVEL
			OH	ORGANIC SILT OR CLAY OF HIGH PLASTICITY WITH OR WITHOUT SAND OR GRAVEL
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

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

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

RELATIVE DENSITY OF COHESIONLESS SOILS	
RELATIVE DENSITY	SPT N60 BLOWS/FOOT
VERY LOOSE	0 - 4
LOOSE	4 - 10
MEDIUM DENSE	10 - 30
DENSE	30 - 50
VERY DENSE	OVER 50

CONSISTENCY OF COHESIVE SOILS		
CONSISTENCY	SPT N60 BLOWS/FOOT	POCKET PENETROMETER MEASUREMENT (TSF)
VERY SOFT	0 - 2	0 - 0.25
SOFT	2 - 4	0.25 - 0.50
MEDIUM STIFF	4 - 8	0.50 - 1.0
STIFF	8 - 15	1.0 - 2.0
VERY STIFF	15 - 30	2.0 - 4.0
HARD	OVER 30	OVER 4.0

NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).
IF THE SEATING INTERVAL (1st 6 INCH INTERVAL) IS NOT ACHEIVED, N IS REPORTED AS REF.

SUBSURFACE EXPLORATION LEGEND

LOG OF BORING B-1

DATE DRILLED:	APRIL 8, 2021	**DRILLING METHOD:**	HOLLOW STEM AUGER		
ELEVATION:	53 FT MSL (GOOGLE EARTH)	**DRILLING EQUP.:**	CME 75 LAR	**GROUNDWATER DEPTH:**	NOT ENCOUNTERED
SAMPLE METHOD:	HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC)	**NOTES:**	ETR=80.8%, N_{60} = 80.8*N=1.35*N		

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0							SM	**OLD PARALIC DEPOSITS (Qop6-7):** *(DISTURBED/REWORKED) SILTY SAND WITH SOME CLAY; DARK REDDISH BROWN, SLIGHTLY MOIST, LOOSE TO MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED CONCRETE DEBRIS, BRICK, AND GRAVEL*	SA
			29	25	8.1	129.2		*(UNDISTURBED) REDDISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
5			19	26					
								ORANGE BROWN	
			15	13	5.4	114.5		*OLIVE BROWN, MOIST, MEDIUM GRAINED DIFFICULT AUGERING*	SA
			50/0"	67/0"				**SAN ONOFRE BRECCIA (Tso):** *NO RECOVERY, VERY DENSE*	
10								*BORING TERMINATED AT 9 FT DUE TO AUGER REFUSAL ON VERY DENSE SAN ONOFRE BRECCIA. NO GROUNDWATER ENCOUNTERED.*	
15									
20									
25									
30									

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FIGURE B.1

LOGGED BY: GN	REVIEWED BY: MS	PROJECT NO.: 2021010

LOG OF BORING B-2

DATE DRILLED: APRIL 8, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: 54 FT MSL (GOOGLE EARTH) **DRILLING EQUP.:** CME 75 **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC) **NOTES:** ETR=73.9%, $N_{60} = \frac{73.9}{60}$*N=1.23*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*2.5 INCHES OF ASPHALT CONCRETE OVER 4.5 INCHES OF AGGREGATE BASE*	
			12	15			SM	**OLD PARALIC DEPOSITS (Qop6-7):** *SILTY SAND; REDDISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, WITH SLIGHT PINHOLE POROSITY*	
			15	18					SA
5			21	26					
			50/1"	62/1"				**SAN ONOFRE BRECCIA (Tso):** *NO RECOVERY, VERY DENSE*	
								BORING TERMINATED AT 6½ FT DUE TO AUGER REFUSAL ON VERY DENSE SAN ONOFRE BRECCIA. NO GROUNDWATER ENCOUNTERED.	
10									
15									
20									
25									
30									

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FIGURE B.2

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LOG OF BORING B-3

DATE DRILLED: APRIL 8, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: 50 FT MSL (GOOGLE EARTH) **DRILLING EQUP.:** CME 75 **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC) **NOTES:** ETR=73.9%, $N_{60} = \frac{73.9}{60}*N = 1.23*N$

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*2.5 INCHES OF ASPHALT CONCRETE OVER 4.5 INCHES OF AGGREGATE BASE*	
			11	14			SM	**OLD PARALIC DEPOSITS (Qop6-7):** *SILTY SAND; DARK REDDISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	RV
			12	15				*REDDISH BROWN WITH OLIVE BROWN AND BLACK SAND GRAINS*	SA
5			21	26				*SOME MINOR CALICHE AND MINOR POROSITY*	
			27	22				*ORANGE BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	DS
10								*VERY DENSE DRILLING*	
			23	28			SP-SM	*POORLY GRADED SAND WITH SILT; LIGHT ORANGE BROWN AND OLIVE BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED SCATTERED CALICHE*	SA
15			33	41				*DENSE*	SA
			50/5"	62/5"			SC	**SAN ONOFRE BRECCIA (Tso):** *CLAYEY SANDSTONE; GRAYISH BROWN WITH ORANGE, SLIGHTLY MOIST TO MOIST, VERY DENSE, FINE TO MEDIUM GRAINED, ABUNDANT GRAVEL AND BROKEN ROCK, VERY DIFFICULT AUGERING*	SA
20								*BORING TERMINATED AT 18½ FT DUE TO AUGER REFUSAL ON VERY DENSE SAN ONOFRE BRECCIA. NO GROUNDWATER ENCOUNTERED.*	
25									
30									

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FIGURE B.3

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LOG OF BORING B-4

DATE DRILLED: APRIL 8, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: 53 FT MSL (GOOGLE EARTH) **DRILLING EQUP.:** CME 75 **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC) **NOTES:** ETR=73.9%, $N_{60} = \frac{73.9}{60}$*N=1.23*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** *SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)*	LAB TESTS
0								*2.5 INCHES OF ASPHALT CONCRETE OVER 4.5 INCHES OF AGGREGATE BASE*	
			13	16			SM	**OLD PARALIC DEPOSITS (Qop6-7):** *(DISTURBED) SILTY SAND; DARK BROWN, SLIGHTLY MOIST TO MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	SA
			22	27			SM-SC	*(UNDISTURBED) SILTY SAND WITH CLAY; REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SOME BLACK MINERALIZATION*	SA
5			22	27					SA
			14	17			SM	*SILTY SAND; REDDISH BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	SA
10			34	27				*ORANGE BROWN*	DS
			50/2"	62/2"			SM	**SAN ONOFRE BRECCIA (Tso):** *SILTY SANDSTONE; GRAYISH BROWN, DRY, VERY DENSE, FINE TO COARSE GRAINED, SOME ROCK FRAGEMENTS* *LIGHT YELLOWISH BROWN, DIFFICULT AUGERING*	SA
15			50/3"	40/3"			SC	*CLAYEY SANDSTONE; ORANGE BROWN WITH LIGHT GRAYISH BROWN, SLIGHTLY MOIST, VERY DENSE, FINE GRAINED*	
			50/3"	62/3"				*VERY DENSE, VERY HARD AUGERING*	SA
								BORING TERMINATED AT 17 FT DUE TO AUGER REFUSAL ON DENSE SAN ONOFRE BRECCIA. NO GROUNDWATER ENCOUNTERED.	
20									
25									
30									

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FIGURE B.4

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LOG OF BORING B-5

DATE DRILLED: APRIL 8, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: 49 FT MSL (GOOGLE EARTH) **DRILLING EQUP.:** CME 75 **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC) **NOTES:** ETR=73.9%, $N_{60} = \frac{73.9}{60} * N = 1.23 * N$

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0								*2.5 INCHES OF ASPHALT CONCRETE OVER 4.5 INCHES OF AGGREGATE BASE*	
			32	39			SM	**OLD PARALIC DEPOSITS (Qop6-7):** *(DISTURBED) SILTY SAND; DARK BROWN, SLIGHTLY MOIST TO MOIST, VERY DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL*	SA
			9	11				*(UNDISTURBED) REDDISH BROWN, MOIST, MEDIUM DENSE, FINE GRAINED*	SA
5			22	27				*REDDISH BROWN, SLIGHTLY MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED*	
			15	18				*MEDIUM GRAINED* *OLIVE BROWN TO ORANGE BROWN* *ORANGE BROWN TO REDDISH BROWN*	SA
10			40	32				*DENSE* *LIGHT YELLOWISH TO ORANGE BROWN*	DS
			24	32					MD SA
15			46	57			SM	**SAN ONOFRE BRECCIA (Tso):** *SILTY SANDSTONE; PALE OLIVE BROWN WITH LIGHT GRAY AND BLACK SAND GRAINS, DRY TO SLIGHTLY MOIST, VERY DENSE, FINE TO COARSE GRAINED, SCATTERED GRAVEL FRAGMENTS*	MD SA EI CR
			50/3"	62/3"				*MORE ABUNDANT ROCK FRAGMENTS, REFUSAL ON COBBLE*	
								BORING TERMINATED AT 18 FT DUE TO AUGER REFUSAL ON DENSE SAN ONOFRE BRECCIA. NO GROUNDWATER ENCOUNTERED.	
20									
25									
30									

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FIGURE B.5

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021010

LOG OF BORING B-6

DATE DRILLED: APRIL 8, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: 46 FT MSL (GOOGLE EARTH) **DRILLING EQUP.:** CME 75 **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC) **NOTES:** ETR=73.9%, $N_{60} = \frac{73.9}{60}$*N=1.23*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	SOIL DESCRIPTION SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0								*2.5 INCHES OF ASPHALT CONCRETE OVER 4.5 INCHES OF AGGREGATE BASE*	
			19	23			SM	**OLD PARALIC DEPOSITS (Qop6-7):** *SILTY SAND; DARK REDDISH BROWN, SLIGHTLY MOIST TO MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL*	MD EI CR
			27	33				*MOIST* *DENSE*	
5			39	31	5.1	130			
								ORANGE BROWN, DRY TO SLIGHTLY MOIST, FINE TO MEDIUM GRAINED	
			20	25				*MEDIUM DENSE, MOIST*	SA
10			23	28			SM/SP	*SILTY SAND/POORLY GRADED SAND; ORANGE BROWN, MOIST, MEDIUM DENSE, MEDIUM GRAINED, SCATTERED GRAVEL* *DARK BROWN, ABUNDANT BLACK SAND GRAINS*	
			65	80			SC	**SAN ONOFRE BRECCIA (Tso):** *CLAYEY SANDSTONE; PALE OLIVE BROWN, DRY, VERY DENSE, FINE TO COARSE GRAINED, WITH ABUNDANT GRAVEL FRAGMENTS* *LIGHT GRAYISH BROWN WITH ORANGE AND DARK GRAY STAINING*	MD EI CR
15			85/11"	68/11"					
			50/5"	62/5"				*PALE OLIVE GRAY*	
								BORING TERMINATED AT 18 FT DUE TO AUGER REFUSAL ON DENSE SAN ONOFRE BRECCIA. NO GROUNDWATER ENCOUNTERED.	
20									
25									
30									

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FIGURE B.5

LOGGED BY: GN | REVIEWED BY: MS | PROJECT NO.: 2021010

LOG OF PERCOLATION BORING P-1

DATE DRILLED: APRIL 8, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: 47 FT MSL (GOOGLE EARTH) **DRILLING EQUP.:** CME 75 LAR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC) **NOTES:** ETR=80.8%, N_{60} = 80.8*N=1.35*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0								*2.5 INCHES OF ASPHALT CONCRETE OVER 4.5 INCHES OF AGGREGATE BASE*	
							SM	**OLD PARALIC DEPOSITS (Qop_{6-7}):** *SILTY SAND; DARK BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL* *REDDISH BROWN*	
5								*MORE FINES*	
								OLIVE BROWN, SLIGHTLY MOIST TO MOIST, DENSE *DIFFICULT AUGERING*	
10								*BORING TERMINATED AT 10 FT AND CONVERTED TO A PERCOLATION WELL. NO GROUNDWATER ENCOUNTERED.*	
15									
20									
25									
30									

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FIGURE B.7

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LOG OF PERCOLATION BORING P-2

DATE DRILLED: APRIL 8, 2021 **DRILLING METHOD:** HOLLOW STEM AUGER

ELEVATION: 46 FT MSL (GOOGLE EARTH) **DRILLING EQUP.:** CME 75 LAR **GROUNDWATER DEPTH:** NOT ENCOUNTERED

SAMPLE METHOD: HAMMER: 140 LBS., DROP: 30 IN. (AUTOMATIC) **NOTES:** ETR=80.8%, N_{60} = 80.8*N=1.35*N

DEPTH (FT)	BULK SAMPLE	CAL/SPT SAMPLE	BLOWS PER FOOT N	N_{60}	MOISTURE (%)	DRY DENSITY (pcf)	SOIL CLASS. (USCS)	**SOIL DESCRIPTION** SUMMARY OF SUBSURFACE CONDITIONS (USCS; COLOR, MOISTURE, DENSITY, GRAIN SIZE, OTHER)	LAB TESTS
0								*2.5 INCHES OF ASPHALT CONCRETE OVER 4.5 INCHES OF AGGREGATE BASE*	
							SM	**OLD PARALIC DEPOSITS (Qop6-7):** *SILTY SAND; DARK BROWN, MOIST, MEDIUM DENSE, FINE TO MEDIUM GRAINED, SCATTERED GRAVEL* *REDDISH BROWN*	
5								*MORE FINES*	
	X							*OLIVE BROWN, SLIGHTLY MOIST, DENSE*	SA
10								*BORING TERMINATED AT 10 FT AND CONVERTED TO A PERCOLATION WELL. NO GROUNDWATER ENCOUNTERED.*	
15									
20									
25									
30									

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FIGURE B.8

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APPENDIX C
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.*

- **GRADATION ANALYSIS (ASTM D6913):** *The grain size distribution of selected soil samples was determined in accordance with ASTM D6913. The results of the tests are summarized on Figures C.1 through C.21.*

- **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.*

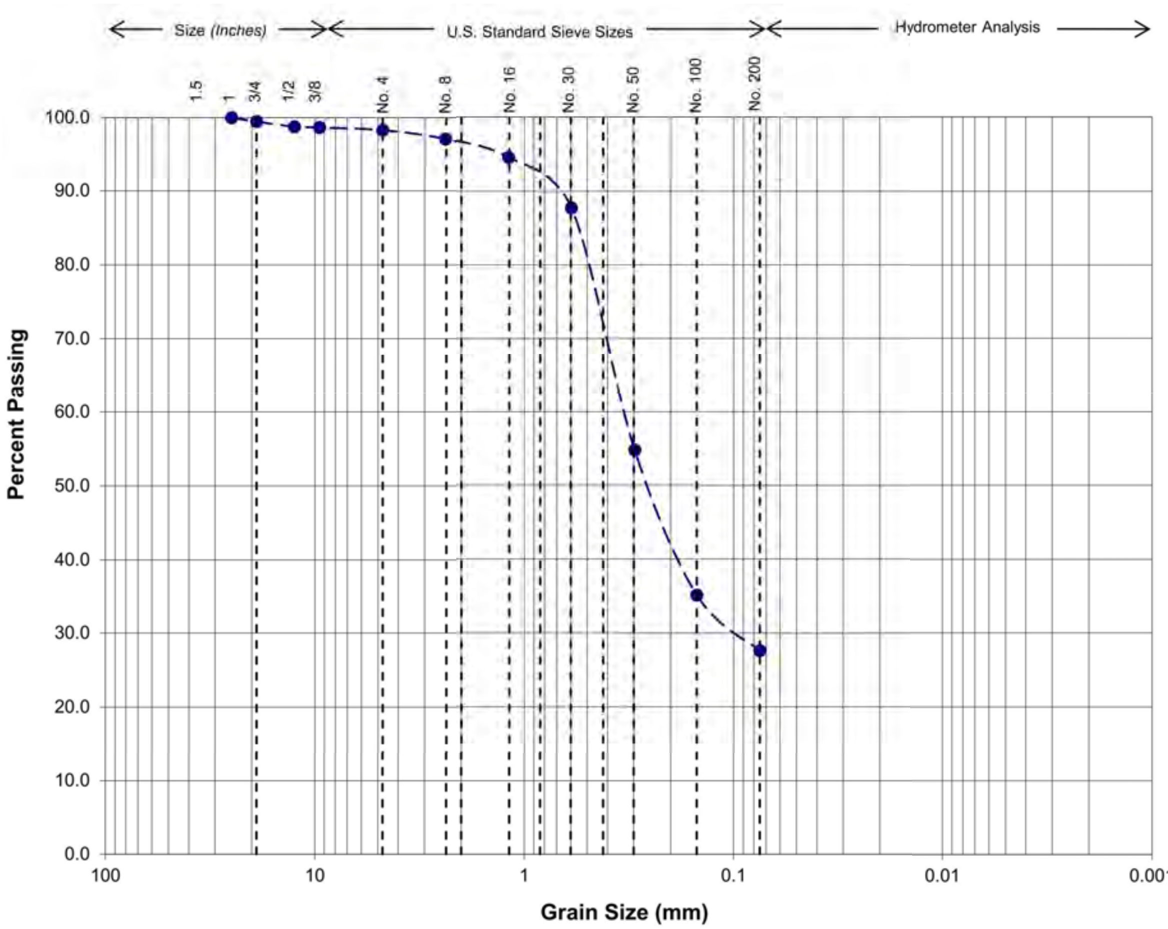
- **MAXIMUM DENSITY AND OPTIMUM MOISTURE CONTENT (ASTM D1557 METHOD A,B,C):** *The maximum dry density and optimum moisture content of typical soils were determined in the laboratory in accordance with ASTM Standard Test D1557, Method A, Method B, Method C.*

- **DIRECT SHEAR (ASTM D3080):** *Direct shear tests were performed on remolded and relatively undisturbed samples in general accordance with ASTM D3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results of the tests are summarized on Figures C.22 through C.24.*

- **R-VALUE (ASTM D2844):** *The resistance Value, or R-Value, of near-surface site soils was 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.*

- **CORROSIVITY (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.*

- **EXPANSION INDEX (ASTM D4829):** *The expansion index of selected materials was evaluated in general accordance with ASTM D4829. Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours.*



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<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 - 1
Depth (ft):	0 - 5
USCS Soil Type:	SM
Passing No. 200 (%):	28

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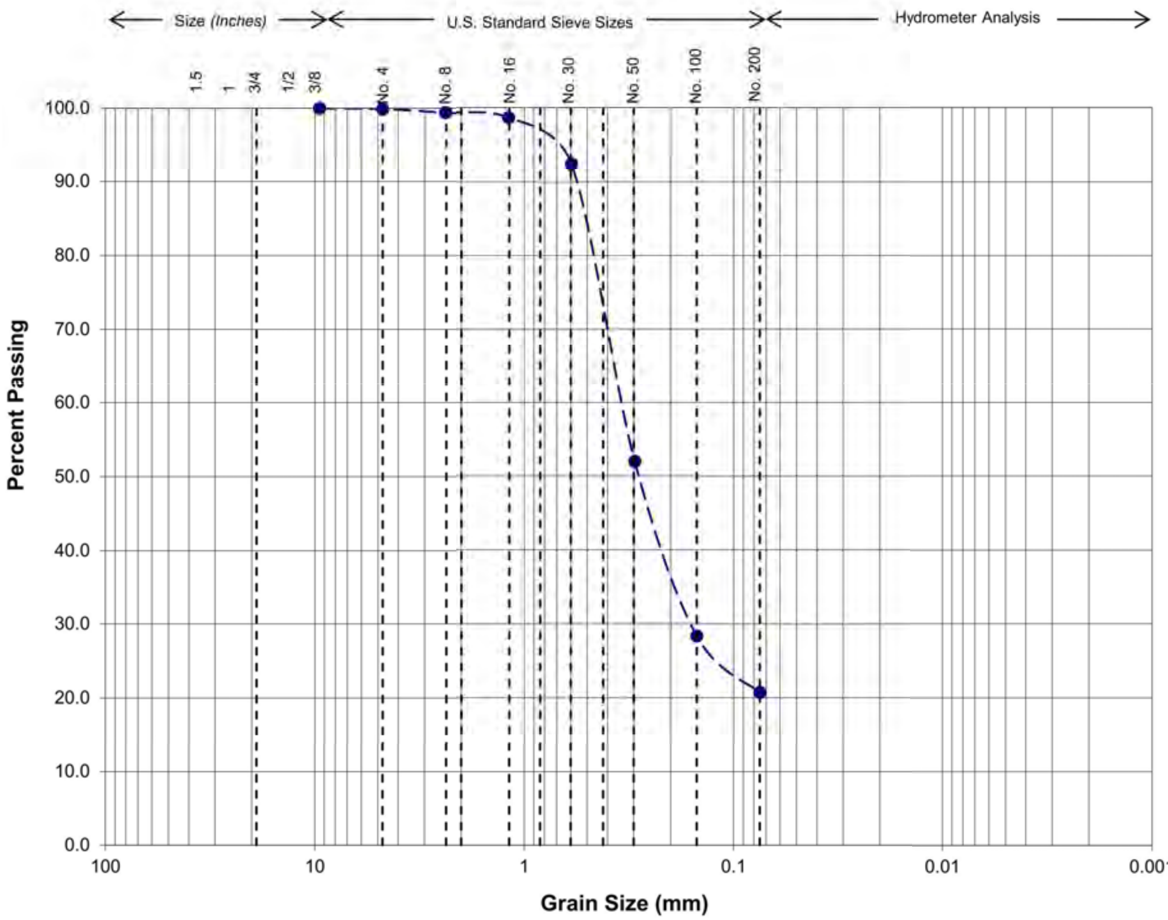

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location:	B - 1
Depth (ft):	7 - 8
USCS Soil Type:	SM
Passing No. 200 (%):	21

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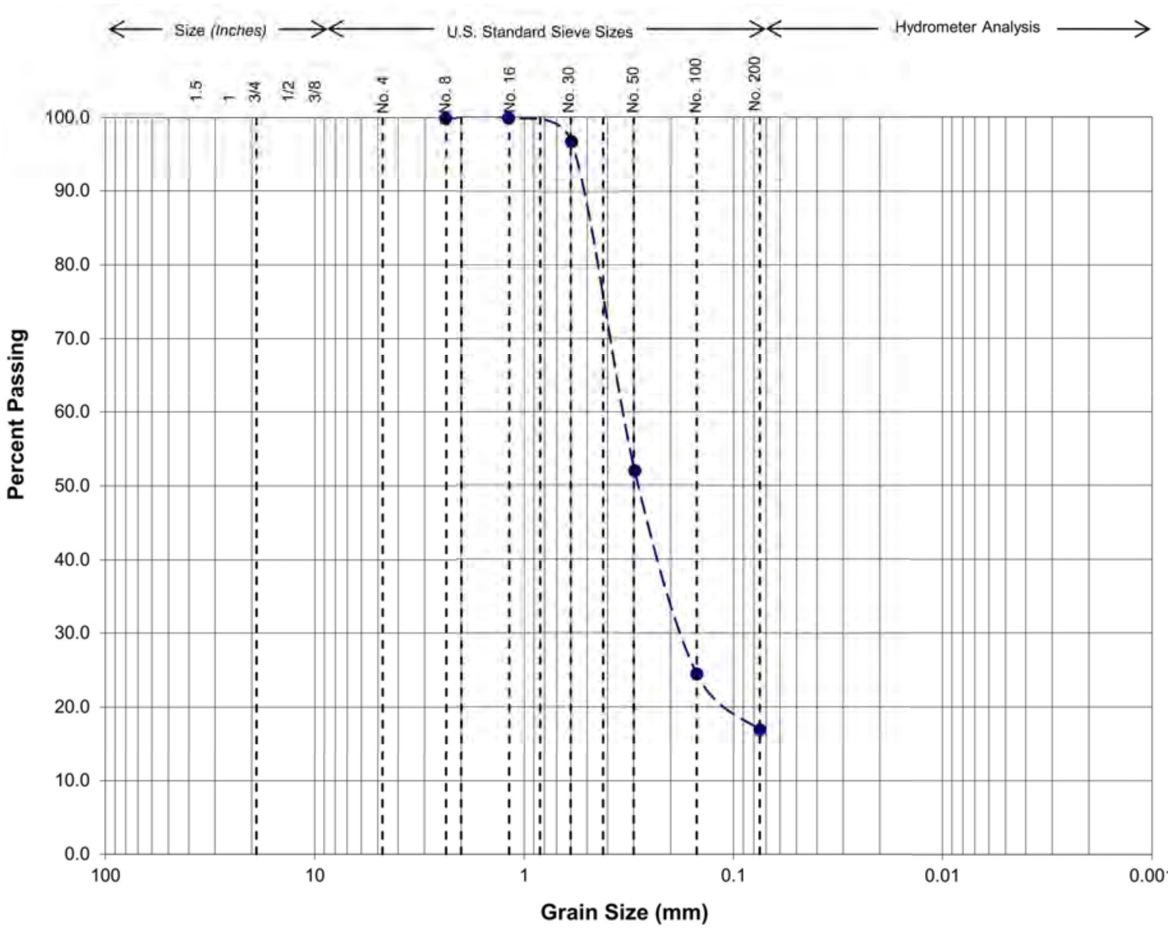
BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.3



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-2

Depth (ft): 2 ½ - 4

USCS Soil Type: SM-SC

Passing No. 200 (%): 17

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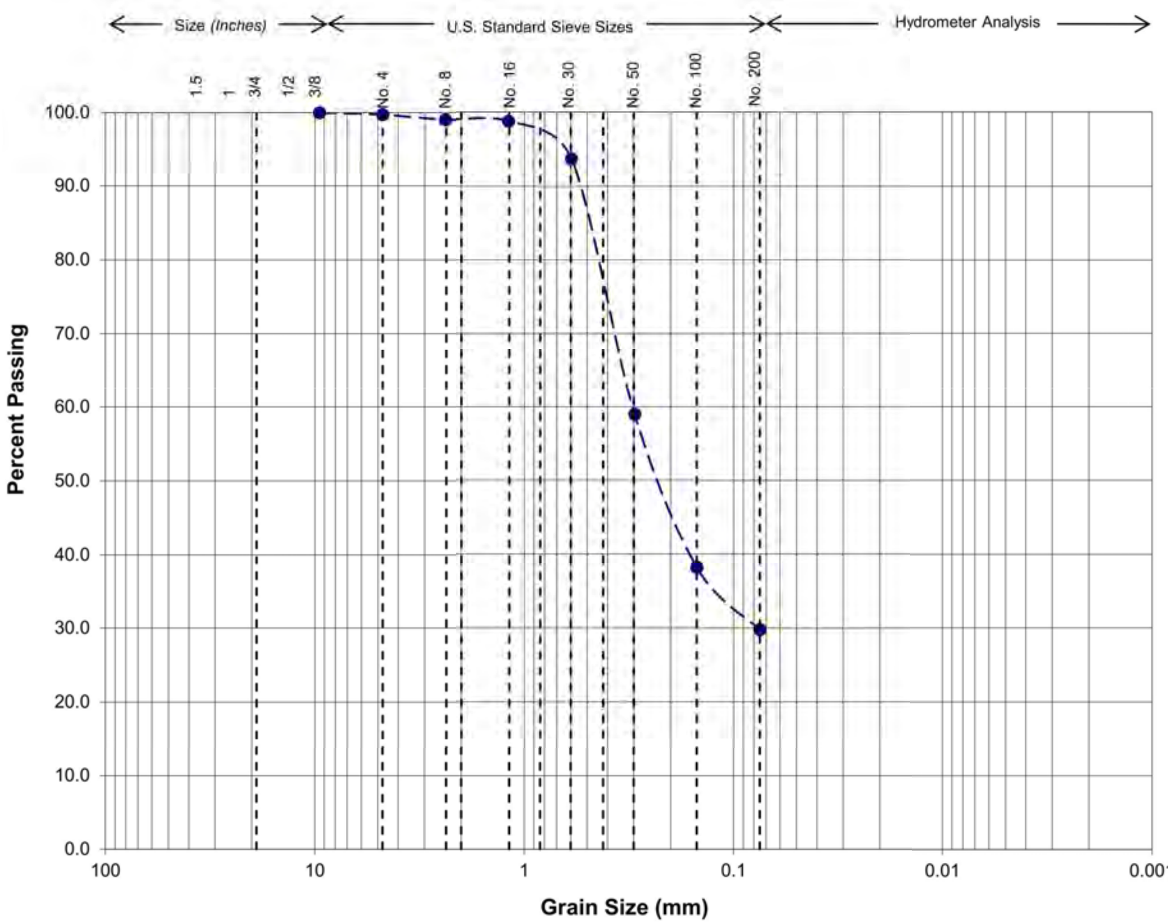
BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.4



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-3

Depth (ft): 2½ - 4

USCS Soil Type: SM-SC

Passing No. 200 (%): 30

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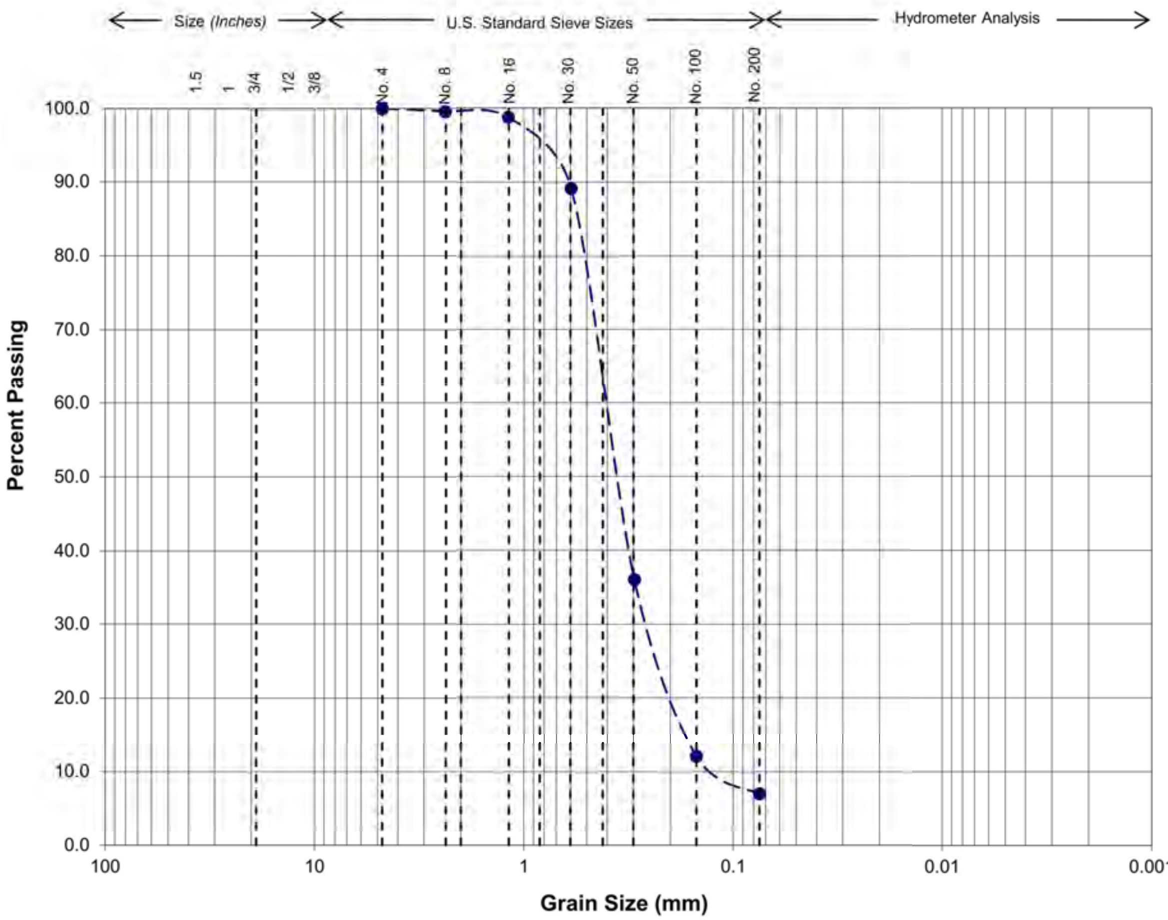

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-3

Depth (ft): 12 $\frac{1}{2}$ - 14

USCS Soil Type: SP-SM

Passing No. 200 (%): 7

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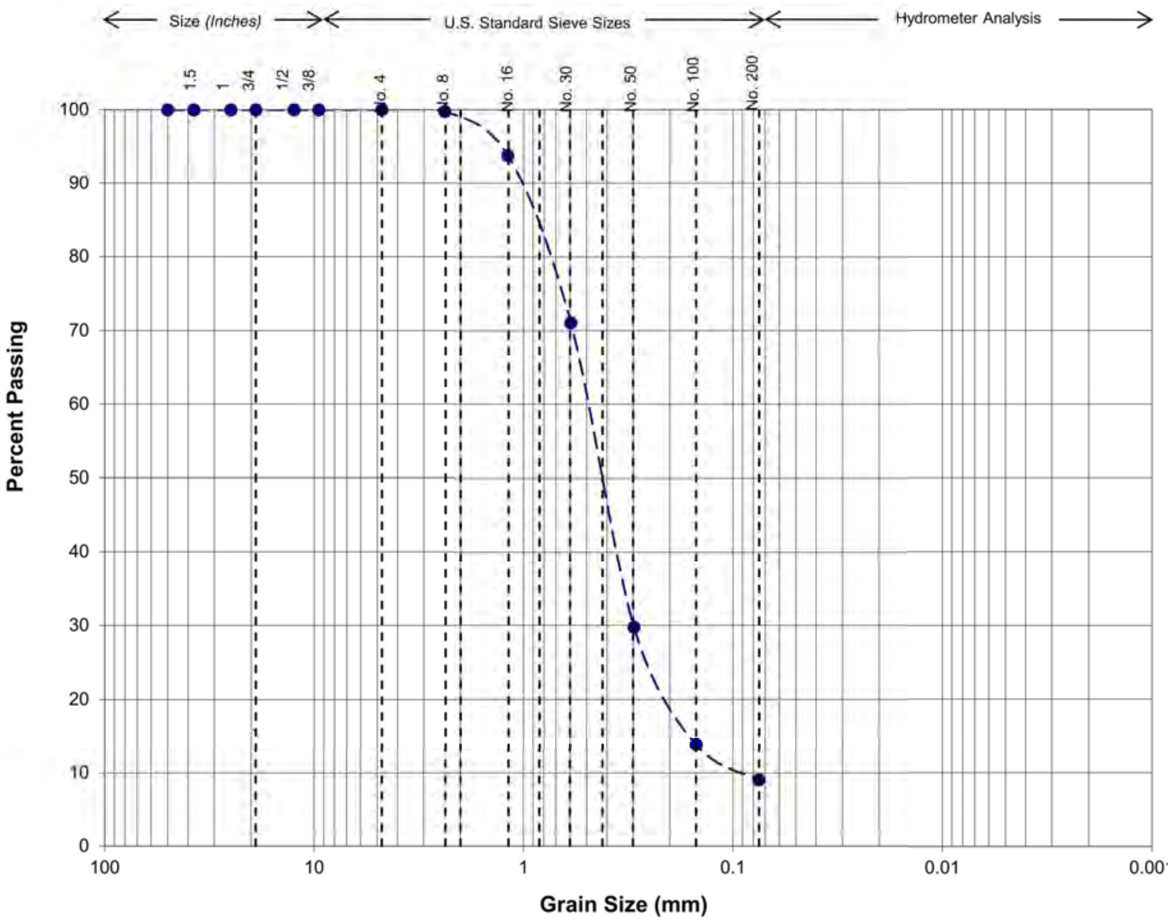

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<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location: B-3

Depth (ft): 15 - 16 $\frac{1}{2}$

USCS Soil Type: SP-SM

Passing No. 200 (%): 9

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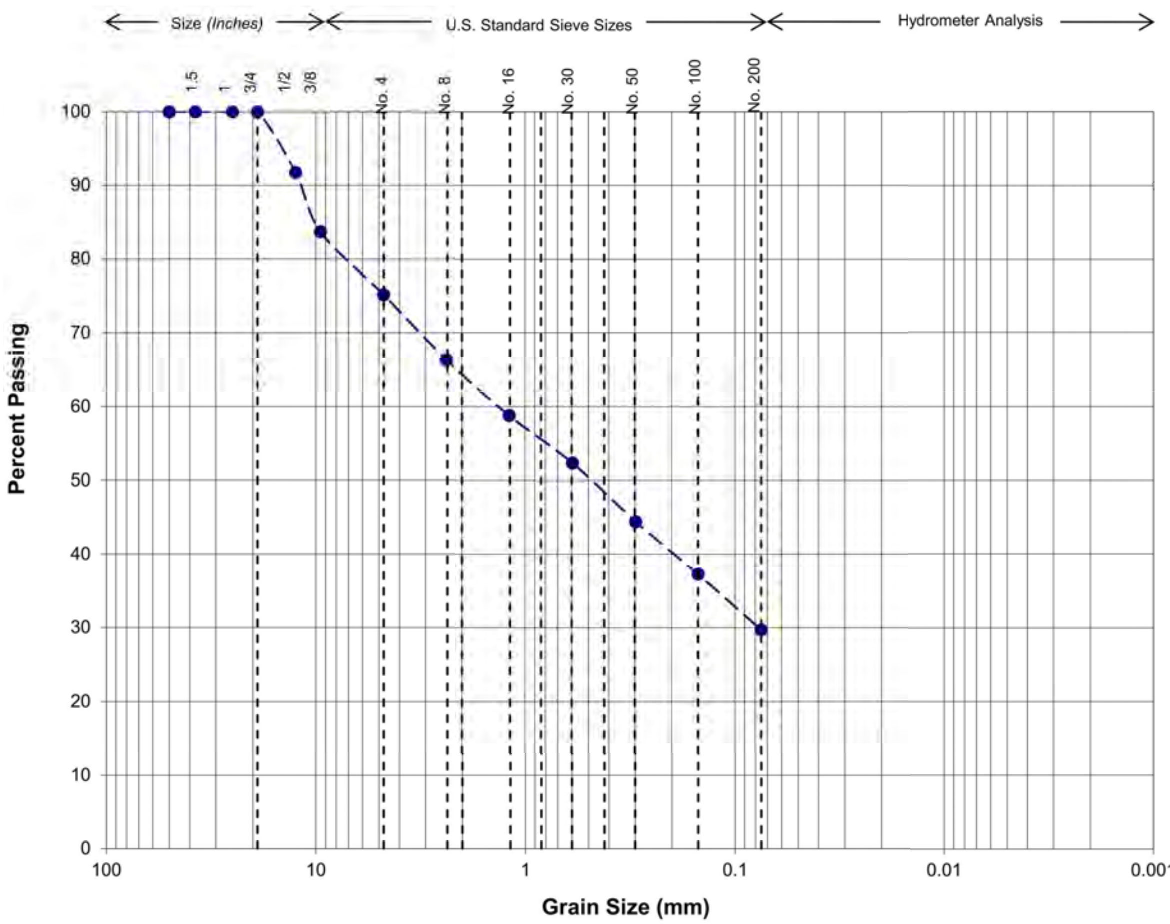
BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.7



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-3

Depth (ft): 17 ½ - 18

USCS Soil Type: SC

Passing No. 200 (%): 30

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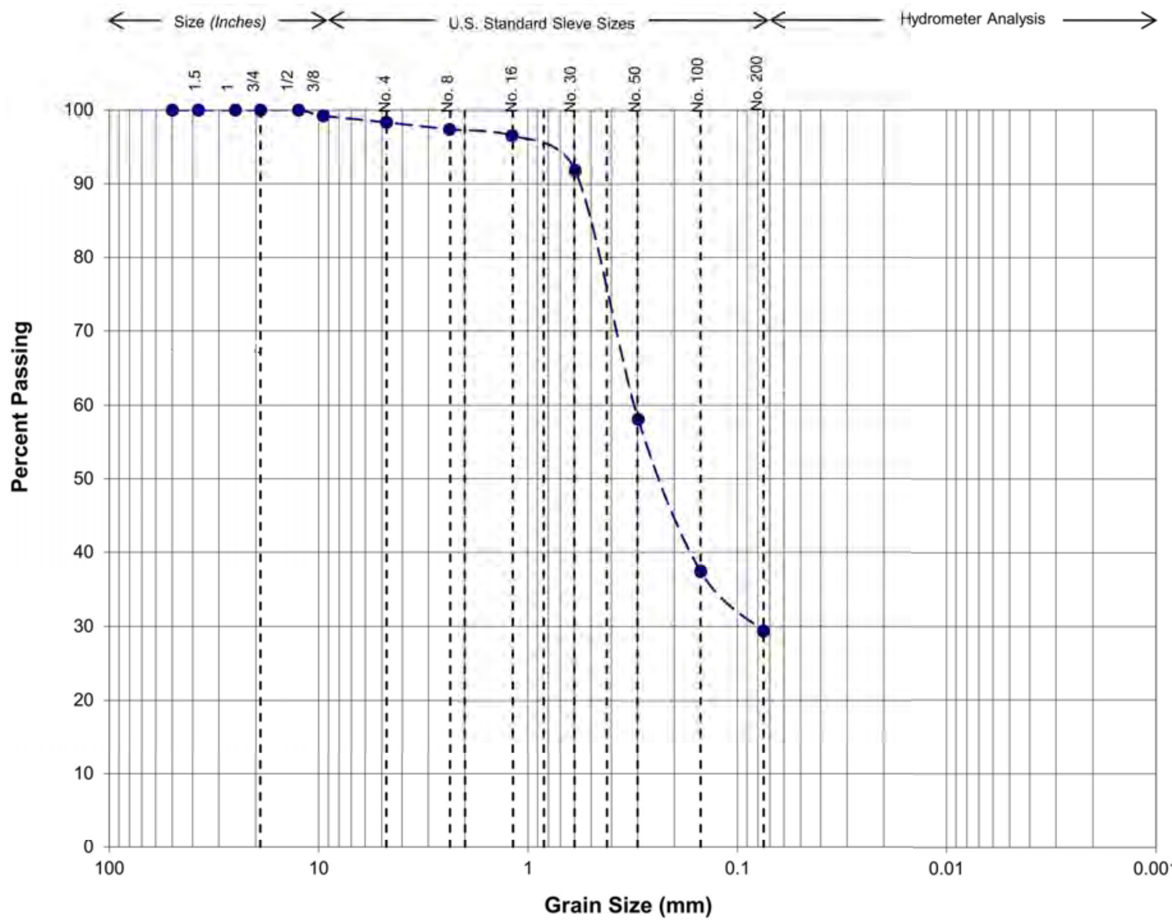
BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.8



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-4

Depth (ft): 2 - 3 $\frac{1}{2}$

USCS Soil Type: SM-SC

Passing No. 200 (%): 29

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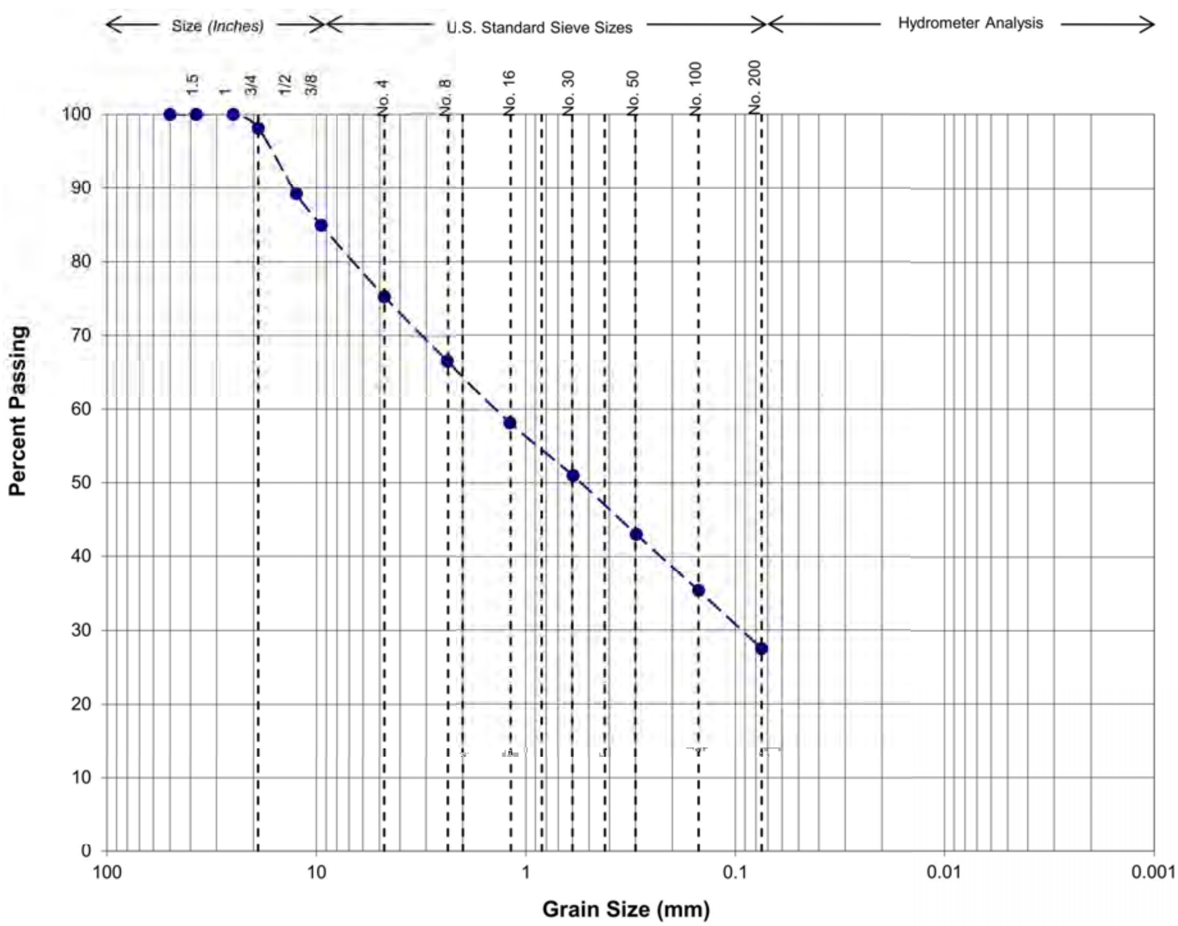

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-4

Depth (ft): 5 - 6 ½

USCS Soil Type: SM-SC

Passing No. 200 (%): 27

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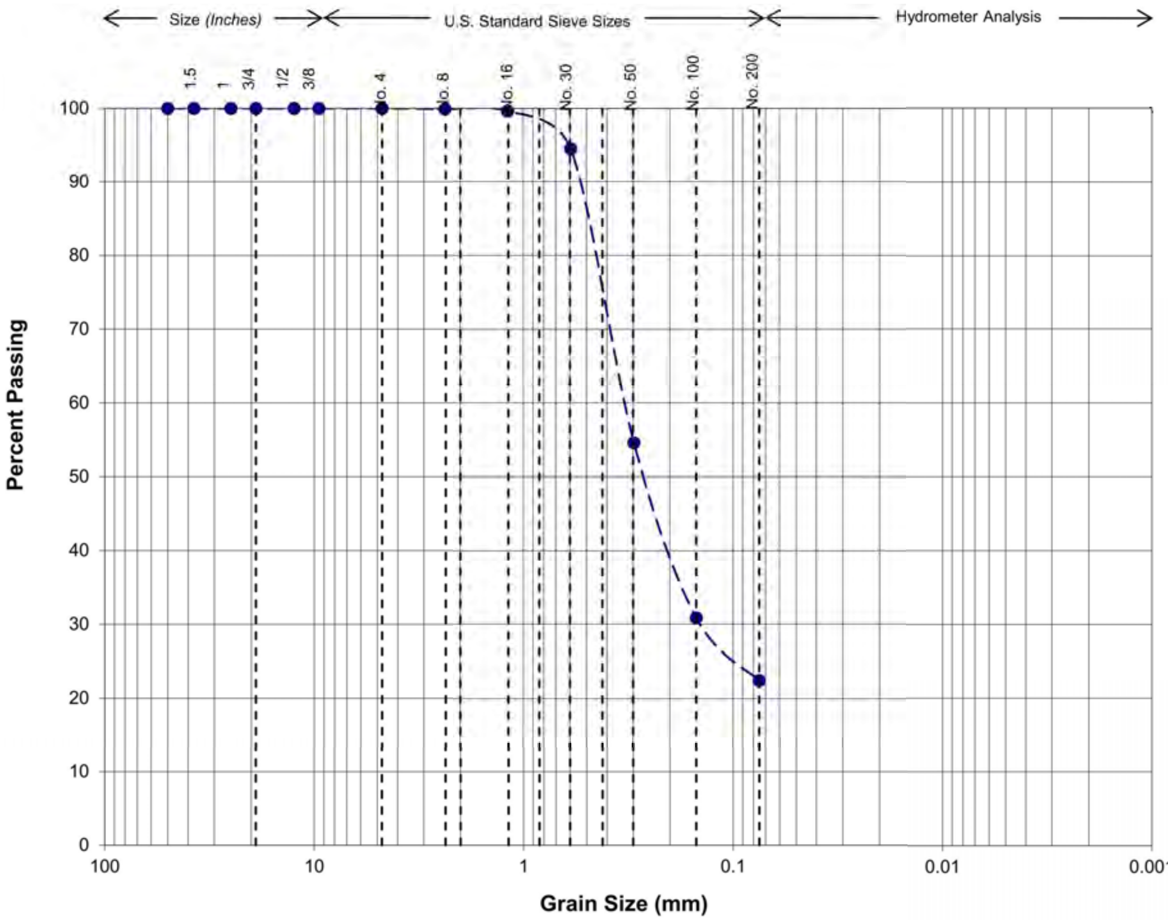

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-4

Depth (ft): 7 ½ - 9

USCS Soil Type: SM

Passing No. 200 (%): 22

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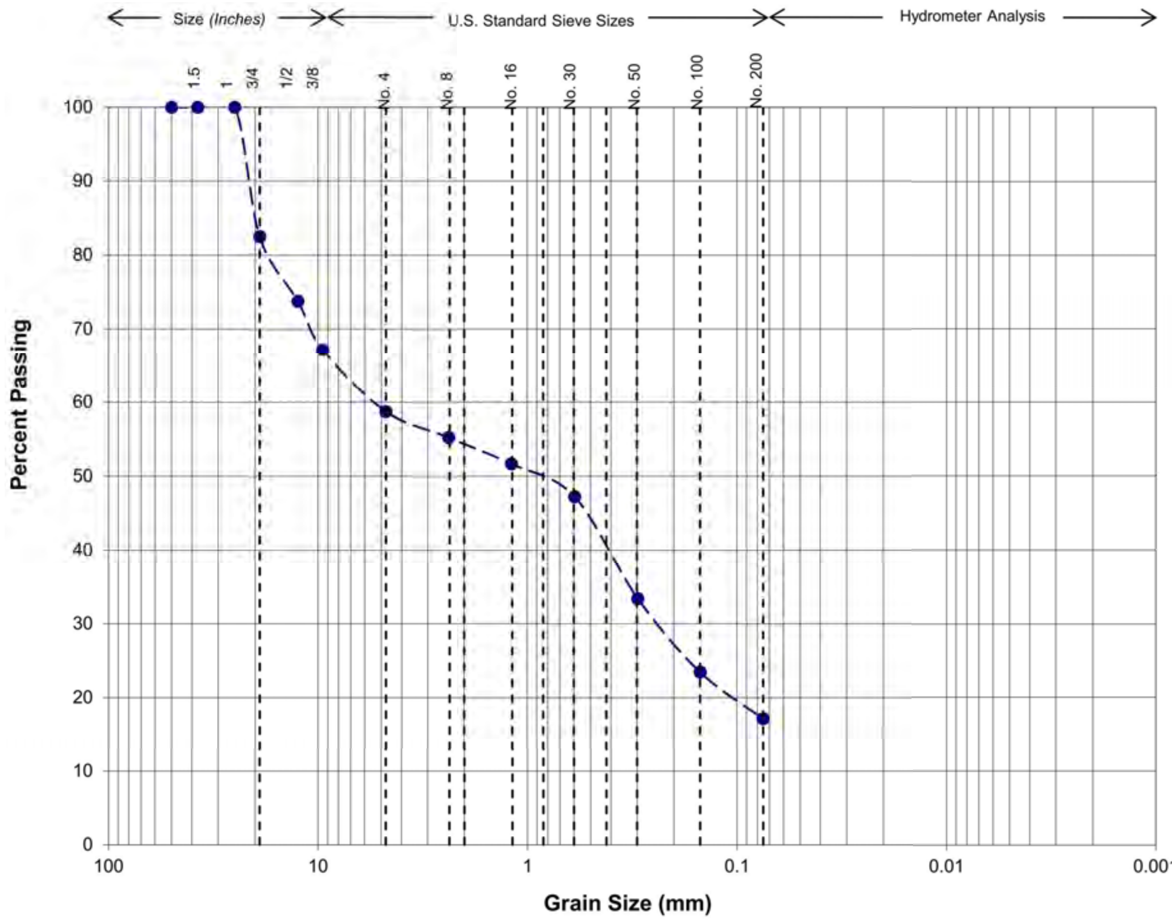

<table>
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<tr><td>Coarse</td><td>Fine</td><td>Coarse</td><td>Medium</td><td>Fine</td></tr>
</table>

Sample Location:	B-4
Depth (ft):	$12\frac{1}{2}$ - $13\frac{1}{2}$
USCS Soil Type:	SM
Passing No. 200 (%):	17

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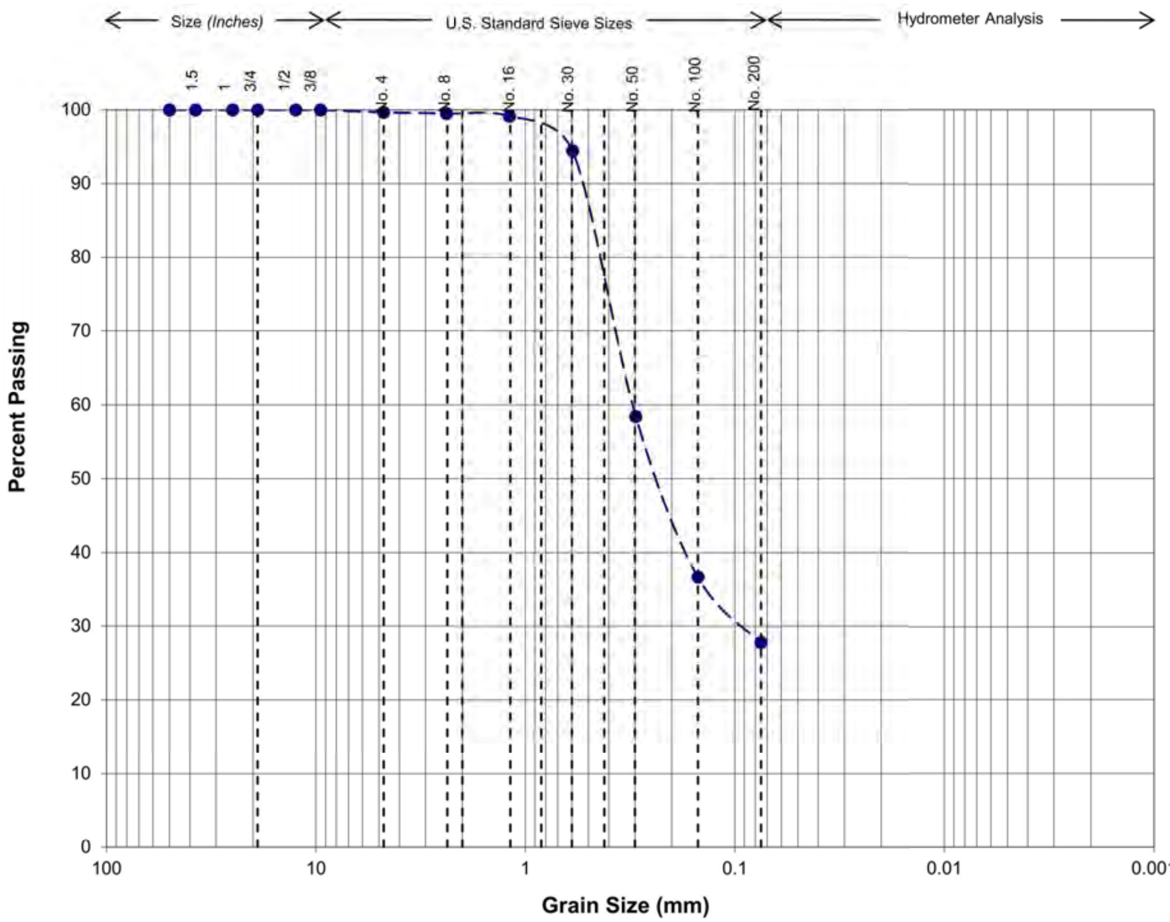

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-4

Depth (ft): 16 - 17

USCS Soil Type: SC

Passing No. 200 (%): 28

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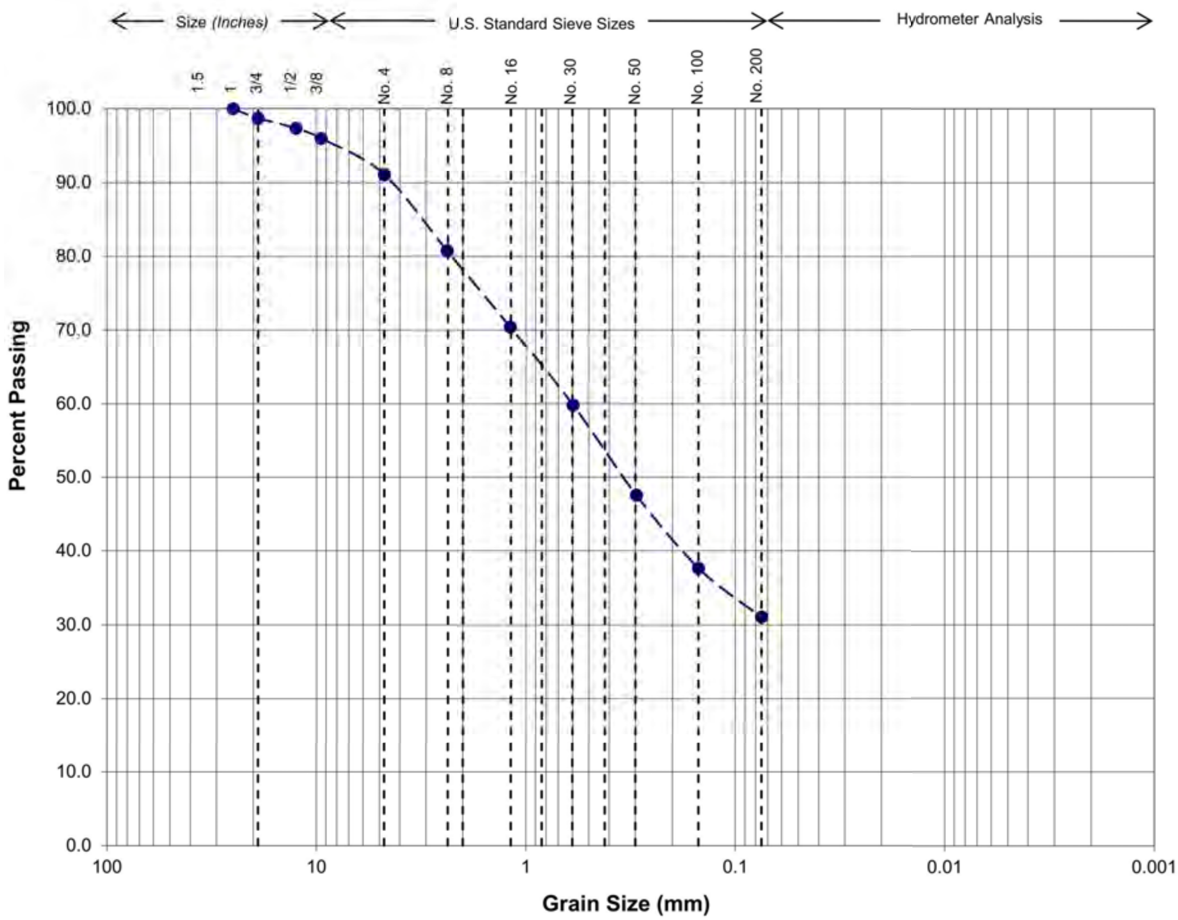

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-5

Depth (ft): 1 - 2½

USCS Soil Type: SM

Passing No. 200 (%): 31

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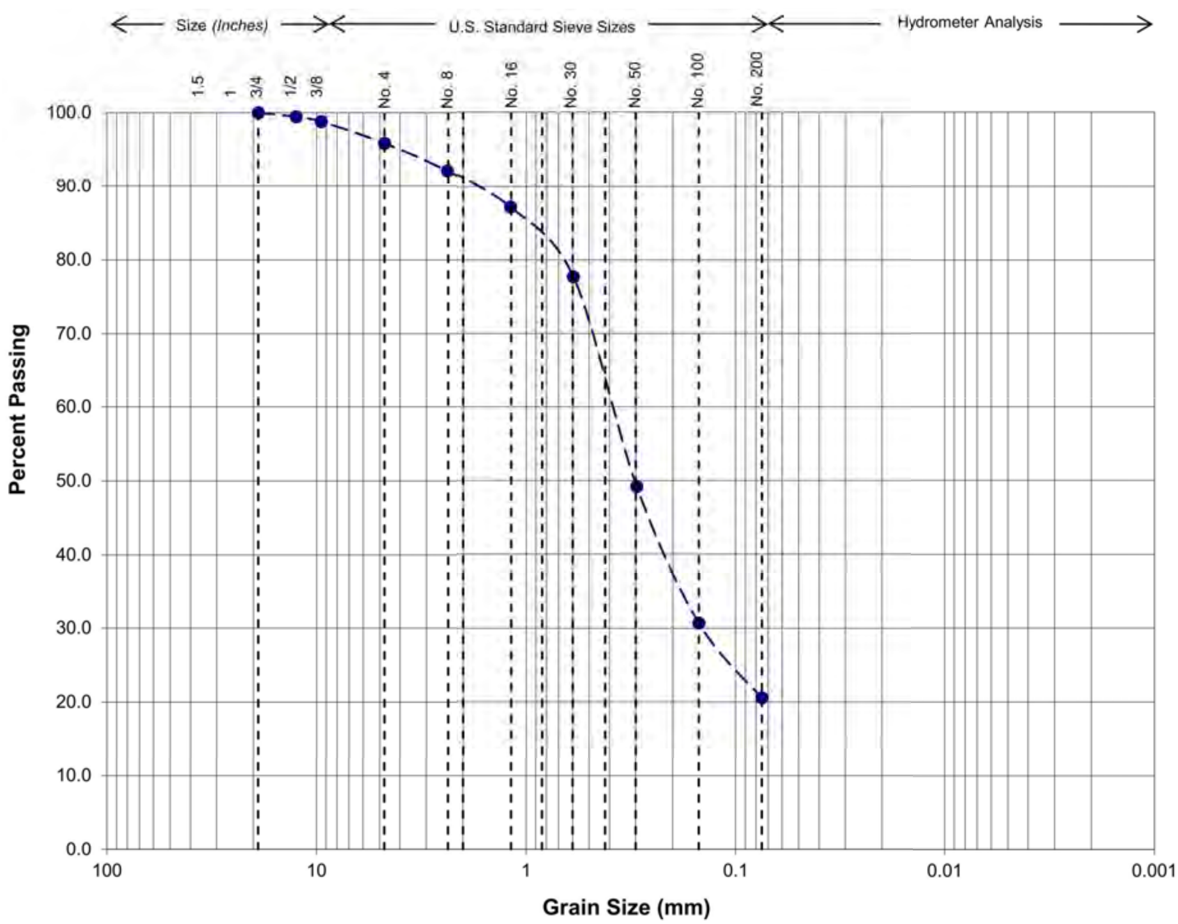
BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.14



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

Sample Location: B-5

Depth (ft): $3\frac{1}{2}$ - $6\frac{1}{2}$

USCS Soil Type: SM-SC

Passing No. 200 (%): 21

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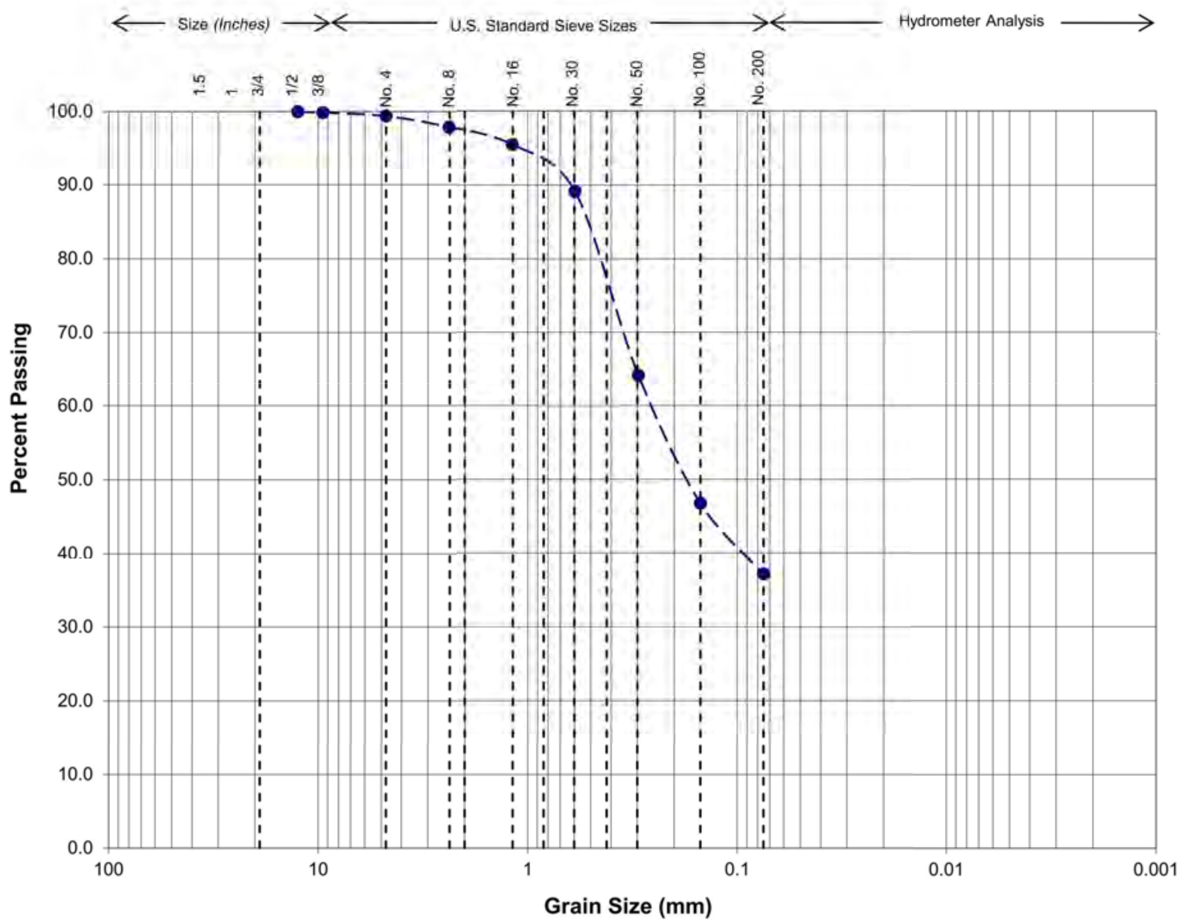

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-5

Depth (ft): 6 $\frac{1}{2}$ - 9

USCS Soil Type: SM-SC

Passing No. 200 (%): 37

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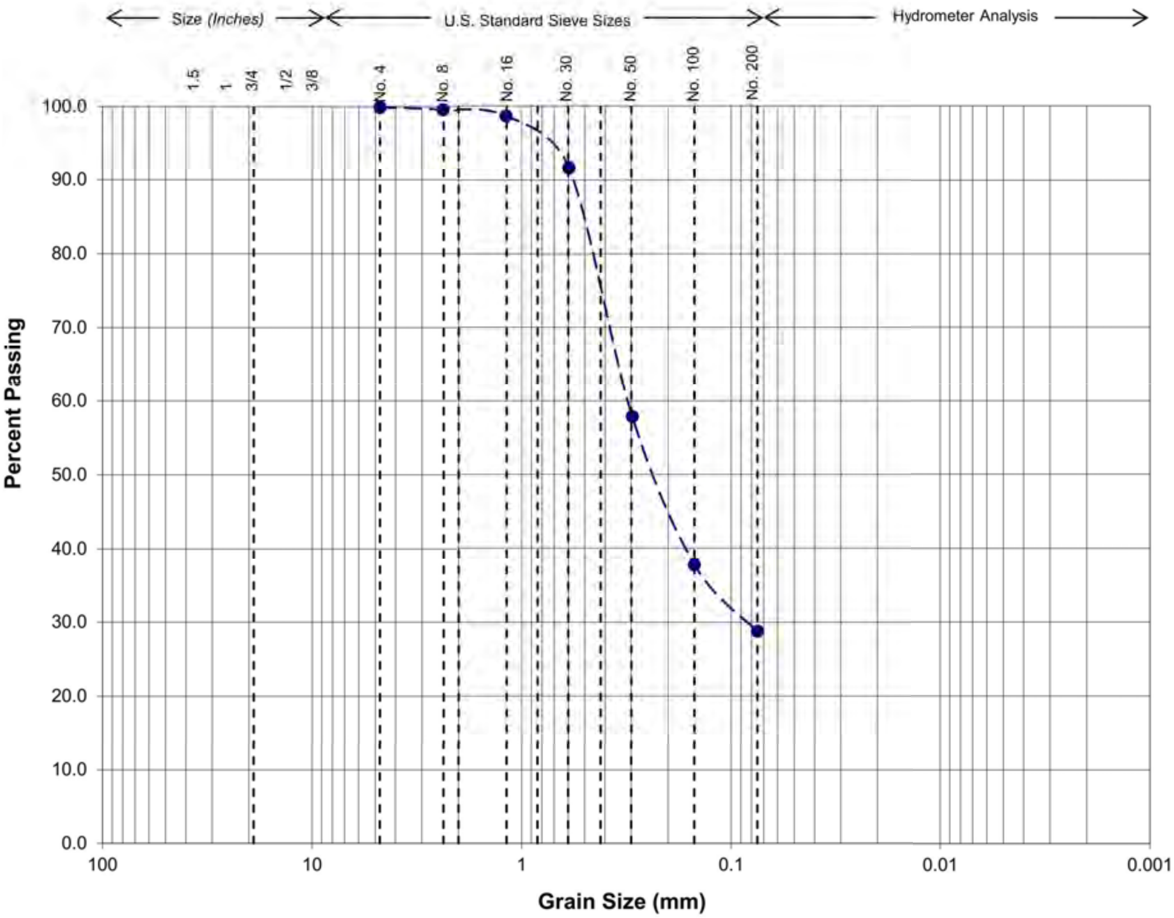

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-5

Depth (ft): 9 - 12

USCS Soil Type: SM

Passing No. 200 (%): 29

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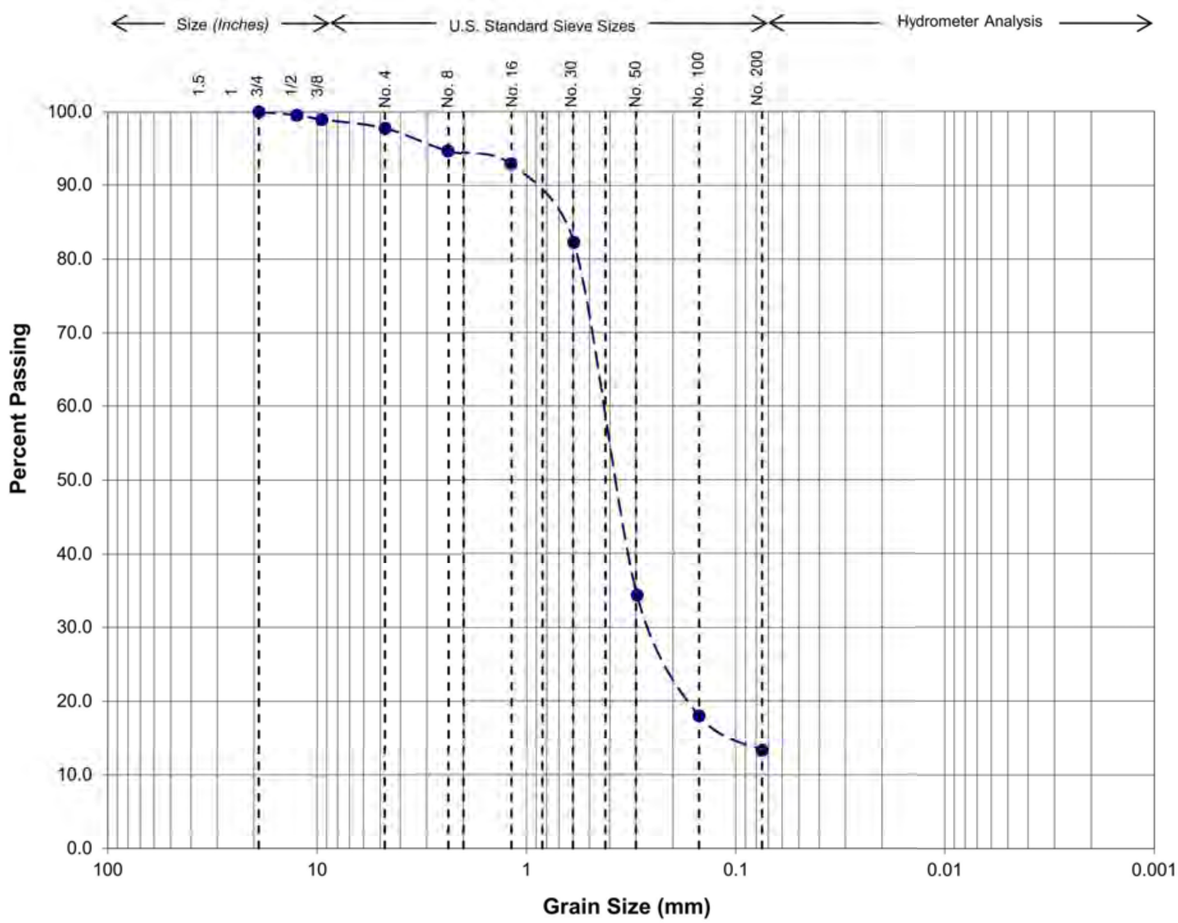

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-5

Depth (ft): 12 - 15

USCS Soil Type: SM

Passing No. 200 (%): 13

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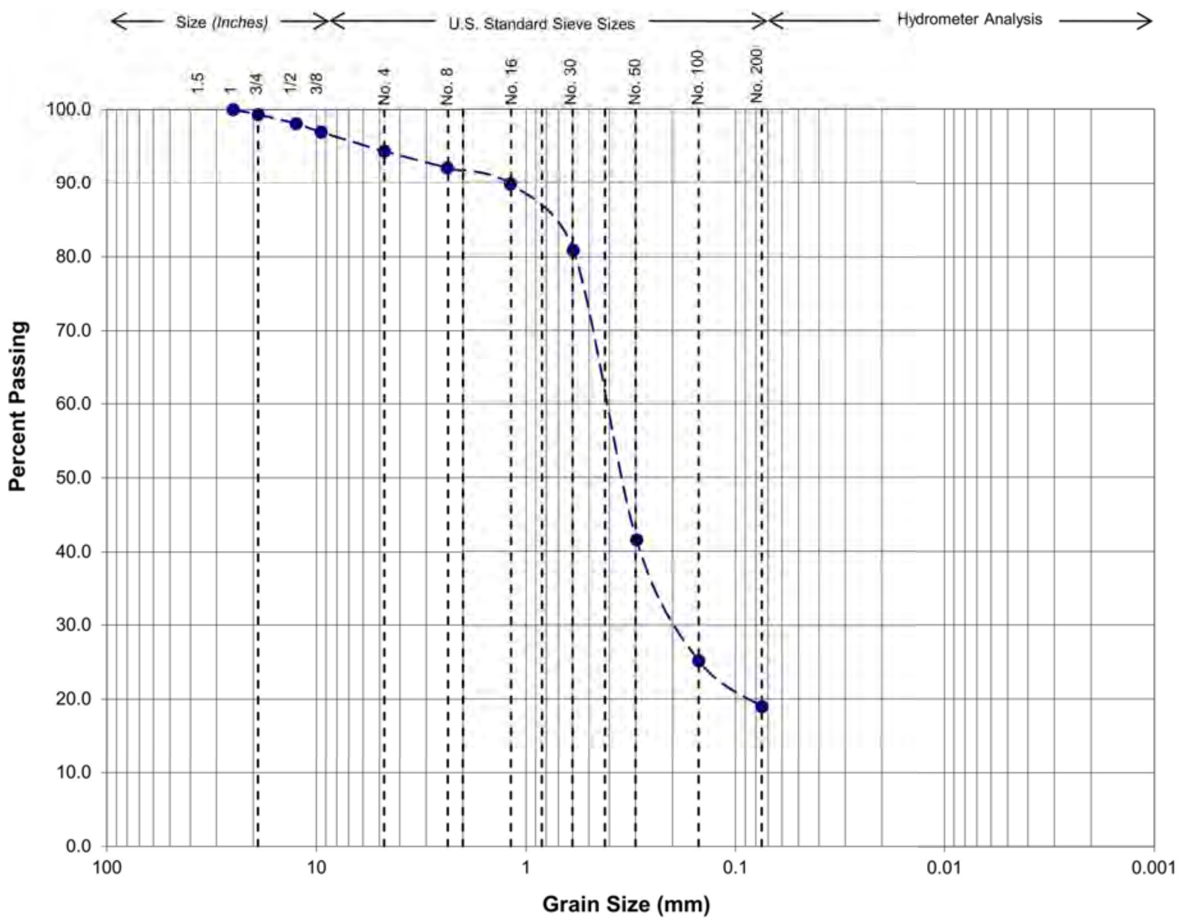

Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-5

Depth (ft): $15\frac{1}{2}$ - $17\frac{1}{2}$

USCS Soil Type: SM

Passing No. 200 (%): 19

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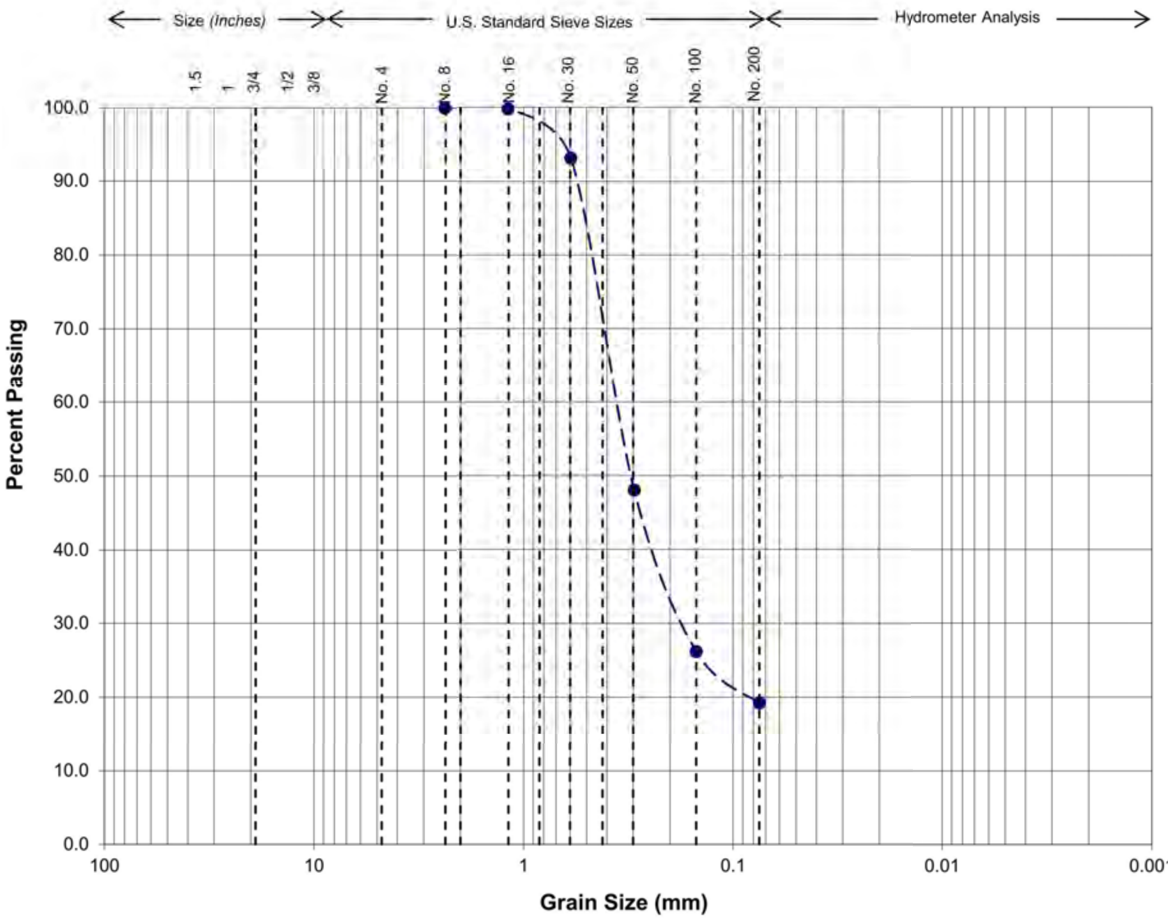
BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.19



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: B-6

Depth (ft): 7 ½ - 9

USCS Soil Type: SM

Passing No. 200 (%): 19

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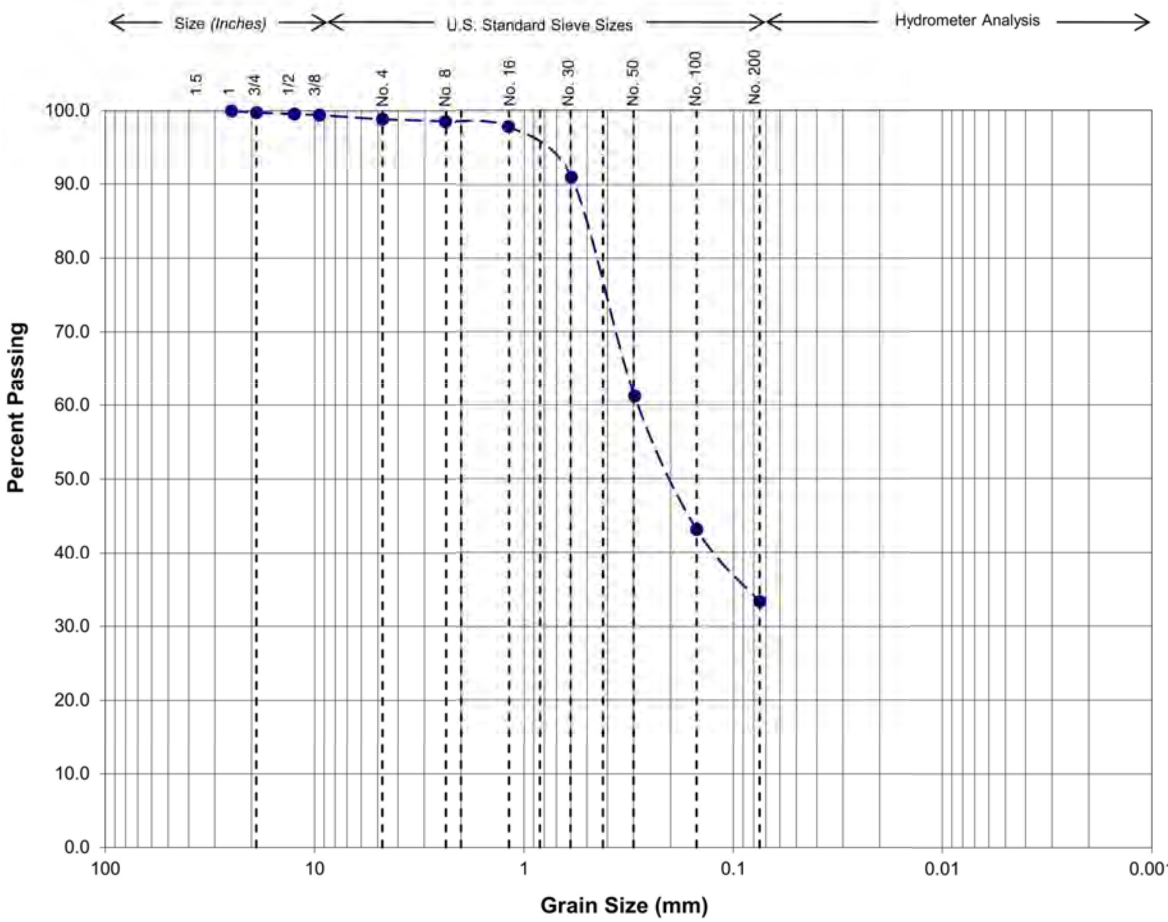
CLASSIFICATION TEST RESULTS

PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT

OCEANSIDE TRANSIT CENTER

OCEANSIDE, CA 92054

BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.20



Gravel		Sand			Silt or Clay
Coarse	Fine	Coarse	Medium	Fine	

Sample Location: P-2

Depth (ft): 7 - 10

USCS Soil Type: SM

Passing No. 200 (%): 33

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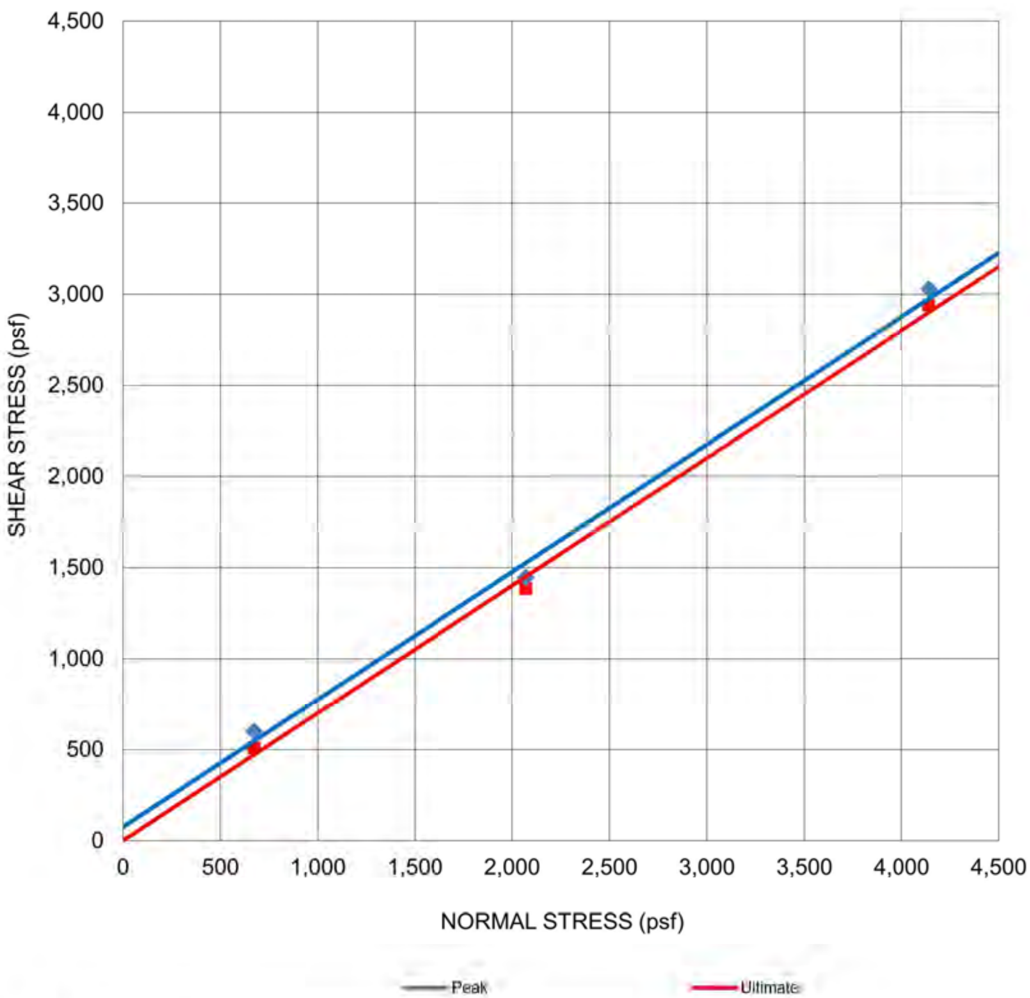
CLASSIFICATION TEST RESULTS

PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT

OCEANSIDE TRANSIT CENTER

OCEANSIDE, CA 92054

BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.21



	Peak	Ultimate
Friction Angle (Φ):	35 °	35 °
Apparent Cohesion (C):	75 psf	0 psf

Soil Unit: Quaternary Old Paralic Deposits (Qop)

Sample Location: B-3

Depth (ft): 7 ½ - 9

Notes: In-Situ

USCS Soil Type: SM

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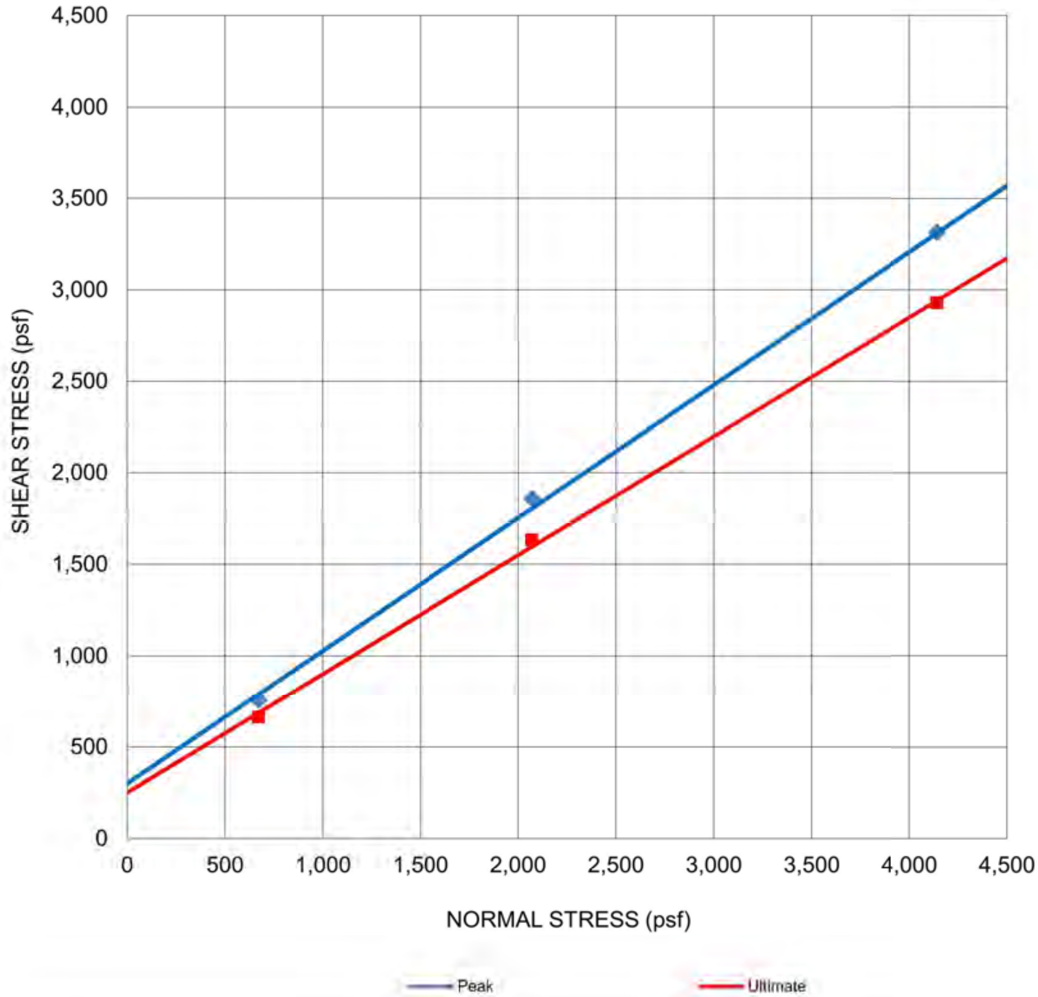
DIRECT SHEAR TEST RESULTS

PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT

OCEANSIDE TRANSIT CENTER

OCEANSIDE, CA 92054

BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.22



Friction Angle (Φ):	36 °	33 °
Apparent Cohesion (C):	300 psf	250 psf

Soil Unit: Quaternary Old Paralic Deposits (Qop)

Sample Location: B-4

Depth (ft): 10 - 11 $\frac{1}{2}$

Notes: In-Situ

USCS Soil Type: SM

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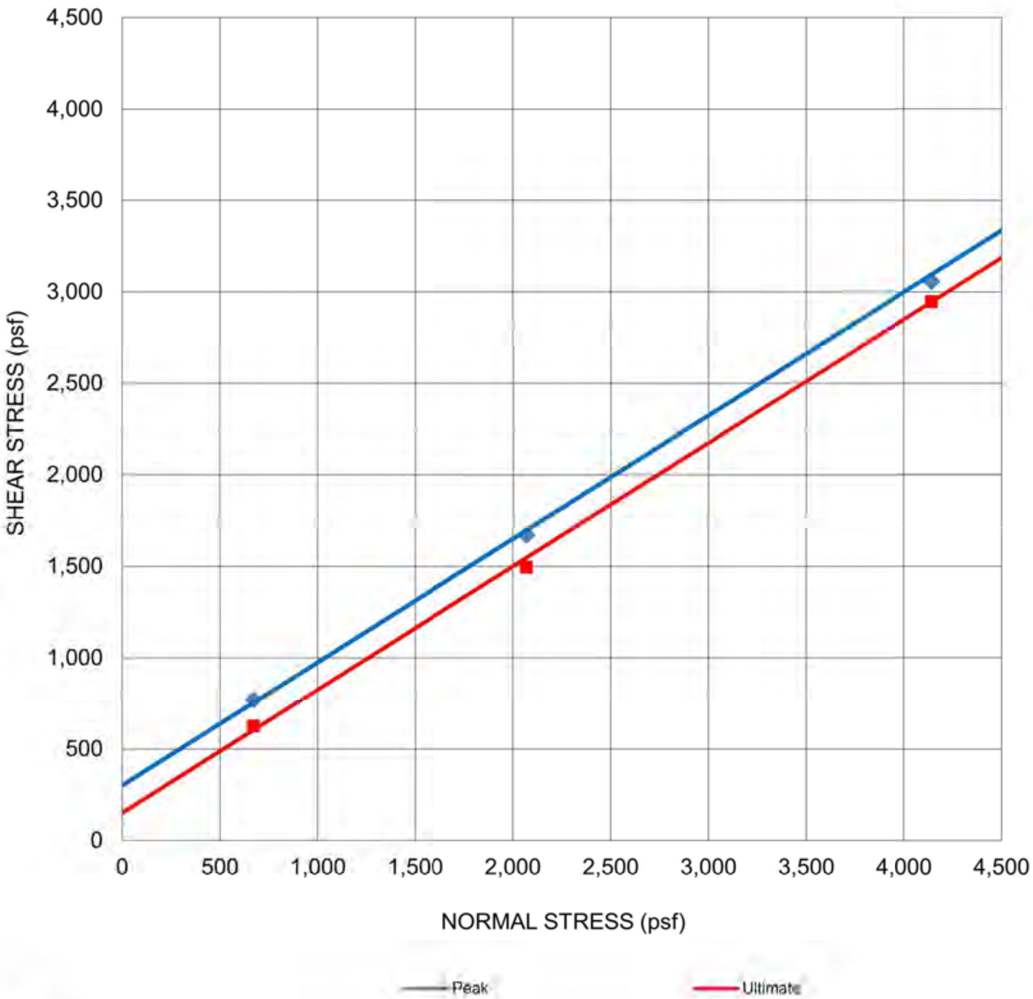
DIRECT SHEAR TEST RESULTS

PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT

OCEANSIDE TRANSIT CENTER

OCEANSIDE, CA 92054

BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.23



	Peak	Ultimate
Friction Angle (Φ):	34 °	34 °
Apparent Cohesion (C):	300 psf	150 psf

Soil Unit: Quaternary Old Paralic Deposits (Qop)

Sample Location: B-5

Depth (ft): 10 - 11 ½

Notes: In-Situ

USCS Soil Type: SM

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DIRECT SHEAR TEST RESULTS

PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT

OCEANSIDE TRANSIT CENTER

OCEANSIDE, CA 92054

BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.24

Density of Soil in Place (ASTM D2937)

Sample Location	Sample Depth (ft)	Soil Description	Moisture (%)	Dry Density (pcf)
B-1	2 $\frac{1}{2}$ - 4	Dark Brown Silty Sand with Some Clay	8.1	129.2
B-1	8 - 8 $\frac{1}{2}$	Olive Brown Silty Sand	5.4	114.5
B-6	5 - 6 $\frac{1}{2}$	Reddish Brown Silty Sand with Clay	5.1	130.0

Maximum Dry Density and Optimum Moisture Content (ASTM D1557)

Sample Location	Sample Depth (ft.)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (%)
B-5	12 - 15	Yellow/Orange Brown Silty Sand	119.0	8.0
B-5	15 - 17 $\frac{1}{2}$	Pale Olive Brown Silty Sandstone	129.4	8.0
B-6	$\frac{1}{2}$ - 5	Dark Reddish Brown Silty Sand	132.0	8.5
B-6	14 - 17 $\frac{1}{2}$	Gray Brown with Orange and Dark Gray Clayey Sandstone	146.0	6.7

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

Sample Location	Sample Depth (ft.)	Soil Description	R-Value
B-3	1 - 5	Dark Reddish Brown Silty Sand	24

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

Sample Location	Sample Depth (ft.)	pH	Resistivity (Ohm-cm)	Sulfate Content (ppm)	Sulfate Content (%)	Chloride Content (ppm)	Chloride Content (%)
B - 3	16 $\frac{1}{2}$ - 18	8.7	6700	<30	0.003	53	0.005
B - 5	15 $\frac{1}{2}$ - 17 $\frac{1}{2}$	8.7	5700	<30	0.003	64	0.006
B - 6	$\frac{1}{2}$ - 5	9.1	2400	90	0.009	53	0.005
B - 6	14 - 17 $\frac{1}{2}$	9.1	3500	<30	0.003	32	0.003

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LAB TEST RESULTS

PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT
OCEANSIDE TRANSIT CENTER
OCEANSIDE, CA 92054

BY: GN | DATE: AUGUST 2021 | PROJECT: 2021010 | FIGURE: C.25

Expansion Index (ASTM D4829)

Sample Location	Sample Depth (ft.)	Expansion Index	Expansion Potential
B - 3	16 $\frac{1}{2}$ - 18	1	Very Low
B - 5	15 $\frac{1}{2}$ - 17 $\frac{1}{2}$	1	Very Low
B - 6	$\frac{1}{2}$ - 5	1	Very Low
B - 6	14 - 17 $\frac{1}{2}$	3	Very Low

EXPANSION INDEX	EXPANSION POTENTIAL
0 - 20	VERY LOW
21 - 50	LOW
51 - 90	MEDIUM
91 - 130	HIGH
131 AND ABOVE	VERY HIGH

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LAB TEST RESULTS

PROPOSED NCTD OCEANSIDE TRANSIT CENTER REDEVELOPMENT
OCEANSIDE TRANSIT CENTER
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BY: GN	DATE: AUGUST 2021	PROJECT: 2021010	FIGURE: C.26

APPENDIX D

WORKSHEET C.4-1: CATEGORIZATION OF INFILTRATION FEASIBILITY CONDITION

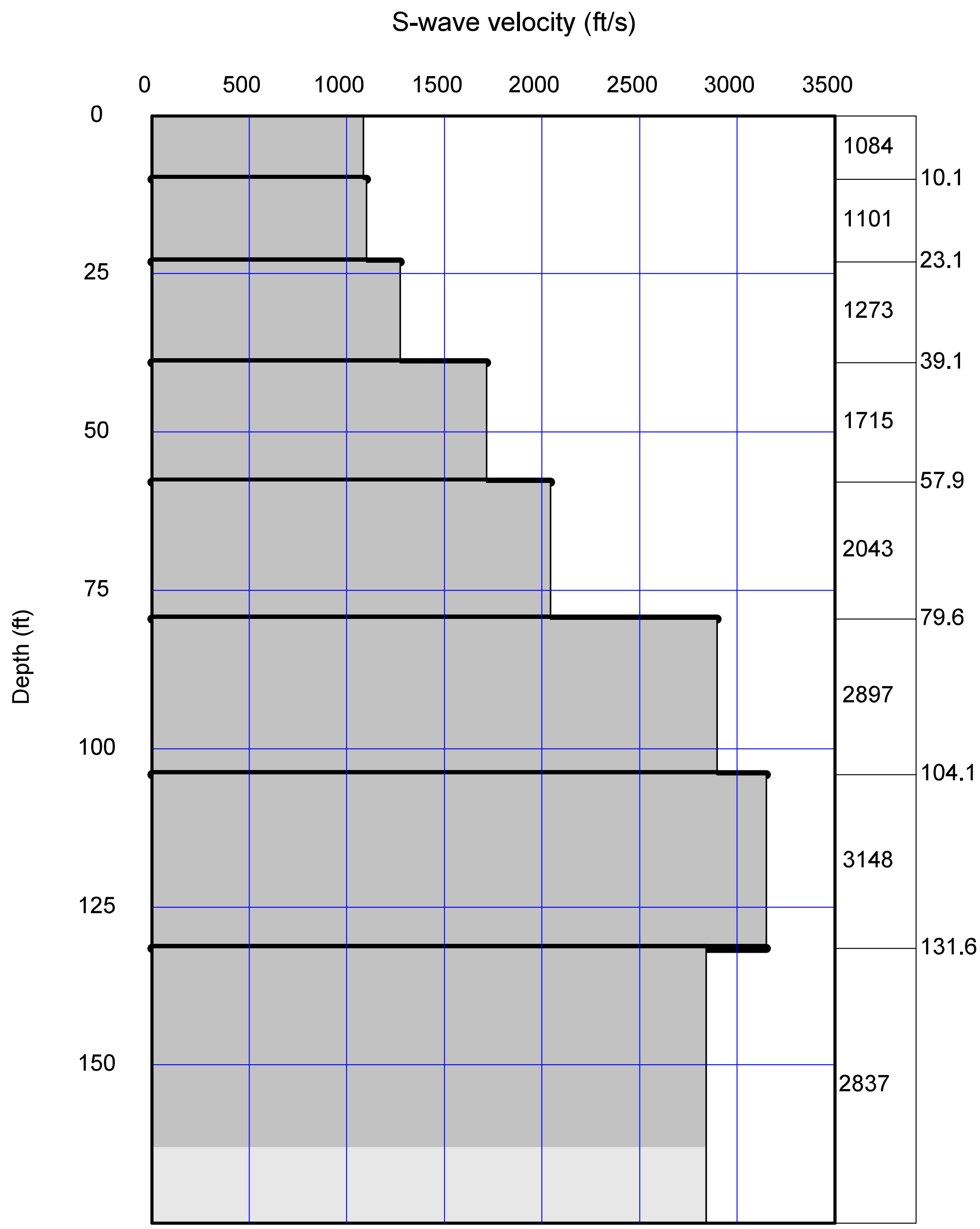
Worksheet C.4-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		**Worksheet C.4-1**	
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 at locations P-1 and P-2, based on the on-site infiltration study was calculated to be greater than 0.5 inches per hour (1.16 and 1.10 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	
Provide basis: *C2.1 A geologic investigation was performed at the subject site. See NOVA 2021.* *C2.2 Settlement and soil volume change due to stormwater infiltration is not a concern with: (i) low expansive soils, (ii) no potential for liquefaction, and (iii) no potential for hydro collapse.* *C2.3 Infiltration has the potential to cause slope failures. BMPs are to be sited a minimum of 50 feet away from any slope.* *C2.4 BMPs are to be sited a minimum of 10 feet away from all underground utilities.* *C2.5 Stormwater infiltration can result in damaging ground water mounding during wet periods. C2.6 Infiltration has the potential to increase lateral pressure and reduce soil strength which can impact foundations and retaining walls. BMPs are to be sited a minimum of 10 feet away from any foundations or retaining walls.* *C2.7 Other Factors: NOVA does not know of other factors that could affect implementation of stormwater infiltration BMPs. However, the complete design is not known at this point. Risk factors could be further identified (for example, the proximity of BMPs to utilities, retaining walls, etc.) in review of the final design.*			

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

APPENDIX E
RECORDS OF GEOPHYSICAL TESTING



SHEAR-WAVE VELOCITY MODEL: Average Vs 100ft = 1,605.7 ft/sec

Site Classification (ASCE 7-16 Ch. 20)- "C" (Very Dense Soil and Soft Rock)

Client: NOVA Services, Inc. **Project No. 2021010**

Project Name: Toll Brothers Project, Oceanside, California

Survey Line End Coordinates: 33.19110 -117.37819 / 33.19136, -117.37777

Date: 4/8/21

PROFESSIONAL GEOPHYSICIST
DONN C. SCHWARTZKOPF
No. 1002
STATE OF CALIFORNIA

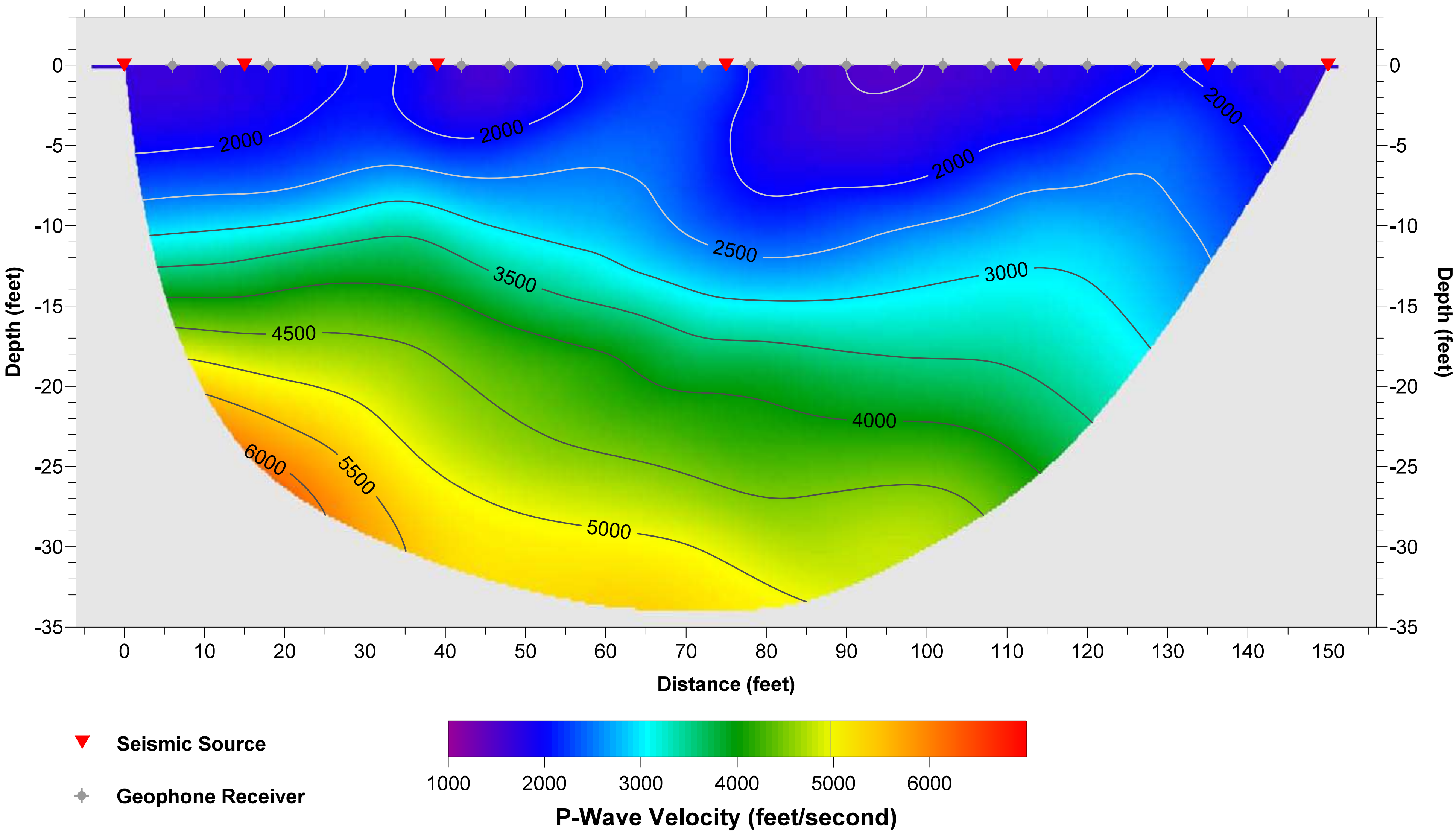
TERRA GEOSCIENCES
P.O. Box 1090
Loma Linda, CA 92354

TG Project No. 213618-1

SEISMIC LINE S-1

South 36° East→

REFRACTION TOMOGRAPHIC MODEL



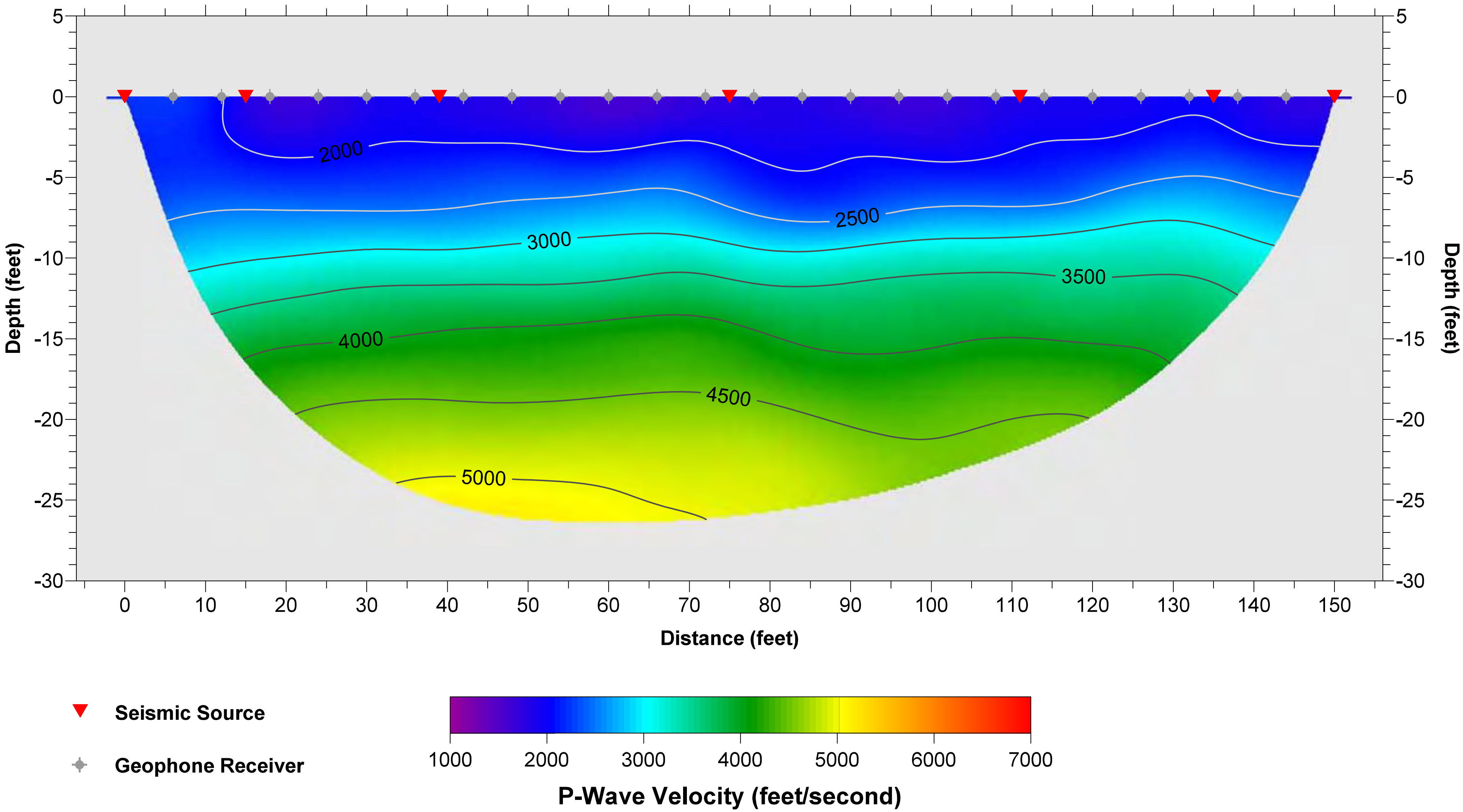
SCALE: Vertical Exaggeration 2X

RMS error 3.6%, Rayfract Version 4.02

SEISMIC LINE S-2

North 36° West →

REFRACTION TOMOGRAPHIC MODEL



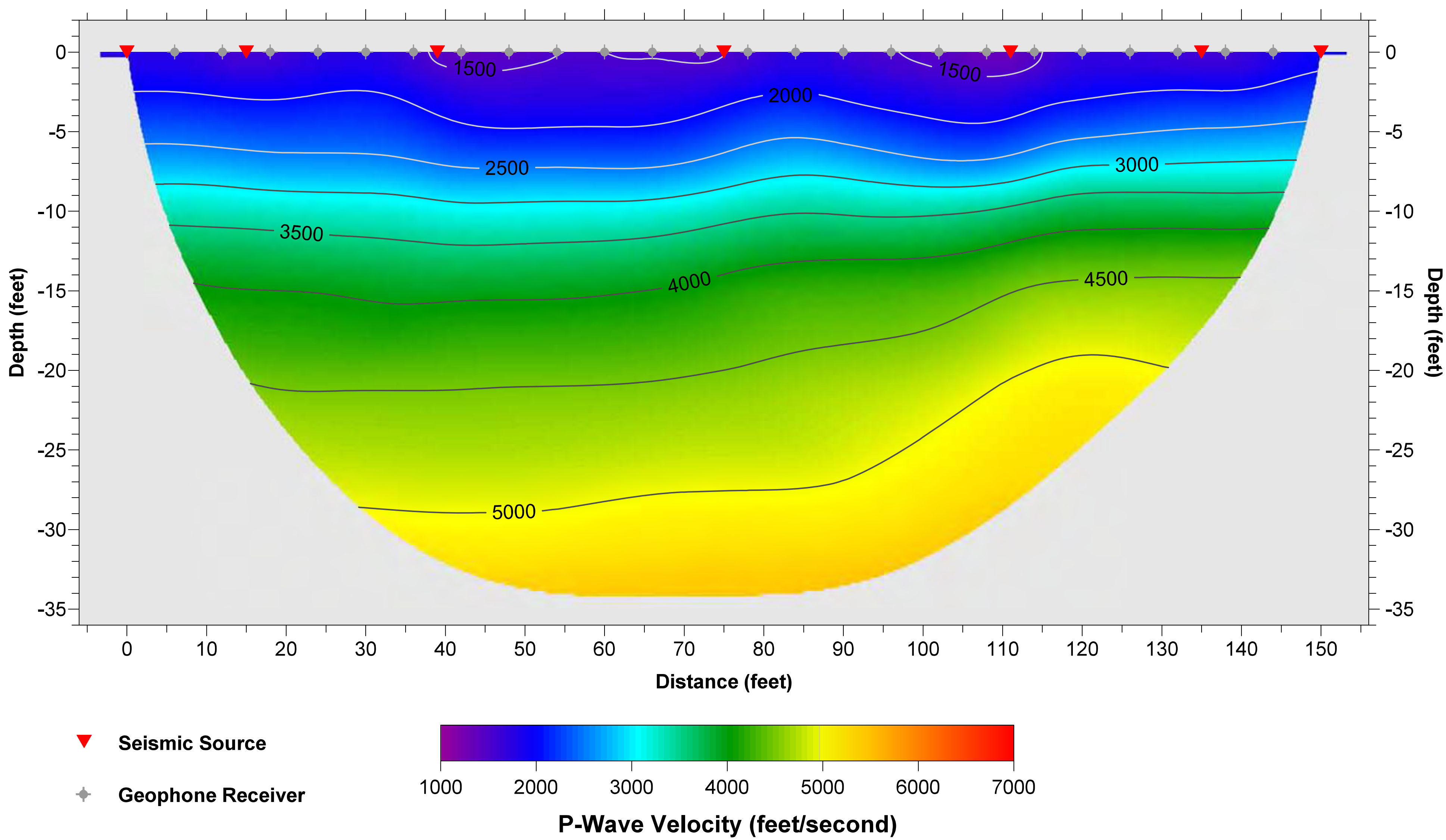
SCALE: Vertical Exaggeration 2X

RMS error 2.5%, Rayfract Version 4.02

SEISMIC LINE S-3

South 36° East →

REFRACTION TOMOGRAPHIC MODEL



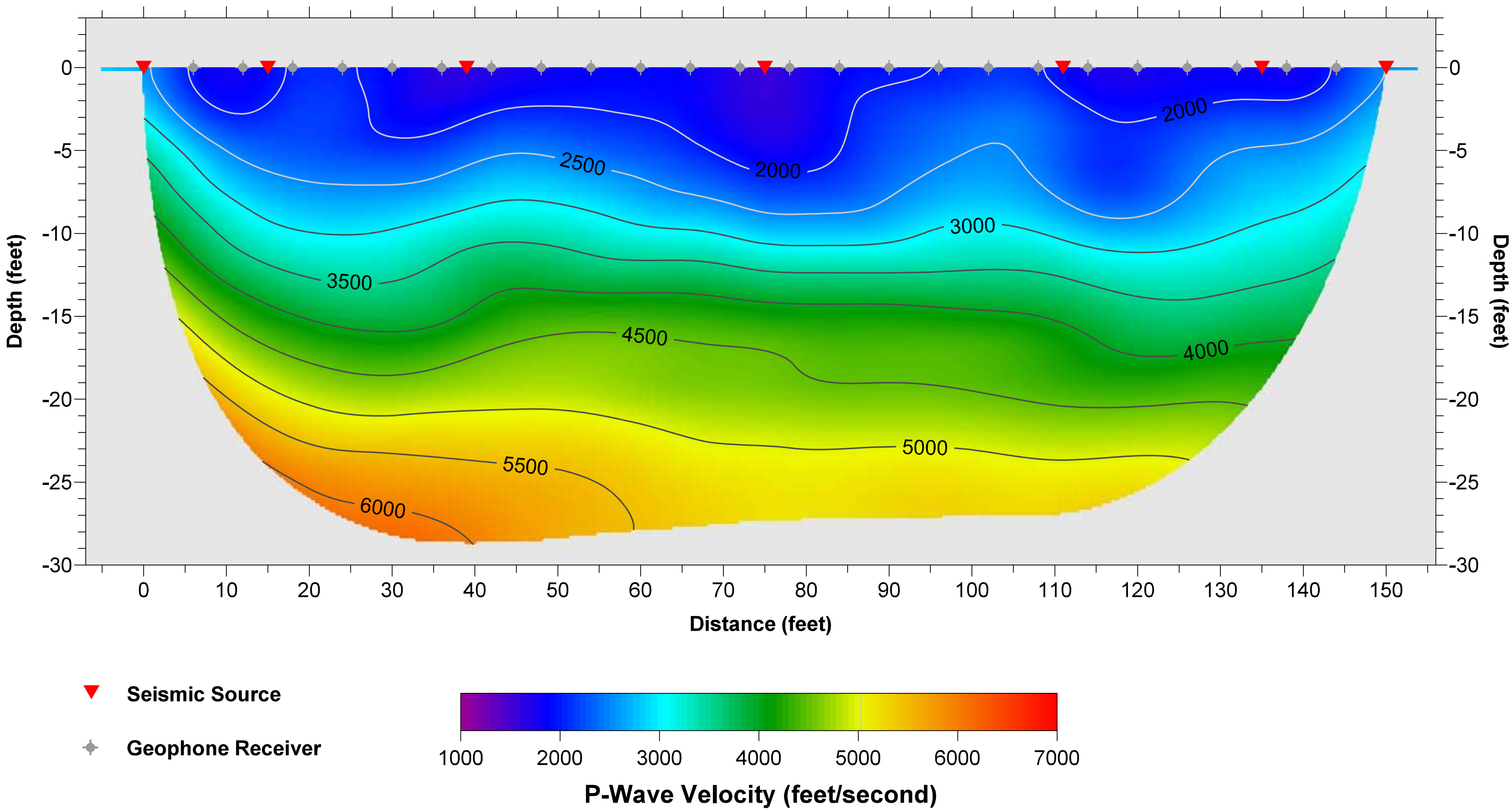
SCALE: Vertical Exaggeration 2X

RMS error 2.1%, Rayfract Version 4.02

SEISMIC LINE S-4

South 54° West →

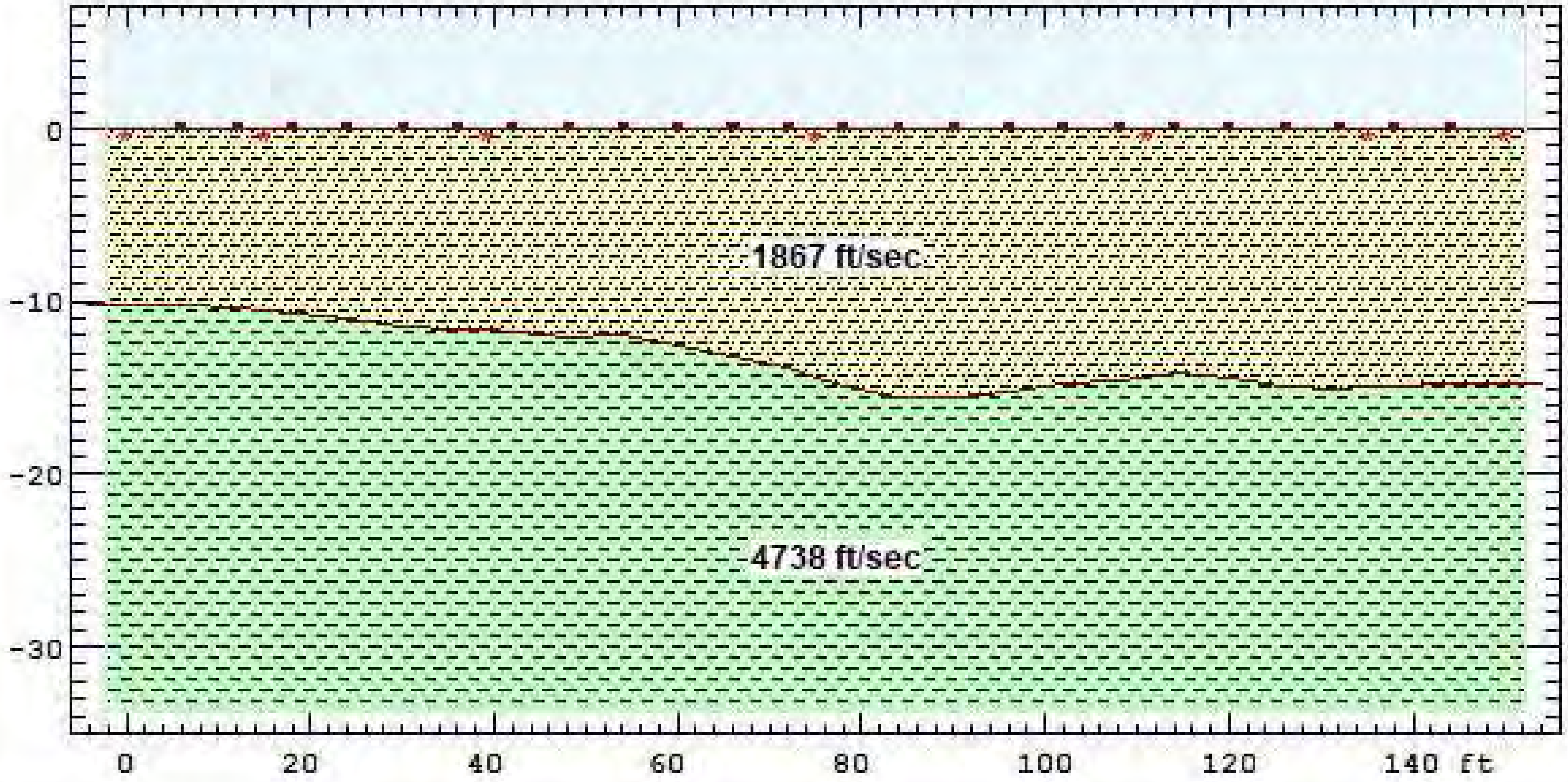
REFRACTION TOMOGRAPHIC MODEL

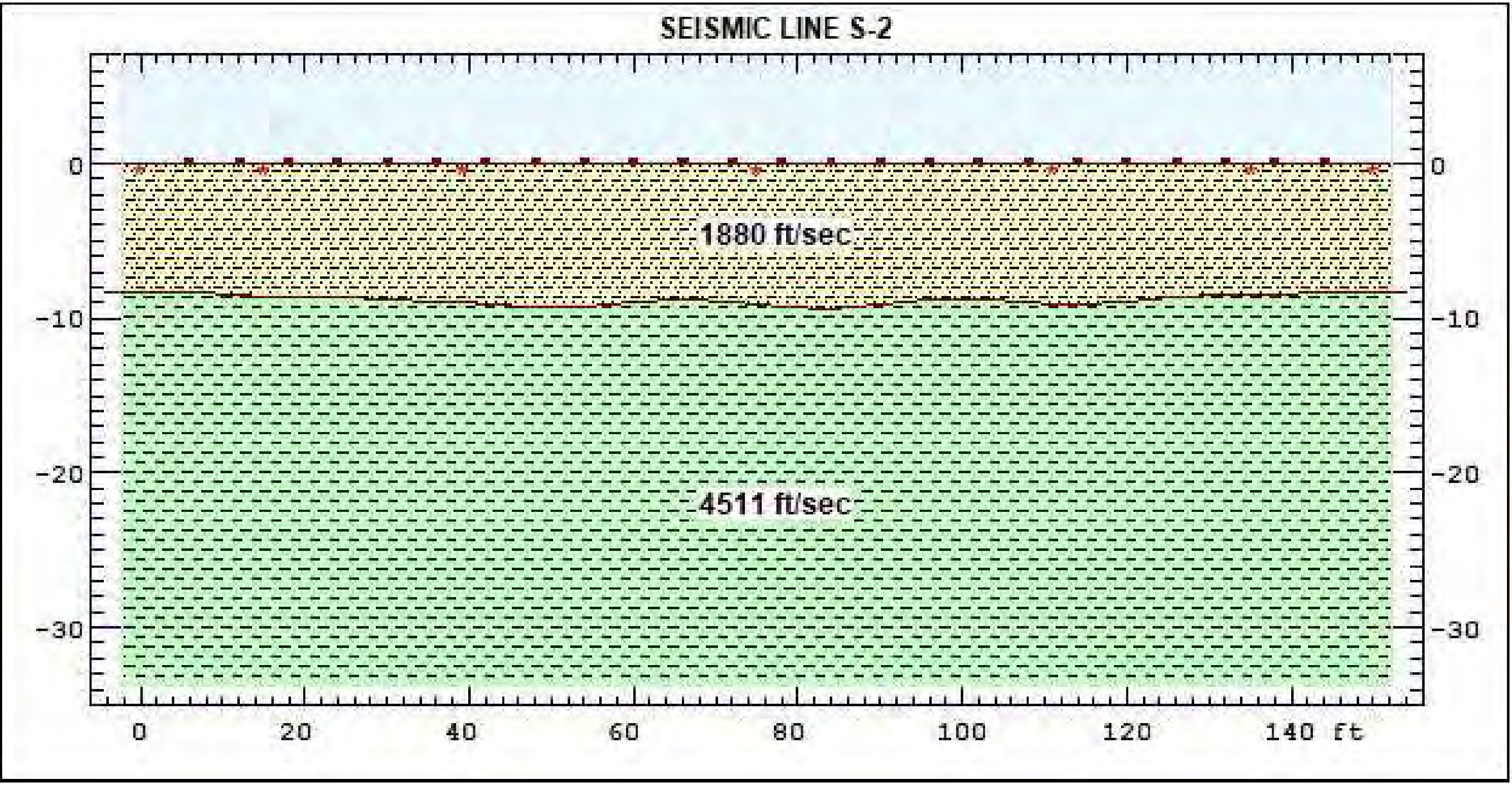


SCALE: Vertical Exaggeration 2X

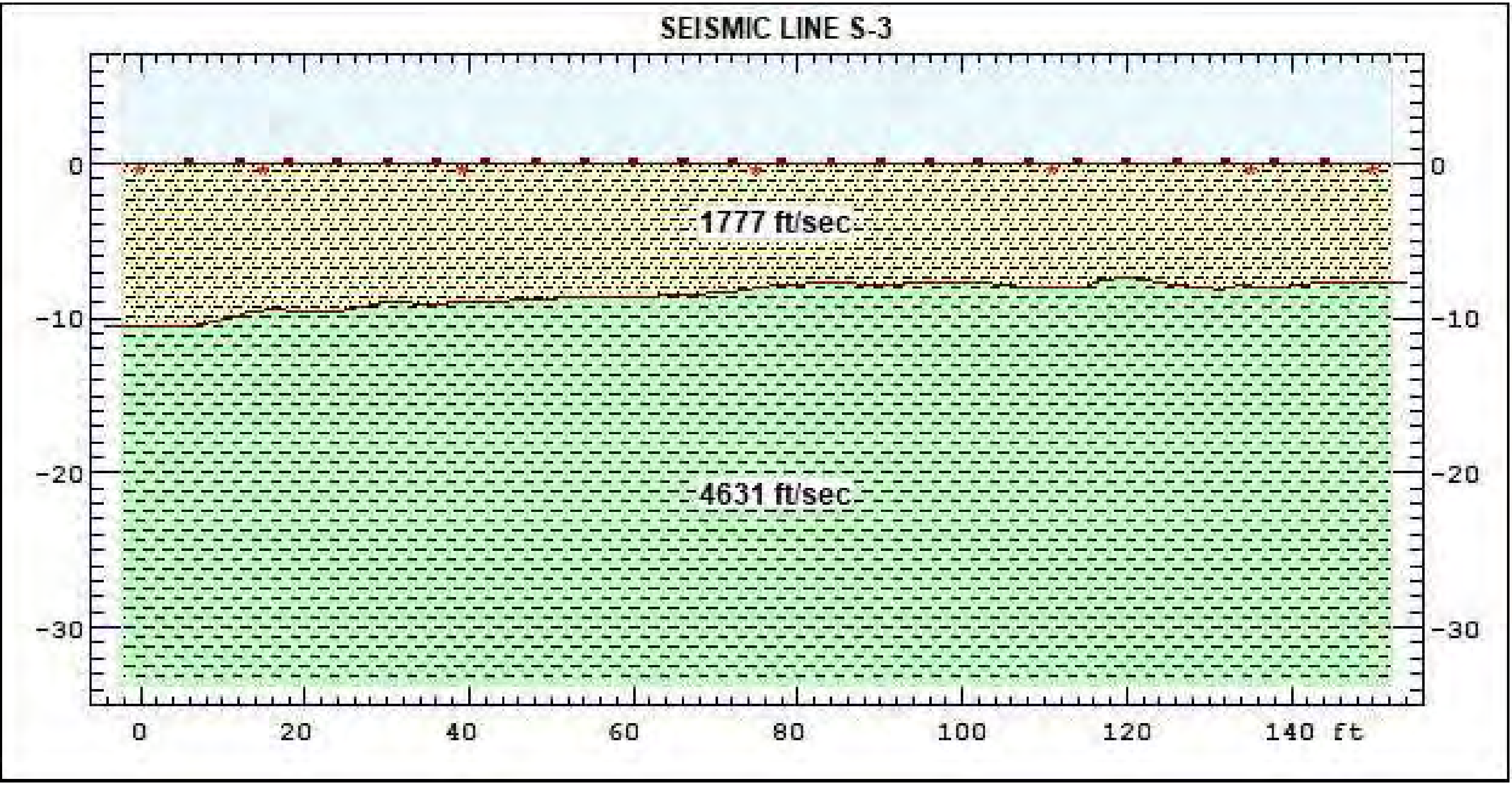
RMS error 4.6%, Rayfract Version 4.02

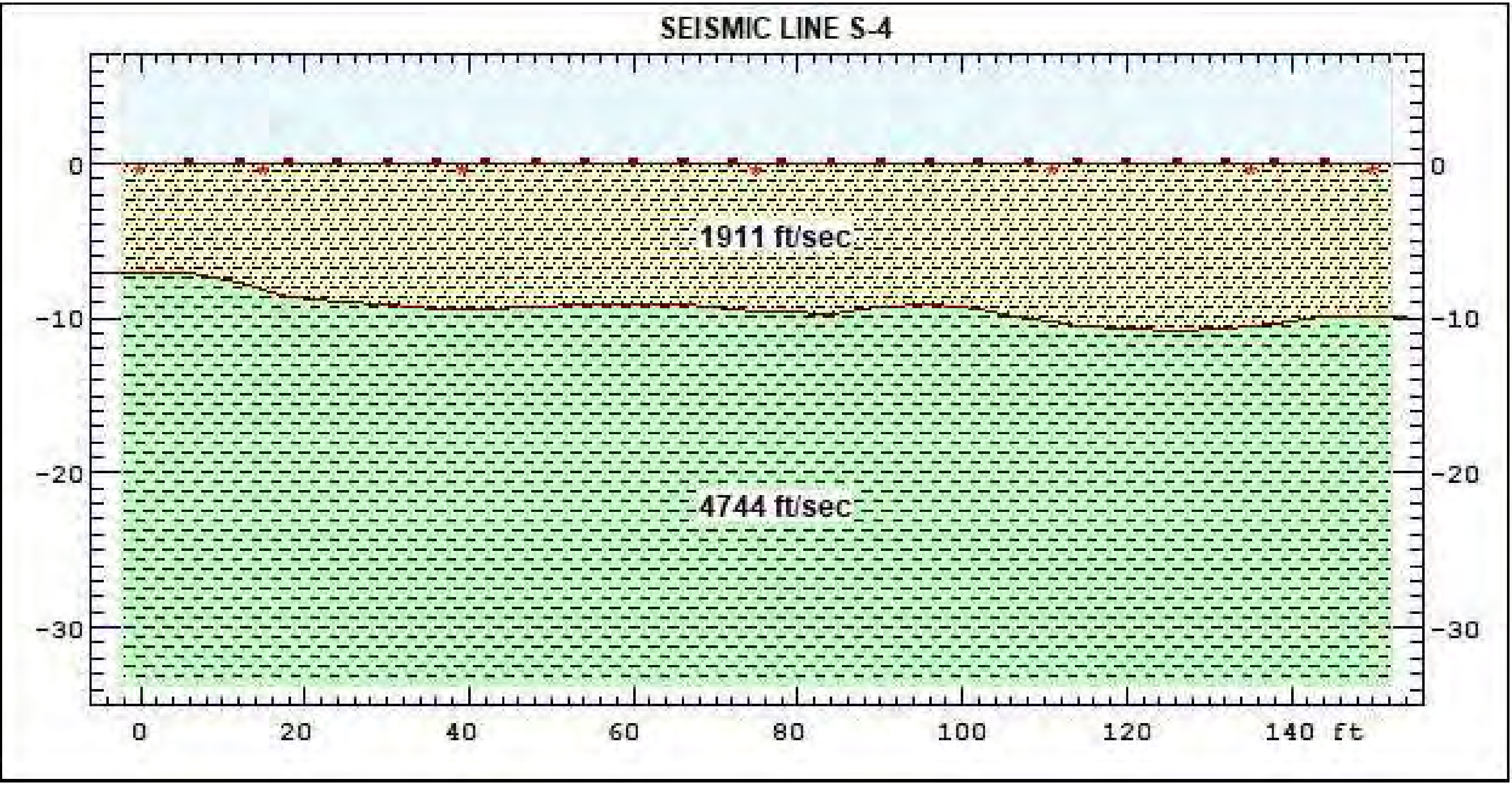
SEISMIC LINE S-1





SEISMIC LINE S-2
1880 ft/sec
4511 ft/sec
0
-10
-20
-30
0
20
40
60
80
100
120
140 ft


SEISMIC LINE S-3
1777 ft/sec
4631 ft/sec
0
-10
-20
-30
0
20
40
60
80
100
120
140 ft


SEISMIC LINE S-4
1911 ft/sec
4744 ft/sec
0
-10
-20
-30
0
20
40
60
80
100
120
140 ft