Appendix D. Roman Creek Geotechnical Design Report

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GEOTECHNICAL DESIGN REPORT

CIP 8188: ROMAN CREEK MITIGATION PROJECT

Vista, California

December 19, 2019
Revised March 27, 2020

Prepared for:

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March 27, 2020

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Subject: Geotechnical Design Report

CIP 8188: Roman Creek Mitigation Project

We are pleased to present this geotechnical design report summarizing results of our subsurface exploration and engineering evaluations, along with the recommendations for design and construction of the subject project. This geotechnical design report provides recommendations for the proposed cut/fill slopes and related earthwork activities, and pedestrian bridge.

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

Respectfully submitted,

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MF/GG

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1.0 INTRODUCTION

1.1 PROJECT DESCRIPTION

The City of Vista is considering the implementation of the Roman Creek Mitigation Project (Project) at Buena Vista Park in Vista, California. The Project is located within the Buena Vista Park boundaries, with the proposed improvements generally located along Roman Creek in the western and southern portions of Buena Vista Park. The proposed Project would create management areas (or units) within the limits of the existing Buena Vista Park to facilitate planning and implementation of hydromodification improvements, compensatory mitigation, and habitat restoration activities within the Roman Creek Mitigation Site (Site). The Project proposes to reestablish riparian habitats within the Mitigation Site by performing cut and fill grading within existing slopes along the Roman Creek. Generally, slopes are proposed to be in the range of about 3 to 13 feet in height. Maximum slope ratios of 2H:1V are considered for slopes of up to 4 feet high. Taller slopes (up to 13 feet high) are also considered, but with slope inclinations of about 6H:1V or flatter. Additionally a 110-foot long pedestrian bridge is proposed near the southeastern edge of the project replacing the existing shorter bridge to continue to provide a looped trail over the other proposed improvements. The Project site location is shown on Figure 1 in Appendix A.

1.2 PURPOSE AND SCOPE

The purpose of this geotechnical report is to review existing geotechnical/geologic data, perform subsurface exploration, laboratory testing, evaluate geologic hazards, and present results of our geotechnical analyses, and provide limited geotechnical design recommendations for the Project.

Our scope of work for this Project included the following tasks:

Literature Review: Reviewed available geologic maps and reports pertinent to the Project site. A list of references used in preparation of this report is presented in Section 8.0.

Pre-Field Activities: Performed a site reconnaissance to visually evaluate the accessibility of the site for drilling equipment, and to locate and mark the proposed boring locations. Underground Service Alert (USA) of Southern California was contacted to identify subsurface utilities and obtain clearance for drilling at the site.

Preparation and Submittal of Work Plan: Prior to the commencement of the field investigation, a work plan was prepared and submitted for approval to City of Vista and a boring permit was obtained from the County of San Diego. This work plan included the field work scope, equipment, boring backfill details, schedule, site access, work impacts, hazards, spills, safety and emergency protocol.

Field Exploration and Laboratory Testing: Performed subsurface exploration that included drilling of five 8-inch-diameter, hollow-stem auger borings to a maximum depth of about 47 feet below ground surface (bgs), and three 3.5-inch diameter, hand auger borings to a maximum depth of approximately 5 feet bgs. Laboratory testing was performed on selected soil samples, collected to evaluate the engineering properties of the subsurface soils. Percolation testing was performed in three of the boreholes to evaluate the site for storm water infiltration requirements. Additionally, one of the hollow-stem auger borings was converted to a monitoring well. The boring logs and laboratory test results are included in Appendices B and C, respectively.

Geotechnical Analysis: Geotechnical analysis was performed on the collected data to develop design recommendations for the proposed cut/fill slopes and related earthwork activities. Additionally, preliminary foundation design parameters for the proposed bridge are provided.

Report Preparation: Relevant geotechnical data were compiled in this report along with our findings and recommendations for the proposed Project.

2.0 GEOTECHNICAL FIELD AND LABORATORY INVESTIGATIONS

2.1 SUBSURFACE EXPLORATION

The field exploration consisted of advancing five 8-inch-diameter, hollow stem auger borings to a maximum depth of approximately 47 feet bgs. In addition, three 3.5-inch, hand auger borings were performed for those areas where site accessibility was restricted. Hand auger borings were advanced to a maximum depth of five feet bgs. Borings were designated in general accordance with Caltrans (2010) standards with hollow stem auger borings designated with “A”, and hand auger boring locations designated with “HA”. Investigation locations were numbered from 001 through 008. Percolation testing was performed in Borings A-19-001, A-19-002, and A-19-005 to assess infiltration capabilities at the site. The percolation testing was performed at an approximate depth of 5 feet bgs in Borings A-19-001 and A-19-002, and at an approximate 10 feet bgs in Boring A-19-005. Drilling of A-19-001, A-19-002, and A-19-005 was resumed after the completion of the infiltration testing at an adjacent borehole. Additionally, a groundwater level monitoring well was constructed within Boring A-19-003. The well extends to a depth of approximately 47 feet bgs. The intent of this well is to monitor the levels of stabilized groundwater at the site during seasonal variations and after substantial rainfall events.

The boring locations were marked in the field by measuring from known locations of existing features and using a global positioning system (GPS) device. The coordinates for each boring location are shown in Table 2–1. The approximate boring locations are shown on Figure 2 in Appendix A.

Table 2–1. Boring Locations

Boring Name	Approximate Latitude	Approximate Longitude	Approximate Depth (feet)
A-19-001[(1)]	33.15205	-117.24656	40.5
A-19-002[(1)]	33.15200	-117.24721	25.8
A-19-003[(2)]	33.15236	-117.24693	47.1
HA-19-004	33.15282	-117.24659	5.0
A-19-005[(3)]	33.15280	-117.24716	26.5
HA-19-006	33.15382	-117.24700	4.5
HA-19-007	33.15490	-117.24675	5.0
A-19-008	33.15618	-117.24703	25.9

Notes:

(1) Percolation testing performed at depths between 3 to 5 feet bgs.

(2) Boring converted to groundwater monitoring well at the end of drilling.

(3) Percolation testing performed at depths between 8.5 to 10 feet bgs.

Standard Penetration Tests (SPT) were performed within the hollow stem auger 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 also 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 collected from Borings A-19-001, A-19-003, and A-19-008. Bag soil samples were collected from the hand auger borings where a change in material was noted.

The test borings were logged in the field by a member of HDR technical staff. Each soil sample collected was reviewed and described in general accordance with ASTM D2487-11 (Unified Soil Classification System). All samples were sealed and packaged for transportation to a subconsultant's laboratory (AP Laboratory). Corrosion testing was performed by HDR's corrosion laboratory and AP Laboratory.

After completion of drilling, the borings were backfilled with cement-bentonite grout and surface restored to its original condition. Soil cuttings generated during drilling were placed in 55-gallon drums and transported to a designated area for future analytical testing and disposal. Analytical testing was performed by a third party following the Environmental Protection Agency (EPA) methods for soil disposal. Soils cuttings generated during drilling were disposed as non-hazardous. Boring logs are included in Appendix B.

At the time of the field exploration, the design had not been developed enough to include borings for the now proposed pedestrian bridge. The nearest borings (A-19-001 and A-19-003) to the bridge are approximately 150 feet south and 100 feet west of the anticipated westernmost bridge abutment (about 210 feet west of easternmost abutment). An additional field exploration program consisting of at minimum two borings (one at each proposed abutment) to a minimum depth of 50 feet bgs is recommended to obtain site specific data. Foundation recommendations for the bridge provided in this report are based on data obtained from the nearest borings and should be considered preliminary.

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:

- Atterberg limits;
- In-situ moisture content and density;
- Grain-size distribution;
- Percent passing No. 200 sieve;
- Laboratory Compaction (maximum dry density and optimum moisture content);
- Expansion Index;
- Consolidation;
- Direct Shear; and
- Corrosivity (soluble sulfate contents, chloride, pH, and resistivity).

All laboratory tests were performed in general accordance with ASTM International procedures, except corrosivity tests, which were performed in accordance with Caltrans procedures. Results of the laboratory tests are presented in Appendix C.

3.0 GEOLOGY AND 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. Based on a review of the Geologic Map of the Oceanside 30' x 60' Quadrangle (Kennedy, M.P., and Tan, S.S., 2007), the site is generally underlain by alluvial flood plain deposits (map unit Qa). In addition, cretaceous rocks (map unit Kt) and sedimentary bedrock of the Santiago Formation (map unit (Tsa) are located in close proximity to the Project site. A geologic map is presented on Figure 3 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.

3.2 SUBSURFACE EARTH MATERIALS

In general, the subsurface soils within the upper 24 feet consisted of loose to medium dense clayey sand, and sand, and medium stiff to very stiff sandy lean clay. The soils below 24 feet, consisted of dense to very dense sands and clayey sands to the maximum depth drilled (47 feet). A summary of the laboratory test results for near-surface soils is presented in Table 3-1.

Table 3-1. Near-Surface Data Summary

Boring Number	Depth (ft)	Soil Type	In-Situ Moisture Content (avg %)	Optimum Moisture Content (%)	Maximum Dry Unit Weight (pcf)	Plasticity Index	Friction Angle	Cohesion (psf)
A-19-001	0-3	SM	-	8.0	132.4	-	-	-
A-19-001	3-6.5	SC	9.8	-	-	15	32	100
A-19-002	5	CL	19	-	-	36	-	-
A-19-003	0-5	SC	7.6	-	-	-	-	-
A-19-003	5-6.5	SM	-	-	-	NP	-	-
HA-19-004	0-5	SC	11.1	-	-	16	-	-
A-19-005	5	SC	8.5	-	-	-	31	150
HA-19-006	3-4.5	SW-SM	19.1	-	-	NP	-	-
HA-19-007	0-5	SM	7.2	-	-	-	-	-
A-19-008	0-3	SM	-	7.3	134.1	-	-	-
A-19-008	3-5	SC	17.5	-	-	-	28	400

3.3 GROUNDWATER

Groundwater was encountered at depths ranging from about 2.5 to 19.3 feet bgs (corresponding to groundwater table elevations ranging from about 290 to 326.5 feet North American Vertical Datum of 1988 [NAVD88]) during the current field investigation. Previous groundwater monitoring well data reported by Merkel & Associates, Inc (2010) between May and October 2010 indicates measurements to depth to groundwater ranging from about 8.4 to 13.5 feet bgs (corresponding to groundwater table elevations ranging from about 299.9 to 310.4 feet NAVD88]).

A groundwater level monitoring well was constructed within Boring A-19-003. The well extends to a depth of approximately 47 feet bgs. The intent of this well is to monitor the levels of stabilized groundwater in disturbed areas west of Roman Creek. During the drilling of Boring A-19-003 and construction of the well on April 24, 2019, groundwater was encountered at approximately 17.5 feet bgs. Groundwater was measured at a depth of about 8 feet bgs in the well on April 25, 2019. Groundwater information from our field exploration is presented in Table 3-2.

Table 3-2. Summary of Groundwater Information

Boring Number	Boring Elevation (feet- NAVD88)	Boring Depth (feet)	Boring Bottom Elevation (feet-NAVD88)	Groundwater	
				Depth (feet)	Elevation (feet-NAVD88)
A-19-001	309.0	40.5	268.5	19.0	290.0
A-19-002	310.0	25.8	284.2	19.3	290.7
A-19-003	311.0	47.1	264.0	17.5	293.5
HA-19-004	315.0	5.0	310.0	NE	--
A-19-005	318.0	26.5	291.5	16.0	302.0
HA-19-006	329.0	4.5	324.5	2.5	326.5
HA-19-007	330.0	5.0	325.0	NE	--
A-19-008	329.0	25.9	303.1	14.0	315.0

Note:
NE: Not Encountered; NAVD 88: North American Vertical Datum of 1988

Fluctuations of the Roman Creek surface water levels, 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.

3.4 PERCOLATION TESTING

Percolation testing was performed within Borings A-19-001, A-19-002, and A-19-005 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 2-inch diameter pipe was installed in the boreholes 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 drop in water level was measured at approximately in 30-minute intervals. The percolation testing was then performed by measuring the infiltration of the water over time. The test was performed until a stabilized rate was achieved. 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 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 the Porchet Method. Test data is presented in Appendix B and summarized in Table 3-3.

Table 3-3. Percolation Test Data Summary

Test Location	Test Depth (feet)	Infiltration Rate (in/hr)	USCS Soil Type
A-19-001	3 to 5	0.07	SC
A-19-002	3 to 5	0.3	SC
A-19-005	8.5 to 10	5.6	SC

Note: USCS = Unified Soil Classification System

The infiltration rates presented on Table 3-3 do not contain a factor of safety. A factor of safety of at least 2.0 is recommended by Caltrans (2011a) and others. Clayey sands and sandy clay soils 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's Civil Engineer. The designer should take into consideration the variability of the native soils when selecting factors of safety, storage, and other design elements.

3.5 Engineering Properties of Subsurface Materials

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

3.5.1 Shear Strength

Based on the direct shear test results on selected samples collected from our borings, the cohesion intercept (c) and friction angle (ϕ) representing the effective ultimate shear strength of the on-site soils were found to range from about 100 psf to 400 psf and 28 degrees to 32 degrees, respectively.

3.5.2 In-situ Moisture Content and Density and Compaction

Samples collected in the upper 5 feet throughout the Project were tested for in-situ moisture content and density. Two compaction tests were also performed on select soil samples in accordance with ASTM D1557.

In-situ moisture contents and dry densities ranged from 7 to 19 percent (13 percent average) and from 111 to 127 pcf (119 average), respectively. Laboratory optimum moisture contents and maximum dry densities ranged from 7 to 8 percent (7 percent average) and from 132 to 134 pcf (133 average), respectively.

3.5.3 Expansive Soils

Two locations (A-19-001 and A-19-008) were tested for expansion index (EI). The EI test indicates the tendency of the soil to expand when wetted or contract when dried. The result of two tests indicated that the soil in the upper five feet had EI of 0 and 3, corresponding to a very low expansion potential. The expansion potential of the final subgrade soils should be evaluated and the findings presented herein should be confirmed and/or modified as necessary.

3.5.4 Corrosion Potential

Selected samples of the subsurface soils were subjected to analytical testing to evaluate the potential for corrosion to concrete and ferrous metals. These tests are only an indicator of soil corrosion potential for the samples tested. Other soils at the Project site may be more, less, or of a similar corrosive nature. The test results are included in Appendix C and are summarized in Table 3-4.

Caltrans corrosion guidelines (2018) define a corrosive soil as a material in which any of the following conditions exist: a chloride content greater than 500 ppm; soluble sulfate content greater than 1,500 ppm; or a pH of 5.5 or less. Based on the guidelines established by Caltrans, the subsurface soils at the site are not considered corrosive towards concrete.

Table 3-4. Summary of Corrosion Test Results

Boring No.	Sample Depth (feet)	pH	Minimum Resistivity (ohm-cm)	Sulfates (ppm)	Chlorides (ppm)
A-19-002	5	7.6	1,120	11	8
A-19-005	5	7.8	5,214	37	39
A-19-008	3.5	7.7	960	216	74

Notes:

ohm-cm = ohm centimeters; ppm = parts per million.

Although electrical resistivity is only one factor in corrosion, resistivity measurements permit classification of relative corrosion potential. The relative level of corrosion potential, commonly accepted by the engineering community as indicated by resistivity levels, is shown in the table below:

Range in Electrical Resistivity	Relative Corrosion Potential (Ferrous Metals)
Less than 700 ohm-cm	Very corrosive
700 to 2,000 ohm-cm	Corrosive
2,000 to 5,000 ohm-cm	Moderately corrosive
5,000 to 10,000 ohm-cm	Mildly corrosive
Greater than 10,000 ohm-cm	Non-corrosive

Based on the measured resistivity of the tested soil samples, the subsurface soils are considered mildly corrosive to corrosive to buried ferrous metals. Imported fill materials should be tested to confirm that their corrosion potential is not more severe than that noted herein. For sensitive buried metallic elements, a corrosion engineer should be consulted.

3.6 FAULTING AND SEISMICITY

3.6.1 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 moment magnitude, and slip rate (average amount of slip per year). The data was developed by the U.S. Geological Survey (2006) for a probabilistic seismic hazard analysis and refined by Caltrans (2017). A fault map with the nearby faults is provided on Figure 4 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 (RCFZ) Oceanside Section	9.0	6.8	1.1
RCFZ Del Mar Section	10.3	6.8	1.1
Newport-Inglewood (Offshore)	10.6	6.9	0.8 – 2.1
Elsinore Julian Section	18.9	7.7	4.0
Elsinore Temecula Section	18.9	7.7	4.0
RCFZ San Diego Section	20.3	6.8	1.1
Coronado Bank	24.7	7.4	2.0
Elsinore Glen Ivy Section	32.5	7.7	4.0
San Jacinto (Anza)	44.0	7.7	7-17
San Jacinto (San Jacinto Valley-Southern Ext.)	45.9	7.7	7-17

Notes:

1. R_{rup} = closest distance from the site to fault rupture plane which is calculated using Caltrans (2017) methodology.
2. Slip rates are estimates, provided by Southern California Earthquake Data Center (2018)

3.6.2 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, 14 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
Southern California	1920	5.0
Greater Los Angeles Area	1923	6.0
Long Beach, California	1933	6.4
Newport Beach, California	1933	5.3
Trabuco Canyon, California	1938	5.2
Pine Valley, California	1940	5.0
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

3.7 SEISMIC HAZARDS

3.7.1 Fault Rapture

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

3.7.2 Liquefaction

The term liquefaction describes a phenomenon in which saturated, cohesionless soils temporarily lose shear strength (liquefy) due to increased pore water pressures induced by strong, cyclic ground motions during an earthquake. Structures founded on or above potentially liquefiable soils may experience bearing capacity failures due to the temporary loss of foundation support, vertical settlements (both total and differential), and/or undergo lateral spreading. The factors known to influence liquefaction potential include soil type, relative density, grain size, confining pressure, depth to groundwater, and the intensity and duration of the seismic ground shaking. Liquefaction is most prevalent in loose to medium dense, silty, sandy, and gravelly soils below the groundwater table.

Due to the anticipated relatively shallow depth to groundwater (within 20 feet bgs) and the soil types present, the potential for liquefaction at the Project site exists. The liquefaction evaluation was conducted using a peak horizontal ground acceleration of 0.5g weighted for a Moment Magnitude (Mw) of 6.6 and a design groundwater level of 8 feet bgs. Based on this evaluation, granular subsurface soils between approximate depths of 8 to 24 feet bgs are susceptible to liquefaction. The liquefaction potential should be further evaluated during final design once specific design elements and their location within the Mitigation Site become available.

3.7.3 Seismically-Induced Settlement

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. Dry dynamic settlement is considered relatively small due to the presence of high groundwater table. The liquefaction-induced settlement was estimated to be about 1.4 inches for the site. This evaluation should be confirmed or modified during final design after conducting field investigation.

3.7.4 Lateral Spreading

Liquefaction-induced lateral spreading is defined as the finite lateral displacement of ground as a result of pore pressure build-up or liquefaction in shallow underlying soils during an earthquake. Lateral spreading can occur on sloping ground or where nearby steep banks are present. The Roman Creek is located in close proximity to the site. The Roman Creek includes areas of relatively flat and vegetated bottomlands with slightly sloping streambanks that transition to upland habitats. Based on the site configuration, liquefaction-induced lateral spreading potential exists at the site. Lateral spreading should be evaluated during the final design once specific design elements and their location within the Mitigation Site become available.

3.7.5 Seiches and Tsunamis

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. Based on the absence of enclosed bodies of water near the site, seiche and tsunami risks at the site are considered negligible.

3.7.6 Earthquake-Induced Flooding

Earthquake-induced flooding is caused by dam failures or other water-retaining structure failures as a result of seismic shaking. A review of the San Diego County General Plan, Dam Inundation Map (2011) indicates that the site is not located within a dam inundation area. Earthquake-induced flooding is considered low.

3.8 SEISMIC DESIGN CRITERIA

To reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the applicable building codes. The seismic parameters were calculated using United States Geological Survey (USGS) U.S. Design Maps Application (USGS, 2020) and in accordance with the 2016 California Building Code (CBC) and ASCE/SEI 7-16 (2016). According to ASCE/SEI 7-16, sites subject to liquefaction should be classified as Site Class F, which requires a site-specific response analysis. However, ACSE/SEI 7-16 states that for a short period (less than ½ second) structure on liquefiable soils, Site Class D or E may be used instead of Site Class F to estimate design seismic loading on the structure. The selection of Site Class D or E is based on the assessment of the site soil profile assuming no liquefaction.

For structures with a period shorter than ½ second, seismic design parameters for Site Class D are provided in Table 3-7. Structures that have a period longer than ½ second will require a site-specific response analysis.

Table 3-7. Preliminary Seismic Design Parameters

Category	Coefficient
Site Class	D
Latitude	33.15248
Longitude	-117.24644
Mapped (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_S	0.916
Mapped (5% damped) spectral response acceleration parameter at long period (1.0 sec), S_1	0.337
Short period (0.2 sec) site coefficient, F_a	1.134
Long period (1.0 sec) site coefficient, F_v	1.963
Spectral response acceleration parameter at short period (0.2 sec), S_{MS}	1.038
Spectral response acceleration parameter at long period (1.0 sec), S_{M1}	0.662
Design (5% damped) spectral response acceleration parameter at short period (0.2 sec), S_{DS}	0.692
Design (5% damped) spectral response acceleration parameter at long period (1.0 sec) S_{D1}	0.441
Peak Ground Acceleration (PGA) (g)	0.397
Geometric Mean PGA (PGA_M) (g)	0.478
Design Magnitude(1) Mw	6.6

(1) Design magnitude based on USGS Probabilistic Deaggregation with 2% chance of exceedance in 50 years (2,475 year return interval) (USGS, 2020).

3.9 FLOODING

Our review of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM) 06073C0786J and 06073C0788J (FEMA, 2012a and 2012b) found that the Project area is located within an area designated as Zone X (areas determined to be outside the 0.2% annual chance of flood).

3.10 SCOUR POTENTIAL

The proposed pedestrian bridge is located within the Roman Creek, scour potential should be considered, and evaluated during final design.

4.0 SLOPE STABILITY

Based on the preliminary information provided by the design team, maximum slope ratios of 2H:1V are anticipated for permanent cut slopes of up to 4 feet high. Taller slopes (up to 13 feet high) are also considered, but with slope inclinations of about 6H:1V or flatter.

The proposed slopes were evaluated using the limit equilibrium method (Spencer's method) available in SLOPE/W slope stability software (Geo-Slope International, 2018). For our slope stability analyses, we have considered three cross sections (A-A', B-B', and C-C'). These cross sections represent typical slope design scenarios for the Project. Approximate locations of cross sections are presented on Figure 2 in Appendix A. Design groundwater level was modeled at approximately groundwater elevation of about +303 feet NAVD 88 for cross sections A-A', +312 feet NAVD88 for cross section B-B', and +315 feet NAVD88 for cross section C-C'. "Entry and Exit" feature in SLOPE/W software was used to search for the critical slip surface in all analyses. The subsurface soil conditions and soil strength parameters used in the slope stability analyses were established based on the results of our geotechnical investigation in the vicinity of the area of interest.

For pseudo-static analysis, a horizontal seismic coefficient (k_h) of 0.15 was used in accordance with the recommendations from the Southern California Earthquake Center (SCEC) DMG Special Publication 117 (2002). The resultant factors of safety from our slope stability analyses were compared to acceptable industry standards of 1.5 for static conditions and 1.1 for seismic conditions.

Our analysis indicates that the slopes have a global factor of safety of at least 1.5 and 1.1 for static and pseudo-static conditions. Additionally, slope stability was analyzed in conjunction with liquefaction. Based on the results of our slope stability analyses, the proposed cut slopes are considered globally stable provided the recommendations in Section 5.2 of this report are met. Graphical outputs of the slope stability analyses are presented in Appendix D.

Table 4-1. Summary of Slope Stability Analysis Results

Cross Section	Proposed Slope Type	Analyzed Slope Geometry	Calculated Static Factor of Safety	Calculated Pseudo-Static Factor of Safety	Calculated Pseudo-Static with Liquefaction Factor of Safety
A-A'	Cut	6H:1V	4.2	2.0	1.1
B-B'	Cut	6H:1V	3.8	1.8	1.1
C-C'	Cut	2H:1V	2.4	1.7	1.4

5.0 GEOTECHNICAL RECOMMENDATIONS

Geotechnical recommendations presented in this report are based on the conditions encountered at the test boring locations and our understanding of the current Project plan. Our recommendations may need to be revised, as necessary, based on the actual soil conditions or any modification of the current plans, and incorporated into the final design plans and specifications. Conclusions and recommendations presented in this report should be reviewed and verified by the Geotechnical Engineer during site construction and revised accordingly if exposed geotechnical conditions vary from our current understanding and interpretations.

5.1 BRIDGE FOUNDATIONS

It is our understanding that a pre-fabricated 110-foot long pedestrian bridge by Western Wood Structures, Inc. is proposed to be installed. Borings were not drilled at the location of the proposed bridge. However, for estimation purposes, data from nearby borings may be used to establish approximate foundation parameters for the design of the proposed bridge. Depending on the anticipated loading, either a shallow or deep foundation system may be considered to support the pre-fabricated bridge.

Data from nearby borings indicate that subsurface soils near the proposed bridge location consist of medium dense sands with 10-foot thick clay layer at approximately 15 feet below ground surface. The borings drilled during this investigation indicate liquefiable soils may be encountered between approximately 8 to 24 feet bgs. If the estimated total settlements (static and seismic) as described in Section 3.7.3 are in excess of the tolerable settlement for the pedestrian bridge, the liquefaction potential should be mitigated. Alternatives for liquefaction mitigation may include ground improvement (i.e., vibro-replacement dry stone columns, compaction grouting, deep soil mixing) or using a deep foundation system that extends below the bottom of the liquefiable layer.

5.1.1 Shallow Foundations

For the anticipated lightly loaded structure, a shallow foundation system may be used provided that these are founded on firm and unyielding soils. For preliminary foundation design purposes, an allowable bearing capacity of 2,500 psf may be used for spread footing design with a minimum footing width of 2 feet, and provided that the foundations are constructed in accordance with the recommendations in Section 6.1, including overexcavation and the use of geogrid and crushed rock. These values may be increased by one-third when considering loads of short duration, such as those imposed by wind or seismic forces.

A minimum overexcavation of 3 feet below the bottom of footing will be required and replaced with geogrid-reinforced (Tensar TriAx TX5 geogrid or similar) backfill. The lateral limits of the overexcavation should extend at minimum 3 feet beyond the edge of footings.

It should be noted that the above described overexcavation and replacement method may reduce the effects of liquefaction, but it does not completely eliminate them. After a seismic event, repairs or replacement may be necessary.

5.1.2 Deep Foundations

As an alternative to shallow foundations, a deep foundation system may be used consisting of either cast-in-drilled-hole (CIDH) piles or driven steel piles. CIDH piles may be difficult to construct due to the presence of groundwater and potential for caving of granular soils. Consideration should be given to noise in the event of selecting driven piles.

For preliminary purposes, axial capacities were calculated for a 1-foot diameter CIDH pile using the Ensoft software SHAFT (2017). An ultimate axial capacity chart is provided in Appendix D. It should be noted that the calculation was performed for a 1-foot diameter CIDH and neglecting end bearing. Capacities for other diameters can be extrapolated linearly. The pile should at minimum be embedded past the anticipated liquefiable layer (minimum depth of 25 feet bgs). Additionally, based on the limited data currently available, the pile capacity chart extends to a maximum depth of approximately 37 feet bgs. Specific recommendations for other deep foundation alternatives and/or deeper pile depths can be provided once site specific exploration is performed.

5.2 SLOPE DESIGN

Permanent cut slopes should be constructed no steeper than 2H:1V. Cut slopes must be observed during construction by the Geotechnical Engineer. In cases where fill slopes are considered, slope inclination should be limited to 2H:1V or flatter. The specific heights of cut/fill slopes should be evaluated by the geotechnical consultant during final design.

5.3 CEMENT TYPE AND CORROSION MEASURES

A discussion of soil corrosion results is included in Section 3.5.4. The test results included in this report should only be used as a screening process for an indication of soil corrosivity. In general, foundation elements should be designed for a corrosive environment toward buried ferrous metals, and a non-corrosive environment for buried concrete structures. Type II Portland Cement is an appropriate concrete type for the Project, and appropriate strength and mix requirements should be selected based on individual structures' design life and structural requirements.

6.0 CONSTRUCTION CONSIDERATIONS

6.1 EARTHWORK

Prior to construction, the site should be cleared of all existing improvements and debris. Existing utility and irrigation lines should also be removed 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.

If soft, pumping or yielding subgrade is exposed during grading, stabilization methods may be required. Additionally, the bottom of the overexcavation may also be difficult to compact using conventional methods of fill placement and compaction. The contractor should consider the moisture conditions when selecting equipment for earthwork and compaction. During seasonal rains, handling of saturated soils may pose problems in equipment access and cleanup. These conditions could seriously impede grading by causing an unstable subgrade condition. Typical remedial measures include the following:

- Drying: Drying unstable subgrade involves disking or ripping wet subgrade to a depth of approximately 18 to 24 inches and allowing the exposed soil to dry. Multiple passes of the equipment (likely on a daily basis) will be needed because as the surface of the soil dries, a crust forms that reduces further evaporation. Frequent disking will help prevent the formation of a crust and will promote drying. This process could take several days to several weeks depending on the depth of ripping, the number of passes, and the weather. Given the fine-grained soils onsite and high moisture content, this may not be a practical solution.

- Removal and Replacement with Crushed Rock and Geotextile Fabric: Unstable subgrade could be over-excavated 18 to 24 inches below planned excavation depth and replaced with crushed rock ranging from ¾ inch to 2 inch in size, underlain by geotextile fabric. The geotextile fabric should consist of a woven geotextile, such as Mirafi 600X or equivalent. The final depth of removal will depend upon the conditions observed in the field once overexcavation begins. The geotextile fabric should be placed in accordance with the manufacturer's recommendations.

The on-site soil is suitable to be used as fill, provided it is free of organics, debris, and oversize particles (e.g., cobbles, boulders, rubble, etc.) larger than 6 inches in the largest dimension. Any imported fill soil should be approved by the Geotechnical Engineer prior to placement as fill. Import soils should be uncontaminated, granular in nature, and free of organic material and should have very low expansion potential (with an Expansion Index less than 21 per ASTM D 4829) and a low corrosion impact to the proposed improvements. Fill soils should be placed in loose lifts not exceeding 8 inches, moisture-conditioned as necessary to optimum or slightly above, and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM D1557.

6.2 PILE CONSTRUCTION

Performance of CIDH piles is heavily dependent on construction methods and procedures. Construction methods that create large zones of disturbance around the drilled shafts can lead to lower than expected skin friction due to excessive stress relief around the shaft length. The pile foundations should be constructed only by qualified contractors experienced in this type of construction, and under strict construction monitoring and quality control. Based on the existing borings and well data within the vicinity of the site, free groundwater is expected to be encountered at depths of approximately 8 feet bgs. The piling contractors should carefully review the boring logs and perform their own assessment of potential construction difficulties. It should be noted that boring logs at the time of this report are for locations about at least 100 feet away from the proposed bridge. Additionally, information was only collected to a depth of about 47 feet bgs.

The CIDH piles will be constructed partly in granular (sandy) soils; therefore, caving should be anticipated and will need to be prevented. Polymer slurry or a combination of temporary casing and polymer slurry may be required to stabilize the sides of the CIDH pile excavation. The use of alternative excavation methods must be subject to review by the geotechnical engineer for compatibility with the design assumptions. This should be carefully considered by the contractor in selecting construction methods, equipment, casing, drilling fluid, and other means and methods for constructing a quality pile.

The concrete for the CIDH piles should be placed using a down-hole tremie, or similar provision, such that the falling concrete does not strike the sides of the shaft. Once concrete pumping is initiated, a minimum head of 5 feet of concrete above the bottom of the tremie should be established and maintained throughout the concrete placement to prevent contamination of the concrete (soil inclusions). If steel casing is used, the casing should be removed slowly, and the minimum concrete head maintained to prevent soil caving and "necking" of the pile as the concrete is placed. Concrete should be placed in newly excavated pile shafts as soon as practical. The pile excavation should not be allowed to remain open for more than 12 hours. The concrete must be capable of propagating between the reinforcing bars to come in contact with the soil and avoid arching during extraction of the casing. The reinforcing cage should be placed carefully in the hole in a manner such that the soil is not disturbed.

The quality of construction is of primary importance in the construction of drilled, cast-in-place piles. The timely placing of concrete and the installation within specified tolerances must be respected. The pile must remain within 2 inches of the theoretical plan location and remain within 2 percent of vertical, as measured from the as-constructed position.

Maintenance of the full design cross-section for the entire pile length is a concern if and when casing is extracted during pile casting. Sometimes the suction created by pulling the casing allows soil intrusion into the shaft resulting in reduced pile cross-section. Post-construction evaluation of the piles using non-destructive testing should be considered. All piles should be subjected to Gamma-Gamma testing and/or cross-hole sonic log testing. Plastic tubing should be installed in all the CIDH piles in the event defective piles are detected so the remainder of the piles supporting that column can be evaluated. The structural engineer should detail the number and location of inspection tubes for CIDH piles.

6.3 SLOPE CONSTRUCTION

Permanent cut and/or fill slopes should be constructed no steeper than 2H:1V. Fill should be compacted to a minimum of 90 percent of the maximum dry density as determined by ASTM D1557, as outlined in Section 6.1. In order to achieve compaction, the fill slopes may be overbuilt and cut back to final grade, or they may be surface rolled to provide a compacted finished surface.

Cut slopes must be observed during construction by the Geotechnical Engineer. Any loose, fractured, or otherwise unsuitable material that may be exposed during slope construction should be overexcavated and replaced with compacted fill. If, during the course of grading, adverse or potentially adverse geotechnical conditions are encountered in the slopes, the Geotechnical Engineer shall evaluate the conditions and provide appropriate recommendations.

Runoff should not be permitted to flow over cut or fill slopes in such a way as to cause erosion. In order to reduce or mitigate erosion for proposed slopes, a drainage channel should be constructed at the top of slopes. Riprap may need to be considered along cut/fill slopes within drainage areas to mitigate the effects of scour and erosion. Additionally, slope cover consisting of drought-tolerant plants or native vegetation is desirable to provide protection from erosion and provide surficial slope stability.

6.4 GROUNDWATER CONTROL

Based on the current field exploration, groundwater was encountered in our borings at approximate depths between 2.5 to 19 feet bgs. Surface water is anticipated within the Roman Creek. 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. In an event of encountering significant groundwater, the contractor may implement a specific dewatering system. Dewatering systems should be designed and installed by a specialty contractor. If CIDH pile type is selected, groundwater control and installation using the wet method will likely be required.

6.5 ADDITIONAL GEOTECHNICAL SERVICES

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

- Over-excavation and soil removal and/or exposed excavation bottom;
- Placement of compacted fill;
- Footing construction,
- Pile installation;
- Slope grading; and

- When any unusual subsurface conditions are encountered.

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

7.0 LIMITATIONS

This report has been prepared for the use of HDR and City of Vista for the proposed Roman Creek Mitigation Project. The report may not be used by others without the written consent of our client and our firm. The findings and conclusions presented in this report have been based on 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.

Our geotechnical scope of services did not include a site specific environmental assessment or evaluations regarding the presence or absence of hazardous substances in the soil. Additionally, the conclusions 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.

8.0 REFERENCES

The following references were used in preparation of this report:

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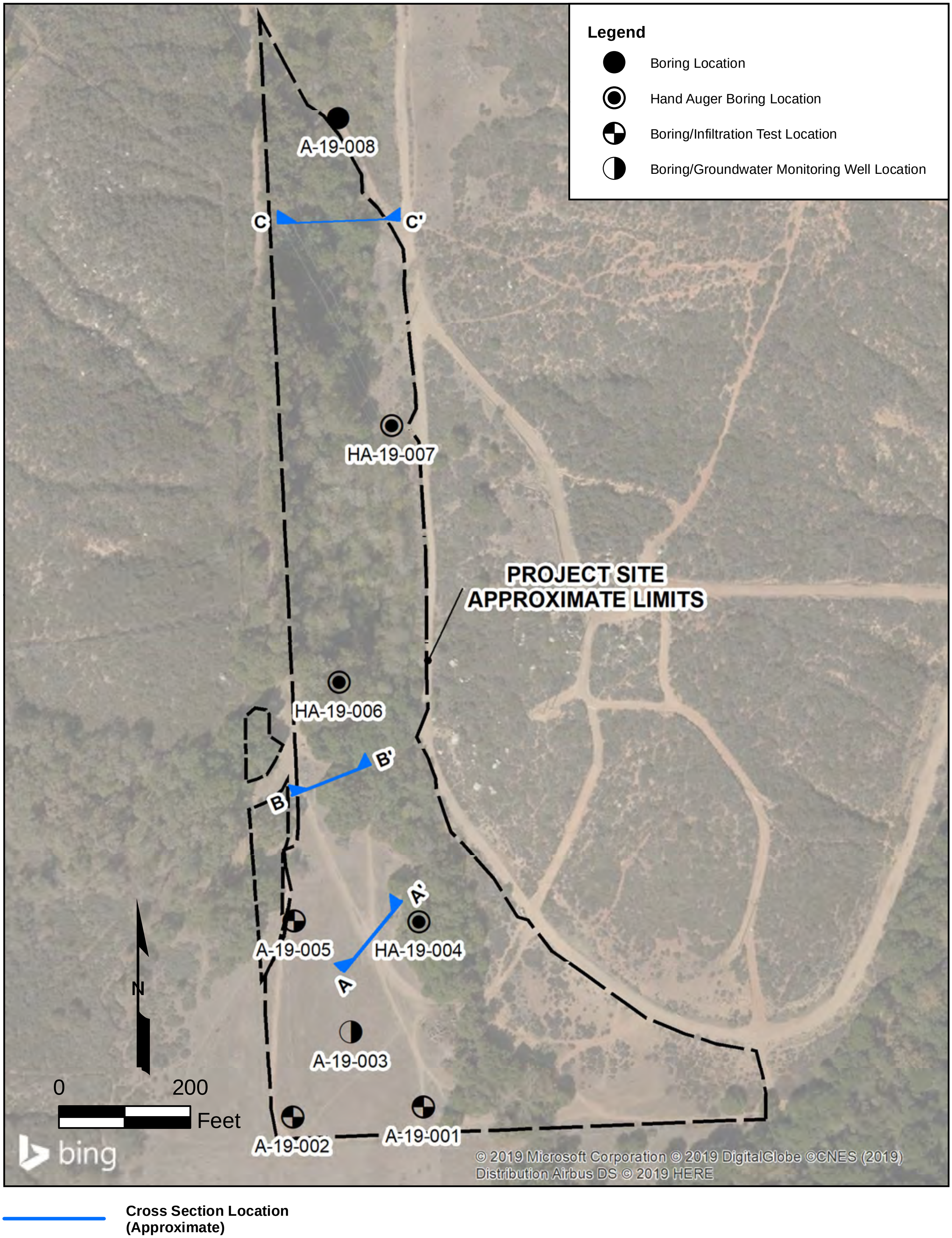
Appendix A
Figures

SITE LOCATION MAP
ROMAN CREEK MITIGATION PROJECT
VISTA, CALIFORNIA

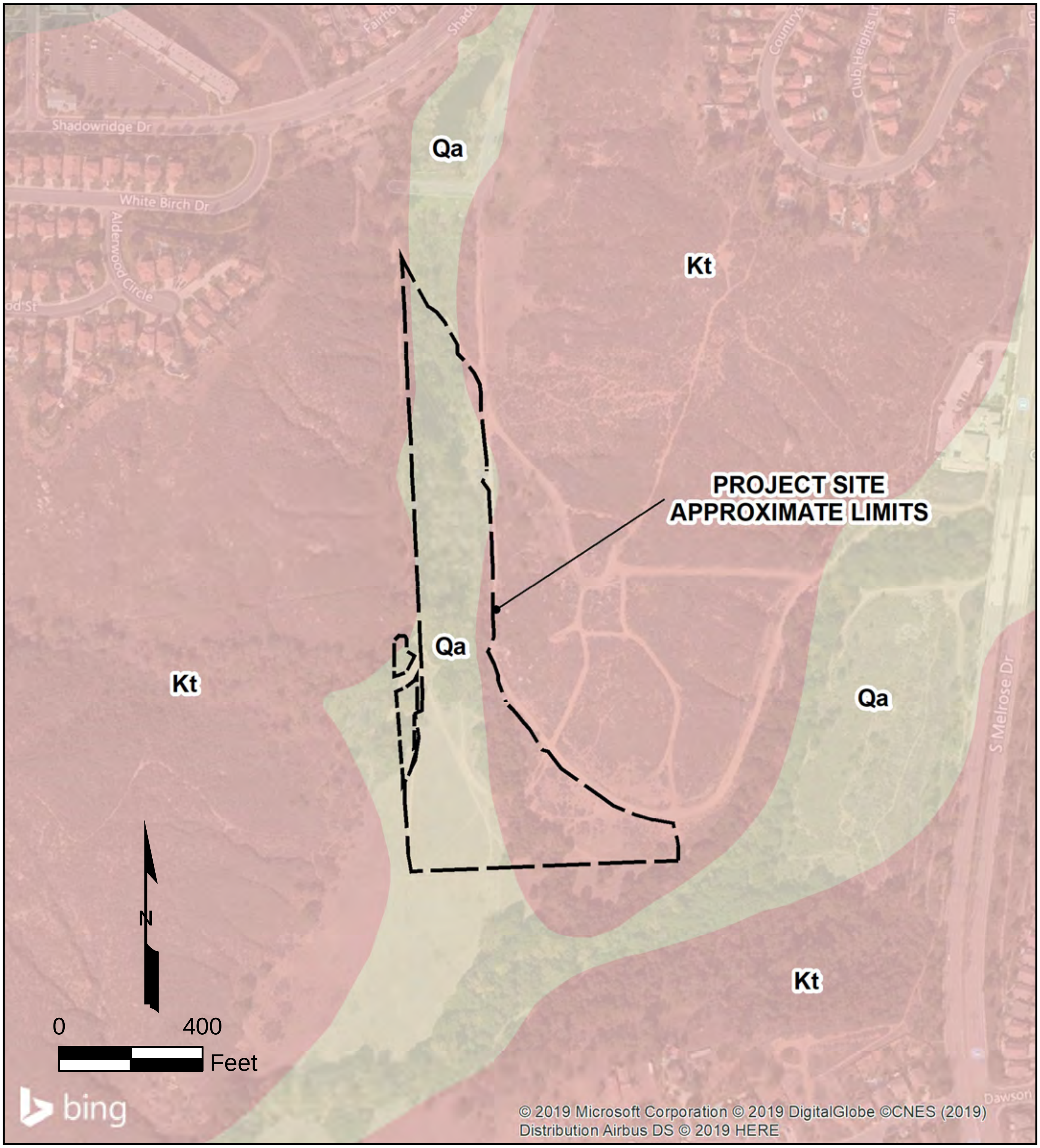
HDR

Figure 1



BORING LOCATION MAP
ROMAN CREEK MITIGATION PROJECT
VISTA, CALIFORNIA

Figure 2

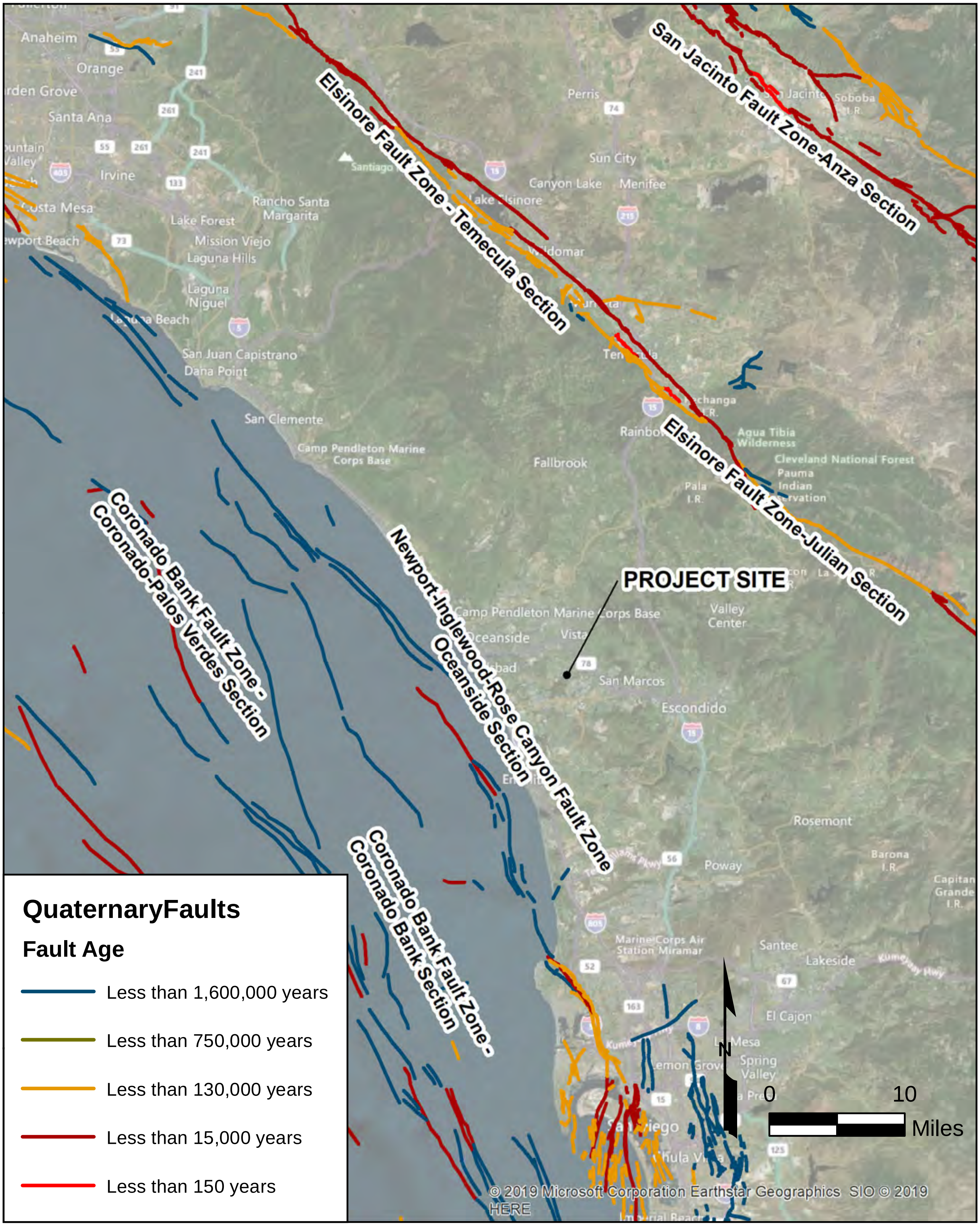


Reference: Kennedy, M.P., Tan, S.S., 2007, Geologic Map of Oceanside

Kt	Tonalite, undivided (mid-Cretaceous): Massive, coarse-grained, light-gray hornblende-biotite tonalite
Tsa	Santiago Formation (middle Eocene): Coarse-grained Tertiary age formations of sedimentary origin. Sandstone, conglomerate and claystone
Qa	Alluvial flood-plain deposits (late Holocene): Active and recently active flood-plain deposits. Consists of unconsolidated sandy, silty, or clay-bearing alluvium

GEOLOGY MAP
ROMAN CREEK MITIGATION PROJECT
VISTA, CALIFORNIA

Figure 3



FAULT MAP
ROMAN CREEK MITIGATION PROJECT
VISTA, CALIFORNIA

Figure 4

Appendix B

Field Investigation

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

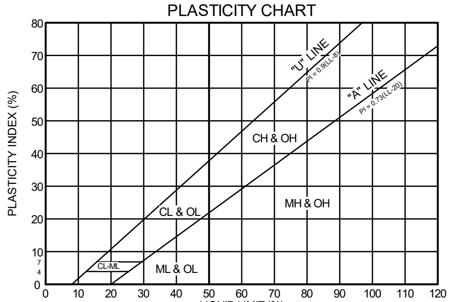
U.S. STANDARD SIEVE	200	40	10	4	3/4"	3"	12"
SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

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).

PLASTICITY CHART



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%

Boring Legend

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Figure

LOGGED BY: MF | DATE: START 4/24/19 END 4/24/19

STATION & OFFSET: NA, NA | LATITUDE: 33.15205 | LONGITUDE: -117.24656 | ELEVATION (ft): 309

DRILL RIG: CME-75 | DRILL METHOD: HSA | DRILLING COMPANY: 2R Drilling | BOREHOLE DEPTH (ft): 40.5

CASING TIP DEPTH: NA | BIT DIAMETER: 8" | GROUNDWATER DATA: DEPTH: 19.0 ft DEPTH:

HAMMER TYPE: Automatic | HAMMER EFFICIENCY: 81% | NOT ENCOUNTERED ☐ | TIME: | TIME:

CHECKED BY (DATE): MG | 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); dark grayish brown; moist; coarse to fine SAND; few mica; trace GRAVEL	Bulk sample collected from 0 to 5 feet
						17			MD EI SA			
305				5 8 14			127	8.9	DS		Clayey SAND (SC); medium dense; dark grayish brown; moist; medium to fine SAND; few mica; low plasticity CLAY	
	5			9 15 21		34	124	10.6	PI HY			
300				3 5 5		30		20.3	SA			
	10										increased moisture content	
295	15			4 6 12			109	21.4			dark gray	
290								▽			wet	
	20			3 7 26		20			SA		dense; olive brown; wet; coarse to fine SAND; trace GRAVEL	
285											Poorly-graded SAND with SILT (SP-SM); very dense; olive brown to gray; wet; coarse to fine SAND; few mica	
	25			50/4"								
280	30											

HDR BORING LOG IRVINE 2019-05 CITY OF VISTA-ROMAN CREEK.GPJ FOLSOM 3-30-11.GDT 6/19/19

HDR

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
A-19-001

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			50/4"								
275												
	35			50/5"							olive gray to brown	
270												
	40			50/6"								

Boring terminated at 40.5 feet bgs due to drilling refusal.
Groundwater measured at 19 feet bgs.
Boring backfilled with cement-bentonite grout.

HDR BORING LOG IRVINE 2019-05 CITY OF VISTA-ROMAN CREEK.GPJ FOLSOM 3-30-11.GDT 6/19/19

HDR

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
A-19-001

LOGGED BY: MF | DATE: START 4/24/19 END 4/25/19

STATION & OFFSET: NA, NA | LATITUDE: 33.15200 | LONGITUDE: -117.24721 | ELEVATION (ft): 310

DRILL RIG: CME-75 | DRILL METHOD: HSA | DRILLING COMPANY: 2R Drilling | BOREHOLE DEPTH (ft): 25.8

CASING TIP DEPTH: NA | BIT DIAMETER: 8" | GROUNDWATER DATA: DEPTH: 19.3 ft DEPTH:

HAMMER TYPE: Automatic | HAMMER EFFICIENCY: 81% | NOT ENCOUNTERED ☐ | TIME: | TIME:

CHECKED BY (DATE): MG | 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); brown; moist; medium to fine SAND	
				5 7 9							medium dense	
305	5			5 10 25		50	111	19.0	PI HY		Sandy lean CLAY (CL); very stiff; dark grayish brown; moist; medium to fine SAND; low plasticity	
300	10			7 10 10		34		16.2	PI SA		Clayey SAND (SC); medium dense; dark grayish brown; moist; coarse to medium SAND	
295	15			15 50/6"		25	119	13.4	SA		very dense; olive brown; moist; coarse to fine SAND	
								▽			Silty SAND (SM); very dense; olive brown; moist; medium to fine SAND wet	
290	20			50/5"								
285	25			50 50/3"							Poorly-graded SAND with SILT (SP-SM); very dense; olive gray; wet; coarse to fine SAND; trace CLAY	

Boring terminated at 25.8 feet bgs.
Groundwater measured at 19.3 feet bgs.
Boring backfilled with cement-bentonite grout.

HDR BORING LOG IRVINE 2019-05 CITY OF VISTA-ROMAN CREEK.GPJ FOLSOM 3-30-11.GDT 6/19/19

HDR

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
A-19-002

LOGGED BY: MF				DATE: START 4/24/19	END 4/24/19
STATION & OFFSET: NA, NA		LATITUDE: 33.15236	LONGITUDE: -117.24693	ELEVATION (ft): 311	
DRILL RIG: CME-75	DRILL METHOD: HSA	DRILLING COMPANY: 2R Drilling		BOREHOLE DEPTH (ft): 47.083	
CASING TIP DEPTH: NA	BIT DIAMETER: 8"	GROUNDWATER DATA:		DEPTH: 17.5 ft	DEPTH:
HAMMER TYPE: Automatic	HAMMER EFFICIENCY: 81%	NOT ENCOUNTERED ☐		TIME:	TIME:
CHECKED BY (DATE): MG	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
310	0	Bulk				33		7.6	SA		Clayey SAND (SC); dark grayish brown; moist; coarse to fine SAND	Bulk sample collected from 0 to 5 feet
305	5			5 6 7		18			PI SA		Silty SAND (SM); medium dense; olive brown; moist; coarse to fine SAND	
300	10			5 8 10		30		17.7	SA		Clayey SAND (SC); medium dense; olive brown; moist; coarse to fine SAND; increased moisture	
295	15			4 6 7	2.0	54	105	22.2	PI		Sandy lean CLAY (CL); stiff; dark grayish brown; moist; medium to fine SAND; low plasticity wet	
290	20			4 6 8								
285	25			50/5"		27			SA		Clayey SAND (SC); very dense; olive gray; wet; coarse to fine SAND; few mica	Difficult drilling 25-30 feet
	30											

HDR

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
A-19-003

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
	30			50/3"								Slow drilling 30-50 feet. Very little recovery
280												
	35			50/1"								
275												
											Poorly-graded SAND with SILT (SP-SM); very dense; olive gray; wet; medium to fine SAND; few mica; few CLAY	
	40			50/2"								
270												
	45			50/3"								
265												
				50/1"								Rig chatter at 47 feet

Boring terminated at 47.1 feet bgs due to drilling refusal.
Groundwater measured at 17.5 feet bgs.
Boring converted to monitoring well.

HDR

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
A-19-003

LOGGED BY: MG | DATE: START 4/25/19 END 4/25/19
STATION & OFFSET: NA, NA | LATITUDE: 33.15282 | LONGITUDE: -117.24659 | ELEVATION (ft): 315
DRILL RIG: Hand Auger | DRILL METHOD: Hand | DRILLING COMPANY: HDR | BOREHOLE DEPTH (ft): 5
CASING TIP DEPTH: NA | BIT DIAMETER: 4" | GROUNDWATER DATA: | DEPTH: | DEPTH:
HAMMER TYPE: | HAMMER EFFICIENCY: % | NOT ENCOUNTERED ☒ | TIME: | TIME:
CHECKED BY (DATE): MG | 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); grayish brown; moist; coarse to fine SAND; little fines	
						31		11.1	PI SA			
310	5											

Hand auger terminated at 5 feet bgs.
Groundwater not encountered.
Boring backfilled with soil cuttings.

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
HA-19-004

LOGGED BY: MF		DATE: START 4/24/19	END 4/25/19
STATION & OFFSET: NA, NA	LATITUDE: 33.15280	LONGITUDE: -117.24716	ELEVATION (ft): 318
DRILL RIG: CME-75	DRILL METHOD: HSA	DRILLING COMPANY: 2R Drilling	BOREHOLE DEPTH (ft): 26.5
CASING TIP DEPTH: NA	BIT DIAMETER: 8"	GROUNDWATER DATA: DEPTH: 16.0 ft	DEPTH:
HAMMER TYPE: Automatic	HAMMER EFFICIENCY: 81%	NOT ENCOUNTERED ☐ TIME:	TIME:
CHECKED BY (DATE): MG	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); brown; moist; medium to fine SAND	
315												
	5			5 12 8		25	120	8.5	DS SA CR		medium dense; trace GRAVEL	
310				5 6 8		28		7.2	PI SA		few mica	
	10											
305												
	15			10 10 12		27	112	17.0 ▽	PI SA		moist; coarse to fine SAND; trace GRAVEL wet	
300												
	20			4 5 7								
295												
	25			10 12 50							Poorly-graded SAND with SILT (SP-SM); dense; olive brown to gray; wet; coarse to fine SAND; trace CLAY; few mica	

Boring terminated at 26.5 feet bgs.
Groundwater measured at 16 feet bgs.
Boring backfilled with cement-bentonite grout.

HDR

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
A-19-005

LOGGED BY: MG | DATE: START 4/25/19 END 4/25/19

STATION & OFFSET: NA, NA | LATITUDE: 33.15382 | LONGITUDE: -117.24700 | ELEVATION (ft): 329

DRILL RIG: Hand Auger | DRILL METHOD: Hand | DRILLING COMPANY: HDR | BOREHOLE DEPTH (ft): 4.5

CASING TIP DEPTH: NA | BIT DIAMETER: 4" | GROUNDWATER DATA: | DEPTH: 2.5 ft | DEPTH:

HAMMER TYPE: | HAMMER EFFICIENCY: % | NOT ENCOUNTERED ☐ | TIME: | TIME:

CHECKED BY (DATE): MG | 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, clayey SAND (SC-SM); grayish brown; moist; coarse to fine SAND	Tree roots
								▽			Clayey SAND (SC); dark grayish brown; moist; coarse to fine SAND wet	
325						5		19.1	PI SA		Well-graded SAND with SILT (SW-SM); yellowish brown; wet	

Hand auger terminated at 4.5 feet bgs.
Groundwater measured at 2.5 feet bgs.
Boring backfilled with soil cuttings.

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
HA-19-006

LOGGED BY: MG | DATE: START 4/25/19 END 4/25/19

STATION & OFFSET: NA, NA | LATITUDE: 33.15490 | LONGITUDE: -117.24675 | ELEVATION (ft): 330

DRILL RIG: Hand Auger | DRILL METHOD: Hand | DRILLING COMPANY: HDR | BOREHOLE DEPTH (ft): 5

CASING TIP DEPTH: NA | BIT DIAMETER: 4" | GROUNDWATER DATA: DEPTH: DEPTH:

HAMMER TYPE: | HAMMER EFFICIENCY: % | NOT ENCOUNTERED [X] | TIME: | TIME:

CHECKED BY (DATE): MG | EFFICIENCY MEASURED [ ] | GW NOT MEASURED [X] | 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); light olive brown; moist; coarse to fine SAND; trace coarse to fine GRAVEL	
											grayish brown	
325	5					22		7.2	SA			

Hand auger terminated at 5 feet bgs.
Groundwater not encountered.
Boring backfilled with soil cuttings.

LOGGED BY: MF | DATE: START 4/25/19 END 4/25/19
STATION & OFFSET: NA, NA | LATITUDE: 33.15618 | LONGITUDE: -117.24703 | ELEVATION (ft): 329
DRILL RIG: CME-75 | DRILL METHOD: HSA | DRILLING COMPANY: 2R Drilling | BOREHOLE DEPTH (ft): 25.9
CASING TIP DEPTH: NA | BIT DIAMETER: 8" | GROUNDWATER DATA: DEPTH: 14.0 ft DEPTH:
HAMMER TYPE: Automatic | HAMMER EFFICIENCY: 81% | NOT ENCOUNTERED ☐ | TIME: | TIME:
CHECKED BY (DATE): MG | 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					33			MD EI SA		Silty SAND (SM); dark gray; moist; fine SAND	Bulk sample collected from 0 to 3 feet
325				7 12 15			112	17.5	DS		Clayey SAND (SC); medium dense; dark gray; moist; fine SAND	
	5			3 3 3		40		18.9	SA		loose	
320	10			5 10 12		45	108	20.2	PI		medium dense; gray; moist; fine SAND	
315								▽			wet	
	15			4 3 4							loose; medium to fine SAND	
310	20			10 50/6"		16	117	11.5	SA		Silty SAND with GRAVEL (SM); very dense; olive brown to olive gray; wet; medium to fine SAND; fine GRAVEL	
305	25			35 50/5"							Poorly-graded SAND with SILT (SP-SM); very dense; olive brown to gray; wet; coarse to fine SAND; few mica	

Boring terminated at 25.9 feet bgs.
Groundwater measured at 14 feet bgs.
Boring backfilled with cement-bentonite grout.

HDR

Boring Log

Roman Creek Mitigation
Vista, CA

Date
JUN 2019

Boring
A-19-008

Boring/Excavation Percolation Testing Field Log Date: 5/6/2019

Project Location	Roman Creek Mitigation	Boring Test Number	A-19-001
Earth Description	Clayey Sand	Diameter of Boring	8"
Tested by	MG	Diameter of Casing	2"
Liquid Description	Clean Tap Water	Depth of Boring	58"
Measurment Method	WLM	Depth to Invert of BMP	N/A
		Depth to Water Table	19
		Dept to Initial Water Depth (d_1)	

Time Interval Standard

Start Time for Pre-Soak	10:03AM (4/24/19)	Water Remaining in Boring	yes
Start Time for Standard	08:00 AM (4/25/19)	Standard Time Interval Between Readings	30 min

Reading Number	Time Start/End (hh:mm)	Elapsed Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/ Comments
1	8:33 9:03	30	36.6 - 36.84 0.24	0.48	Clayey Sand
2	9:03 9:33	30	35.28 - 35.76 0.48	0.96	
3	9:33 10:03	30	34.8 - 35.28 0.48	0.96	
4	10:03 10:33	30	34.08 - 34.56 0.48	0.96	Percolation rate consistent in the last 3 readings. Infiltration Rate = 0.07 in/hr.

Notes:

HDR

Boring/Excavation Percolation Testing Field Log

Date: 4/24/2019

Project Location	Roman Creek Mitigation	Boring Test Number	A-19-002
Earth Description	Clayey Sand	Diameter of Boring	8"
Tested by	MG	Diameter of Casing	2"
Liquid Description	Clean Tap Water	Depth of Boring	58
Measurment Method	WLM	Depth to Invert of BMP	N/A
		Depth to Water Table	19
		Dept to Initial Water Depth (d_1)	

Time Interval Standard

Start Time for Pre-Soak	10:30AM	Water Remaining in Boring	yes
Start Time for Standard	1:00PM	Standard Time Interval Between Readings	30 min

Reading Number	Time Start/End (hh:mm)	Elapsed Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/ Comments
1	1:00 1:30	30	43.68 - 44.52 0.84	1.68	Clayey Sand
2	1:30 2:00	30	42.36 - 43.92 1.56	3.12	
3	2:00 2:30	30	42.12 - 43.68 1.56	3.12	
4	2:30 3:00	30	41.64 - 43.32 1.68	3.36	
5	3:00 3:30	30	40.2 - 41.88 1.68	3.36	
6	3:32 4:02	30	39.36 - 41.04 1.68	3.36	Percolation rate consistent in the last 3 readings. Infiltration Rate = 0.34 in/hr.

Notes:

Boring/Excavation Percolation Testing Field Log

Date: 4/24/2019

Project Location	Roman Creek Mitigation	Boring Test Number	A-19-005
Earth Description	Clayey Sand	Diameter of Boring	8"
Tested by	MG	Diameter of Casing	2"
Liquid Description	Clean Tap Water	Depth of Boring	111.6
Measurment Method	WLM	Depth to Invert of BMP	N/A
		Depth to Water Table	16
		Dept to Initial Water Depth (d_1)	

Time Interval Standard

Start Time for Pre-Soak	11:15AM	Water Remaining in Boring	yes
Start Time for Standard	12:45PM	Standard Time Interval Between Readings	30 min

Reading Number	Time Start/End (hh:mm)	Elapsed Time Δtime (mins)	Water Drop During Standard Time Interval Δd (inches)	Percolation Rate for Reading (in/hr)	Soil Description/Notes/ Comments
1	12:45 1:15	30	109.32 - 115.68 6.36	12.72	Clayey Sand
2	1:19 1:49	30	102.36 - 110.76 8.4	16.8	
3	1:50 2:20	30	103.2 - 111.72 8.52	17.04	
4	2:22 2:52	30	103.2 - 111.6 8.4	16.8	
5	2:54 3:24	30	103.08 - 111.36 8.28	16.56	
6	3:25 3:55	30	103.68 - 111.84 8.16	16.32	Percolation rate consistent in the last 3 readings. Infiltration Rate = 5.59 in/hr.

Notes:

Appendix C

Geotechnical Laboratory Test Results

AP Engineering and Testing, Inc.
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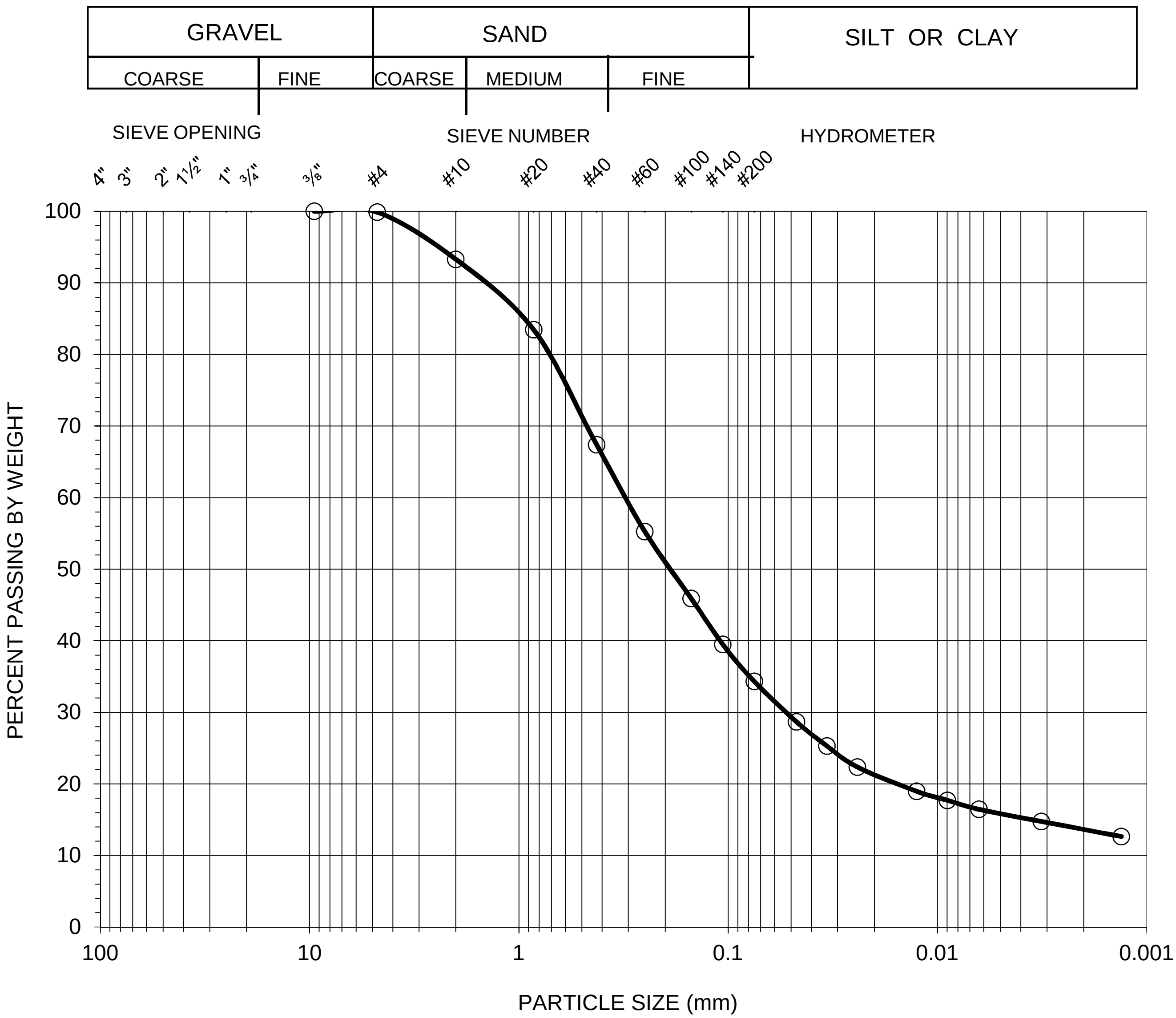
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913 & D 7928

Client Name:	HDR	Tested by:	NG	Date:	05/15/19
Project Name:	Roman Creek Mitigation	Computed by:	JP	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



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-19-001	2	5	0	66	34	27:12:15	SC

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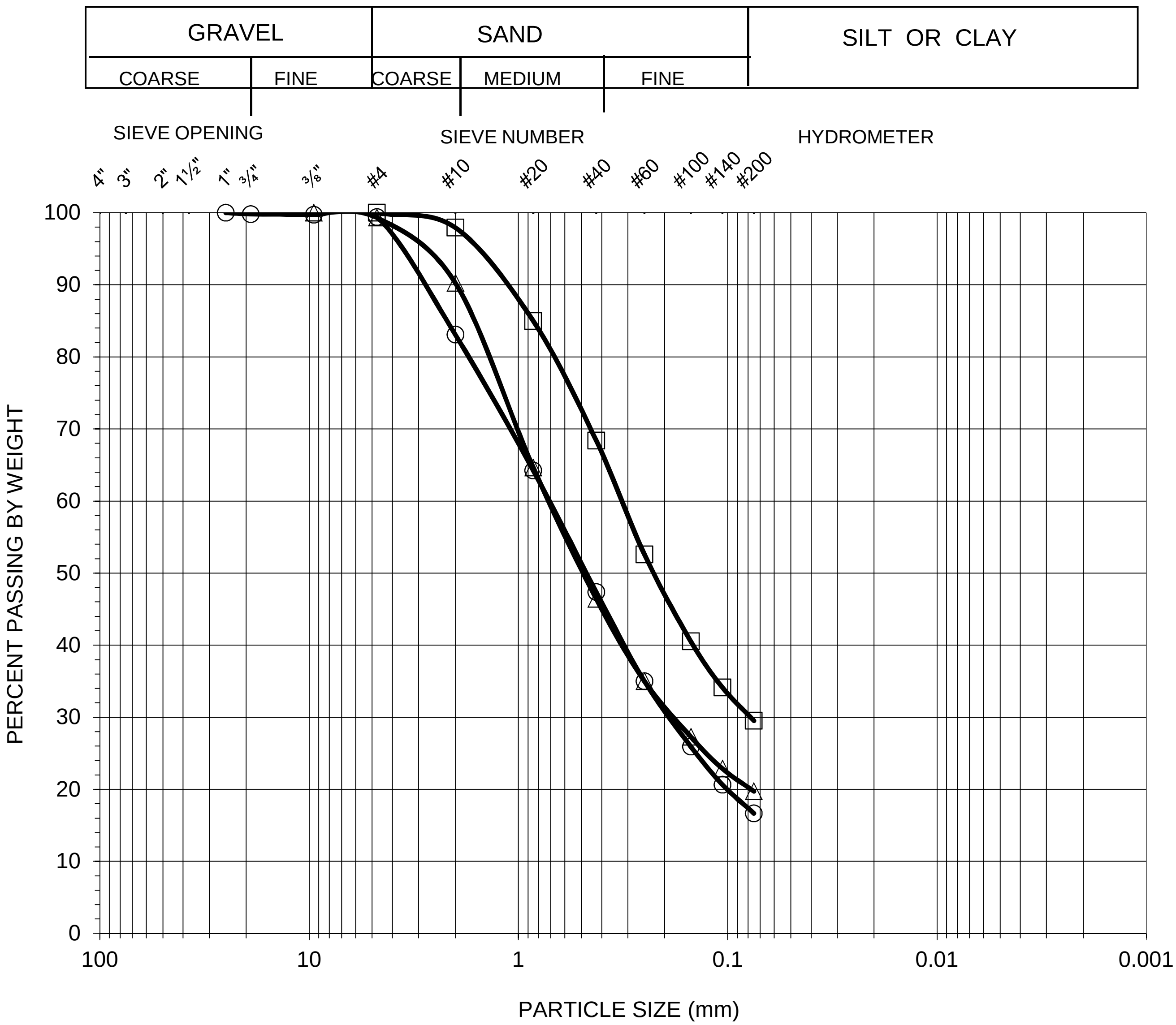
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



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-19-001	B	0-5	1	82	17	N/A	SM
□	A-19-001	3	8.5	0	70	30	N/A	SC*
△	A-19-001	5	20	1	79	20	N/A	SC*

**Note: Based on visual classification of sample*

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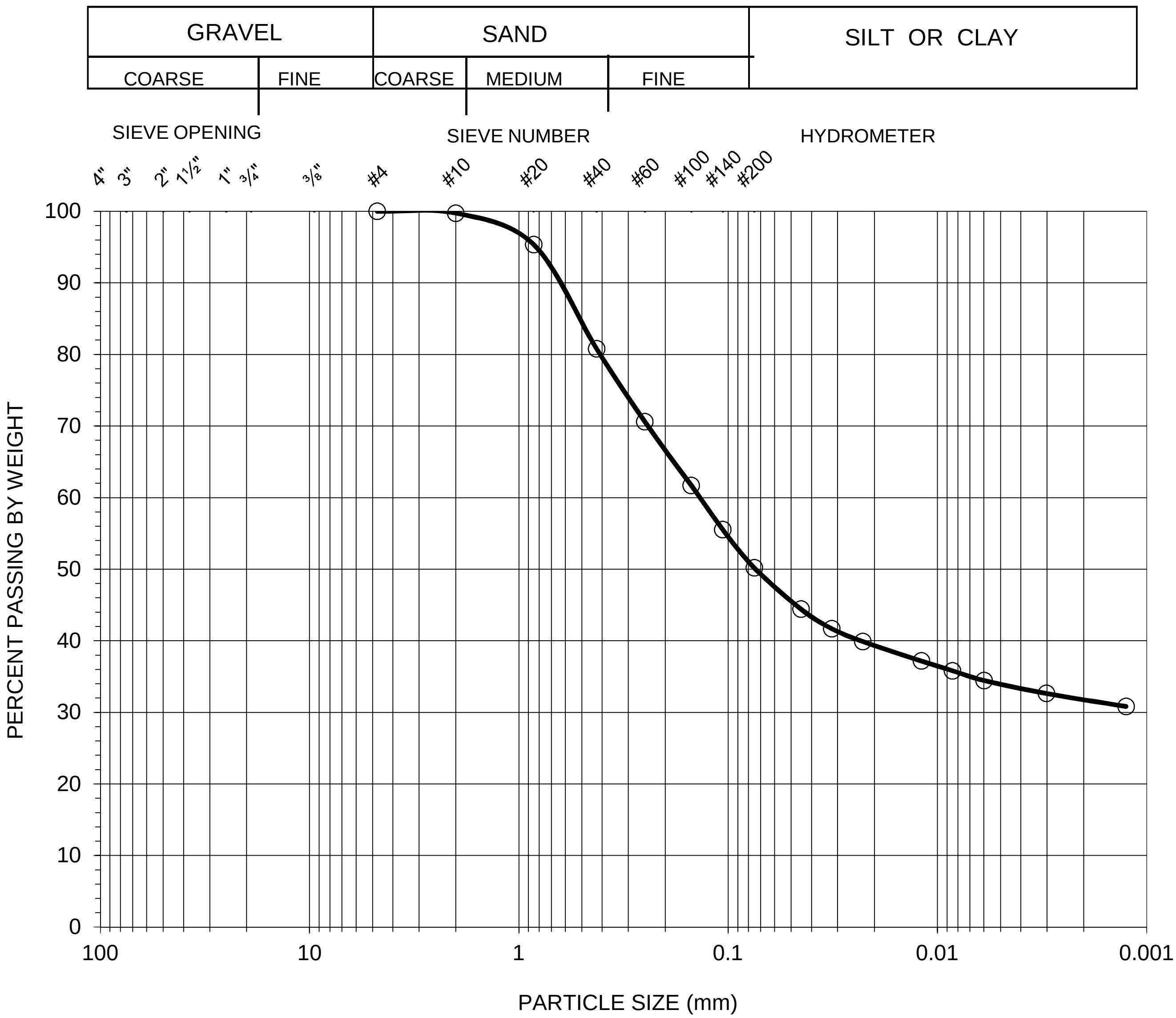
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913 & D 7928

Client Name:	HDR	Tested by:	NG	Date:	05/15/19
Project Name:	Roman Creek Mitigation	Computed by:	JP	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



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-19-002	2	5	0	50	50	48:12:36	CL

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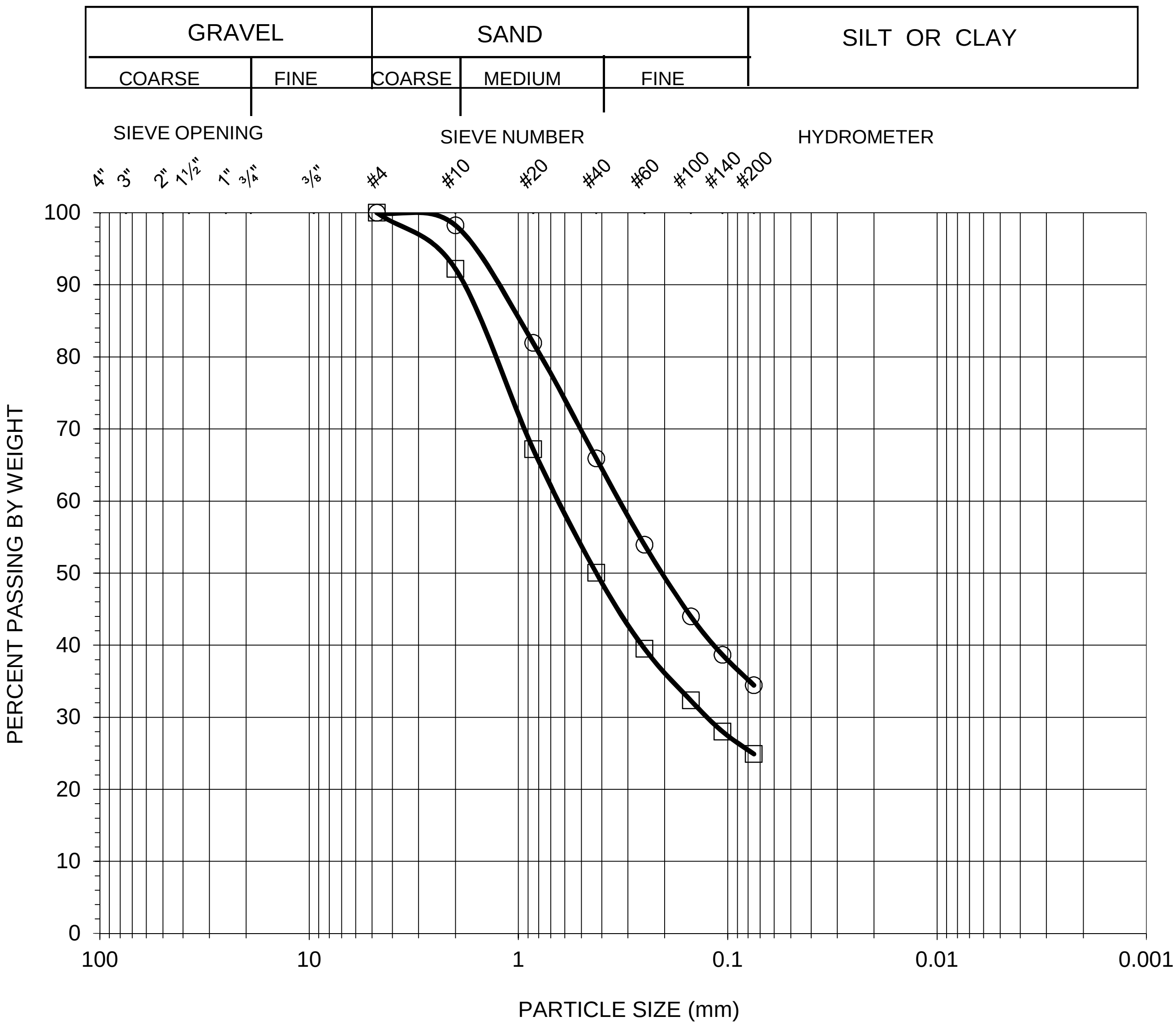
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	08/17/16
Project Name:	Roman Creek Mitigation	Computed by:	LS	Date:	08/17/16
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



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-19-002	3	10	0	66	34	40:10:30	SC
□	A-19-002	4	15	0	75	25	N/A	SC*

**Note: Based on visual classification of sample*

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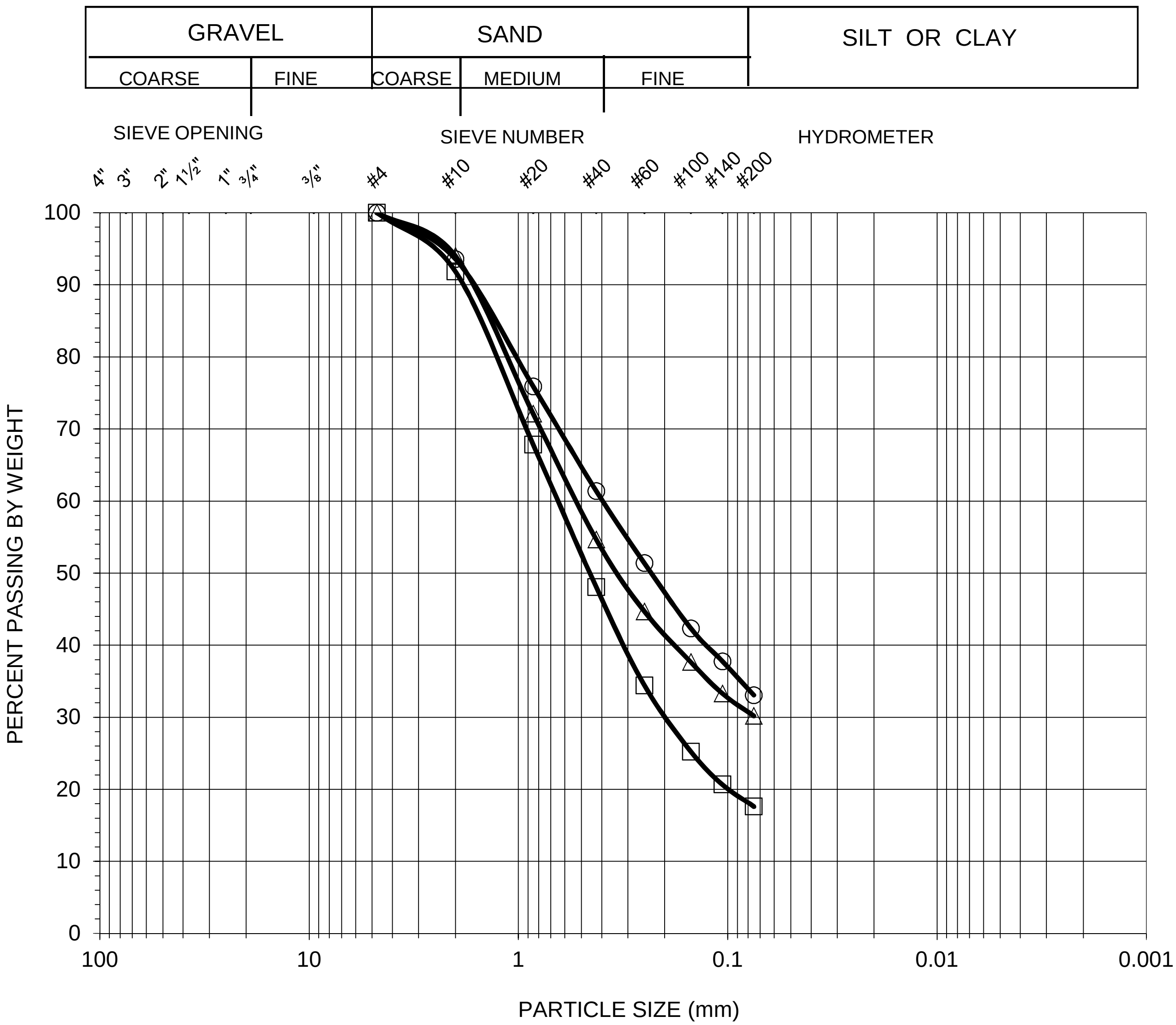
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



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-19-003	B	0-5	0	67	33	N/A	SC*
□	A-19-003	1	5	0	82	18	N/P	SM
△	A-19-003	2	10	0	70	30	N/A	SC*

**Note: Based on visual classification of sample*

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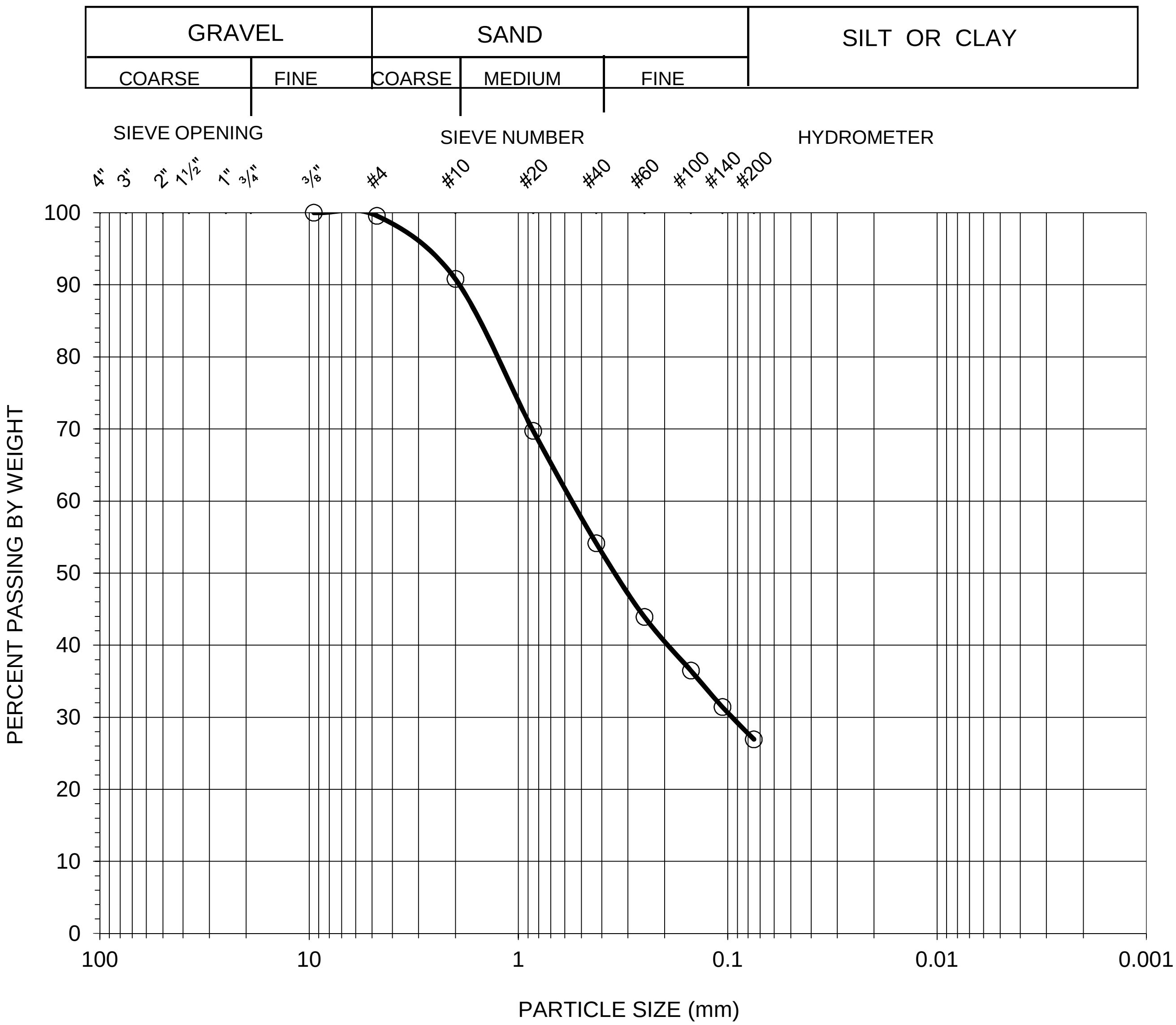
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NR	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



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-19-003	5	25	0	73	27	N/A	SC*

**Note: Based on visual classification of sample*

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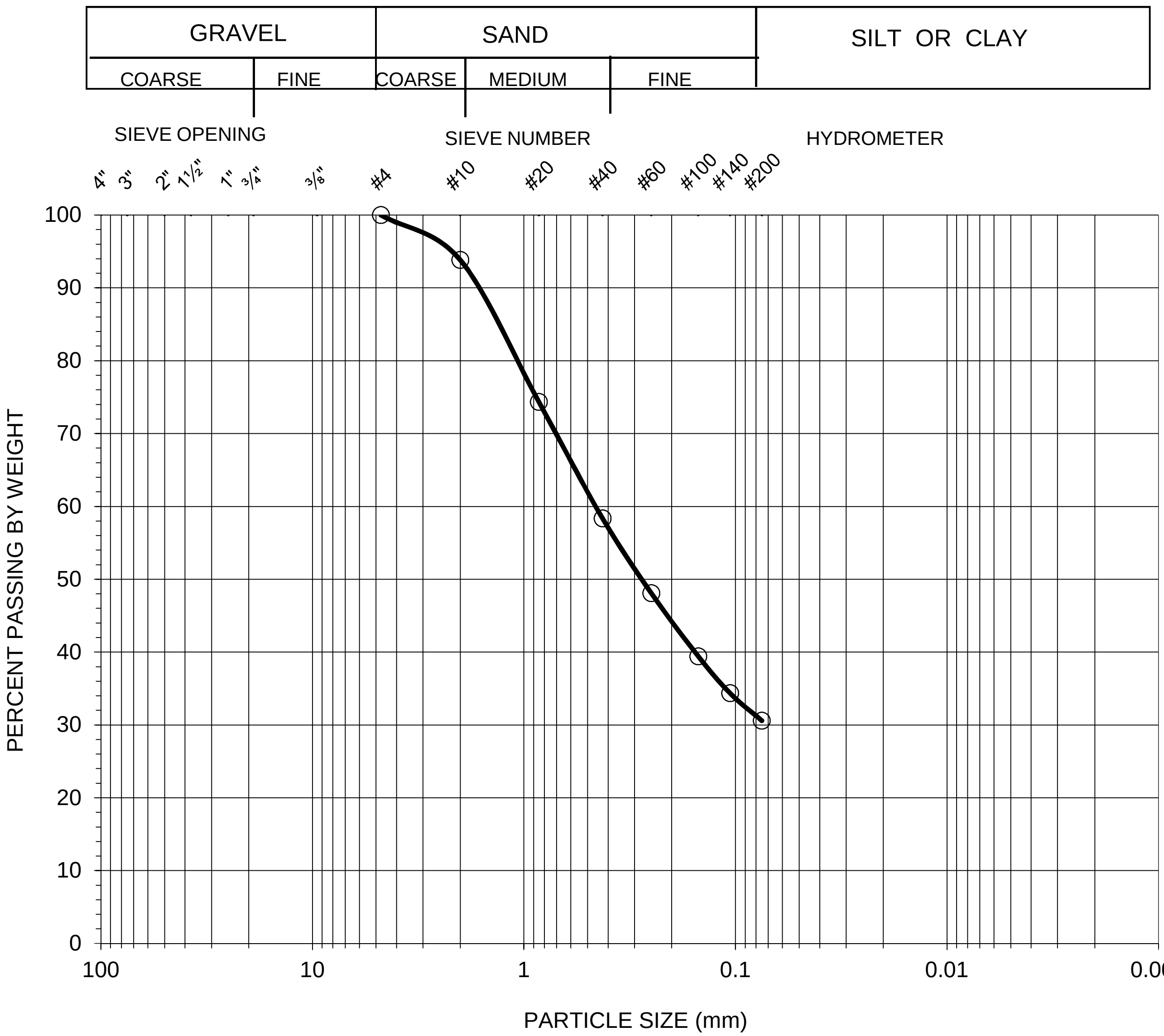
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	HA-19-004	1 & 2	2 & 4.5	0	69	31	28:12:16	SC

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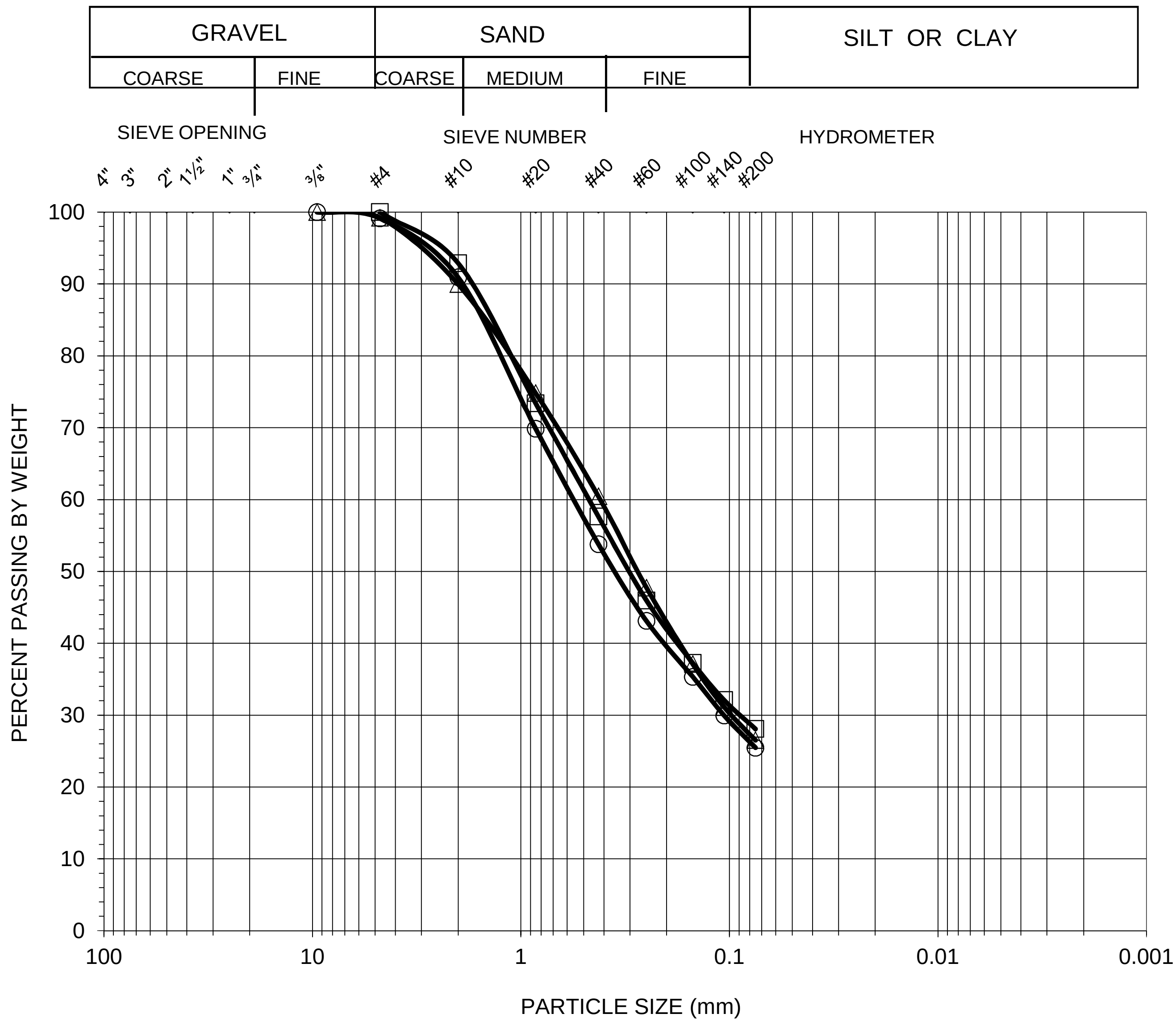
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



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-19-005	1	5	1	74	25	N/A	SC*
□	A-19-005	2	8.5	0	72	28	24:16:8	SC
△	A-19-005	3	15	1	72	27	25:15:10	SC

Note: Based on visual classification of sample

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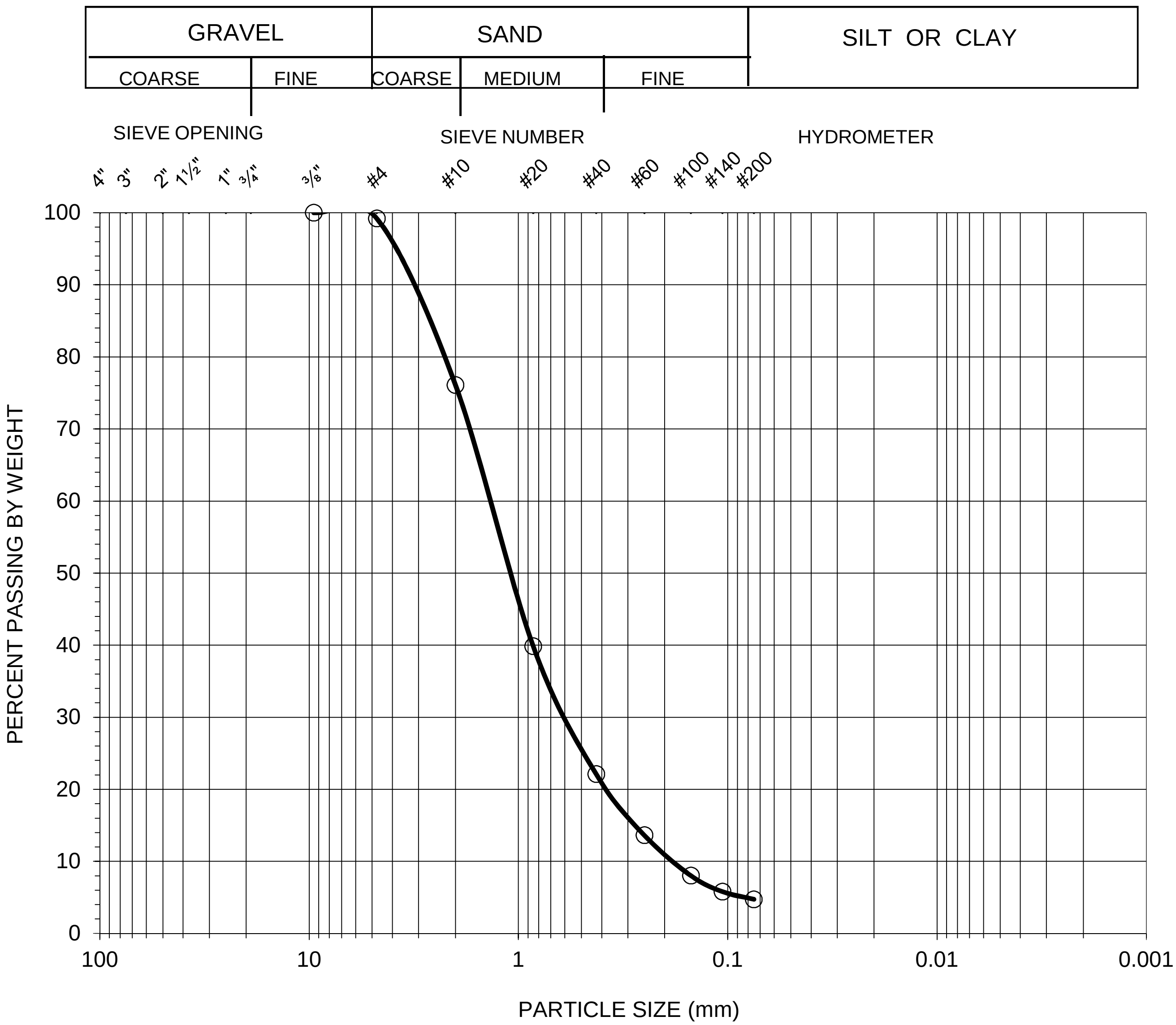
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent Gravel	Percent Sand	Percent Silt & Clay	Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
○	HA-19-006	2 & 3	3 & 4	1	94	5	N/P	SW-SM

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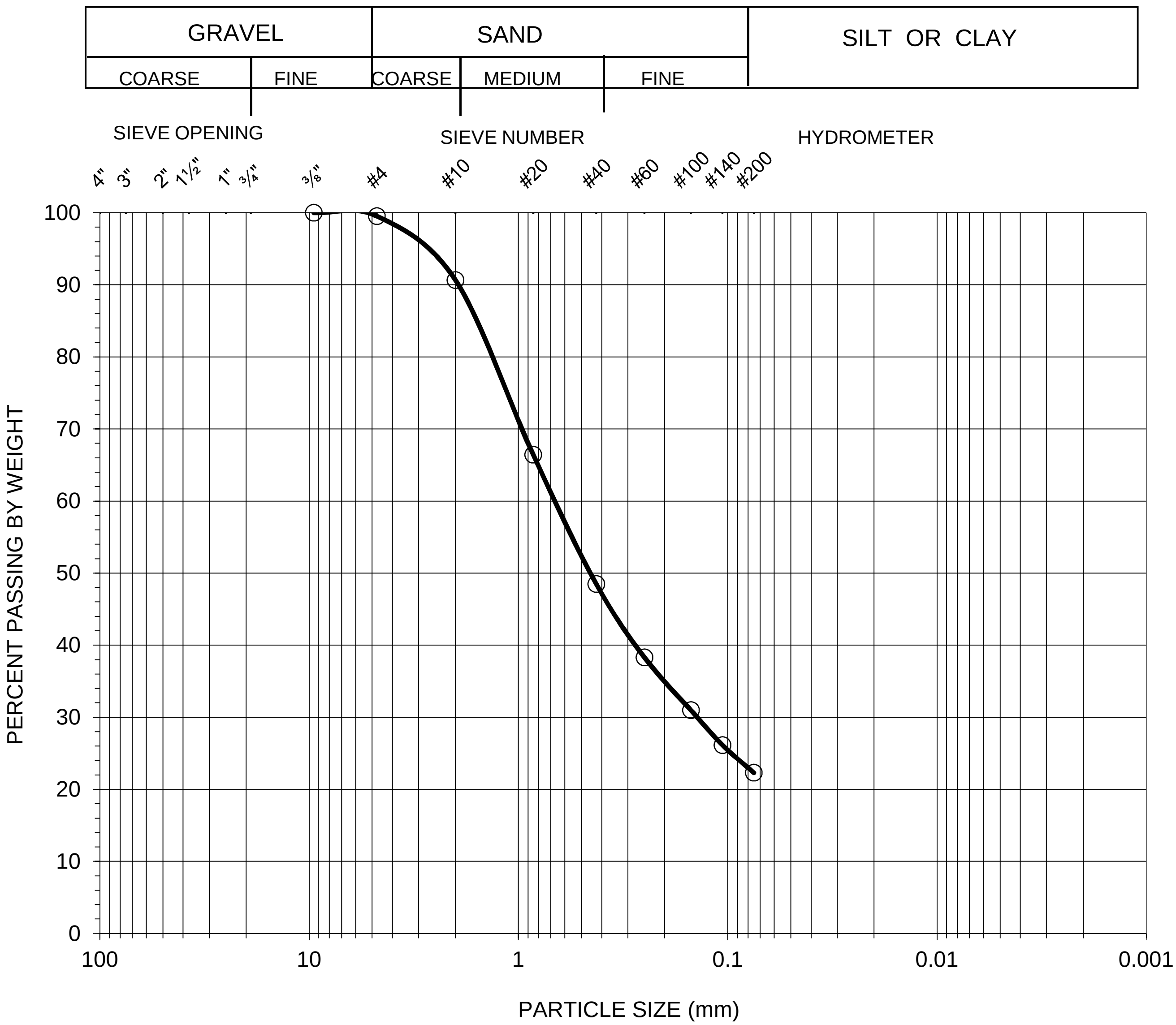
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19



Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	HA-19-007	3	4.5	0	78	22	N/A	SM

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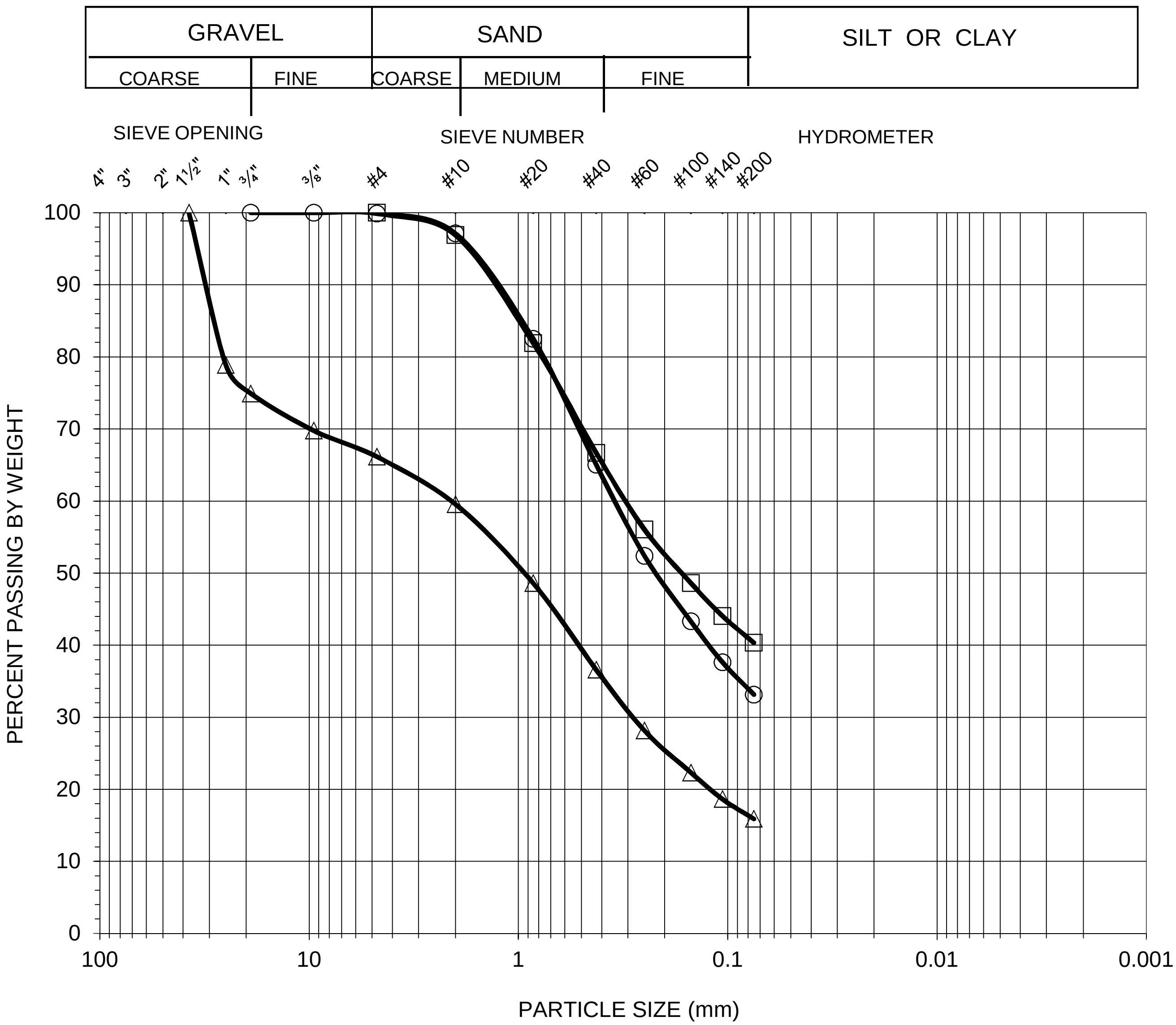
GRAIN SIZE DISTRIBUTION CURVE
ASTM D 6913

Client Name:	HDR	Tested by:	NG	Date:	05/22/19
Project Name:	Roman Creek Mitigation	Computed by:	NR	Date:	05/22/19
Project Number:	10156443	Checked by:	AP	Date:	05/22/19

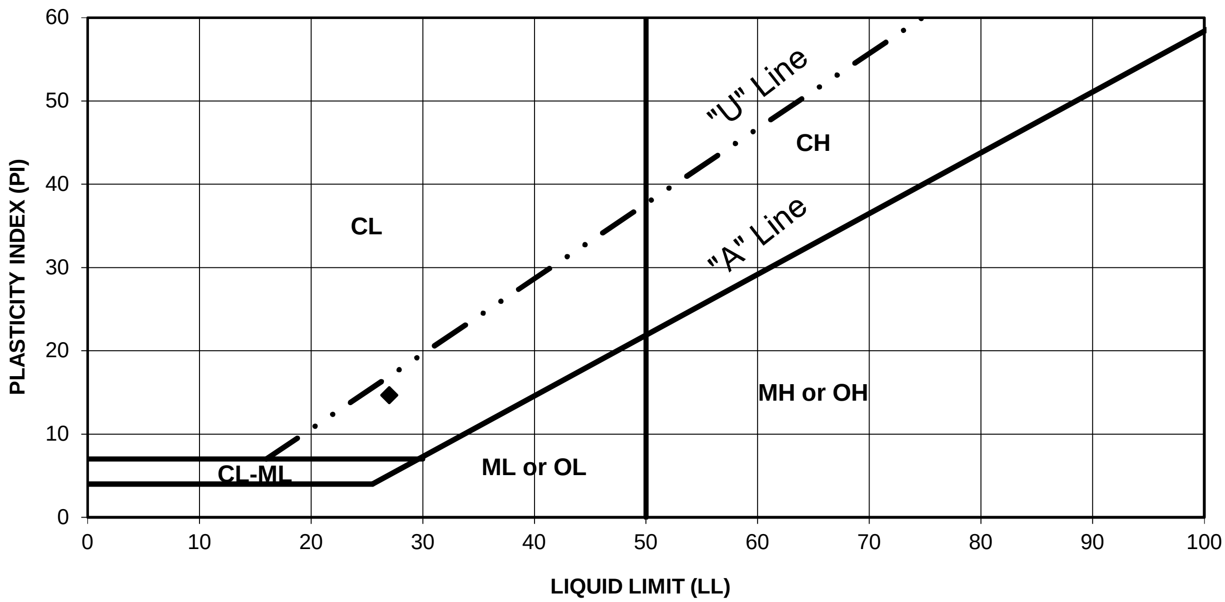


Symbol	Boring No.	Sample No.	Sample Depth (feet)	Percent			Atterberg Limits LL:PL:PI	Soil Type U.S.C.S
				Gravel	Sand	Silt & Clay		
○	A-19-008	B	0-3	0	67	33	N/A	SM
□	A-19-008	2	5	0	60	40	N/A	SC*
△	A-19-008	5	20	34	50	16	N/A	SM

**Note: Based on visual classification of sample*

ATTERBERG LIMITS
ASTM D 4318

Project Name:	Roman Creek Mitigation	**Tested By:**	DK	Date:	05/14/19
Project No.:	10156443	**Checked By:**	AP	Date:	05/22/19



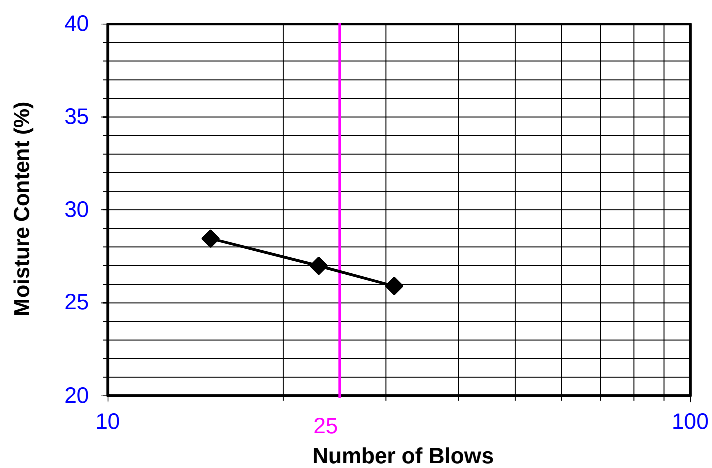
PROCEDURE USED

☐ 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-19-001	2	5	27	12	15	CL

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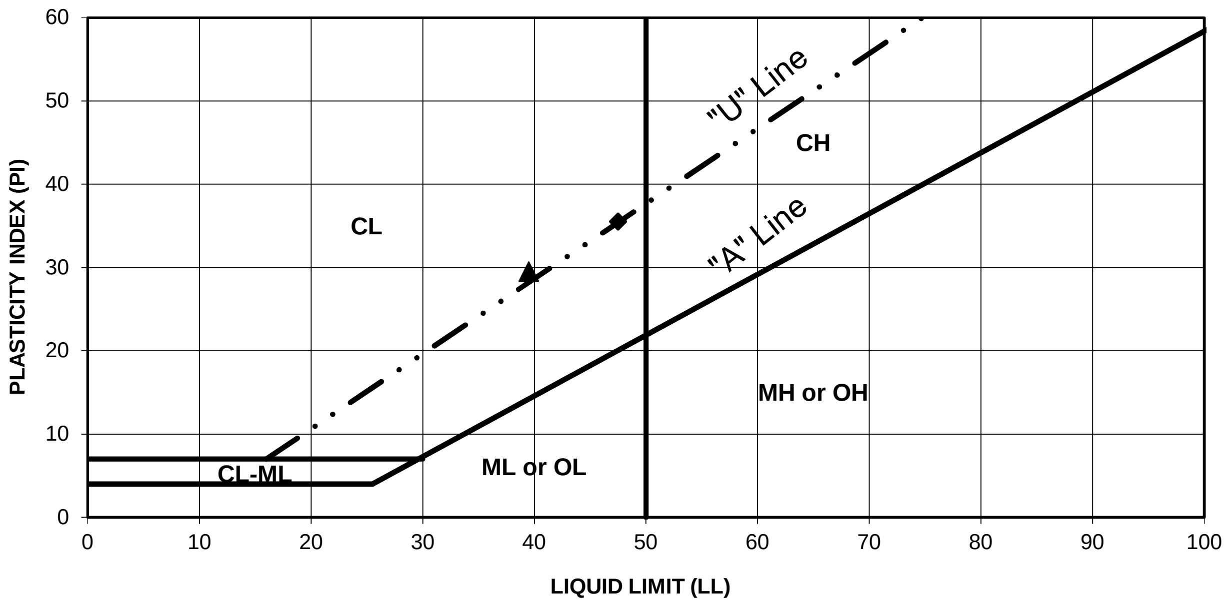
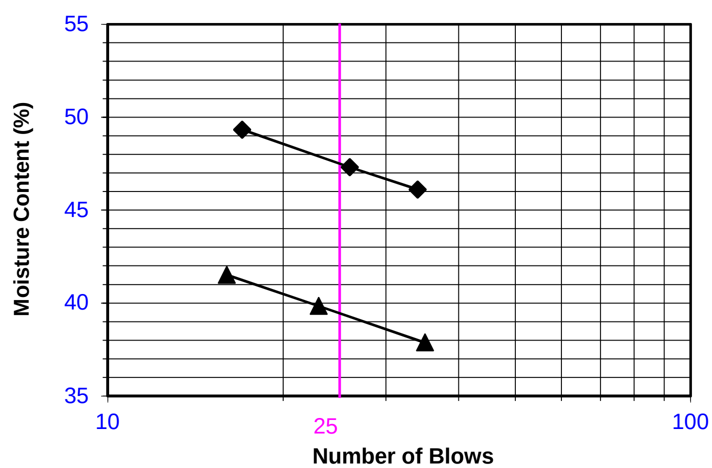
ATTERBERG LIMITS
ASTM D 4318

Project Name: Roman Creek Mitigation **Tested By:** DK Date: 05/14/19
Project No.: 10156443 **Checked By:** AP Date: 05/22/19



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	A-19-002	2	5	48	12	36	CL
▲	A-19-002	3	10	40	10	30	CL

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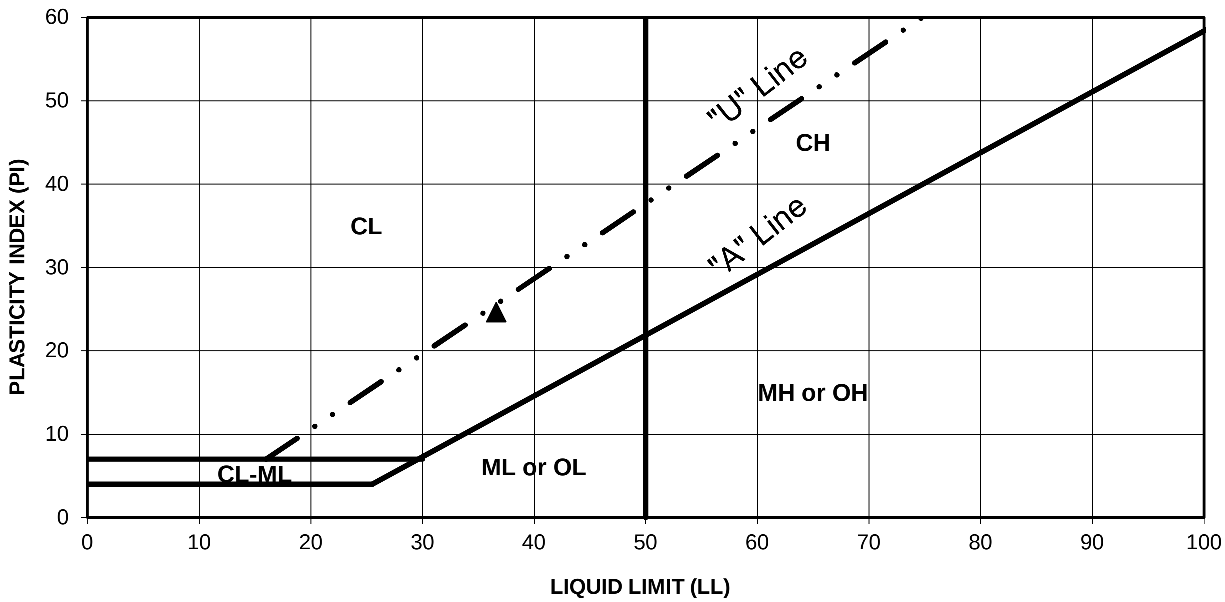
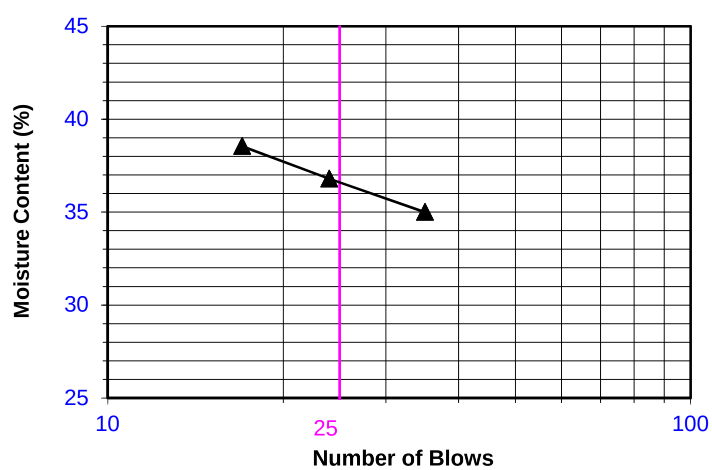
ATTERBERG LIMITS
ASTM D 4318

Project Name: Roman Creek Mitigation **Tested By:** DK Date: 05/14/19
Project No.: 10156443 **Checked By:** AP Date: 05/22/19



PROCEDURE USED

- [ ] Wet Preparation
- [x] Dry Preparation
- [x] Procedure A Multipoint Test
- [ ] Procedure B One-point Test

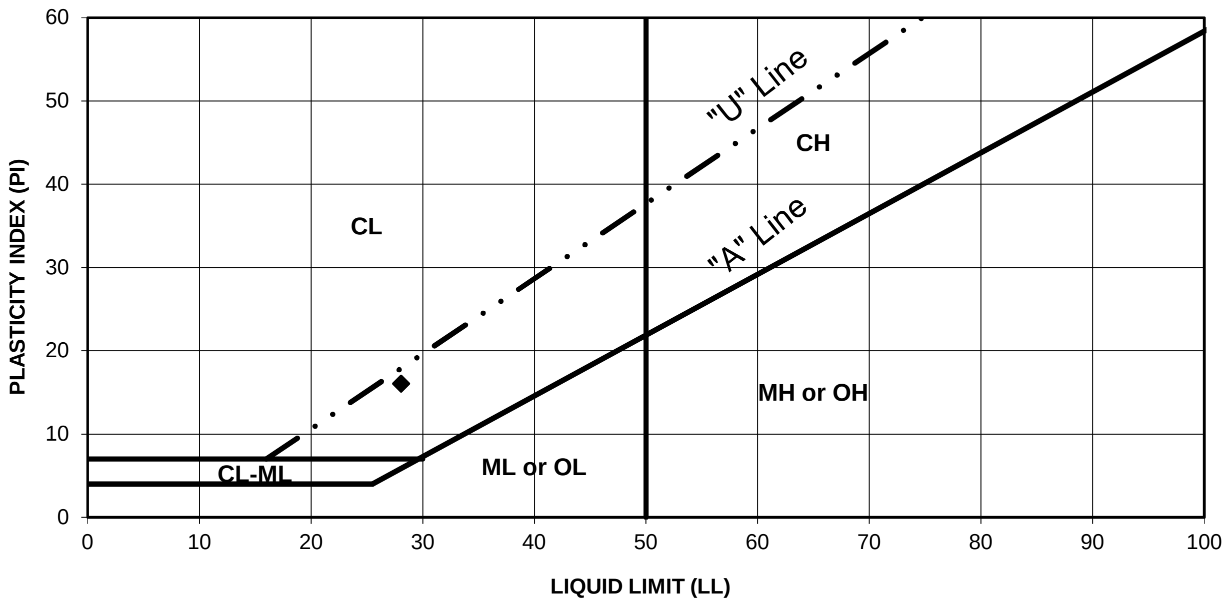


Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	A-19-003	1	5	NP	NP	NP	
▲	A-19-003	3	15	37	12	25	CL

* NP denotes "non-plastic"

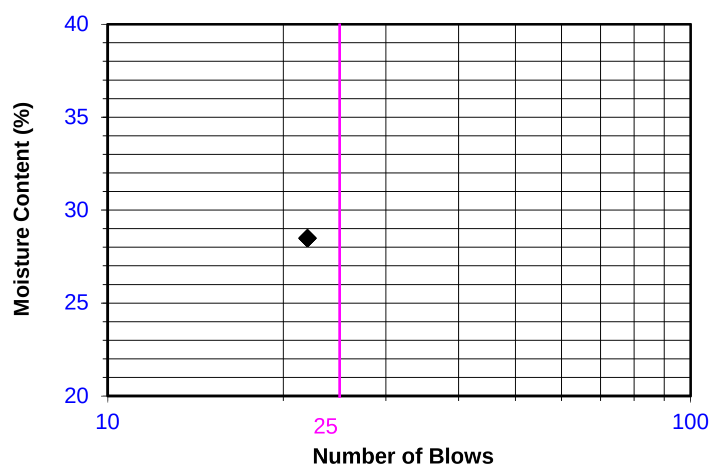
ATTERBERG LIMITS
ASTM D 4318

Project Name: Roman Creek Mitigation **Tested By:** DK Date: 05/14/19
Project No.: 10156443 **Checked By:** AP Date: 05/22/19



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [ ] Procedure A Multipoint Test
- [X] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	HA-19-004	1 & 2	2 & 4.5	28	12	16	CL

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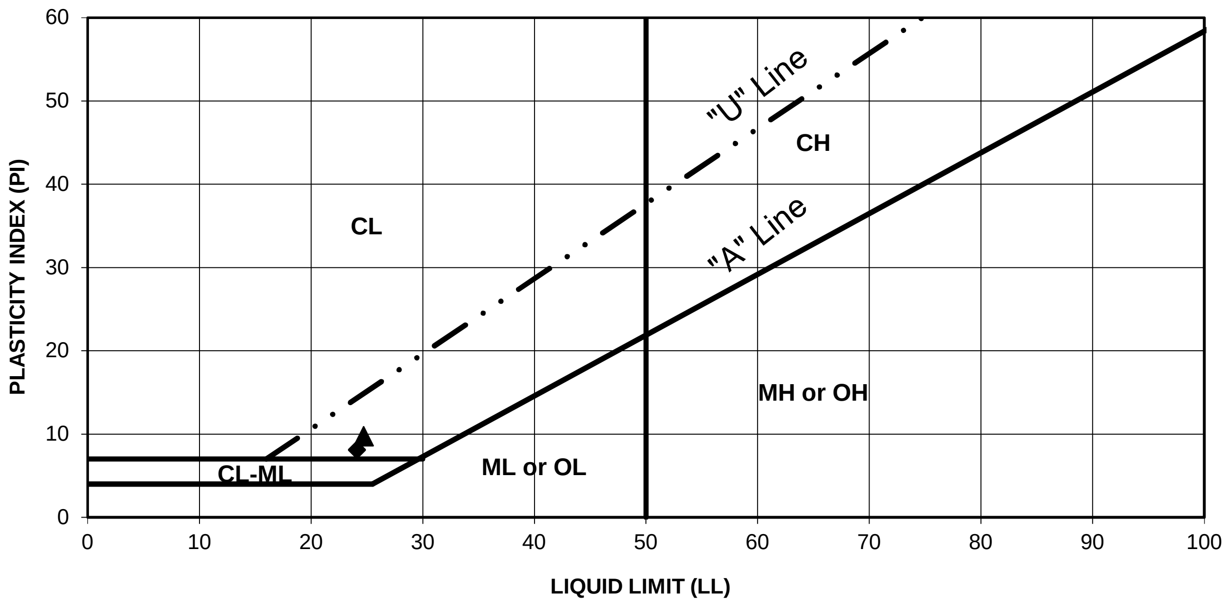
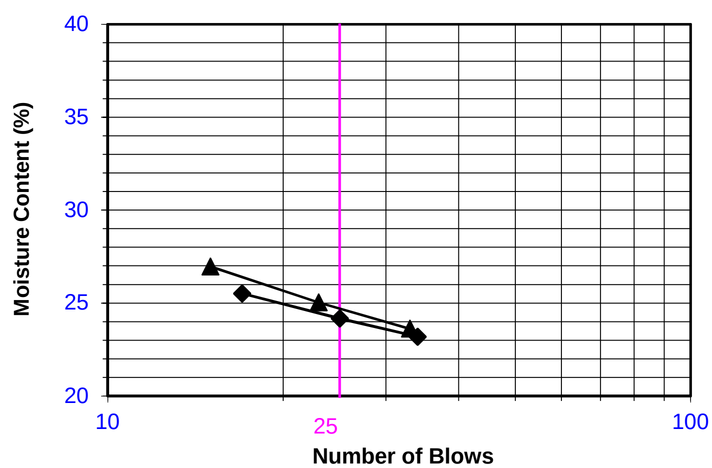
ATTERBERG LIMITS
ASTM D 4318

Project Name: Roman Creek Mitigation **Tested By:** DK Date: 05/14/19
Project No.: 10156443 **Checked By:** AP Date: 05/22/19



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	A-19-005	2	8.5	24	16	8	CL
▲	A-19-005	3	15	25	15	10	CL

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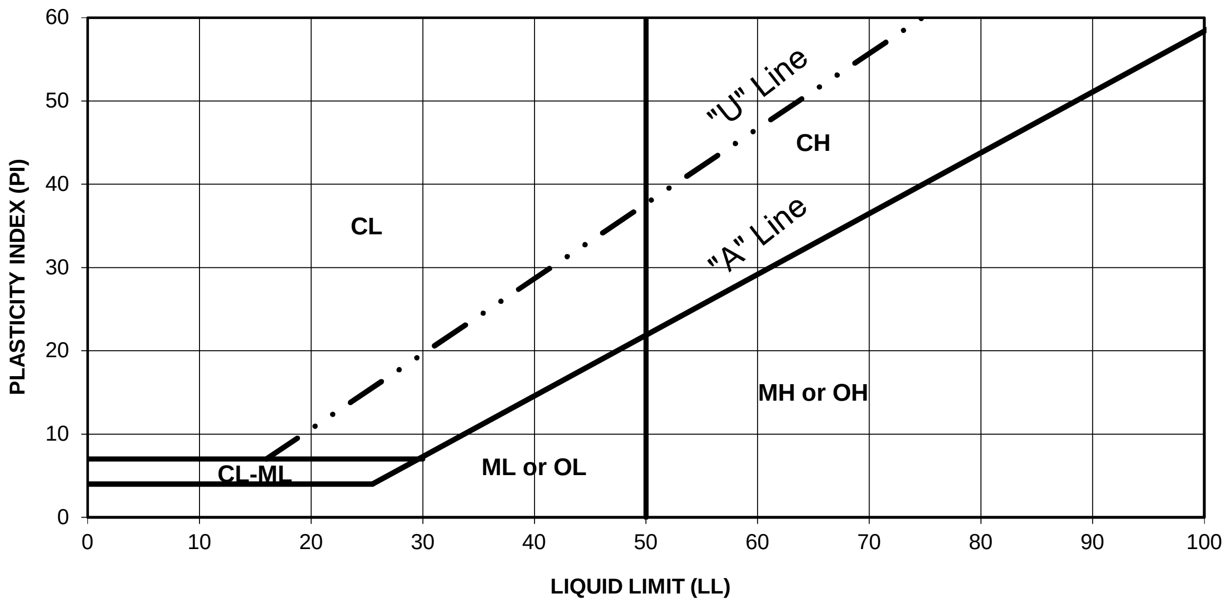
ATTERBERG LIMITS
ASTM D 4318

Project Name: Roman Creek Mitigation **Tested By:** DK Date: 05/14/19
Project No.: 10156443 **Checked By:** AP Date: 05/22/19



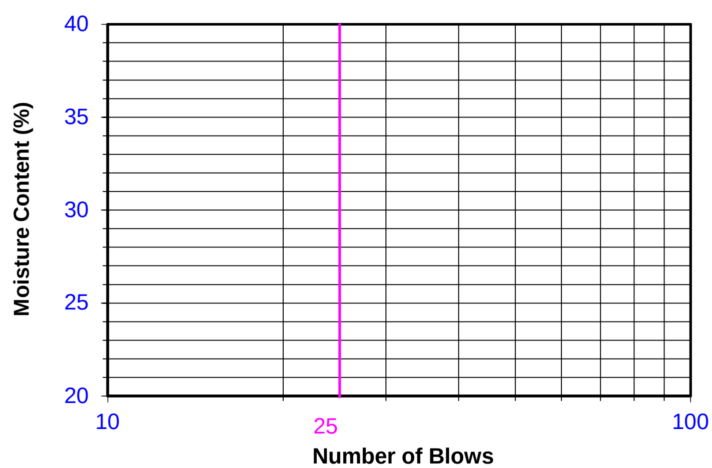
PROCEDURE USED

☐ 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
	HA-19-006	2 & 3	3 & 4	NP	NP	NP	

* NP denotes "non-plastic"

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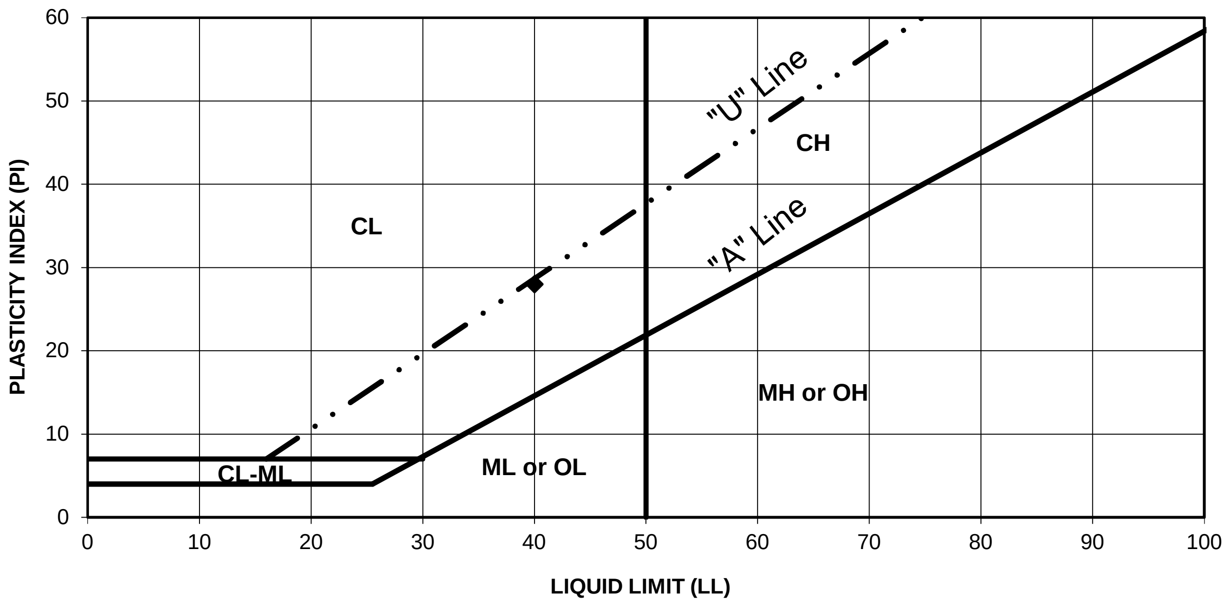
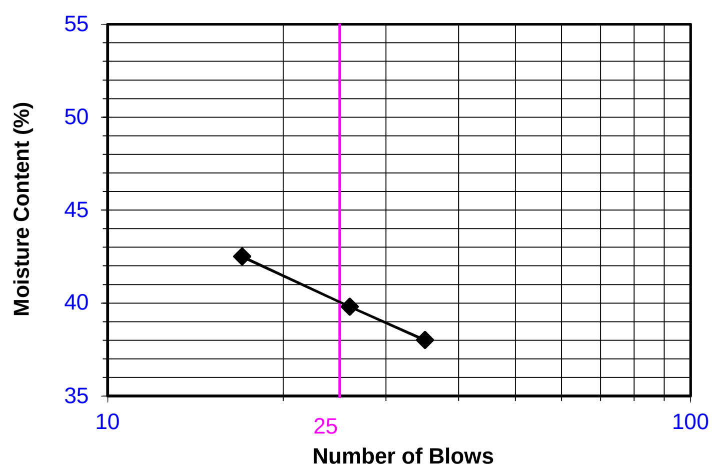
ATTERBERG LIMITS
ASTM D 4318

Project Name: Roman Creek Mitigation **Tested By:** DK Date: 05/14/19
Project No.: 10156443 **Checked By:** AP Date: 05/22/19



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
♦	A-19-008	3	10	40	12	28	CL

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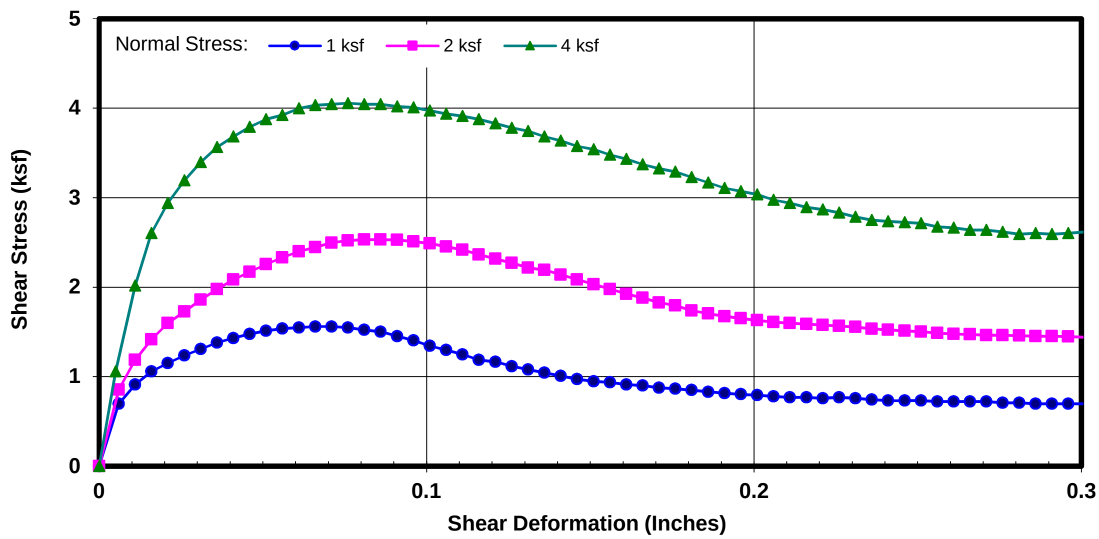
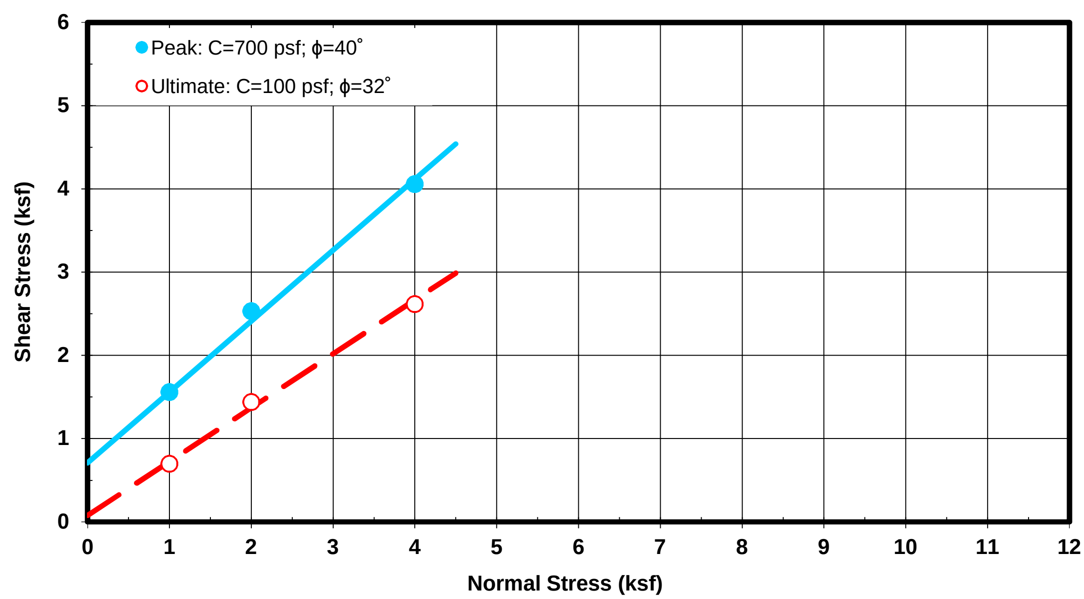
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Roman Creek Mitigation
Project No.: 10156443
Boring No.: A-19-001
Sample No.: 1 **Depth (ft):** 3.5
Sample Type: Mod. Cal.
Soil Description: Clayey Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: NG **Date:** 05/14/19
Computed By: JP **Date:** 05/22/19
Checked by: AP **Date:** 05/22/19

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)
138.4	127.1	8.9	12.0	74	100	1	1.560	0.697
						2	2.531	1.441
						4	4.056	2.616





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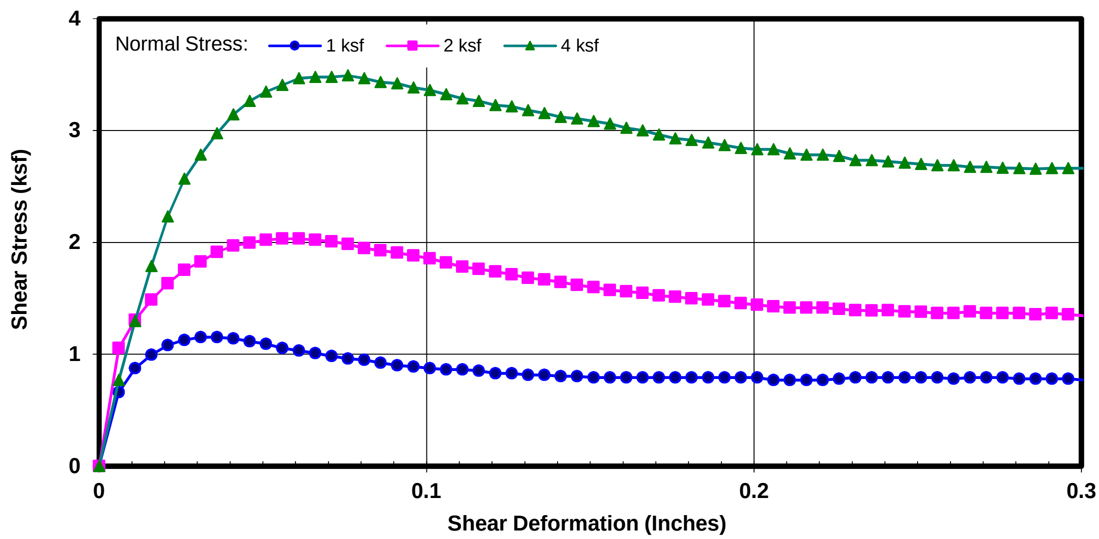
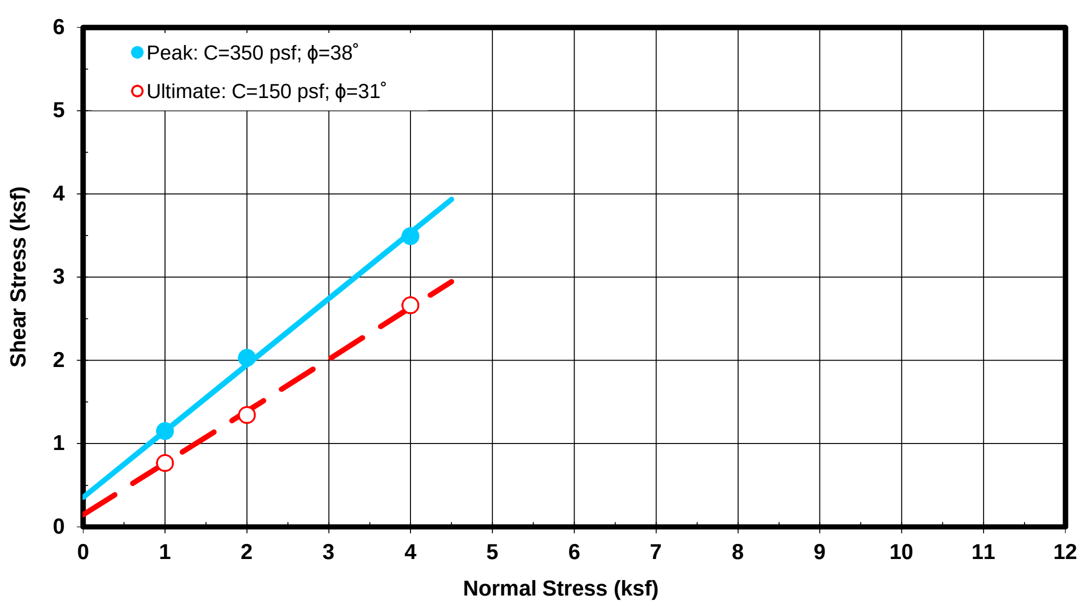
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	Roman Creek Mitigation	**Tested By:** ST	**Date:** 05/15/19
Project No.:	10156443	**Computed By:** JP	**Date:** 05/22/19
Boring No.:	A-19-005	**Checked by:** AP	**Date:** 05/22/19
Sample No.:	1 **Depth (ft):** 5		
Sample Type:	Mod. Cal.		
Soil Description:	Clayey Sand		
Test Condition:	Inundated **Shear Type:** Regular		

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)
130.4	120.3	8.5	13.6	57	91	1	1.152	0.768
						2	2.033	1.344
						4	3.492	2.664





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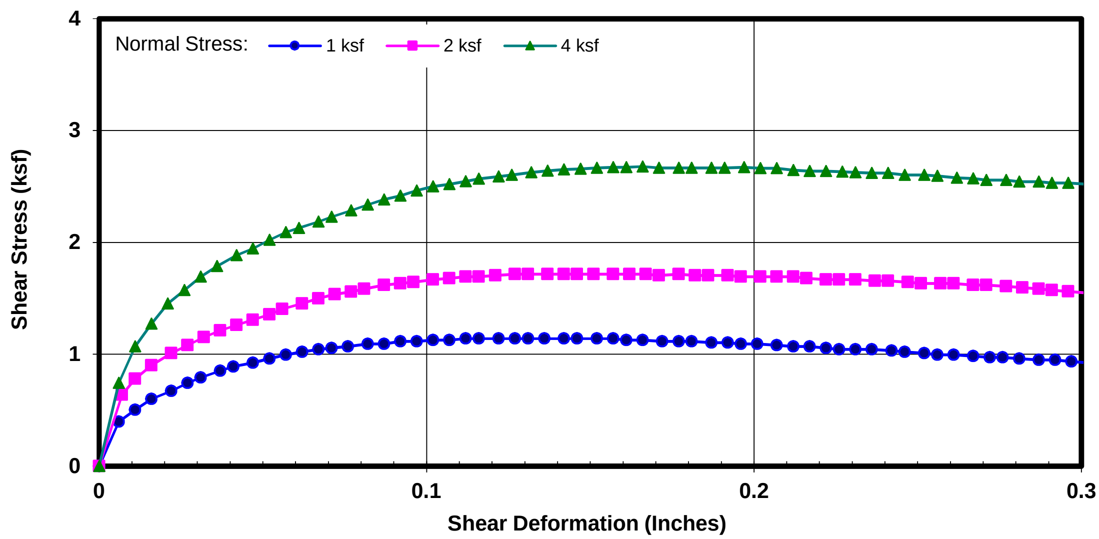
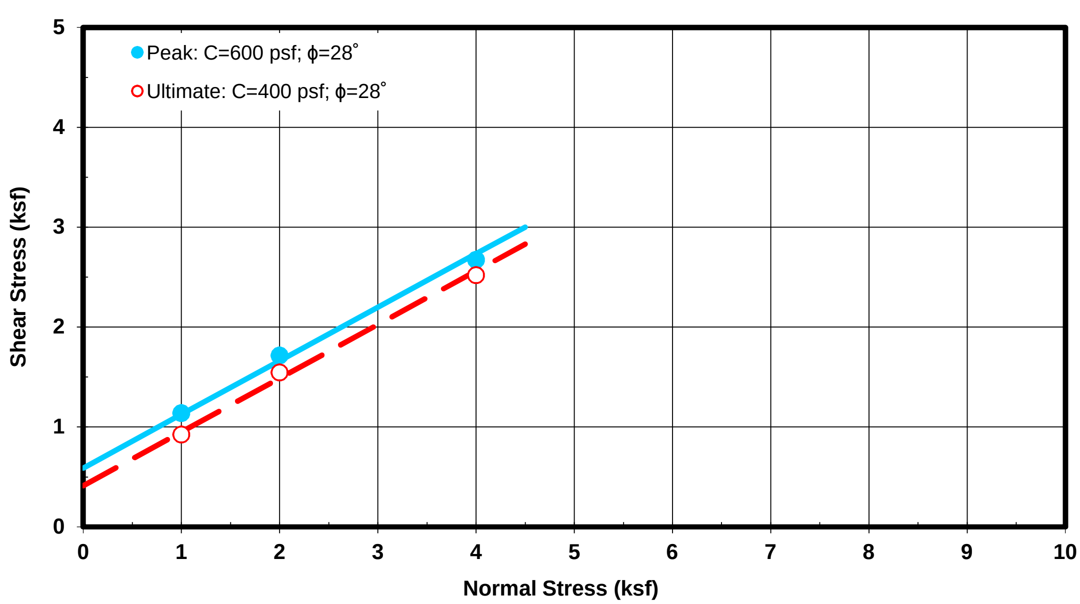
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: Roman Creek Mitigation
Project No.: 10156443
Boring No.: A-19-008
Sample No.: 1 **Depth (ft):** 3.5
Sample Type: Mod. Cal.
Soil Description: Sandy Clay
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 05/15/19
Computed By: JP **Date:** 05/22/19
Checked by: AP **Date:** 05/22/19

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)
131.6	112.0	17.5	18.2	93	97	1	1.140	0.924
						2	1.716	1.546
						4	2.672	2.520





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COMPACTION TEST

Client: HDR | AP Number: 19-0524

Project Name: Roman Creek Mitigation | Tested By: JT | Date: 05/21/19

Project No. : 10156443 | Calculated By: NR | Date: 05/22/19

Boring No.: A-19-001 | Checked By: AP | Date: 05/22/19

Sample No.: B | Depth(ft.): 0-5

Visual Sample Description: Silty Sand

Compaction Method: [X] ASTM D1557 [ ] ASTM D698

METHOD: A

MOLD VOLUME (CU.FT): 0.0333

Preparation Method: [ ] Moist [X] Dry

Wt. Comp. Soil + Mold (gm.)	3966	3982	3804	3915		
Wt. of Mold (gm.)	1836	1836	1836	1836		
Net Wt. of Soil (gm.)	2130	2146	1968	2079		
Container No.						
Wt. of Container (gm.)	143.02	138.87	141.98	140.25		
Wet Wt. of Soil + Cont. (gm.)	483.97	596.36	601.82	678.57		
Dry Wt. of Soil + Cont. (gm.)	461.73	554.77	582.69	620.02		
Moisture Content (%)	6.98	10.00	4.34	12.20		
Wet Density (pcf)	140.87	141.93	130.16	137.50		
Dry Density (pcf)	131.68	129.03	124.74	122.55		

Maximum Dry Density (pcf): **132.4**

Maximum Dry Density w/ Rock Correction (pcf): **N/A**

Optimum Moisture Content (%): **8.0**

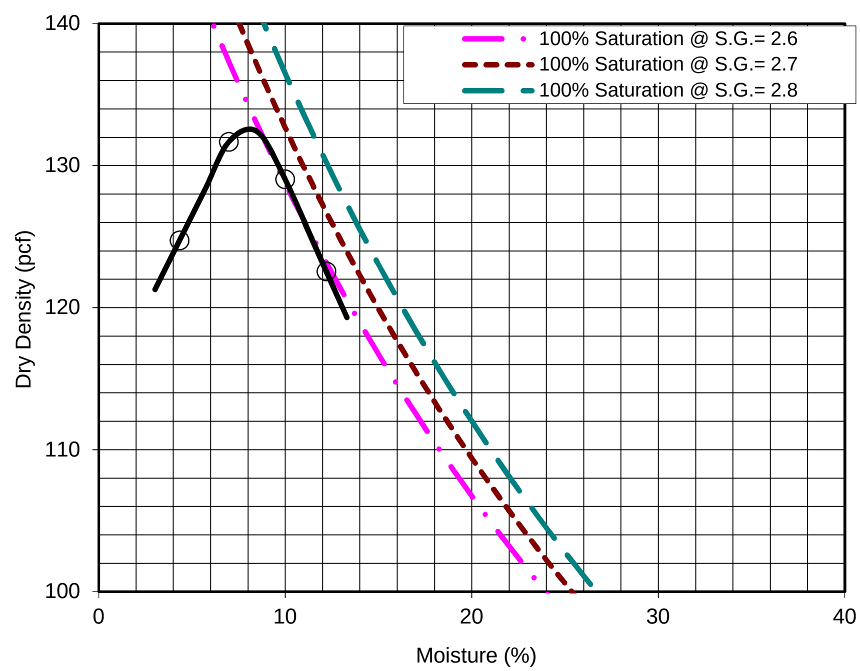
Optimum Moisture Content w/ Rock Correction (%): **N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *1.4%*
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)



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COMPACTION TEST

Client:	HDR			AP Number:	19-0524
Project Name:	Roman Creek Mitigation	Tested By:	JT	Date:	05/21/19
Project No. :	10156443	Calculated By:	NR	Date:	05/22/19
Boring No.:	A-19-008	Checked By:	AP	Date:	05/22/19
Sample No.:	B	Depth(ft.):	0-3		
Visual Sample Description:	Silty Sand				

Compaction Method: [X] ASTM D1557 / [ ] ASTM D698

Preparation Method: [ ] Moist / [X] Dry

METHOD: A

MOLD VOLUME (CU.FT): 0.0333

Wt. Comp. Soil + Mold (gm.)	4021	3971	3939	3813		
Wt. of Mold (gm.)	1836	1836	1836	1836		
Net Wt. of Soil (gm.)	2185	2135	2103	1977		
Container No.						
Wt. of Container (gm.)	149.31	144.96	135.63	129.62		
Wet Wt. of Soil + Cont. (gm.)	481.46	527.33	558.50	599.47		
Dry Wt. of Soil + Cont. (gm.)	456.91	490.35	535.60	582.31		
Moisture Content (%)	7.98	10.71	5.73	3.79		
Wet Density (pcf)	144.51	141.20	139.09	130.75		
Dry Density (pcf)	133.83	127.55	131.56	125.98		

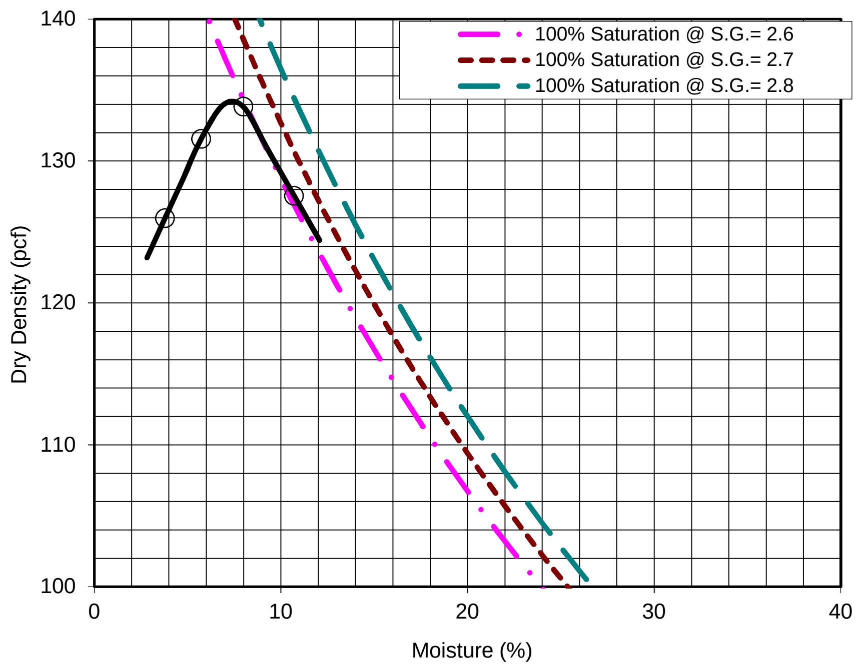
Maximum Dry Density (pcf)	**134.1**	Optimum Moisture Content (%)	**7.3**
Maximum Dry Density w/ Rock Correction (pcf)	**N/A**	Optimum Moisture Content w/ Rock Correction (%)	**N/A**

PROCEDURE USED

[X] *METHOD A: Percent of Oversize:* *0.3%*
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)



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EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: HDR
Project Name: Roman Creek Mitigation
Project No.: 10156443

AP Job No.: 19-0524
Date: 05/21/19

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
A-19-008	B	0-3	Silty Sand	120.1	7.5	50.4	2	3

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

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EXPANSION INDEX TEST RESULTS

ASTM D 4829

Client Name: HDR
Project Name: Roman Creek Mitigation
Project No.: 10156443

AP Job No.: 19-0524
Date: 05/21/19

Boring No.	Sample No.	Depth (ft)	Soil Description	Molded Dry Density (pcf)	Molded Moisture Content (%)	Init. Degree Saturation (%)	Measured Expansion Index	Corrected Expansion Index
A-19-001	B	0-5	Silty Sand	120.4	7.6	51.5	0	0

ASTM EXPANSION CLASSIFICATION

Expansion Index	Classification
0-20	V. Low
21-50	Low
51-90	Medium
91-130	High
>130	V. High

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CORROSION TEST RESULTS

Client Name: HDR
Project Name: Roman Creek Mitigation
Project No.: 10156443

AP Job No.: 19-0524
Date: 05/16/19

Boring No.	Sample No.	Depth (feet)	Soil Type	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
A-19-005	1	5	SC	5214	7.8	37	39

NOTES:
Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

Table 1 - Laboratory Tests on Soil Samples

HDR, Irvine
City of Vista Rowan Creek
HDR Lab #19-0256LAB
16-May-19

Sample ID			A-19-002 @ 5'	A-19-008 @ 3.5'
Resistivity		**Units**		
as-received		ohm-cm	1,520	1,200
minimum		ohm-cm	1,120	960
pH			7.6	7.7
Electrical Conductivity		mS/cm	0.05	0.22
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	13	39
magnesium	Mg^{2+}	mg/kg	2.3	15
sodium	Na^{1+}	mg/kg	49	180
potassium	K^{1+}	mg/kg	8.0	15
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	101	131
fluoride	F^{1-}	mg/kg	ND	ND
chloride	Cl^{1-}	mg/kg	8.0	74
sulfate	SO_4^{2-}	mg/kg	11	216
phosphate	PO_4^{3-}	mg/kg	ND	1.0
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	4.8	2.6
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

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

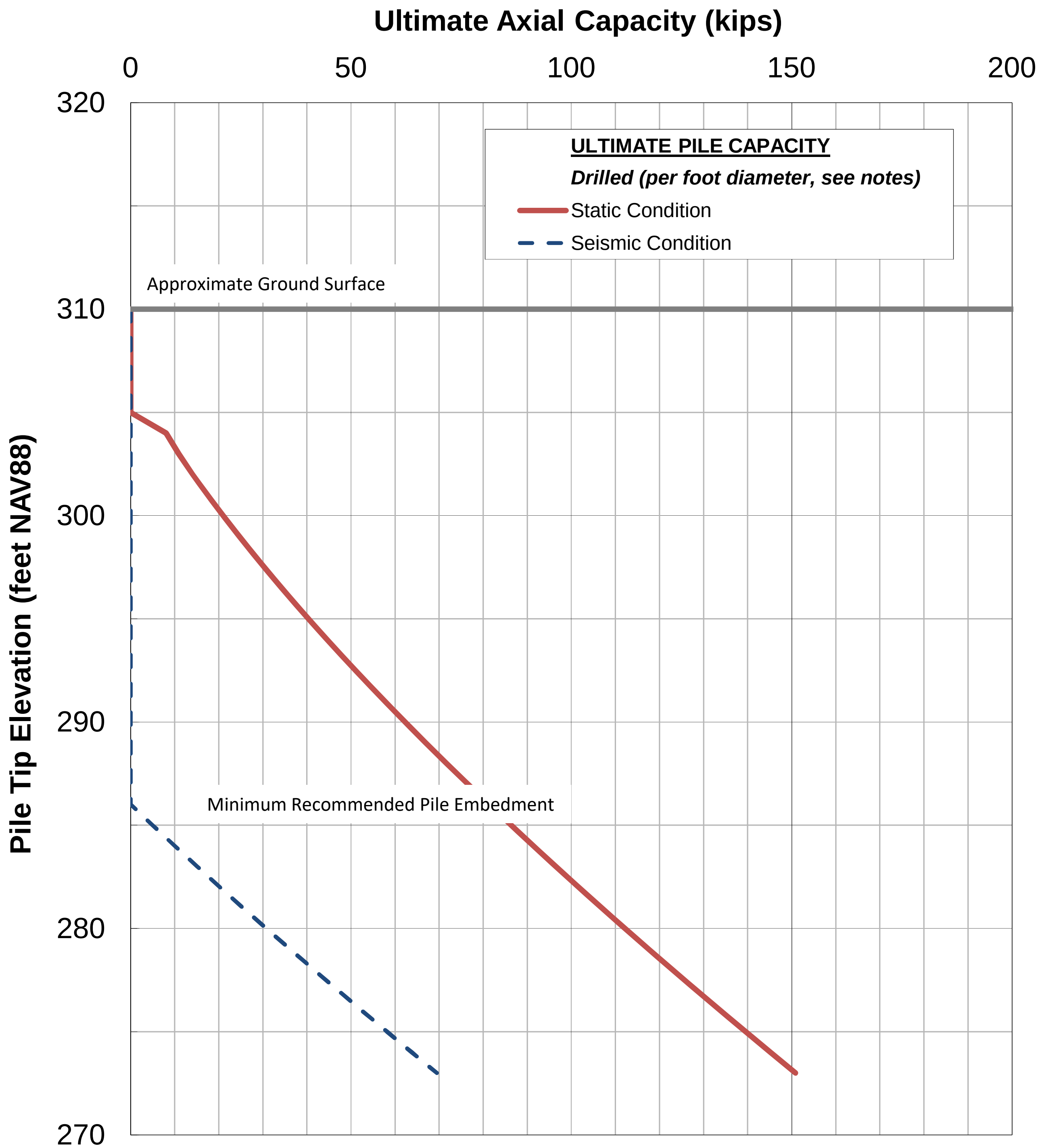
Table 1 - Laboratory Tests on Soil Samples

HDR, Irvine
BNSF Needles 3MT
Your #10122333, HDR Lab #19-0490LAB
8-Aug-19

Sample ID			A-19-035 @ 15'	A-19-055 @ 30'	A-19-055 @ 65'	A-19-057 @ 70'	A-19-058 @ 50-55'
Resistivity		**Units**					
as-received		ohm-cm	208,000	208,000	56,000	13,200	68,000
minimum		ohm-cm	5,200	3,560	1,800	1,360	5,200
pH			9.7	9.8	9.2	8.5	9.0
Electrical Conductivity		mS/cm	0.19	0.19	0.30	0.31	0.12
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	28	21	26	54	62
magnesium	Mg^{2+}	mg/kg	4.8	5.1	4.8	7.0	6.6
sodium	Na^{1+}	mg/kg	244	219	365	347	55
potassium	K^{1+}	mg/kg	31	37	51	105	53
Anions							
carbonate	CO_3^{2-}	mg/kg	101	89	89	72	ND
bicarbonate	HCO_3^{1-}	mg/kg	241	256	564	637	332
fluoride	F^{1-}	mg/kg	ND	ND	ND	ND	ND
chloride	Cl^{1-}	mg/kg	1.0	15	0.9	1.4	9.6
sulfate	SO_4^{2-}	mg/kg	8.3	21	11	13	31
phosphate	PO_4^{3-}	mg/kg	ND	ND	ND	ND	ND
Other Tests							
ammonium	NH_4^{1+}	mg/kg	ND	ND	ND	ND	ND
nitrate	NO_3^{1-}	mg/kg	4.9	5.6	8.5	9.8	24
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	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 Pile Capacity Slope Stability Analyses

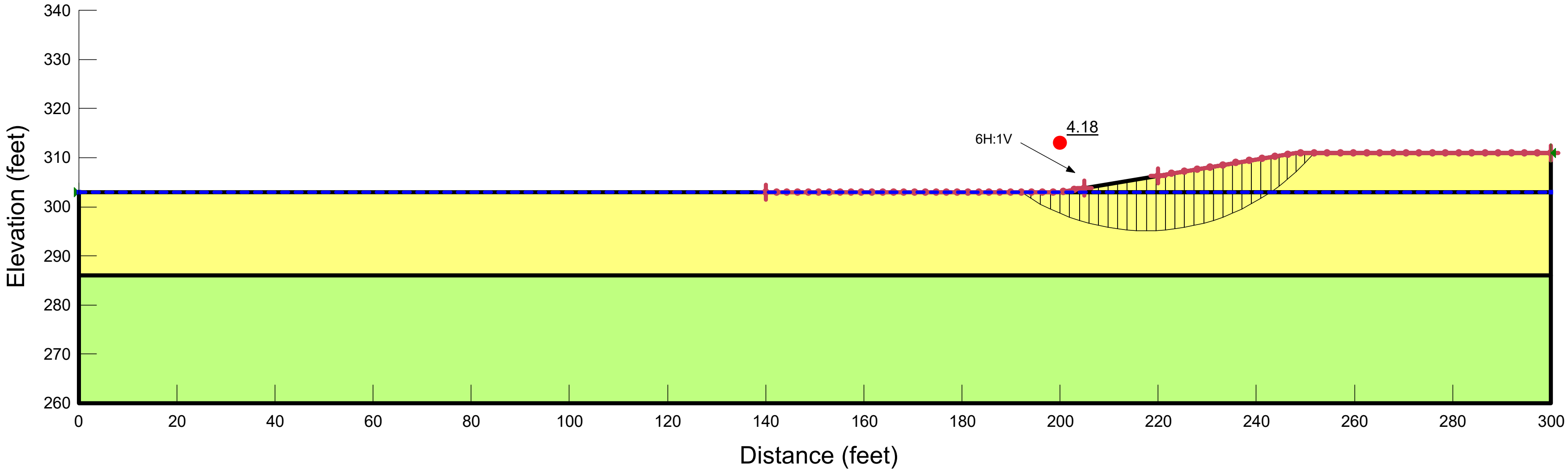


Notes:

1. Values presented are ultimate capacities and should be factored as appropriate. End bearing not considered.
2. Calculated for one-foot diameter pile. Capacities for other diameters may be extrapolated linearly.
3. 70% of compression pile capacity can be used for tension pile capacity, plus downdrag load as appropriate.
4. Analysis performed for single pile and is applicable to piles spaced at least three diameters on center.
5. Design groundwater depth of 8 feet used in analyses.
6. An estimated downdrag force of 25 kips per foot diameter should be added to the pile load for seismic analyses.

DRILLED PILE ULTIMATE PILE CAPACITY General Design Case Roman Creek Vista, California	PROJECT NO : DESIGNED BY: CHECKED BY: DATE:	10156443 MG MF 3/24/2020	HDR

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	Sand with Silt	120	0	38



Slope Stability Analysis

Cross Section A-A'

Proposed Conditions | 6H:1V Slope

Static Case

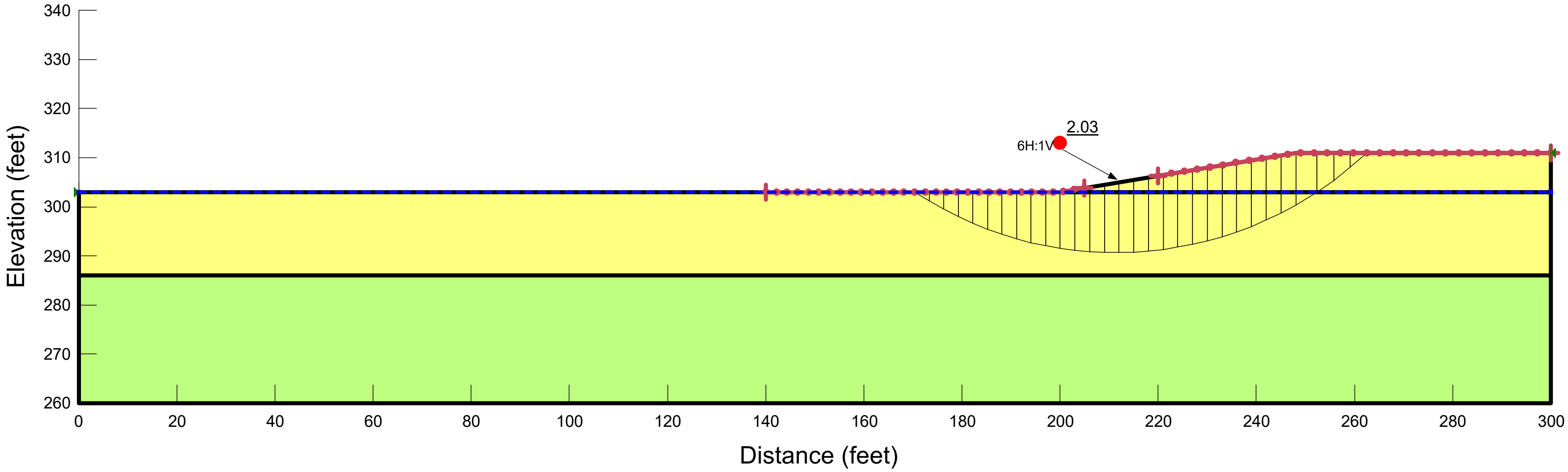
Roman Creek Mitigation Project

Vista, San Diego County, California

Figure

1

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	Sand with Silt	120	0	38



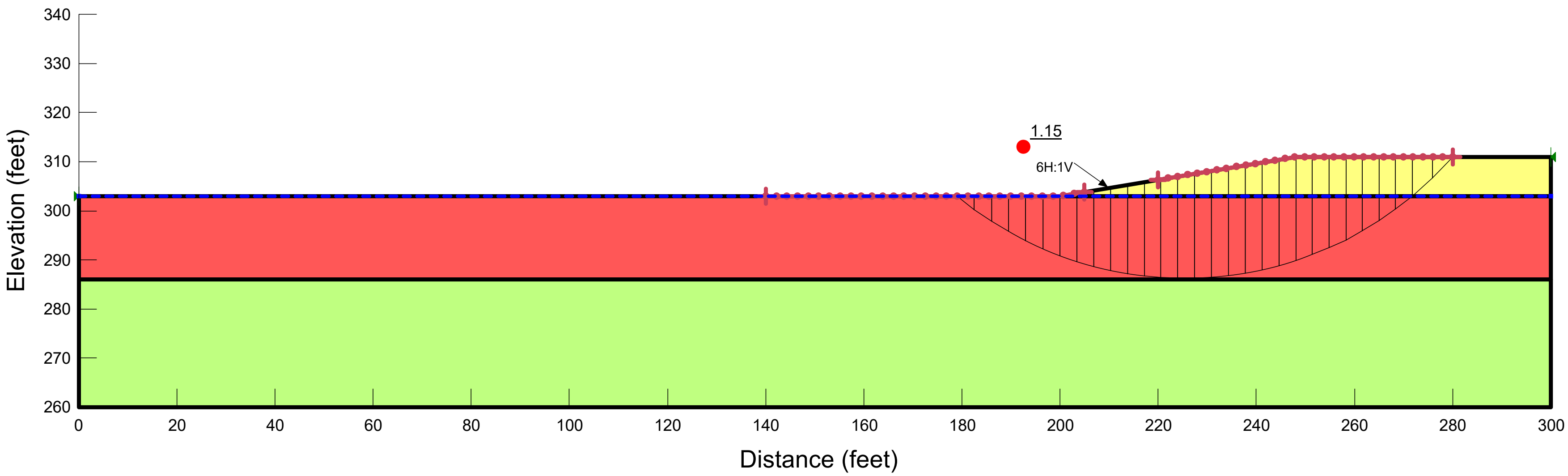
Kh= 0.15

HDR

Slope Stability Analysis
Cross Section A-A'
Proposed Conditions | 6H:1V Slope
Seismic Case
Roman Creek Mitigation Project
Vista, San Diego County, California

Figure
2

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	liquefiable soil	120	400	0
	Sand with Silt	120	0	38



Kh= 0.15

Slope Stability Analysis

Cross Section A-A'

Proposed Conditions | 2H:1V Slope

Liquefaction Case

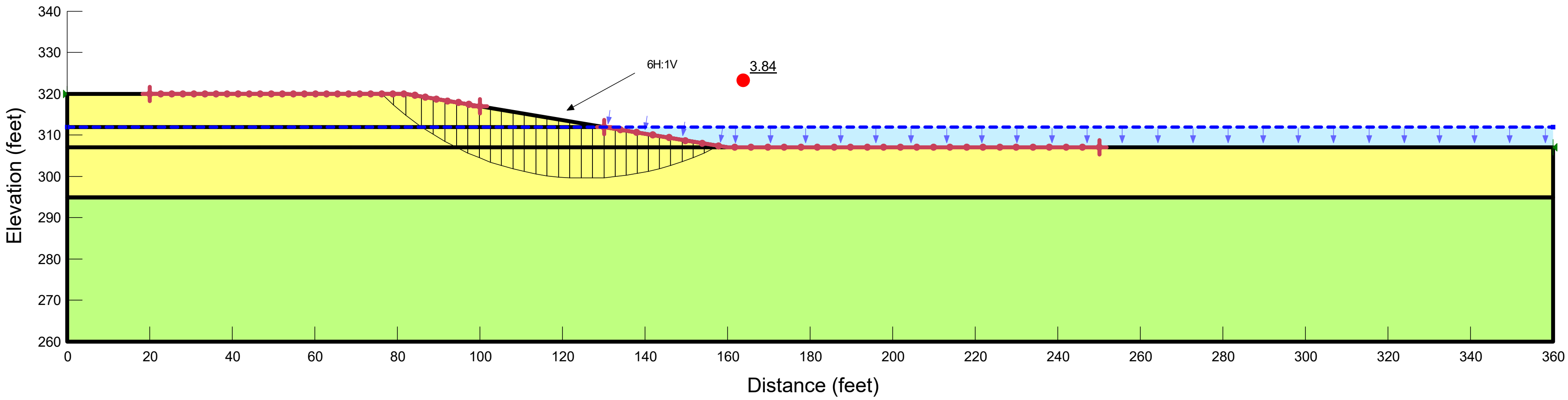
Roman Creek Mitigation Project

Vista, San Diego County, California

Figure

3

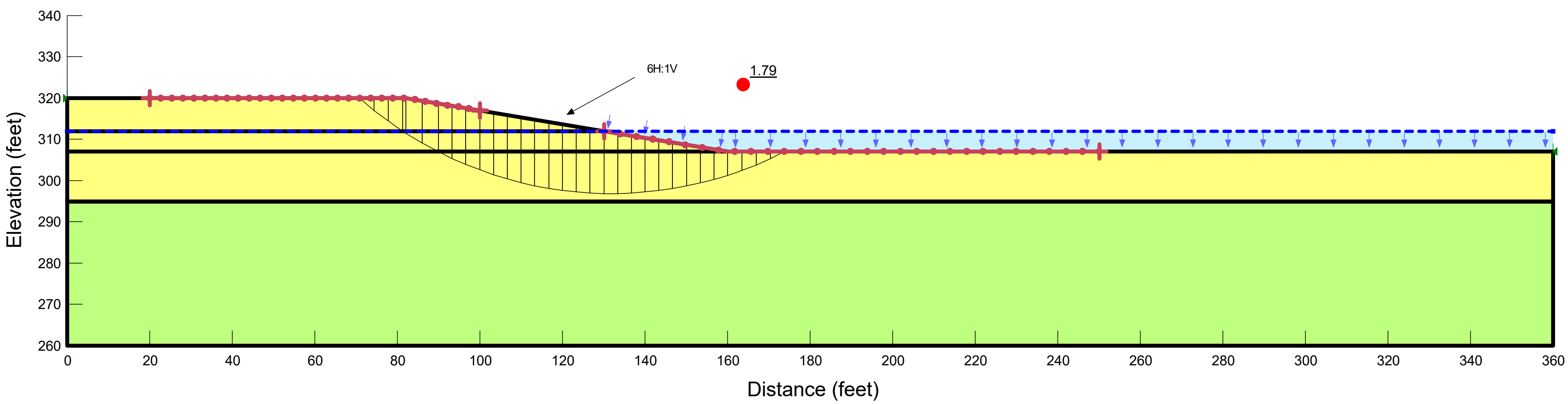
Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	Sand with Silt	120	0	38



Slope Stability Analysis
Cross Section B-B'
Proposed Conditions | 6H:1V Slope
Static case
Roman Creek Mitigation Project
Vista, San Diego County, California

Figure
4

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	Sand with Silt	120	0	38



Kh= 0.15

Slope Stability Analysis

Cross Section B-B'

Proposed Conditions | 6H:1V Slope

Seismic Case

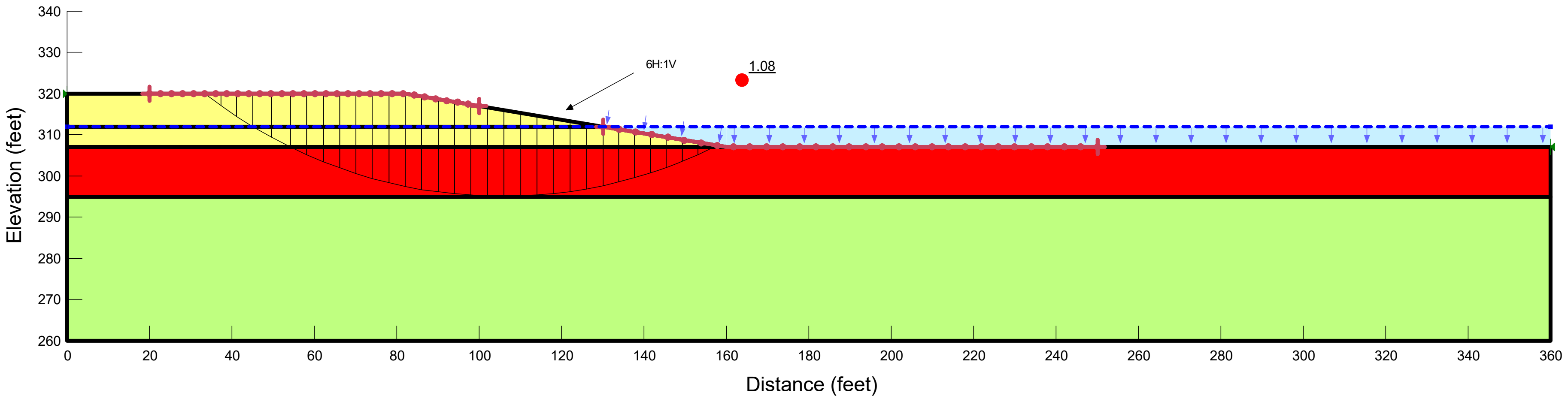
Roman Creek Mitigation Project

Vista, San Diego County, California

Figure

5

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	liquefiable sand	120	400	0
	Sand with Silt	120	0	38



Kh= 0.15

Slope Stability Analysis

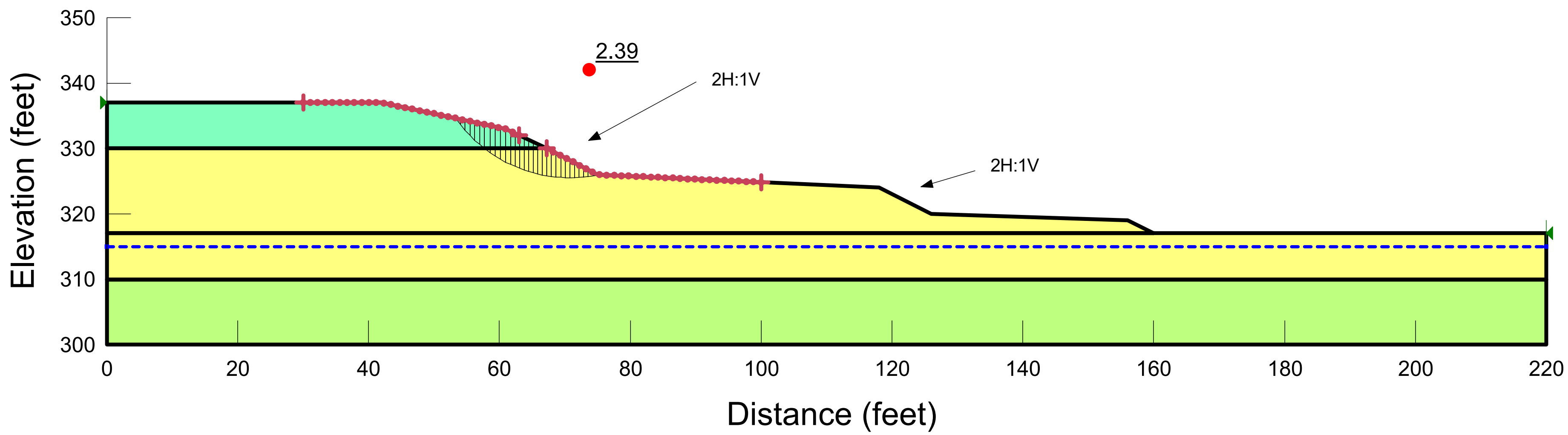
Cross Section B-B'
Proposed Conditions | 6H:1V Slope
Liquefiable case

Roman Creek Mitigation Project
Vista, San Diego County, California

Figure

6

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	Sand with Silt	120	0	38
	Silty Sand	120	0	30



Slope Stability Analysis

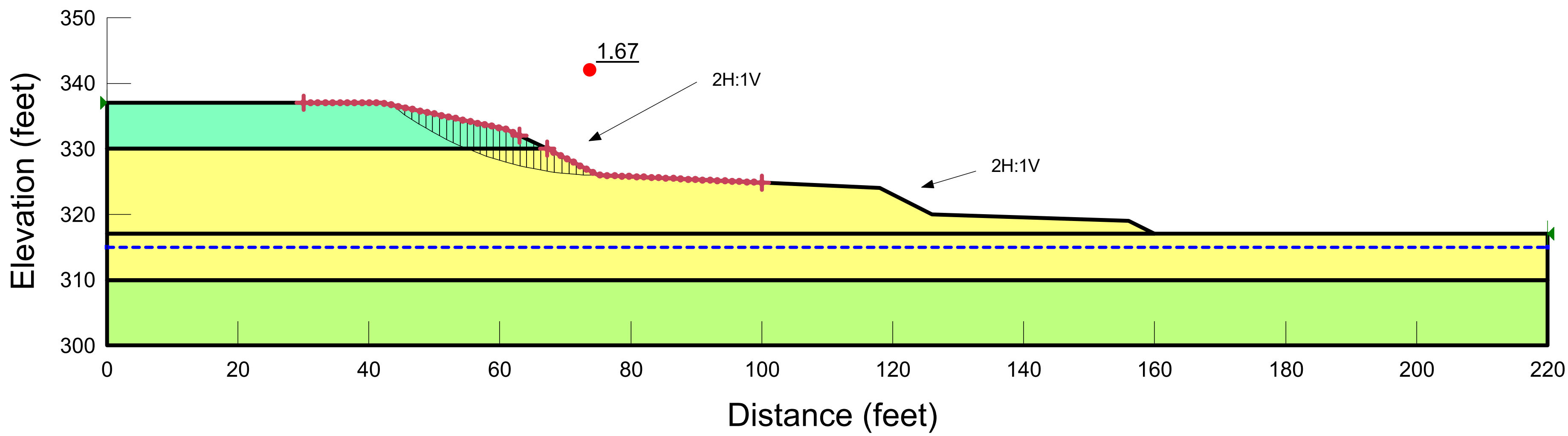
Cross Section C-C'
Proposed Conditions | 2H:1V
Static Case

Roman Creek Mitigation Project
Vista, San Diego County, California

Figure

7

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	Sand with Silt	120	0	38
	Silty Sand	120	0	30



Kh= 0.15

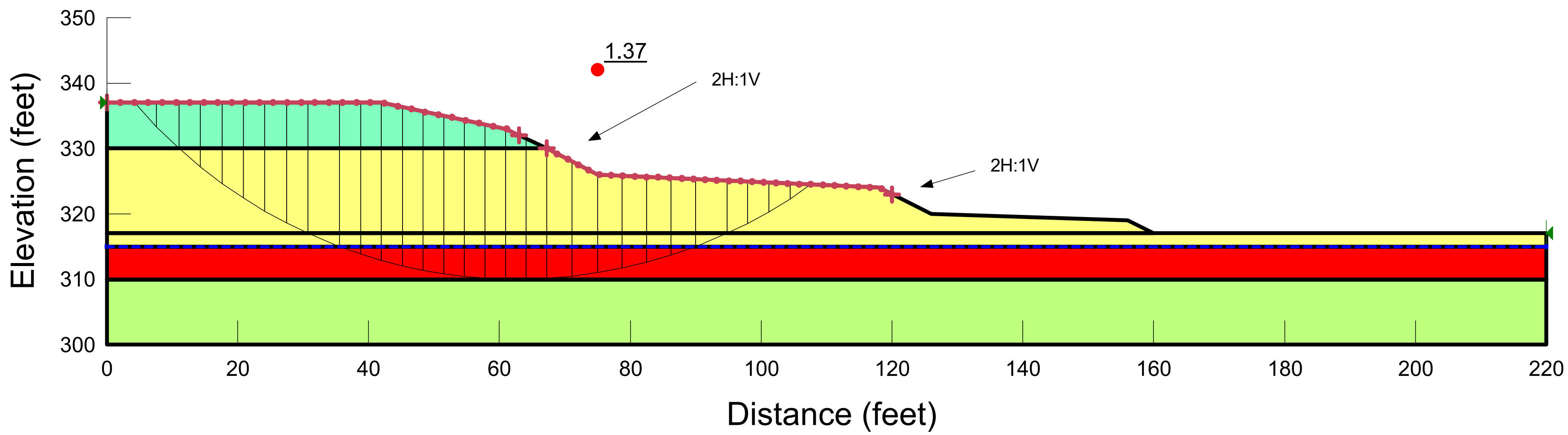
Slope Stability Analysis

Cross Section C-C'
Proposed Conditions | 2H:1V
Seismic Case
Roman Creek Mitigation Project
Vista, San Diego County, California

Figure

8

Color	Name	Unit Weight (pcf)	Cohesion' (psf)	Phi' (°)
	Clayey Sand	120	100	31
	liquefiable sand	120	400	0
	Sand with Silt	120	0	38
	Silty Sand	120	0	30



Kh= 0.15

Slope Stability Analysis

Cross Section C-C'

Proposed Conditions | 2H:1V

Liquefaction Case

Roman Creek Mitigation Project

Vista, San Diego County, California

Figure

9