Geotechnical Design Report

Verdi Montgomery Avenue Vicinity Pedestrian Rail Undercrossing Project

January 2020

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January 22, 2020

City of Encinitas
505 S. Vulcan Avenue
Encinitas, CA 92024

Attn: Ms. Christy Villa
Project Manager, City of Encinitas

Subject: Verdi-Montgomery Avenue Vicinity Pedestrian Rail Undercrossing Project (Verdi Avenue Undercrossing Project), Geotechnical Design Report

HDR has completed the subsurface exploration and geotechnical engineering services for the above referenced project. This geotechnical design report presents the results of the subsurface exploration and provides geotechnical conclusions and recommendations regarding earthwork and the design and construction of the Verdi Avenue Undercrossing Project.

In summary, the project is considered feasible from a geotechnical perspective provided that the recommendations presented in this report are incorporated into design and construction.

If you have any questions regarding this report, please do not hesitate to contact the undersigned. We appreciate this opportunity to be of service. Respectfully submitted,

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1 Introduction

1.1 Project Description

The proposed Verdi Avenue Undercrossing Project (Project) is located within the City of Encinitas on a portion of the Los Angeles to San Diego and San Luis Obispo (LOSSAN) Rail Corridor operated by the North County Transit District (NCTD). The LOSSAN Corridor between Los Angeles Union Station and the Santa Fe Depot in downtown San Diego is referred to as "LOSSAN South." LOSSAN South is the second busiest passenger rail line in the United States after the Northeast Corridor between Washington D.C. and Boston. This important 127.5-mile rail line serves a vital function in providing intercity, commuter and freight rail services in California, and is a major transportation resource in Southern California.

The Project includes the addition of a pedestrian undercrossing structure and pathway to connect San Elijo Avenue near Verdi Avenue to the San Elijo State Beach, in the City of Encinitas. The Project requires a crossing of the existing NCTD double-tracked railway, as well as associated retaining walls, crosswalks, and other minor features. We understand that the preferred design includes the construction of a railroad bridge, rather than a culvert or other crossing feature. It is anticipated that the proposed bridge will be approximately 62 feet in length and carry two railroad tracks. The bridge will consist of one abutment at each end and two intermediate bents. The abutments and bents are to be supported on deep foundations consisting of cast-in-drilled-hole (CIDH) piles. We understand that the proposed pedestrian pathway will lead beneath the railroad bridge to an at-grade crosswalk across existing South Coast Highway (Highway 101). The approximate Project location is shown on Figure 1, Vicinity Map, in Appendix A.

1.2 Purpose and Scope

The purpose of this investigation was to review existing geotechnical data and evaluate data from our subsurface exploration and laboratory testing, present results of geotechnical analyses, and provide geotechnical design recommendations for the proposed Project.

The scope of work for the geotechnical design of this Project included the following tasks:

- **Literature Review:** Review of various documents pertinent to the Project alignment and proposed bridge. A list of references used in preparation of this report is presented in Section 6. Relevant existing geotechnical data are included in Appendix D. Locations of previous exploratory borings advanced by Ninyo and Moore (2016) are shown on Figure 2, Existing Data Map in Appendix A.
- **Pre-Field Exploration Activities:** Prior to the commencement of the field investigation, a work plan was prepared and submitted for approval to NCTD and a boring permit was obtained from the County of San Diego Department of Environmental Health (CSDEH). This work plan included the field work scope, equipment, boring backfill details, schedule, site access, work impacts, hazards,

spills, safety and emergency protocol. In addition, a site reconnaissance was performed to visually evaluate the accessibility of the site for drilling equipment and to locate and mark the proposed boring locations. Utility clearance was performed by Bombardier Signal Department and an independent third-party geophysical subconsultant (Southwest Geophysics, Inc.) prior to drilling.

- **Field Exploration and Laboratory Testing:** The subsurface exploration program included drilling, logging, and sampling borings as described in Section 2.1. Laboratory testing was performed on selected soil samples collected from the field exploration to evaluate the engineering properties of the subsurface soils. The approximate location of borings is presented on Figure 3, Investigation Location Map in Appendix A. Boring logs and laboratory test results are included in Appendices B and C, respectively.
- **Seismic Analysis:** Regional seismicity and encountered subsurface conditions were used to perform a ground motion analysis of the Project alignment for use in structural analysis and design. Seismic hazards were identified and are presented in Section 3.11.
- **Geotechnical Design and Analysis:** Geotechnical analysis was performed using the collected data to develop recommendations for design and construction of the proposed Project. Recommendations for earthwork, existing embankment slope remediation, bridge foundations, and lateral earth pressure for retaining walls, allowable bearing capacity, soldier pile walls and tiebacks, infiltration, trench backfill, and cement type and corrosion measures are presented in Sections 4 and 5.
- **Report Preparation:** Relevant geotechnical data were compiled in this report along with our findings, conclusions, and recommendations for the proposed Project.

2 Geotechnical Field and Laboratory Investigations

2.1 Subsurface Exploration

HDR's field exploration consisted of advancing three 4-inch-diameter, mud rotary borings to a maximum depth of about 101 feet below ground surface (bgs), and one 8-inch-diameter, hollow-stem boring to a maximum depth of about 51 feet bgs. The mud rotary borings were initially advanced using an 8-inch-diameter hollow-stem auger, but converted to mud rotary at depths between 5 to 20 feet bgs. Borings were designated as A-18-001 through A-18-004. Boring A-18-002 was converted to an infiltration test at a depth of 10 feet bgs to assess infiltration capabilities at the site. Drilling of Boring A-18-002 was resumed after the completion of the infiltration testing at an adjacent borehole. The approximate location of the existing subsurface investigation locations and current boring explorations are shown on Figure 2 and 3, respectively in Appendix A.

The boring locations were marked in the field by measuring from known locations of existing features using a measuring wheel and/or tape measure or were located using GPS coordinates.

Standard Penetration Tests (SPT) were performed within the borings using a 140-pound automatic hammer falling freely for 30 inches. The samplers were driven for a total penetration of 18 inches and the blow counts per 6 inches of penetration were recorded in the boring logs. Drive samples were collected from the borings using a Modified California Ring sampler. The field sampling procedures were conducted in accordance with ASTM Standard Specifications D 1586 and D 3550 for SPT and split-barrel sampling of soil, respectively. In addition to driven samples, bulk soil samples were also collected from Borings A-18-002, A-18-003, and A-18-004.

The test borings were logged in the field by a member of HDR technical staff. Each soil sample collected was reviewed and described in accordance with the Unified Soil Classification System (ASTM D2487). All samples were sealed and packaged for transportation to a subconsultant's laboratory. At the location where infiltration testing was performed, the boring was converted to an infiltration test as described in Section 3.8. After completion of drilling, the borings were backfilled with bentonite grout in accordance with the requirements of the CSDEH Monitoring Well Program Geotechnical Boring Construction Permit obtained from the County. Soil cuttings were drummed for offsite disposal. Geotechnical logs of the borings are included in Appendix B.

2.2 Geotechnical Laboratory Testing

Laboratory tests were performed on selected soil samples to evaluate the geotechnical engineering properties of subsurface materials. The following laboratory tests were performed:

- In-situ moisture content and density
- Atterberg limits

- Grain-size distribution and hydrometer
- Laboratory Compaction (maximum dry density and optimum moisture content)
- Direct Shear
- Triaxial Compression
- Corrosivity (soluble sulfate contents, chloride, pH, and resistivity).
- All laboratory tests were performed in general accordance with ASTM procedures, except corrosivity tests, which were performed in accordance with Caltrans procedures. Results of the laboratory tests are summarized in Table C-1 and presented in Appendix C.

3 Geotechnical Findings

3.1 Geologic Setting

The Project area is located in the coastal section of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 1998). The province varies in width from approximately 30 to 100 miles. In general, the province consists of rugged mountains under-lain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. In the portion of the province in San Diego County that includes the Project area, basement rocks are generally overlain by Quaternary and Tertiary age sedimentary rock and alluvial soils. A geologic map is presented on Figure 4 in Appendix A.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are considered active faults. The Elsinore, San Jacinto, and San Andreas faults are active fault systems located northeast of the Project area and the Newport-Inglewood Rose Canyon, Coronado Bank, San Diego Trough, and San Clemente faults are active faults located west of the Project area. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. Further discussion of faulting relative to the site is provided in Section 3.9.

3.2 Site Geology

Geologic units encountered during the field investigation or mapped in the project vicinity included Artificial Fill (Qaf), Beach Deposits (Qb), Fine-grained Tertiary Deposits (Tsh), Old Lacustrine (Qol), and Very Old Lacustrine Deposits (Qvol). Generalized descriptions of these units are provided below. More detailed descriptions are provided on the geotechnical boring logs in Appendix B.

3.2.1 Artificial Fill (Qaf)

Fill soils were generally observed along the existing railroad embankment. The existing embankment heights ranged from roughly 8 to 10 feet above adjacent South Coast Highway to the southwest. However, the embankments are not constructed entirely of fill, with fill soils generally limited to about two feet in thickness where observed. Fill may be thicker directly beneath existing tracks where embankment thickness is greatest.

Fill soils generally consisted of light brown, moist, medium dense, silty sand, gravels, and cobbles, with scattered clays intermixed. Fill materials generally match the constituency of the adjacent Old Lacustrine deposits and were likely borrowed from local sources.

3.2.2 Beach Deposits (Qb)

Beach Deposits generally consist of unconsolidated late Holocene marine sediments consisting of fine- and medium-grained poorly-graded sand. These are the sands typically associated with Southern California beaches, and are generally limited to within about 100 to 200 feet of the shoreline within the project vicinity. Beach deposits were not observed in the exploratory borings performed at the project site.

3.2.3 Fine-Grained Tertiary Deposits (Tsh) – Del Mar Formation

Materials of the Tertiary Age, consisting of fine-grained sandstone, siltstone, mudstone, shale, and siliceous and calcareous sediments. In the project vicinity, these materials are generally observed as a part of the Del Mar Formation consisting of weakly to moderately cemented siltstones and sandstones with occasional scattered gravels and varying clay content. This formation was observed in all borings, beginning at depths ranging from about 10 to 15 feet bgs (corresponding elevations 52 to 56 feet based on the North American Vertical Datum of 1988 [NAVD88]) and extending to the maximum depth explored (about 101 feet bgs, corresponding elevation -30 feet NAVD88).

3.2.4 Old Lacustrine Deposits (Qol)

Old Lacustrine, Playa, and Estuarine (Paralic) deposits which generally consist of medium dense to dense moderately dissected fine-grained sand, silt, and clay from lake, playa, and estuarine deposits of various types. The materials observed during our field investigation and previous investigations near the project were generally in a medium-dense to dense or hard condition, and ranged widely from clayey soils to silty sands, with cobbles and gravels occasionally observed. Where the old lacustrine deposits were observed, they generally overlie the Tertiary deposits of the Del Mar formation, described above.

3.2.5 Very Old Lacustrine Deposits (Qvol)

These deposits are generally similar to the Old Lacustrine deposits described above. Their key difference is that they are generally older, and therefore underlie the Old Lacustrine deposits, and are generally in a dense to very dense condition.

3.3 Existing Surface Conditions

Surficial erosion has occurred along various portions of the alignment in the form of erosion rills (gullies). Deep erosion rills on the order of 2 to 5 feet deep and approximately 1 to 3 feet wide were observed during the site visits conducted between January and March 2018 at numerous locations near existing slopes. A drainage swale running parallel to the tracks on the eastern side shows significant erosion. The erosion rills in general are a result of previous heavy rainfall.

Vegetation onsite varies from overgrown native and non-native shrubbery, trees, and grasses, with most area covered by ice plant-type groundcover, to non-vegetated pathways and slope faces. Soils observed at the ground surface vary in accordance with the geologic units described in Section 3.2. These vary in constituency generally from silty to clayey sands and fill associated with existing construction.

3.4 Subsurface Earth Materials

The subsurface soils encountered in the borings and observed during our field investigation at the project site are predominantly localized artificial fills, and alluvium consisting of estuarine and colluvial deposits. Section 3.2 of this report and the boring logs presented in Appendix B describe in more detail the subsurface units encountered during exploration. Groundwater data can be found in Section 3.5.

Artificial Fill (Qaf) was noted where grades were constructed for the existing railroad construction. These materials were not sampled during our investigation due to their proximity to the tracks. Beneath the fill materials (where observed) is about a 10- to 15-foot-thick layer of old or very old Lacustrine Deposits (Qol or Qvol). Beneath the lacustrine deposits, Del Mar Formation bedrock was encountered to the maximum depth explored as described above.

3.5 Groundwater

Groundwater was generally within the Del Mar formation during our field investigation. Due to the slow exfiltration rate of groundwater from this formation, standing water was not observed within our borings. However, groundwater depth was estimated based on the degree of saturation of soil samples recovered from the borings. This process requires judgment and therefore the estimated depths to groundwater are not considered exact.

During our field exploration groundwater was estimated at a depth of 20 feet bgs in Boring A-18-001 but was not evident in other borings. It is possible that either the degree of saturation noted in Boring A-18-001 was due to perched groundwater in that area or that saturation of samples in other borings existed but was not visibly apparent due to sample disturbance. Nearby borings from the Ninyo & Moore (2016) exploration encountered groundwater at depths ranging from about 16 to greater than 30 feet (not encountered in 30-foot boring) bgs within the depths explored (corresponding groundwater table elevations from about 35 to lower-than-15 feet NAVD88).

Minor surface drainages traverse the project site, which may influence groundwater levels in temporary or perched conditions. Due to the site’s proximity to the Pacific Ocean, groundwater levels may be tidally influenced and lower-bounded by sea level.

Design groundwater elevation was considered to be approximately 20 feet bgs, corresponding to elevation 52 feet NAVD88. Fluctuations of the groundwater level, localized zones of perched water, and an increase in soil moisture should be anticipated during and following the rainy seasons or periods of locally intense rainfall or storm water runoff.

3.6 Engineering Properties of Subsurface Materials

Engineering properties of the subsurface materials were modeled based on results of geotechnical field and laboratory tests performed during our exploration. Results of these laboratory tests are summarized in Table C-1 and presented in Appendix C. These test results are briefly discussed below:

3.6.1 Density and Compaction

The in-situ dry density of the soils in the upper 5 feet averaged approximately 110 pounds per cubic foot (pcf). The moisture content of the soils in the upper five feet averaged approximately six percent. Results of one compaction test (per ASTM D1557) indicated a maximum dry density of 129.5 pcf at an optimum moisture content of 8.6 percent. Based on the laboratory test results, the calculated average relative compaction of the existing subgrade soils in the upper five feet is about 85 percent.

3.6.2 Shear Strength

Based on the direct shear test results, the cohesion intercept (c) and friction angle (ϕ) representing the effective ultimate shear strength of the near surface on-site soils ranged from about 100 pounds per square foot (psf) to 300 psf and 25 to 30 degrees, respectively. Undrained shear strengths for bedrock formational materials encountered along the proposed alignment was 5,300 psf. Based on the laboratory test results, SPT blow counts, and soil types, generalized shear strength parameters and unit weights selected for design are presented in Table 3-1 and grouped based on soil type. Soil strength parameters used in the design analyses are presented in Section 4.2.

Table 3-1. Generalized Soil Design Parameters

Soil Type	Depth bgs (Feet)	Total Unit Weight (pcf)	Friction Angle(1) (degrees)	Cohesion (1) (psf)
Silty or Clayey Sand	0-15	120	30	—
Del Mar Formation	>15	130	—	5,000

Notes:
(1) Ultimate shear strength parameters based on SPT blow counts (NAFVAC, 1986) and laboratory test results.

3.6.3 Corrosion Potential

Samples of the near subsurface soils were subjected to analytical testing to evaluate the potential for corrosion to concrete and ferrous metals using Caltrans Corrosion Guidelines (2018). Caltrans specifications define a corrosive soil as a material in which any of the following conditions exist: a chloride content greater than 500 parts per million (ppm); soluble sulfate content greater than 1,500 ppm; or a pH of 5.5 or less. The tests included in this report are only a screening process for indication of soil corrosivity. A summary of corrosion test results is presented in Table 3-2 and a summary of corrosion potential guidelines is presented in Table 3-3.The subsurface soils at the site have a high corrosion potential to buried concrete materials and are corrosive to buried ferrous metal materials. See Section 4.7 for additional recommendations.

Table 3-2. Summary of Corrosion Test Results

Boring No	Sample Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Sulfates (ppm)	Chlorides (ppm)
A-13-002	2	8.1	1,200	130	122
A-13-002	30	7.4	480	2,530	49

Notes:

ft = feet; ohm-cm = ohm centimeters; ppm = parts per million

Table 3-3. Summary of Corrosion Potential

Boring No	Sample Depth (feet)	(1) Caltrans Corrosion Criteria	(2) NACE Corrosion Potential	(3)Sulfate Attack Potential
A-13-002	2	Not Corrosive	Corrosive	Negligible
A-13-002	30	Corrosive	Severely Corrosive	Severe

Notes:

(1) Corrosivity screening established using the Caltrans Corrosion Guidelines (2018).

(2) Corrosivity screening established using the National Association of Corrosion Engineers, 1984.

(3) Corrosivity screening established using Portland Cement Association, 1988.

3.6.4 Hydrocollapse Potential

Due to the soil types encountered and results of our laboratory tests, hydrocollapse of near-surface soils is not anticipated to have a substantial impact on the design and performance of the Project.

3.6.5 Expansion Potential

Some high-plasticity clay soils are prone to expansion when wetted. Based on our review of the City of Encinitas Housing Element (2015), expansive soils in the area are generally located to the east of the project (described as ‘east of Interstate 5’). Based on this local description as well as the soils encountered during our and other nearby field investigations and laboratory testing, expansion potential of onsite soils is considered low and therefore is not anticipated to have a substantial impact on the design and performance of the Project.

3.7 Scour and Erosion Potential

Because the project does not involve a waterway, scour is not anticipated to be a design element. However, exposed sloped surfaces are prone to erosion and surficial runoff and local drainage should be addressed appropriately. Surficial protection ranging from engineered mats to vegetative cover or gravel beds and drainage swales may be required to mitigate excessive erosion.

3.8 Infiltration Rate

Percolation testing was performed within Boring A-18-002 in general accordance with County of San Diego Department of Environmental Health, Land and Water Quality Division test procedures (CSDEH, 2013). This method is also in accordance with the recommendations provided by Caltrans (2011a).

A 3-inch diameter pipe was installed in the borehole with the bottom and side annular space filled with 3/4 inch gravel. The test zone was then pre-soaked with clean water by filling with water, and allowing the water to percolate. The percolation testing was then performed by measuring the infiltration of the water over time. After completion of the percolation testing, the pipe was removed, and the boring was backfilled with bentonite cement slurry. The ground surface was restored to match its original condition.

Infiltration rates were somewhat variable during testing, as water levels changed and the influence of soil layers within the test also changed throughout the test. Therefore, interpretation and judgment of field data results is required. The in-situ percolation rate was converted to vertical infiltration rates using modified inverse borehole method procedures recommended by San Bernardino County (2011) and others. We recommend the design vertical infiltration rates presented in Table 3-4.

Table 3-4. Recommended Design Vertical Infiltration Rates

Test Location	Test Depth (feet)	Infiltration Rate (in/hr)	USCS Soil Type
A-17-002	4 - 10.0	0.25	SM/SC

Notes:
USCS = Unified Soil Classification System

The design value presented in Table 3-4 does not contain a factor of safety. A factor of safety of at least 2.0 is recommended by Caltrans (2011a) and others. Clayey upper soils as well as bedrock materials were observed during our field investigation which may control the behavior of infiltration basins as well as underground water migration.

Our scope of work was limited to testing, and does not include evaluation of the general suitability of the project site for the infiltration system, evaluation of the storage capacity, nor actual design of the infiltration system. The actual infiltration rate may vary from the values reported herein. The design elevation and size of the proposed infiltration systems should account for the expected variability in infiltration rates. The proposed storm water management system design should be performed by the project Civil Engineer. The designer should take into consideration the variability of the native soils when selecting factors of safety, storage, and other design elements. Additional infiltration basin construction and design recommendations are provided in Section 4.5.

3.9 Faulting and Seismicity

3.9.1 Faults

Like most of Southern California, the Project area is considered to be seismically active. Our review of available in-house literature indicates that there are no known active or potentially active faults that have been mapped at the site, and the site is not located within an State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Bryant and Hart, 2007).

There are several major faults in the San Diego region, including the Newport-Inglewood Rose Canyon Fault Zone (RCFZ), San Andreas, San Jacinto, Elsinore, Palos Verdes–Coronado Bank, San Diego Trough, and San Clemente faults. The prevailing zone of faulting within this region is the RCFZ recognized as a trend of related fault traces. Table 3-5 lists 10 of the most noteworthy faults near the Project and reports the following fault parameters; distance, maximum magnitude, and slip rate (average amount of slip per year). The data was developed by the U.S. Geological Survey (2008) for a probabilistic seismic hazard analysis and refined by Caltrans (2019). A fault map is provided on Figure 5 in Appendix A.

Table 3-5. Principal Active Faults

Fault Name	R_{RUP} (miles)(1)	Maximum Moment Magnitude(1)	Slip Rate (millimeters /year)(2)
Rose Canyon Fault Zone (RCFZ) Del Mar Section	2.4	6.8	1.1
RCFZ Oceanside Section	2.5	6.8	1.1
Coronado Bank	17.6	7.4	2.0
Newport-Inglewood (Offshore)	12.2	6.9	0.8 – 2.1
RCFZ San Diego Section	11.7	6.8	1.1
Elsinore Julian Section	26.9	7.7	4.0
Elsinore Temecula Section	26.9	7.7	4.0
San Diego Trough	27.8	7.3	1.5
Elsinore Glen Ivy Section	41.2	7.7	4.0
RCFZ Silver Strand Section - Spanish Bight	19.8	6.8	1.1

Notes**:**
R_{rup} = closest distance from the site to fault rupture plane which is calculated using Caltrans (2018) methodology.
Slip rates are estimates, provided by Southern California Earthquake Data Center (2018).

3.10 Seismicity

The seismicity of the region surrounding the project site was evaluated using the earthquake database from USGS website (https://earthquake.usgs.gov/earthquakes/search/). Based on the review of the available data, 13 earthquake events with magnitudes equal or greater than 5.0 have occurred within a radius of 60 miles of the site in the last 100 years. The location of the earthquake, year of occurrence, and earthquake magnitude are summarized in Table 3-6.

Table 3-6. List of Historic Earthquakes

Earthquake Location	Date of Earthquake	Earthquake Magnitude
Long Beach, California	1933	6.4
Newport Beach, California	1933	5.3
Trabuco Canyon, California	1938	5.2
Pine Valley, California	1940	5.0
San Clemente Island, California	1951	5.8
Hemet, California	1963	5.3
Borrego Springs, California	1969	5.5
Anza, California	1980	5.3
San Clemente Island, California	1986	5.5
Anza, California	2001	5.0
Anza, California	2005	5.2
Borrego Springs, California	2010	5.4
Borrego Springs, California	2016	5.2

3.11 Seismic Hazards

3.11.1 Fault Rupture

Based on our review of the referenced reports and geologic maps, the Project alignment is not traversed by active or potentially active faults. Therefore, the risk of surface fault rupture for the project is considered low.

3.11.2 Seismic Ground Shaking

A probabilistic seismic hazard analysis was performed using the USGS Unified Hazard Tool (USGS, 2018) to evaluated anticipated ground motions at the project site. The estimated peak ground accelerations for different seismic levels per AREMA are summarized in . The probabilities of exceedance of the seismic events for Level I (100-year return period), Level II (475-year return period), and Level III (2,475-year return period) were reduced using the risk factors per Chapter 9 of AREMA (2015). The risk factors used in Table 3-7 were estimated and should be verified by the structural engineer. Additional seismic design information is provided in Section 4.2.1.

Table 3-7. AREMA Risk Factors

Risk Factor	Value (1)
Immediate Safety	
Occupancy Factor	4
Hazardous Material Factor	1
Community Lifelines Factor	3
Immediate Value	
Railroad Utilization Factor	4
Detour Availability Factor	1
Replacement Value	
Span Length Factor	1
Bridge Length Factor	1
Bridge Height Factor	0.75

Notes:
(1) Values used for risk factors were estimated according to Chapter 9 in AREMA (2015).

3.11.3 Liquefaction and Seismically-Induced Settlement

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine- to medium-grained, cohesionless soils. Effects of liquefaction can include sand boils, settlement, bearing capacity failures, and lateral spreading. Seismically-induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). This settlement occurs primarily within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. The Project is located near the border of an area designated as potentially liquefiable by the County of San Diego (2009a).

Due to the lack of observed groundwater in upper alluvial and fill soils, and the very dense/hard nature of Del Mar Formation below, liquefaction is not expected at the project site.

3.11.4 Lateral Spreading

Lateral spreading is a type of landslide motion generally characterized by progressive cracking and ground motion near a slope face. Lateral spreading is generally associated with liquefiable soils which allow the slope face and surrounding area to flow during or shortly after earthquake ground motions.

Due to the lack of expected liquefaction at the project site, lateral spreading is not expected at the project site.

3.11.5 Seiches and Tsunami

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Although near the Pacific Ocean, there is a relatively steep grade between the project site and the shore. CGS (2009) maps the project site just outside a tsunami inundation area. Therefore, the risks of seiche and tsunami at the site are considered low.

3.11.6 Earthquake-induced Flooding

Earthquake-induced flooding is caused by dam failures or other water-retaining structure failures as a result of seismic shaking. Our review of the San Diego County Dam Inundation Areas Map (County of San Diego, 2009b) found that the project area is not located within areas of potential susceptibility to dam inundation. The potential for earthquake-induced flooding is considered low.

3.12 Flooding

Our review of the San Diego County Dam Inundation Areas Map (County of San Diego, 2009b) found that the project area is not located within a 100 year floodplain or floodway. Therefore, the potential for flooding along the proposed alignment is considered low.

We understand that the new construction may create a localized low-point where water may collect. A careful hydraulic and hydrology analysis of localized runoff and drainage should be performed to prevent localized ponding or flooding of the undercrossing structure.

3.13 Slope Stability

The project area is located within a relatively flat terrain. Existing and proposed slopes are considered stable for the static and pseudo-static conditions with final slopes of 2H(horizontal): 1V(vertical) or shallower. Due to the shallow nature of existing relatively competent bedrock, proposed retaining walls with footings extending into the rock formation are considered stable. If steeper proposed slopes or other major earthwork modifications are proposed, they should be reviewed by the geotechnical engineer.

3.14 Historic Landslides

Our review of the City of Encinitas Housing Element (2015) found that the project area is not mapped in an area of known landslides. Landslides are known to occur regionally, generally where the steepest slopes are exposed along erodible creeks and waterways. The area is mapped within a zone defined as ‘marginally susceptible’ which is considered ‘unlikely to mobilize under natural conditions’.

During the site reconnaissance and review of recent aerial photographs, evidence of recent movement was not observed. Slopes did show typical signs of erosion and some areas of shallow surficial slumping, which is typical for all slopes and part of the natural degradation process.

Based on these observations and the field investigation performed, the potential for gross instability of existing slopes is considered low. Normal surficial slope degradation processes, such as erosion, slope creep, and shallow surficial slumping, can be anticipated.

3.15 Static Settlement

Deep, saturated layers of silts and clays which are prone to settlement issues are generally not prevalent in the project area. Based on the proposed improvements and the lack of compressible soils present, static settlement is not anticipated to be a design issue.

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4 Geotechnical Recommendations

Based upon our evaluation of the subsurface conditions and geologic information, we conclude that the proposed Project is feasible from a geotechnical standpoint provided that the recommendations presented in this report are properly incorporated in the design and construction of the Project. The recommendations in this report are considered a minimum and may be superseded by updated geotechnical recommendations or more stringent requirements of the structural engineer and/or the governing agencies. HDR should be notified, in a timely manner, of changes in the Project plans that might impact recommendations in this report.

4.1 Earthwork

4.1.1 Site Preparation

Prior to construction, the site should be cleared of all existing improvements and debris. Existing utility and irrigation lines should also be either removed or protected in place if they interfere with the proposed construction. Cavities resulting from removal of the existing underground structures and lines should be excavated to expose competent material before being properly backfilled and compacted.

4.1.2 Overexcavation

Beneath proposed spread footing at Retaining Wall No. 2, and in areas particularly sensitive to settlement such as near the bridge abutments, removal and recompaction of approximately two (2) feet below the existing grade or two (2) feet below the finish subgrade or footing bottom, whichever is deeper, should be anticipated. Laterally, the compacted fills should extend a minimum of 2 feet beyond the subballast outer edges wherever track is removed and replaced. The exact extent of removals can best be determined during grading when direct observation and evaluation of exposed materials are possible. Other local conditions may be encountered which could require additional removals, such as deeper than anticipated fill materials. Overexcavation is not required around structures such as bridge abutments or bents when the foundation is supported on piles.

Temporary excavation slope considerations are presented in Section 5.3.

4.1.3 General Fill Placement and Compaction

Exposed subgrade soil surfaces, including all excavation or removal bottoms, should be observed by a representative of the geotechnical engineer prior to placement of fill. Competent excavation bottoms should be scarified to a minimum depth of 8 inches, moisture-conditioned to above the optimum-moisture content, and then compacted to a minimum of 95 percent relative compaction (per ASTM D1557).

If soft, pumping subgrade is exposed during grading, stabilization methods may be required. This may consist of overexcavating an additional 12 to 18 inches of depth and placing crushed aggregate (grading from 3/4-inch to 2-1/2-inches in size). As a viable alternative, a double geogrid layer, consisting of Tensar BX1200 or equivalent biaxial geogrid, in combination with a 6-inch thick layer of crushed aggregate, as indicated above, may be considered. These conditions should be evaluated by the geotechnical engineer at the time of removals.

4.1.4 Fill Material

The soils encountered at the boring locations are generally suitable for use as compacted structural fill, provided that they are free of organic material, debris and oversized material. Soils to be placed as fill, whether onsite or import material, should meet the requirements specified in AREMA (2015) and be approved by the geotechnical engineer. Import soils should be free of environmentally regulated substances, granular in nature (with percent passing No. 200 sieve less than 35 percent), free of organic material, free of rock greater than 3 inches in maximum size, have very low expansion potential (with an expansion index less than 21 per ASTM D4829 and plasticity index less than 15) and have a low corrosion impact (classified as non-corrosive by Caltrans, see Section 3.6.3) to the proposed improvements. All fill soils should be placed in thin (under 8 inches uncompacted), loose lifts with each lift properly moisture controlled to zero to two percent above optimum moisture content and compacted to a minimum of 95 percent relative compaction per ASTM D 1557. Subballast and aggregate base should be compacted to a minimum of 95 percent relative compaction (ASTM D 1557).

4.1.5 Rippability

Based on our findings from the previous and current field explorations, we anticipate that the surficial soil deposits can be excavated with moderate effort using conventional drilling/ earthwork equipment. Various excavations will be made into formational material (described as Del Mar Formation, see Sections 3.2 and 3.4). In these locations, we expect moderate to heavy excavation effort due to the presence of gravel and cobble layers. Locations along the proposed alignment where gravel and cobbles have been encountered may require heavy excavation equipment.

4.2 Structures

We understand that the preferred design includes the construction of a railroad bridge for pedestrian undercrossing and associated retaining walls. Design parameters for these structures are provided below.

4.2.1 Response Spectra

A description of site seismicity is provided in Section 3.11.2. Additionally, the response spectra for the site based on both the AREMA and Caltrans guidelines were calculated for the proposed bridge. The AREMA response spectra were obtained using the horizontal accelerations shown in for the corresponding design event in accordance with Chapter 9 of AREMA (2015) with 5 percent damping. As noted in Section 3.11.2, seismic return period was adjusted based on estimated risk factors. These factors were estimated based on the current understanding of the bridge design, and should be verified by the structural engineer. The ARS curves and tabulated data are provided on Figures 1 and 2 in Appendix E. Spectral response accelerations for each return period were based on input values from Site Class B, and were adjusted to Site Class C (site's estimated Site Class) in accordance with AREMA (2015).

Additionally, a Caltrans ARS was developed for retaining structures which retain highway loading. The Caltrans ARS curve was developed using the ARS online tool version 2.3.09 (Caltrans 2019). This tool combines three different spectra, using deterministic and probabilistic methods and returns an envelope spectrum. The Caltrans methodology is described in more detail in *Methodology for Developing Design Response Spectrum for Use in Seismic Design Recommendations* (Caltrans, 2012) and the associated ARS curve and tabular data are presented on Figures 1 and 2 in Appendix E.

4.2.2 Foundation Type

We understand that the proposed bridge structure will be structurally similar to the nearby Santa Fe Drive Pedestrian Undercrossing (TYLin, 2012). Based on our review of the as-built plans for Santa Fe Drive, we anticipate that the bridge will be about 62 feet in length and carry two tracks, and contain three spans (two abutments and two intermediate bents). We anticipate that each support will contain six 24-inch diameter CIDH piles in a single row, and the piles will be spaced at about 5.5 feet on center. Pile cutoff elevations are estimated at about 6 feet below rail at each abutment, and about 16 feet below rail at each bent. Rail elevation at the proposed bridge location is about 72 feet NAVD88.

We understand that retaining wall foundations will consist of either spread footings or soldier piles with tiebacks.

The following sections provide recommended geotechnical parameters for design and construction of the proposed foundations.

4.2.3 Axial Capacity of Piles

Soil strength data collected from the geotechnical investigation and laboratory testing program were used to estimate axial pile capacities. It should be noted that the pile capacities shown are based on soil strengths alone without consideration of pile materials and connections. The piles and related connections should be evaluated for structural capacity as part of the structural design. Settlements of piles generally result from the settlement of the supporting soils and elastic compression of piles. The estimated settlement for piles constructed based on the design recommendations in this report is less than one half inch.

Design of CIDH piles (constructed as described in Section 5.1), of 24 inches in diameter, was performed on the basis of shaft friction, neglecting end bearing, using Ensoft SHAFT software (2012). Ultimate axial pile capacity is estimated with respect to elevation, with capacity beginning at bedrock (approximately 56 feet NAVD88), and therefore the axial capacity within fill at the abutments is neglected.

Based on our analysis, the ultimate compressive capacity of the piles is 14.6 kips per foot of embedment below elevation 56 feet NAVD88 (approximate bedrock contact elevation). Tension capacity is estimated as 70 percent of compressive capacity, or 10.2 kips per foot embedment below elevation 56 feet NAVD88. For service loading, a factor of safety of 2.5 should be applied. For AREMA Seismic Level 1, 2, and 3, factors of safety of

2.5, 1.8, and 1.0 should be applied, respectively. To avoid group effects, all piles should be spaced at a minimum center-to-center spacing of 5.5 feet.

4.2.4 Lateral Capacity of Piles

The lateral resistance and deflections of vertical pile foundations are governed by the resistance-displacement characteristics of near-surface soils and the material strength of piles. The parameters presented in Table 4-2 can be used in the lateral pile capacity analysis (*LPILE* program, Ensoft Inc., 2016). For the application of a ‘safety factor’ against overturning calculations determined using AREMA criteria, the embedment length should be determined by increasing the lateral load by a factor of 1.5 for Service, 1.3 for Seismic Level I, and 1.1 for Seismic Level III, and using the below LPILE criteria to determine critical embedment length. Based on our analyses, a preliminary estimation of critical embedment length for lateral loading is in the range of 20 to 25 feet (to about elevation 36 feet). These values should be checked when design lateral loading is known.

The estimated lateral capacities presented below are for single piles and do not consider a reduction for group action. Group action reduction factors are based on the pile configuration and spacing. Based on the estimated spacing described in the sections above, appropriate reduction factors are 0.75 for loading in row (loading in direction of train travel) and 0.48 average for loading in line (transverse to tracks). Table 4-1 presents the lateral load reduction factors to be applied for various pile spacing for in-line loading based on Caltrans Amendments to AASHTO LRFD Bridge Design Specifications (2014) to be considered if other pile layouts are under consideration. For spacing in between those provided below, a linear interpolation may be utilized to calculate the reduction factor.

The deflection, shear, and moment development of piles based on deflections of 0.25 inch, 0.5 inch, and 1.0 inch deflection for both pinned and fixed head connections are presented on Figures 2 and 3 in Appendix E. As described above, group reduction factors should be applied as appropriate.

Table 4-1. Lateral Load Reduction Factors

Center-to-Center Pile Spacing in the Direction of Loading	Ratio of Load Resistance of Piles in Group to Single Pile		
	Row 1	Row 2	Row 3+
7D	1.0	1.0	0.90
5D	1.0	0.85	0.70
3D	0.75	0.55	0.40

Source: Caltrans Amendments to AASHTO LRFD Bridge Design Specifications- Sixth Edition (2014).

Notes:
D = diameter or width of the pile

Table 4-2. Soil Parameters for Lateral Pile Capacity Analysis - Abutments

Depth of Layer (feet)	LPILE Model	Total Unit Weight (pcf)	Internal Friction Angle (degree)	Cohesion (psf)	p-y Modulus K (pci)	Strain Factor (E50)
0 - 10	SAND	120	30	—	100	—
> 10	STIFF CLAY w/o Free Water	130	—	5,000	—	0.01

Notes:

[1] Pile cut off modeled at elevation 66 feet NAVD88. Distance from pile top to ground surface conservatively modeled as zero in LPILE program. Groundwater modeled at elevation 52 feet MSL.

Table 4-3. Soil Parameters for Lateral Pile Capacity Analysis - Bents

Depth of Layer (feet)	LPILE Model	Effective Unit Weight (pcf)	Internal Friction Angle (degree)	Cohesion (psf)	p-y Modulus K (pci)	Strain Factor (E50)
> 0	STIFF CLAY w/o Free Water	130	—	5,000	—	0.01

Notes:

[1] Pile cut off modeled at elevation 56 feet NAVD88. Distance from pile top to ground surface conservatively modeled as zero in LPILE program. Groundwater modeled at elevation 52 feet MSL.

4.3 Retaining Walls

We understand that retaining walls are proposed generally on the downslope (southwest) side of the railroad embankment. These walls will provide space for pedestrian access to the proposed bridge undercrossing structure. We estimate that total retaining wall lengths will be in the range of about 100 feet, and exposed wall heights will reach approximately 10 feet at maximum.

4.3.1 Lateral Earth Pressures

Earth-retaining structures should be designed using the lateral earth pressures provided in Table 4-4. A soil unit weight of 120 pounds per cubic foot (pcf) may be used for calculating the actual weight of the soil over the wall footing. The magnitude of these pressures depends on the amount that the wall can yield horizontally under load. If the wall can yield enough to mobilize full shear strength of backfill soils, then the wall can be designed for "active" pressure. If the wall cannot yield under the applied load, the shear strength of the soil cannot be mobilized and the earth pressure will be higher. Such walls should be designed for "at-rest" conditions. If the wall moves toward the soils, the resulting resistance developed by the soil is the "passive" resistance.

Table 4-4. Lateral Earth Pressures

	Equivalent Fluid Pressure (pcf)	
	Sand Backfill (Sand Equivalent of 30 or greater)	
Condition	Level Backfill	2H:1V Backfill
Active	37	56
At-Rest	56	84
Passive	375 to maximum 3,750 psf	140 (sloping down)

The values in Table 4-4 do not contain a factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design. The design values in Table 4-4 are based upon drained conditions. Proper drainage should be provided behind the walls to prevent buildup of hydrostatic pressure behind the walls. Resistance to lateral loads can be provided by friction developed between the bottom of footings and the supporting soil and by the passive soil pressure, as presented above, developed on the face of the footings. For design purposes, an ultimate coefficient of friction of 0.5 may be used. AASHTO (2017) Table 10.5.5.2.2-1 recommends reduction factors to be applied depending on construction method and load state. For lateral resistance of soldier pile wall foundations, see Section 4.3.2.

Surcharge loading from nearby active rail should be considered in the design of retaining structures. In addition to the above lateral pressures from retained earth, lateral pressures from other superimposed loads, such as those from adjacent structures or vehicles, should be added per Section 5 of Chapter 8 of AREMA (2015) and/or Section 6 of Caltrans *Trenching and Shoring Manual* (Caltrans 2011b). For surcharge loading onto wing walls or other retaining wall structures, loads should be calculated according to AREMA Chapter 8 Section 20.3.2.

We understand that some structures (i.e. walls retaining highway loading) are designed using Caltrans methodology, and others (i.e. bridge, retaining walls supporting rail) are designed using AREMA methodology. These two different approaches to seismic loading are presented below:

> **AREMA**: Per the SCRRA Design Criteria Manual (2014), the ground acceleration value used for calculating seismic earth pressure was 0.22g, corresponding to the Level II seismic event PGA. For seismic loading and level backfill, a triangular pressure distribution of 8 pcf (equivalent fluid pressure), may be used in addition to the static earth pressures and should be factored as appropriate. This seismic earth pressure may be assumed to act with a similar load distribution as static pressures, and is applicable for both cantilever and braced conditions. Forces resulting from wall inertia effects are expected to be relatively minor for non-gravity walls and may be ignored in estimating the seismic lateral earth pressure.
>
> **CALTRANS**: The peak ground acceleration value of 0.43g was used, corresponding to the Caltrans seismic design method outlined in Section 4.2.1. For seismic loading and level backfill, a triangular pressure distribution of 18 pcf (equivalent fluid pressure), may be used in addition to the static earth pressures and should be factored as appropriate. This seismic earth pressure may be

assumed to act with a similar load distribution as static pressures, and is applicable for both cantilever and braced conditions. Forces resulting from wall inertia effects are expected to be relatively minor for non-gravity walls and may be ignored in estimating the seismic lateral earth pressure.

It is recommended that all retaining walls be backfilled with non-expansive granular soils, i.e., backfill Types 1 and 2 per Section 5.2.5, Chapter 8 of AREMA (2015). Backfill for retaining walls should be compacted to a minimum of 95 percent relative compaction (based on ASTM D1557) moisture controlled to zero to two percent above optimum moisture content. During construction of retaining walls, the backcut should be made in accordance with the requirements of Cal/OSHA Construction Safety Orders (California DIR, 2015). Relatively light construction equipment should be used to achieve the compaction requirement behind retaining walls.

4.3.2 Soldier Pile Walls and Tiebacks

Soldier pile walls are proposed on the downslope (southwest) side of the tracks. Earth pressures should be designed in accordance with Caltrans methodology (2011b) and as presented in Section 4.3.1 of this report. An active earth pressure coefficient k_a of 0.32 may be used in analyses. Passive pressures of 500 pcf may be used within the Del Mar Formation (considered at about elevation 52 feet NAVD88). A passive arching factor of up to three may be used, provided that pile center to center spacing is at least three pile diameters. The upper 1.5 pile diameters should be ignored in determining passive resistance. For the axial capacity of drilled shafts containing soldier piles, allowable side friction values of 750 psf may be used. An allowable end bearing value of 8 kips per square foot may be used for piles with proper bottom cleanout construction practices.

We understand that a soldier-pile-with-tieback wall is proposed to support the rail north of the bridge to provide pedestrian access. Tiebacks should de designed to derive their load carrying capacity from the soil behind the active wedge behind the wall. This wedge is defined by a plane drawn at approximately 60 degrees above horizontal from the bottom of the wall, i.e. the non-retained ground elevation. Tiebacks should have a minimum unbonded length of 10 and 15 feet for bars and strands, respectively. Apparent unbonded length should meet the requirements set forth in PTI (2014), Sections 8.6.2.2 and 8.6.2.3. All tiebacks should have a minimum bonded length of 15 feet and be spaced at least four feet on center, with the bond zone beginning at least five feet behind the failure plane as defined above. The center of the bonded zone should be at least 15 feet below ground. Prior to installation of tiebacks, the contractor should verify site conditions such that there is no conflict with existing utilities, foundations and/or other subsurface structures. Tiebacks should be located such that they are not within three feet of existing utilities if gravity-grouted or five feet of existing utilities if pressure-grouted.

Tieback grout-to-ground bond ultimate capacity will vary depending on whether the anchor is founded in soil or in rock. The ultimate capacity can be expected to range from about 1.5 to 5 kips per square foot (ksf) for gravity or pressure grouted anchors, respectively, when grouted in soil. These values can be increased to about 10 to 30 ksf when grouted in bedrock. However, these values are highly dependent on contractor methodology, and a factor of safety of at least 2.0 is recommended by FHWA (1999).

In order to evaluate tieback anchor capacity, it is recommended to perform anchor load tests in the field using performance or proof testing procedures. Anchor load testing should be performed according to the FHWA (1999) or Caltrans Special Provisions Section 50-560. All tiebacks should be tested to verify anchor design criteria including length, diameter, grouting pressure, etc.

The acceptable creep criteria for anchors subject to either performance or proof tests should not exceed 0.04 inches between 1 and 10 minutes for total movement. If movements are less than 0.04 inches, the anchor is considered acceptable. If the total movement exceeds 0.04 inches, the load is held for an additional 50-minute period, and the anchor is considered to be acceptable if the total movement between 6 and 60 minutes is less than 0.08 inches.

Each production anchor should be locked-off at the design load if the test is considered satisfactory. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be reset until the anchor is locked-off within 10% of the design load.

Corrosion protection should be provided for temporary and permanent anchors according to Caltrans Special Provisions Section 50-560 and/or Class I protection based on FHWA requirements (FHWA, 1999). These may include the use of PVC, HDPE or polypropylene sheathing, centralizers, corrosion inhibiting grease and cementitious grout. The contractor should be responsible for providing corrosion protection to tiebacks and any of its elements that may be exposed to corrosive attack from surrounding soil.

The Geotechnical Engineer or their representative should be present during installation and testing of tiebacks.

4.4 Allowable Bearing Capacity

We understand that a spread footing will be used to support Retaining Wall No. 2, which supports highway loading. Spread footings at this location require two feet of over excavation and recompaction/replacement of soil as engineered fill as described in Section 4.1.

For this foundation, an allowable bearing capacity of 2,500 psf may be used with a minimum embedment of 18 inches below the lowest adjacent grade and minimum width of 3 feet. This allowable bearing pressure may be increased by 1,000 psf for an additional foot of embedment or by 500 psf for an additional foot of width, to a maximum value of 3,500 psf. This value may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces. The recommended allowable bearing capacity for shallow footings is generally based on a total allowable static settlement of 1 inch and differential settlement of ½ inch over a horizontal distance of 30 feet for shallow footings.

4.5 Infiltration Basin Design

Due to the relatively low infiltration rates estimated during our field testing program (see Section 3.8), the use of a dedicated infiltration-only basin is not recommended. Additionally, clayey upper soils as well as deeper bedrock materials may inhibit large-scale infiltration at the site. However, best management practice (BMP) may not preclude the use of bioswale-type pretreatment or detention options.

Effective infiltration BMP design requires proper design assumptions and proper device maintenance. The application of each BMP should consider the possible requirements for water pretreatment, device siltation/clogging, consequences of under/over performance, and other considerations. The potential for requiring water pretreatment should be considered, depending on design application. Where infiltration is intended, the soil at the bottom of the proposed BMP should not be compacted, and should be inspected during construction by HDR or our geotechnical representative for consistency with the design recommendations herein.

With time, the bottoms of infiltration systems tend to plug with organics, sediments, and other debris. Long term maintenance will likely be required to remove these deleterious materials to maintain design percolation rates. Restrictions on locations of Infiltration systems include being located at least 10 feet from any existing or proposed foundation system, being located away from slopes, and other considerations. Due to the site's proximity to slopes, active rail, highway pavement, and other features, BMP methods should be considered carefully and should be located and designed appropriately. Design plans and proposed infiltration methods should be reviewed by the geotechnical engineer during design. For additional recommendations see the references from Caltrans (2011a) and CSDEH (2013).

The potential for underground contamination and the implications of installing a BMP should be considered during design. Although soil contamination analysis is outside the scope of our efforts, we understand that a separate environmental document has been prepared for the project which should be reviewed concurrent to BMP design.

4.6 Trench Backfill

Utility trenches should be backfilled with compacted fill in accordance with Section 10.4, Chapter 8 of AREMA or Sections 306-1.2 and 306-1.3 of the Standard Specifications for Public Works Construction, ("Greenbook"), current edition. Fill material should be placed in horizontal layers of thickness compatible to the type of equipment being used and should be compacted to at least 90 percent relative compaction (ASTM D1557) by mechanical means only. Utility pipes should be placed on properly placed bedding materials extended to a depth in accordance with the pipe manufacturer's specification. The pipe bedding should extend to at least 12 inches over the top of the pipe for the full trench width. The bedding material may consist of compacted free-draining sand, gravel, or crushed rock with a maximum particle size of ¾". Bedding materials should also conform to the pipe manufacture's specifications. If sand is used, the sand should have a Sand Equivalent (California Standard Test Method 217) of 30 or greater. If gravel is used for the bedding material, the gravel should be wrapped with the filter fabric (Mirafi 140N or equivalent). Controlled Low Strength Materials (CLSM) may also be used within the bedding zone and the CLSM should conform to Section 201-6 of the Greenbook. We recommend that the materials other than CLSM used for the bedding zone be placed and compacted with mechanical means. Densification by water jetting should not be allowed.

Above the bedding zone, trenches can be backfilled with the onsite material, provided it is free of debris, organic material and oversized material greater than 3 inches in largest dimension. Oversized rock (cobbles and/or boulders) should either be removed from the alignment or pulverized for use in backfill. Gravel larger than ¾ inches in diameter should be mixed with at least 80 percent soil by weight passing the No. 4 sieve. CLSM may also be used to backfill the trenches.

Backfill should be placed in thin lifts, loose lift thickness being compatible with the earthwork equipment but not exceeding 12 inches, moisture-conditioned as necessary, and mechanically compacted to a minimum 90 percent relative compaction (ASTM D 1557). The upper 12 inches of trench backfill in pavement areas should be compacted to a minimum 95 percent relative compaction.

4.7 Cement Type and Corrosion Measures

A discussion of soil corrosion results is included in Section 3.6.3. The tests included in this report are only a screening process for indication of soil corrosivity. In general, foundation elements should be designed for a severely corrosive environment toward buried ferrous metals, and a corrosive environment for buried concrete structures. Concrete mix design should follow the recommendations within the LOSSAN Service Life Design Guide (SANDAG, 2014). Based on our review of the LOSSAN Design Guide, the project is generally categorized as being within an ‘inland low-exposure’ zone. Type V Portland Cement is an appropriate concrete type on the Project, and appropriate strength and mix requirements should be selected based on structures’ design life and structural requirements.

5 Construction Considerations

5.1 Pile Construction

We understand that the bridge foundations for the Project will be constructed using CIDH methods.

The drilling operations are recommended to be observed and evaluated by a representative of the geotechnical engineer to allow further evaluation of the actual subsurface conditions. Groundwater is expected to be approximately 20 feet below the existing grade, although exfiltration of the groundwater within the Del Mar formation may be slow. Due to the nature of sandy, gravelly soils, the presence of cobbles, and relatively shallow groundwater table, caving or drilling refusal may be encountered during pile construction and temporary casing or drilling slurry may be necessary to facilitate the construction of the piles. The installation/removal of temporary casing or the use of slurry for borehole stability should be in accordance with the Caltrans Standard Specifications (Caltrans, 2010) and/or AREMA (2015) to reduce the potential for adversely affecting the frictional resistance of the soils and thereby reduce the load capacity of the piles. If the wet method is utilized for the installation of piles, Gamma-Gamma tests should be performed to verify the integrity of the piles and detect presence of anomalies. Cross-hole Sonic Logging (CSL) can be performed as a complementary test to better identify the location and size of the anomalies within the pile. The tests should be performed in accordance with Caltrans specifications.

To maintain a relatively clean hole and to achieve high quality pile construction, it is recommended that the entire construction operation including drilling of the pile borehole, lowering of the steel casing and/or reinforcing cage, and placing concrete be carried out consecutively. The pile excavation should not be allowed to remain open for more than 12 hours. Piles within 10 feet at their nearest point to one another are considered adjacent piles. One adjacent pile may only be drilled a minimum 24 hours after placement of concrete in another adjacent pile. We further recommend that a tremie pipe with pumped concrete be used to avoid concrete segregation during pile construction.

Although specific pile construction techniques should be selected by the contractor in conjunction with the design team, it is critical that certain elements of pile construction be maintained in order for the recommendations in this report to remain applicable. The contractor’s final pile design details should be reviewed and approved by the design team including representatives of the geotechnical engineer.

5.2 Groundwater Control

Based on the current and previous field explorations, groundwater levels are expected to be deeper than 10 feet below the existing grade. However, localized perched groundwater may exist at shallower depths on a seasonal basis. Relatively shallow groundwater inflow may be controlled by a system of collection ditches and sump pumps.

5.3 Temporary Excavations

Excavations for pile caps or other appurtenant structures that are 5 feet or deeper should be laid back or shored in accordance with CAL/OSHA (California DIR, 2015) requirements before personnel are allowed to enter. Soil type “B” may be assumed for formational site soils, with soil type “C” used for fill or cohesionless alluvial soils which are anticipated to be shallow in depth (Section 3.4). For temporary excavations greater than 5 feet deep that cannot be adequately sloped for stability, some form of temporary external support will be required. In consideration of the type of construction, the most practical method is expected to be excavation bracing. The lateral earth pressure for this type of shoring is estimated as 25H psf (evenly distributed), where H is the depth of excavation and the resulting lateral pressure distribution is rectangular pressure. This above lateral pressure is only appropriate for level backfill and a drained condition behind the shoring. Shoring should also be designed to resist lateral surcharge from train loading, adjacent vehicular traffic, construction equipment, and existing structures. The contractor should be responsible for the structural design and safety of all temporary shoring systems.

5.4 Additional Geotechnical Services

The proposed construction involves various activities that would require geotechnical observation and testing. These include:

- Removal and/or excavation bottom;
- Placement of compacted fill;
- Pile installation;
- Footing excavation; and
- When any unusual conditions are encountered.

These and other soils-related activities should be observed and tested by a qualified representative of the geotechnical engineer.

5.5 Limitations

This report has been prepared for the use of HDR, City of Encinitas, NCTD, and SANDAG for the proposed Verdi Avenue Undercrossing Project. The report may not be used by others without the written consent of our client and our firm. The conclusions and recommendations presented in this report have been based upon the generally accepted principles and practices of geotechnical engineering utilized by other competent engineers at this time and place. No other warranty is either expressed or implied.

Additionally, the conclusions and recommendations presented in this report have been based upon the subsurface conditions encountered at discrete and widely spaced locations and at specific intervals below the ground surface. Soil and groundwater conditions were observed and interpreted at the exploration locations only. This information was used as the basis of analyses and recommendations provided in this report. Conditions may vary between the exploration locations and seasonal fluctuations in the groundwater level may occur due to variations in rainfall and local groundwater management practices. If conditions encountered during construction differ from those described in this report, our recommendations may be subject to modification and such variances should be brought to our attention to evaluate the impact upon the recommendations presented in this report.

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Appendix A. Figures

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SITE LOCATION MAP

VERDI AVENUE UNDERCROSSING PROJECT
SAN DIEGO COUNTY, CALIFORNIA

Figure 1

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EXISTING DATA MAP

VERDI AVENUE UNDERCROSSING PROJECT
SAN DIEGO COUNTY, CALIFORNIA

Figure 2

INVESTIGATION LOCATION MAP

VERDI AVENUE UNDERCROSSING PROJECT
SAN DIEGO COUNTY, CALIFORNIA

Figure 3

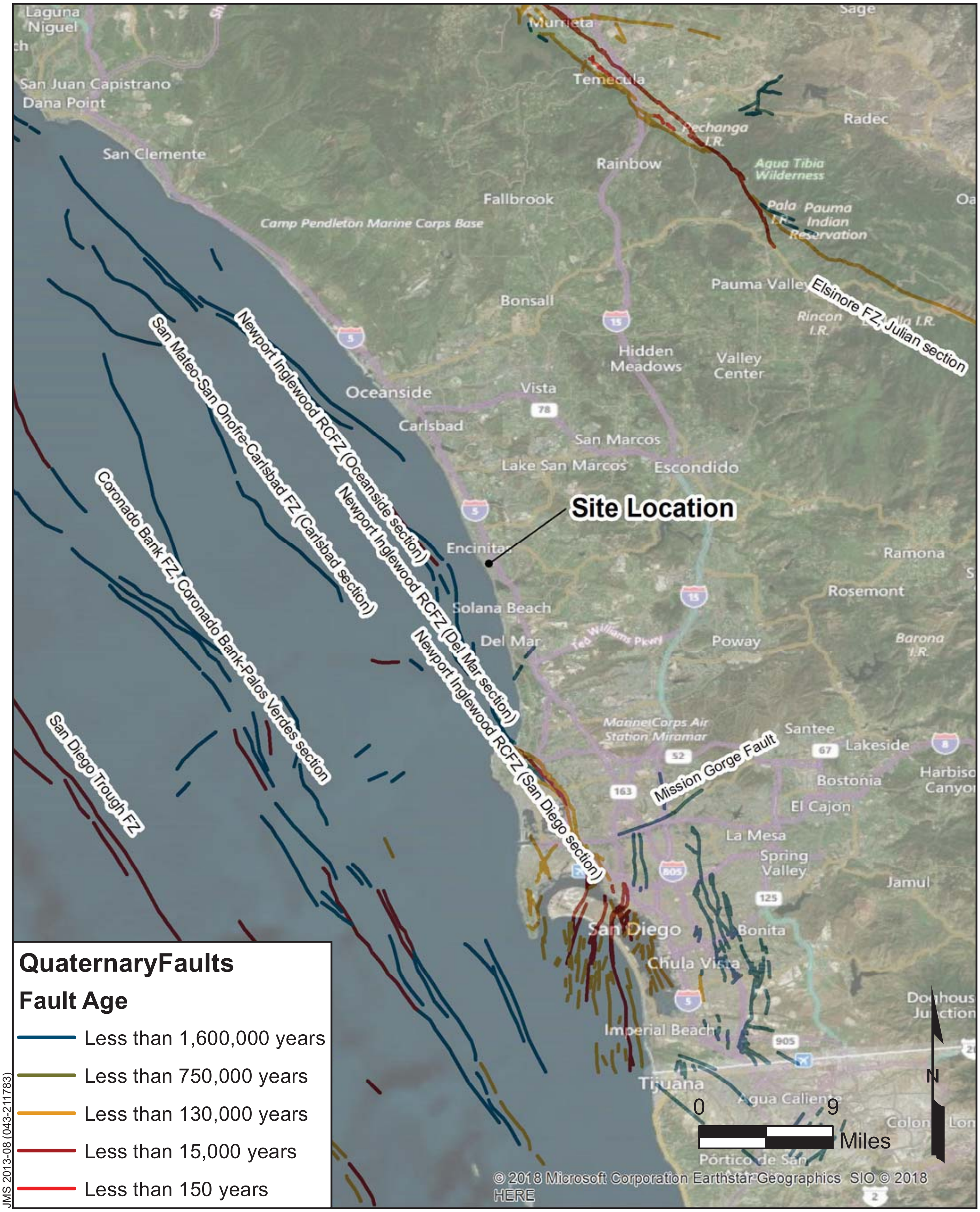
Qb	**Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand
Tsh	**Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments
Qol	**Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types
Qvol	**Very Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - moderately to well-consolidated, highly dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types

Reference: CGS, 2010

GEOLOGIC MAP

VERDI AVENUE UNDERCROSSING PROJECT
SAN DIEGO COUNTY, CALIFORNIA

Figure 4



Reference: USGS and CGS, 2006

JMS 2013-08 (043-211783)

FAULT MAP

VERDI AVENUE UNDERCROSSING PROJECT
SAN DIEGO COUNTY, CALIFORNIA

HDR

Figure 5

Appendix B. Geotechnical Boring Logs

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UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$C_u \geq 4$ AND $1 \leq C_c \leq 3$	GW	WELL-GRADED GRAVEL	▨
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	POORLY-GRADED GRAVEL	▨
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL	▨
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	▨
	SANDS >50% OF COARSE FRACTION PASSES NO 4. SIEVE	CLEAN SANDS <5% FINES	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	WELL-GRADED SAND	▨
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	POORLY-GRADED SAND	▨
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND	▨
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	▨
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY	▨
			PI>4 AND PLOTS<"A" LINE	ML	SILT	▨
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT	▨
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY	▨
			PI PLOTS <"A" LINE	MH	ELASTIC SILT	▨
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT	▨
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	▨

OTHER SYMBOLS

	MATERIALS		SAMPLERS
■	Asphalt	☒	SPT (2" OD)
▨	Aggregate Base	⧗	Modified California (3" OD)
▨	Boulders & Cobbles	⧗	California (2.5" OD)
▨	Fill	▨	Bulk
▨	Topsoil	▮	Shelby Tube
▨	WELL	▮	HQ Core
▨	Concrete Grout/Fill	▨	Sonic Core
▨	Bentonite/Grout Seal	▽	INITIAL WATER LEVEL MEASUREMENT(WITH DATE)
▨	Sand Pack + Solid Pipe	▼	STABILIZED WATER LEVEL MEASUREMENT(WITH DATE)
▨	Sand Pack + Slotted Pipe		

GRAIN SIZES

U.S. STANDARD SIEVE	200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

PENETRATION RESISTANCE

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT (N_{60})	CONSISTENCY	BLOWS/FOOT*	UNC. COMP. STRENGTH (TSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 1	0 - 1/4
LOOSE	5 - 10	SOFT	2 - 4	1/4 - 1/2
MEDIUM DENSE	11 - 30	MEDIUM STIFF	5 - 8	1/2 - 1
DENSE	31 - 50	STIFF	9 - 15	1 - 2
VERY DENSE	OVER 50	VERY STIFF	16 - 30	2 - 4
		HARD	OVER 30	OVER 4

* 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).

NOTES

bgs	BELOW GROUND SURFACE
c	COHESION
CD	CONSOLIDATED DRAINED TRIAXIAL
CN	CONSOLIDATION
CR	CORROSIVITY
CU	CONSOLIDATED UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
HY	HYDROMETER
MD	MAX DENSITY (COMPACTION)
N_{60}	BLOW COUNT, Corrected for Hammer Energy Only
PI	PLASTICITY INDEX
PR	PERMEABILITY
RV	R-VALUE
SA	SIEVE ANALYSIS
SE	SAND EQUIVALENT
TC	CYCLIC TRIAXIAL
TR	TIME RATE OF CONSOLIDATION
UC	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL

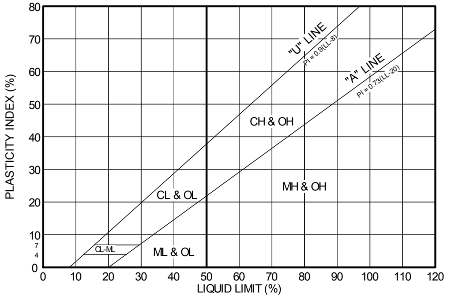
INCREASING VISUAL MOISTURE CONTENT

↑
WET
MOIST
DRY

COMPONENT PERCENTAGE

MOSTLY	>50%
SOME	30 - 50%
LITTLE	15 - 29%
FEW	5 - 14%
TRACE	<5%

PLASTICITY CHART



Boring Legend

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Figure

LEGEND 2018-02 VERDI GRADE SEPARATION.GPJ FOLSOM 3-30-11.GDT 4/30/18

LOGGED BY: **MF** DATE: START **3/30/18** END **3/30/18**

STATION & OFFSET: **75+75, 50ft R** LATITUDE: **33.02755** LONGITUDE: **-117.28735** ELEVATION (ft): **67**

DRILL RIG: **Diedrich D-120HT** DRILL METHOD: **HSA** DRILLING COMPANY: **Tri-County** BOREHOLE DEPTH (ft): **75.5**

CASING TIP DEPTH: **NA** BIT DIAMETER: **8"** GROUNDWATER DATA: DEPTH: DEPTH:

HAMMER TYPE: **Automatic** HAMMER EFFICIENCY: **80%** NOT ENCOUNTERED ☒ TIME: TIME:

CHECKED BY (DATE): **JMS** EFFICIENCY MEASURED ☐ GW NOT MEASURED ☒ DATE: DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	0										Clayey SAND (SC); olive brown; moist; medium to fine SAND; trace coarse GRAVEL	Possible Fill. Rig chatter from 0' to 3'
65												
	5			3 3 3							Clayey SAND with GRAVEL (SC); Paralic Deposits (Qol); loose; brown; moist; coarse to fine SAND; angular to subangular coarse GRAVEL	Very little recovery
60												
	10			4 5 9				5.9			medium dense; fragments of GRAVEL in sampler	Very little recovery. Few COBBLES in soil cuttings
55												
	15			15 50/5"		18		20.3			SANDSTONE; Delmar Formation (Tsh); very soft to soft; olive brown; decomposed; (recovered as Silty SAND [SM]; very dense; moist; fine SAND)	
50												
	20			67/6"				23.0			gray; wet	Possible perched groundwater. Converted to mud rotary at 20.5' bgs
45												
	25			50/2"		40		24.8	PI		bluish gray; decomposed; some CLAY content; interlayered SANDSTONE and CLAYSTONE; intebedded highly cemented layers; (recovered as Clayey SAND [SC], very dense, moist, fine SAND)	Very little recovery
40												
	30											

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-001

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	30	☒		46 50/3"							micaceous SAND	
35												
	35	⧓		82/6"			110	18.7			greenish blue	
30												
	40	☒		13 24 29								
25												
	45	⧓		58/5"			125	11.3			green	
20												
	50	☒		18 32 43							bluish green	
15												
	55	⧓		50/4"			119	14.9				
10												
	60											
5												

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-001

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	65	☒		50/5"						▨	bluish gray	Very little recovery
0										▨		
	70									▨		
-5										▨		
	75	⧓		128/6"			120	13.2		▨		

Boring terminated at 75.5 feet bgs.
Groundwater not encountered during drilling.
Boring backfilled with cement-bentonite grout.

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-001

LOGGED BY: **MF**
DATE: START **3/29/18** END **3/29/18**
STATION & OFFSET: **76+50, 30 ft L**
LATITUDE: **33.02743**
LONGITUDE: **-117.28701**
ELEVATION (ft): **71**
DRILL RIG: **Diedrich D-120HT**
DRILL METHOD: **HSA**
DRILLING COMPANY: **Tri-County**
BOREHOLE DEPTH (ft): **100.3**
CASING TIP DEPTH: **NA**
BIT DIAMETER: **8"**
GROUNDWATER DATA:
DEPTH: ____ DEPTH: ____
HAMMER TYPE: **Automatic**
HAMMER EFFICIENCY: **80%**
NOT ENCOUNTERED ☒
TIME: ____ TIME: ____
CHECKED BY (DATE): **JMS**
EFFICIENCY MEASURED ☐
GW NOT MEASURED ☒
DATE: ____ DATE: ____

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	0										Silty SAND (SM); Fill (Qaf); dark brown; moist; medium to fine SAND; trace fine GRAVEL; trace roots	Bulk sample collected from 0' to 3'
70									MD			
		⧗		3 3 3					CR		loose	
	5	⧗		4 6 4			102	7.8	DS		Silty SAND (SM); Paralic Deposits (Qol); loose; moist; medium to fine SAND; trace fine GRAVEL	
65												
		⧗		14 27 36			122	11.5			Clayey SAND (SC); Paralic Deposits (Qol); dense; greenish gray; moist; medium to fine SAND; low plasticity; some calcification	Possibly decomposed bedrock
	10	⧖		9 15 17				16.2				Converted to mud rotary at 10' bgs
60												
	15	⧗		16 19 11			112	118.3	UU		SILTSTONE; Delmar Formation (Tsh); very soft to soft; gray; decomposed; (recovered as Sandy CLAY [CL]; very stiff; moist; medium to fine SAND; low plasticity CLAY)	
55												
	20	⧖		27 19 29		19		21.8	PI		SANDSTONE; Delmar Formation (Tsh); very soft to soft; olive brown; decomposed; (recovered as Silty SAND [SM]; very dense; moist; medium to fine SAND; interbedded CLAY; few oxidation)	
50												
	25	⧗		25 31 60/5"			108	20.8			reddish SAND	
45												
	30											

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-002

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	30			26 50/3"					CR		bluish gray; micaceous	
40												
	35			50/2"								Very little recovery
35												
	40			50/5"			99	32.7			increased SILT content; interbedded SILTSTONE and SANDSTONE	
30												
	45			15 31 34								
25												
	50			57/6"		41	122	11.3			reddish brown	
20												
	55											
15												
	60			50/6"							bluish gray	
10												

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-002

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	65											
5												
	70	⧖		24 35 42				20.5				
0												
	75											
-5												
	80	⧗		50/5"			120	16.1			reddish brown lenses	
-10												
	85											
-15												
	90	⧖		50/5"								
-20												
	95											
-25												

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-002

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	100	☒		50/3"							bluish gray	

Boring terminated at 100.3 feet bgs.
Groundwater not encountered during drilling.
Boring backfilled with cement-bentonite grout.

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-002

LOGGED BY: **MF**
DATE: START **3/30/18** END **3/30/18**
STATION & OFFSET: **77+40, 50 ft R**
LATITUDE: **33.02718**
LONGITUDE: **-117.28712**
ELEVATION (ft): **66**
DRILL RIG: **Diedrich D-120HT**
DRILL METHOD: **HSA**
DRILLING COMPANY: **Tri-County**
BOREHOLE DEPTH (ft): **75.67**
CASING TIP DEPTH: **NA**
BIT DIAMETER: **8"**
GROUNDWATER DATA: DEPTH: DEPTH:
HAMMER TYPE: **Automatic**
HAMMER EFFICIENCY: **80%**
NOT ENCOUNTERED ☒ TIME: TIME:
CHECKED BY (DATE): **JMS**
EFFICIENCY MEASURED ☐
GW NOT MEASURED ☒ DATE: DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
65	0	▨									Clayey SAND (SC); Fill (Qaf); brown; moist; medium to fine SAND; trace ballast	Bulk sample collected from 0' to 3'
60	5	☒		7 10 13		33					Clayey SAND (SC); Paralic Deposits (Qol); dense; brown; moist; medium to fine SAND;	Converted to mud rotary at 5' bgs
55	10	⧗		11 13 18		87	105	18.9	PI DS		CLAYSTONE; Delmar Formation (Tsh); very soft to soft; greenish gray; decomposed; (recovered as Fat CLAY [CH]; very stiff; moist; little fine SAND; high plasticity CLAY)	
50	15	☒		15 30 19				23.6			SANDSTONE; Delmar Formation (Tsh); very soft to soft; olive gray; decomposed; (recovered as silty SAND [SM]; very dense; moist; fine to medium SAND; signs of calcification)	
45	20	⧗		50/3"				24.0			micaceous SAND	
40	25	☒		31 50/4"							pale brown; fine SAND; trace sea shells	
	30											

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-003

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
35	30			50/4"			113	17.4			olive gray; medium to fine SAND	
30	35			11 20 27							greenish gray	
25	40			22 37 60			107	19.7	DS		CLAYSTONE; Delmar Formation (Tsh); very soft to soft; bluish gray; decomposed; (recovered as sandy lean CLAY [CL]; hard; moist; fine to medium SAND; low plasticity CLAY)	
20	45			23 28 41				22.3			greenish to bluish gray	
15	50			105/6"		30	123	12.6			SANDSTONE; Delmar Formation (Tsh); very soft to soft; bluish gray; decomposed; (recovered as silty SAND [SM]; very dense; moist; fine to medium SAND)	
10	55			50/5"				18.3				
5	60			55/6"			110	18.6				

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-003

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	65	☒		27 57						▨	bluish green; fine SAND	
0										▨		
	70									▨		
-5										▨		
	75	⧓		91 50/2"			117	15.7		▨	interbedded cemented SILSTONE	

Boring terminated at 75.75 feet bgs.
Groundwater not encountered during drilling.
Boring backfilled with cement-bentonite grout.

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-003

LOGGED BY: **MF**
DATE: START **3/29/18** END **3/29/18**
STATION & OFFSET: **78+50, 50 ft L**
LATITUDE: **33.02693**
LONGITUDE: **-117.28668**
ELEVATION (ft): **70**
DRILL RIG: **Diedrich D-120HT**
DRILL METHOD: **HSA**
DRILLING COMPANY: **Tri-County**
BOREHOLE DEPTH (ft): **51**
CASING TIP DEPTH: **NA**
BIT DIAMETER: **8"**
GROUNDWATER DATA:
DEPTH:
DEPTH:
HAMMER TYPE: **Automatic**
HAMMER EFFICIENCY: **80%**
NOT ENCOUNTERED ☒
TIME:
TIME:
CHECKED BY (DATE): **JMS**
EFFICIENCY MEASURED ☐
GW NOT MEASURED ☒
DATE:
DATE:

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	0	▨									Silty SAND (SM); Paralic Deposits (Qol); dark brown; moist; fine SAND	Bulk sample collected from 0' to 5'
65	5	☒		15 26 18				3.5			very dense; light brown; slightly cemented; some calcification	Possibly decomposed bedrock
60	10	⧓		9 16 20		20	108	11.9			Clayey SAND (SC); Paralic Deposits (Qol); medium dense; mottled black; red olive spots; moist; fine SAND; iron oxide staining; micaceous	
55	15	☒		8 14 18				21.7			CLAYSTONE; Delmar Formation (Tsh); very soft to soft; green; decomposed; (recovered as Sandy Lean CLAY [CL]; very stiff; moist; fine SAND; low plasticity CLAY)	
50	20	⧓		18 50/2"		23	114	14.1			SANDSTONE; Delmar Formation (Tsh); very soft to soft; pale brown; decomposed; (recovered as Silty SAND [SM]; very dense; moist; medium to fine SAND; low plasticity SILT; micaceous; trace calcification)	
45	25	☒		16 34 45							CLAYSTONE; Delmar Formation (Tsh); very soft to soft; greenish gray; decomposed; (recovered as Sandy Lean CLAY [CL]; hard; moist; fine SAND; low plasticity CLAY)	Trace sea shells
40	30											

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-004

ELEVATION (ft)	DEPTH (ft)	SAMPLER	SAMPLE NO./ CORE RUN	FIELD BLOWS/6 in	POCKET PEN (ksf)	% FINES	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS	MATERIAL GRAPHIC	DESCRIPTION	REMARKS
	30			50/4"								No recovery
35	35			47 50/3"				11.3			SANDSTONE; Delmar Formation (Tsh); very soft to soft; bluish gray; decomposed; (recovered as Clayey SAND [SC]; very dense; moist; fine SAND; high plasticity CLAY; trace mica)	
30	40			39 50/3"			100	19.2	PI			Increased moisture
25	45			10 41 50/3"								
20	50			21 50/6"			106	19.7			iron oxide staining	

Boring terminated at 51 feet bgs.
Groundwater not encountered during drilling.
Boring backfilled with cement-bentonite grout.

Boring Log

Verdi Avenue Undercrossing Project
San Diego County, California

Date
MAR 2018

Boring
A-18-004

Appendix C. Geotechnical Laboratory Test Results

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TABLE C-1
SUMMARY OF SOIL LABORATORY DATA (Imperial Units)

Project: Verdi Avuene Undercrossing
Project No.: 10027160

Sheet 1 of 2

														Direct Shear Strength				UU		Unconfined							
						Gradation			Compaction		Atterberg Limits			Peak		Ultimate		Triaxial Test		Compression	Consolidation		R- Value	Chemical Analyses			
Boring No.	Sample Depth (ft)	Soil Type (USCS)	Sample Elev. (ft)	Moisture Content (%)	Dry Density (pcf)	Gravel (%)	Sand (%)	Fines	Max. Dry Density (pcf)	Optimum Moisture Content (%)	LL	PL	PI	ϕ' (deg)	c' (psf)	ϕ' (deg)	c' (psf)	Maximum Deviator Stress (ksf)	Axial Strain (%)	S_u (ksf)	Collapse (%)	Collapse Pressure (ksf)		pH	Resistivity (Ω-cm)	Sulfate (ppm)	Chloride (ppm)
A-18-001	10.0	SC	57	5.9																							
A-18-001	15.0	SM	52	20.3		0	82	18																			
A-18-001	20.0	SM	47	23.0																							
A-18-001	25.0	SC	42	24.8		0	60	40			33	22	11														
A-18-001	35.0	SC	32	18.7	110.1																						
A-18-001	45.0	ML	22	11.3	124.5																						
A-18-001	55.0	SM	12	14.9	118.9																						
A-18-001	75.0	SM	-8	13.2	119.9																						
A-18-002	0-3	SM	70						129.5	8.6																	
A-18-002	2.0	SM	69																					8.1	1,200	130	122
A-18-002	5.0	SM	66											30	150	30	100										
A-18-002	8.0	SC	63	11.5	121.9																						
A-18-002	10.0	SC	61	16.2																							
A-18-002	15.0	CL	56															10.6	4.8								
A-18-002	20.0	SM	51	21.8		1	80	19			NP	NP	NP														
A-18-002	25.0	SM	46	20.8	107.7	0	59	41																			
A-18-002	30.0	SM	41																					7.4	480	2,530	49
A-18-002	40.0	SM	31	32.7	98.6																						
A-18-002	50.0	SM	21	11.3	122.3																						
A-18-002	70.0	SM	1	20.5																							
A-18-002	80.0	SM	-9	16.1	119.8																						

TABLE C-1

SUMMARY OF SOIL LABORATORY DATA (Imperial Units)

Project: Verdi Avuene Undercrossing

Project No.: 10027160

Sheet 2 of 2

														Direct Shear Strength				UU		Unconfined							
						Gradation			Compaction		Atterberg Limits			Peak		Ultimate		Triaxial Test		Compression	Consolidation			Chemical Analyses			
Boring No.	Sample Depth (ft)	Soil Type (USCS)	Sample Elev. (ft)	Moisture Content (%)	Dry Density (pcf)	Gravel (%)	Sand (%)	Fines	Max. Dry Density (pcf)	Optimum Moisture Content (%)	LL	PL	PI	ϕ' (deg)	c' (psf)	ϕ' (deg)	c' (psf)	Maximum Deviator Stress (ksf)	Axial Strain (%)	S_u (ksf)	Collapse (%)	Collapse Pressure (ksf)	R- Value	pH	Resistivity (Ω-cm)	Sulfate (ppm)	Chloride (ppm)
A-18-003	5.0	SC	61			0	67	33																			
A-18-003	10.0	CH	56			0	13	87			58	21	37	26	700	26	300										
A-18-003	15.0	SC	51	23.6														10.6	4.8								
A-18-003	20.0	SM	46	24.0																							
A-18-003	30.0	SM	36	17.4	112.6																						
A-18-003	40.0	CL	26											26	2050	25	200										
A-18-003	45.0	SM	21	22.3																							
A-18-003	50.0	SM	16	12.6	123.0	0	70	30																			
A-18-003	55.0	ML	11	18.3																							
A-18-003	60.0	ML	6	18.6	110.0																						
A-18-003	75.0	ML	-9	15.7	116.8																						
A-18-004	5.0	SM	65	3.5																							
A-18-004	10.0	SM	60	11.9	107.8	0	80	20																			
A-18-004	15.0	CL	55	21.7																							
A-18-004	20.0	SM	50	14.1	114.0	0	77	23																			
A-18-004	35.0	SC	35	11.3																							
A-18-004	40.0	SC	30	19.2	99.5						56	25	31														
A-18-004	50.0	SC	20	19.7	106.2																						

Notes: NP denotes "Non Plastic"

The laboratory tests were performed in general accordance with the following standards:

Dry Density Test - ASTM Test Method D2937

Moisture Content Test - ASTM Test Method D2216

No. 200 Wash Test - ASTM Test Method D1140

Compaction Test - ASTM Test Method D1557

Resistance R-Value and Expansion Pressure - Cal Test 301

Grain Size Analysis and Hydrometer - ASTM Test Method D422

Direct Shear Test - ASTM Test Method D3080

One-Dimensional Consolidation Test - ASTM Test Method D2435

Atterberg Limits Test - ASTM Test Method D4318

Corrosivity Tests - DOT CA 532/643 - pH, DOT CA 417 - soluble sulfates, DOT CA 422 - chlorides, DOT CA 643 - minimum resistivity

Consolidated Undrained (CU) Triaxial Test - ASTM Test Method D4767

Unconsolidated Undrained (UU) Triaxial Test - ASTM Test Method D2850

Unconfined Compression Test - ASTM Test Method D2166

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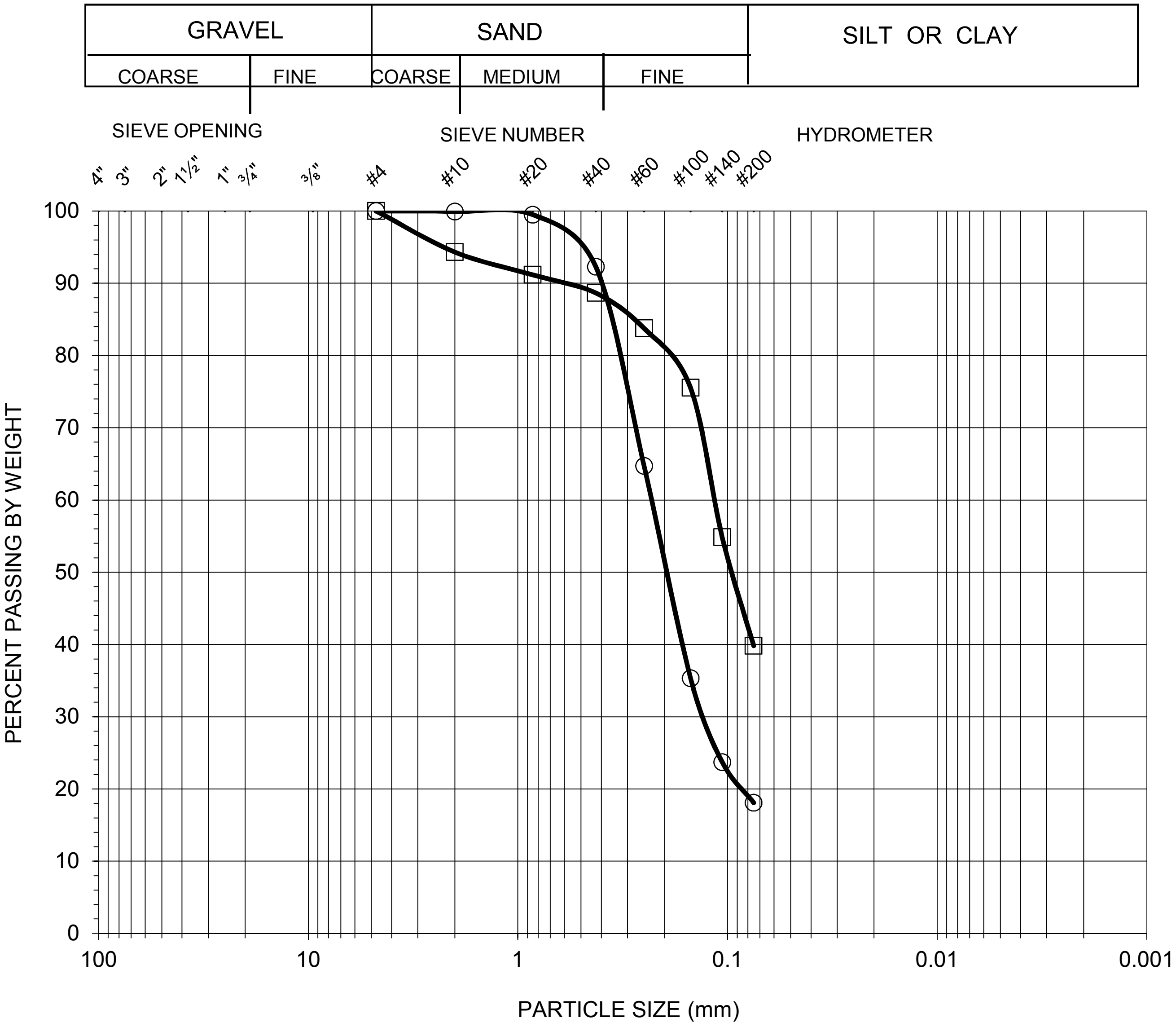
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	04/16/18
Project Name:	Verdi Ave UC	Computed by:	JP	Date:	04/17/18
Project Number:	10027160	Checked by:	AP	Date:	04/17/18



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-18-001	2	15	0	82	18	N/A	SM
□	A-18-001	4	25	0	60	40	33:22:11	SC

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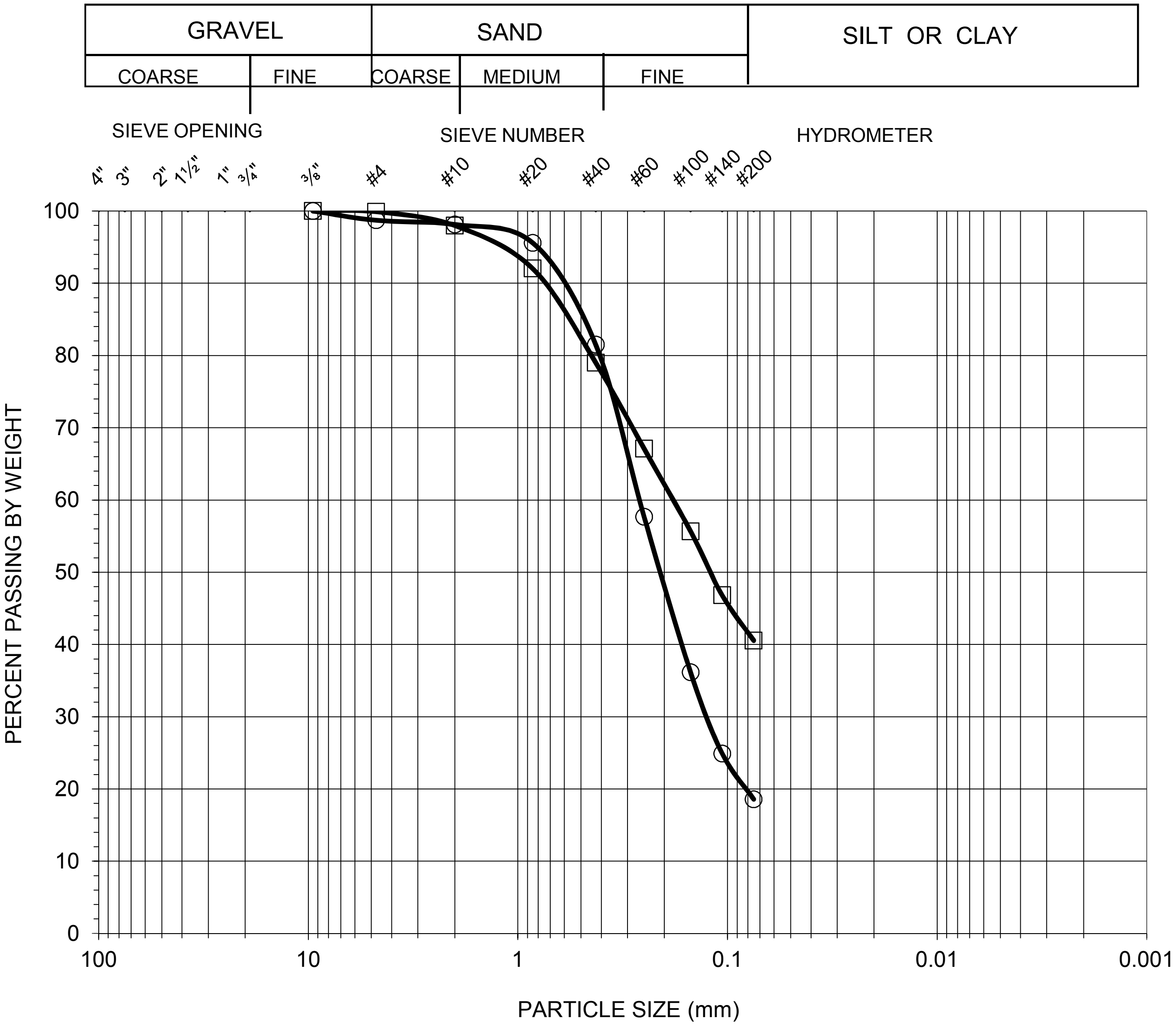
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name: HDR
Project Name: Verdi Ave UC
Project Number: 10027160

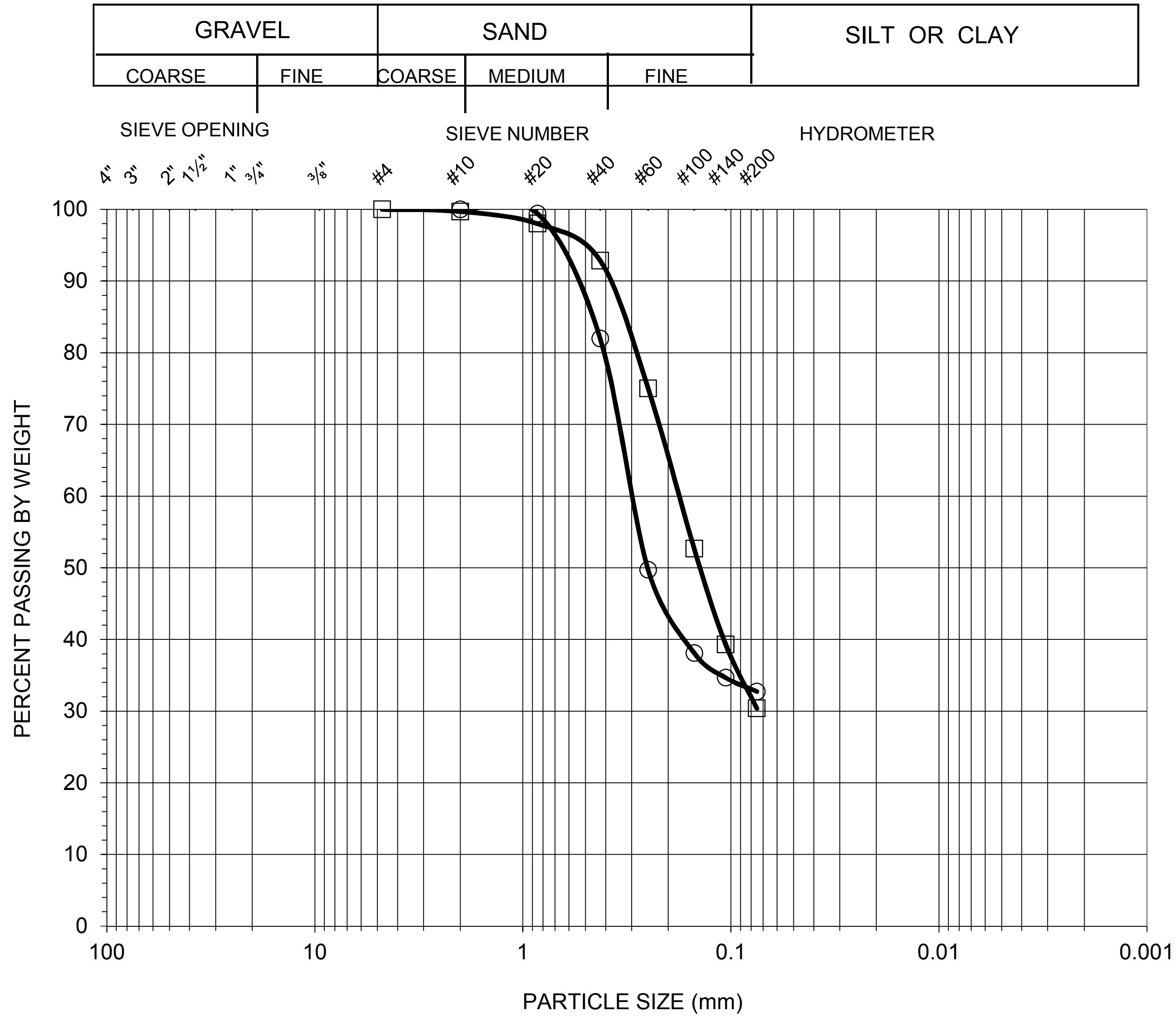
Tested by: NG Date: 04/16/18
Computed by: JP Date: 04/17/18
Checked by: AP Date: 04/17/18



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-18-002	3	20	1	80	19	N/P	SM
□	A-18-002	9	50	0	59	41	N/A	SM

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	04/16/18
Project Name:	Verdi Ave UC	Computed by:	JP	Date:	04/17/18
Project Number:	10027160	Checked by:	AP	Date:	04/17/18



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-18-003	1	5	0	67	33	N/A	SC*
□	A-18-003	10	50	0	70	30	N/A	SM

**Note: Based on visual classification of sample*

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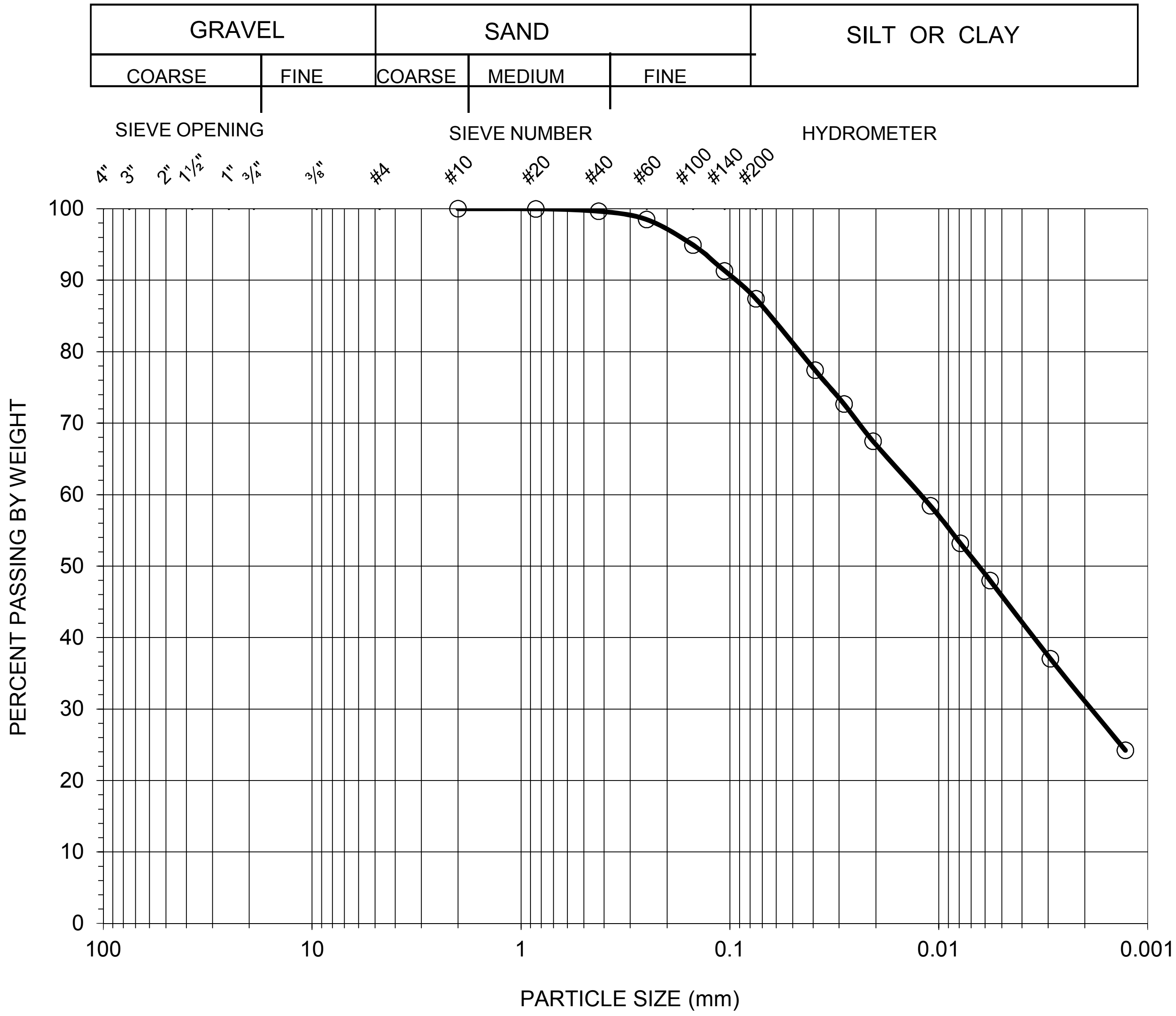
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913 & D 7928

Client Name: HDR
Project Name: Verdi Ave UC
Project Number: 10027160
Tested by: NG Date: 04/16/18
Computed by: JP Date: 04/17/18
Checked by: AP Date: 04/17/18



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-18-003	2	10	0	13	87	58:21:37	CH

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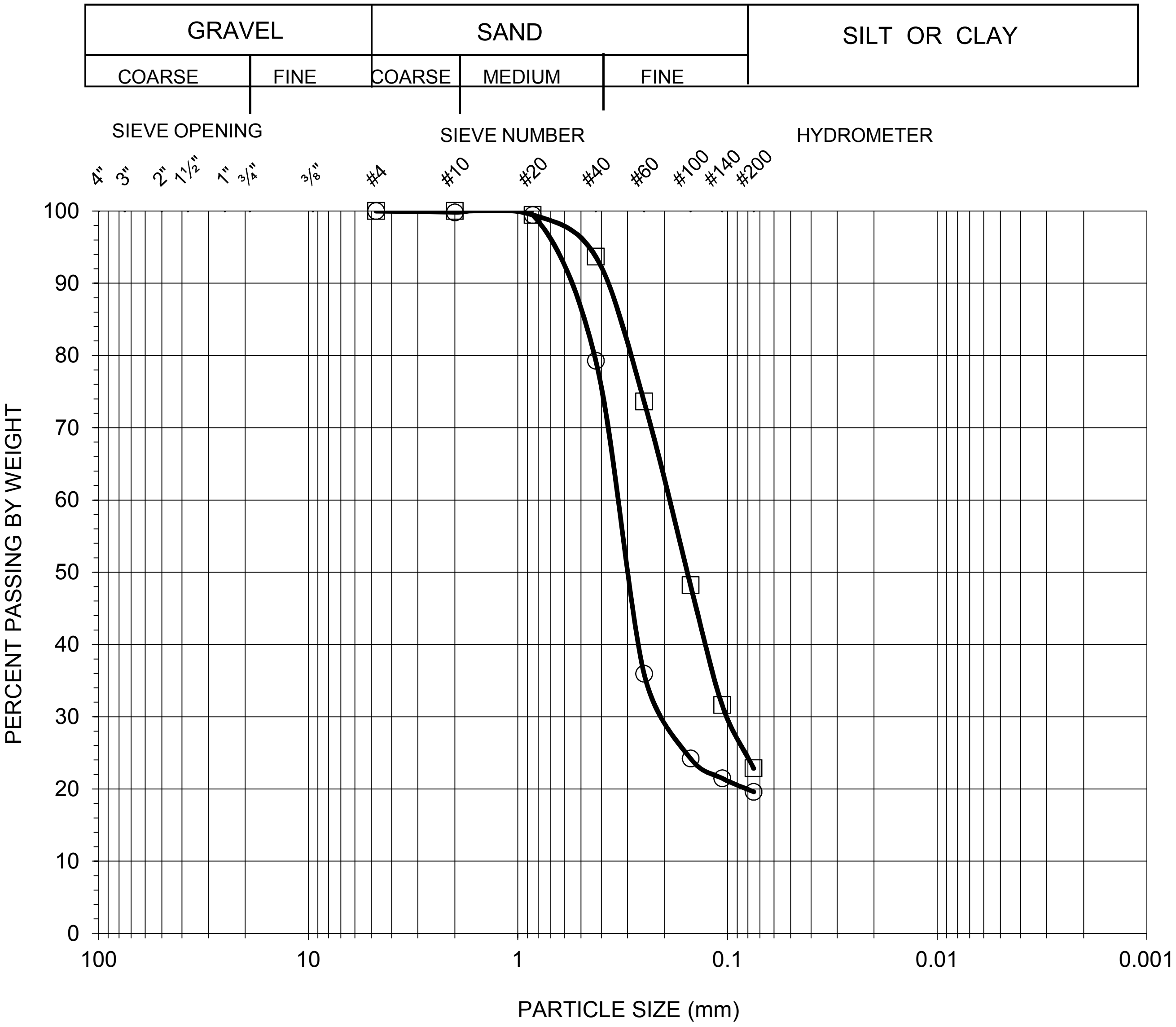
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name: HDR
Project Name: Verdi Ave UC
Project Number: 10027160

Tested by: NG Date: 04/16/18
Computed by: JP Date: 04/17/18
Checked by: AP Date: 04/17/18



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-18-004	2	10	0	80	20	N/A	SC*
□	A-18-004	4	20	0	77	23	N/A	SM

**Note: Based on visual classification of sample*

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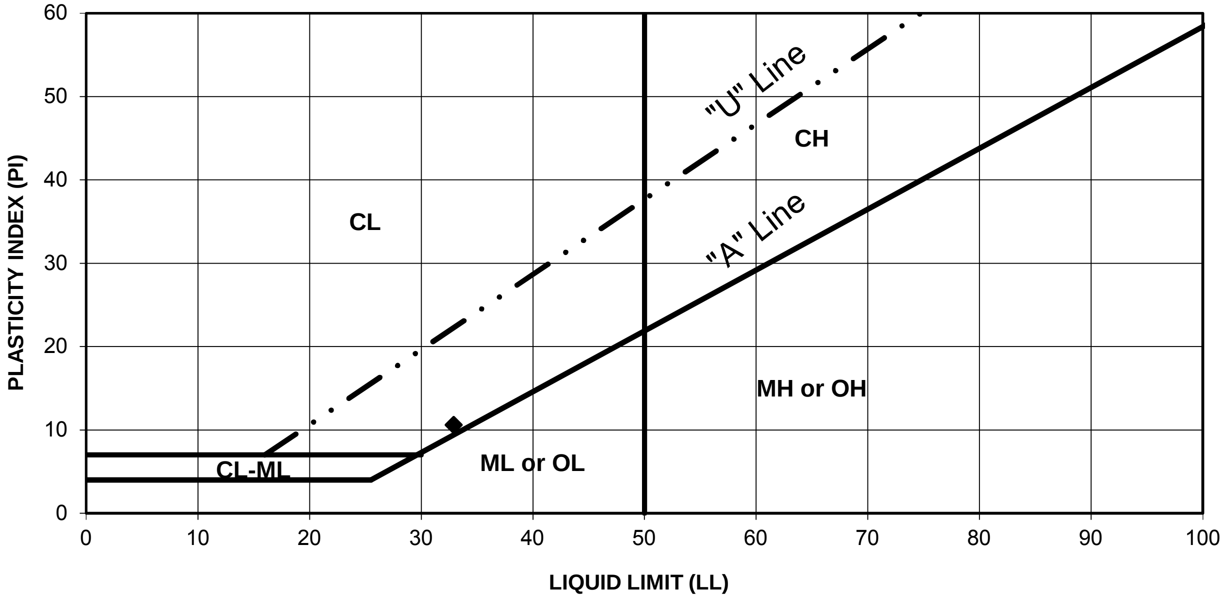
ATTERBERG LIMITS
ASTM D 4318

Project Name: Verdi Ave UC **Tested By:** LS Date: 04/16/18
Project No.: 10027160 **Checked By:** AP Date: 04/17/18



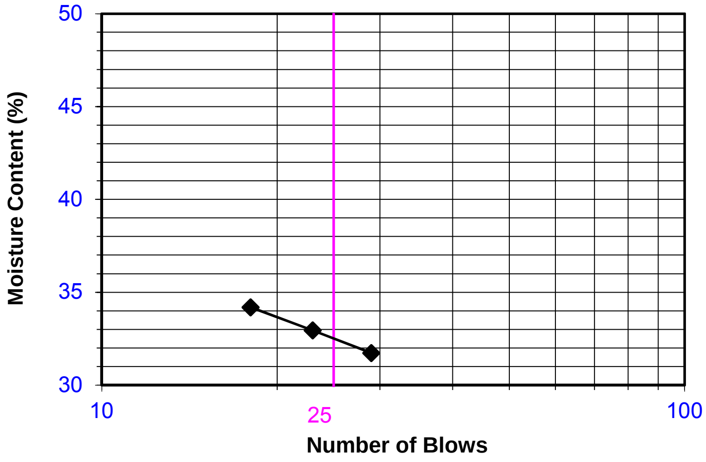
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

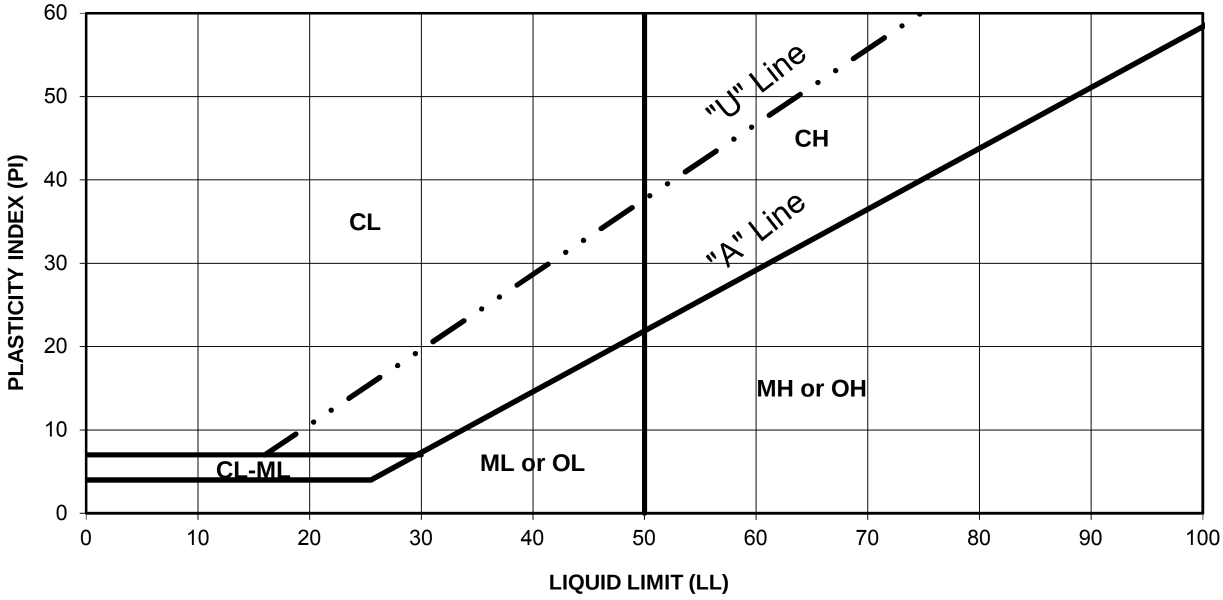
☐ Procedure B
One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
◆	A-18-001	4	25	33	22	11	CL

ATTERBERG LIMITS
ASTM D 4318

Project Name: Verdi Ave UC **Tested By:** LS Date: 04/16/18
Project No.: 10027160 **Checked By:** AP Date: 04/17/18



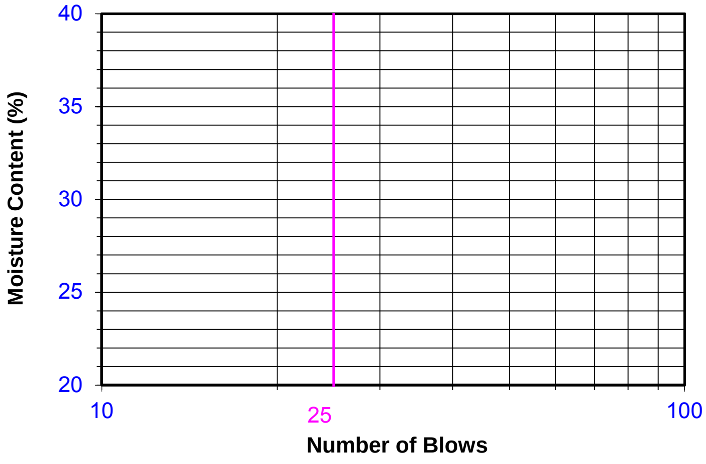
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

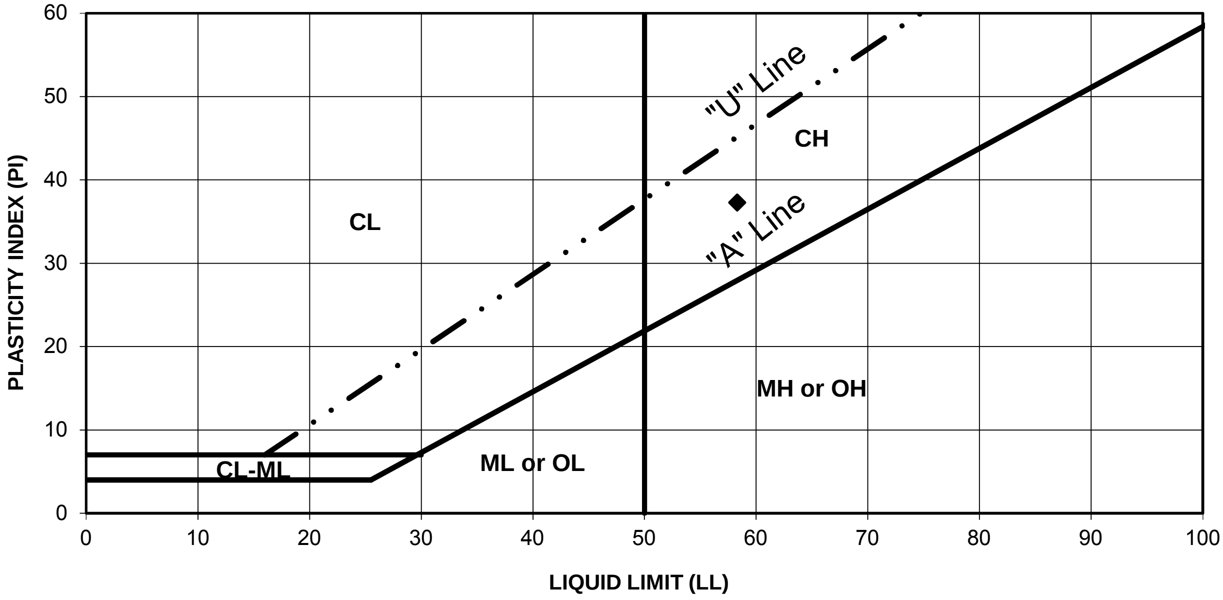


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	A-18-002	3	20	NP	NP	NP	

* NP denotes "non-plastic"

ATTERBERG LIMITS
ASTM D 4318

Project Name: Verdi Ave UC **Tested By:** LS Date: 04/16/18
Project No.: 10027160 **Checked By:** AP Date: 04/17/18



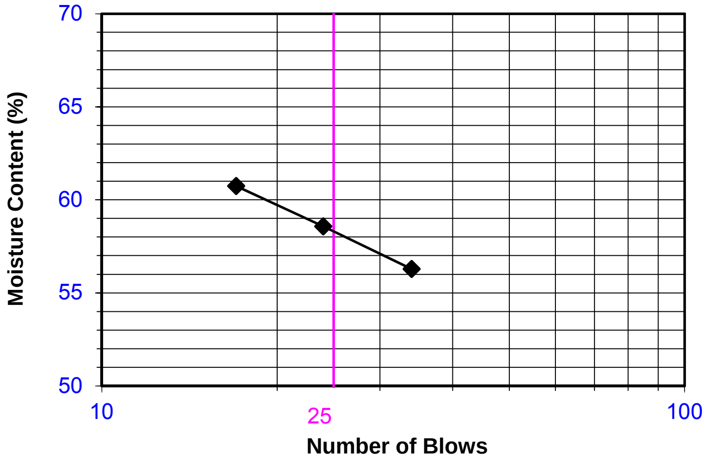
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test

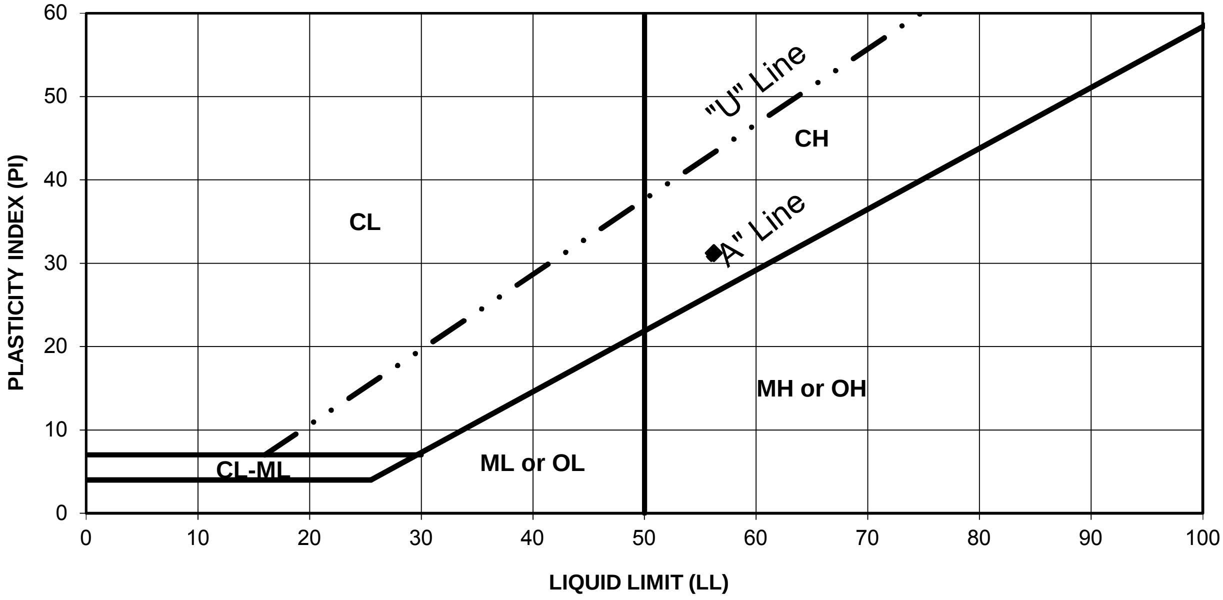


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	A-18-003	2	10	58	21	37	CH

ATTERBERG LIMITS
ASTM D 4318

Project Name: Verdi Ave UC
Tested By: LS
Date: 04/16/18

Project No.: 10027160
Checked By: AP
Date: 04/17/18



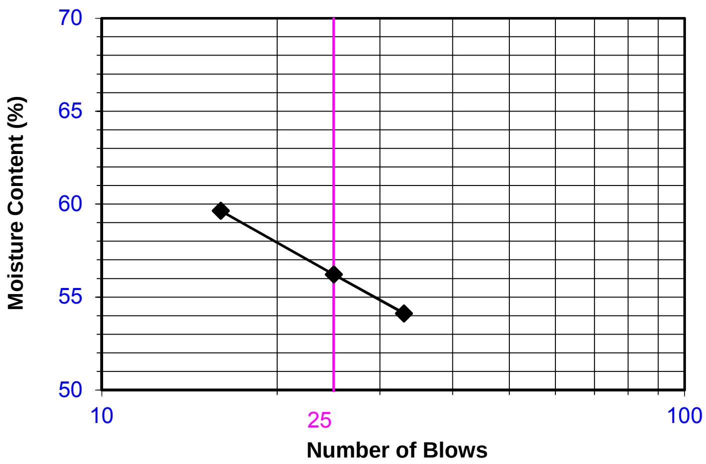
PROCEDURE USED

☐ Wet Preparation

☒ Dry Preparation

☒ Procedure A
Multipoint Test

☐ Procedure B
One-point Test



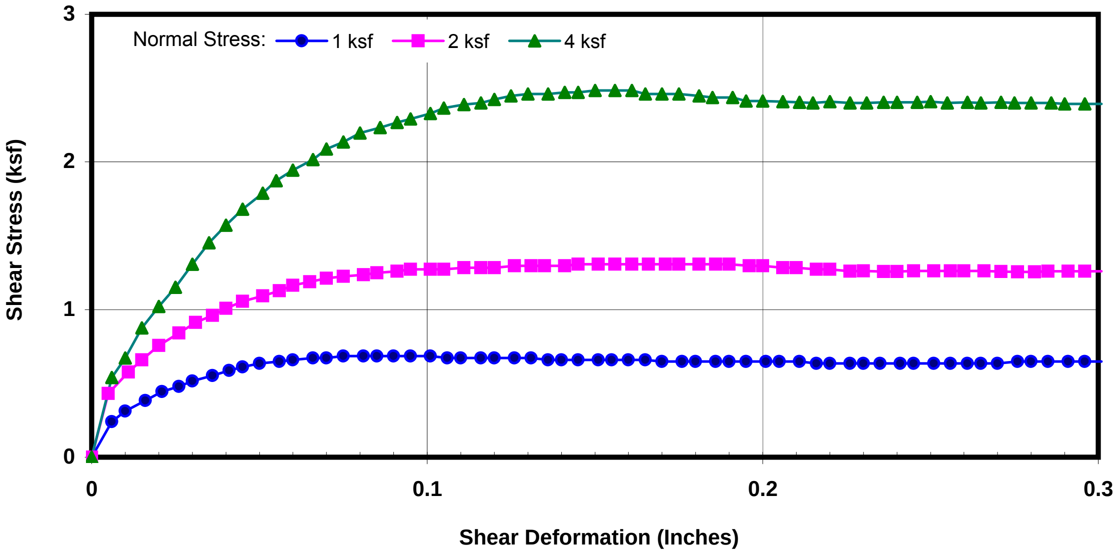
Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	A-18-004	7	40	56	25	31	CH

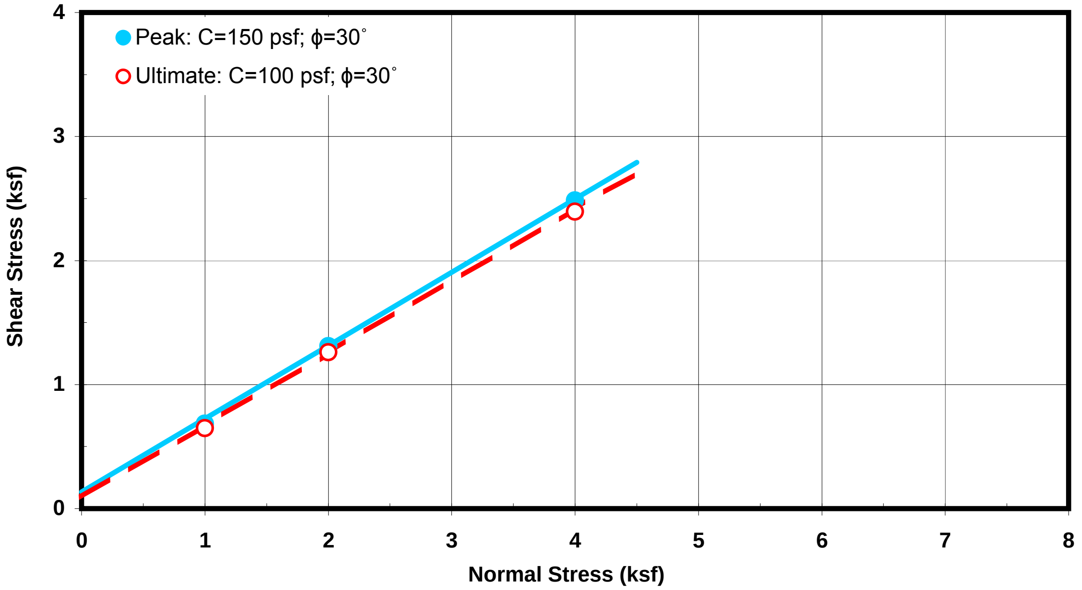
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Verdi Ave UC
Project No.: 10027160
Boring No.: A-18-002
Sample No.: 2 **Depth (ft):** 5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 04/16/18
Computed By: JP **Date:** 04/17/18
Checked by: AP **Date:** 04/17/18

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
110.2	102.2	7.8	21.9	33	91	1	0.684	0.648
						2	1.308	1.260
						4	2.484	2.394





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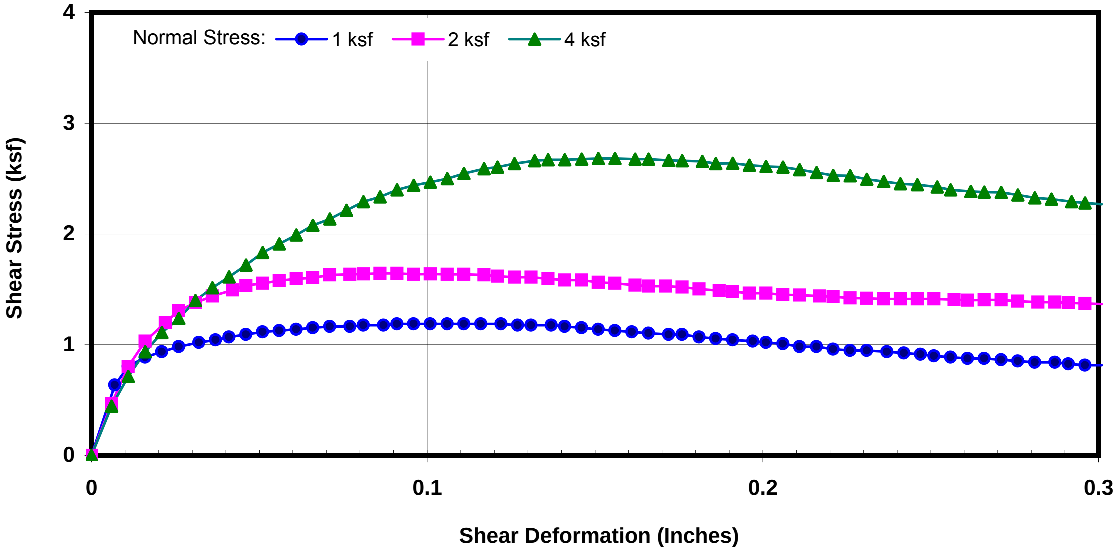
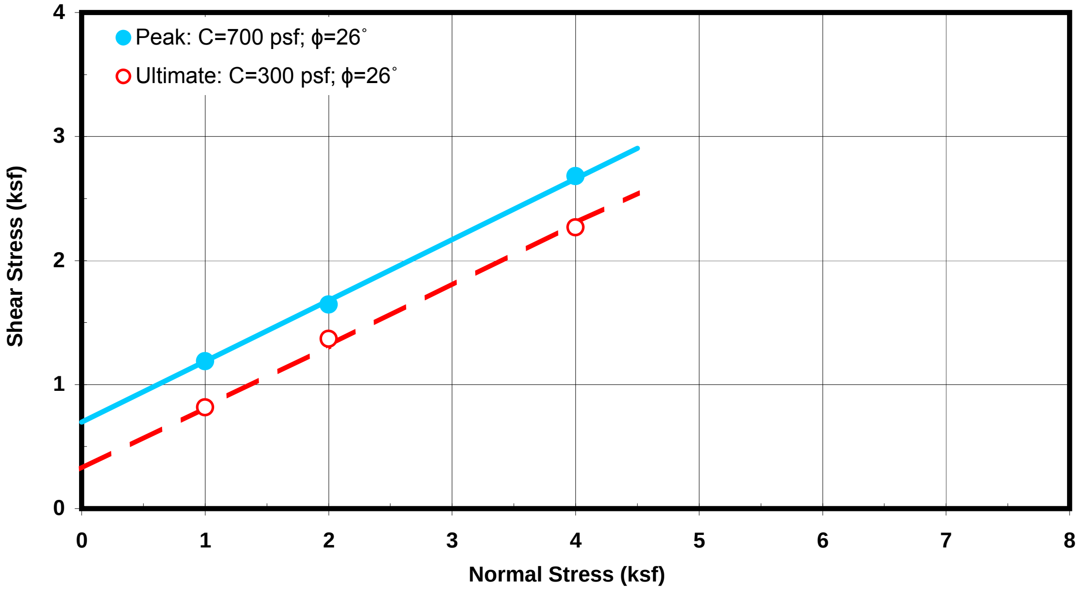
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Verdi Ave UC
Project No.: 10027160
Boring No.: A-18-003
Sample No.: 2 **Depth (ft):** 10
Sample Type: Mod. Cal.
Soil Description: Fat Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 04/13/18
Computed By: JP **Date:** 04/17/18
Checked by: AP **Date:** 04/17/18

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
125.3	**105.3**	**18.9**	**22.1**	**85**	**100**	**1**	**1.188**	**0.816**
						2	**1.645**	**1.368**
						4	**2.681**	**2.268**



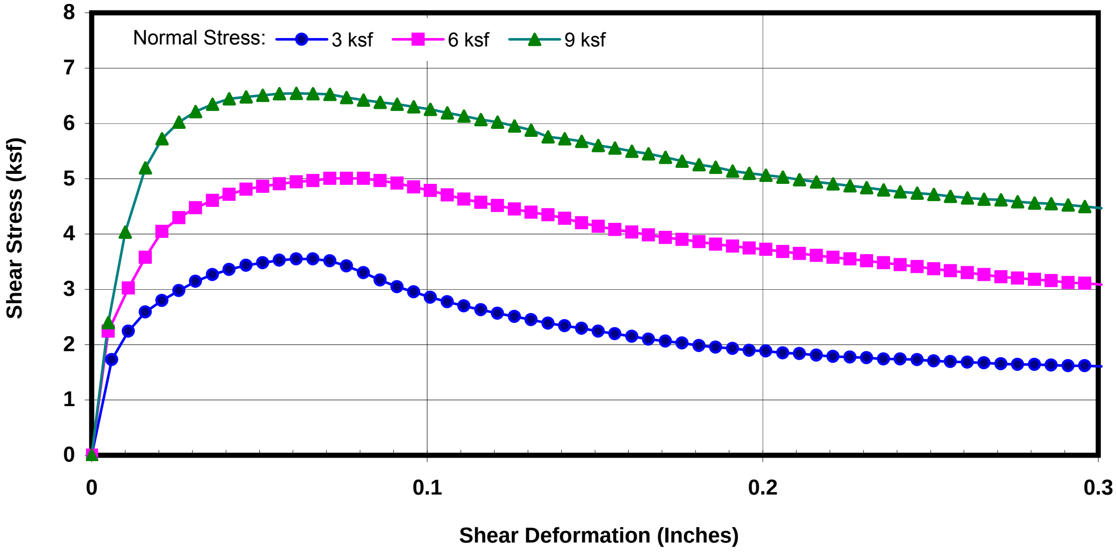


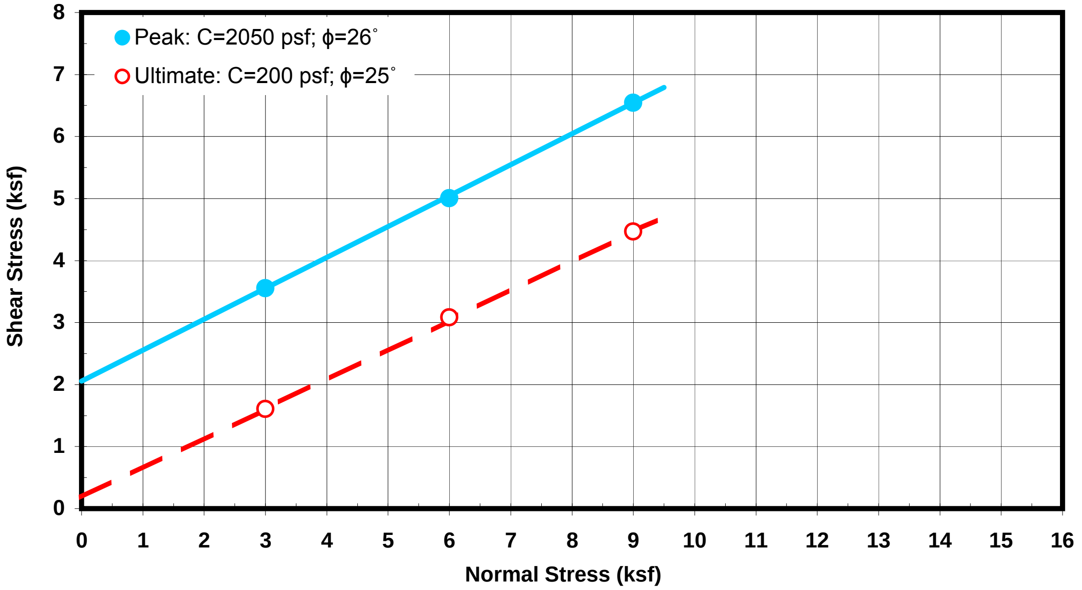
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Verdi Ave UC
Project No.: 10027160
Boring No.: A-18-003
Sample No.: 8 **Depth (ft):** 40
Sample Type: Mod. Cal.
Soil Description: Clay w/sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 04/16/18
Computed By: JP **Date:** 04/17/18
Checked by: AP **Date:** 04/17/18

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
127.8	106.8	19.7	21.4	92	100	3	3.552	1.608
						6	5.004	3.084
						9	6.545	4.466





UNCONSOLIDATED UNDRAINED TRIAXIAL TEST (UU,Q)
ASTM D 2850

Client Name: HDR
Project Name: Verdi Ave UC
Project No.: 10027160
Boring No.: A-18-002
Sample No.: 2 Depth (feet): 15
Soil Description Sandy Clay

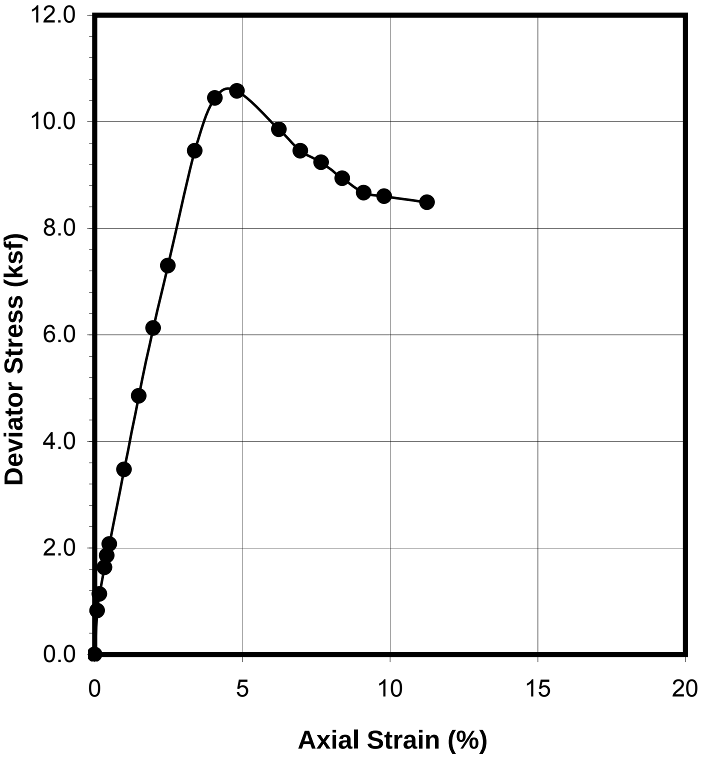
Tested By: ST Date: 04/16/18
Checked by: AP Date: 04/17/18
Sample Type: Mod. Cal.

Sample Diameter (inch): 2.410
Sample Height (inch): 6.038
Sample Weight (g): 954.20
Wt. of Wet Soil+Container (g): 370.58
Wt. of Dry Soil+Container (g): 336.35
Wt. of Container (g): 149.64

Wet Unit Weight (pcf): 131.9
Dry Unit Weight (pcf): 111.5
Moisture Content (%): 18.3
Void Ratio for Gs=2.7: 0.51
% Saturation: 96.8

TEST DATA

Cell Pressure (ksf): 1.20
Back Pressure (ksf): 0.0
Tested Total Confining Pressure (ksf): 1.20
Shear Rate (%/min): 0.3
Maximum Deviator Stress (ksf): 10.58
Ultimate Deviator Stress (ksf): 8.49
Ultimate Undrained Shear Strength (ksf): 4.24
Axial Strain @ Maximum Stress (%) 4.82



Load (lbs)	Def. (inch)	Area (sq.in)	Deviator Stress (ksf)	Axial Strain (%)
0	0.000	4.56	0.00	0.00
26	0.005	4.57	0.82	0.08
36	0.010	4.57	1.13	0.17
52	0.020	4.58	1.64	0.33
59	0.025	4.58	1.85	0.41
66	0.030	4.58	2.07	0.50
111	0.060	4.61	3.47	0.99
156	0.090	4.63	4.85	1.49
198	0.120	4.65	6.13	1.99
237	0.150	4.68	7.30	2.48
310	0.205	4.72	9.45	3.39
345	0.246	4.76	10.45	4.08
352	0.291	4.79	10.58	4.82
333	0.377	4.87	9.86	6.25
322	0.421	4.90	9.46	6.97
317	0.463	4.94	9.24	7.67
309	0.506	4.98	8.94	8.39
302	0.550	5.02	8.66	9.11
302	0.592	5.06	8.60	9.81
303	0.680	5.14	8.49	11.26

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COMPACTION TEST

Client: HDR
AP Number: 18-0416
Project Name: Verdi Ave UC
Tested By: LS
Date: 04/16/18
Project No. : 10027160
Calculated By: JP
Date: 04/17/18
Boring No.: A-18-002
Checked By: AP
Date: 04/17/18
Sample No.: B
Depth(ft.): 0-3
Visual Sample Description: Silty Sand

Compaction Method ☒ ASTM D1557
☐ ASTM D698

METHOD: A
MOLD VOLUME (CU.FT): 0.0333

Preparation Method ☐ Moist
☒ Dry

Wt. Comp. Soil + Mold (gm.)	3954	3953	3853	3896		
Wt. of Mold (gm.)	1848	1848	1848	1848		
Net Wt. of Soil (gm.)	2106	2105	2005	2048		
Container No.						
Wt. of Container (gm.)	149.74	175.53	208.71	236.96		
Wet Wt. of Soil + Cont. (gm.)	362.91	428.77	429.60	473.78		
Dry Wt. of Soil + Cont. (gm.)	347.36	405.11	417.05	447.93		
Moisture Content (%)	7.87	10.31	6.02	12.25		
Wet Density (pcf)	139.29	139.19	132.57	135.45		
Dry Density (pcf)	129.13	126.18	125.04	120.66		

Maximum Dry Density (pcf): **129.5**
Maximum Dry Density w/ Rock Correction (pcf): **N/A**
Optimum Moisture Content (%): **8.6**
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

☒ *METHOD A: Percent of Oversize:* *0.2%*
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

☐ *METHOD B: Percent of Oversize:* *N/A*
Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)

☐ *METHOD C: Percent of Oversize:* *N/A*
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)

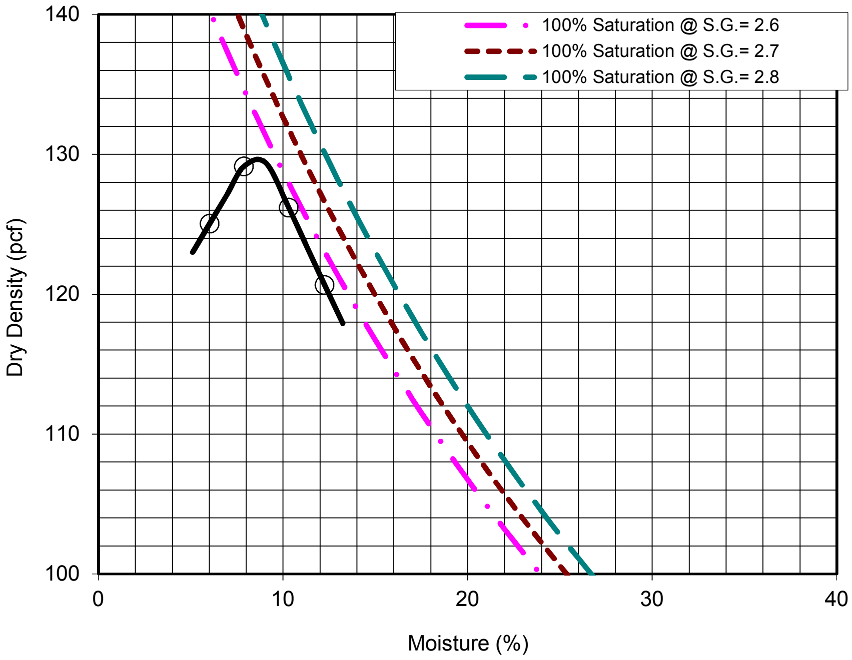

Table 1 - Laboratory Tests on Soil Samples

HDR, Irvine
Verdi-Montgomery Avenue Vicinity Pedestrian Rail Undercrossing
Your #10027160, HDR Lab #18-0237LAB
9-Apr-18

Sample ID			A-18-002 @ 2'	A-18-002 @ 30'
Resistivity		**Units**		
as-received		ohm-cm	6,800	1,000
minimum		ohm-cm	1,200	480
pH			8.1	7.4
Electrical Conductivity		mS/cm	0.18	0.97
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	7.1	742
magnesium	Mg^{2+}	mg/kg	12	54
sodium	Na^{1+}	mg/kg	160	125
potassium	K^{1+}	mg/kg	39	27
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	98	46
fluoride	F^{1-}	mg/kg	ND	ND
chloride	Cl^{1-}	mg/kg	122	49
sulfate	SO_4^{2-}	mg/kg	130	2,530
phosphate	PO_4^{3-}	mg/kg	3.0	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	15	8.6
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Minimum resistivity per CTM 643, Chlorides per CTM 422, Sulfates per CTM 417

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

Appendix D. Existing Geotechnical Data by Others

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Explanation of Test Pit, Core, Trench and Hand Auger Log Symbols

PROJECT NO.	DATE

DEPTH (FEET)

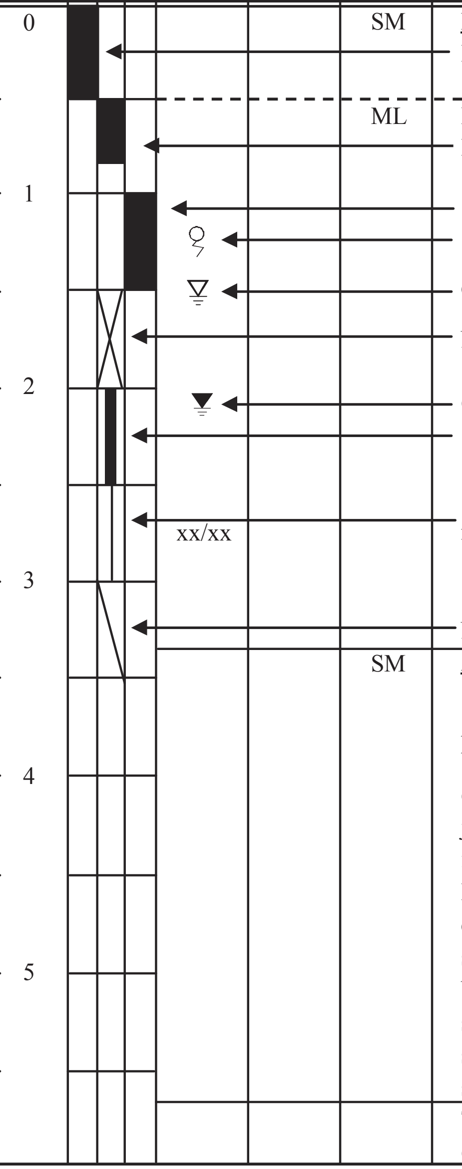
SAMPLES: Bulk, Driven, Sand Cone

MOISTURE (%)

DRY DENSITY (PCF)

CLASSIFICATION U.S.C.S.

EXCAVATION LOG EXPLANATION SHEET



<u>FILL</u>:

Bulk sample.

Dashed line denotes material change.

Drive sample.

Sand cone performed.

Seepage

Groundwater encountered during excavation.

No recovery with drive sampler.

Groundwater encountered after excavation.

Sample retained by others.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches

No recovery with Shelby tube sampler.

<u>ALLUVIUM</u>:

Solid line denotes unit change.

Attitude: Strike/Dip

b: Bedding
c: Contact
j: Joint
f: Fracture
F: Fault
cs: Clay Seam
s: Shear
bss: Basal Slide Surface
sf: Shear Fracture
sz: Shear Zone
sbs: Sheared Bedding Surface

The total depth line is a solid line that is drawn at the bottom of the excavation log.

SCALE: 1 inch = 1 foot

FIGURE

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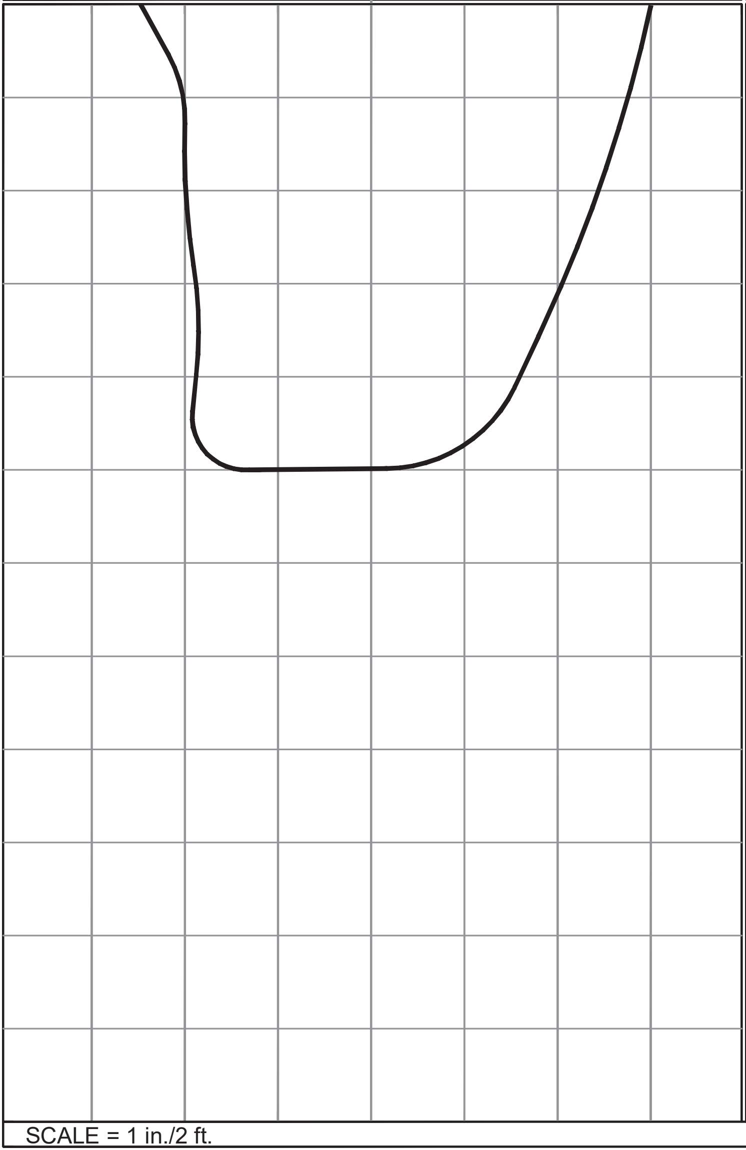
TEST PIT LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE
105991023	2/16

DATE EXCAVATED 10/26/11 TEST PIT NO. TP-1
GROUND ELEVATION 65'± (MSL) LOGGED BY BTM
METHOD OF EXCAVATION 328 Bobcat Mini Excavator
LOCATION See Figure 2



SCALE = 1 in./2 ft.

DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM	<u>OLD PARALIC DEPOSITS</u>: Brown to gray, moist, medium dense, silty fine SAND.
2	■						
4							
							Total Depth = 5 feet. Groundwater not encountered. Backfilled with soil on 10/26/11. <u>Note:</u> Groundwater, though not encountered at the time of excavation, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
6							
8							
10							
12							

FIGURE A-49

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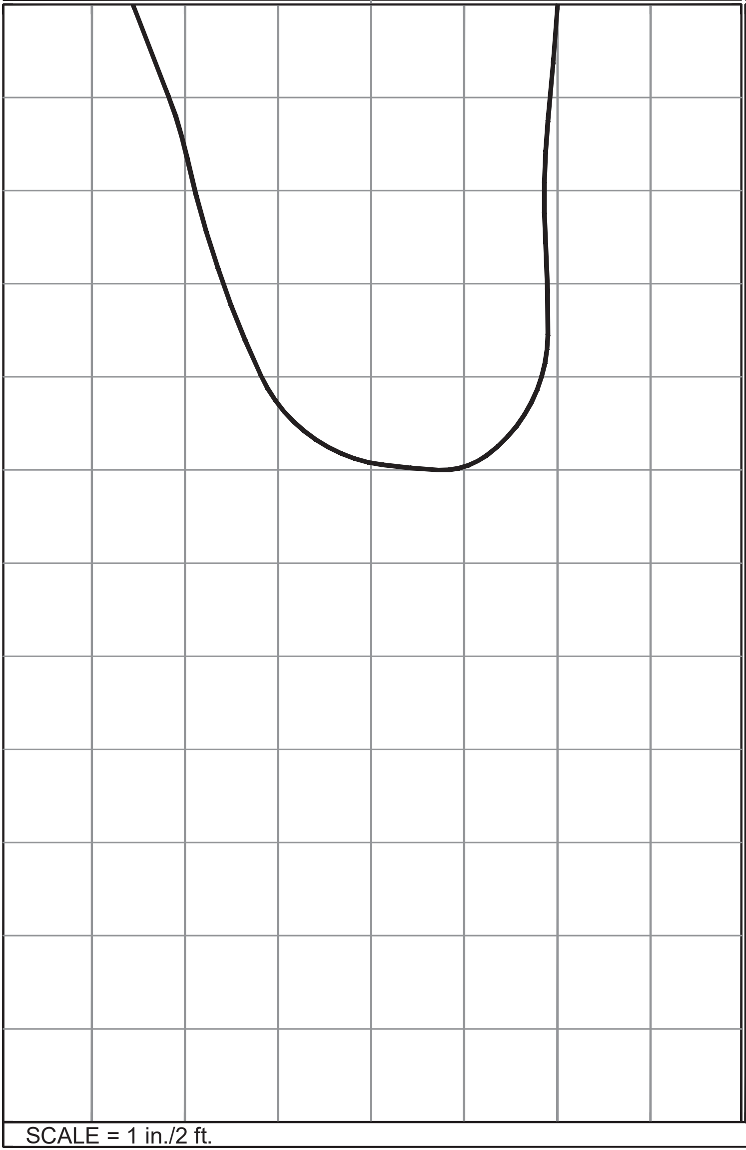
TEST PIT LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE
105991023	2/16

DATE EXCAVATED 10/26/11 TEST PIT NO. TP-2
GROUND ELEVATION 70'± (MSL) LOGGED BY BTM
METHOD OF EXCAVATION 328 Bobcat Mini Excavator
LOCATION See Figure 2



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SM	<u>OLD PARALIC DEPOSITS</u>: Brown to gray, moist, medium dense, silty fine SAND.
2							
4	■						Difficulty excavating; concretions.
							Total Depth = 5 feet. Groundwater not encountered. Backfilled with soil on 10/26/11. <u>Note:</u> Groundwater, though not encountered at the time of excavation, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
6							
8							
10							
12							

SCALE = 1 in./2 ft.

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TEST PIT LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

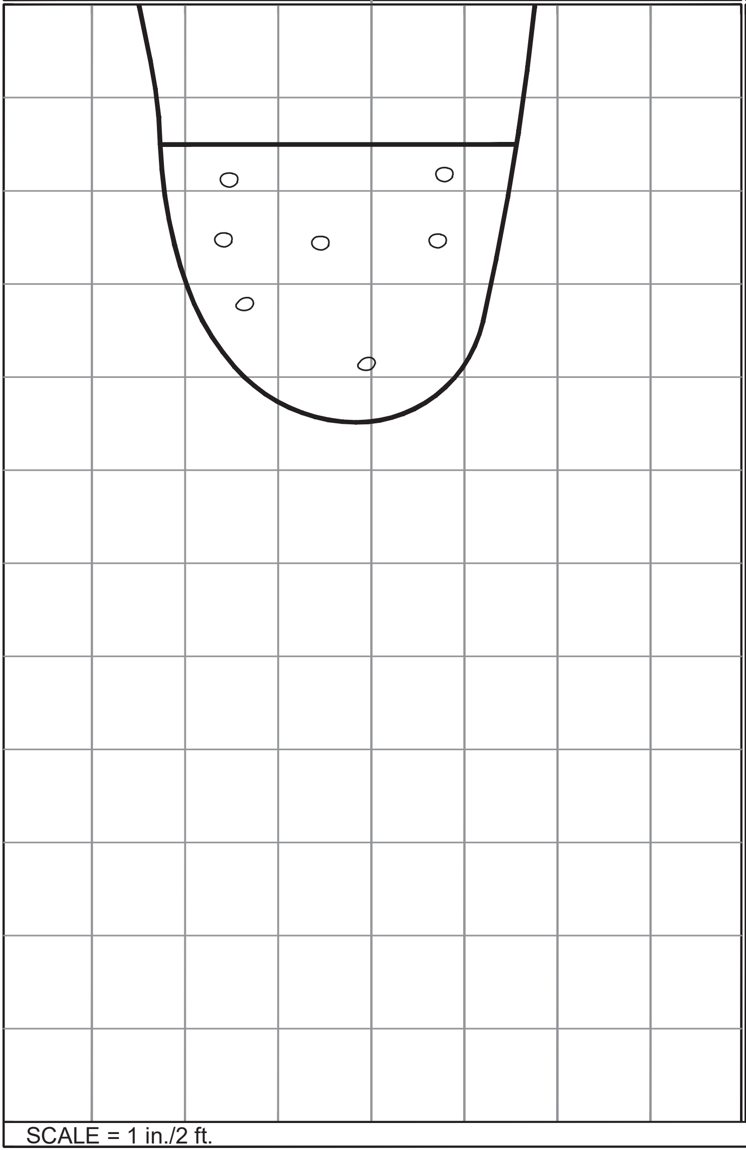
PROJECT NO.	DATE
105991023	2/16

DATE EXCAVATED 9/12/11 TEST PIT NO. TP-3

GROUND ELEVATION 75'± (MSL) LOGGED BY MBG

METHOD OF EXCAVATION Manual

LOCATION See Figure 2



SCALE = 1 in./2 ft.

DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SM	<u>FILL:</u> Light brown, dry to damp, loose, silty fine SAND; trace debris.
2	■	■		2.1	102.1	SM	<u>OLD PARALIC DEPOSITS</u>: Yellowish reddish brown, moist, medium dense, silty fine SAND; rounded gravel and cobbles up to 4 inches in diameter; scattered roots.
4	■						Moist to wet.
							Total Depth = 4.5 feet. Groundwater not encountered. Backfilled with soil on 9/12/11. <u>Note:</u> Groundwater, though not encountered at the time of excavation, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
6							
8							
10							
12							

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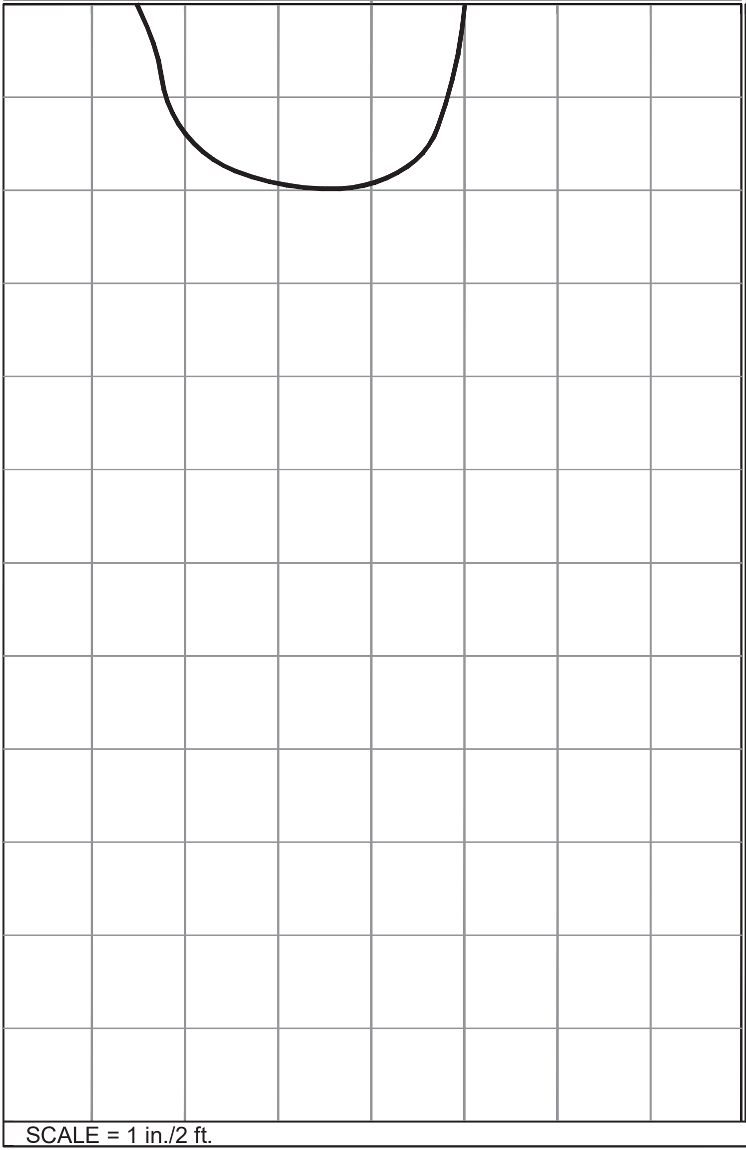
TEST PIT LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE
105991023	2/16

DATE EXCAVATED 10/07/11 TEST PIT NO. TP-4
GROUND ELEVATION 55'± (MSL) LOGGED BY MAC
METHOD OF EXCAVATION Manual
LOCATION See Figure 2



DEPTH (FEET)	SAMPLES Bulk	Driven	Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SM	<u>ALLUVIUM</u>: Brown, damp, medium dense, silty fine to coarse silty SAND; trace gravel.
2							Total Depth = 2 feet. (Refusal) Groundwater not encountered. Backfilled with soil on 10/07/11. <u>Note:</u> Groundwater, though not encountered at the time of excavation, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
4							
6							
8							
10							
12							

SCALE = 1 in./2 ft.

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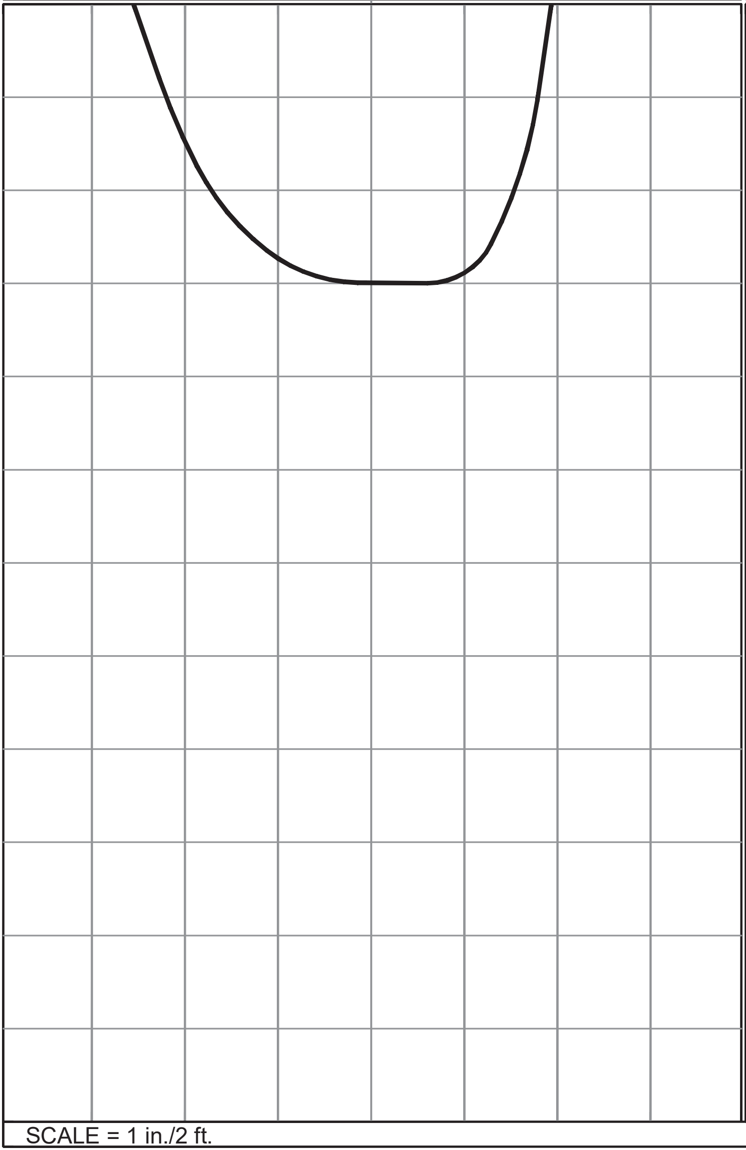
TEST PIT LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE
105991023	2/16

DATE EXCAVATED 9/12/11
TEST PIT NO. TP-5
GROUND ELEVATION 60'± (MSL)
LOGGED BY MBG
METHOD OF EXCAVATION Manual
LOCATION See Figure 2



DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0	■					SM	OLD PARALIC DEPOSITS: Yellowish brown, dry to damp, medium dense, silty fine SAND; scattered roots.
	■	■		1.6	99.3		
2	■						Moist.
							Total Depth = 3 feet. Groundwater not encountered. Backfilled with soil on 9/12/11. Note: Groundwater, though not encountered at the time of excavation, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
4							
6							
8							
10							
12							

SCALE = 1 in./2 ft.

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TEST PIT LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE
105991023	2/16

DATE EXCAVATED 10/07/11

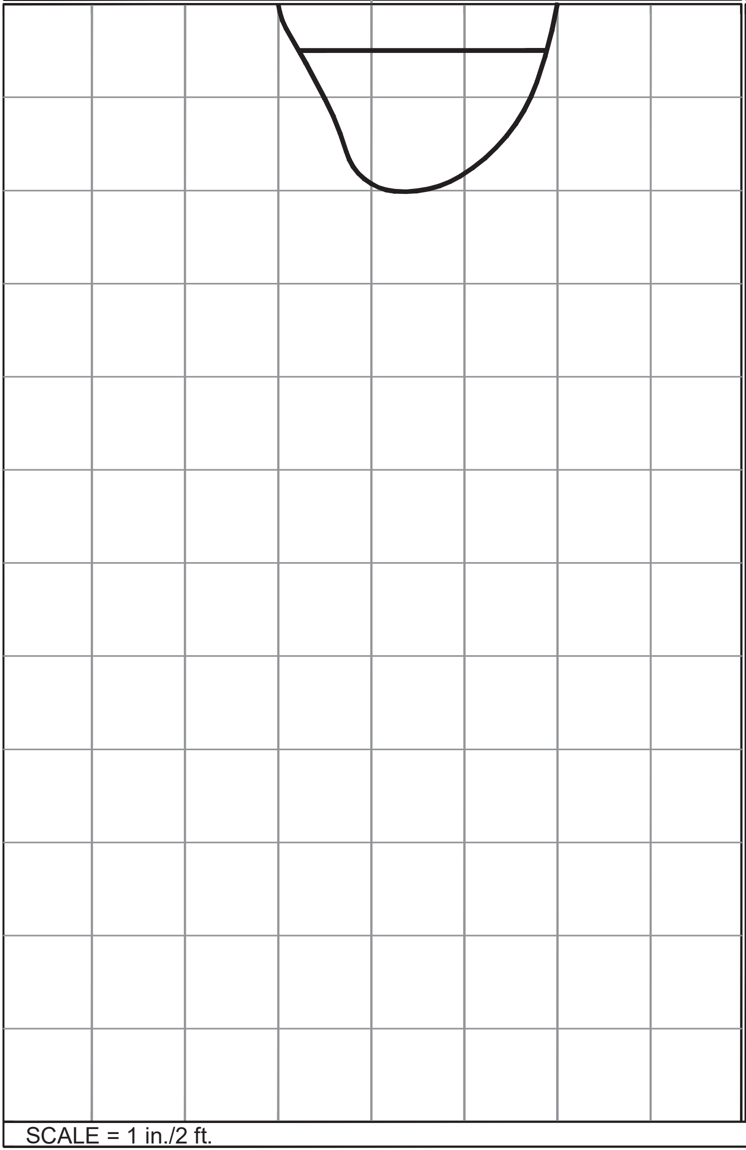
TEST PIT NO. TP-6

GROUND ELEVATION 50'± (MSL)

LOGGED BY MAC

METHOD OF EXCAVATION Manual

LOCATION See Figure 2

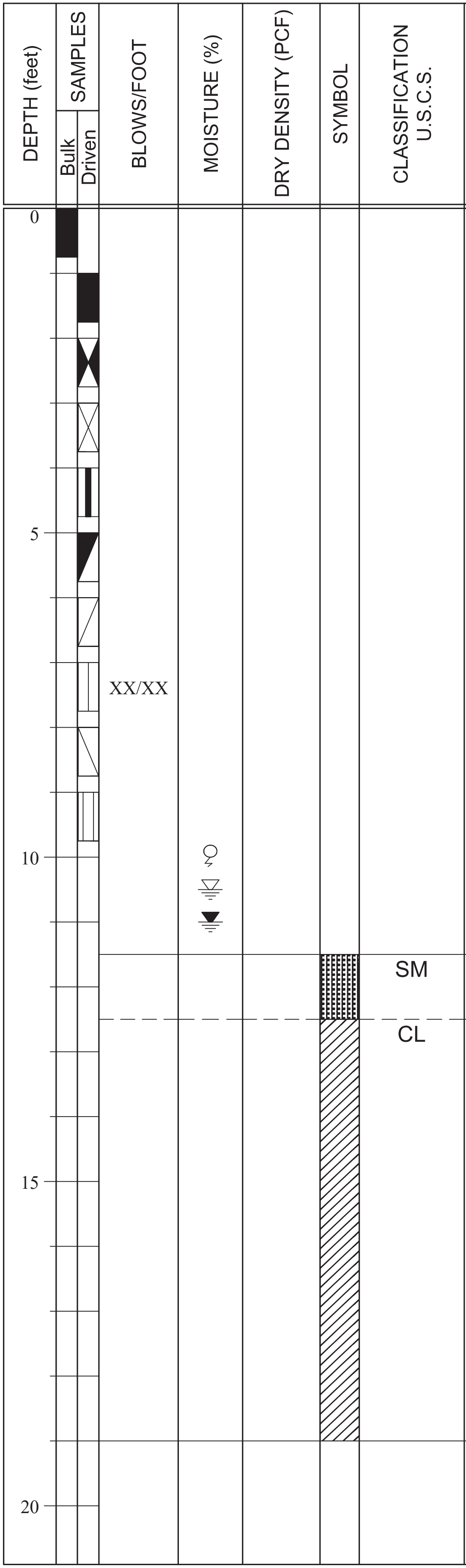


DEPTH (FEET)	SAMPLES Bulk	SAMPLES Driven	SAMPLES Sand Cone	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION
0						SP-SM	FILL: Brown, damp, dense, poorly graded, fine to coarse SAND with silt and gravel.
	■					SM	ALLUVIUM: Brown, damp, dense, silty fine to coarse SAND; trace gravel; trace clay.
2							Total Depth = 2 feet. (Refusal) Groundwater not encountered. Backfilled with soil on 9/12/11. Note: Groundwater, though not encountered at the time of excavation, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
4							
6							
8							
10							
12							

SCALE = 1 in./2 ft.

FIGURE A-54

BORING LOG EXPLANATION SHEET



Bulk sample.

Modified split-barrel drive sampler.

2-inch inner diameter split-barrel drive sampler.

No recovery with modified split-barrel drive sampler, or 2-inch inner diameter split-barrel drive sampler.

Sample retained by others.

Standard Penetration Test (SPT).

No recovery with a SPT.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.

No recovery with Shelby tube sampler.

Continuous Push Sample.

Seepage.
Groundwater encountered during drilling.
Groundwater measured after drilling.

MAJOR MATERIAL TYPE (SOIL):
Solid line denotes unit change.

Dashed line denotes material change.

Attitudes: Strike/Dip
b: Bedding
c: Contact
j: Joint
f: Fracture
F: Fault
cs: Clay Seam
s: Shear
bss: Basal Slide Surface
sf: Shear Fracture
sz: Shear Zone
sbs: Shear Bedding Surface

The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

Explanation of Boring Log Symbols

PROJECT NO.	DATE	FIGURE

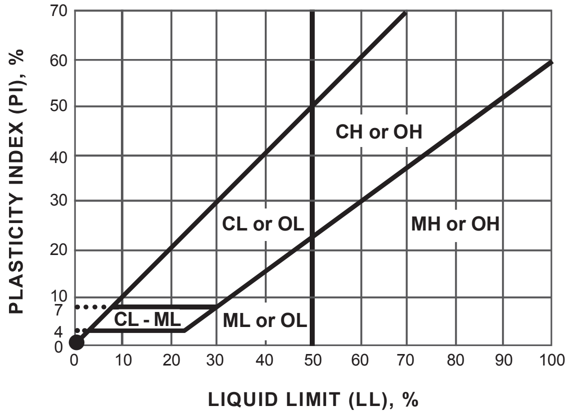
SOIL CLASSIFICATION CHART PER ASTM D 2488

PRIMARY DIVISIONS			SECONDARY DIVISIONS		
			GROUP SYMBOL		GROUP NAME
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	▨	GW	well-graded GRAVEL
			▨	GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	▨	GW-GM	well-graded GRAVEL with silt
			▨	GP-GM	poorly graded GRAVEL with silt
			▨	GW-GC	well-graded GRAVEL with clay
			▨	GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	▨	GM	silty GRAVEL
			▨	GC	clayey GRAVEL
			▨	GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	▨	SW	well-graded SAND
			▨	SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	▨	SW-SM	well-graded SAND with silt
			▨	SP-SM	poorly graded SAND with silt
			▨	SW-SC	well-graded SAND with clay
			▨	SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	▨	SM	silty SAND
			▨	SC	clayey SAND
			▨	SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	▨	CL	lean CLAY
			▨	ML	SILT
			▨	CL-ML	silty CLAY
		ORGANIC	▨	OL (PI > 4)	organic CLAY
			▨	OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	▨	CH	fat CLAY
			▨	MH	elastic SILT
		ORGANIC	▨	OH (plots on or above "A"-line)	organic CLAY
			▨	OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		▨	PT	Peat

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

PLASTICITY CHART



APPARENT DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

CONSISTENCY - FINE-GRAINED SOIL

CONSIS-TENCY	SPOOLING CABLE OR CATHEAD		AUTOMATIC TRIP HAMMER	
	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)	SPT (blows/foot)	MODIFIED SPLIT BARREL (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

Explanation of USCS Method of Soil Classification

PROJECT NO.	DATE	FIGURE

DATE DRILLED 1/31/14 BORING NO. B-13
GROUND ELEVATION 49' ± (MSL) SHEET 1 OF 1
METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Mole) (Pacific)
DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"
SAMPLED BY GS LOGGED BY GS REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■					▦	SM	<u>FILL:</u> Light brown, damp, loose, silty SAND; scattered organics. Moist; medium dense.
						▦	SM	<u>OLD PARALIC DEPOSITS:</u> Yellowish brown, moist, medium dense, silty SAND; trace clay.
		■	30	10.5	106.3	▨	SC	Dark grayish brown, moist, medium dense, clayey SAND.
10		■	24			▦	SM	Yellowish brown, moist, medium dense, silty SAND; trace clay.
	■					▨	SC	Dark brown, moist, medium dense, clayey SAND.
		■	35	9.3	114.5	▦	SM	Mottled reddish brown and gray, moist, medium dense, silty SAND; trace clay.
20 30 40								Total Depth = 16.5 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 1/31/14. <u>Note:</u> Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

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BORING LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991023	2/16	A-24

DATE DRILLED 1/31/14 BORING NO. B-14
GROUND ELEVATION 33' ± (MSL) SHEET 1 OF 1
METHOD OF DRILLING 6" Diameter Hollow Stem Auger (Mole) (Pacific)
DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"
SAMPLED BY GS LOGGED BY GS REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						SM	FILL: Light brown, damp, loose to medium dense, silty SAND.
	■						SC	Yellowish brown, moist, medium dense, clayey SAND; scattered roots.
							SP	Yellowish brown, moist, loose to medium dense, poorly graded fine SAND.
		■	38	5.7	112.4		SM	OLD PARALIC DEPOSITS: Mottled reddish brown and gray, moist, medium dense, silty fine SAND; trace clay; scattered roots.
10		■	75	5.6	112.5			Dense.
								Difficult drilling.
							CL	Grayish brown, moist, hard, sandy CLAY.
	■							Wet.
		■	54	15.8	114.3			
								Total Depth = 16.5 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 1/31/14.
20								Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

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BORING LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991023	2/16	A-25

DATE DRILLED 10/4/11 BORING NO. B-3

GROUND ELEVATION 35'± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 6" Hollow-Stem Auger (Pacific Drilling) (Mole Rig)

DRIVE WEIGHT 140 lbs. (Cathead) DROP 30"

SAMPLED BY MBG LOGGED BY MBG REVIEWED BY JG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Light brown, damp, medium dense, silty fine SAND.
			37	9.3	115.0			
								DEL MAR FORMATION: Light olive brown, damp, moderately cemented, fine sandy SILTSTONE. Light olive brown, moist, weakly cemented, silty, fine-grained SANDSTONE.
			23	8.7	117.6			
10			50/5"					Medium-grained sandstone.
			50/6"	▽				Mottled yellowish brown, light olive brown and medium brown; strongly cemented; trace of clay. @ 16': Saturated.
			50/3"					Dark olive brown; clayey and silty; fine-grained sandstone.
20								Total Depth = 19.5 feet. Groundwater encountered at approximately 16 feet. Backfilled with approximately 4 cubic feet of bentonite grout on 10/4/11.
								Note: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report.
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

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BORING LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991023	2/16	A-7

DATE DRILLED 2/12/14 BORING NO. B-15
GROUND ELEVATION 58' ± (MSL) SHEET 1 OF 1
METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja)
DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"
SAMPLED BY AQP LOGGED BY AQP REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 6 inches thick.
							GM	BASE: Grayish brown, damp, dense, silty GRAVEL with sand; approximately 3 inches thick.
							SM	FILL: Mottled reddish brown and gray, damp to moist, medium dense, silty SAND; few clay.
		■	62				SM	OLD PARALIC DEPOSITS: Reddish brown, damp to moist, dense, silty SAND.
10		■	50					DELMAR FORMATION: Mottled light olive brown and red, damp, moderately indurated, silty CLAYSTONE.
		■	50/4"					Light olive brown, damp, moderately cemented, silty SANDSTONE.
20	■							
		■	50/5"					Light reddish gray. Mottled light olive brown and red, damp, moderately to strongly cemented, sandy SILTSTONE; scattered gravel.
	■							
30		■	50/5"					Total Depth = 30.4 feet. Groundwater not encountered during drilling. Backfilled with approximately 10 cubic feet of bentonite grout and patched with hot mix asphalt shortly after drilling on 2/12/14. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

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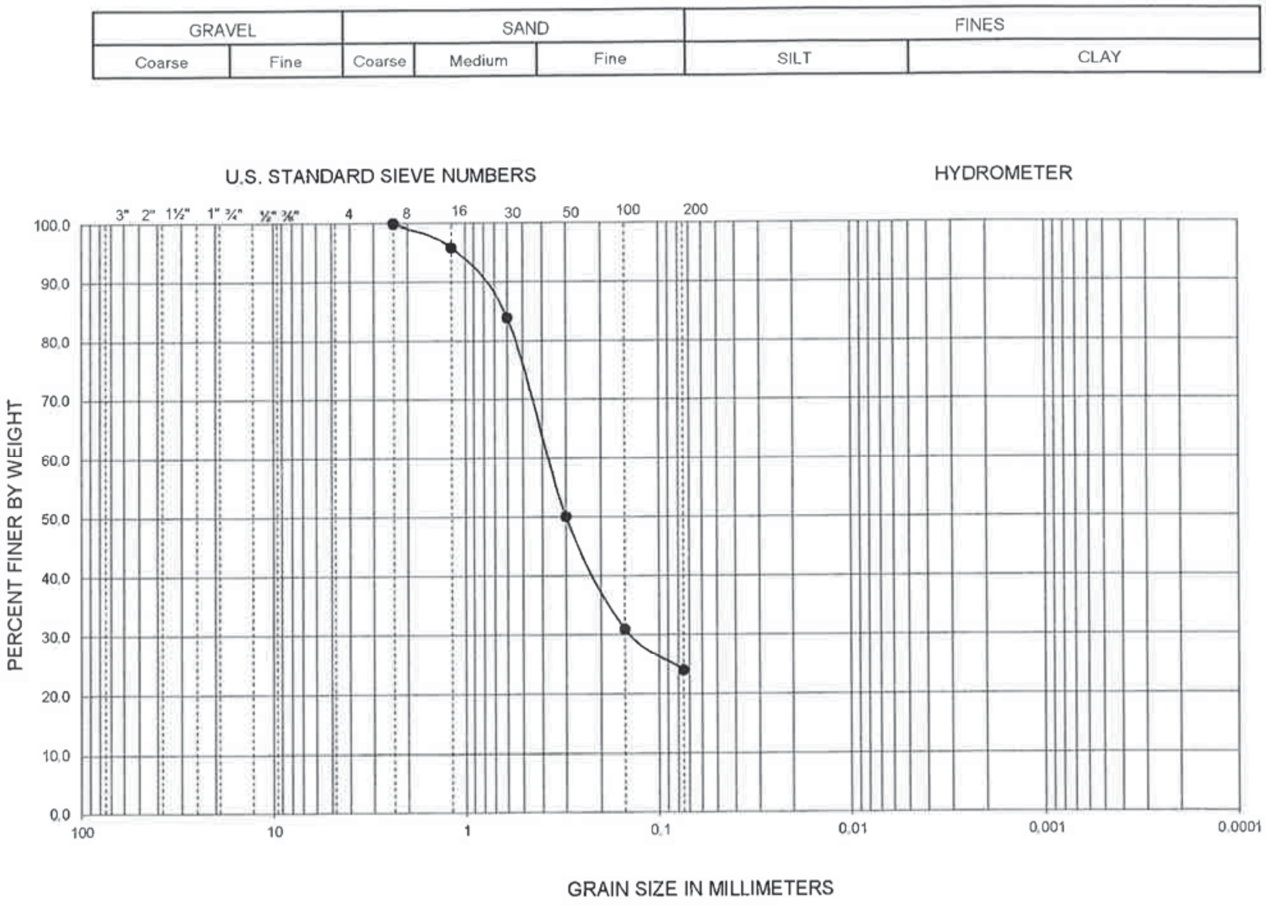
BORING LOG

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

PROJECT NO.	DATE	FIGURE
105991023	2/16	A-26



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	TP-1	2.0-3.0	--	--	--	--	--	--	--	--	24	SM

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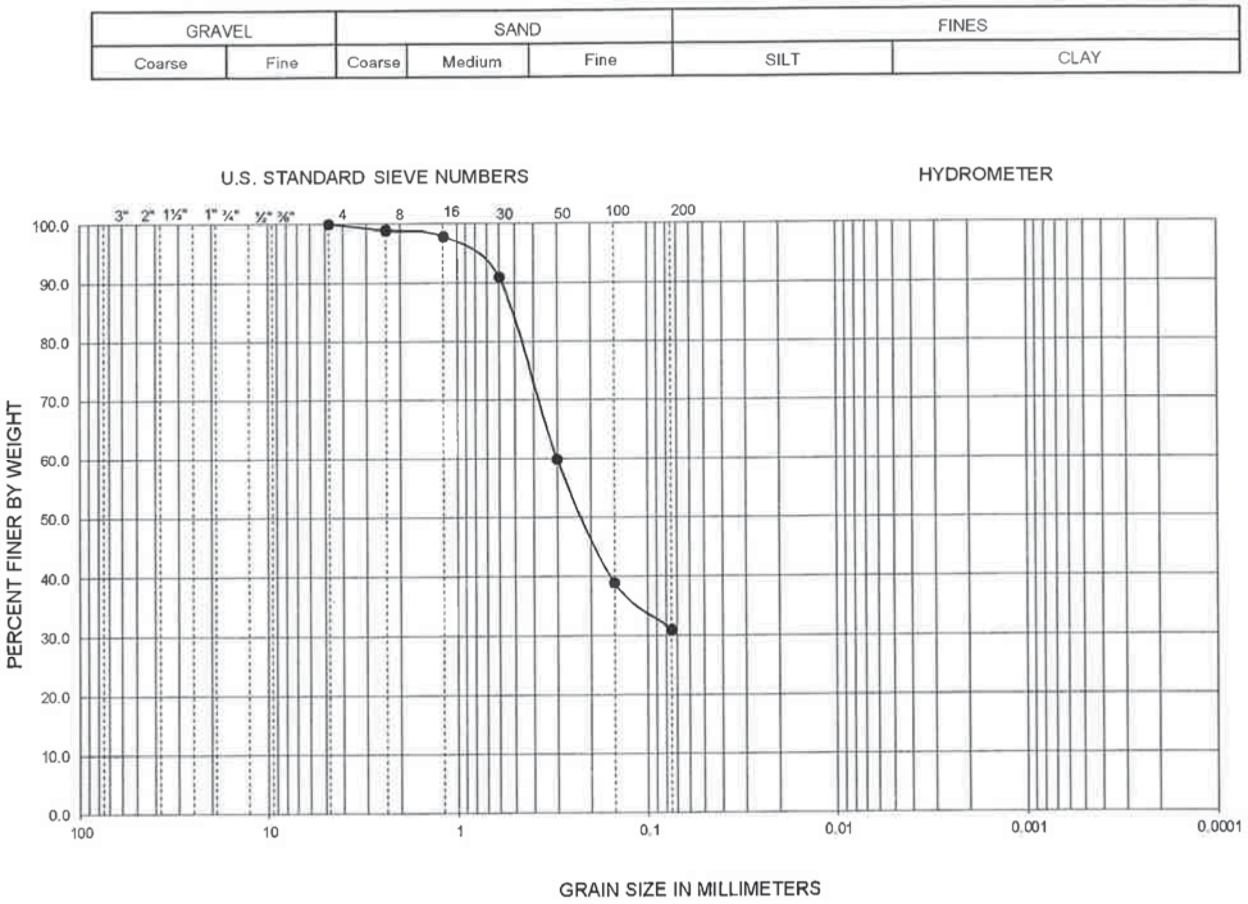
GRADATION TEST RESULTS

PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE
B-41



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	TP-2	4.0-5.0	--	--	--	--	--	--	--	--	31	SM

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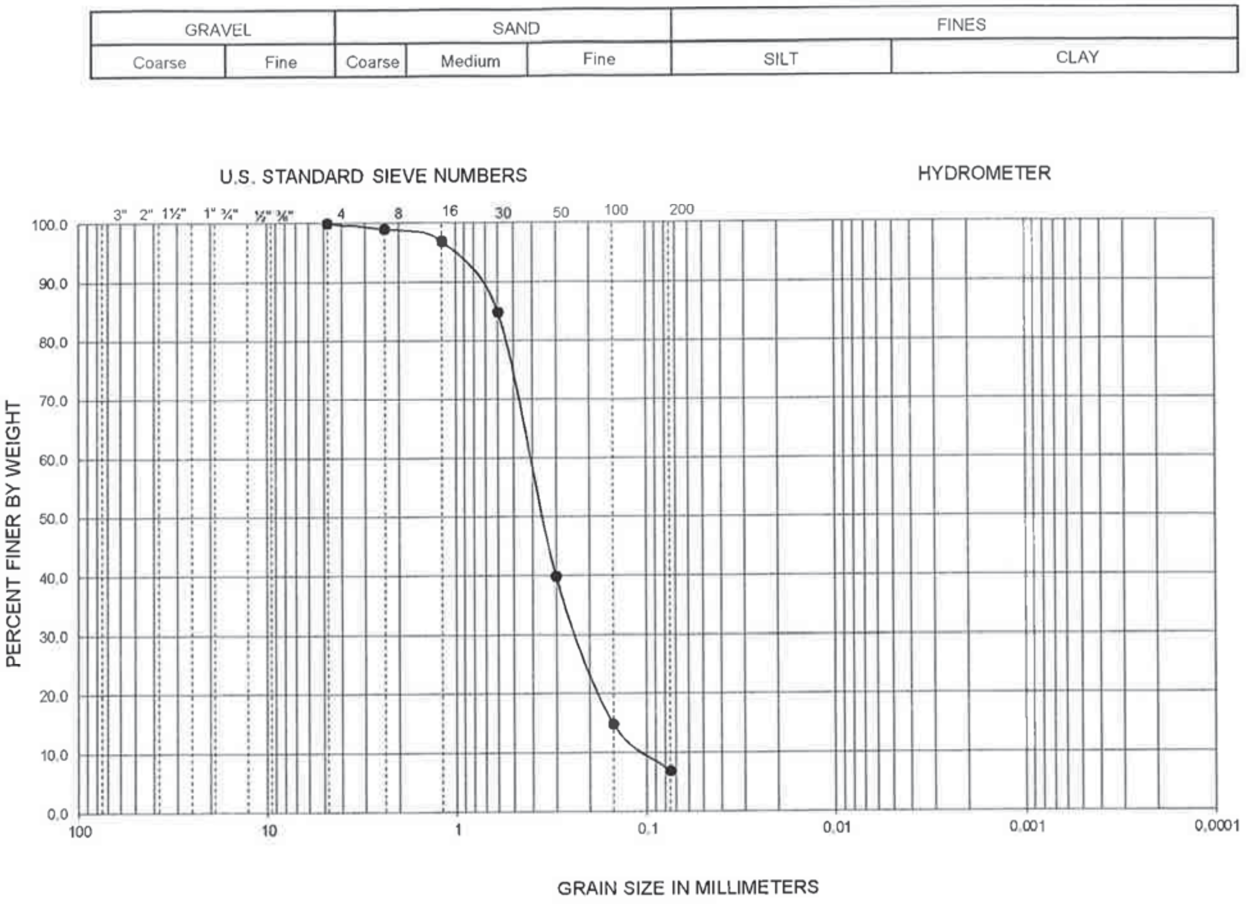
GRADATION TEST RESULTS

PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE
B-42



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	TP-3	0.0-2.5	--	--	--	0.11	0.25	0.30	2.7	1.9	7	SP-SM

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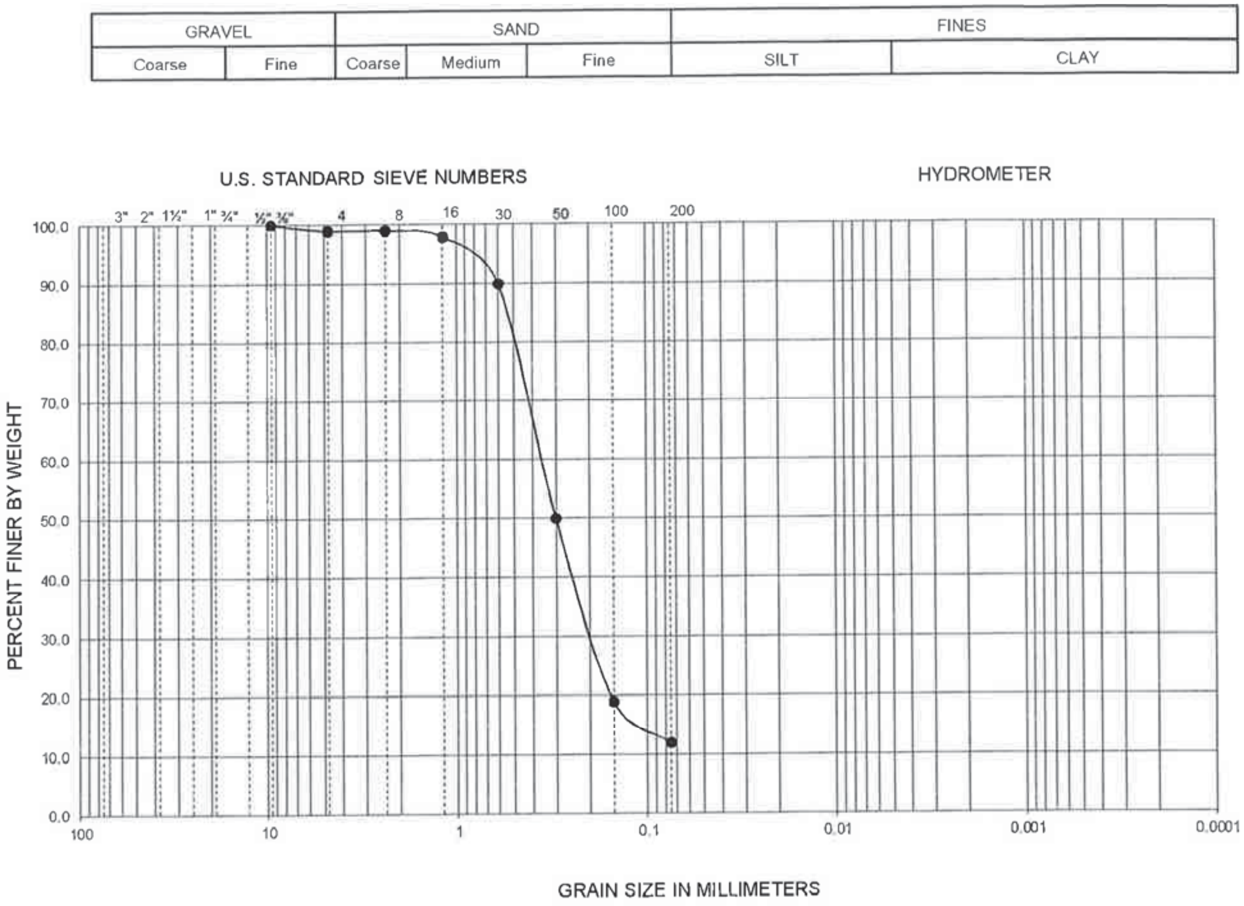
GRADATION TEST RESULTS

PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE
B-43



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	TP-5	0.0-3.0	--	--	--	--	--	--	--	--	12	SM

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GRADATION TEST RESULTS

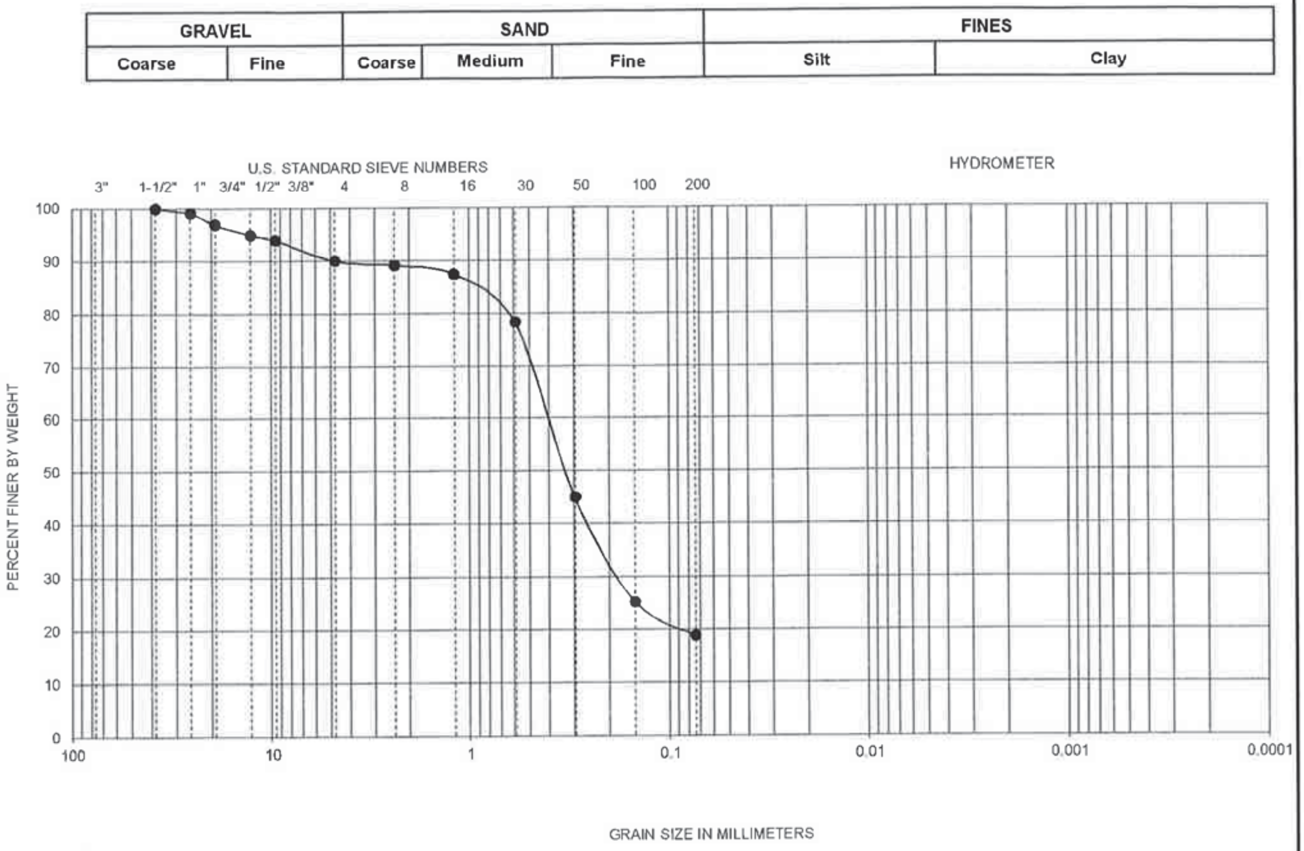
PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE

B-44

105991023 SIEVE TP-5 @ 0.0-3.0

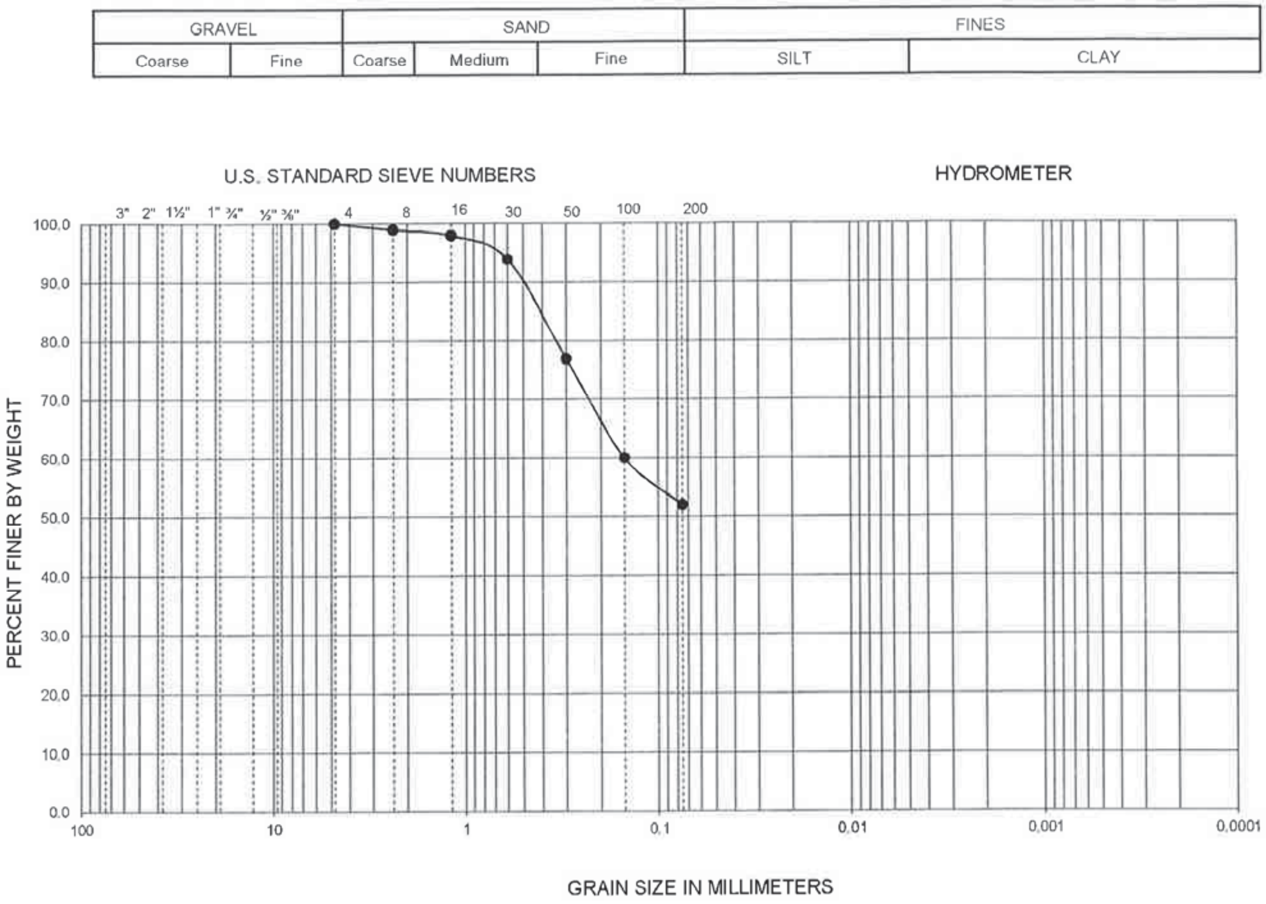


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-13	0.0-4.0	--	--	--	--	--	--	--	--	19	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

Ninyo & Moore		GRADATION TEST RESULTS	FIGURE
PROJECT NO.	DATE	SAN ELIJO LAGOON DOUBLE TRACK PROJECT ENCINITAS AND SOLANA BEACH, CALIFORNIA	B-26
105991023	2/16		

105991023 SIEVE B-13 @ 0.0-4.0



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	USCS
●	B-14	13.0-14.0	--	--	--	--	--	--	--	--	52	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

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GRADATION TEST RESULTS

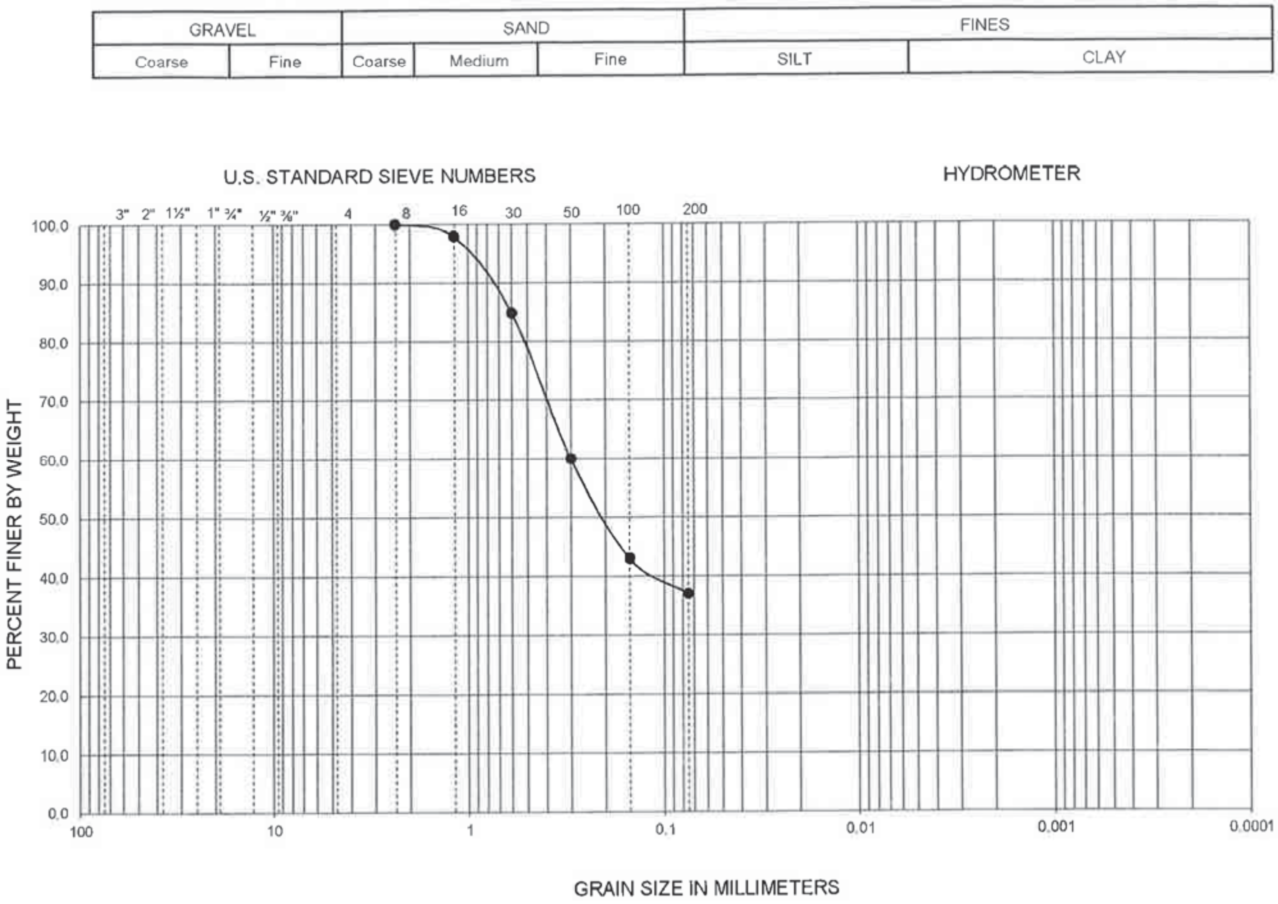
PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE

B-27

105991023 SIEVE B-14 @ 13.0-14.0



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (%)	Equivalent USCS
●	B-15	20.0-24.0	--	--	--	--	--	--	--	--	37	SM

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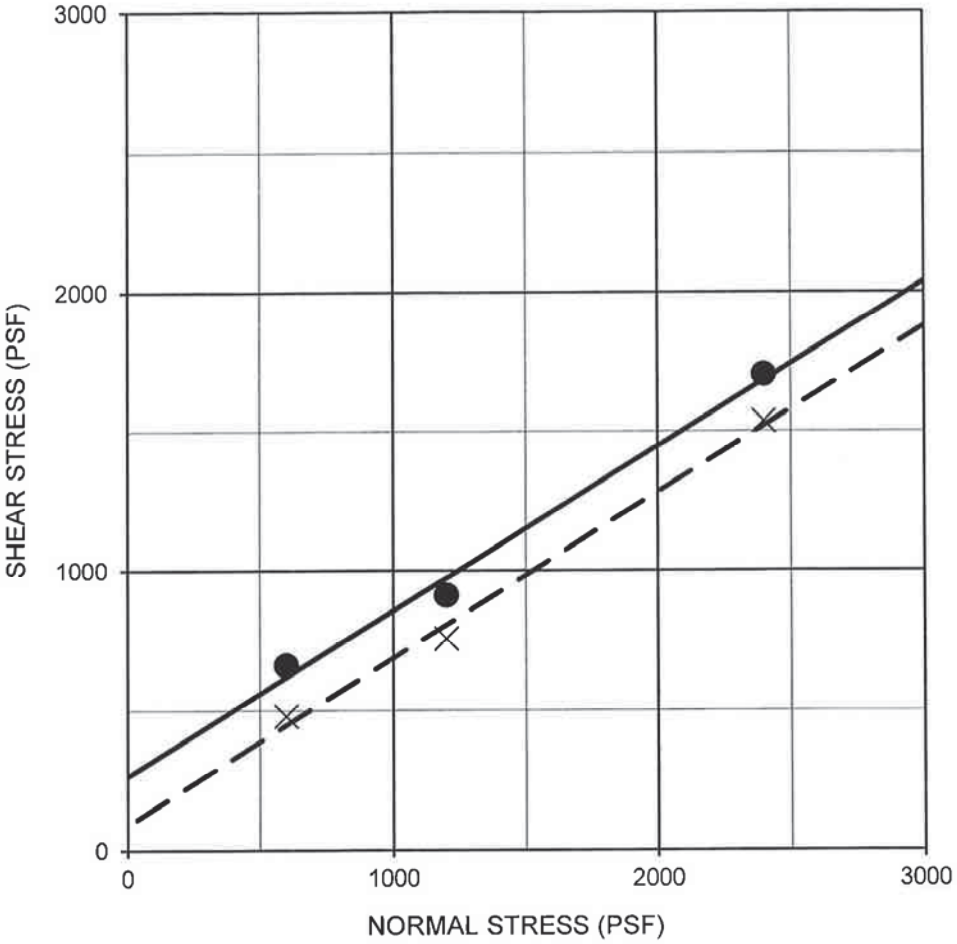
GRADATION TEST RESULTS

PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE

B-28



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Silty SAND	—●—	B-13	8.0-9.5	Peak	260	31	SM
Silty SAND	- - X - -	B-13	8.0-9.5	Ultimate	90	31	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

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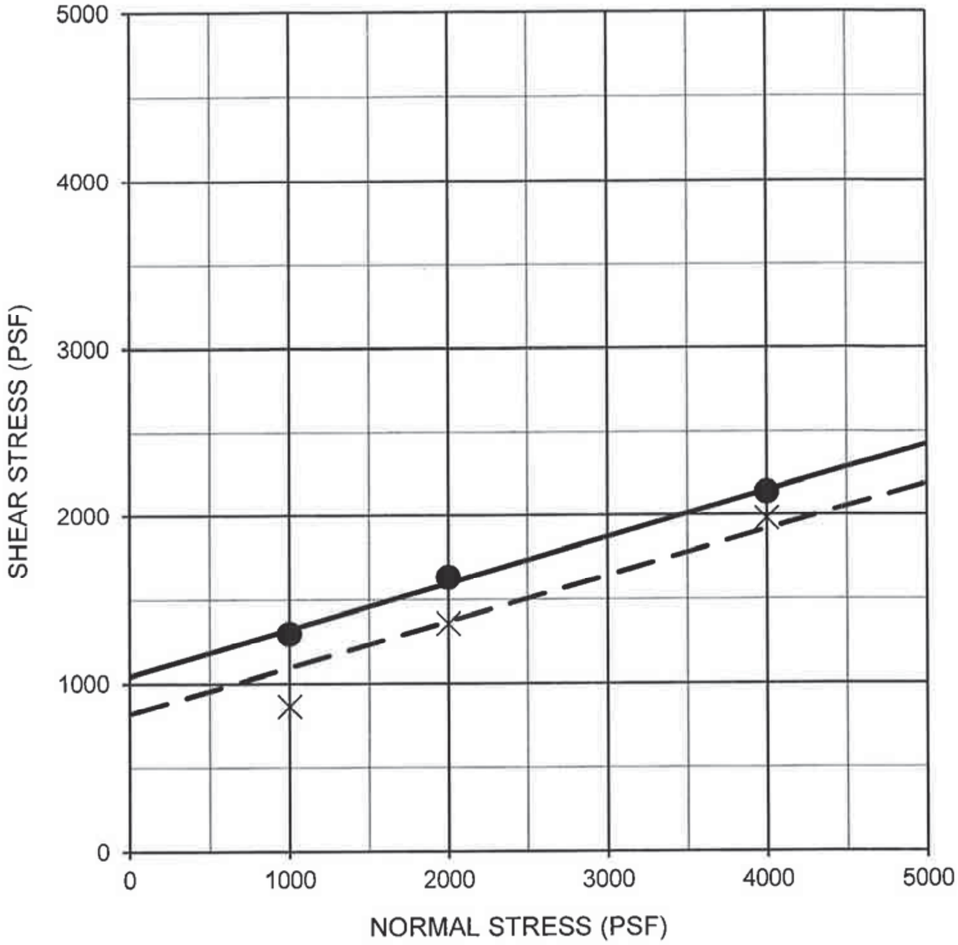
DIRECT SHEAR TEST RESULTS

PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE
B-62

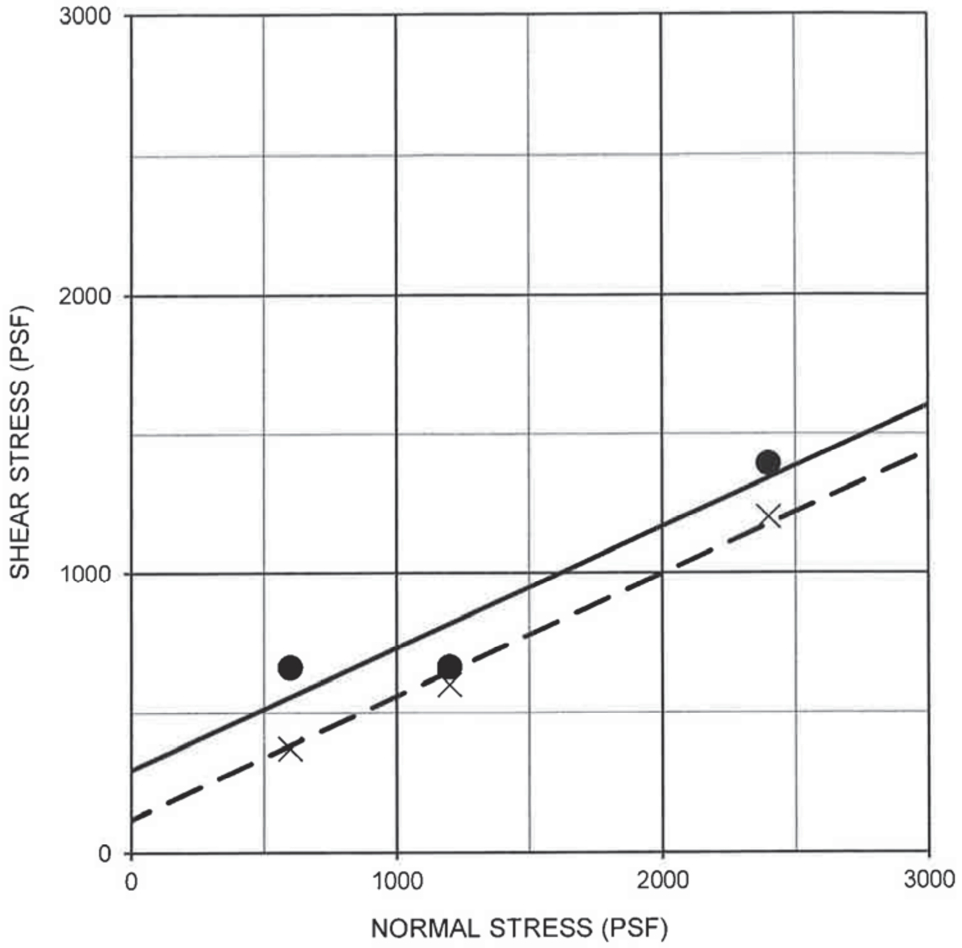
105991023_DIRECT SHEAR B-13 @ 8.0-9.5



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Sandy CLAY	—●—	B-14	15.0-16.5	Peak	1,040	15	CL
Sandy CLAY	- - X - -	B-14	15.0-16.5	Ultimate	820	15	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	SAN ELIJO LAGOON DOUBLE TRACK PROJECT ENCINITAS AND SOLANA BEACH, CALIFORNIA	B-63
105991023	2/16		



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion, c (psf)	Friction Angle, ϕ (degrees)	Soil Type
Sandy CLAYSTONE	—●—	B-15	10.0-11.5	Peak	300	24	Formation
Sandy CLAYSTONE	- - X - -	B-15	10.0-11.5	Ultimate	120	24	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

Ninyo & Moore		DIRECT SHEAR TEST RESULTS	FIGURE
PROJECT NO.	DATE	SAN ELIJO LAGOON DOUBLE TRACK PROJECT ENCINITAS AND SOLANA BEACH, CALIFORNIA	B-64
105991023	2/16		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-4	3.0-5.0	12.0	102.6	23.2	0.040	40	Low
B-14	0.5-5.0	8.0	118.1	14.9	0.002	2	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

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EXPANSION INDEX TEST RESULTS

PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE
B-68

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH[1]	RESISTIVITY[1] (Ohm-cm)	SULFATE CONTENT[2]		CHLORIDE CONTENT[3] (ppm)
				(ppm)	(%)	
B-1	55.0-57.0	8.2	110	4,200	0.420	3,300
B-2	5.0-6.5	7.0	2,100	180	0.018	85
B-2	20.0-21.4	6.6	700	180	0.018	95
B-8	30.0-31.0	6.4	115	960	0.096	3,850
B-8	47.0-48.0	6.6	105	1000	0.100	3,700
B-10	7.0-8.0	5.9	120	1650	0.165	4,300
B-10	36.0-37.0	5.8	130	1680	0.168	4,800
B-14	0.5-5.0	7.1	3,000	30	0.003	210

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore

CORROSIVITY TEST RESULTS

PROJECT NO.	DATE
105991023	2/16

SAN ELIJO LAGOON DOUBLE TRACK PROJECT
ENCINITAS AND SOLANA BEACH, CALIFORNIA

FIGURE
B-51

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH[1]	RESISTIVITY[1] (Ohm-cm)	SULFATE CONTENT[2]		CHLORIDE CONTENT[3] (ppm)
				(ppm)	(%)	
B-16	1.0-5.0	7.8	2,500	180	0.018	120
HA-14	0.5-2.5	6.7	50	3,200	0.320	295
HA-18	1.0-3.0	7.2	60	1,800	0.180	8,400
TP-1	2.0-3.0	7.1	290	370	0.037	1,380
TP-3	0.0-2.5	6.8	1,100	160	0.016	395
TP-8	0.0-2.0	7.0	2,100	20	0.002	310
TP-10	0.5-2.0	7.0	2,300	20	0.002	330

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

Ninyo & Moore		CORROSIVITY TEST RESULTS	FIGURE
PROJECT NO.	DATE	SAN ELIJO LAGOON DOUBLE TRACK PROJECT ENCINITAS AND SOLANA BEACH, CALIFORNIA	B-52
105991023	2/16		

Appendix E. Engineering Analysis Results

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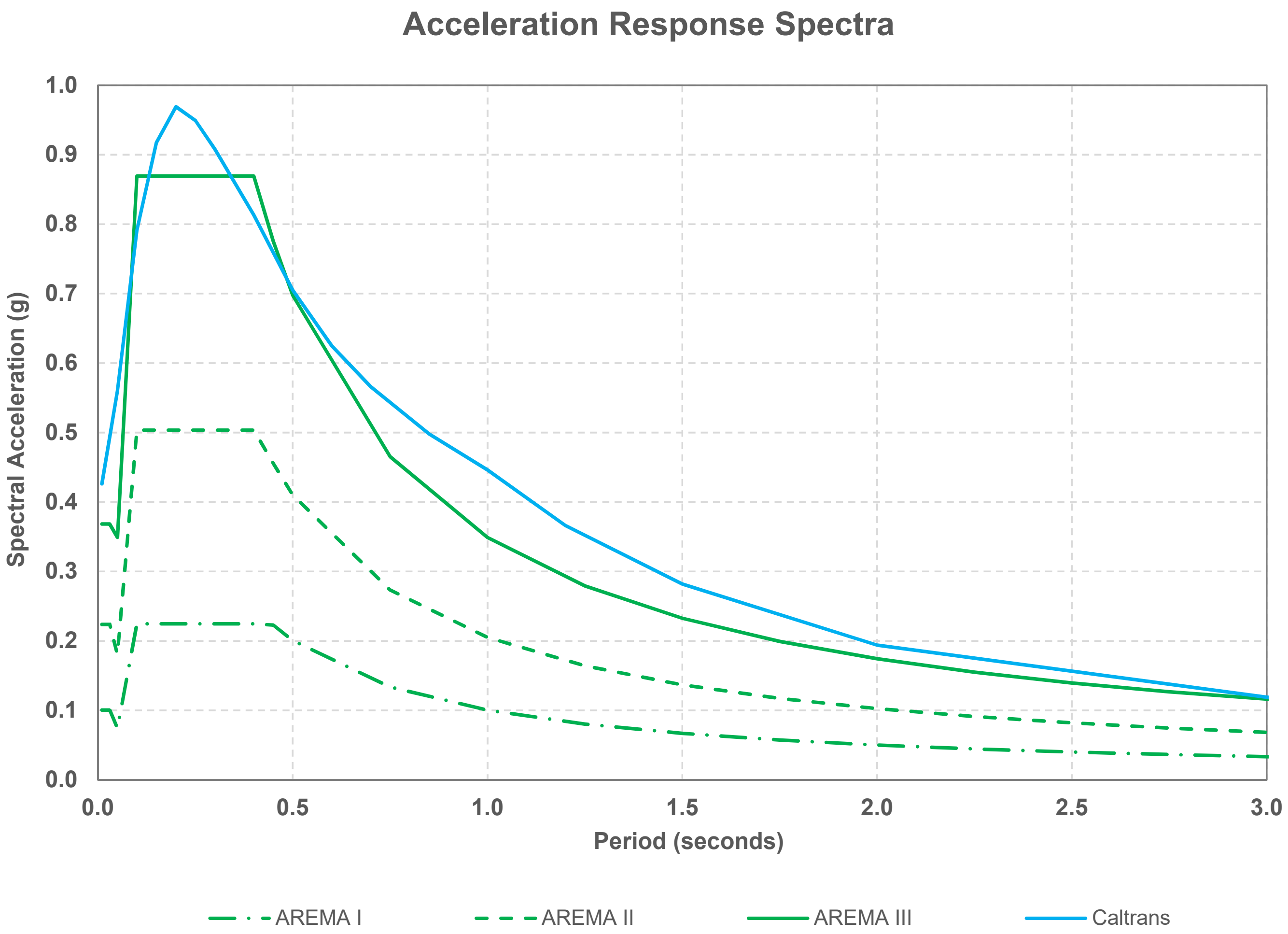

Latitude:	33.0268	**Caltrans PGA:**	0.43g
Longitude:	-117.287		
V_{s30}:	537 m/s		
Site Class:	C		
Moment Magnitude:	6.7		

AREMA Parameters

Limit State	Return Period (yrs)	PGA_M (g)
Serviceability (AREMA I)	100	0.10
Ultimate (AREMA II)	453	0.22
Survivability (AREMA III)	1516	0.37

Notes:

- ARS values are considered approximate. Values are likely to change prior to final design.
- Return periods estimated based on AREMA CH 9 Section 1.3.2.
- 5% critical damping with damping adjustment factor of one used to develop AREMA ARS curves.
- Magnitude based on USGS Dynamic Conterminous 2014 (v4.2.0) deaggregation for 475 year return interval.
- V_{s30} estimated based on generalized soil profile and only used for developing Caltrans ARS curve.

Figure 1

Acceleration Response Spectra Tabular Data

AREMA ARS				Caltrans ARS	
	Serviceability	Ultimate	Survivability		
T	Acc.	Acc.	Acc.	T	Acc.
(Sec)	(g)	(g)	(g)	(Sec)	(g)
0	0.10	0.22	0.37	0.01	0.43
0.01	0.10	0.22	0.37	0.05	0.56
0.02	0.10	0.22	0.37	0.10	0.79
0.03	0.09	0.20	0.36	0.15	0.92
0.04	0.07	0.18	0.35	0.20	0.97
0.05	0.22	0.50	0.87	0.25	0.95
0.1	0.22	0.50	0.87	0.30	0.91
0.15	0.22	0.50	0.87	0.40	0.81
0.2	0.22	0.50	0.87	0.50	0.71
0.25	0.22	0.50	0.87	0.60	0.63
0.3	0.22	0.50	0.87	0.70	0.57
0.35	0.22	0.50	0.87	0.85	0.50
0.4	0.22	0.46	0.78	1.00	0.45
0.45	0.20	0.41	0.70	1.20	0.37
0.5	0.13	0.27	0.47	1.50	0.28
0.75	0.10	0.21	0.35	2.00	0.19
1	0.08	0.16	0.28	3.00	0.12
1.25	0.07	0.14	0.23	4.00	0.08
1.5	0.06	0.12	0.20	5.00	0.07
1.75	0.05	0.10	0.17		
2	0.04	0.09	0.16		
2.25	0.04	0.08	0.14		
2.5	0.04	0.07	0.13		
2.75	0.03	0.07	0.12		
3	0.03	0.06	0.10		
3.5	0.03	0.05	0.09		
4	0.00	0.00	0.00		

Figure 2

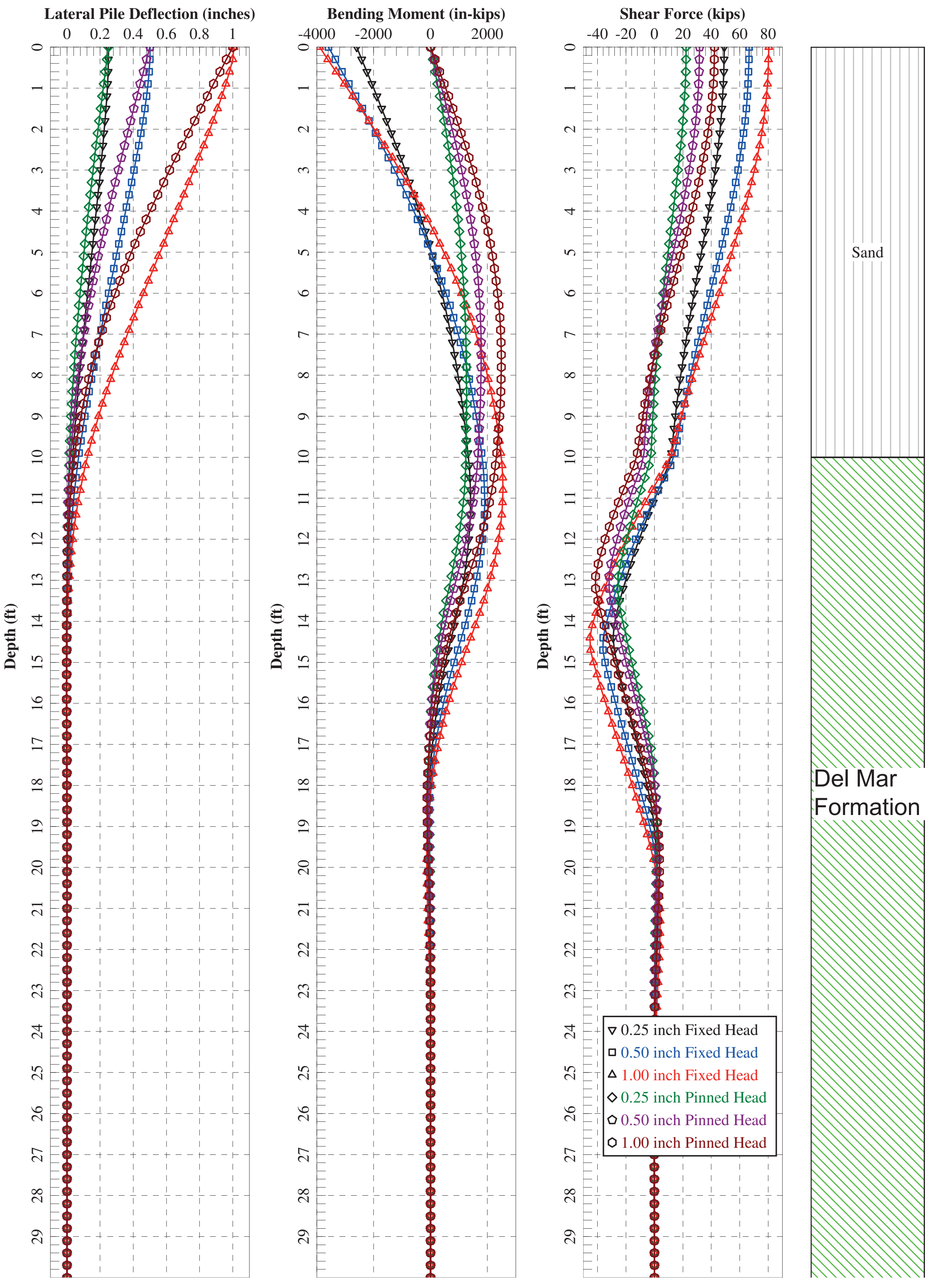

Figure 3 - Lateral Loading at Abutment Piles

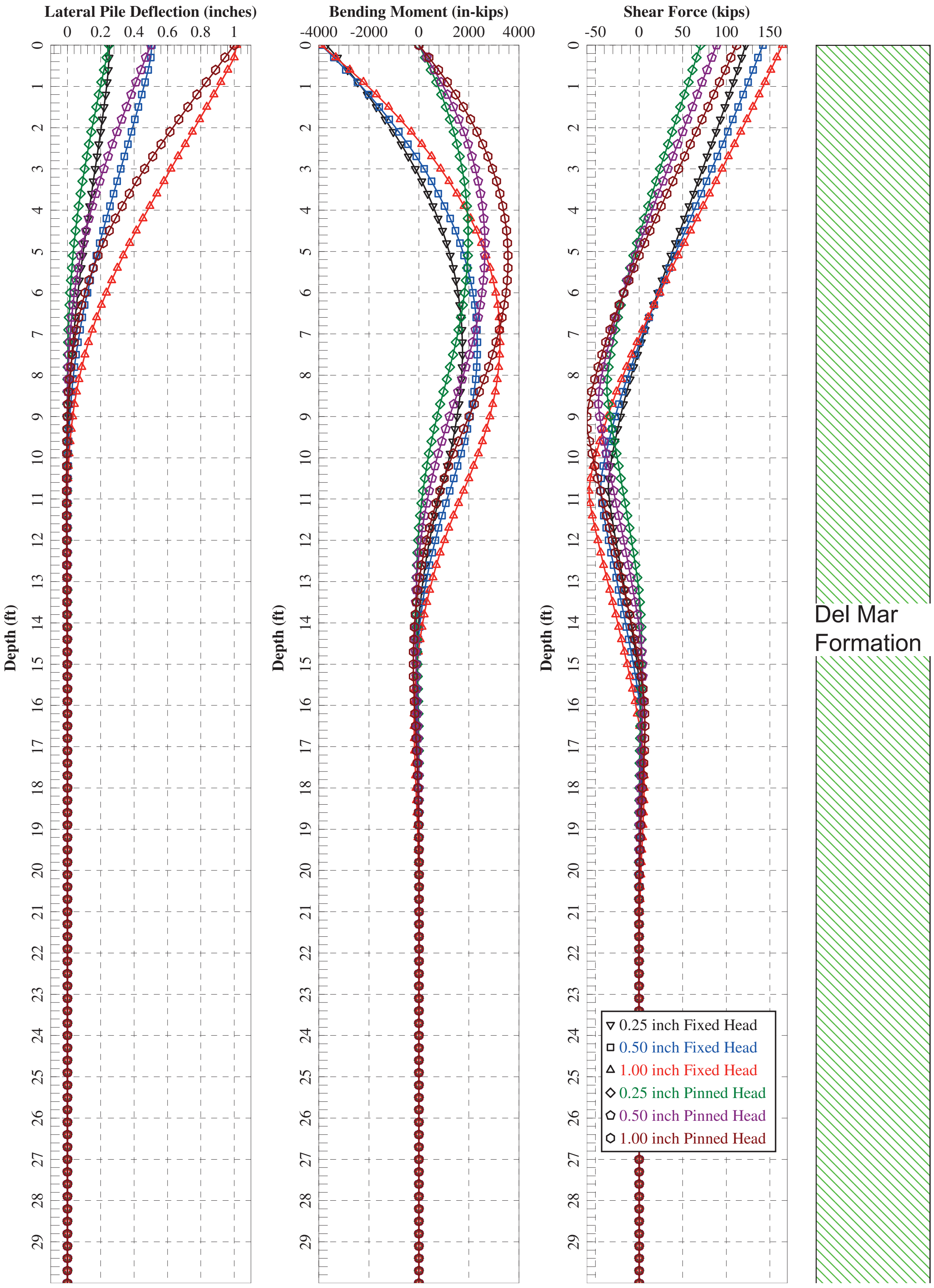

Figure 4 - Lateral Loading at Bent Piles