INLAND FOUNDATION ENGINEERING, INC.
Consulting Geotechnical Engineers and Geologists
P.O. Box 937, San Jacinto, CA 92581

December 30, 2020
Project No. R206-025

RIVERSIDE COUNTY FLOOD CONTROL and
WATER CONSERVATION DISTRICT
Attention: Mr. Helio K. Takano, P.E., Engineering Project Manager
1995 Market Street
Riverside, California 92501

Subject: Geotechnical Investigation Report
Wildomar MDP Lateral C, Stage 3
Project No. 7-0-00075-03

Dear Mr. Takano:

We are pleased to submit the results of our preliminary geotechnical report conducted for the referenced project. Our study was conducted in general conformance with our proposal dated March 5, 2020 and subsequent discussions with the District.

The report includes geotechnical engineering recommendations for project design and construction, along with field and laboratory data. The primary geotechnical issues that will require mitigation appear to be the limited right-of-way and loose alluvial soil within the area of the proposed stilling basin and transition structure. Shoring will likely be necessary for excavation and foundation construction in this area.

We appreciate the opportunity of being of service to you on this project. If there are any questions, please contact our office.

Respectfully,
INLAND FOUNDATION ENGINEERING, INC.

Daniel R. Lind

DANIEL R. LIND
NO. 2451
CERTIFIED ENGINEERING GEOLOGIST

Daniel R. Lind, P.G., C.E.G.
Principal Geologist

ALLEN D. EVANS

Allen D. Evans, P.E., G.E.
Principal

DRL:ADE:es
Distribution: Addressee

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INTRODUCTION

This report presents the results of the geotechnical investigation conducted for the proposed Riverside County Flood Control and Water Conservation District Wildomar MDP Lateral C, Stage 3 project [Project No. 7-0-00075-03]. The project will consist of the design and construction of an incised detention basin on the southeast corner of the intersection of Bundy Canyon Road and Monte Vista Drive in the City of Wildomar. In addition to the detention basin and appurtenant structures, the project also includes the design and construction of approximately 900 feet of RCB storm drain, and relocation of approximately 900 feet of an existing EVMWD 8-inch sewer line. The following plans were reviewed and utilized in the preparation of this report.

- Preliminary 30% Plans for Wildomar MDP Lateral C, Stage 3, Sheets 1-11, prepared by Riverside County Flood Control and Water Conservation District, dated January 2020.

- Elsinore Valley Municipal Water District, Wildomar MDP Lateral C STG 3 Sewer Improvement Plans in the City of Wildomar (Preliminary 30%), Sheets 1 and 2, prepared by Riverside County Flood Control and Water Conservation District, undated.

- As-Built Plans, Riverside County Flood Control & Water Conservation District, Tract 23281, Bundy Canyon Channel, Sheets 1-7 of 7, prepared by Gabel, Cook & Becklund, dated June 25, 1992.

SCOPE OF SERVICE

The purpose of the geotechnical investigation was to develop geotechnical parameters for design and construction of the proposed project. The scope of the geotechnical services included:

- *Review of the general geologic conditions and specific subsurface conditions of the project site.*

- *Evaluation of the engineering and geologic data collected for the project.*

- *Preparation of this report with geotechnical conclusions and recommendations for design and construction.*

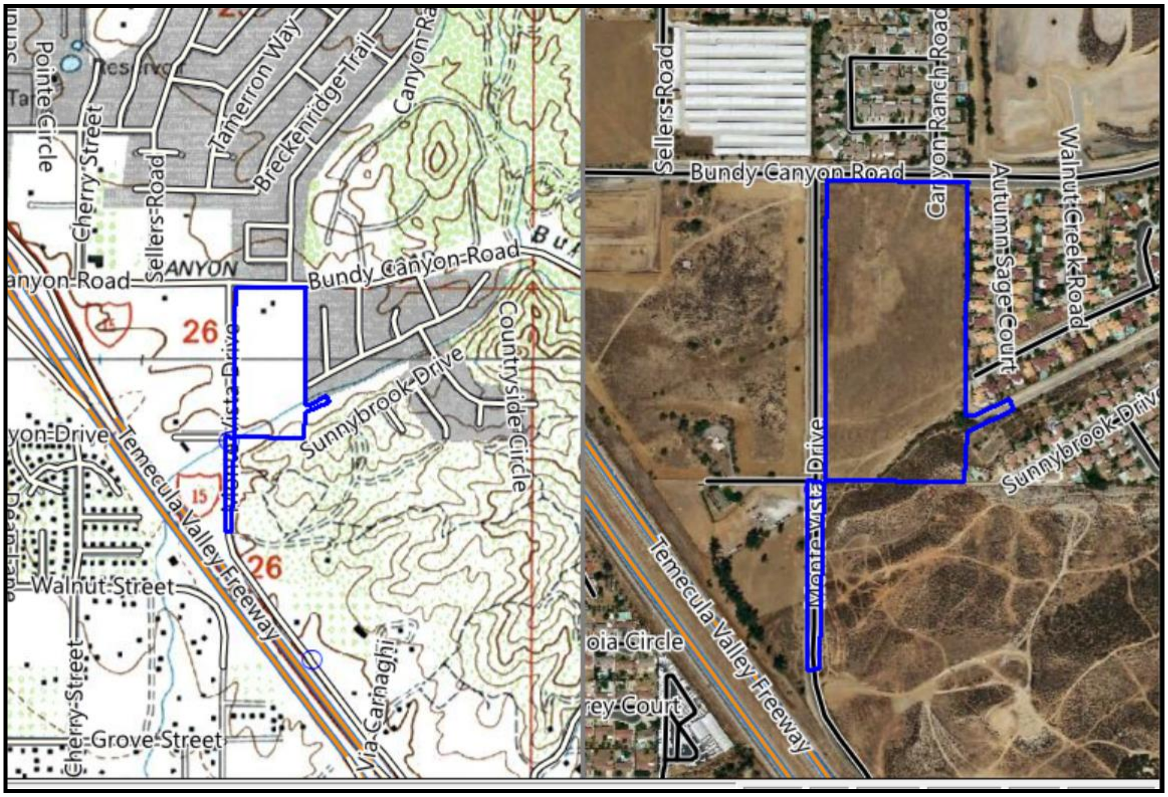
The tasks performed in order to achieve these objectives included:

- *Collection and review of soil and geologic data in order to develop an exploration program*
- *Subsurface exploration to evaluate the nature and stratigraphy of the subsurface soils and to obtain representative samples for laboratory testing*
- *Visual reconnaissance of the detention basin site, storm drain alignment and surrounding area to ascertain the presence of unstable or adverse geologic conditions*
- *Laboratory testing of representative samples to evaluate the classification and engineering properties of the soils*
- *Seismic refraction survey to evaluate the site excavation characteristics and depth to bedrock*
- *Seismic shear wave survey to evaluate the average shear wave velocity within the upper 100 feet for purposes of the site-specific ground motion analysis*
- *Infiltration testing to evaluate the permeability of the site soil and bedrock for basin design*
- *Analysis of the data collected and the preparation of this report with our geotechnical conclusions and recommendations.*

SITE DESCRIPTION

The proposed flood control project is located in the northerly portion of Section 26, Township 6 South, Range 4 West, S.B.B.&M. The basin site is located on the southeast corner of the intersection of Bundy Canyon Road and Monte Vista Drive in the City of Wildomar and occupies approximately 15 acres. The locations of the proposed detention basin and associated flood control improvements are shown on Figure 1 below.

Figure 1: USGS Topographic Map, Wildomar 7.5' Quadrangle, and Aerial Photograph (2018)



The referenced project plans indicate the topography of the site generally slopes to the south-southwest. Ground surface elevations across the detention basin site range from approximately 1,435 feet msl at the northerly site boundary to approximately 1,404 feet msl at the southwest site corner. A 10 to 15 feet high knoll is present on the north end of the site that slopes to the south from Bundy Canyon Road. A ± 90 feet high hill is present off-site and adjacent to the southeast corner of the site.

A large pile of rock, debris and gravelly fill material, approximately six feet high, was present in the central portion of the site at the time of our investigation. Plastic silt fencing was also present.

Historic imagery shows an apparent single-family residence and several other structures in the north-central portion of the site. The residence and other structures were apparently demolished sometime between January 2006 and June 2009.

At the time of our field investigation, vegetation consisted of a moderate growth of seasonal weeds and grass across most of the site. Mowing operations were being conducted during our study. A group of mature trees were present within the westerly portion of the site and along the southeast site boundary.

The existing improved Bundy Canyon Channel - Lateral A outlets on to the southeast portion of the basin site. Runoff from Bundy Canyon Channel flows in a natural drainage course along the south site boundary that has been designated as protected riparian habitat. Site runoff is collected within an existing concrete lined channel that runs from the southwest site corner south along the east side of Monte Vista Drive.

An existing EVMWD sewer line crosses the southerly portion of the proposed basin site.

PROJECT DESCRIPTION

The District intends to construct an incised detention basin on the southeast corner of the intersection of Bundy Canyon Road and Monte Vista Drive in the City of Wildomar. The proposed basin will be located on District owned parcels and will occupy approximately 15 acres. The basin site is bounded to the east by existing residential development and to the south by a natural drainage course (Bundy Canyon). Bundy Canyon Channel Lateral A is currently a concrete-lined trapezoidal channel that directs flows southwesterly into a natural wash. Flows run through the wash, drain into a trapezoidal channel along Monte Vista Drive, flow into a culvert, and cross beneath Monte Vista Drive and Interstate 15 (I-15). The proposed basin will attenuate the flows that would otherwise flow directly downstream across I-15. The proposed project has the following components:

- **Detention (flow-through) Basin:** An incised detention basin located on District owned parcels (APNs 367-110-007 and -008). The northern portion of the basin will have a depth of approximately 45 feet and the southern portion of the basin will have a depth of approximately 18 feet. The maximum basin side slopes will be 3H:1V.

- **Basin Inlet Channel**: The basin inlet channel will generally be located in the southeast basin corner and will tie into an existing trapezoidal channel (Bundy Canyon Channel Lateral A). It will include a transition structure (125 LF), USBR Type III stilling basin (107 LF), Caltrans double RCB (144 LF) and a riprap chute. The stilling basin will be incised in the existing ground with reinforced concrete sidewalls as high as 25 feet.

- **Basin Outlet Storm Drain**: The outlet drain will be located in the southwest corner of the basin and will include a grated intake structure, double RCB (17 LF), and transition structure (30.5 LF). From the transition structure, a single RCB (789 LF) and double RCB (60 LF) will convey flow south along the east side of Monte Vista Drive.

- **Basin Intake Structure**: The basin intake structure will be designed to convey 1,080 cfs from the detention basin into the basin outlet storm drain. The intake structure will be designed with a SPPWC slope protection barrier (Std. 360-2).

- **54-inch Storm Drain Relocation**: An existing storm drain will be relocated with approximately 144 LF of 54-inch RCP with approximately 4 feet of cover. The storm drain will convey flows from the existing storm drain in Valley Vista Circle on the east side of the basin into an impact basin on the detention basin bottom.

- **Low Flow Storm Drain**: Approximately 158 LF of 36-inch RCP will convey small storm flows from the USBR Type III stilling basin to the basin bottom.

- **Storm Drain Extension**: Approximately 177 LF of 48-inch RCP storm drain will convey flows from an existing storm drain in Monte Vista Drive to an impact basin in the northwest corner of the basin bottom.

- **EVMWD Sewer**: Approximately 914 feet of Elsinore Valley Municipal Water District (EVMWD) 8-inch PVC sewer line is to be relocated from Valley Vista Circle along the centerline of the detention basin maintenance road to an existing sewer in Monte Vista Drive. The relocated sewer will cross above the new basin inlet channel and basin outlet storm drain.

- **Retaining Walls**: Several retaining walls will be constructed around and in the basin. The walls will be designed using Caltrans Standard Plan Nos. B3-7B, B3-4B and B3-1B. The three types are all cantilever retaining walls with spread footings.

GEOLOGIC SETTING

Regional Geology: The subject site is situated within a natural geomorphic province in southwestern California known as the Peninsular Ranges, which is characterized by steep, elongated ranges and valleys that trend northwesterly. This geomorphic province encompasses an area that extends 125 miles, from the Transverse Ranges and the Los Angeles Basin, south to the Mexican border, and beyond another 795 miles to the tip of Baja California (Norris & Webb, 1990; Harden, 1998). This province is believed to have originated as a thick accumulation of predominately marine sedimentary and volcanic rocks during the late Paleozoic and early Mesozoic. Following this accumulation, in mid-Cretaceous time, the province underwent a pronounced episode of mountain building. The accumulated rocks were then complexly metamorphosed and intruded by igneous rocks, known locally as the Southern California Batholith. A period of erosion followed the mountain building, and during the late Cretaceous and Cenozoic time, sedimentary and subordinate volcanic rocks were deposited upon the eroded surfaces of the batholithic and pre-batholithic rocks.

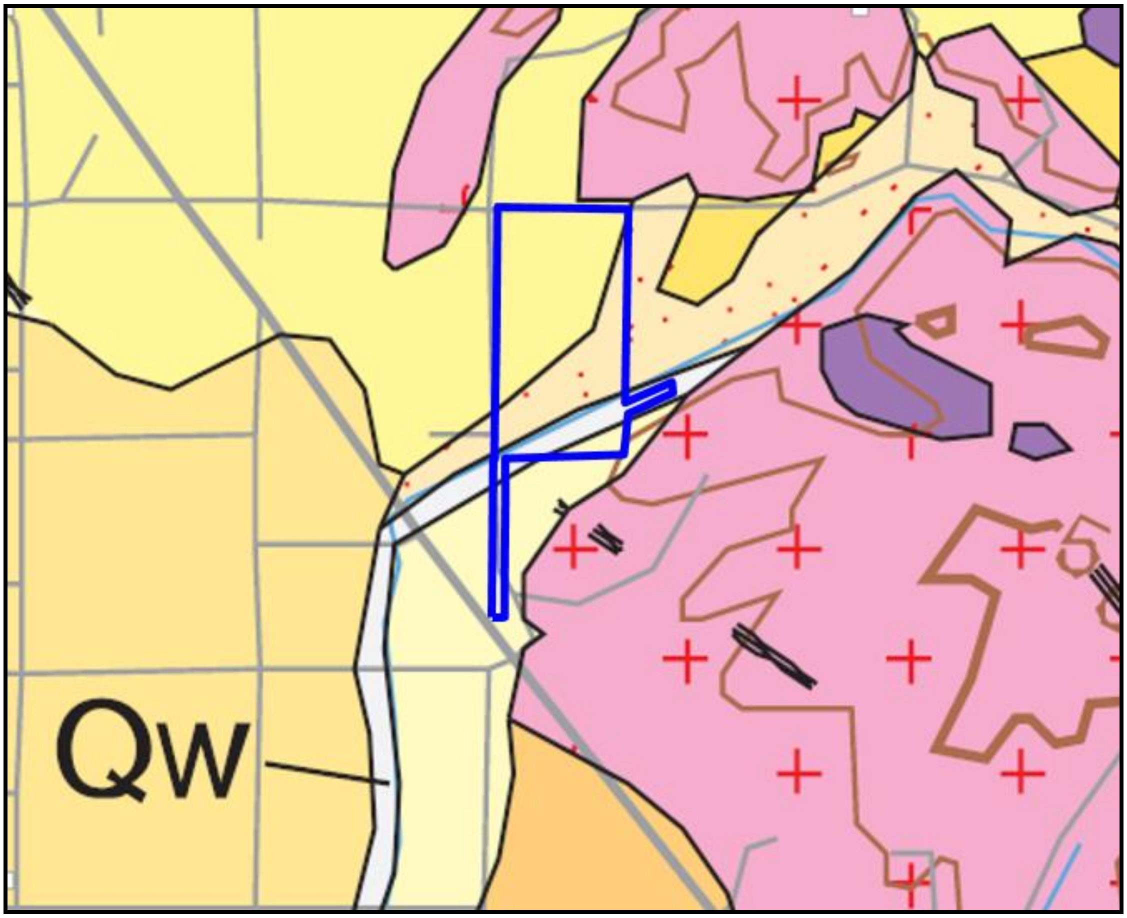
Local Geology: More specifically, the site lies along the westerly fringe of the Perris Block, an eroded mass of Cretaceous and older crystalline rock. Thin sedimentary and volcanic units mantle the bedrock in a few places with alluvial deposits filling in the lower valley areas. The Perris Block is a structurally stable, internally unfaulted mass of crustal rocks bounded on the west by the Elsinore-Chino fault zones, on the east by the San Jacinto fault zone, and on the north by the Cucamonga fault zone (Woodford, et al., 1971). On the south, the Perris Block is bounded by a series of sedimentary basins that lie between Temecula and Anza (Morton and Matti, 1989).

Locally, as mapped by Morton & Miller (2006), the study area is underlain by several distinct geologic units including young alluvial fan deposits (map symbol Qyf), very young wash deposits (map symbol Qw), young axial channel deposits (map symbol Qya), and monzogranite and granodiorite bedrock (map symbol Kpvg).

The young (Holocene and late Pleistocene) alluvial fan deposits (Qyf) are described as unconsolidated to moderately consolidated silt, sand, pebbly cobbly sand, and bouldery alluvial fan deposits. The very young (late Holocene) wash deposits (Qw) in the Bundy Canyon Wash are described as unconsolidated sand and gravel deposits. The mapped young (Holocene and late Pleistocene) axial-channel deposits (Qya) are described as slightly to moderately consolidated silt, sand, and gravel deposits. The granitic bedrock (map symbol Kpvg) is desribed by Morton and Miller as pale gray, massive, medium grained hypidiomorphic-granular biotite monzogranite.

Figure 2 below is a portion of the USGS Geologic Map of the San Bernardino and Santa Ana 30’ x 60’ Quadrangle (Morton & Miller, 2006) depicting the mapped geologic units in the vicinity of the project.

Figure 2: **USGS Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangl (Morton & Miller, 2006)**

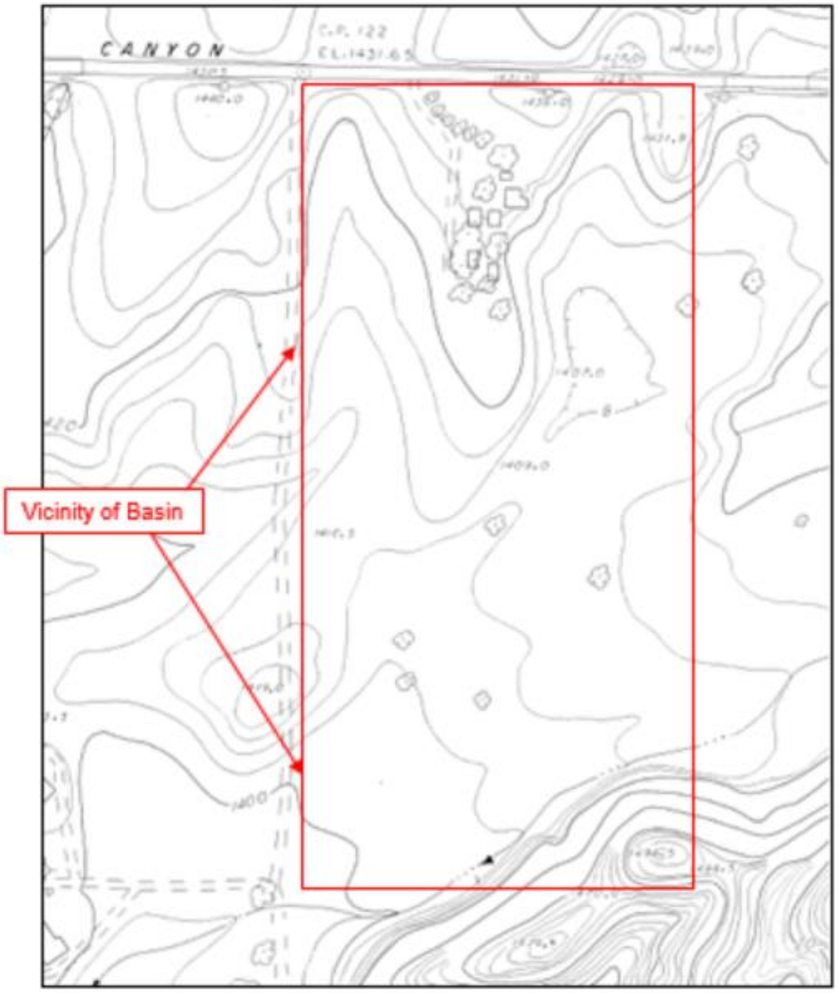


🟨	Qyf	**Young alluvial-fan deposits (Holocene and late Pleistocene)**—Unconsolidated to moderately consolidated silt, sand, pebbly cobbly sand, and bouldery alluvial-fan deposits having slightly to moderately dissected surfaces
⬜	Qw	**Very young wash deposits (late Holocene)**—Unconsolidated sand and gravel deposits
⁚	Qya	**Young axial-channel deposits (Holocene and late Pleistocene)**—Slightly to moderately consolidated silt, sand, and gravel deposits.
+	Kpvg	**Monzogranite to granodiorite**—Pale gray, massive, medium-grained hypidiomorphic-granular biotite monzogranite

Geomorphically, the study area is situated along a gently south-southwesterly sloping alluvial fan that has been largely created by outwash originating from Bundy Canyon to the east. A ± 90 feet high hill is present off-site and adjacent to the southeast corner of the site. The elevated knoll adjacent to Bundy Canyon Road in the northerly portion of the site is roughly peninsular-shaped. Slopes on the order of 10 feet high are present on the east side of the elevated area that exhibit minor shallow sloughing.

Figure 3 below is a portion of Riverside County Flood Control and Water Conservation District Topographic Map, Section 26, T. 6 S., R. 4 W., SBB&M, based on aerial photography dated November 23, 1964. This map shows the location of former structures near the north-central portion of the site and the mapped topography of the site at that time.

Figure 3: RCFCWCD Topographic Map, Section 26, T. 6 S., R. 4 W., SBB&M, 1964



We reviewed a report entitled "Preliminary Geotechnical Investigation, Tentative Tract Map No. 31409, Wildomar Area, Riverside County, California", dated September 8, 2003 and prepared by LOR Geotechnical Group, Inc. The report was prepared for a proposed residential development on the site. According to the report, several structures occupied the site during the 2003 study. These included a fenced, single family residence within the north-center portion of the site, and a water tank and pump/well house within the southern portion of the site. Based on the description and review of aerial photographs, it appears that the former pump/well house on the south portion of the site was located in the vicinity of GPS coordinates ±33.6236°N / 117.2659°W (WGS 84).

Figure 4 below is a three-dimensional view of the project vicinity (1:9028 scale).

Figure 4: **Three-dimensional View (Terrain Navigator Pro, 2018)**

Groundwater: Groundwater was encountered within five exploratory borings at depths of approximately 19 to 39 feet below ground surface (bgs). Table 1 below shows the locations and depths where groundwater was encountered:

Table 1: Location and Depth of Encountered Groundwater

Boring No.	Depth to Encountered Groundwater (ft.)	Approximate Ground Surface Elevation (ft. msl)
B-01	26	1,393
B-12	34	1,410
B-15	39	1,415
B-16	19	1,405
B-21	29	1,405

Groundwater was encountered in boring B-01 near the south end of the proposed storm drain adjacent to Monte Vista Drive within granitic bedrock. Groundwater was encountered in borings B-12 and B-21 within alluvial deposits above bedrock. Groundwater was encountered in borings B-15 and B-16 at the approximate depth of the alluvial bedrock contact.

Water well records complied by Watermaster Support Services and the Western Municipal Water District (2019) were reviewed for this project. State Well No. 06S/04W-27P001S, (measured; June 2006), located approximately one mile southwest of the site, had a groundwater depth of ±145 feet.

Groundwater records compiled by the California Department of Water Resources (DWR) were also reviewed. State Well No. 06S/04W-26L001S, (measured; March 1968, located approximately 0.3 miles to the south of the proposed basin), had a measured groundwater depth of 61 feet below the existing ground surface. State Well No. 06S/04W-26E002S, (measured; March 1968, located approximately 0.45 miles to the southwest of the proposed basin) had a measured groundwater depth of 44 feet below the existing ground surface at that time.

Faulting/Seismicity: The project site does not lie within a mapped State of California Alquist-Priolo Earthquake Fault Zone or mapped Riverside County fault zone. There are at least 36 major "potentially active/active" (late Quaternary) faults that are within a 100 kilometer (62 mile) radius of the subject site (Blake, 1989-2000b). Of these, there are no known active faults that traverse the site based on available published literature, nor is there any surficial geomorphic or photogeologic evidence suggestive of faulting. In addition, the site is not located within a State of California Earthquake Fault Zone for fault rupture hazard (CGS, 2018). The nearest known active fault is the Glen Ivy North Fault (Elsinore Fault Zone), located approximately 0.25 miles to the southwest. The Glen Ivy Fault is one of the central strands of the Elsinore Fault Zone System (Glen Ivy segment), which runs from the Los Angeles Basin to the north and into Mexico to the south. The Glen Ivy Fault is a right-lateral, strike-slip fault, approximately 36 kilometers in length (CDMG, 1996 and Cao, et al., 2003).

Figure 5 shows an enlarged portion of the 2010 Fault Activity Map of California (CGS, 2010) depicting the site location and mapped faults in the region.

Figure 5: 2010 Fault Activity Map of California (CGS, 2010)

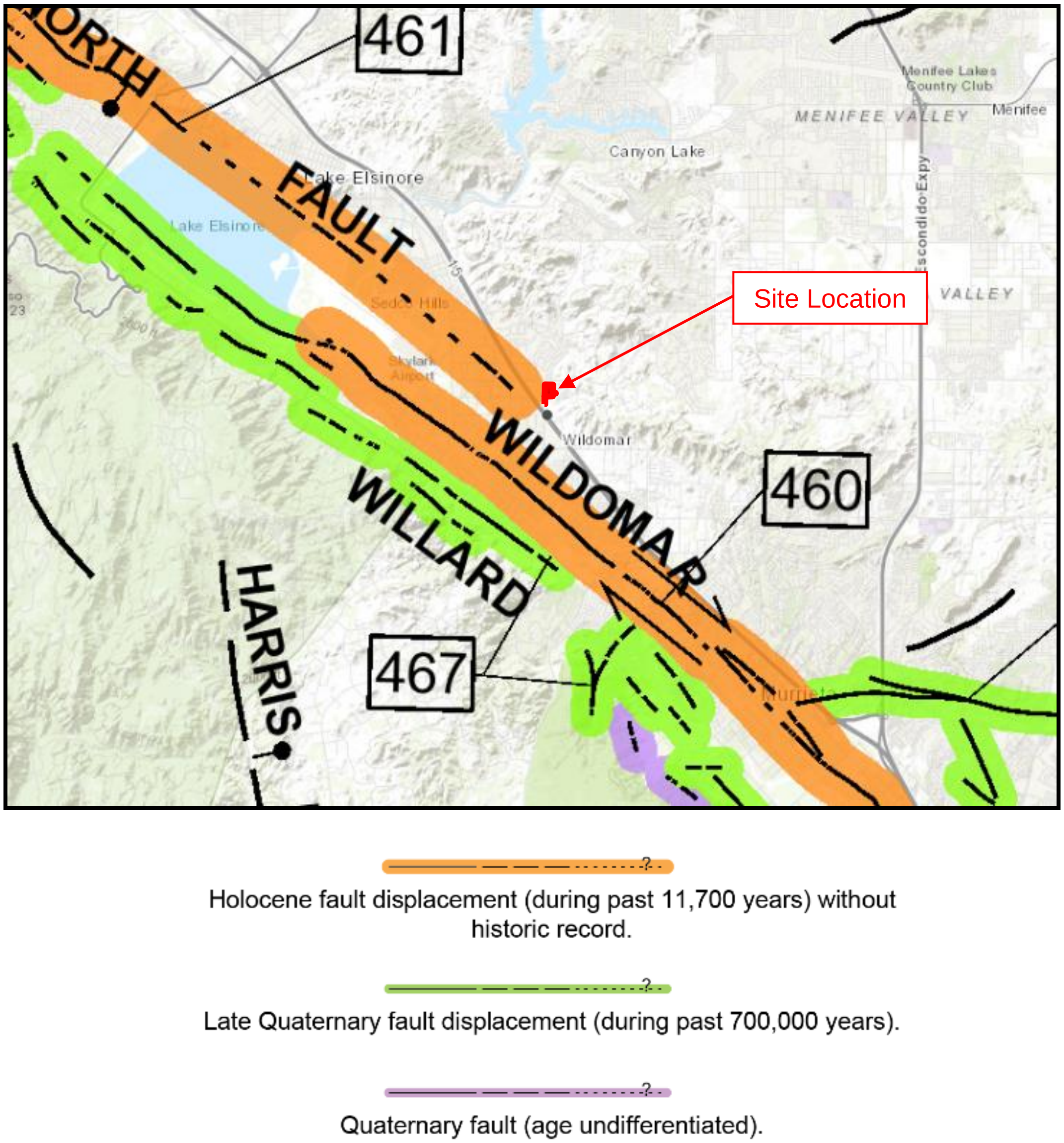
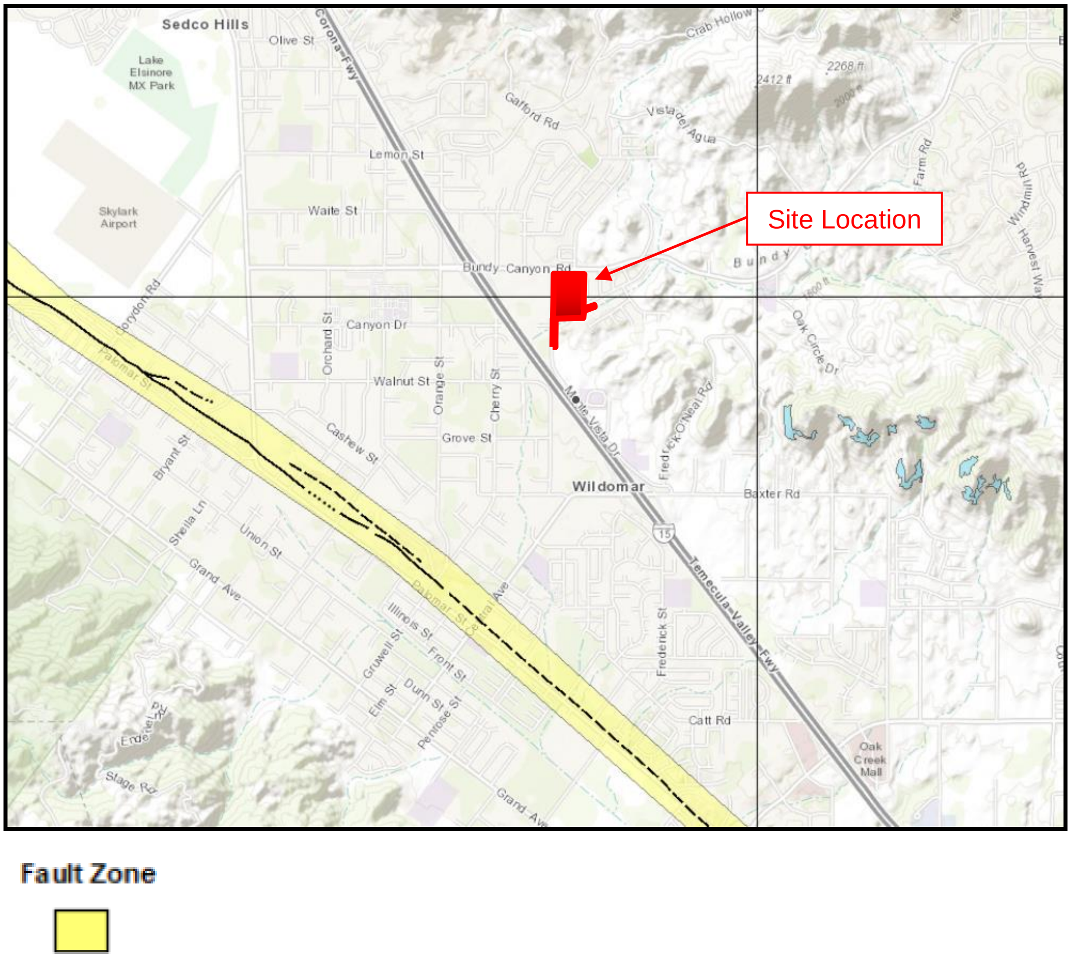


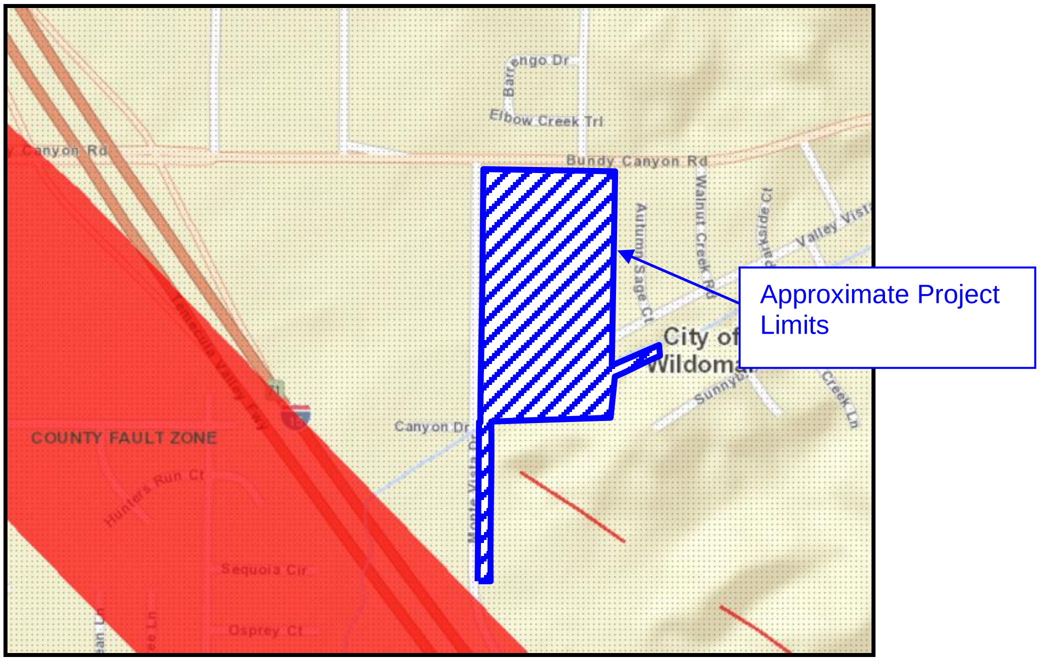
Figure 6 below shows a portion of the State of California *Earthquake Zones of Required Investigation Map* (CGS, 2020) for the project area.

Figure 6: State of California Earthquake Zones of Required Investigation Map (CGS, 2020)



A Riverside County fault zone associated with the Glen Ivy Section of the Elsinore Fault Zone (Glen Ivy North Fault) is mapped just to the south-southwest of the southernmost limit of the project. An unnamed fault is located south and east of the project site. This appears to be the same fault also shown on the referenced geologic map by Morton & Miller (Figure 2). Figure 7 below shows a portion of the Riverside County RCIT GIS map with the location of the mapped County fault zone (shaded in red).

Figure 7: Riverside County RCIT GIS Map



Major faults influencing the site, approximate distances and maximum earthquake magnitudes are shown in Table 2.

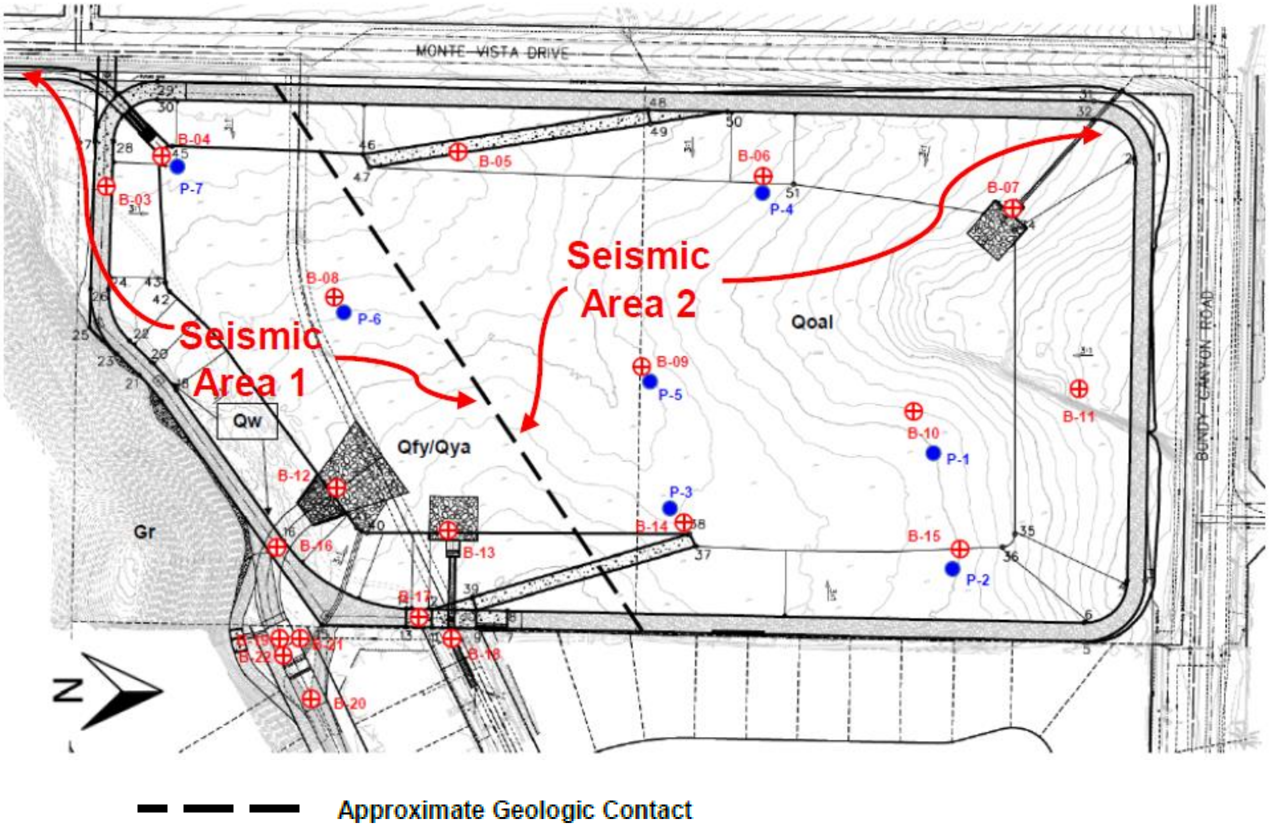
Table 2: Fault Zone, Approximate Distances and Maximum Earthquake Magnitudes

Fault Zone	Approximate Distance (Km)	Earthquake Magnitude (M_w)
Elsinore-Glen Ivy	0.40	6.8
Elsinore-Temecula	1.75	6.8
San Jacinto-San Jacinto Valley	29.8	7.2
San Jacinto-Anza	34.5	7.2

Evaluation of the potential for surface fault rupture included an examination of one non-stereo and nine stereo pairs of vertical black and white aerial photographs dating from 1962 to 2018 (see References for a listing) to aid in assessing the geologic and geomorphic characteristics with respect to the site and vicinity. The photogeologic analysis did not reveal indicators suggestive of active fault-related features on the subject site. No surficial indications or geomorphic features were observed within the aerial photographs or field reconnaissance that are suggestive of active faulting.

Seismic Parameters: Because of the differing geologic conditions between the wash deposits in the south portion of the site and the rest of the site, separate site-specific ground motion analyses were conducted. Within this report the separate areas are referred to as Seismic Area 1 and Seismic Area 2. For purposes of our analyses, Seismic Areas 1 and 2 are separated by the mapped geologic contact shown on Figure 8 below. The coordinates (WGS 84) used for Seismic Area 1 are 33.6210°N / -117.2666°W. The coordinates used for the Seismic Area 2 analysis are 33.6245°N / -117.2662°W.

Figure 8: **Seismic Area Designations**



Mapped spectral acceleration parameters, coefficients, and other related seismic parameters were evaluated using the OSHPD Seismic Design Maps web application (OSHPD, 2020) and the California Building Code (CBC, 2019). The site-specific ground motion analyses were performed in accordance with Section 21 of ASCE 7-16. The results of the site-specific analysis are summarized in Table 3 below. The site-specific ground motion analyses are described in Appendix D.

Table 3: Summary of Seismic Design Parameters

Factor or Coefficient	Seismic Area 1	Seismic Area 2
S_S	1.67 g	1.66 g
S_1	0.62 g	0.62 g
S_{DS}	1.33 g	1.11 g
S_{D1}	0.58 g	0.70 g
S_{MS}	2.00 g	1.66 g
S_{M1}	0.86 g	1.05 g
T_S	0.4 sec	0.6 sec
MCE_GPGA	0.72 g	0.72 g
Site Class	C	D

Secondary Seismic Hazards: Secondary permanent or transient seismic hazards generally associated with severe ground shaking during an earthquake include, but are not limited to ground rupture, liquefaction, seiches or tsunamis, landsliding, rockfalls, and seismically-induced settlement. These are discussed below.

Ground Rupture: Ground rupture is generally considered most likely to occur along pre-existing faults. Since there are no faults that are known to traverse the site, the potential for ground rupture is considered to be low.

Liquefaction: In general, liquefaction is a phenomenon that occurs where there is a loss of strength or stiffness in the soil that can result in the settlement of buildings, ground failures, or other hazards. The main factors contributing to this phenomenon are: 1) cohesionless, granular soil with relatively low density (usually of Holocene age); 2) shallow ground water (generally less than 50 feet); and 3) moderate to high seismic ground shaking.

The results of our analysis indicate the alluvial soil within the unimproved drainage course along the south site boundary is susceptible to liquefaction in its existing condition if historic high groundwater conditions are present. We evaluated the liquefaction potential of the soil profiles encountered in Boring Nos. B-03/B-04 and B-21. The results of our evaluation indicate a potential for liquefaction to depths of about 20 and 28 feet, respectively, within the loose alluvial soil at those locations. The liquefaction analysis is described, and results are shown in Appendix E.

Seiches/Tsunamis: Based on the distance to large, open bodies of water and the elevation of the site with respect to sea level, the possibility of seiches/tsunamis is considered nil.

Landsliding/Rockfalls: Due to the low-lying relief of most of the site, the potential for landsliding due to seismic shaking is considered low. However, a ± 90 foot tall hillside is present within the southeast corner of the site. Evidence of landsliding or rockfalls associated with this feature was not observed. Our site observations do not indicate the presence of landslides along the hills to the south that would be subject to seismic instability. Surficial materials will be subject to downhill displacement during seismic shaking.

Seismically-Induced Settlement: Seismically-induced settlement generally occurs within areas of loose granular soil as a result of liquefaction or “dry sand” settlement. The results of the liquefaction analysis performed for Boring Nos. B-3/B-4 and B-21 indicate a potential seismically-induced settlement within the alluvial soil on the order of 6 and 4 inches, respectively, at those locations. Refer to Appendix E for discussion of liquefaction analysis and results.

SOIL SURVEY REVIEW

The USDA Soil Survey of Western Riverside Area, California and the NRCS Soilweb website were reviewed. This review reveals several agricultural soil types (Series) present within the project area. These include Cajalco fine sandy loam (CaF2), Placentia fine sandy loam (PiD), Yokohl loam (YbE3), Monserate sandy loam (MnD2), Greenfield sandy loam (GyD2), Honcut sandy loam (Hnc), Tujunga loamy sand (TvC), Cieneba rocky sandy loam (CkF2), and River Wash sediments (RsC). These soil series are further described as:

Cajalco fine sandy loam, 15 to 35 percent slopes, eroded (CaF2) – Soils of the Cieneba series consist of well-drained soils developed in decomposing gabbro and other basic igneous rocks. In a typical profile, the surface layer is yellowish-brown fine sandy loam about 10 inches thick. The subsoil is brown fine sandy loam and loam. At a depth of about 22 inches is weathered gabbro. Runoff is medium, and the hazard of erosion is moderate. Typically, fines content (passing No. 200 Sieve) is 25 to 50 percent, shrink-swell potential is low, have fair to poor stability, slight to medium compressibility, and poor resistance to piping.

Placentia fine sandy loam, 5 to 15 percent slopes (PiD) – Soils of the Placentia series consist of moderately well-drained soils on alluvial fans and terraces. In a typical profile, the surface layer is brown heavy clay loam 21 inches thick. The subsoil is brown sandy clay loam. The substratum is stratified sandy, gravelly, or cobbly alluvium. Runoff is medium and the hazard of erosion is moderate. Typically, fines content (passing No. 200 Sieve) is 40 to 60 percent, shrink-swell potential is low, have fair to poor stability, very high to slight compressibility, and poor to good resistance to piping.

Yokohl loam, 8 to 25 percent slopes, severely eroded (YbE3) – Soils of the Yokohl series consist of well-drained soils on old alluvial fans and terraces. These soils developed in alluvium made up of granitic materials. In a typical profile, the upper 10 inches is reddish-brown loam. The subsoil is reddish-brown and dark brown clay about 16 inches thick. At a depth of about 26 inches is a hardpan of reddish-brown coarse sand. Runoff is slow to medium and the hazard of erosion is slight. Typically, fines content (passing No. 200 Sieve) is 60 to 90 percent, shrink-swell potential is moderate to high, slow permeability, have poor to good stability, medium to high compressibility, and poor to good resistance to piping.

Monserate sandy loam, shallow, 5 to 15 percent slopes, eroded (MnD2) – Soils of the Monserate series consist of well-drained soils that developed in alluvium from predominately granitic materials. In a typical profile, the surface layer is brown and yellowish-red sandy loam about 10 inches thick. The subsoil is reddish-brown sandy clay loam. At a depth of about 28 inches is a dark brown layer that is cemented with iron and silica. Typically, fines content (passing No. 200 Sieve) is 35 to 60 percent, shrink-swell potential is low to moderate, have fair to good stability, slight to high compressibility, and poor to good resistance to piping.

Greenfield sandy loam, 2 to 8 percent slopes, eroded (GyD2) – Soils of the Greenfield series are on alluvial fans and terraces. These soils developed on weathered granite. In a typical profile, the surface layer is brown sandy loam about 26 inches thick. The subsoil is brown sandy loam and pale-brown loam and extends to a depth of about 60 inches. Runoff is slow to medium and the hazard of erosion is slight moderate. Typically, fines content (passing No. 200 Sieve) is 25 to 60 percent, shrink-swell potential is low, poor to good stability, slight to high compressibility, and poor to good resistance to piping.

Honcut sandy loam, 2 to 8 percent slopes (Hnc) – Soils of the Honcut series consist of well-drained soils on alluvial fans. In a typical profile, the surface layer is dark brown sandy loam about 22 inches thick. The underlying material is brown fine sandy loam or loamy sand and extends to a depth of greater than 60 inches. Runoff is slow to medium and the hazard of erosion is slight to moderate. Typically, fines content (passing No. 200 Sieve) is 25 to 35 percent, shrink-swell potential is low, poor to fair stability, slight to medium compressibility, and poor resistance to piping.

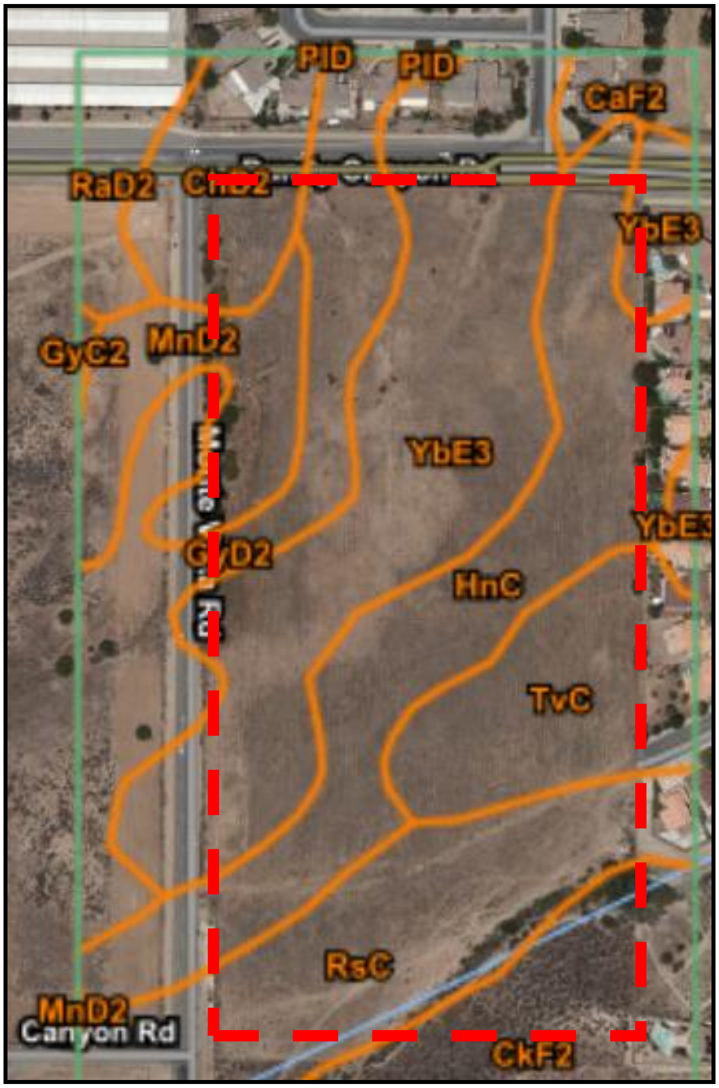
Tujunga loamy sand, channeled, 0 to 8 percent slopes (TvC) – Soils of the Tujunga series consist of excessively drained on alluvial fans and floodplains. These soils developed on weathered granite. In a typical profile, the surface layer is light gray loamy sand about 10 inches. Below this layer is light gray fine sand. Runoff is very slow and the hazard of erosion by wind is high. Typically, fines content (passing No. 200 Sieve) is 0 to 20 percent, shrink-swell potential is low, poor to fair stability, slight to medium compressibility, and poor resistance to piping.

Cieneba rocky sandy loam, 15 to 50 percent slopes, eroded (CkF2) – Soils of the Cieneba series consist of somewhat excessively drained soils on uplands. These soils formed in coarse-grained igneous rock. In a typical profile, the surface layer is brown sandy loam about 14 inches thick. Underlying this is light yellowish-brown gravelly coarse sand. At a depth of about 22 inches is slightly acid, weathered granodiorite. Runoff is medium, and the hazard of erosion is moderate. Typically, fines content (passing No. 200 Sieve) is 15 to 30 percent, shrink-swell potential is low, have fair stability, very slight to slight compressibility, and poor resistance to piping.

Riverwash (RsC) – Riverwash soils are on slopes of 0 to 8 percent in valley fills and on alluvial fans. These sandy, gravelly, or cobbly areas lie in the beds of the major streams and larger creeks.

Figure No. 9 is a portion of a NRCS soil survey map (NRCS, 2020) depicting the mapped agricultural soil types in the vicinity of the proposed basin.

Figure 9: NRCS Soil Survey Map (NRCS, 2020)



SUBSURFACE CONDITIONS

Field exploration consisted of drilling 22 exploratory borings ranging in depth from approximately 2.5 to 55 feet. The borings were excavated by means of both truck- and track-mounted rotary auger drill rigs. The approximate locations of the exploratory borings are shown on Figure Nos. A-25 and A-26. The drilling and sampling procedures are described in Appendix A. Laboratory test results are presented in Appendix B.

Based on geologic mapping and the subsurface exploration, the project site is underlain by young alluvial fan/axial channel deposits, very young wash deposits, and granitic bedrock. Artificial fill was encountered on portions of the site. As encountered within our exploratory borings, the earth materials are generally described below.

Artificial Fill: Artificial fill was encountered within borings B-02 (±3.5 feet), B-09 (±5 feet), B-14 (±5 feet), and B-17 (±3 feet). These materials consisted of silty sand (SM) and silty gravel with sand (GW-GM) generally in a loose condition. Artificial fill associated with previous structures and grading in other areas of the site should be anticipated. This could include remnants of the former structures, on-site septic systems including septic tanks, leachlines and/or seepage pits, and other debris. A large pile of oversized rock and debris is located near the center portion of the property. Existing riprap was encountered in our borings B-19, B-20, and B-22 at the outlet of the existing Bundy Canyon Channel – Lateral A.

Younger Alluvium/Axial Valley Deposits: Younger alluvial and axial valley deposits (Holocene-late Pleistocene age) are predominately present on the southerly portion of the site, and were encountered within borings B-01 through B-05, B-08, B-12, B-13, B-17 and B-18. These soils included mostly brown and light-brown fine- to coarse-grained silty sand (SM), silty clayey sand (SC-SM), fine- to medium-grained clayey sand (SC), and fine- to coarse-grained sand with silt (SP-SM). Typically, these soils were loose to medium dense with increasing density with depth.

Older Alluvium: Older alluvial deposits (presumed late Holocene-Pleistocene age) were encountered within borings B-06 through B-07, B-09 through B-11, B-14, B-15, and B-18 at depths ranging from near the ground surface to approximately 8.0 feet. These soils included brown and reddish-brown silty clayey sand (SC-SM), clayey sand (SC), silty sand (SM), sand with silt (SP-SM) and sandy clay (CL). These soils were typically loose to dense and well indurated, with increasing density with depth. Standard Penetration Testing (SPT) across the site indicated that penetration blow counts (N-values) within the older alluvial soils were notably higher than those observed within the younger soils.

Bedrock: Granitic bedrock was encountered within borings B-01, B-04 through B-07, B-11, B-13 through B-16 at depths ranging from approximately 3.0 to 38.5 feet below the existing ground surface. The bedrock was generally dense and ranged from highly to slightly weathered, with decreased weathering with depth. Drilling refusal was encountered above our target depth within boring B-05, at a depth of 26 feet. Target depths of the borings were achieved in the remaining borings drilled within the granitic bedrock.

Wash Deposits: Very young (late Holocene) wash deposits were encountered within borings B-03, B-16, B-19, and B-20 within the Bundy Canyon wash on the southeast portion of the site. These soils consisted of fine- to coarse-grained silty sand (SM), sand with silt (SP-SM). These deposits are generally loose to medium dense and not suitable for support of foundations or embankments loads in their existing condition.

Groundwater was encountered within five exploratory borings (B-01, B-12, B-15, B-16 and B-21) at depths of approximately 19 to 39 feet below the existing ground surface. Based on the depth to groundwater where encountered, groundwater is not generally expected to be an influence in the design or construction of the proposed project, with the exception of the area of the unimproved Bundy Canyon wash in and along the southeast and south areas of the proposed basin. Groundwater was encountered in boring B-16 at a depth of approximately 19 feet, about three feet below the bottom of the proposed RCB inlet channel. Groundwater levels beneath and adjacent to the Bundy Canyon wash will be subject to significant seasonal variation. Groundwater could be an influence on the construction if conducted during or after periods of significant precipitation.

For purposes of our analysis, we assumed a historic high groundwater elevation of 1,405 feet msl. This is the approximate bottom elevation of the Bundy Canyon wash along the south end of the proposed basin. Very moist to saturated near-surface conditions are expected in this area following prolonged periods of precipitation.

We understand that a water well was formerly present within the southern portion of the site (LOR, 2003). We recommend that the District review water well records to determine if the well was properly abandoned per Riverside County Department of Environmental Health guidelines.

The moisture content of the soil at the time of our investigation ranged from approximately 2 to 14 percent within the planned excavation depths. The dry unit weight of the alluvial soil tested ranged from 104 to 135 pcf. The apparent dry unit weight of granitic bedrock samples ranged from 108 to 136 pcf. The dry unit weight and moisture content values of samples obtained from the exploratory borings are shown on the borings logs in Appendix A.

Sand equivalent values of soil samples tested within the depth of excavation ranged from 17 to 56. Sand equivalent values of representative samples are listed below and in Appendix B.

Table 4: Summary of Sand Equivalent Test Results

Boring No.	Depth (ft.)	SE
B-01	3.2 – 6.0	20
B-02	7.0 – 17.0	56
B-03	3.0 – 14.5	39
B-07	6.5 – 10.0	24
B-11	12.0 – 16.0	17
B-13	6.5 – 12.0	48
B-16	11.0 – 17.0	24
B-17	6.5 – 12.5	31
B-18	2.5 – 5.0	40

A soil corrosivity evaluation for this project was conducted by HDR Engineering, Inc. The soil corrosivity evaluation report prepared by HDR is presented in Appendix I.

Detailed descriptions of the subsurface soil conditions encountered are presented on the boring logs in Appendix A.

Seismic Refraction Rippability Survey: A seismic refraction rippability survey was conducted at selected locations within the proposed detention basin site. A copy of this report is presented as Appendix H. The seismic refraction survey report, prepared by Terra Geosciences, indicates that in general the site is characterized by three major subsurface layers. These include:

> Velocity Layer V1: The surficial layer (V1) yielded a seismic velocity range of 823 to 1,382 fps, which is presumed to be comprised of variable younger (Holocene age) alluvial deposits and/or localized fill.
>
> No excavating difficulties are expected to be encountered within the uppermost V-1 layer.
>
> Velocity Layer V2: The second layer (V2) has a seismic velocity range of 1,594 to 2,333 fps, which is believed to be comprised of older alluvial deposits.
>
> No excavating difficulties are expected to be encountered within the second V2 layer.

Velocity Layer V3: The third layer (V2) indicates the presence of highly- to moderately-weathered granitic bedrock, having a seismic velocity of 4,334 to 8,007 fps. In addition to granitic rock, seismic velocities typically ranging from 5,000 to 6,500± fps are also representative for saturated sediments, indicating the possibility of a saturated groundwater table.

The third layer is believed to consist of highly- to moderately weathered granitic bedrock. Moderate to hard excavation difficulties within this velocity layer should be anticipated during grading.

The entire Terra Geosciences report is appended and should be read for more details concerning rippability of the site.

Infiltration: Seven borings within the footprint of the proposed detention basin were converted to percolation test wells. The testing procedures and test results are described in Appendix C. Table 5 below provides a summary of the test data with calculated infiltration rates (I_c). Note that the values shown do not include safety factors.

Table 5: Percolation Test Data and Infiltration Rates

Percolation Test No.	Depth Below Existing Ground Surface (ft)	Infiltration Rate (I_c) (in/hr)
P-1 (B-10)	32.3	8.3
P-2 (B-15)	30.3	1.9
P-3 (B-14)	36.5	13.4
P-4 (B-06)	33p.5	4.1
P-5 (B-09)	28.5	9.5
P-6 (B-08)	27.5	5.9
P-7 (B-04)	28.5	11.6

For comparison, flexible wall permeability testing of in-situ drive samples obtained from below the proposed basin bottom was performed in accordance with ASTM D5084. Dimensionally, the coefficient of permeability is equivalent to the one-dimensional infiltration rate under atmospheric conditions. Differences can result from sample disturbance, degree of saturation and other factors. Flexible wall permeability test results are summarized in Appendix B and in Table 6 below.

Table 6: **Flexible Wall Permeameter Test Data**

Boring No.	Sample Depth (ft.)	Coefficient of Permeability (in/hr)
B-04	23.5	0.12
B-06	28.0	0.02
B-10	28.5	0.00
B-14	36.0	0.34
B-21	25.5	0.00

As shown, laboratory permeability and borehole test data can vary by several orders of magnitude. We recommend that design infiltration rates for larger basins, such as the Wildomar Basin, be evaluated using the results of in-situ double-ring infiltration tests (ASTM D3385) performed within the actual basin bottom soil when practical.

CONCLUSIONS AND RECOMMENDATIONS

On the basis of our field and laboratory exploration and testing, the proposed detention basin and storm drain construction is feasible from a geotechnical standpoint. Our conclusions and recommendations are presented in the following sections.

Overall Feasibility: Based on our investigation, the proposed Wildomar Basin project is feasible from geotechnical and geologic standpoints, provided that our recommendations are properly implemented. Predominately granular, alluvial soils overlying granitic bedrock are present within the basin site and along the outlet storm drain alignment to the south. The alluvial soils are generally loose to dense and may be susceptible to caving.

A soil corrosion evaluation report has been prepared by HDR Engineering, Inc. and is presented in Appendix I. General recommendations for mitigation are included.

The most significant condition encountered is the potential for near-surface groundwater within the existing unimproved Bundy Canyon wash along the south perimeter of the basin site. Groundwater levels in this area are expected to vary significantly depending on seasonal precipitation and runoff from the Bundy Canyon Channel – Lateral A to the east. Dewatering may be necessary for excavation within and near the existing wash. Groundwater was not encountered within the planned excavation limits in other areas of the project. Except for the Bundy Canyon wash area, historical groundwater levels are expected to be below the planned excavation limits.

A related groundwater issue is the potential for liquefaction and seismic settlement in saturated alluvial soil below the existing unimproved Bundy Canyon wash. Our analysis indicates the soil encountered in boring B-21 near the proposed stilling basin is potentially liquefiable to a depth of approximately 28 feet, about eight feet below the bottom stilling basin slab. The soil encountered in boring B-3 at EVMWD sewer Station 31+50 is potentially liquefiable to a depth of approximately 20 feet, about 15 feet below the sewer invert elevation.

Expected Soil Types to be Encountered: The subsurface materials that will be encountered during construction of this project primarily consist of granular alluvial deposits overlying granitic bedrock. The alluvial soils generally consist of loose to dense silty sand (SM), clayey sand (SC), silty clayey sand (SC-SM), and sand with silt (SP-SM). The underlying granitic bedrock is generally dense to very dense and slightly to highly weathered.

Excavation and Rippability: Alluvial deposits at the basin site and along the outlet storm drain alignment are expected to be readily excavated with conventional excavation equipment. Caving should be anticipated.

Based on the results of the seismic refraction survey and the conditions encountered in our borings, we expect that most granitic bedrock within the basin site and along the outlet storm drain can be readily excavated with conventional excavation equipment. Difficult excavation may be periodically encountered in less-weathered bedrock. Breaking and/or blasting to obtain planned invert depths may also be necessary, depending on the excavation equipment used.

Basin Slope Stability: The proposed north basin slope was analyzed for the following conditions, with the resulting factors of safety (FS) shown.

Static stability, basin full	FS = 1.7
Static stability, basin empty	FS = 1.7
Pseudo-static stability, basin full	FS = 1.1
Pseudo-static stability, basin empty	FS = 1.1
Rapid drawdown stability	FS = 0.7

Minimum factors of safety of 1.5 and 1.1 are considered acceptable for static and seismic conditions, respectively, based on current standards in Riverside County. The slopes are expected to perform satisfactorily with routine maintenance.

The rapid drawdown factor of safety of 0.7 indicates that slope failure within the alluvial soil is likely if the basin water level drops faster than the soil pore water pressure can dissipate.

Slope stability analysis procedures and results are discussed in Appendix F.

Seepage Analysis: The primary focus of the seepage analysis was to evaluate seepage from the existing Bundy Canyon Channel downslope toward the proposed transition structure and stilling basin. We considered the area of analysis to consist of two primary soil types; the surficial alluvial soil and underlying granitic bedrock.

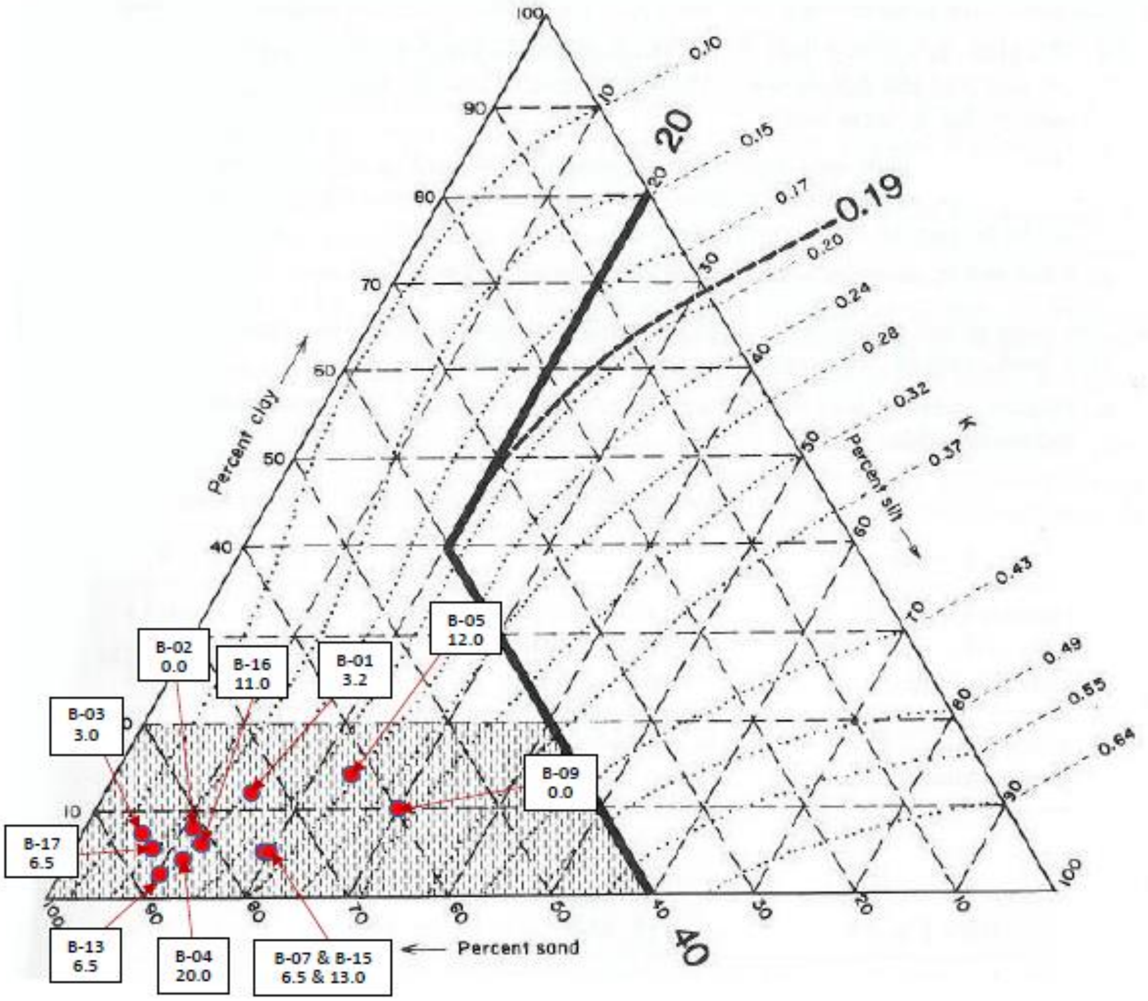
The seepage analysis was performed using the Slide 6.0 computer program (RocScience, 2013). Slide 6.0 uses two-dimensional finite element analysis to evaluate saturated / unsaturated, steady state or transient flow conditions. For this project, we used a steady state analysis.

A schematic cross-section of the seepage model is shown on Figure G-2. As shown, the phreatic surface (wetted front) of the seepage zone generally coincides with the underlying bedrock and remains below the improved Bundy Canyon Channel and the proposed transition structure and stilling basin.

Seepage analysis results are presented in Appendix G.

Soil Erodibility Factor (K): The K factor can be determined using the nomograph method, which requires that a particle size analysis be done to determine the percentages of sand, very fine sand, silt and clay.

On the basis of classification testing, the value for K is estimated to be between 0.10 and 0.32 as indicated on the following chart.



Erickson triangular nomograph used to estimate soil erodibility (K) factor. USDA nomograph from Erickson 1977 as referenced by Goldman et al., 1986.

Unusual Soil Conditions or Groundwater Conditions: No unusual or unanticipated soil conditions were encountered.

Soil Compressibility, Preliminary Soil Strength: The site is generally underlain by alluvial deposits overlying granitic bedrock. Based on sampler blow count data, the alluvial soil generally ranges from loose to dense. Consolidation test results indicate the alluvial soil is moderately to severely compressible. The underlying granitic bedrock is generally incompressible within the range of loads considered. Cohesive strengths of the alluvial soil and bedrock are negligible.

Water Soluble Sulfates: Testing indicates negligible concentrations of water-soluble sulfates. This is addressed in the report of the Soil Corrosivity Evaluation, which is appended.

Temporary Excavations and Shoring: All excavation should be configured and shored in accordance with the requirements of Cal/OSHA. The soil should be classified as Type C. Cohesionless soil will be encountered at depths that will likely be subject to caving when exposed in unshored vertical excavation sidewalls. If a trench shield is used, careful monitoring will be required.

The contractor should have a "competent person" on-site for the purpose of assuring safety within and about all construction excavations. Unshored excavations should have a maximum slope of 1.5:1 (H:V) and should not exceed twenty feet in height. Shoring, shields, or other protective systems should be used in accordance with all specifications, recommendations, and limitations provided by the manufacturer. Shoring should be designed using an at-rest earth pressure of 60 pounds per cubic foot. A registered professional engineer should design shoring or benching for excavations deeper than twenty feet.

Excavation for the proposed stilling basin and transition structure will likely require shoring due to the limited right-of-way. Potential temporary excavation scenarios were evaluated for the south side of the stilling basin that consisted of a) near-vertical excavation within the right-of-way and b) a 1.5:1 (h:v) temporary slope. The results of the analysis are shown on Figures C-7 and C-8 and indicate factors of safety less than 1.0 for both scenarios. Similar conditions are present on the north side of the stilling basin.

Protection of Existing Facilities: Where existing utilities are exposed during excavation, we recommend they be assessed for sensitivity to post-construction settlement. Typically, this is a concern for rigid pipelines, such as water and sewer lines.

Allowable Bearing Pressure: Structure foundations should be designed with a maximum allowable bearing pressure of 2,500 pounds per square foot (psf). This is suitable for native soil with an in-place relative compaction of at least 85 percent and for fill soil compacted to at least 90 percent relative compaction. Footing depths should be at least 12 inches below the lowest adjacent final grade. Settlement of foundations designed using the recommended allowable bearing pressures is expected to be less than one inch.

This firm should review the locations and design loads of any proposed permanent foundation, when available, to verify that the above recommendations remain appropriate.

Earth Pressure: Cantilever walls or shoring supporting compacted fill soils should be designed using an active equivalent fluid pressure of 45 pounds per cubic foot (pcf) for level backfill. Cantilever shoring supporting native soil should be designed with a minimum active equivalent earth pressure of 40 pcf. Braced shoring or walls supporting native soil should be designed for an at-rest earth pressure of 60 pcf, with the resultant applied at mid-height. The above values area based on:

- Rankine active pressure coefficient of 0.33
- At-rest pressure coefficient of 0.50
- Total native soil density (wet) of 120 pcf
- Total compacted backfill density (wet) of 135 pcf

Any applicable construction surcharge or seismic loads should be added to the above pressures. The effects of seismic forces may be characterized as an equivalent fluid pressure of 15 pounds per cubic foot.

A maximum passive equivalent fluid pressure of 240 pcf should be used for design of foundations on compacted fill or competent native soil. This value includes a safety factor of 1.5.

The equivalent fluid pressure recommendations are for drained conditions with level backfill. Structures subject to undrained conditions should be constructed with weepholes to allow for drainage of the adjacent soil. Preliminarily, weepholes should be spaced a maximum distance of 10 feet apart and should be at least 2.5 inches in diameter. Alternatively, imported granular backfill with an outlet drain could be used to provide for proper drainage behind structures. This firm should be contacted for specific structure drainage recommendations, if required.

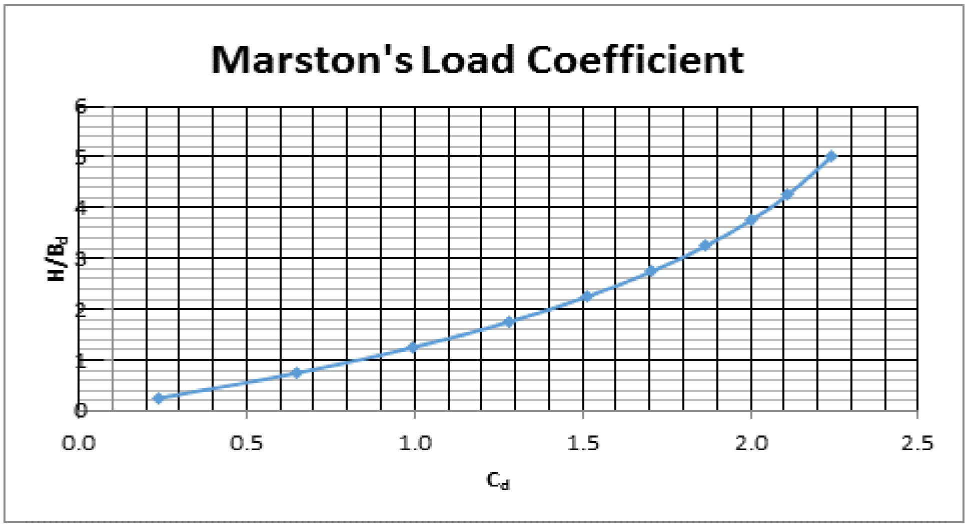
Coefficient of Friction: A coefficient of friction of 0.45 between soil and concrete may be used with dead load forces only.

Unit Weight: Our recommendations are based on a total unit weight of 135 pounds per cubic foot for compacted backfill. Our testing indicates the product of the Rankine ratio and sidewall friction coefficient ($K\mu'$) to be approximately 0.19. Loads under trench conditions are to be estimated using Marston’s Formula:

$$W_c = C_d w B_d^2$$

Where: W_c = Load on Pipe
w = Unit Weight of Backfill (135 pcf)
B_d = Trench Width (ft.)

H = Height of Fill above Pipe (H<20ft.)
C_d = Marston’s Load Coefficient as shown in the following chart:



Corrosivity: A corrosivity evaluation for this project was conducted by HDR Engineering, Inc. The report indicates that the soils are classified as severely corrosive to ferrous metals and aggressive to copper. The entire report is appended and includes test data.

Earthwork/Backfilling: All earthwork and backfilling should be performed in accordance with District requirements and the current edition of the Standard Specifications for Public Works Construction (Greenbook).

1. **Clearing and Grubbing:** All structure, slab and pavement areas and all surfaces to receive compacted fill should be cleared of existing loose soil, vegetation, tree roots, artificial fill, debris, and other unsuitable materials.

2. **General Site Grading:** The on-site materials are generally suitable for use as compacted fill, but may require screening to remove rocks larger than 12 inches in diameter. Rocks larger than 12 inches in maximum dimension may be generated during basin and pipeline excavation and should not be placed in fill, unless done so per an approved rock disposal plan. Soils should be brought to near optimum moisture content, and compacted in 6 to 8-inch thick lifts to a minimum of 90 percent relative compaction, based on either California Test Method (CTM) 216 or ASTM Standard D1557. Soils should be mechanically compacted.

   Within the existing wash along the south perimeter of the project site, all alluvial soil should be removed to granitic bedrock. This generally encompasses the area of the proposed inlet channel (including the transition structure and stilling basin), the south maintenance road and the EVMWD relocated sewer alignment. Excavation to bedrock will necessitate removals on the order of 30 feet below existing surface grades. Following removal, soil should be moisturized and compacted as recommended above. Fills deeper than 10 feet below finish grade should be compacted to 95 percent relative compaction.

Foundations for retaining walls and other structures should be supported on recompacted native soil or processed granitic bedrock. Granitic bedrock was encountered at the elevation of the proposed retaining wall footings for the impact basin in the northwest corner or the basin. Prior to concrete placement, the exposed surface at this location should be scarified, moisture conditioned and recompacted to a depth of at least 12 inches.

Existing alluvial soil should be removed to bedrock below the proposed impact basin for the relocated 54-inch RCP storm drain west of Valley Vista Circle and below the rip-rap chute in the southeast basin corner. The removed material should be moisture-conditioned and replaced as compacted fill.

Chemical or compaction grouting may be feasible alternatives to removing and recompacting alluvial soil. A specialty ground improvement contractor should be contacted for more information,

3. **Pipe Bedding**: Per District Standard Drawing Number M815, RCP should be supported by at least four (4) inches of filter material consisting of 1-inch x No. 4 coarse aggregate per Section 90-1.02C(4)(b) of the Caltrans Standard Specifications.

   Pipe bedding material, as defined by Greenbook Section 306-6, is the material supporting, surrounding the pipe, and extending to one (1) foot above the top of the pipe. Per District Standard Drawing Number M815, pipe bedding should consist of controlled low strength material (CLSM) that conforms to Greenbook Section 201-6. To protect the pipe, we recommend that CLSM be placed to at least 12 inches above the top of pipe.

4. **Trench Backfill:** Greenbook Section 306-12 defines backfill as starting at the top of the bedding zone. Soil and bedrock materials excavated for the detention basin and storm drain construction are expected to be suitable for use as trench backfill. On-site excavated material should be screened to remove all material larger than 12 inches prior to placement as backfill. All backfill should be non-expansive, with an expansion index of less than 20. Trench backfill should be compacted at near optimum moisture content by mechanical means as necessary for the achievement of satisfactory compaction. Controlled density fill (CDF) or sand-cement slurry may be used in lieu of mechanically compacted native soils as backfill material. Unless otherwise specified by the drawings, specifications or encroachment permits, the minimum acceptable relative

compaction should be 90 percent, based on CTM 216 or ASTM D1557. The upper 12 inches of backfill within pavement areas which should be compacted to a minimum of 95 percent relative compaction.

5. **Structure Backfill:** Structure backfill should conform with Greenbook Section 217-3. Soil and bedrock materials excavated for the detention basin and storm drain construction are expected to be suitable for use as structure backfill, but will require screening to remove material larger than four (4) inches in maximum dimension. Compaction of structure backfill should be as recommended above for trench backfill.

6. **Import Soil:** If required, import soil should be granular, non-plastic, and non-expansive. Suggested criteria for import soil is presented in the following table.

Import Soil Suggested Criteria	
Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	85 – 100
Percent Passing No. 200 Sieve	15 – 40
Plasticity Index	Less than 15
Expansion Index (ASTM D4829)	20 or less (very low)
Organic content	Less than 1 percent by weight
Sulfates	< 1,000 ppm
Min. Resistivity	> 10,000 ohm-cm

To retard particle migration, imported pipe embedment material should meet the following criteria:

Imported Embedment Material Criteria
D_{15} >0.015 mm, and D_{50} <1.25 mm

where D_{15} and D_{50} represent bedding material particle sizes corresponding to 15 and 50 percent passing by weight, respectively. Washed concrete sand (ASTM C33) satisfies these criteria.

7. **Construction Observation and Testing:** All earthwork should be monitored by a representative of the District, and compaction testing performed, to verify conformance with the recommendations in this report and other project requirements. Maximum dry density-optimum moisture content testing should be performed in accordance with either the ASTM Standard D1557 test method or California Test Method No. 216. Field density testing should be performed in accordance with either the ASTM Standard D1556 (sand cone) or ASTM 6938 (nuclear) test method. Recommended maximum compaction testing intervals are 250 feet for each 2-foot vertical lift or as otherwise deemed necessary by the District representative. Some backfill and compaction methodologies may dictate more frequent test intervals.

   Should testing indicate insufficient compaction, additional testing may be necessary in order to define the area requiring recompaction. Without further testing, it should be assumed that the area between a failing test and a passing test is not properly compacted. As a guideline for evaluation, one test may be taken at a distance from the failing test equal to 20 percent of the distance to the next passing test. If the test reveals satisfactory compaction, the area between the failing test and the passing test should be recompacted. If the test reveals inadequate compaction, the process should be repeated in order to delineate the unsatisfactory area. After recompaction of “failing” areas, retesting should be conducted in order to confirm satisfactory compaction. At least one retest is required for each failing test, even if failing tests are for the purpose of delineating the area requiring additional work.

LIMITATIONS

The findings and recommendations presented in this report are based upon an interpolation of the soil conditions between boring locations. Should conditions be encountered during construction that appears to be different than those indicated by this report, this office should be notified.

This report was prepared for Riverside County Flood Control and Water Conservation District for their use in the design of the proposed Wildomar MDP Lateral C Stage 3 project. This report may only be used by Riverside County Flood Control and Water Conservation District for this purpose. The use of this report by parties or for other purposes is not authorized without written permission by Inland Foundation Engineering, Inc. Inland Foundation Engineering, Inc. will not be liable for any projects connected with the unauthorized use of this report.

The recommendations of this report are considered to be preliminary. The final design parameters may only be determined or confirmed at the completion of construction on the basis of observations made during the construction operation. To this extent, this report is not considered to be complete until the completion of both the design process and the site preparation. The information in this report represents professional opinions that have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No other warranty, either expressed or implied, is made as to the professional advice included in this report.

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Riverside County Flood Control District, 1990, Photo Numbers 14-13 and 14-14, Scale 1”=1,600’, dated January 22, 1990.

Riverside County Flood Control District, 1995, Photo Numbers 14-12 and 14-13, Scale 1”=1,600’, dated January 28, 1995.

Riverside County Flood Control District, 2000, Photo Numbers 14-13 and 14-14, Scale 1”=1,600’, dated March 18, 2000.

Riverside County Flood Control District, 2005, Photo Numbers 14-13 and 14-14, Scale 1”=1,600’, dated April 13, 2005.

Riverside County Flood Control District, 2010, Photo Numbers 14-12 and 14-13, Scale 1”=1,600’, dated April 2, 2010.

Terrain Navigator, Wildomar NE, CA, 2018, USGS Reference Code 33117-E3-TF-012.

APPENDIX A –
Field Exploration

APPENDIX A

FIELD EXPLORATION

The field exploration consisted of 22 exploratory borings with both truck- and track-mounted drill rig at the approximate locations shown on Figures A-25 and A-26. Logs of the materials encountered were made during drilling by a staff geologist and are presented on Figures A-3 through A-24.

Representative soil samples were obtained within the borings by driving a thin-walled steel penetration sampler with successive 30-inch drops of a 140-pound hammer. The numbers of blows required to achieve each six inches of penetration were recorded on the boring logs. Two different samplers were used; a Standard Penetration Test (SPT) sampler and a modified California sampler with brass sample rings. Representative bulk soil samples were also obtained from the auger cuttings. Samples were placed in moisture sealed containers and transported to our laboratory for further testing and evaluation. Laboratory tests results are discussed and included in Appendix B.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

PRIMARY DIVISIONS			GROUP SYMBOLS		SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIALS IS LARGER THAN #200 SIEVE SIZE	GRAVELS MORE THAN HALF OF COARSE FRACTION IS LARGER THAN #4 SIEVE	CLEAN GRAVELS (LESS THAN) 5% FINES	GW		WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
			GP		POORLY GRADED GRAVELS OR GRAVEL-SAND MIXTURES, LITTLE OR NO FINES
		GRAVEL WITH FINES	GM		SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES
			GC		CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN #4 SIEVE	CLEAN SANDS (LESS THAN) 5% FINES	SW		WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP		POORLY GRADED SANDS OR GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM		SILTY SANDS, SAND-SILT MIXTURES
			SC		CLAYEY SANDS, SAND-CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF OF MATERIALS IS SMALLER THAN #200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT IS LESS THAN 50	ML		INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS
			CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL		ORGANIC SILTS AND ORGANIC SILT-CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS	LIQUID LIMIT IS GREATER THAN 50	MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS, ELASTIC SILTS
			CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH		ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
	HIGHLY ORGANIC SOILS		PT		PEAT, MUCK AND OTHER HIGHLY ORGANIC SOILS
TYPICAL FORMATIONAL MATERIALS	SANDSTONES		SS		
	SILTSTONES		SH		
	CLAYSTONES		CS		
	LIMESTONES		LS		
	SHALE		SL		

CONSISTENCY CRITERIA BASES ON FIELD TESTS

RELATIVE DENSITY – COARSE – GRAIN SOIL

RELATIVE DENSITY	SPT * (# BLOWS/FT)	RELATIVE DENSITY (%)
VERY LOOSE	<4	0-15
LOOSE	4-10	15-35
MEDIUM DENSE	10-30	35-65
DENSE	30-50	65-85
VERY DENSE	>50	85-100

CONSISTENCY – FINE-GRAIN SOIL

		TORVANE	POCKET ** PENETROMETER
CONSISTENCY	SPT* (# BLOWS/FT)	UNDRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2-4	0.13-0.25	0.25-0.5
Medium Stiff	4-8	0.25-0.5	0.5-1.0
Stiff	8-15	0.5-1.0	1.0-2.0
Very Stiff	15-30	1.0-2.0	2.0-4.0
Hard	>30	>2.0	>4.0

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM -1586 STANDARD PENETRATION TEST)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp but no visible water
WET	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbled or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

EXPLANATION OF LOGS

A-2

LOG OF BORING B-01

DRILLING RIG CME-75
DRILLING METHOD Rotary Auger
LOGGED BY FWC
GROUND ELEVATION +/- 1393 ft

DATE DRILLED 5/26/20

Outlet Storm Drain

Station 20+75

HAMMER TYPE Auto-Trip
HAMMER WEIGHT 140-lb.
HAMMER DROP 30-inches
BORING DIAMETER 8-inches

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0									
1			SILTY SAND, fine- to coarse, olive-brown, moist, medium dense to very dense.			AU			
2	SM								
3						AU			
4			GRANITE, moderately weathered, olive (5Y 4/3).				32 42	13	117
5	SM								
6						AU			
7			GRANITE, moderately weathered, light olive-brown (2.5Y 5/4).						
8	SM						30 55	6	124
9						AU			
10			GRANITE, highly weathered, olive-brown (2.5Y 4/4).						
11									
12							26 55	10	130
13									
14	SM								
15									
16							50	7	123
17									
18									
19			GRANITE, highly weathered, olive (5Y 5/3).						
20							50	5	121
21									
22									
23	SM								
24							50/5"	6	114
25									
26			∇						
27							50	8	116
			End of boring at 27.5 feet. Groundwater encountered at 26 feet. Mottling encountered at 9 feet. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT RCFCD
PROJECT NAME Wildomar Basin
PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA
PROJECT NUMBER R206-025

FIGURE NO. A-3

LOG OF BORING B-02

DRILLING RIG **CME-75**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1396 ft**

DATE DRILLED **5/26/20**

Outlet Storm Drain

Station 24+60

HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0–3	SM	▩	SILTY SAND,, fine- to medium, brown, moist, loose.	▮		AU AU			
4–7	SM	░	SILTY SAND, with trace clay, fine- to medium, very dark grayish-brown (10YR 3/2), moist, loose.	▮	☒	SS	5 4	5	109
7–17	SM	░	SILTY SAND, fine- to medium, dark olive-brown (2.5Y 3/3), moist, medium dense, with thin interbeds of sand.	▮		AU			
8				▮	☒	SS	7 9	7	119
12				▮	☒	SS	9 12	5	117
16				▮	☒	SS	10 10	8	110
17–25	SM	░	SILTY SAND, fine- to very coarse, brown (7.5YR 4/4), moist, medium dense.	▮		AU			
20				▮	☒	SS	7 10	9	119
24				▮	☒	SS	11 13	4	117
26–28	SC	▨	CLAYEY SAND, fine- to coarse, dark olive-brown (2.5Y 3/3), moist, dense.						
28					☒	SS	8 10	7	125
			End of boring at 28.5 feet. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-4

LOG OF BORING B-03

DRILLING RIG CME-75
DRILLING METHOD Rotary Auger
LOGGED BY FWC
GROUND ELEVATION +/- 1403 ft

DATE DRILLED 5/26/20

Maint. Road, SW Basin Corner
EVMWD Sewer Station 31+50

HAMMER TYPE Auto-Trip
HAMMER WEIGHT 140-lb.
HAMMER DROP 30-inches
BORING DIAMETER 8-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0	SM		**YOUNGER ALLUVIUM** SILTY SAND, fine- to coarse, brown, slightly moist to moist, loose.						
3	SP-SM		SAND with SILT, with trace gravel, fine- to coarse, very dark grayish-brown (2.5Y 3/2), moist, loose to medium dense, with thin interbeds of silty sand.	▮		AU			
4				▮	☒	SS	6 7	6	114
8				▮	☒	SS	2 6	3	123
12				▮	☒	SS	11 10	7	123
15	SM		SILTY SAND, very fine- to fine, brown (10YR 4/3), moist, medium dense.						
16	SM		SILTY SAND, fine- to coarse, brown (10YR 4/3), moist, medium dense, with thin interbeds of sand.		☒	SS	6 10	16	112
20					☒	SS	14 14	4	125
			End of boring at 20.5 feet. No groundwater encountered. Backfilled with native soils.						

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CLIENT RCFCD
PROJECT NAME Wildomar Basin
PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA
PROJECT NUMBER R206-025

FIGURE NO.
A-5

LOG OF BORING B-04

DRILLING RIG **CME-75**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1404 ft**

DATE DRILLED **5/21/20**

SW Basin Corner

HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0									
1	SM		**YOUNGER ALLUVIUM** SILTY SAND, fine- to coarse, dark grayish-brown (10YR 4/2), dry to moist, loose.			AU			
3	SM		SILTY SAND, fine- to coarse-grained, dark grayish-brown (10YR 4/2), slightly moist, loose.		☒	SS AU	5 5	3	118
6	SM		SILTY SAND, with trace clay, fine- to coarse, dark olive-brown (2.5Y 3/3), moist, loose.			AU			
7					☒	SS	3 7	13	117
9	SP-SM		SAND with SILT, with trace gravel, fine- to very coarse, dark olive-brown (2.5YR 3/3), moist, very loose to medium dense, with thin interbeds of silty sand.			AU			
11					⧗	SPT	1 2	3	
15					☒	SS	9 15	5	117
19					☒	SS	11 17	4	124
20	SM		SILTY SAND with GRAVEL, fine- to very coarse, dark olive-brown (2.5YR 3/3), slightly moist, dense, with thin interbeds of sand with silt.			AU			
24					☒	SS	21 27	4	126
28	SC-SM		SILTY, CLAYEY SAND, very fine- to coarse, dark olive-brown (2.5YR 3/3), moist, dense.						
29	SM		GRANITE, highly weathered, dark olive-brown 2.5YR 3/3).		☒	SS	20 35	5	128

End of boring at 29.5 feet. No groundwater encountered. Backfilled with native soils.

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CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-6

LOG OF BORING B-05

DRILLING RIG CME-75
DRILLING METHOD Rotary Auger
LOGGED BY FWC
GROUND ELEVATION +/- 1407 ft

DATE DRILLED 5/23/20

West Side of Basin

HAMMER TYPE Auto-Trip
HAMMER WEIGHT 140-lb.
HAMMER DROP 30-inches
BORING DIAMETER 8-inches

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0 1 2	SC	▨	**YOUNGER ALLUVIUM** CLAYEY SAND, very fine- to fine, red-brown, moist, loose.	▮		AU			
3 4 5	SC-SM	▨	**SILTY, CLAYEY SAND,** very fine- to fine-, dark olive-brown (2.5YR 3/3), slightly moist, loose.		☒	SS	3 3	4	104
6 7 8 9	SC-SM	▨	**OLDER ALLUVIUM** SILTY, CLAYEY SAND, with trace gravel, fine- to coarse-grained, dark brown (10YR 3/5), slightly moist, dense, moderately cemented.	▮	☒	AU SS	33 50	6	116
10 11 12	SM	▨	SILTY SAND, with trace clay, fine- to medium, dark brown (10YR 3/5), slightly moist, dense, moderately cemented.	▮	☒	AU SS	26 50	11	121
13 14 15 16	SM	▨	SILTY SAND, with trace clay, fine- to medium, dark brown (10YR 3/5), slightly moist, dense, weakly cemented.	▮		AU			
17	SM	▨	SILTY SAND, fine- to very coarse, dark brown (10YR 3/5), slightly moist, dense.						
18 19 20 21 22	SC-SM	▨	SILTY SAND, dark olive-brown (2.5YR 3/3), slightly moist, dense, moderately cemented.	▮	☒	AU SS	30 40	4	129
23 24 25 26	SM	▨	GRANITE, moderately to slightly weathered, dark olive (2.5Y 3/3). - very hard drilling -		⧗ ⧗	SPT SPT	23 50 35 25	5 5	123
			End of boring at 26 feet. Auger refusal. No groundwater encountered. Backfilled with native soils.						

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CLIENT RCFCD
PROJECT NAME Wildomar Basin
PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA
PROJECT NUMBER R206-025

FIGURE NO.
A-7

LOG OF BORING B-06

DRILLING RIG **CME-75**
DATE DRILLED **5/22/20**
HAMMER TYPE **Auto-Trip**
DRILLING METHOD **Rotary Auger**
HAMMER WEIGHT **140-lb.**
LOGGED BY **FWC**
HAMMER DROP **30-inches**
GROUND ELEVATION **+/- 1413 ft**
West Side of Basin
BORING DIAMETER **8-inches**

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0									
1			CLAYEY SAND, very fine- to fine, dark yellowish-brown (10YR 3/4, moist, loose to medium dense, moderately cemented.						
2									
3	SC								
4									
5					⧗	SPT	3 7	9	
6	SM		SILTY SAND, fine- to coarse, dark yellowish-brown (10YR 3/4), moist, loose.						
7									
8	SC-SM		**OLDER ALLUVIUM** SILTY, CLAYEY SAND, very fine- to fine, dark brown (10YR 3/3), moist, medium dense to dense, moderately cemented.						
9					⧗	SPT	15 16	6	
10									
11			CLAYEY SAND, fine- to coarse, dark brown (10YR 3/3), moist, dense, moderately cemented.						
12									
13			- very hard drilling -		☒	SS	20 50	5	131
14									
15	SC								
16			- cobbly -						
17					☒	SS	30 40	10	130
18									
19			CLAYEY SAND, very fine- to fine, dark brown (10 YR 3/3), moist, dense.						
20	SC								
21					☒	SS	25 55	14	124
22			SILTY SAND, fine- to medium, dark brown (10YR 3/3), moist, dense.						
23									
24									
25					☒	SS	30 55	9	130
26									
27	SM								
28									
29					☒	SS	33 50	14	116
30									
31									
32									
33			GRANITE, moderately weathered, dark yellowish-brown (10YR 3/4).						
34	SM				☒	SS	30 50	4	126
			End of boring at 34 feet. No groundwater encountered. Backfilled with native soils.						

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CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-8

LOG OF BORING B-07

DRILLING RIG **CME-75**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1422 ft**

DATE DRILLED **5/20/20**

Northwest Basin Corner

HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0 1 2 3	SC		**OLDER ALLUVIUM** CLAYEY SAND, very fine- to fine, red-brown, moist, loose.			AU AU			
4 5 6	SC-SM		SILTY, CLAYEY SAND, very fine- to medium, dark brown (10YR 3/3), slightly moist, dense, moderately cemented.		☒	SS	20 25	7	111
7 8 9 10	SC-SM		SILTY, CLAYEY SAND, fine- to coarse-grained, dark brown (10YR 3/3), moist, dense, weakly to moderately cemented.		☒	AU SS	26 26	8	112
11 12 13 14	SC		CLAYEY SAND, very fine- to fine, dark brown (10YR 3/3), slightly moist, dense.		☒ ☒	SS AU SS	32 40 25	6 5	119 127
15 16 17 18	SM		SILTY SAND, with trace clay, fine- to medium, dark olive-brown (2.5Y 3/3), slightly moist, dense, weakly cemented.		☒	AU SS	25 25 30	5	122
19 20 21 22 23 24 25 26 27	SM		SILTY SAND, with trace clay and gravel, fine- to medium, dark olive-brown (2.5Y 3/3), moist, dense.		⧗ ⧗	AU SPT SPT	11 12 9	7 7	
28 29 30 31 32 33	SM		SILTY SAND, fine- to coarse, dark olive-brown (2.5Y 3/3), slightly moist, dense.		⧗	AU SPT AU	15 18 17	3	
34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55	SC-SM		GRANITE, highly weathered, dark olive-brown (2.5Y 3/3).		⧗ ⧗ ⧗ ⧗ ⧗	SPT SPT SS SPT SPT SPT	27 50 50/5" 50/3" 50/5" 50 50/5"	9 3 4 5 5 6	
			End of boring at 55.42 feet. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT	RCFCD
PROJECT NAME	Wildomar Basin
PROJECT LOCATION	SEC Bundy Canyon & Monte Vista Wildomar, CA
PROJECT NUMBER	R206-025

FIGURE NO.

A-9

LOG OF BORING B-08

DRILLING RIG **CME-75**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1407 ft**

DATE DRILLED **5/21/20**

South Center Basin

HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0 1 2 3 4	SM		**YOUNGER ALLUVIUM** SILTY SAND, with trace gravel, fine- to medium, dark olive-brown (2.5Y 3/3), moist, loose to medium dense.		☒	SS	11 12	2	110
5 6 7 8	SM		SILTY SAND, fine- to coarse, dark olive-brown (2.5Y 3/3), slightly moist, medium dense, with thin interbeds of silty sand with trace clay.		☒	SS	7 12	6	123
9 10	SP-SM		SAND with SILT, fine- to coarse, dark olive-brown (2.5Y 3/3), moist, medium dense.						
11 12 13	SM		SILTY SAND, with trace clay, fine- to medium, dark olive-brown (2.5Y 3/3), moist, loose.		☒	SS	5 5	10	107
14 15 16 17 18	SM		SILTY SAND, with trace clay, very fine- to coarse, dark olive-brown (2.5Y 3/3), moist, medium dense, with thin interbeds of silty, clayey sand.		☒	SS	9 10	12	111
19 20	SM		SILTY SAND, medium, dark olive-brown (2.5Y 3/3), moist, loose.		⧗	SPT	4 5	4	
21 22 23	SM		SILTY SAND, fine- to medium, dark olive-brown (2.5Y 3/3), moist, loose.		⧗	SPT	3 5	6	
24	SM		SILTY SAND, medium-grained, dark olive-brown (2.5Y 3/3), moist, loose.						
25 26 27	SW-SM		SAND with SILT, fine- to coarse, darl olive-brown (2.5Y 3/3), slightly moist, medium dense.		⧗	SPT	7 9	3	
			End of boring at 27.5 feet. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-10

LOG OF BORING B-09

DRILLING RIG **CME-75**
DATE DRILLED **5/21/20**
HAMMER TYPE **Auto-Trip**
DRILLING METHOD **Rotary Auger**
HAMMER WEIGHT **140-lb.**
LOGGED BY **FWC**
HAMMER DROP **30-inches**
GROUND ELEVATION **+/- 1411 ft**
Basin Center
BORING DIAMETER **8-inches**

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0 1 2 3 4 5	SC-SM	▩	ARTIFICIAL FILL, SILTY, CLAYEY SAND with GRAVEL, fine- to medium, dark olive-brown (2.5Y 3/3), slightly moist, medium dense.			AU	18 18	8	105
6 7 8 9	SC	▨	**OLDER ALLUVIUM** CLAYEY SAND, very fine- to fine, dark olive-brown (2.5Y 3/3), moist, medium dense to dense, moderately cemented.			AU	16 18	3	111
10 11 12 13	SC	▨	CLAYEY SAND, very fine- to fine, dark olive-brown (2.5Y 3/3), moist, dense, moderately cemented.			AU			
14 15 16 17	SM	░	SILTY SAND, with trace clay, fine- to medium, dark olive-brown (2.5Y 3/3), moist, dense, moderately cemented.			AU	20 20	6	127
18 19 20 21	SM	░	SILTY SAND, fine- to medium, dark olive-brown (2.5Y 3/3), moist, dense, moderately cemented.			AU	50	2	126
22 23 24 25	SC	▨	CLAYEY SAND, very fine- to fine, dark olive-brown (2.5Y 3/3), moist, very dense, moderately cemented.			AU	38 48 18 25	11 10	129
26 27 28	SM	░	SILTY SAND, fine- to medium, dark olive-brown (2.5Y 3/3), moist, very dense, moderately cemented.				23 22	5	
			End of boring at 28.5 feet. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-11

LOG OF BORING B-10

DRILLING RIG **CME-75**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1416 ft**

DATE DRILLED **5/22/20**

North Center Basin

HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0 1 2 3 4 5 6 7	SC	▨	**YOUNGER ALLUVIUM** CLAYEY SAND, very fine- to fine-, dark olive-brown (2.5Y 3/3), moist, loose.		☒	SS	17 24	9	105
8 9 10 11	SC-SM	▨⁘	SILTY, CLAYEY SAND, fine- to medium, dark olive-brown (2.5Y 3/3), slightly moist to moist, dense, moderately cemented.		☒	SS	14 18	6	120
12 13	SC	▨	**OLDER ALLUVIUM** CLAYEY SAND, very fine- to fine, dark olive-brown (2.5Y 3/3), moist, very dense.		☒	SS	40 50/4"	9	117
14 15	SC	▨	CLAYEY SAND, fine- to coarse, dark olive-brown (2.5Y 3/3), slightly moist, very dense, moderately cemented.						
16 17 18 19	SM	⁘	SILTY SAND with GRAVEL, fine- to coarse-grained, light olive-brown (2.5Y 5/3), slightly moist, dense, cobbles - granitic boulder -		☒	SS	37 55	1	126
20 21 22	SC-SM	▨⁘	SILTY, CLAYEY SAND, very fine- to fine, dark yellowish-brown (10YR 3/4), slightly moist, dense, moderately to weakly cemented.		☒	SS	20 50	12	119
23 24	SM	⁘	SILTY SAND, very fine- to fine, dark yellowish-brown (10YR 3/4), moist, dense.						
25	SM	⁘	SILTY SAND, fine- to very coarse, dark yellowish-brown (10YR 3/4), slightly moist, very dense, strongly cemented.		☒	SS	36 50/4"	11	118
26 27 28 29	SC	▨	>CLAYEY SAND, fine- to medium, dark yellowish-brown (10YR 3/4), moist, very dense, strongly cemented.						
30 31	SC	▨	CLAYEY SAND, very fine- to fine, dark yellowish-brown (10YR 3/4), moist, very dense, weakly to moderately cemented.		☒	SS	40 50	12	126
32	SM	⁘	SILTY SAND, with trace gravel, fine- to very coarse, dark olive-brown (2.5Y 3/3), moist, very dense.		☒	SS	50	6	122
			End of boring at 32.5 feet. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-12

LOG OF BORING B-11

DRILLING RIG CME-75
DRILLING METHOD Rotary Auger
LOGGED BY FWC
GROUND ELEVATION +/- 1433 ft
DATE DRILLED 5/26/20
HAMMER TYPE Auto-Trip
HAMMER WEIGHT 140-lb.
HAMMER DROP 30-inches
BORING DIAMETER 8-inches

North End of Basin

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0–6	SM	**OLDER ALLUVIUM** SILTY SAND, fine- to medium, dark yellowish-brown (10YR 3/4), moist, dense, moderately to strongly cemented.	AU	22 50	2	125
6–12	SM	SILTY SAND, fine- to medium, dark yellowish-brown (10YR 3/4), slightly moist, very dense, moderately cemented.	AU	25 50	6	116
12–16	SM	SILTY SAND, with trace clay, fine- to medium, dark yellowish-brown (10YR 3/4), slightly moist to moist, dense, moderately cemented.	AU	18 22	5	114
16–25	SC-SM	SILTY, CLAYEY SAND, fine- to medium, dark yellowish-brown (10YR 3/4), slightly moist, very dense, weakly to strongly cemented.	AU	24 30 27 37 18 50	6 5 9	127 127 119
25–56	SM	GRANITE, highly to moderately weathered, olive (5Y 4/3) to very dark gray (5Y 3/1). - very hard drilling - - moderately to slightly weathered -	AU	35 50 23 50/5" 50 50/3" 50/3" 50/5" 50/3" 50/3"	6 15 5 4 9 1 4 8	119 108 120 121 118 154 136

End of boring at 56.25 feet. No groundwater encountered. Backfilled with native soils.

Inland Foundation Engineering, Inc.

CLIENT RCFCD
PROJECT NAME Wildomar Basin
PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA
PROJECT NUMBER R206-025

FIGURE NO. A-13

LOG OF BORING B-12

DRILLING RIG	CME-75	DATE DRILLED	5/21/20	HAMMER TYPE	Auto-Trip
DRILLING METHOD	Rotary Auger			HAMMER WEIGHT	140-lb.
LOGGED BY	FWC			HAMMER DROP	30-inches
GROUND ELEVATION	+/- 1410 ft		Southeast Basin Corner	BORING DIAMETER	8-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0–3.5	SM		**YOUNGER ALLUVIUM** SILTY SAND, fine- to medium, dark yellowish-brown (10YR 3/4), moist, loose.						
3.5–18.5	SM		SILTY SAND, with trace gravel, fine- to very coarse, dark yellowish-brown (10YR 3/4), slightly moist, loose, interbedded with silty sand.						
4–5				▌	☒	AU SS	4 5	2	119
8–9				▌	☒	SS	5 5	3	126
12–13				▌	☒	SS	10 8	4	121
16–17				▌	☒	SS	7 11	5	122
18.5–24.5	SC-SM		SILTY, CLAYEY SAND, fine- to medium-grained, dark yellowish-brown (10YR 3/4), moist, medium dense.						
20–21					☒	SS	15 15	9	121
24–25					☒	SS AU	12 21	4	125
24.5–33.5	SW-SM		SAND with SILT, fine- to very coarse, dark yellowish-brown (10YR 4/6), moist, dense, with thin interbeds of silty sand.	▌					
29–30				▌	☒	SS	19 31	5	123
33.5–35.5	SC		▽ CLAYEY SAND with GRAVEL, fine- to coarse, dark brown (10YR 3/4), very moist to wet, medium dense to dense.						
35					☒	SS	17 17	6	135
			End of boring at 35.5 feet. Groundwater encountered at 34 feet. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT	RCFCD
PROJECT NAME	Wildomar Basin
PROJECT LOCATION	SEC Bundy Canyon & Monte Vista Wildomar, CA
PROJECT NUMBER	R206-025
FIGURE NO.	A-14

LOG OF BORING B-13

DRILLING RIG **CME-75**
DATE DRILLED **5/20/20**
HAMMER TYPE **Auto-Trip**
DRILLING METHOD **Rotary Auger**
HAMMER WEIGHT **140-lb.**
LOGGED BY **FWC**
HAMMER DROP **30-inches**
GROUND ELEVATION **+/- 1413 ft**
BORING DIAMETER **8-inches**

Southeast Basin Corner

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
1	SM	YOUNGER ALLUVIUM SILTY SAND, fine- to coarse, very dark brown (7.5YR 2.5/2), slightly moist, loose.	AU			
5			SS	3 6	3	112
7	SW-SM	SAND with SILT, with trace gravel, fine- to coarse-grained, very dark brown (7.5YR 2.5/2), moist, medium dense.	AU			
9			SS	10 13	4	125
13	SM	SILTY SAND, very fine- to fine, very dark grayish-brown (7.5YR 2.5/2), moist, loose.	SS	6 7	11	118
14	SM	SILTY SAND, with trace gravel, fine- to coarse, dark brown (10YR 3/3), moist, loose.	AU			
17	SC	CLAYEY SAND, very fine- to fine, dark-brown (10YR 3/3), moist, loose.	SPT	3 3	6	
21			SPT	4 8	6	
24	SM	GRANITE, moderately weathered, grayish-brown (10YR 5/2).				
26			SS SPT	50/4" 50/4"	6 3	111
30			SPT	50/5"	2	
35			SPT	50/5"	4	

End of boring at 35.42 feet. No groundwater encountered. Backfilled with native soils.

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO. A-15

LOG OF BORING B-14

DRILLING RIG **CME-75**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1412 ft**

DATE DRILLED **5/22/20**

East Side of Basin

HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0									
1	GW-GM		ARTIFICIAL FILL, SILTY GRAVEL with SAND, fine- to medium, olive-brown (2.5Y 4/4), slightly moist to moist, medium dense, with cobbles and boulders.			AU			
2									
3									
4									
5					☒	SS	9 10	10	127
6	SC		**OLDER ALLUVIUM** CLAYEY SAND, very fine- to medium, olive-brown (2.5Y 4/4), moist, medium dense.						
7						AU			
8	SC-SM		SILTY, CLAYEY SAND, very fine- to fine, olive-brown (2.5Y 4/4), moist, medium dense.						
9					☒	SS	13 11		
10						AU			
11	SM		SILTY SAND, with trace clay, fine- to medium, dark olive-brown (2.5Y 3/3), moist, medium dense to dense.						
12									
13					☒	SS	11 18	7	95
14	CL		SANDY CLAY, dark olive-brown (2.5Y 3/3), moist, hard, moderately cemented.			AU			
15									
16	SC		CLAYEY SAND, with trace gravel, fine- to coarse, dark reddish-brown (5Y 3/3), moist, dense, moderately cemented.			AU			
17					☒	SS	14 15	11	-3205
18									
19									
20									
21					☒	SS	7 7	11	118
22			- weakly cemented -						
23									
24			- strongly cemented -						
25					☒	SS	19 25	11	118
26			- very hard drilling -						
27						AU			
28	SM		SILTY SAND, very fine- to fine, dark yellowish-brown (10YR 3/4), moist, very dense, moderately cemented.						
29					☒	SS	20 50	14	123
30									
31									
32	GW-GC		GRAVELLY SAND, fine- to medium, yellowish-brown (10Y 5/4), moist, very dense.						
33					☒	SS	50/3"	5	
34									
35	SM		GRANITE, moderately weathered, yellowish-brown (10Y 5/4).						
36					☒	SS	50	4	124
			End of boring at 36.5 feet. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT	**RCFCD**
PROJECT NAME	**Wildomar Basin**
PROJECT LOCATION	**SEC Bundy Canyon & Monte Vista** **Wildomar, CA**
PROJECT NUMBER	**R206-025**

FIGURE NO.
A-16

IFE BORING - GINT STD US LAB.GDT - 8/7/20 11:12 - P:\R206\R206-025 WILDOMAR\GINT.GPJ

LOG OF BORING B-15

DRILLING RIG	**CME-75**	DATE DRILLED	**5/20/20**	HAMMER TYPE	**Auto-Trip**
DRILLING METHOD	**Rotary Auger**			HAMMER WEIGHT	**140-lb.**
LOGGED BY	**FWC**			HAMMER DROP	**30-inches**
GROUND ELEVATION	**+/- 1415 ft**		**Northeast Basin Corner**	BORING DIAMETER	**8-inches**

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0									
1			**OLDER ALLUVIUM**			AU			
2	SC-SM		SILTY, CLAYEY SAND, very fine- to fine, brown ((10Y 4/3), moist, loose to medium dense.						
3									
4						SS	9	7	116
5	SC-SM		SILTY, CLAYEY SAND, with trace gravel, fine- to medium, brown (10Y 4/3), moist, dense.			AU	8		
6						AU	20	3	135
7			SILTY SAND, with trace clay and gravel, fine- to very coarse, yellowish-brown (10YR 5/4), moist, dense, with thin interbeds of sand.			SS	33		
8									
9	SM								
10						SS	17	3	121
11							27		
12	SM		SILTY SAND, very fine- to fine, dark grayish-brown (10YR 4/2), moist, dense.						
13						SS	37	8	112
14						AU	40		
15			SILTY SAND, with trace clay and gravel, fine- to very coarse, brown (10YR 4/3), moist, dense, moderately cemented.						
16	SM								
17						SPT	21	5	
18							18		
19						AU			
20			CLAYEY SAND, very fine- to fine, brown (10YR 5/3), moist, dense, interbedded with sandy clay, moderately cemented.						
21									
22						SPT	12	13	
23	SC						18		
24									
25									
26									
27			CLAYEY SAND, very fine- to fine, olive-brown (2.5Y 5/4), moist, dense, moderately cemented.			SPT	15	10	
28	SC						18		
29									
30			SILTY SAND, fine- to medium, olive-brown (2.5Y 5/4), moist, dense, moderately cemented, with thin interbeds of clayey sand.						
31									
32						SPT	22	9	
33	SM						23		
34									
35									
36									
37	SC		CLAYEY SAND, very fine- to fine, olive-brown (2.5Y 4/3), moist, dense.			SS	28	9	123
38	SM					SPT	40	16	
39		▽	SILTY SAND, with trace clay, very fine- to fine, olive-brown (2.5Y 4/4), very moist, dense.				13		
40							18		
41			GRANITE, highly to moderately weathered, olive-brown (2.5Y 4/3), wet.						
42						SPT	35	11	
43							27		
44									
45	SM								
46						SPT	50	11	
47									
48									
49									
50									
						SPT	40 50/4"	12	
			End of boring at 50.83 feet. Groundwater encountered at 39 feet.. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**

PROJECT NAME **Wildomar Basin**

PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA

PROJECT NUMBER **R206-025**

FIGURE NO. A-17

LOG OF BORING B-16

DRILLING RIG: CME-75 Track Rig
DATE DRILLED: 5/28/20
HAMMER TYPE: Auto-Trip
DRILLING METHOD: Rotary Auger
HAMMER WEIGHT: 140-lb.
LOGGED BY: FWC
HAMMER DROP: 30-inches
GROUND ELEVATION: +/- 1405 ft
BORING DIAMETER: 8-inches

Southeast Basin Corner
EVMWD Sewer Station 36+50

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0									
1			**YOUNG WASH DEPOSITS** SILTY SAND, fine- to coarse, very dark brown (10YR 2/2), slightly moist, loose to medium dense, with thin interbeds of sand.			AU			
2	SM								
3					☒	SS	7 6	2	118
4			SAND with SILT, fine- to very coarse-grained, very dark grayish-brown (10YR 3/2), slightly moist, medium dense, with thin interbeds of silty sand.			AU			
5									
6	SP-SM								
7					☒	SS	13 11	6	115
8									
9			SAND with SILT, with trace gravel, fine- to very coarse, very dark gray (10YR 3/1), moist, loose.			AU			
10	SW-SM								
11					☒	SS	6 7	6	113
12			SILTY SAND, with trace clay and gravel, fine- to very coarse-grained, very dark grayish-brown (10YR 3/2), moist, loose to medium dense.			AU			
13									
14	SM								
15					☒	SS	19 24	9	128
16									
17									
18			SILTY, CLAYEY SAND, fine- to very coarse-grained, very dark grayish-brown (10 3/2), very moist to wet, dense, interbedded with sand.			AU			
19	SC-SM	∇			☒	SS	30 27	7	135
20									
21									
22	SM		GRANITE, slightly weathered, olive.			SS	1/1"		
			End of boring at 22 feet. Groundwater encountered at 19 feet. Auger refusal. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT: RCFCD
PROJECT NAME: Wildomar Basin
PROJECT LOCATION: SEC Bundy Canyon & Monte Vista
Wildomar, CA
PROJECT NUMBER: R206-025

FIGURE NO.
A-18

LOG OF BORING B-17

DRILLING RIG **CME-75 Track Rig**
DATE DRILLED **5/28/20**
HAMMER TYPE **Auto-Trip**
DRILLING METHOD **Rotary Auger**
HAMMER WEIGHT **140-lb.**
LOGGED BY **FWC**
HAMMER DROP **30-inches**
GROUND ELEVATION **+/- 1414 ft**
EVMWD Sewer Station 38+30
BORING DIAMETER **8-inches**

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0									
1			ARTIFICIAL FILL, SILTY SAND, fine- to medium, brown, moist, loose, gypsum board debris.			AU			
2	SM								
3						AU			
4			**YOUNGER ALLUVIUM** SAND with SILT, fine- to very coarse-grained, very dark gray (7.5YR 3/1), moist, loose to medium dense.				6 7	5	108
5	SW-SM								
6									
7			SILTY SAND, with trace gravel, fine- to very coarse, very dark brown (10YR 2/2), moist, medium dense, with thin interbeds of sand with silt.			AU			
8							7 8	5	112
9									
10	SM								
11									
12							12 8	7	121
13			SAND with SILT, fine- to very coarse-grained, dark yellowish-brown (10YR 4/2), moist, medium dense.			AU			
14									
15	SW-SM								
16							9 9	13	109
17			CLAYEY SAND, very fine- to fine, dark yellowish-brown (10YR 3/4), moist, medium dense.			AU			
18	SC								
19			SILTY SAND, fine- to medium, dark grayish-brown (10YR 4/2), moist, medium dense, with thin interbeds of sand.						
20							11 11	6	116
21									
22	SM								
23									
24							10 12	12	120
			End of boring at 24.5 feet. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-19

LOG OF BORING B-18

DRILLING RIG CME-75
DRILLING METHOD Rotary Auger
LOGGED BY FWC
GROUND ELEVATION +/- 1414 ft

DATE DRILLED 5/28/20

West End Valley Vista Circle

HAMMER TYPE Auto-Trip
HAMMER WEIGHT 140-lb.
HAMMER DROP 30-inches
BORING DIAMETER 8-inches

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0			ASPHALT CONCRETE over AGGREGATE BASE, (2.5 inches over 4 inches)						
1	SC		**YOUNGER ALLUVIUM** CLAYEY SAND, very fine- to fine, red-brown, moist, medium dense.			AU			
3	SW-SM		SAND with SILT, fine- to medium, very dark grayish-brown (10YR 3/2), moist, medium dense.			AU			
5	SP-SM		GRAVELLY SAND, fine- to coarse, brown (10YR 4/3), moist, medium dense.		☒	SS AU	11 12	5	116
9					☒	SS	6 8	5	119
11	SC		**OLDER ALLUVIUM** CLAYEY SAND, fine- to coarse, dark yellowish-brown (10YR 4/4), moist, very dense.						
13					☒	SS	23 25	7	131
15	SW-SM		SAND with SILT, fine- to very coarse-grained, dark grayish-brown (10YR 4/4), moist, medium dense.						
16					☒	SS	50/3"	4	116
20					☒	SS	50/4"	4	119

End of boring at 20.33 feet. No groundwater encountered. Backfilled with native soils.

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CLIENT RCFCD
PROJECT NAME Wildomar Basin
PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA
PROJECT NUMBER R206-025

FIGURE NO.
A-20

LOG OF BORING B-19

DRILLING RIG CME-75 Track Rig
DATE DRILLED 5/28/20
HAMMER TYPE Auto-Trip
DRILLING METHOD Rotary Auger
HAMMER WEIGHT 140-lb.
LOGGED BY FWC
HAMMER DROP 30-inches
GROUND ELEVATION +/- 1405 ft
BORING DIAMETER 8-inches

Southeast Basin Corner
at Stilling Basin

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0			YOUNG WASH DEPOSITS <u>SILTY SAND,</u> fine- to coarse, grayish-brown (2.5Y 4/2), dry to moist, loose to medium dense.			AU			
1	SM								
2							23 50	6	114
	GC		<u>GRANITIC BOULDERS,</u> (Rip-Rap).						
			End of boring at 2.56 feet. No groundwater encountered. Auger refusal. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT RCFCD
PROJECT NAME Wildomar Basin
PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA
PROJECT NUMBER R206-025

FIGURE NO.
A-21

LOG OF BORING B-20

DRILLING RIG **CME-75 Track Rig**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1408 ft**

DATE DRILLED **5/28/20**

Transition Structure
at Southeast Basin Corner

HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0			**YOUNG WASH DEPOSITS** SAND with SILT, fine- to coarse, light brown, dry to slightly moist, loose.	▮		AU			
1	SW-SM			▮					
2				▮					
	GC	▨	GRANITIC BOULDERS, (Rip-Rap).						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-22

LOG OF BORING B-21

DRILLING RIG **CME-75 Track Rig**
DATE DRILLED **6/25/20**
HAMMER TYPE **Auto-Trip**
DRILLING METHOD **Rotary Auger**
HAMMER WEIGHT **140-lb.**
LOGGED BY **FWC**
HAMMER DROP **30-inches**
GROUND ELEVATION **+/- 1406 ft**
BORING DIAMETER **8-inches**

Southeast Basin Corner

SUMMARY OF SUBSURFACE CONDITIONS

This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0–2	SM		**YOUNG WASH DEPOSITS** SILTY SAND, fine- to coarse, dark reddish-brown (5YR 3/4), slightly moist to moist, loose to medium dense.	▮		AU			
2				▮		AU			
3	SM		SILTY SAND, fine- to very coarse, dark reddish-brown (5YR 3/4), moist, medium dense to dense, roots.	▮	☒	SS	15 14	7	137
5				▮		AU			
7	SM		SILTY SAND, with trace gravel, fine- to medium, dark reddish-brown (5YR 3/4), moist, medium dense.	▮	☒	SS	12 10	3	132
9	SC-SM		SILTY, CLAYEY SAND, very fine- to medium, dark reddish-brown (5YR 3/2), moist, medium dense.	▮		AU			
11	SM		SILTY SAND, with trace gravel, fine- to coarse, dark reddish-brown (5YR 3/2), moist, medium dense to loose, with thin interbeds of sand.		☒	SS	8 8	5	121
15					☒	SS	6 6	7	114
16	SC		CLAYEY SAND, fine- to medium, dark reddish-brown (5YR 3/2), moist, loose to medium dense.						
19	SM		SILTY SAND, fine- to very coarse, dark reddish-brown (5YR 3/2), moist, dense, with thin interbeds of sand or clayey sand.		☒	SS	10 21	5	130
23					☒	SS	22 28	9	122
24	SC		CLAYEY SAND, very fine- to fine, dark brown (7.5YR 3/2), very moist to wet, medium dense, with thin interbeds of silty sand.	▮		AU			
26				▮	☒	SS	11 12		
29		▽		▮					
31				▮	☒	SS	10 29	12	126

End of boring at 31.5 feet. Groundwater encountered at 29 feet..

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Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-23

LOG OF BORING B-22

DRILLING RIG **CME-75 Track Rig**
DATE DRILLED **6/25/20**
DRILLING METHOD **Rotary Auger**
LOGGED BY **FWC**
GROUND ELEVATION **+/- 1405 ft**

Southeast Basin Corner

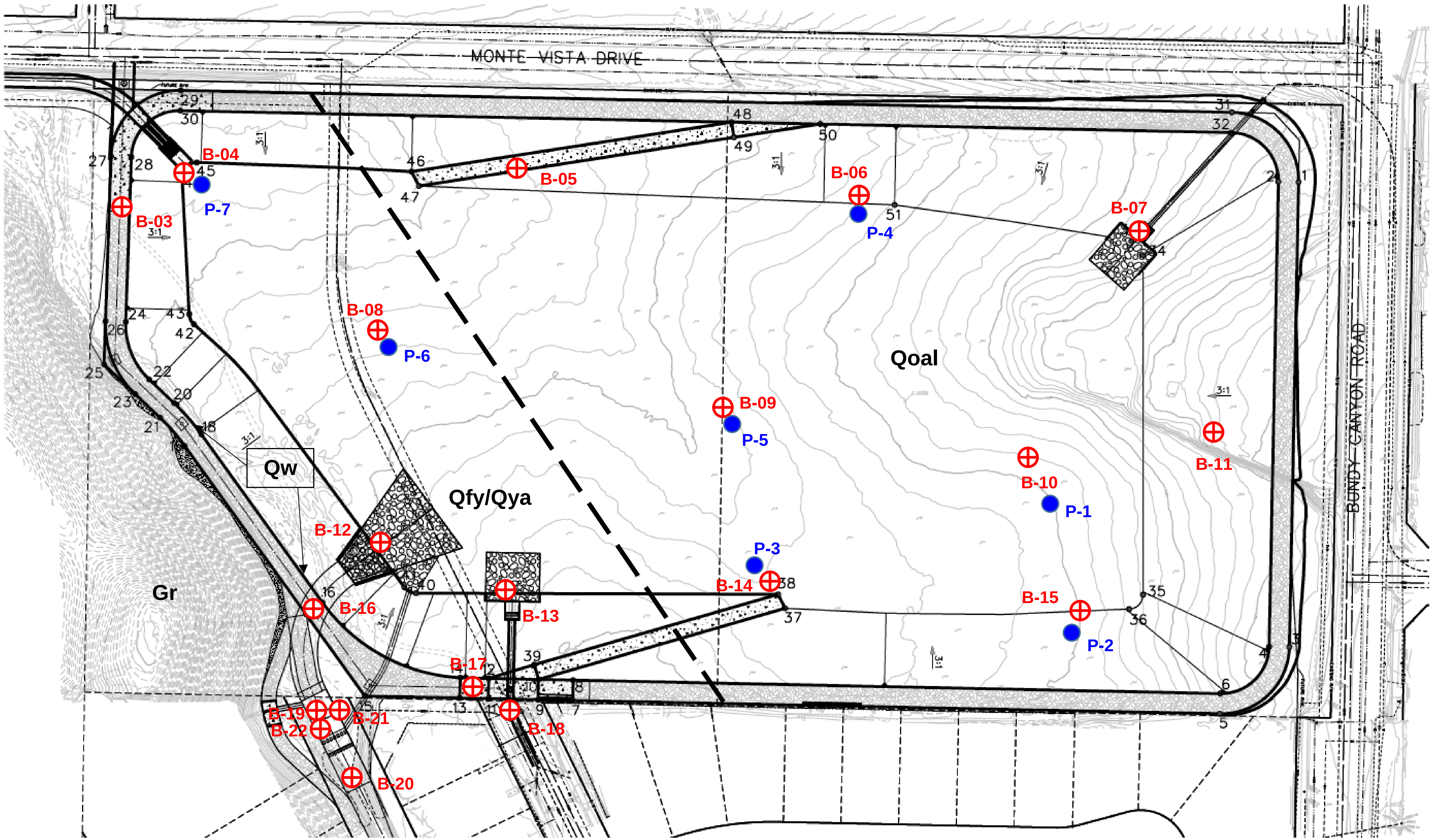
HAMMER TYPE **Auto-Trip**
HAMMER WEIGHT **140-lb.**
HAMMER DROP **30-inches**
BORING DIAMETER **8-inches**

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0			**YOUNG WASH DEPOSITS** SILTY SAND, fine- to coarse, olive-brown, slightly moist, loose.						
1	SM								
2	GC		RIP-RAP,						
			End of boring at 2.1 feet. Auger refusal. No groundwater or mottling encountered.						

Inland Foundation Engineering, Inc.

CLIENT **RCFCD**
PROJECT NAME **Wildomar Basin**
PROJECT LOCATION **SEC Bundy Canyon & Monte Vista**
Wildomar, CA
PROJECT NUMBER **R206-025**

FIGURE NO.
A-24



⊕ **Approximate Location of Exploratory Boring**

● **Approximate Location of Infiltration Test**

— — — **Approximate Geologic Contact**

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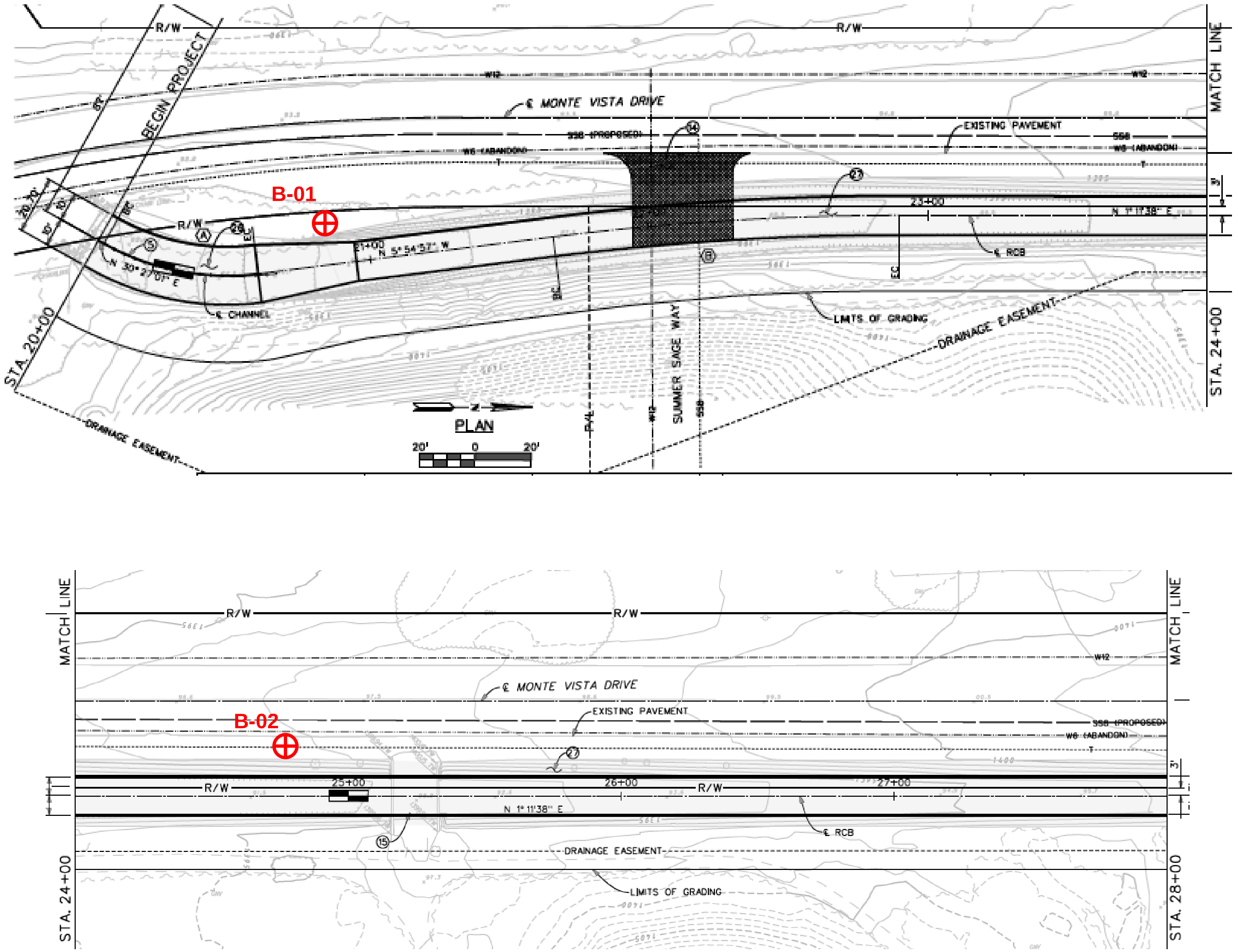
A-25

Geotechnical Investigation
Wildomar MDP Lateral C, Stage 3
Wildomar Area, Riverside County, CA

Drawn By: DRL
Project No. R206-025
Scale: 1” = 100’ ±
Date: Aug. 2020

SITE PLAN



⊕ **Approximate Location of Exploratory Boring**

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A-26

Geotechnical Investigation
Wildomar MDP Lateral C, Stage 3
Wildomar Area, Riverside County, CA

Drawn By: DRL
Project No. R206-025
Scale: 1" = 50' ±
Date: Aug. 2020

APPENDIX B –
Laboratory Testing

APPENDIX B

LABORATORY TESTING

Representative soil samples obtained from our borings were returned to our laboratory for additional observations and testing. Descriptions of the tests performed are provided below.

Unit Weight and Moisture Content: Ring samples were weighed and measured to evaluate their unit weight. A small portion of each sample was then tested for moisture content. The testing was performed per ASTM D2937 and D2216. The results of the testing are shown on the boring logs (Figure Nos. A-3 through A-24).

Maximum Density-Optimum Moisture Content: Seven samples were selected for maximum density testing. This testing was performed per the current ASTM Standard D1557 test method A. The results of this testing are presented graphically on Figure Nos. B-4 and B-5.

Sieve Analysis: Several soil samples were selected for sieve analysis testing in accordance with ASTM D6913. These tests provide information for classifying the soil in accordance with the Unified Classification System. This classification system categorizes the soil into groups having similar engineering characteristics. The results of this testing are shown on Figure Nos. B-6 through B-10.

Atterberg Limits: Several samples were selected for Atterberg limits testing in accordance with ASTM D4318. These tests provide information regarding soil plasticity and are also used for classifying the soil in accordance with the Unified Classification System. The results are shown on Figure No. B-6 through B-10

Sand Equivalent: Nine samples were selected for sand equivalent testing in accordance with ASTM D2419. This test is used to indicate the relative proportions of clay-size or plastic fines and dust in granular soil and fine aggregate. Sand equivalent test results are shown in the following table.

Boring No.	Depth (ft.)	SE
B-01	3.2 – 6.0	20
B-02	7.0 – 17.0	56
B-03	3.0 – 14.5	39
B-07	6.5 – 10.0	24
B-11	12.0 – 16.0	17
B-13	6.5 – 12.0	48
B-16	11.0 – 17.0	24
B-17	6.5 – 12.5	31
B-18	2.5 – 5.0	40

Direct Shear Strength: Several samples were selected for direct shear strength testing in accordance with ASTM D3080. This testing measures the shear strength of the soil under various normal pressures and is used to develop parameters for foundation bearing capacity and lateral earth pressure. Test results are shown on Figure Nos. B-11 through B-20.

Three samples were also transported to AP Engineering and Testing in Pomona, California for direct shear strength testing. Those results are shown on Figure Nos. B-21 through B-23.

CU Triaxial Compression: One sample was transported to AP Engineering and Testing in Pomona, California for consolidated undrained triaxial compression testing in accordance with ASTM D4767. This test is used to evaluate strength and stress-strain relationships of soil under undrained conditions. Test results are shown on Figure Nos. B-24a through B-24h.

Consolidation Testing: Two samples were selected for consolidation testing in accordance with ASTM D2435. This test is used to evaluate the magnitude and rate of settlement of a structure or earth fill. The results of this testing are presented graphically on Figure Nos. B-25 and B-26.

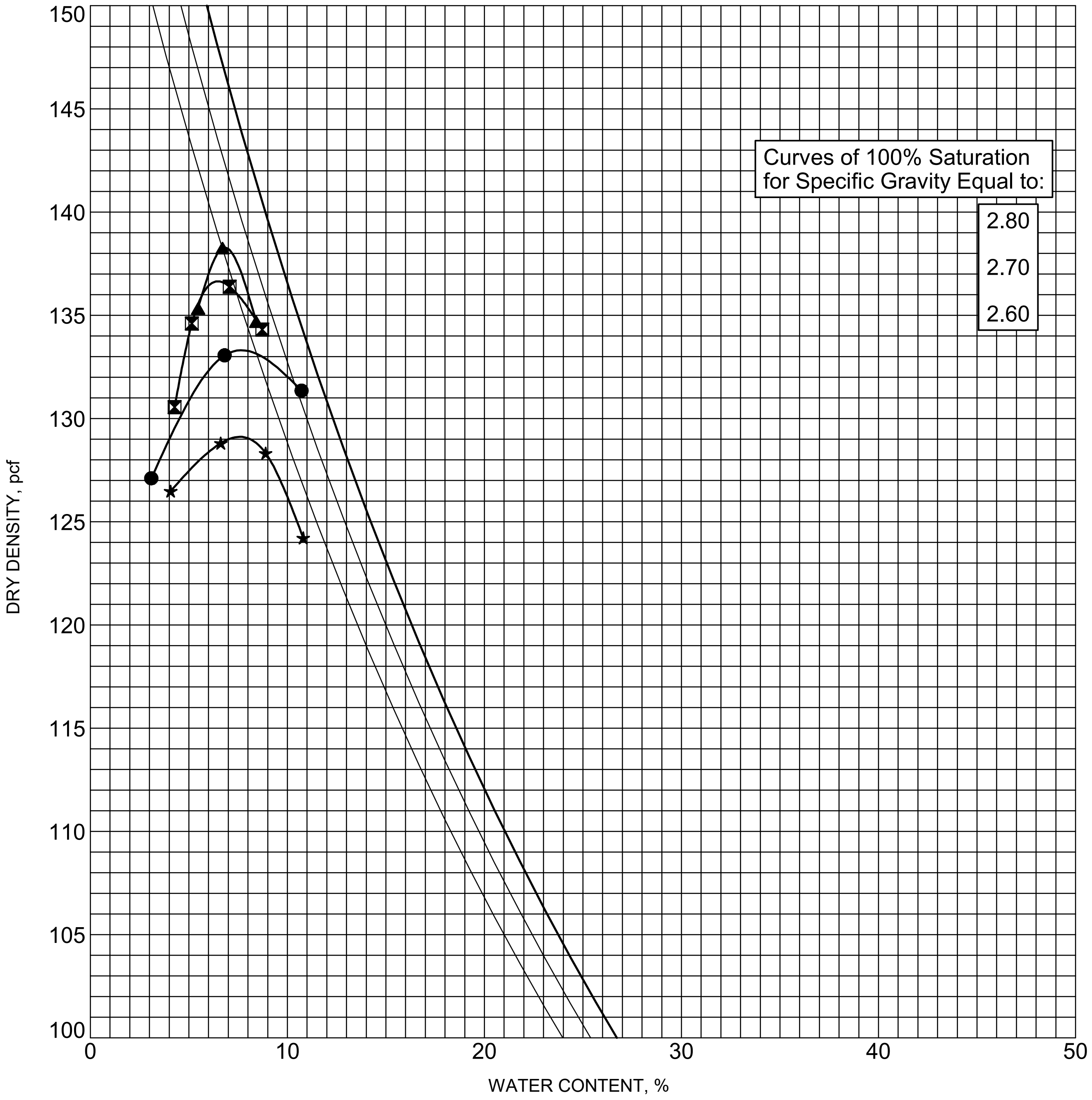
Permeability Testing: Representative, relatively undisturbed ring samples from the proposed detention basin exploration were transported to AP Engineering and Testing in Pomona, California for flexible wall permeability testing in accordance with ASTM

D5084. This testing indicates permeability values of the samples tested are on the order of 10^{-4} to 10^{-8} cm/sec. The permeability test results are shown in the following table.

Sample Location	**Sample Depth (ft.)**	**Soil Type (USCS)**	**Description**	**Coefficient of Permeability, cm/s**
B-04	23.5	SM	Silty Sand	8.50×10^{-5}
B-06	28.0	SM	Silty Sand	1.38×10^{-5}
B-10	28.5	CL	Sandy Clay	3.34×10^{-8}
B-14	36.0	SM	Silty Sand	2.40×10^{-4}
B-21	25.5	CL	Sandy Clay	1.36×10^{-7}

GENERAL

All laboratory testing has been conducted in conformance with the applicable ASTM test methods by personnel trained and supervised in conformance with our QA/QC policy. Our test data only relates to the specific soils tested. Soil conditions typically vary and any significant variations should be reported to our laboratory for review and possible testing. The data presented in this report are for the use of Riverside County Flood Control and Water Conservation District only and may not be reproduced or used by others without written approval of Inland Foundation Engineering, Inc.



	BOREHOLE	DEPTH	Description of Materials	Max DD	Optimum WC
●	**B-02**	**3.5**	**SILTY SAND (SM)**	**133.3 PCF**	**7.6 %**
⧗	**B-03**	**3.0**	**POORLY-GRADED SAND with SILT (SP-SM)**	**136.6 PCF**	**6.5 %**
▲	**B-04**	**20.0**	**SILTY SAND (SM)**	**138.3 PCF**	**6.8 %**
★	**B-07**	**6.5**	**SILTY, CLAYEY SAND (SC-SM)**	**129.1 PCF**	**7.6 %**

Inland Foundation Engineering, Inc.

MOISTURE-DENSITY CURVES (ASTM D1557)

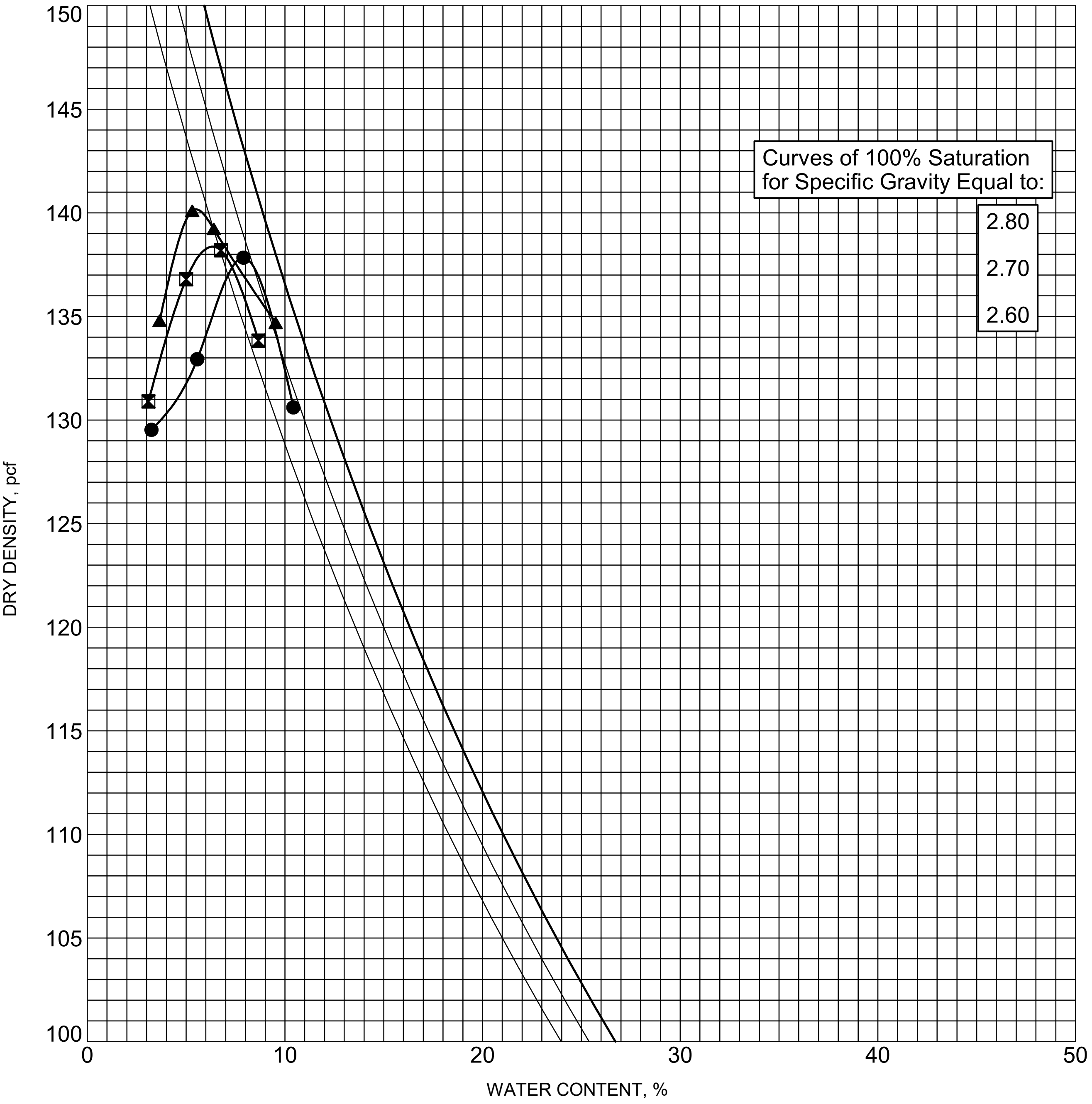
FIGURE NO. B-4

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



	BOREHOLE	DEPTH	Description of Materials	Max DD	Optimum WC
●	**B-12**	**24.0**	**WELL-GRADAED SAND with SILT (SW-SM)**	**137.8 PCF**	**7.9 %**
☒	**B-15**	**13.0**	**SILTY SAND(SM)**	**138.4 PCF**	**6.3 %**
▲	**B-16**	**11.0**	**SILTY SAND(SM)**	**140.2 PCF**	**5.5 %**

Inland Foundation Engineering, Inc.

MOISTURE-DENSITY CURVES (ASTM D1557)

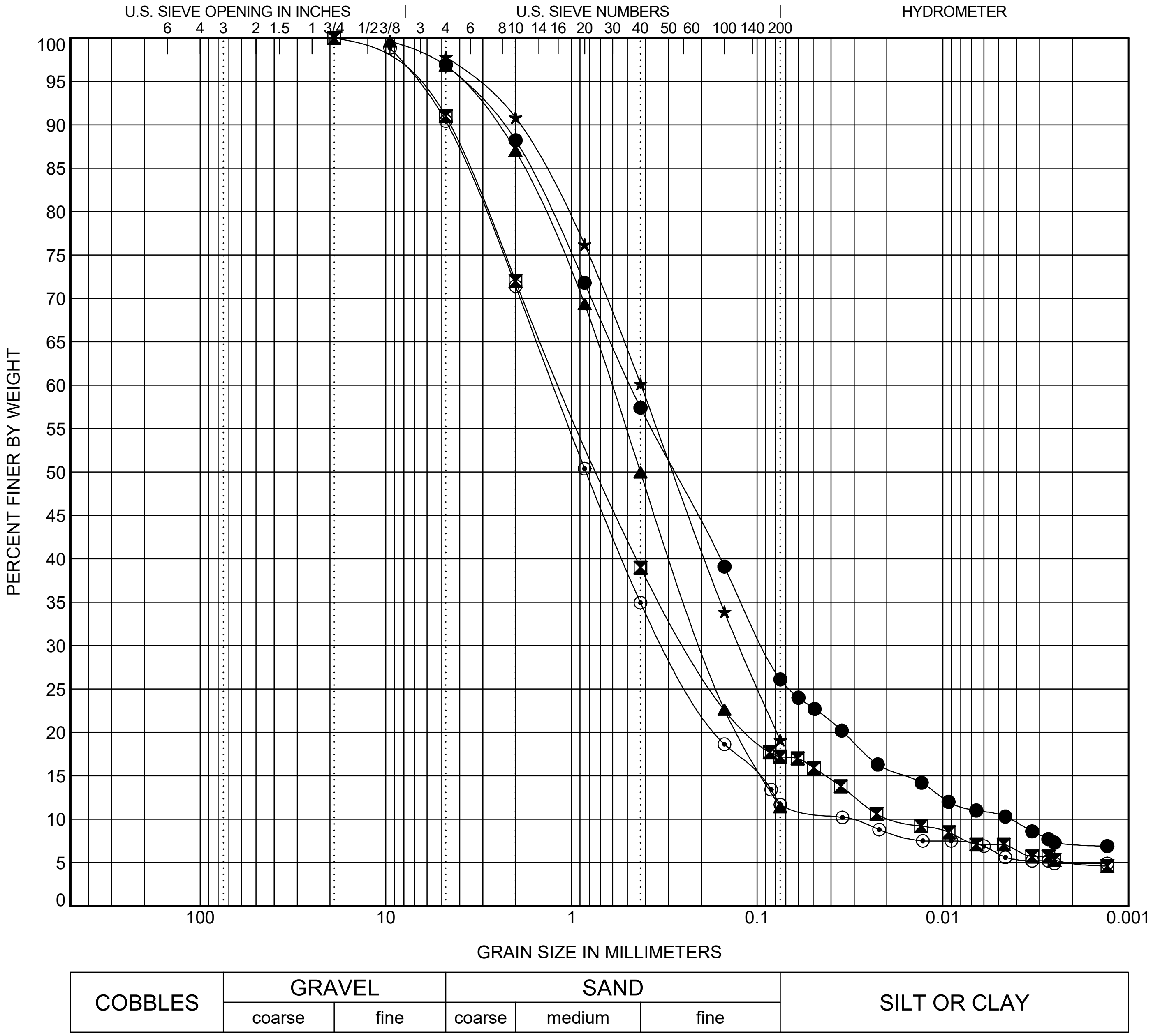
FIGURE NO. B-5

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**3.2**	**SILTY SAND(SM)**	**31**	**24**	**7**	**4.08**	**111.03**
⧗	**B-02**	**0.0**	**SILTY SAND (SM)**				**2.28**	**63.49**
▲	**B-02**	**7.0**	**POORLY-GRADED SAND with SILT (SP-SM)**	**NP**	**NP**	**NP**	**0.94**	**8.84**
★	**B-02**	**7.5**	**SILTY SAND (SM)**					
⊙	**B-03**	**3.0**	**POORLY-GRADED SAND with SILT (SP-SM)**				**2.35**	**38.65**

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**3.2**	**4.75**	**0.482**	**0.092**	**0.004**	**3.1**	**70.8**	**15.6**	**10.5**
⧗	**B-02**	**0.0**	**19**	**1.139**	**0.216**	**0.018**	**9.0**	**73.8**	**10.1**	**7.1**
▲	**B-02**	**7.0**	**9.5**	**0.608**	**0.199**		**2.8**	**85.4**	**11.4**	
★	**B-02**	**7.5**	**4.75**	**0.422**	**0.125**		**2.2**	**78.7**	**19.1**	
⊙	**B-03**	**3.0**	**9.5**	**1.257**	**0.31**	**0.033**	**8.4**	**78.8**	**5.7**	**6.0**

GRADATION CURVES (ASTM D422, ASTM D4318)

Inland Foundation Engineering, Inc.

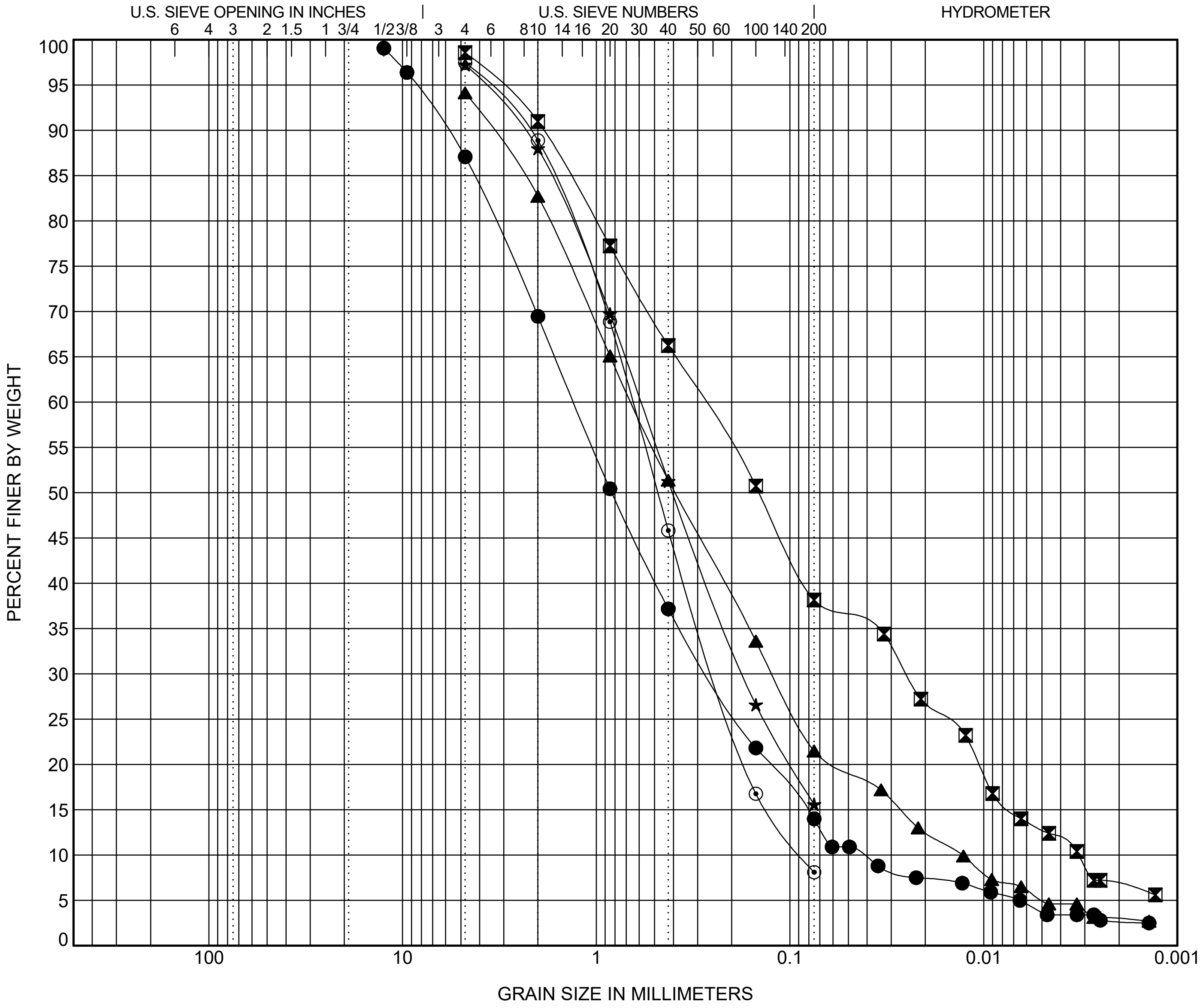
FIGURE NO. B-6

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-04**	**20.0**	**SILTY SAND (SM)**				**1.22**	**30.67**
⧗	**B-05**	**12.0**	**SILTY SAND (SM)**				**0.70**	**86.86**
▲	**B-07**	**6.5**	**SILTY, CLAYEY SAND (SC-SM)**				**1.75**	**50.86**
★	**B-07**	**30.5**	**SILTY SAND (SM)**					
⊙	**B-08**	**26.5**	**WELL-GRADED SAND with SILT(SW-SM)**	**NP**	**NP**	**NP**	**1.02**	**7.46**

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-04**	**20.0**	**12.5**	**1.307**	**0.261**	**0.043**	**12.0**	**73.1**	**10.3**	**3.7**
⧗	**B-05**	**12.0**	**4.75**	**0.28**	**0.025**	**0.003**	**1.5**	**60.4**	**25.3**	**12.8**
▲	**B-07**	**6.5**	**4.75**	**0.657**	**0.122**	**0.013**	**5.8**	**72.7**	**16.4**	**5.1**
★	**B-07**	**30.5**	**4.75**	**0.59**	**0.173**		**2.8**	**81.6**	**15.6**	
⊙	**B-08**	**26.5**	**4.75**	**0.651**	**0.241**	**0.087**	**2.5**	**89.4**	**8.1**	

GRADATION CURVES (ASTM D422, ASTM D4318)

Inland Foundation Engineering, Inc.

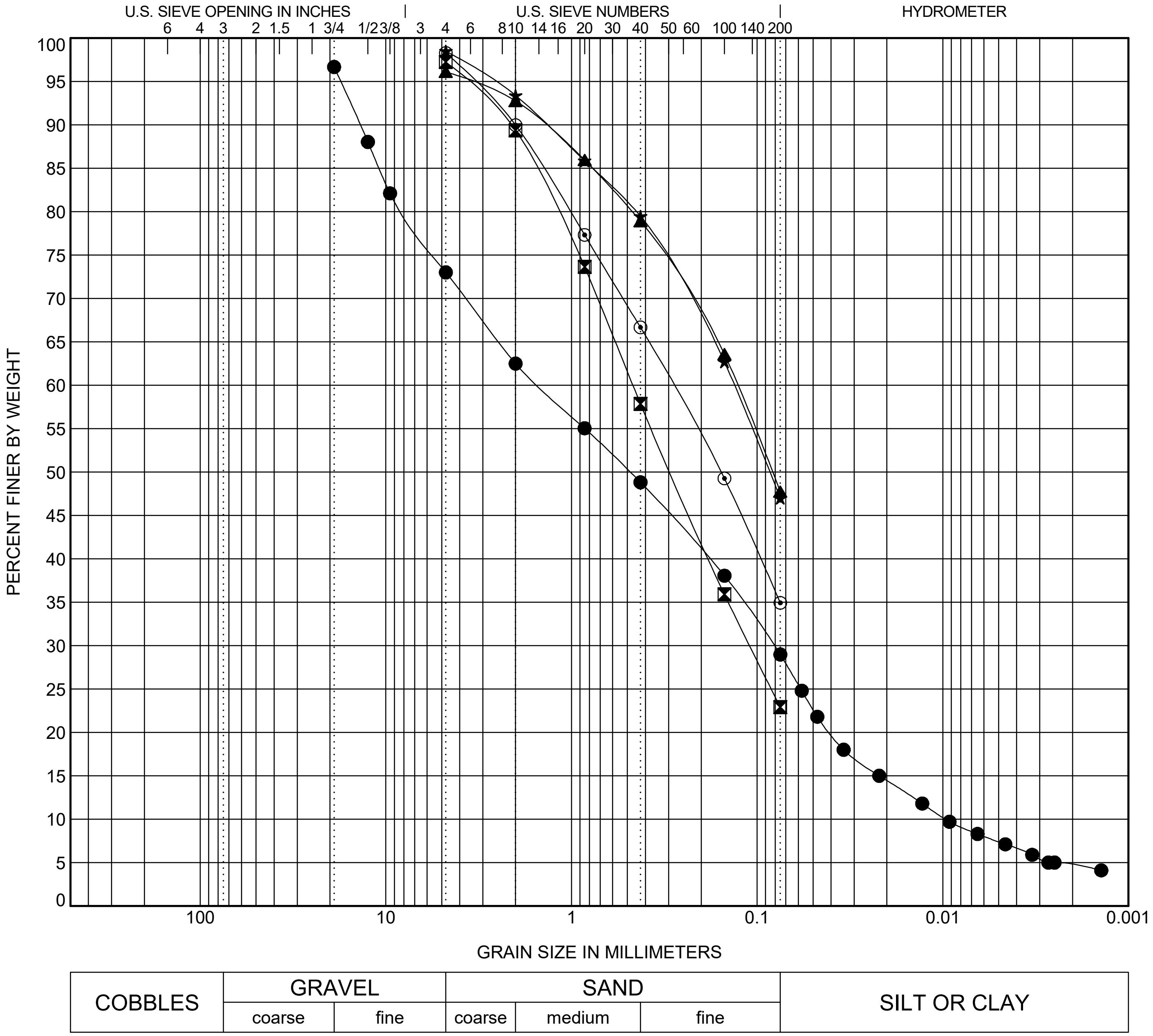
FIGURE NO. B-7

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE		DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-09**	**0.0**	**SILTY, CLAYEY SAND with GRAVEL(SC-SM)**	**27**	**21**	**6**	**0.45**	**155.68**
⧗	**B-09**	**27.5**	**SILTY SAND (SM)**					
▲	**B-10**	**4.5**	**CLAYEY SAND(SC)**	**28**	**19**	**9**		
★	**B-10**	**28.5**	**CLAYEY SAND (SC)**					
⊙	**B-11**	**12.5**	**SILTY SAND (SM)**					

BOREHOLE		DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-09**	**0.0**	**19**	**1.503**	**0.081**	**0.01**	**23.7**	**44.0**	**21.6**	**7.4**
⧗	**B-09**	**27.5**	**4.75**	**0.467**	**0.109**		**2.8**	**74.3**	**22.9**	
▲	**B-10**	**4.5**	**4.75**	**0.128**			**3.9**	**48.4**	**47.7**	
★	**B-10**	**28.5**	**4.75**	**0.133**			**1.5**	**51.5**	**47.0**	
⊙	**B-11**	**12.5**	**4.75**	**0.285**			**1.7**	**63.4**	**34.9**	

GRADATION CURVES (ASTM D422, ASTM D4318)

Inland Foundation Engineering, Inc.

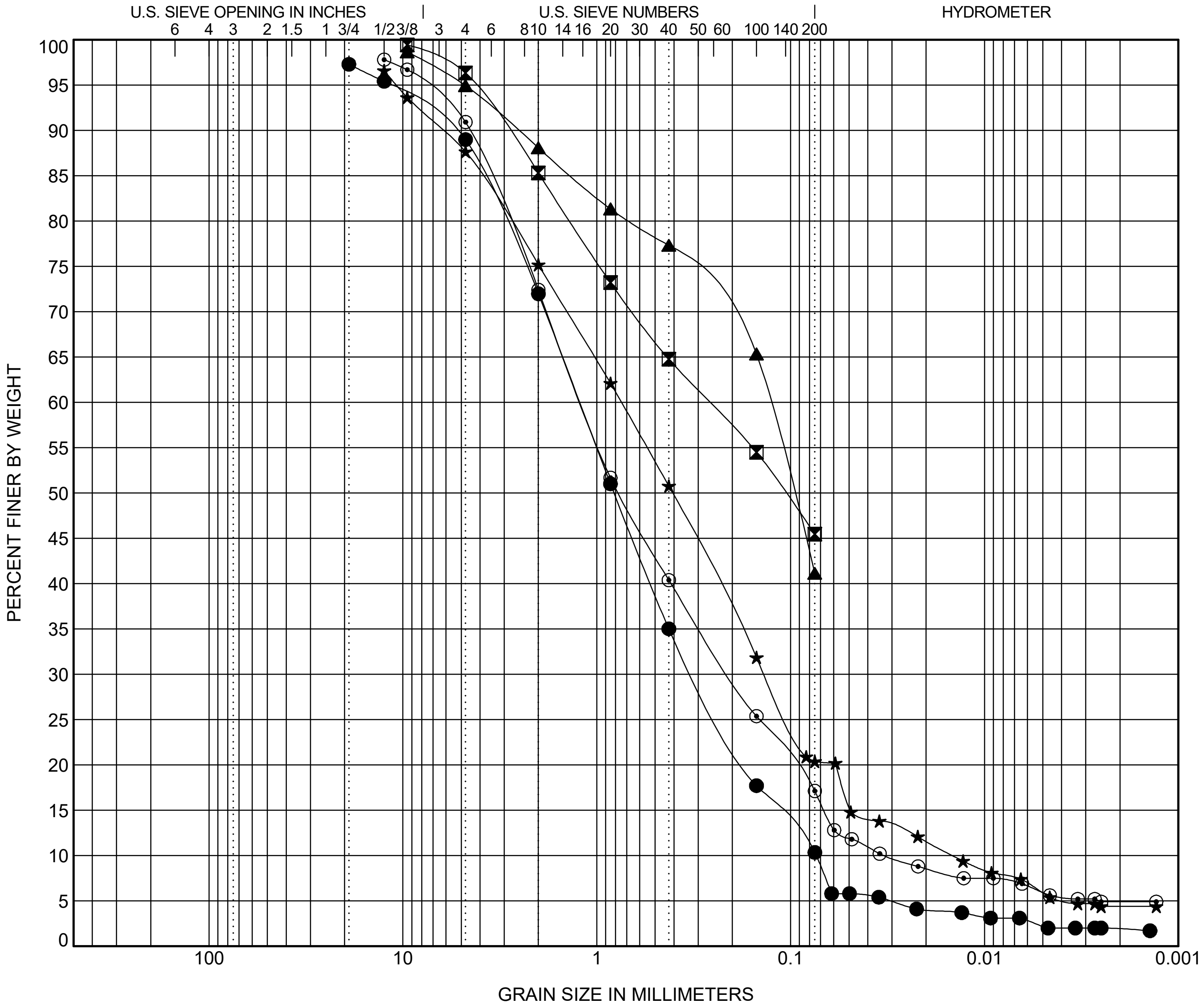
FIGURE NO. B-8

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-13**	**6.5**	**WELL-GRADED SAND with SILT(SW-SM)**	**NP**	**NP**	**NP**	**1.09**	**16.60**
⧗	**B-14**	**16.0**	**CLAYEY SAND (SC)**	**29**	**21**	**8**		
▲	**B-15**	**11.5**	**SILTY SAND (SM)**					
★	**B-15**	**13.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**	**1.70**	**51.39**
⊙	**B-16**	**11.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**	**1.10**	**36.87**

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-13**	**6.5**	**19**	**1.227**	**0.314**	**0.074**	**8.3**	**78.6**	**8.1**	**2.2**
⧗	**B-14**	**16.0**	**9.5**	**0.262**			**3.1**	**50.9**	**45.4**	
▲	**B-15**	**11.5**	**9.5**	**0.129**			**3.7**	**53.8**	**41.1**	
★	**B-15**	**13.0**	**12.5**	**0.747**	**0.136**	**0.015**	**9.0**	**67.3**	**14.5**	**5.9**
⊙	**B-16**	**11.0**	**12.5**	**1.199**	**0.207**	**0.033**	**6.9**	**73.8**	**11.2**	**5.9**

GRADATION CURVES (ASTM D422, ASTM D4318)

Inland Foundation Engineering, Inc.

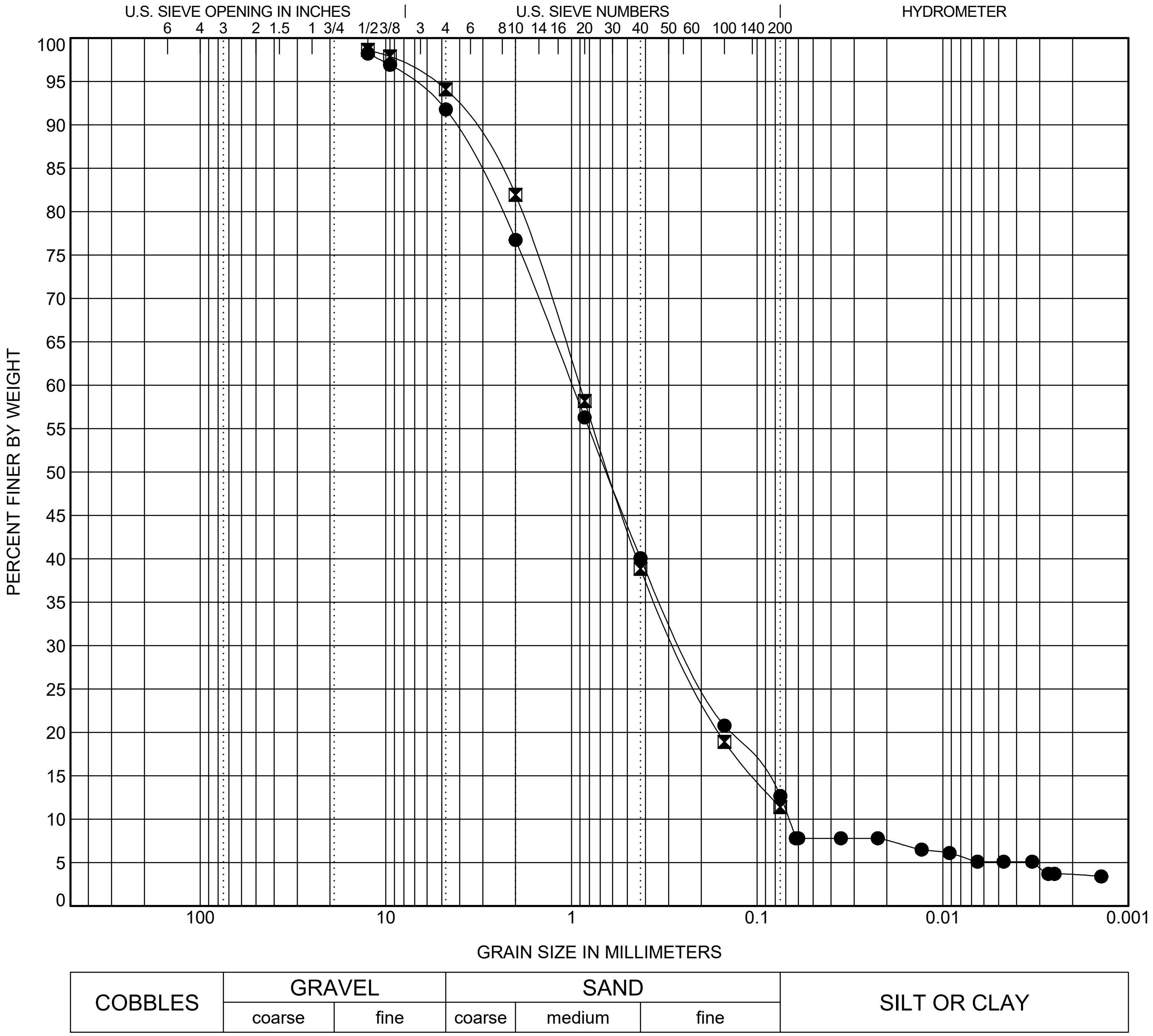
FIGURE NO. B-9

CLIENT RCFCD

PROJECT NAME Wildomar Basin

PROJECT NUMBER R206-025

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



	BOREHOLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-17**	**6.5**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**	**0.91**	**14.71**
☒	**B-18**	**2.5**	**WELL-GRADED SAND with SILT (SW-SM)**	**NP**	**NP**	**NP**	**1.20**	**13.74**

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-17**	**6.5**	**12.5**	**0.993**	**0.247**	**0.067**	**6.4**	**79.1**	**7.6**	**5.1**
☒	**B-18**	**2.5**	**12.5**	**0.907**	**0.268**		**4.5**	**82.7**	**11.4**	

GRADATION CURVES (ASTM D422, ASTM D4318)

Inland Foundation Engineering, Inc.

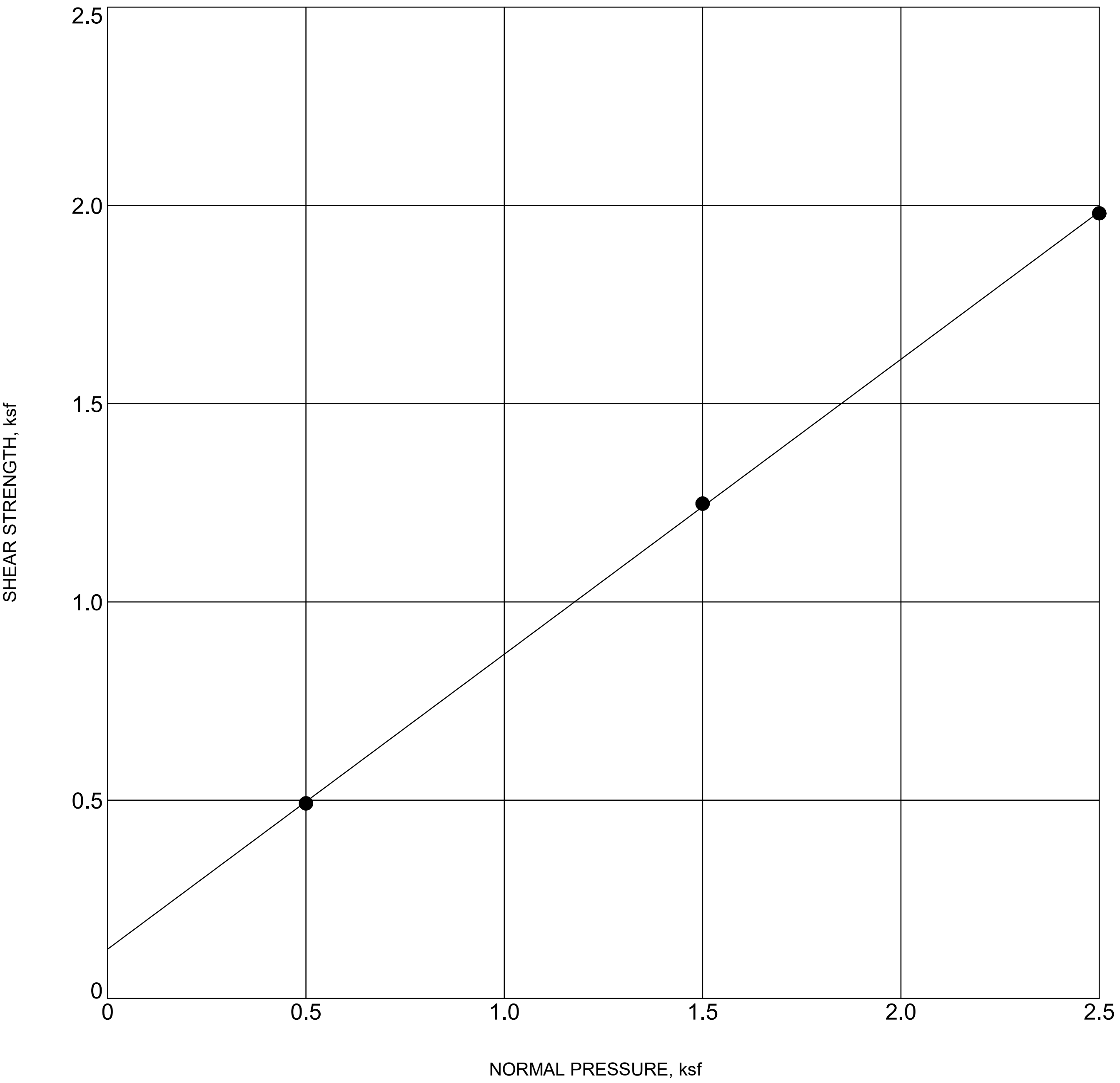
FIGURE NO. B-10

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-02	**7.5**	**SILTY SAND (SM)**	**107**	**15**	**0.1**	**37**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

FIGURE NO. B-11

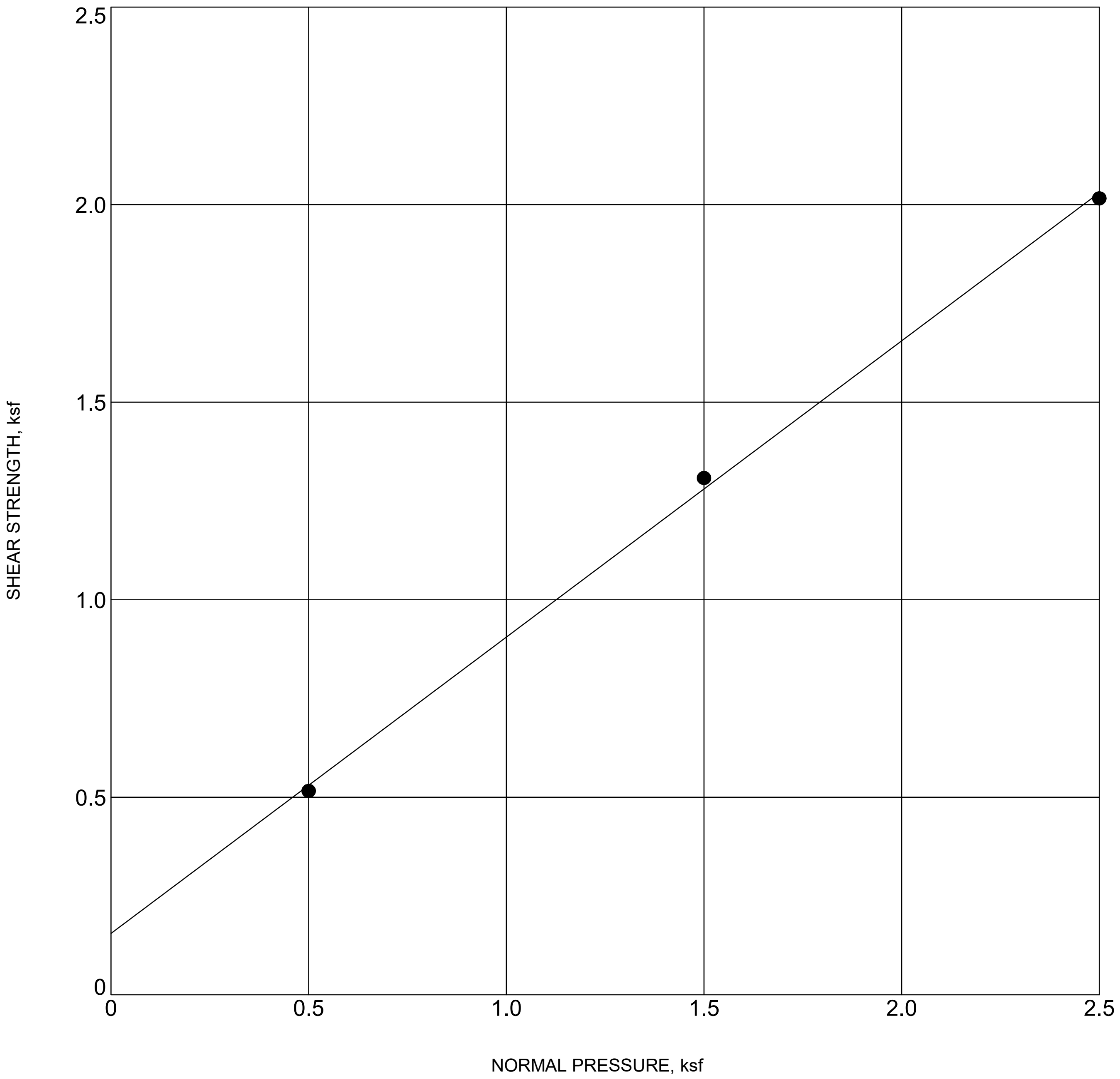
Inland Foundation Engineering, Inc.

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-05	**12.0**	**SILTY SAND (SM)**	**110**	**14**	**0.2**	**37**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

FIGURE NO. B-12

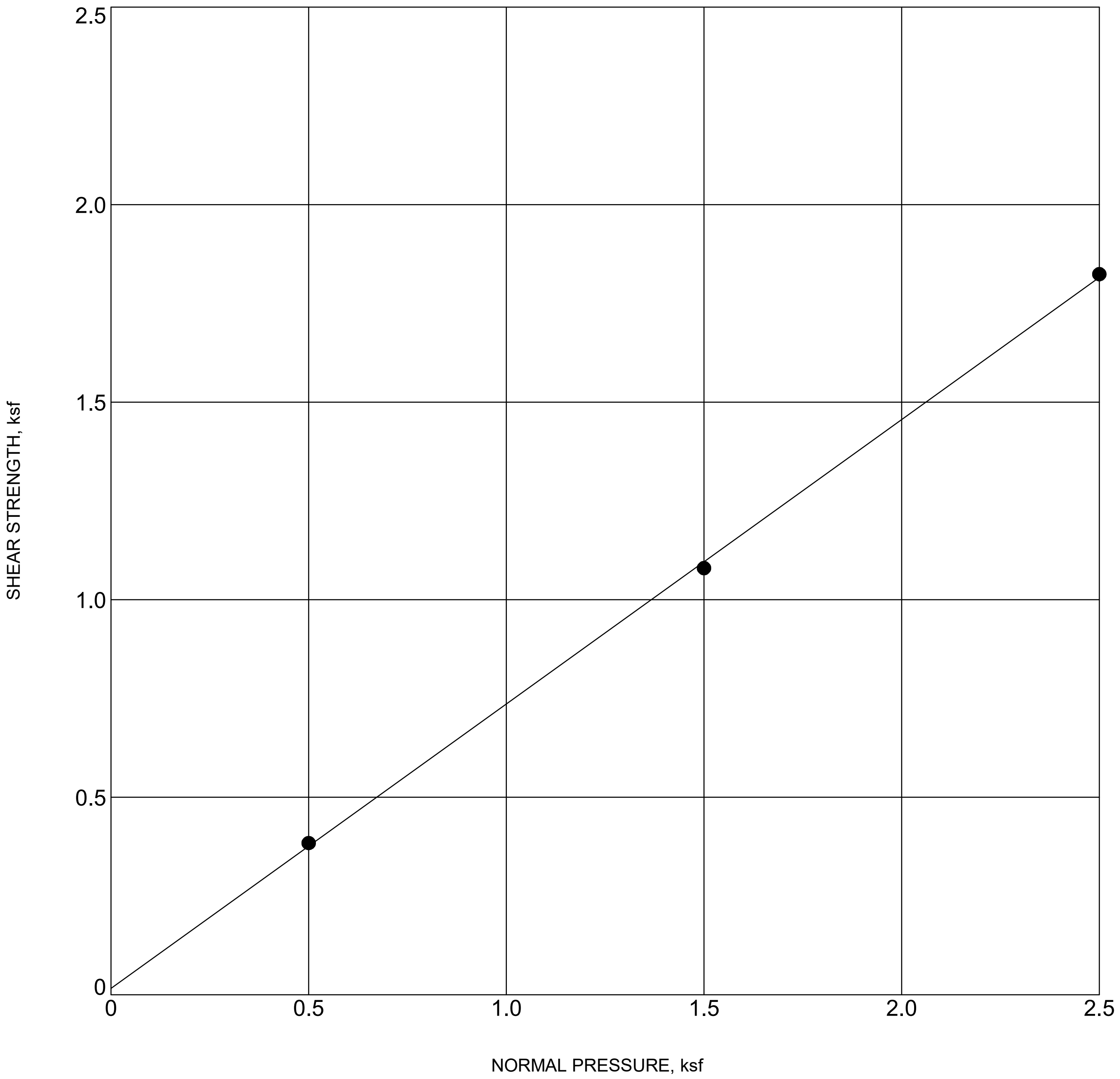
Inland Foundation Engineering, Inc.

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-11	**12.5**	**SILTY SAND (SM)**	**90**	**18**	**0.0**	**36**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

FIGURE NO. B-13

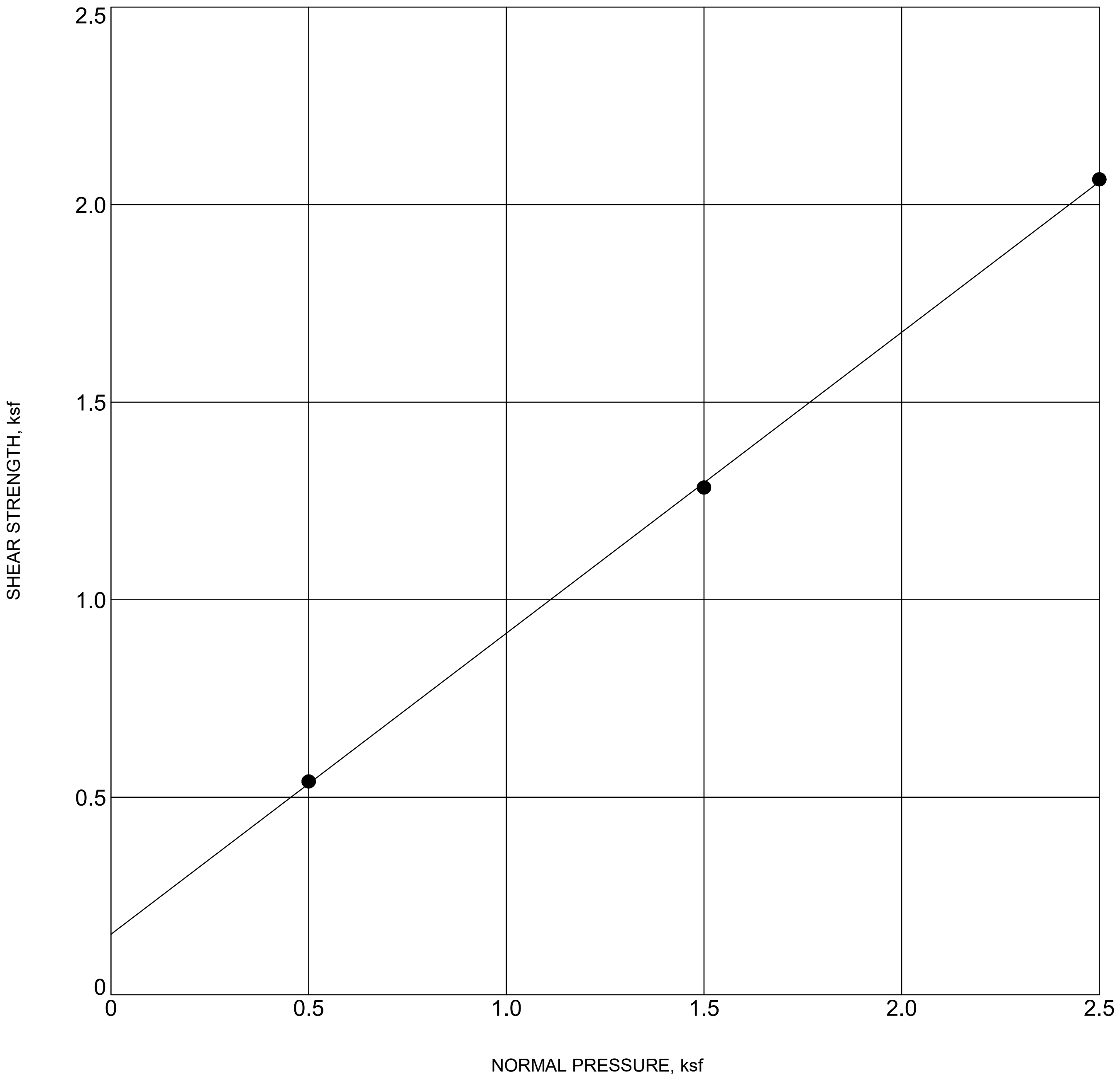
Inland Foundation Engineering, Inc.

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-13	**8.5**	**SILTY SAND (SM)**	**99**	**15**	**0.2**	**37**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

FIGURE NO. B-14

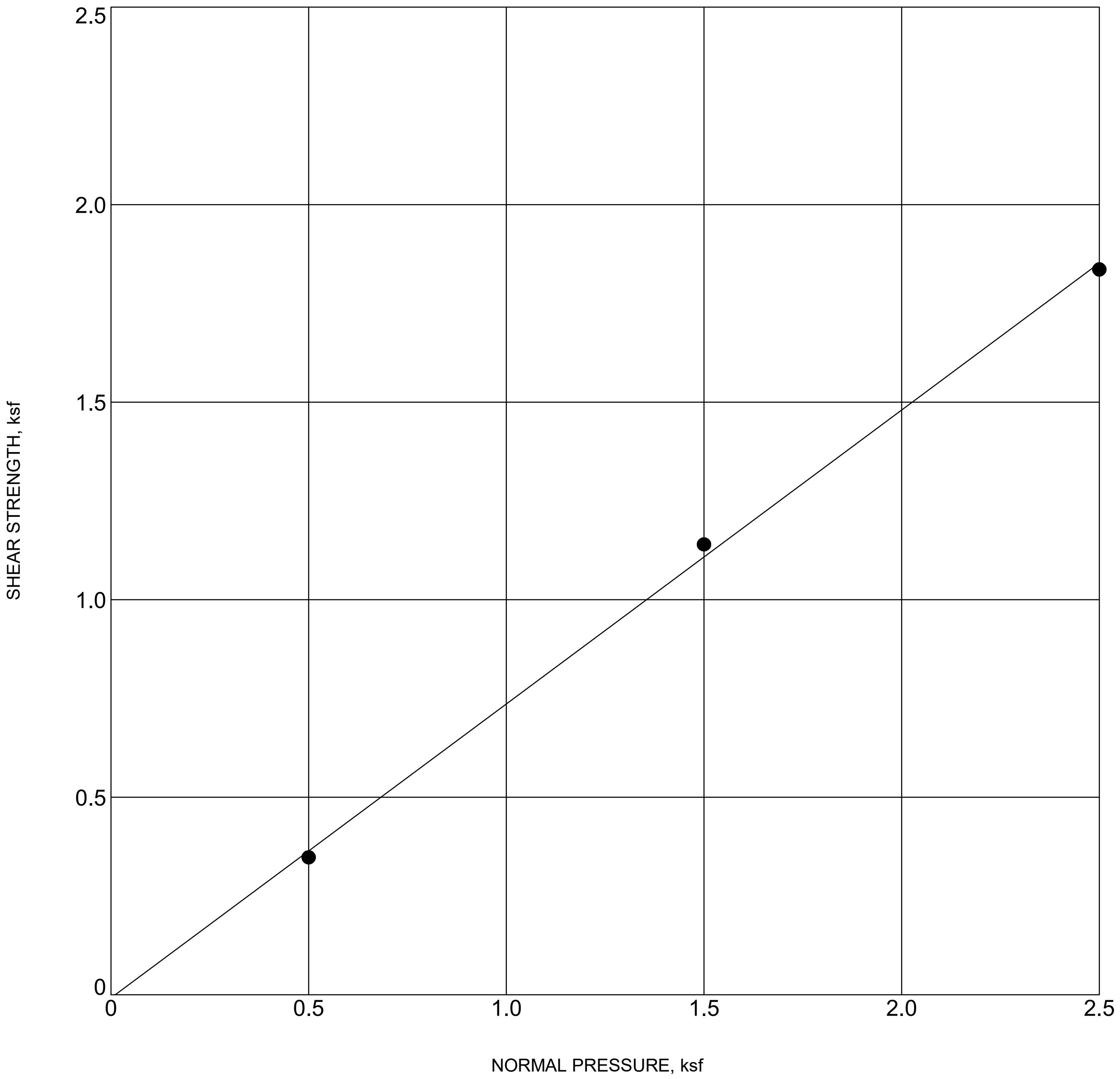
Inland Foundation Engineering, Inc.

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-15	**11.5**	**SILTY SAND (SM)**	**85**	**25**	**0.0**	**36**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

FIGURE NO. B-15

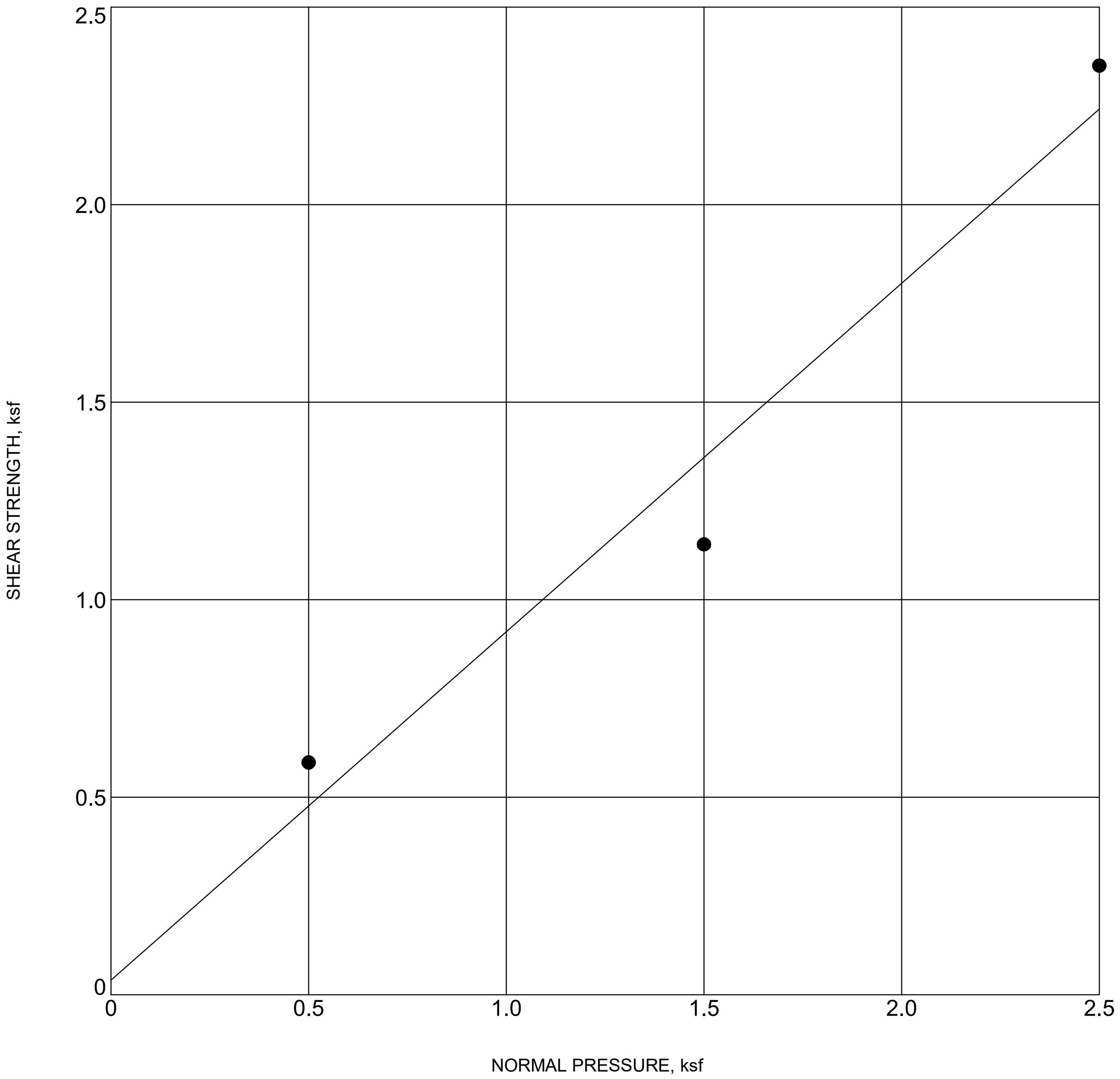
Inland Foundation Engineering, Inc.

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-16	**6.5**	**POORLY-GRADED SAND with SILT (SP-SM)**	**100**	**17**	**0.0**	**41**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

FIGURE NO. B-16

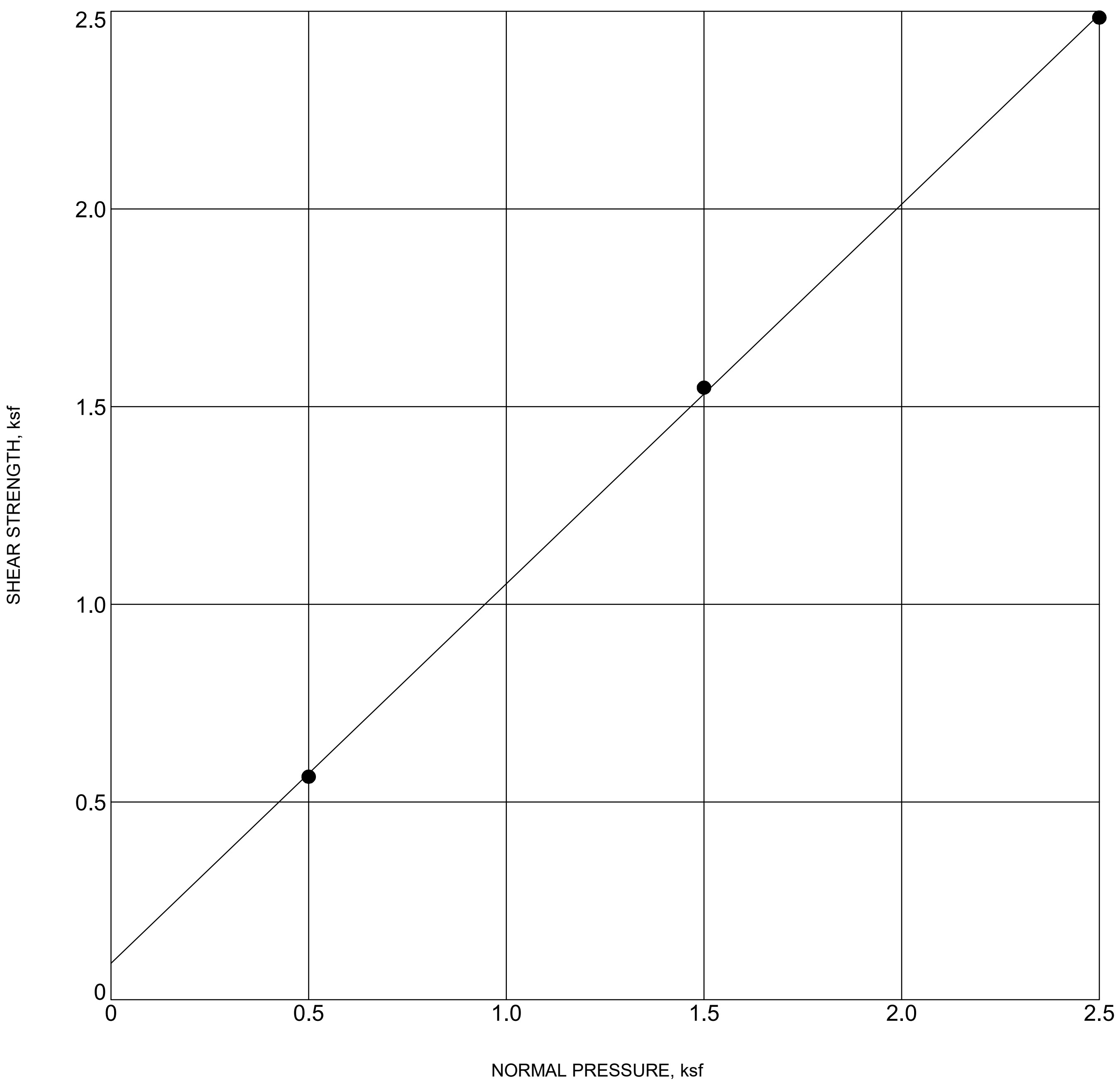
Inland Foundation Engineering, Inc.

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-16	**14.5**	**SILTY SAND (SM)**	**113**	**14**	**0.0**	**44**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

Inland Foundation Engineering, Inc.

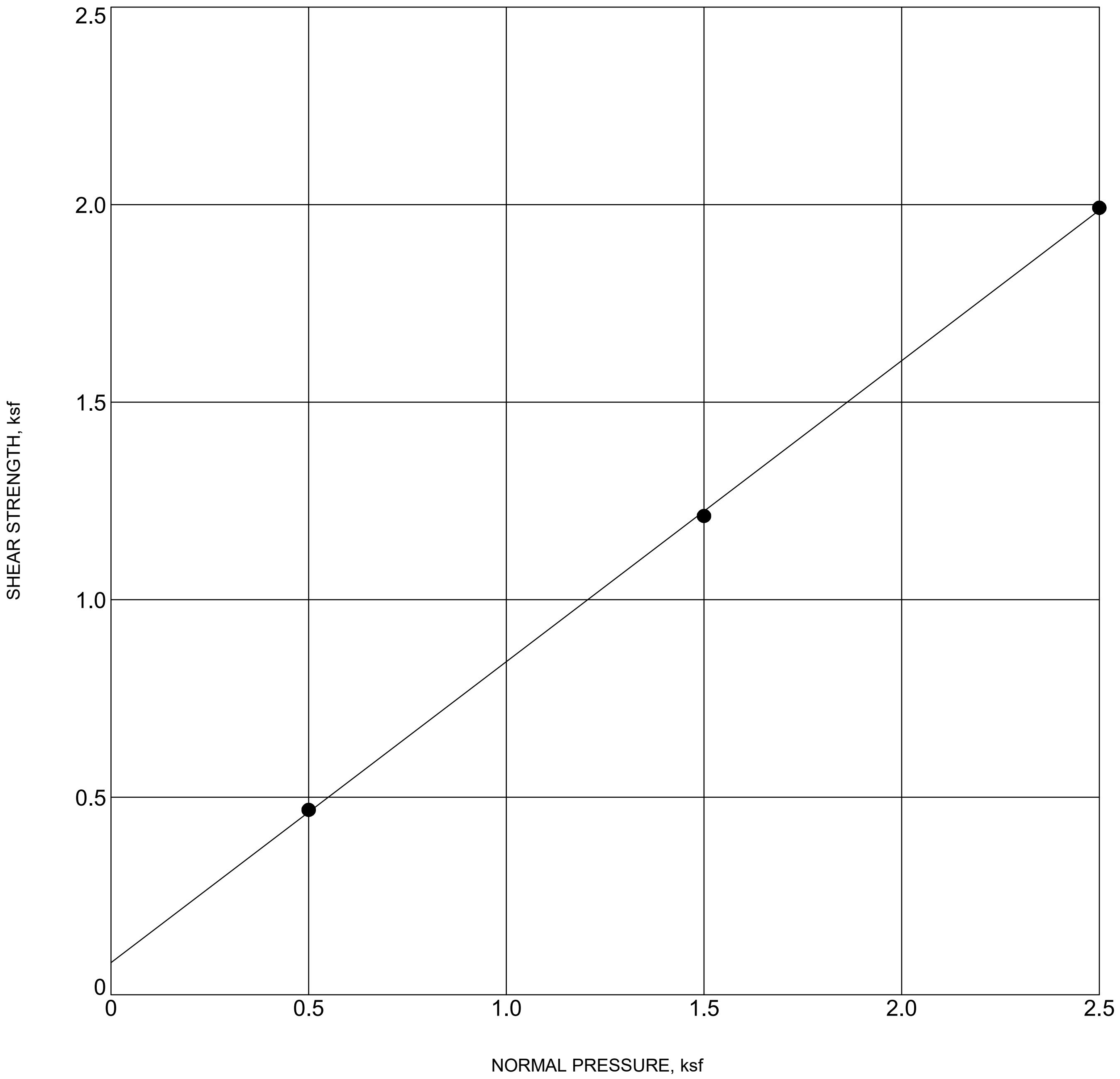
FIGURE NO. B-17

CLIENT RCFCD

PROJECT NAME Wildomar Basin

PROJECT NUMBER R206-025

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-17	**7.5**	**SILTY SAND (SM)**	**94**	**18**	**0.1**	**37**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

Inland Foundation Engineering, Inc.

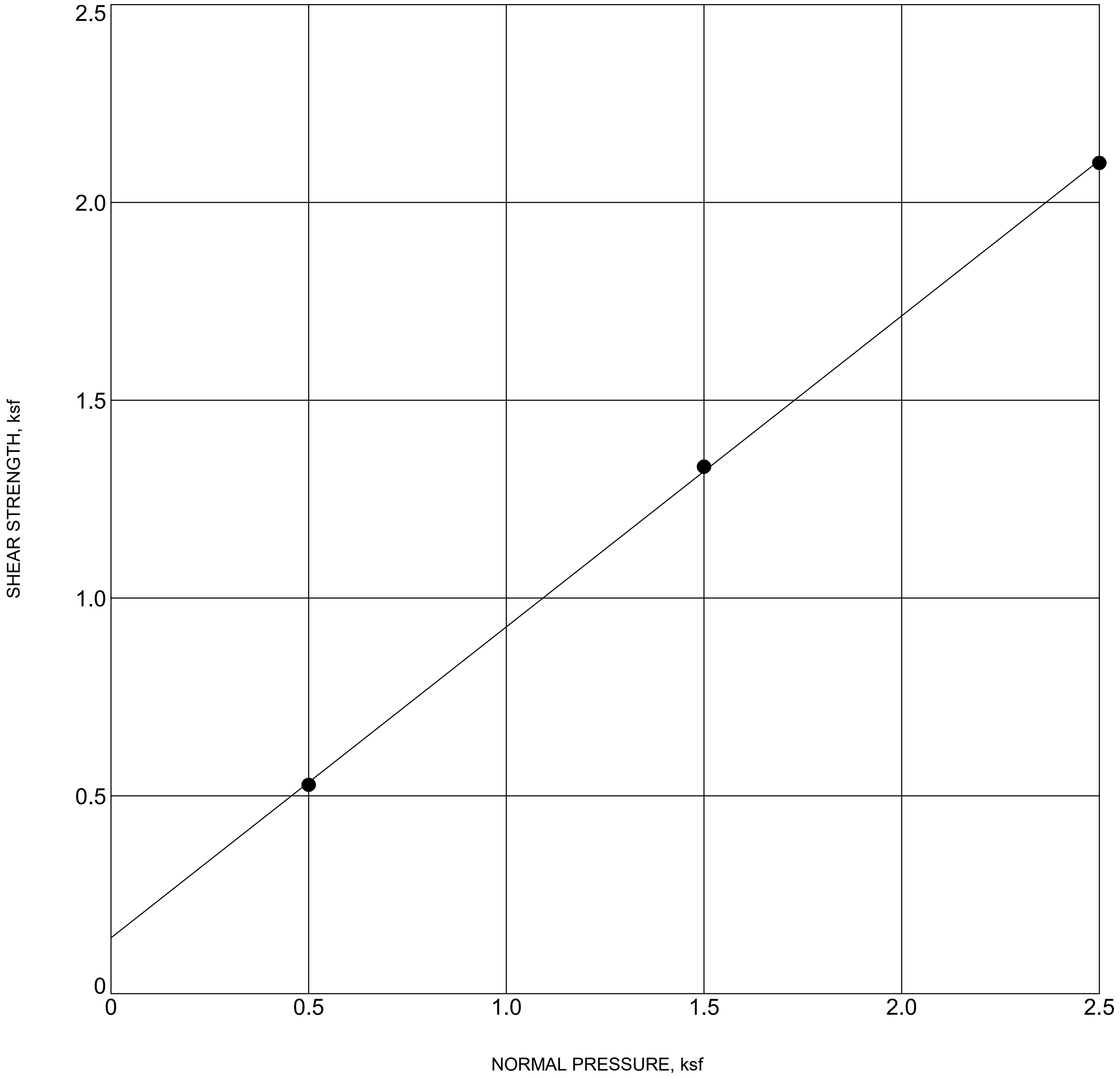
FIGURE NO. B-18

CLIENT RCFCD

PROJECT NAME Wildomar Basin

PROJECT NUMBER R206-025

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-18	**4.5**	**SILTY SAND (SM)**			**0.1**	**38**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

Inland Foundation Engineering, Inc.

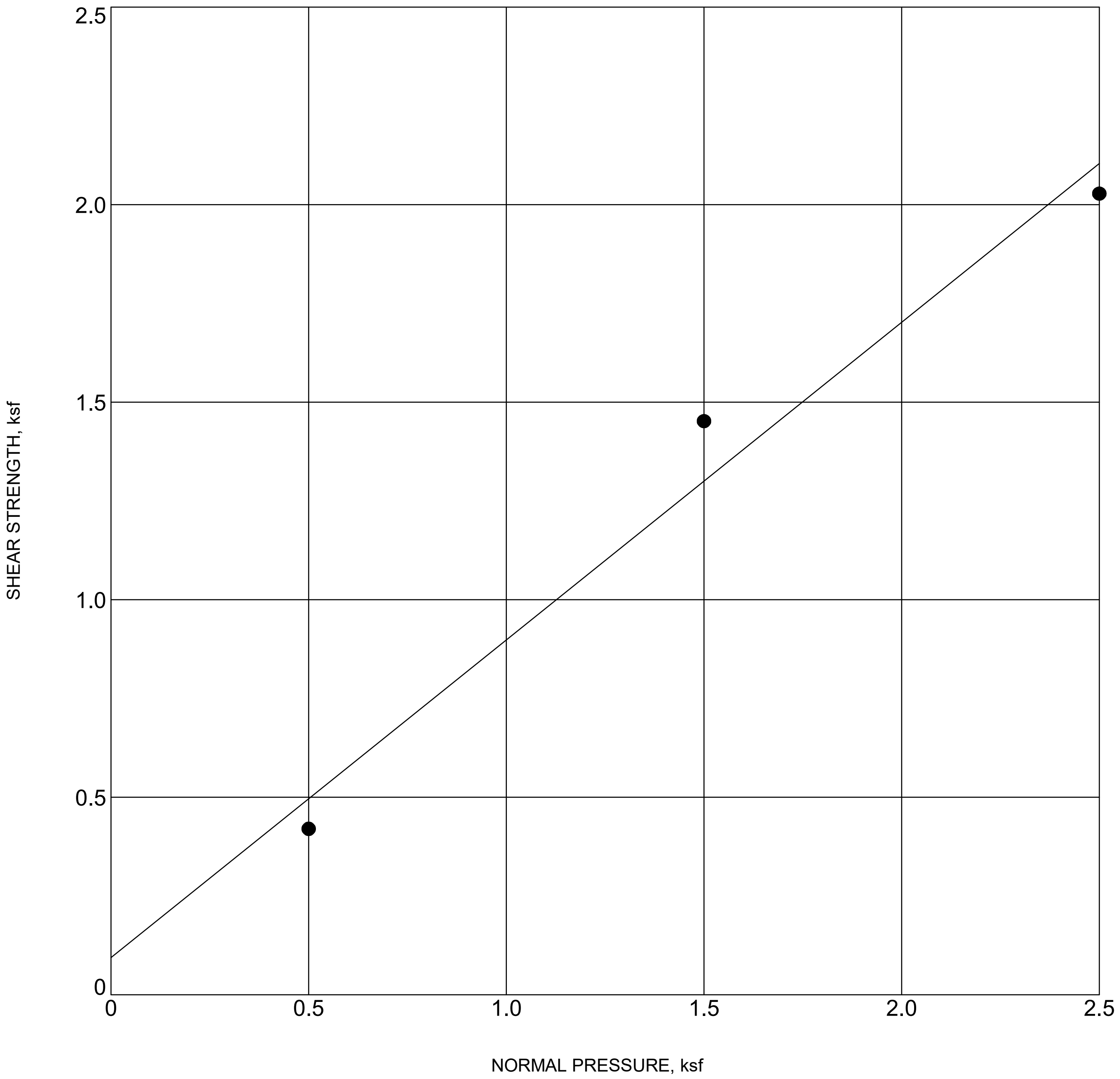
FIGURE NO. B-19

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%	c	ϕ		
B-18	**8.5**	**GRAVELLY SAND (SP-SM)**	**95**	**18**	**0.1**	**39**	Residual	●
							Peak	

DIRECT SHEAR TEST (ASTM D3080)

FIGURE NO. B-20

Inland Foundation Engineering, Inc.

CLIENT RCFCD

PROJECT NUMBER R206-025

PROJECT NAME Wildomar Basin

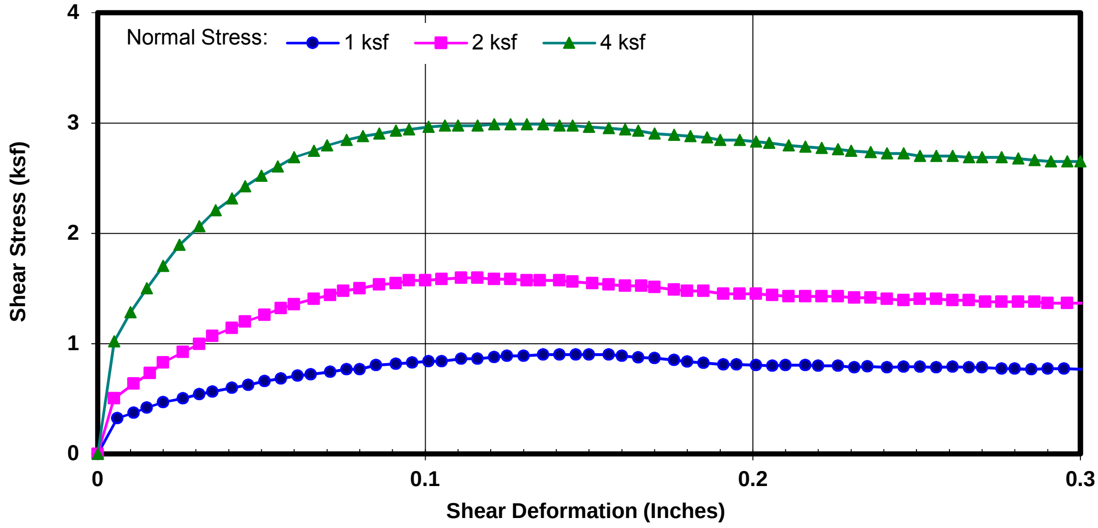
PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA

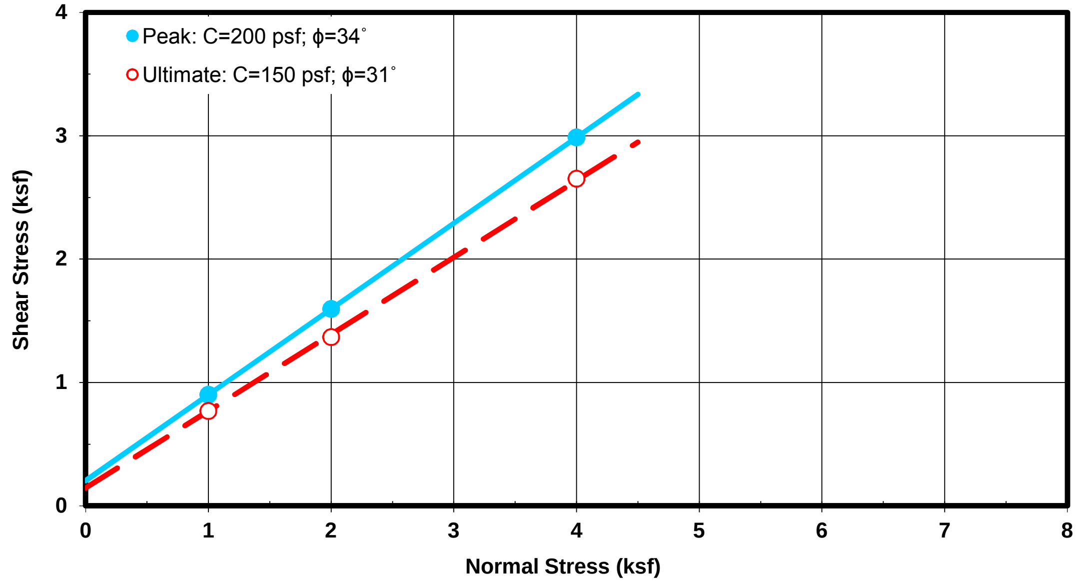
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Inland Foundation Engineering
Project Name: RCFCD
Project No.: R206-025
Boring No.: B-11
Sample No.: - **Depth (ft):** 16.5-17.5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/04/20
Computed By: NR **Date:** 09/08/20
Checked by: AP **Date:** 09/08/20

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)
113.8	111.3	2.2	17.3	12	91	1	0.900	0.768
						2	1.596	1.368
						4	2.988	2.652





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DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client:	Inland Foundation Engineering	**Tested By:**	ST	**Date:**	09/03/20
Project Name:	RCFCD	**Computed By:**	NR	**Date:**	09/08/20
Project No.:	R206-025	**Checked by:**	AP	**Date:**	09/08/20

Boring No.: B-21

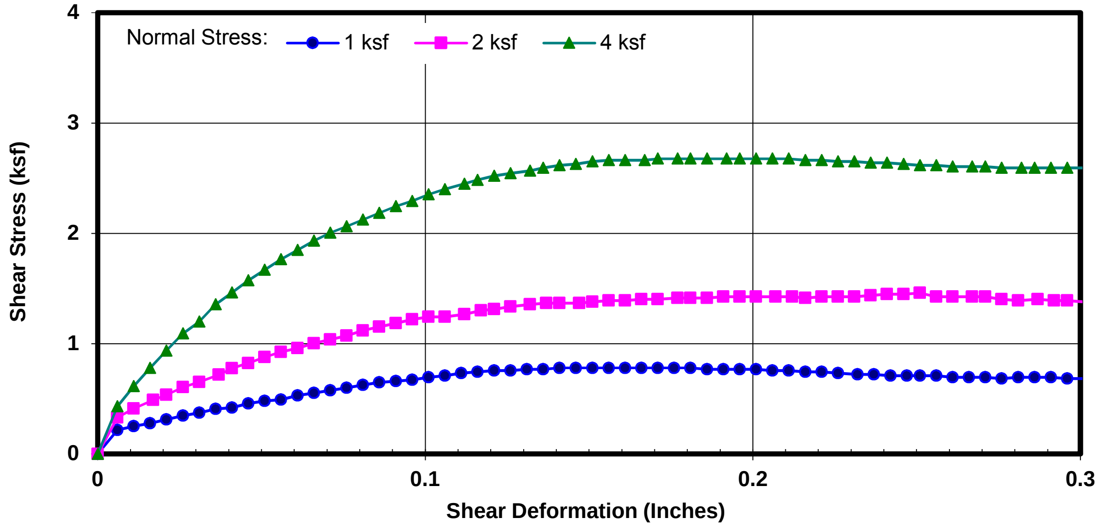
Sample No.: - **Depth (ft):** 6.5-7.5

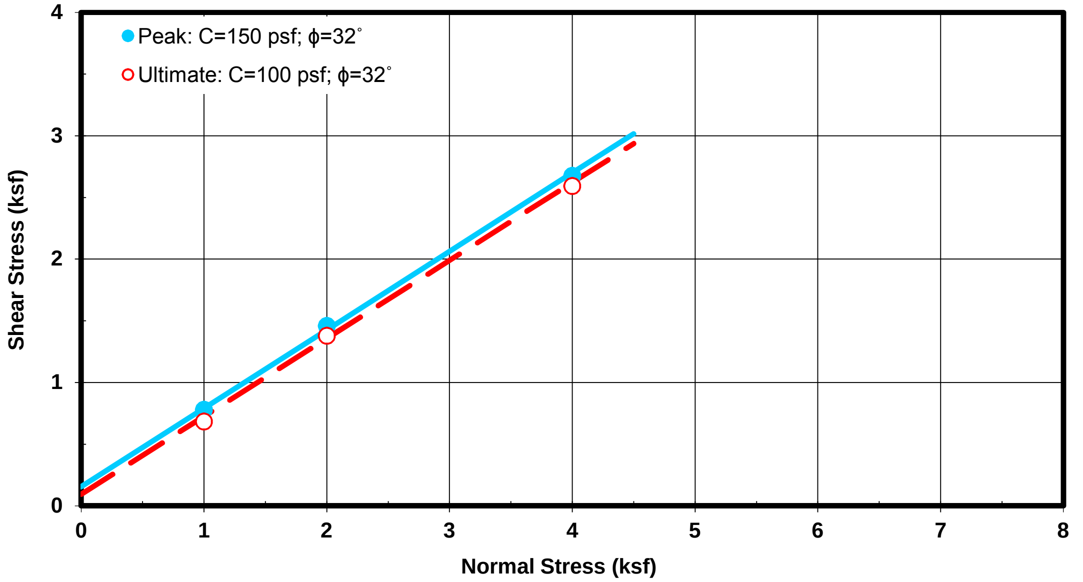
Sample Type: Mod. Cal.

Soil Description: Silty 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)
110.6	108.4	2.1	19.1	10	93	1	0.780	0.684
						2	1.459	1.379
						4	2.676	2.592





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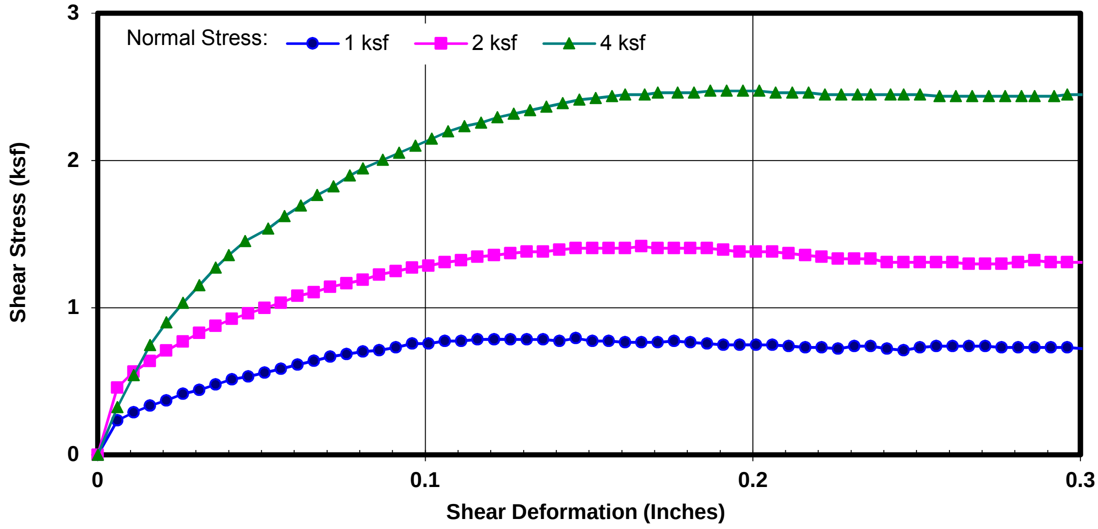
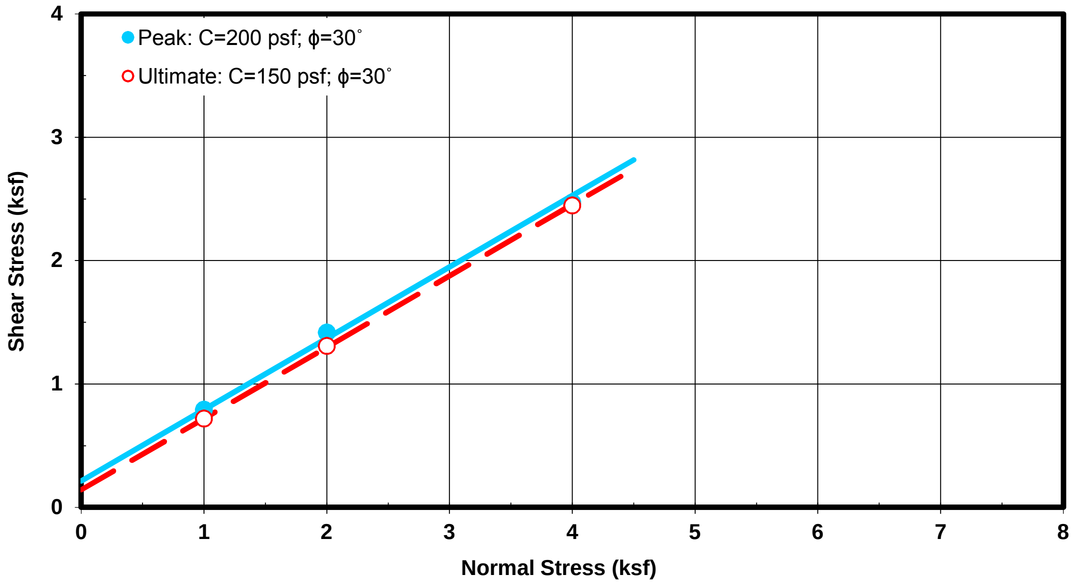
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Inland Foundation Engineering
Project Name: RCFCD
Project No.: R206-025
Boring No.: B-21
Sample No.: - **Depth (ft):** 14.5-15.5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/03/20
Computed By: NR **Date:** 09/08/20
Checked by: AP **Date:** 09/08/20

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)
109.3	106.0	3.1	19.6	14	90	1	0.792	0.720
						2	1.416	1.308
						4	2.472	2.448





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THREE-STAGED
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT
Test Procedure: ASTM D 4767

Project Name:	RCFCD Wildomar	Tested by:	ST	Date:	10-05-20
Project No.:	R206-025	Input Data by:	JP	Date:	10-08-20
Boring No.:	B-8	Reviewed by:	AP	Date:	10-08-20
Sample No.:	-	Sample Description:	Sandy Silt w/gravel		
Depth(ft):	14.5-15				
Sample Type:	Mod Cal	Confining Pressure = 13.9 psi			

Initial Sample Diameter (in)	2.415	2.415	2.415	Avg. =	2.415
Initial Sample Height (in)	5.000	5.000	5.000	Avg. =	5.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	4.581	4.624
Moisture Content (%)	1.77	
Wet Weight (gms)	186.96	
Dry Weight (gms)	184.60	
Container Weight (gms)	50.95	
Density and Saturation		
Wet Weight (gms)	692.38	
Container Weight (gms)	0.00	
Wet Density (pcf)	115.2	
Dry Density (pcf)	113.2	
Initial Void Ratio	0.489	
Initial % Saturation	9.8	

Assumed Specific Gravity = 2.70

Back Pressure Saturation			
B Value (%) =	96	Change in Ht. of the Specimen (in)=	0

Consolidation			
Cell Pressure (psi) =	43.9	Initial Burette Ht.(cm)=	73.5
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	69.3
Eff. Consol. Stress (psi) =	13.9	Final Height (in)=	4.898
Initial Sample Height (in) =	5.000	Initial Volume (cu.in)=	22.903
Change in Height due to Consolidation (in) =	0.1023	Final Volume (cu.in) =	22.647

Shear		At Failure	
Rate of Deformation (in/min)=	0.004	Deviator Stress (ksf) =	3.41
Time to 50% primary Consolidation (min) =	15	Eff. Minor Principal stress (ksf) =	0.93
Failure Mode: Shear Failure		Eff. Major Principal stress (ksf) =	4.34
Deformation After Shearing (in) =	0.229	Axial Strain (%) =	5.00

THREE-STAGED

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	RCFCD Wildomar	Tested by:	ST	Date:	10-05-20
Project No.:	R206-025	Input Data by:	JP	Date:	10-08-20
Boring No.:	B-8	Reviewed by:	AP	Date:	10-08-20
Sample No.:	-	Sample Description:	Sandy Silt w/gravel		
Depth(ft):	14.5-15				
Sample Type:	Mod Cal	Confining Pressure = 27.8 psi			

Initial Sample Diameter (in)	2.415	2.415	2.415	Avg. =	2.415
Initial Sample Height (in)	5.000	5.000	5.000	Avg. =	5.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	4.581	4.749
Moisture Content (%)	1.77	
Wet Weight (gms)	186.96	
Dry Weight (gms)	184.60	
Container Weight (gms)	50.95	
Density and Saturation		
Wet Weight (gms)	692.38	
Container Weight (gms)	0.00	
Wet Density (pcf)	115.2	
Dry Density (pcf)	113.2	
Initial Void Ratio	0.489	
% Saturation	9.8	

Assumed Specific Gravity = 2.70

Back Pressure Saturation

B Value (%) =	96	Change in Ht. of the Specimen (in)=	0

Consolidation

Cell Pressure (psi) =	57.8	Initial Burette Ht.(cm)=	69.3
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	61.5
Eff. Consol. Stress (psi) =	27.8	Final Height (in)=	4.668
Height of Sample after 1st Shearing (in) =	4.668	Initial Volume (cu.in)=	22.903
Change in Height due to Consolidation (in) =	0.0000	Final Volume (cu.in) =	22.171

Shear		At Failure	
Rate of Deformation (in/min)=	0.004	Deviator Stress (ksf) =	8.25
Time to 50% primary Consolidation =	15	Eff. Minor Principal stress (ksf) =	2.67
Failure Mode: Shear Failure		Eff. Major Principal stress (ksf) =	10.92
Deformation After Shearing (in) =	0.226	Axial Strain (%) =	5.00

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THREE-STAGED

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Test Procedure: ASTM D 4767

Project Name:	RCFCD Wildomar	Tested by:	ST	Date:	10-05-20
Project No.:	R206-025	Input Data by:	JP	Date:	10-08-20
Boring No.:	B-8	Reviewed by:	AP	Date:	10-08-20
Sample No.:	-	Sample Description:	Sandy Silt w/gravel		
Depth(ft):	14.5-15				
Sample Type:	Mod Cal	Confining Pressure = 55.6 psi			

Initial Sample Diameter (in)	2.415	2.415	2.415	Avg. =	2.415
Initial Sample Height (in)	5.000	5.000	5.000	Avg. =	5.000

	BEFORE CONSOLIDATION	AFTER CONSOLIDATION
Area (in²)	4.581	4.656
Moisture Content (%)	1.77	16.67
Wet Weight (gms)	186.96	907.98
Dry Weight (gms)	184.60	799.19
Container Weight (gms)	50.95	146.41
Density and Saturation		
Wet Weight (gms)	692.38	
Container Weight (gms)	0.00	
Wet Density (pcf)	115.2	
Dry Density (pcf)	113.2	
Initial Void Ratio	0.489	
% Saturation	9.8	

Assumed Specific Gravity = 2.70

Back Pressure Saturation

B Value (%) =	96	Change in Ht. of the Specimen (in)=	0

Consolidation

Cell Pressure (psi) =	85.6	Initial Burette Ht.(cm)=	61.5
Back Pressure(psi) =	30.0	Final Burette Ht.(cm)=	54.4
Eff. Consol. Stress (psi) =	55.6	Final Height (in)=	4.668
Height of Sample after 2nd Shearing (in) =	4.668	Initial Volume (cu.in)=	22.903
Change in Height due to Consolidation (in) =	0.0000	Final Volume (cu.in) =	21.738

Shear		At Failure	
Rate of Deformation (in/min)=	0.004	Deviator Stress (ksf) =	17.83
Time to 50% primary Consolidation =	15	Eff. Minor Principal stress (ksf) =	5.96
Failure Mode: Shear Failure		Eff. Major Principal stress (ksf) =	23.79
		Axial Strain (%) =	5.00

THREE-STAGED
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Project Name:	RCFCD Wildomar	Cell Pressure:	43.9	psi
Project No:	R206-025	Back Pressure :	30.0	psi
Boring No.:	B-8	Consolidation Pressure :	13.9	psi
Sample No.:	-	Initial Sample Height:	5.000	in
Depth(ft.):	14.5-15	Initial Area of Sample:	4.581	sq. in.
Sample Type:	Mod Cal	Final Sample Ht.* (L):	4.898	in
Sample Description:	Sandy Silt w/gravel	Final Sample Area (A)*:	4.624	sq. in.

Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure (psi)	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
43.9	0	0.000	30.0	0.00	0.00	0.00	0.00	2.00
43.9	25	0.005	32.6	0.78	0.10	0.38	0.39	2.01
43.9	51	0.012	34.5	1.58	0.25	0.65	0.79	2.15
43.9	62	0.025	36.3	1.91	0.51	0.91	0.95	2.05
43.9	65	0.039	37.2	2.01	0.80	1.04	1.01	1.97
43.9	67	0.053	37.8	2.08	1.08	1.12	1.04	1.92
43.9	70	0.066	38.0	2.15	1.35	1.16	1.08	1.92
43.9	72	0.079	38.2	2.21	1.61	1.18	1.11	1.93
43.9	75	0.093	38.3	2.30	1.90	1.19	1.15	1.96
43.9	79	0.106	38.3	2.40	2.17	1.20	1.20	2.00
43.9	82	0.120	38.3	2.49	2.44	1.20	1.24	2.05
43.9	85	0.133	38.3	2.59	2.72	1.19	1.29	2.10
43.9	89	0.147	38.2	2.68	3.00	1.19	1.34	2.16
43.9	93	0.161	38.2	2.79	3.29	1.18	1.39	2.22
43.9	96	0.174	38.1	2.88	3.55	1.16	1.44	2.28
43.9	99	0.188	38.0	2.96	3.84	1.15	1.48	2.33
43.9	102	0.202	37.9	3.06	4.12	1.13	1.53	2.40
43.9	106	0.215	37.8	3.17	4.39	1.12	1.58	2.47
43.9	110	0.228	37.6	3.27	4.66	1.10	1.63	2.54
43.9	114	0.241	37.5	3.38	4.93	1.08	1.69	2.61
43.9	117	0.250	37.4	3.46	5.10	1.06	1.73	2.67

THREE-STAGED
CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

Project Name:	RCFCD Wildomar	Cell Pressure:	57.8	psi
Project No:	R206-025	Back Pressure :	30.0	psi
Boring No.:	B-8	Consolidation Pressure :	27.8	psi
Sample No.:	-	Initial Sample Height:	5.000	in
Depth(ft.):	14.5-15	Initial Area of Sample:	4.581	sq. in.
Sample Type:	Mod Cal	Final Sample Ht.* (L):	4.668	in
Sample Description:	Sandy Silt w/gravel	Final Sample Area (A)*:	4.749	sq. in.

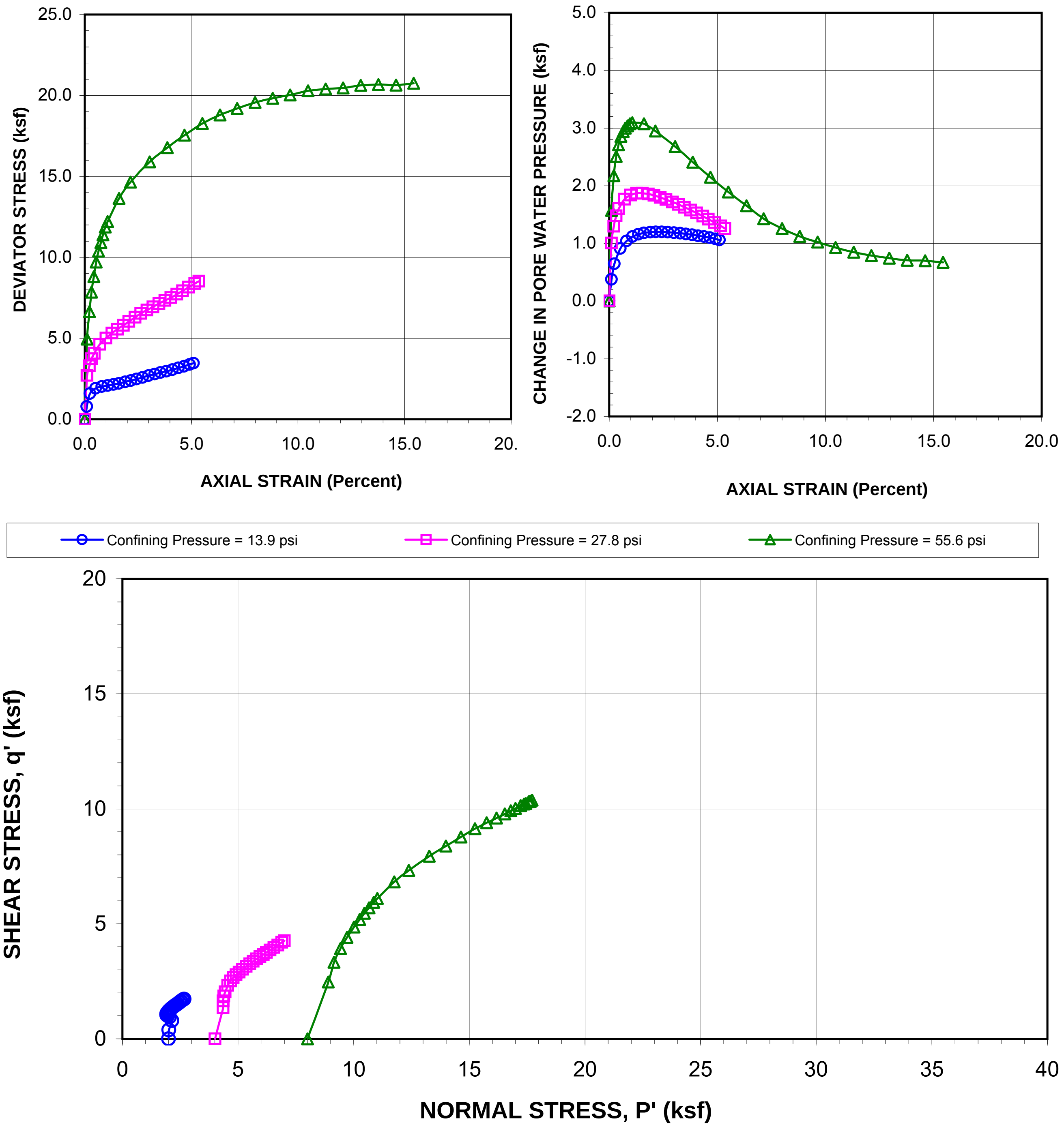
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure (psi)	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
57.8	0	0.000	30.0	0.00	0.00	0.00	0.00	4.00
57.8	89	0.005	37.0	2.70	0.11	1.00	1.35	4.35
57.8	109	0.010	39.0	3.30	0.21	1.30	1.65	4.36
57.8	124	0.015	40.3	3.73	0.32	1.48	1.87	4.39
57.8	135	0.021	41.1	4.08	0.45	1.60	2.04	4.44
57.8	154	0.033	42.3	4.64	0.71	1.76	2.32	4.56
57.8	167	0.047	42.8	5.02	1.00	1.84	2.51	4.68
57.8	178	0.060	43.0	5.32	1.28	1.87	2.66	4.79
57.8	186	0.072	43.0	5.56	1.53	1.87	2.78	4.91
57.8	195	0.085	42.9	5.81	1.82	1.86	2.90	5.05
57.8	204	0.097	42.7	6.05	2.08	1.83	3.02	5.19
57.8	213	0.110	42.5	6.29	2.36	1.80	3.15	5.35
57.8	221	0.123	42.2	6.53	2.63	1.76	3.27	5.51
57.8	229	0.137	41.9	6.75	2.92	1.72	3.37	5.66
57.8	237	0.149	41.6	6.94	3.20	1.67	3.47	5.80
57.8	244	0.162	41.3	7.14	3.48	1.63	3.57	5.95
57.8	251	0.176	41.0	7.33	3.77	1.58	3.67	6.09
57.8	258	0.189	40.6	7.52	4.04	1.53	3.76	6.24
57.8	266	0.202	40.2	7.72	4.33	1.47	3.86	6.39
57.8	274	0.214	39.9	7.94	4.59	1.42	3.97	6.55
57.8	283	0.228	39.5	8.15	4.88	1.36	4.08	6.72
57.8	292	0.241	39.1	8.39	5.16	1.30	4.19	6.89
57.8	297	0.250	38.7	8.52	5.36	1.26	4.26	7.01

THREE-STAGED

CONSOLIDATED UNDRAINED TRIAXIAL TEST WITH PORE PRESSURE MEASUREMENT

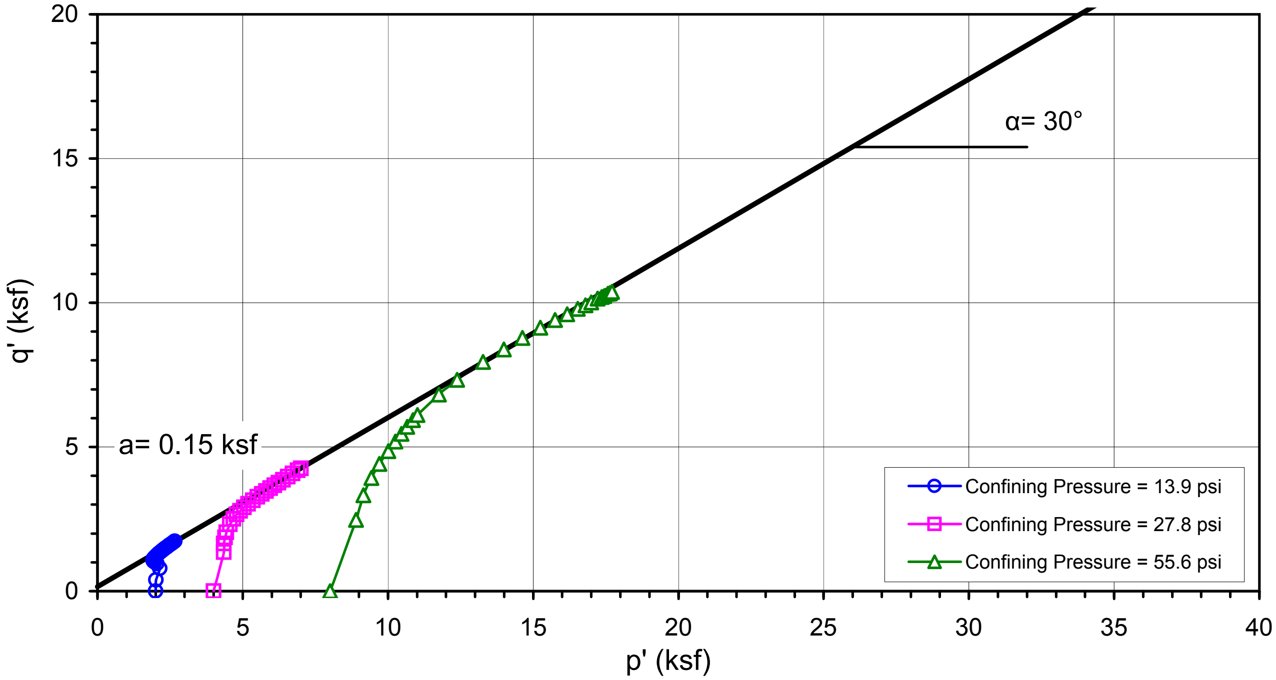
Project Name:	RCFCD Wildomar	Cell Pressure:	85.6	psi
Project No:	R206-025	Back Pressure :	30.0	psi
Boring No.:	B-8	Consolidation Pressure :	55.6	psi
Sample No.:	-	Initial Sample Height:	5.000	in
Depth(ft.):	14.5-15	Initial Area of Sample:	4.581	sq. in.
Sample Type:	Mod Cal	Final Sample Ht.* (L):	4.668	in
Sample Description:	Sandy Silt w/gravel	Final Sample Area (A)*:	4.656	sq. in.

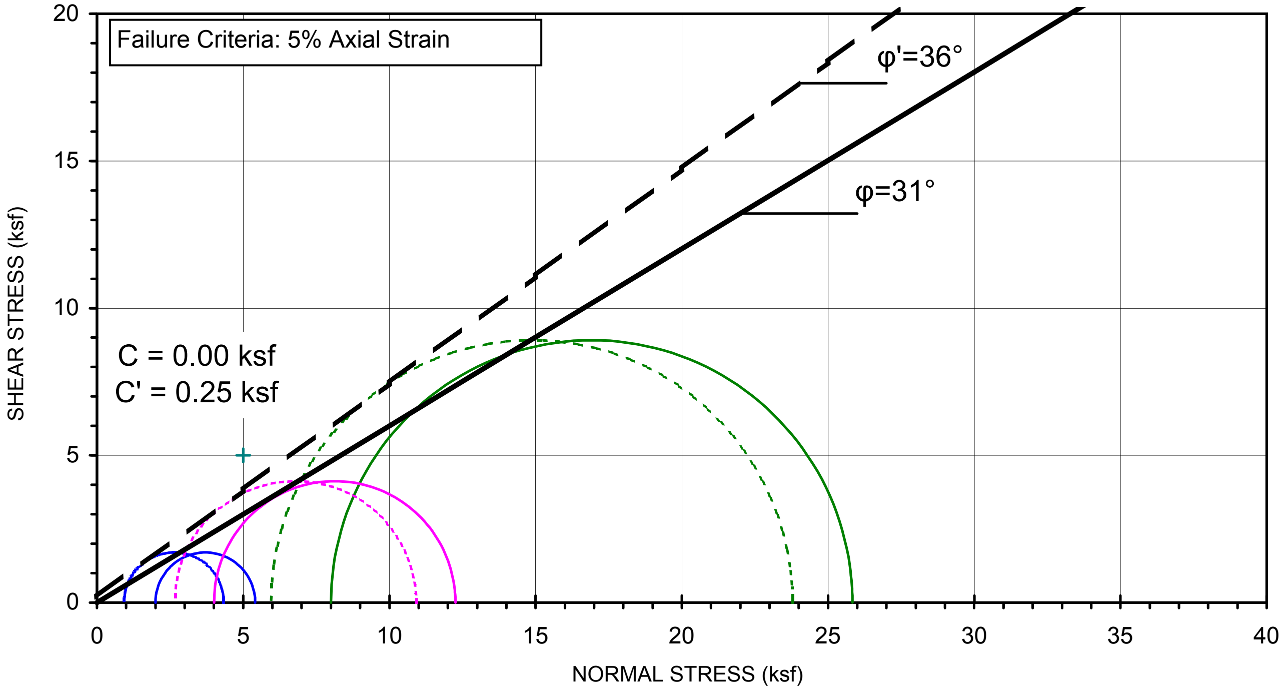
Cell Pressure (psi)	Load (lbs)	Axial Deformation (in)	Back Pressure (psi)	Deviator Stress (S1-S3) (ksf)	Axial Strain (%)	Pore Pressure Change (ksf)	Shear Stress q' (S1-S3)/2 (ksf)	Normal Stress p' (S1'+S3')/2 (ksf)
85.6	0	0.000	30.0	0.00	0.00	0.00	0.00	8.01
85.6	160	0.005	40.9	4.94	0.11	1.57	2.47	8.90
85.6	215	0.010	45.1	6.65	0.21	2.17	3.32	9.15
85.6	255	0.015	47.4	7.86	0.32	2.51	3.93	9.43
85.6	286	0.020	48.8	8.82	0.43	2.71	4.41	9.71
85.6	316	0.025	49.8	9.72	0.54	2.85	4.86	10.01
85.6	338	0.030	50.4	10.38	0.64	2.94	5.19	10.26
85.6	355	0.035	50.8	10.91	0.75	3.00	5.46	10.47
85.6	372	0.040	51.1	11.39	0.86	3.04	5.70	10.66
85.6	388	0.045	51.4	11.87	0.96	3.08	5.94	10.87
85.6	399	0.050	51.5	12.22	1.07	3.09	6.11	11.02
85.6	448	0.075	51.3	13.64	1.61	3.07	6.82	11.76
85.6	484	0.100	50.5	14.65	2.14	2.95	7.32	12.38
85.6	530	0.142	48.6	15.89	3.05	2.68	7.95	13.27
85.6	564	0.181	46.7	16.77	3.87	2.41	8.39	13.99
85.6	595	0.218	44.9	17.55	4.68	2.15	8.78	14.63
85.6	625	0.258	43.1	18.27	5.52	1.89	9.14	15.25
85.6	649	0.296	41.5	18.80	6.35	1.65	9.40	15.75
85.6	669	0.334	39.9	19.20	7.15	1.43	9.60	16.18
85.6	688	0.373	38.7	19.57	7.99	1.26	9.79	16.53
85.6	703	0.412	37.8	19.83	8.82	1.12	9.91	16.80
85.6	717	0.450	37.1	20.03	9.64	1.02	10.01	17.00
85.6	733	0.489	36.5	20.29	10.47	0.93	10.14	17.22
85.6	744	0.528	35.9	20.41	11.31	0.85	10.20	17.36
85.6	753	0.566	35.5	20.47	12.12	0.79	10.23	17.45
85.6	766	0.605	35.2	20.62	12.95	0.74	10.31	17.57
85.6	775	0.643	34.9	20.67	13.78	0.71	10.34	17.63
85.6	782	0.682	34.9	20.65	14.60	0.70	10.32	17.63
85.6	794	0.721	34.7	20.75	15.44	0.67	10.38	17.71



Project Name:	RCFCD Wildomar	Sample Type:	Mod Cal
Project No.:	R206-025	Sample Description:	Sandy Silt w/gravel
Boring No.:	B-8	Dry Unit Weight (pcf):	113.2
Sample No.:	-	Initial Moisture Content (%):	1.8
Depth (ft):	14.5-15	Confining Pressure:	13.9, 27.8, 55.6 psi

CU TRIAXIAL THREE-STAGED TEST WITH PORE PRESSURE MEASUREMENT
ASTM D 4767



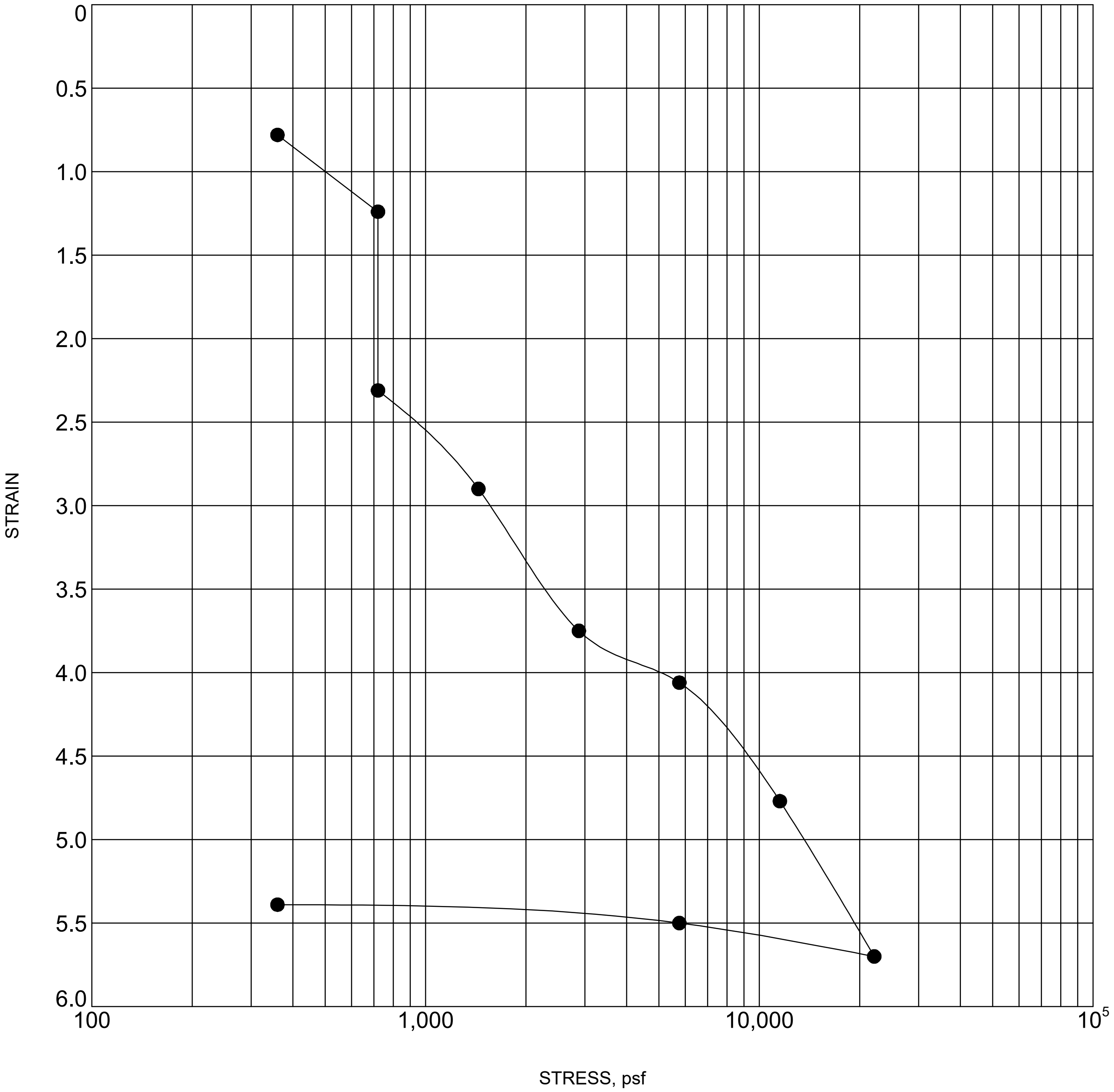


Project Name:	RCFCD Wildomar	Sample Type:	Mod Cal
Project No.:	R206-025	Sample Description:	Sandy Silt w/gravel
Boring No.:	B-8	Dry Unit Weight (pcf):	113.2
Sample No.:	-	Initial Moisture Content (%):	1.8
Depth (ft):	14.5-15	Confining Pressure:	13.9, 27.8, 55.6 psi

CU TRIAXIAL THREE-STAGED TEST WITH PORE PRESSURE MEASUREMENT

ASTM D 4767

B24h



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-03**	**7.5**	**SAND with SILT (SP-SM)**		**17**

CONSOLIDATION TEST (ASTM D2435)

Inland Foundation Engineering, Inc.

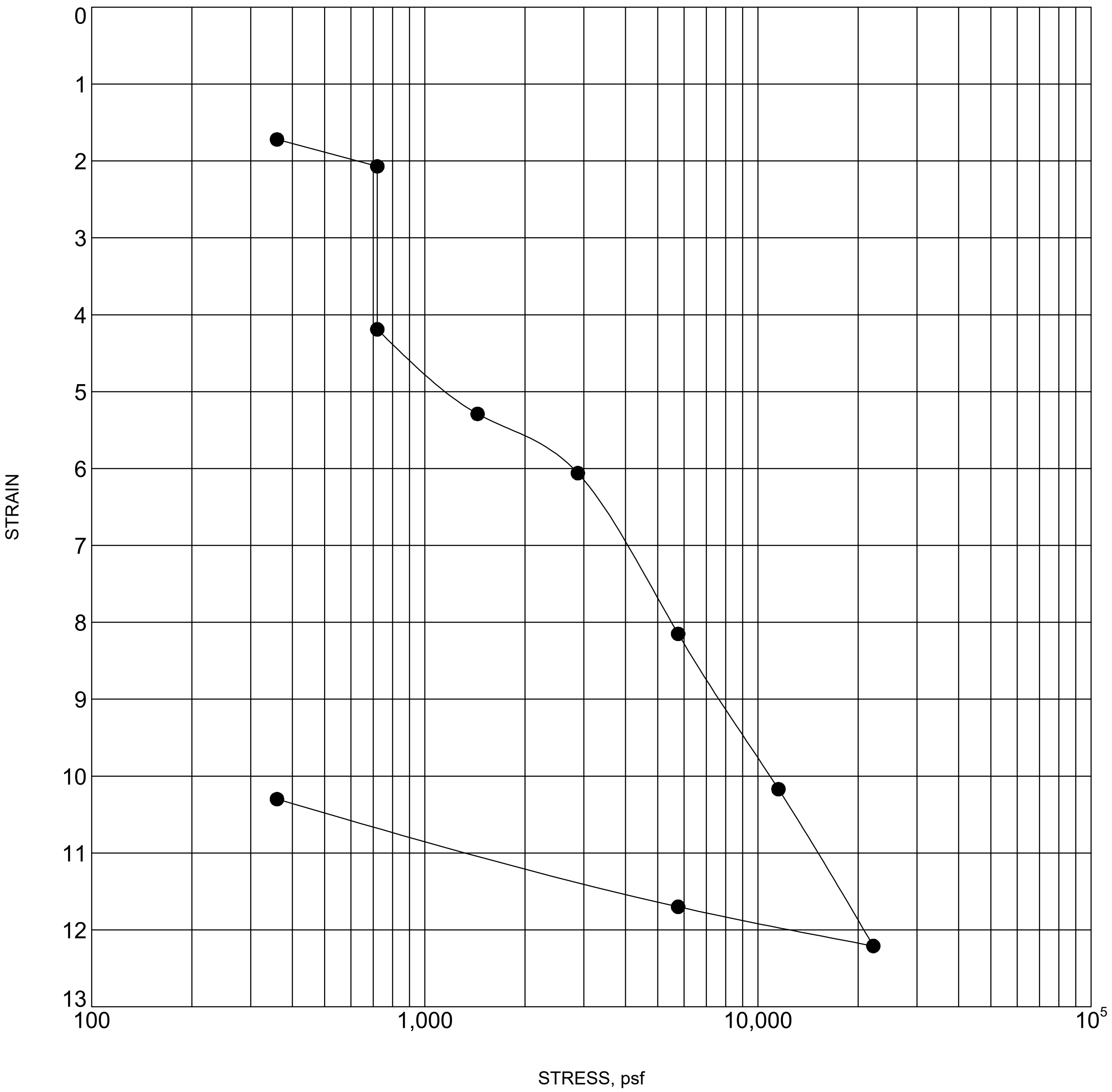
FIGURE NO. B-25

CLIENT RCFCD

PROJECT NAME Wildomar Basin

PROJECT NUMBER R206-025

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-21**	**22.5**	**SILTY SAND (SM)**		**12**

CONSOLIDATION TEST (ASTM D2435)

Inland Foundation Engineering, Inc.

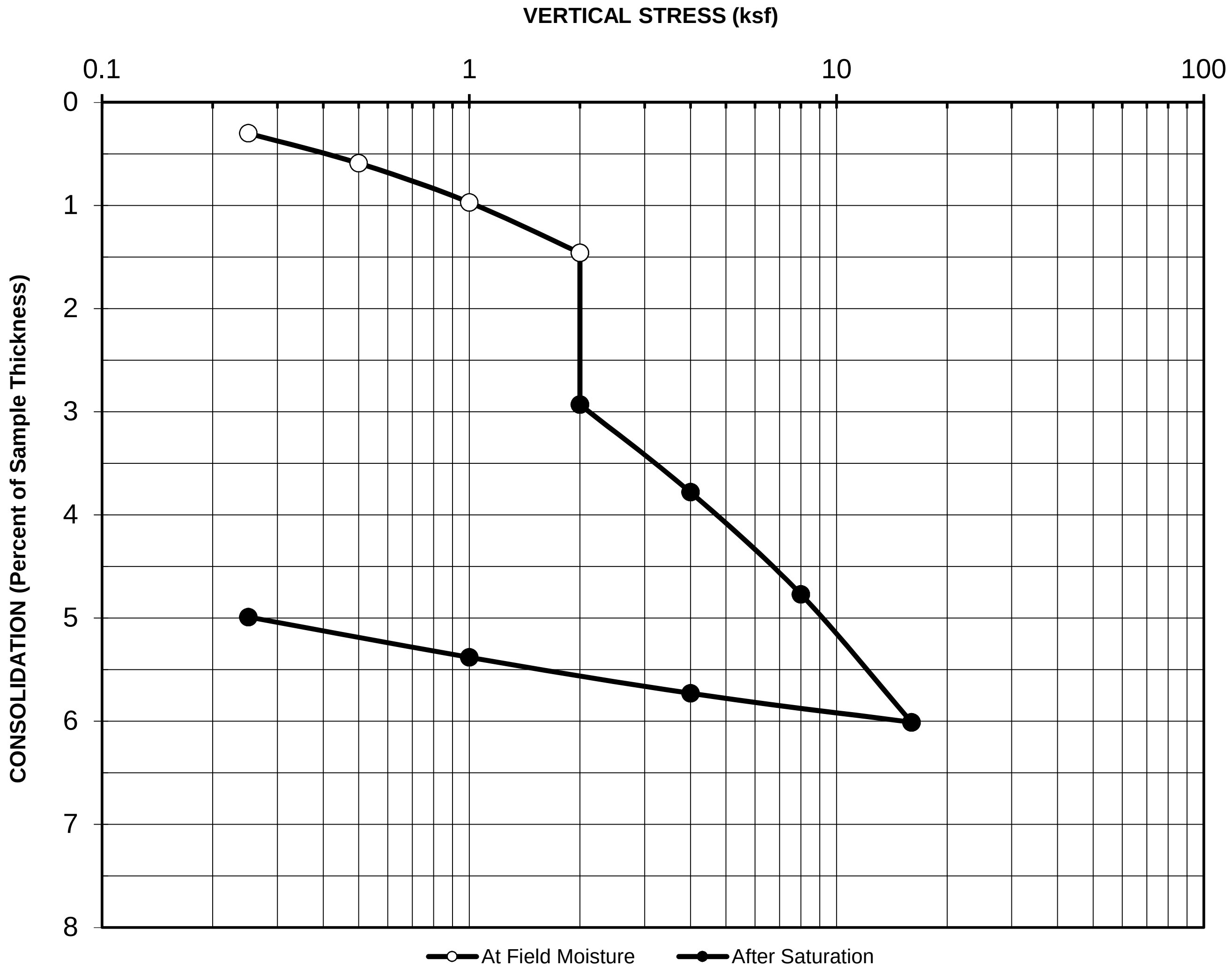
FIGURE NO. B-26

CLIENT RCFCD

PROJECT NAME Wildomar Basin

PROJECT NUMBER R206-025

PROJECT LOCATION SEC Bundy Canyon & Monte Vista
Wildomar, CA



Boring No. : B-21

Sample No.: -

Depth (feet): 22.5-23.5

Sample Type: Mod Cal

Soil Description: Poorly-Graded Sand w/silt

Remarks: Collapse= 1.47% upon inundation

Initial Dry Unit Weight (pcf): 121.9

Initial Moisture Content (%): 3.0

Final Moisture Content (%): 13.5

Assumed Specific Gravity: 2.7

Initial Void Ratio: 0.38

CONSOLIDATION CURVE
ASTM D 2435

Project Name: RCFCD

Project No.: R206-025

Date: 9/1/2020

AP No: 20-0676 **Sheet No:** 1

APPENDIX C –
Infiltration Testing

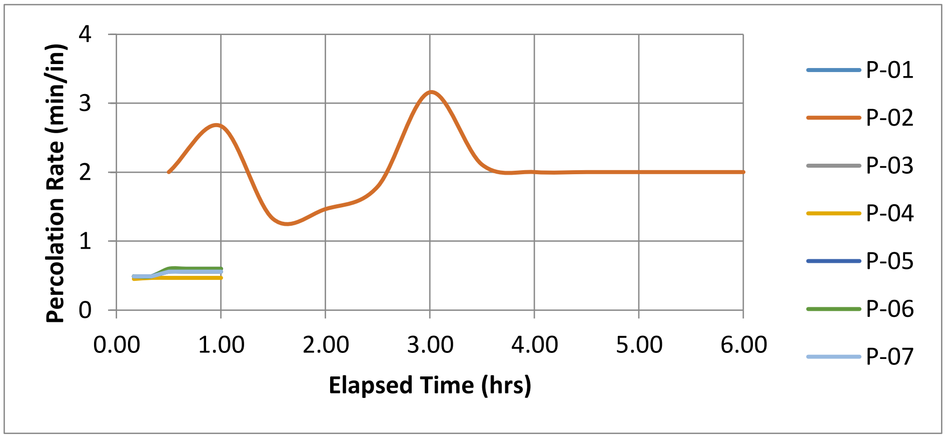
APPENDIX C

INFILTRATION TESTING

Infiltration testing was conducted in general accordance with Appendix A - Infiltration Testing of the Riverside County - Low Impact Development BMP Handbook. Shallow percolation testing was performed per the Riverside County Department of Environmental Health test procedure. A staff geologist conducted the actual percolation testing with equipment and procedures outlined in the Riverside County Technical Guidance Manual.

Seven exploratory borings were converted to percolation test wells within the footprint of the proposed detention basin, at the locations shown on Figure No. A-25. The tests were performed at depths ranging from approximately 27.5 to 36.5 feet below the existing ground surface. Per the specified percolation test procedure, the test holes were filled with water to a depth of at least five (5) times the radius of the test holes. A two-inch thick layer of gravel was placed in the bottom of each test hole.

The test holes were presoaked prior to actual testing. The measured percolation rates at the depths tested ranged from 0.47 to 2 minutes per inch at depths ranging from approximately 27.5 to 36.5 feet below the existing ground surface.



Percolation test rates were converted to infiltration rates (I_c) using the Porchet method and the following equation:

$$I_c = \Delta H 60r / \Delta t(r+2H_{avg})$$

Where:

r = Test Hole Radius (in.)
H_{avg} = Average Height of Water during Test Interval (in.)
ΔH = Change in Water Height during Test Interval (in.), and
Δt = Time Interval (in.)

The corresponding calculated infiltration rates (I_c) ranged from 1.9 to 13.4 inches per hour. These values exclude factors of safety. The table below provides a summary of the test data with values for I_c:

Percolation Test No.	**Percolation Rate (Min./Inch)**	**Depth Below Existing Ground Surface (ft.)**	**Infiltration Rate (I_c) (In./Hr.)**
P-01	0.56	32.25	8.3
P-02	2.0	30.25	1.9
P-03	0.36	36.5	13.4
P-04	0.55	33.5	4.1
P-05	0.47	28.5	9.5
P-06	0.60	27.5	5.9
P-07	0.47	28.5	11.6

APPENDIX D –
Site Specific Analysis

APPENDIX D

SITE SPECIFIC GROUND MOTION ANALYSIS

The site-specific ground motion analysis was conducted for the project in accordance with the 2019 California Building Code and ASCE 7-16. Mapped spectral acceleration parameters, coefficients, and other related seismic parameters were obtained from the OSHPD Seismic Design Maps website (OSHPD, 2020) and the California Building Code (CBC, 2019). The site-specific ground motion analyses were performed following Section 21 of ASCE 7-16.

Because of the differing geologic conditions between the wash deposits in the south portion of the site and the rest of the site, separate site-specific ground motion analyses were conducted. Within this report the separate areas are referred to as Seismic Area 1 and Seismic Area 2. The coordinates (WGS 84) used for Seismic Area 1 are 33.6210°N / -117.2666°W. The coordinates used for the Seismic Area 2 analysis are 33.6245°N / -117.2662°W.

I. Seismic Area 1

- Mapped Spectral Acceleration Parameters (CBC 1613A2.1)

  Based on maps prepared by the USGS (Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Coterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping), a value of 1.668g for the 0.2 second period (S_S) and 0.617g for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1614A.2.1).

- Site Classification (CBC 1613A.2.2 & ASCE 7-16 Chapter 20)

  Our subconsultant Terra Geosciences, conducted a geophysical shear-wave velocity survey at the approximate location shown on Google Earth® imagery below. A copy of the shear wave survey results is appended.

Based on the site-specific measured shear wave value of 373.1 m/sec (1,224.3 feet/second), the soil profile type is Site Class “C”.

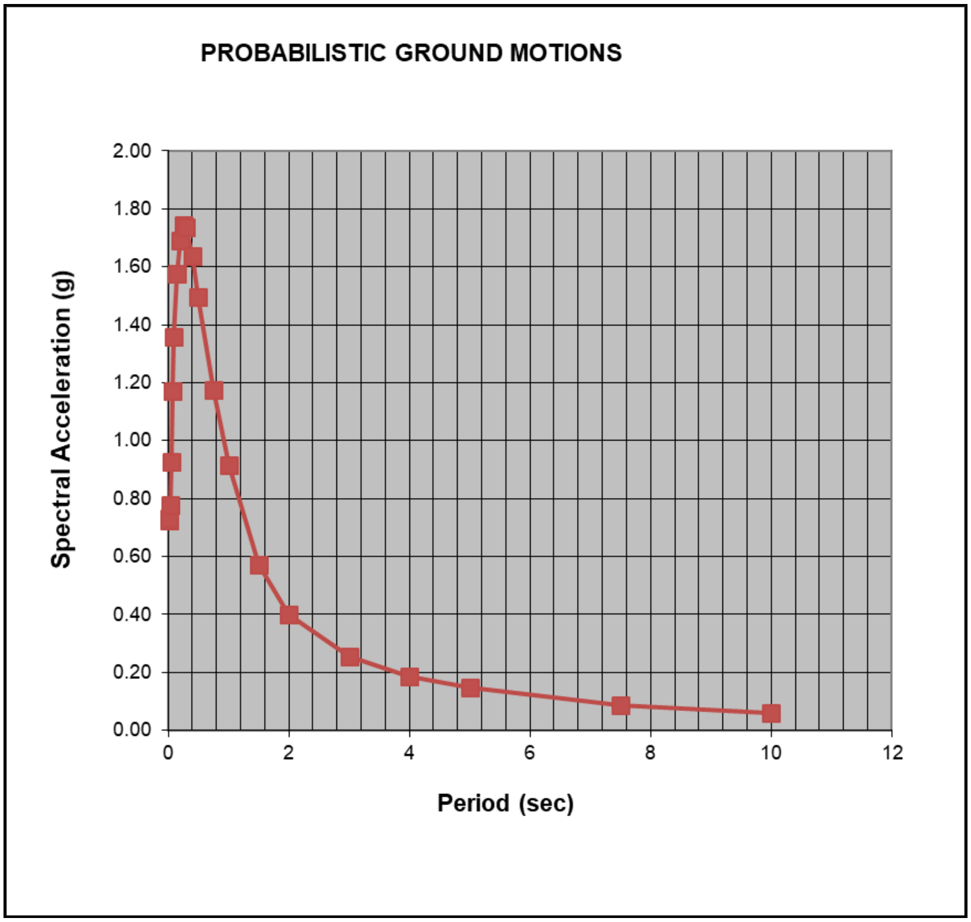
- Site Coefficients (CBC 1613A2.3(1) and 1613A2.3(2)

  Fa = 1.2
  Fv = 1.4

- Probabilistic (MCE_R) Ground Motions (ASCE 7 Section 21.2.1)

  Per Section 21.2.1, the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

  The probabilistic analysis included the use of Open Seismic Hazard Analysis (OpenSHA). The selected Earthquake Rupture Forecast (ERF) was UCERF3 along with a Probability of Exceedance of 2% in 50 years. The average of four Next Generation Attenuation West-2 Relations (2014 NGA) were utilized to produce a response spectrum. These included Chiou & Youngs (2014), Abramson et al., (2014), Boore, et al., (2014) and Campbell & Borzignia (2014). The Probabilistic Risk Targeted Response Spectrum was determined as the product of the ordinates of the probabilistic response spectrum and the applicable risk coefficient (C_R). These values were then modified to produce a spectrum based on the maximum rotated components of ground motion. The resulting MCE_R Response Spectrum is indicated below:



- <u>Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)</u>

  The deterministic MCE_R response acceleration at each period shall be calculated as an 84th-percentile 5 percent damped spectral response acceleration in the direction of mazimum rotated response computed at that period. The largest such accleration calculated for the characteristic earthquakes on all known active faults within the region shall be used. Analyses were conducted with the average of four Next Generation Attenuaton West-2 Relations (2014 NGA), including Chiou & Youngs (2014), Abramson et al., (2014), Boore, et al., (2014), and Campbell & Borzignia (2014).

  Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3: Field, et al., 2013), discussions with the California Geologic Survey (CGS), and based on the length and maximum magnitude of each of the segments of the Elsinore Fault Zone, the largest moment magnitude (Mw) for this fault is 7.78, considering a cascading event along the entire fault zone.

Following is a summary of the Deterministic Spectral Response Acceleration Values and Comparison with Deteministic Lower Limit.

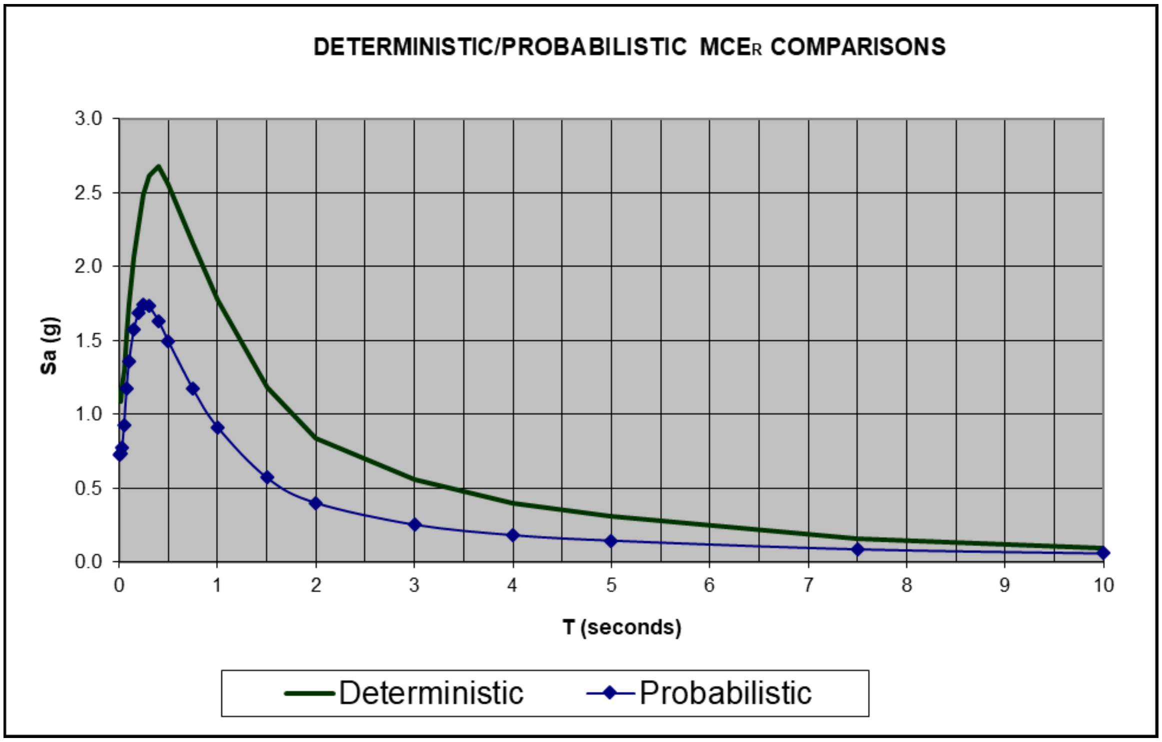
Deterministic Summary and Comparison with Deterministic Lower Limit – Section 21.2.2

T	Median Sa (Average)	*Corrected* Sa (per ASCE7-16)*	Scaled Sa (Average)
0.010	0.97	1.07	1.07
0.020	0.99	1.09	1.09
0.030	1.03	1.13	1.13
0.050	1.18	1.29	1.29
0.075	1.41	1.55	1.55
0.100	1.59	1.75	1.75
0.150	1.87	2.06	2.06
0.200	2.07	2.28	2.28
0.250	2.24	2.49	2.49
0.300	2.32	2.61	2.61
0.400	2.33	2.68	2.68
0.500	2.18	2.56	2.56
0.750	1.75	2.16	2.16
1.000	1.36	1.77	1.77
1.500	0.89	1.18	1.18
2.000	0.62	0.84	0.84
3.000	0.40	0.56	0.56
4.000	0.28	0.40	0.40
5.000	0.21	0.31	0.31
7.500	0.11	0.16	0.16
10.000	0.07	0.10	0.10
PGA	0.97		0.97
Max Sa=	2.68		
Fa =	1.20	Per ASCE7-16 21.2.2	
1.5XFa=	1.8		
Scaling Factor=	1.00		

* Correction is the adjustment for Maximum Rotated Value if Applicable

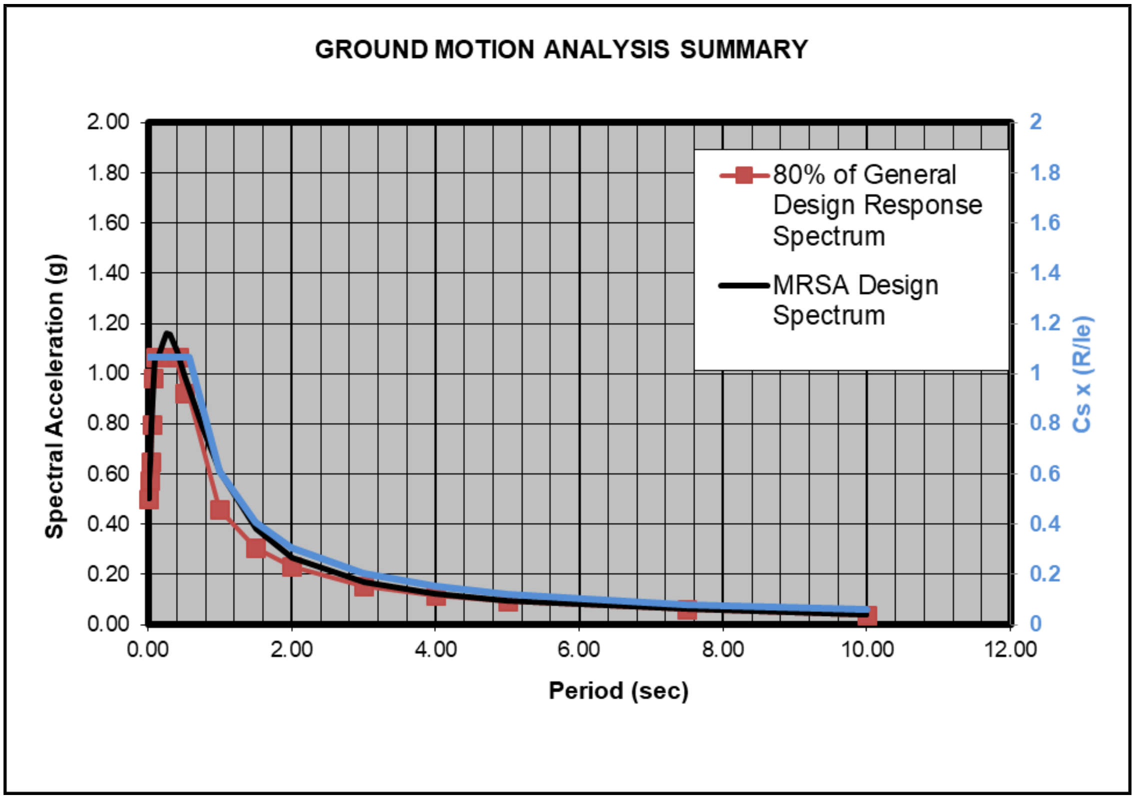
Site-Specific MCE_R (ASCE 7 21.2.3)

The site-specific MCE_R spectral response acceleration at any period, S_{aM}, shall be taken as the lesser of the spectral response accelerations for the probabilistic ground motions of Section 21.2.1 and the deterministic ground motions of Section 21.2.2. The deterministic ground motions were compared with the probabilistic ground motions that were determined per Section 21.2.1. These are plotted in the following diagram:



- Design Response Spectrum (ASCE 7 Section 21.3)

Per Section 21,3, the Design Response Spectrum was developed by the following equation: S_a = 2/3S_{aM}, where S_{aM} is the MCE$_R$ spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration shall not be taken less than 80 percent of S_a. These are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



- Design Acceleration Parameters (ASCE 7 Section 21.4)

Where the site-specific procedure is used to determine the design ground motion per Section 21.3, the parameter S_{DS} shall be 90 percent of the peak spectral acceleration, Sa, at any period larger than 0.2 s. The parameter S_{D1} shall be taken as the greater of the products of Sa * T for the periods between 1 and 5 seconds. The parameters $S_{MS,}$ and S_{M1} shall be taken as 1.5 times S_{DS} and S_{D1}, respectively. The values so obtained shall not be less than 80 percent of the values determined per Section 11.4.4 for S_{MS}, SM1 and Section 11.4.5 for S_{DS} and S_{D1}.

S_{DS} is taken as 90% of the highest value for Sa at any period over 0.2 seconds except that it cannot be less than 80% of the maximum value in the General Design Spectrum. In this case, the value of S_{DS} is 1.07g based on upon the lower limit of 80 percent of the general design spectrum. A value of 0.61g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4).

For the MCE_R 0.2 second period, a value of 1.601g (S_{MS}) was computed, along with a value of 0.915g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

- Site-Specific MCE_G Peak Ground Accelerations (ASCE 7 Section 21.5)

The probablistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.72g. The deterministic geometric mean peak ground acceleration (largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.97g. The site-specific MCE_G peak ground acceleration was calculated to be 0.72g, which was determined by using the lesser of the probablistic (0.72g) or the deteministic (0.97g) geometric mean peak ground accelerations.

The site specific analysis for Seismic Area 1 is summarized below.

II. Seismic Area 2

- Mapped Spectral Acceleration Parameters (CBC 1613A2.1)

A value of 1.664g for the 0.2 second period (S_s) and 0.615g for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1614A.2.1).

- Site Classification (CBC 1613A.2.2 & ASCE 7-16 Chapter 20)

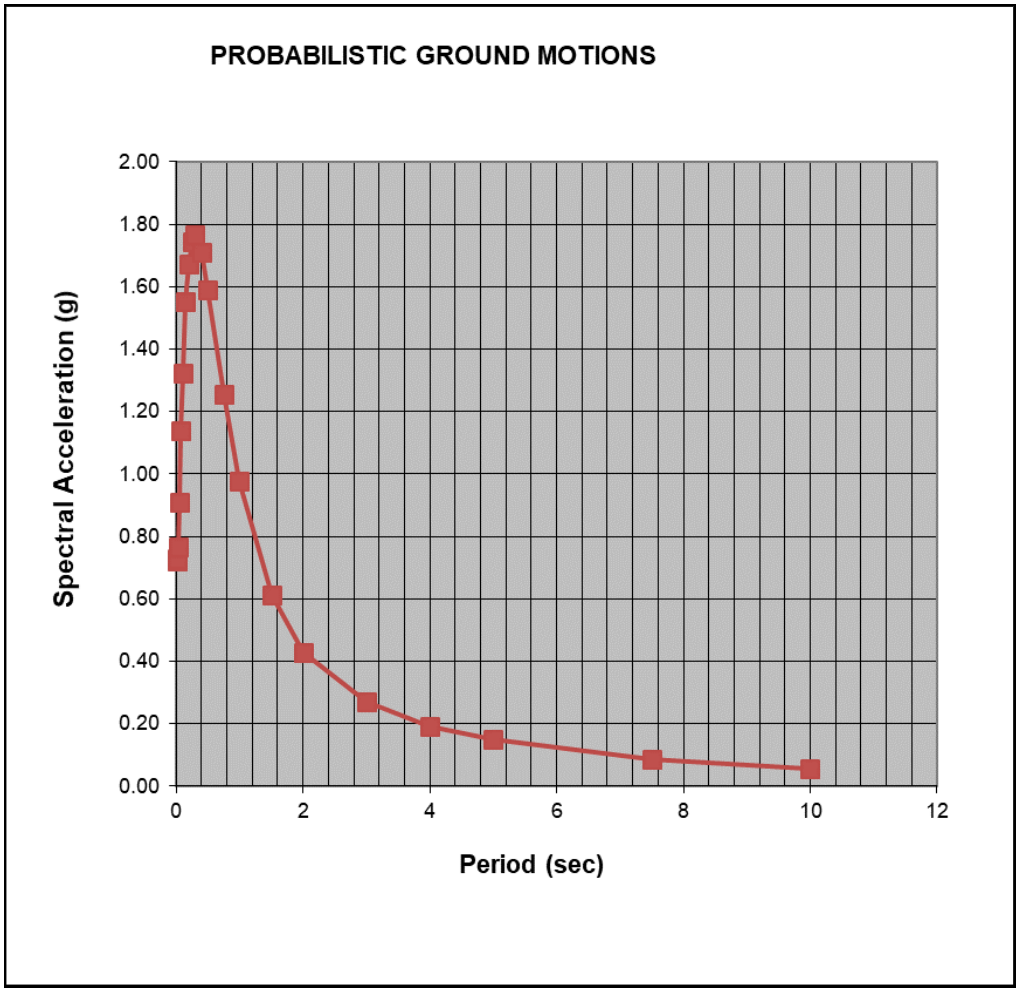
Based on the seismic refraction survey and exploration boring data, the inferred soil profile type for Seismic Area 2 is Site Class “D”**.**

- Site Coefficients (CBC 1613A2.3(1) and 1613A2.3(2)

Fa = 1.0
Fv = 2.5

- <u>Probabilistic (MCE$_R$) Ground Motions (ASCE 7 Section 21.2.1)</u>

The MCE$_R$ Response Spectrum is indicated below:



- <u>Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)</u>

Following is a summary of the Deterministic Spectral Response Acceleration Values and Comparison with Deteministic Lower Limit.

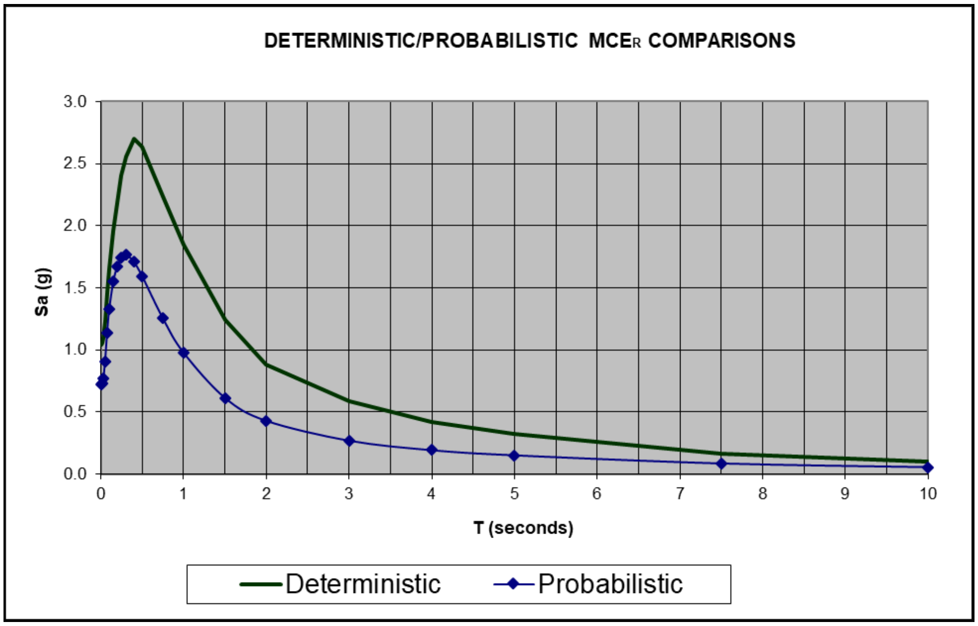
Deterministic Summary and Comparison with Deterministic Lower Limit – Section 21.2.2

T	**Median Sa** (Average)	***Corrected* Sa*** *(per ASCE7-16)*	**Scaled Sa** (Average)
0.010	0.95	1.04	1.04
0.020	0.96	1.06	1.06
0.030	0.99	1.09	1.09
0.050	1.12	1.23	1.23
0.075	1.33	1.46	1.46
0.100	1.50	1.65	1.65
0.150	1.78	1.96	1.96
0.200	1.98	2.18	2.18
0.250	2.16	2.40	2.40
0.300	2.27	2.55	2.55
0.400	2.34	2.70	2.70
0.500	2.24	2.63	2.63
0.750	1.81	2.24	2.24
1.000	1.42	1.85	1.85
1.500	0.94	1.24	1.24
2.000	0.65	0.88	0.88
3.000	0.42	0.58	0.58
4.000	0.29	0.42	0.42
5.000	0.22	0.33	0.33
7.500	0.11	0.17	0.17
10.000	0.07	0.10	0.10
PGA	0.94		0.94
Max Sa=	2.70		
Fa =	1.00	Per ASCE7-16 21.2.2	
1.5XFa=	1.5		
Scaling Factor=	1.00		

* Correction is the adjustment for Maximum Rotated Value if Applicable

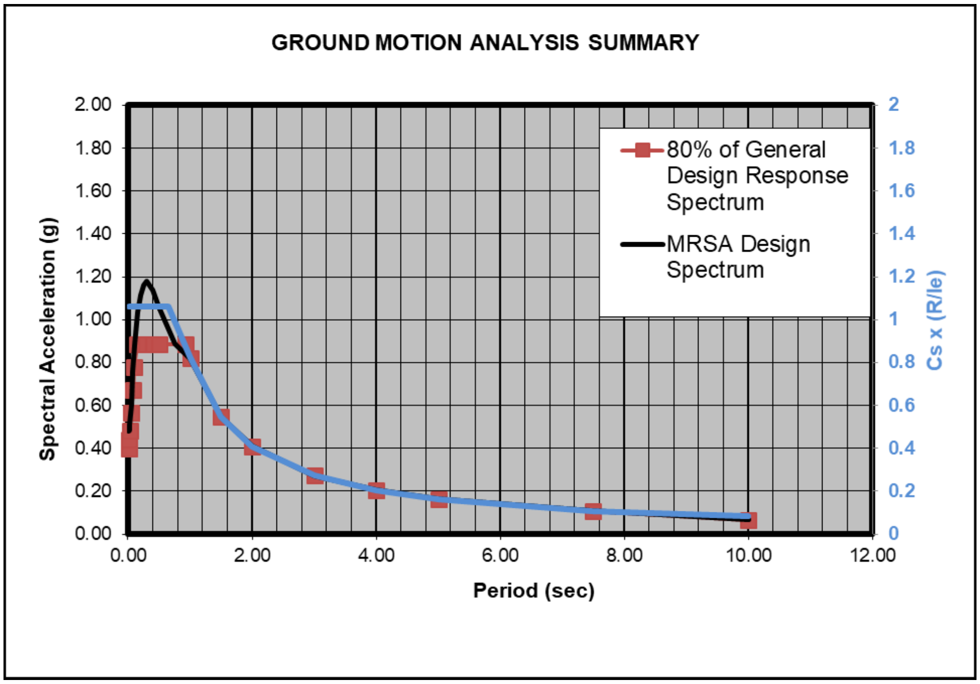
Site-Specific MCE$_R$ (ASCE 7 21.2.3)

Deterministic and probabilistic ground motions determined per Section 21.2.1 are plotted in the following diagram:



- Design Response Spectrum (ASCE 7 Section 21.3)

The design spectral response acceleration values are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



- Design Acceleration Parameters (ASCE 7 Section 21.4)

  The value of S_{DS} is 1.06g. A value of 0.82g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4).
  For the MCE_R 0.2 second period, a value of 1.591g (S_{MS}) was computed, along with a value of 1.230g (S_{M1}) for the MCE_R 1.0 second period (ASCE 7-16, 21.2.3).

- Site-Specific MCE_G Peak Ground Accelerations (ASCE 7 Section 21.5)

  The probablistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.72g. The deterministic geometric mean peak ground acceleration (largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.94g. The site-specific MCE_G peak ground acceleration was calculated to be 0.72g, which was determined by using the lesser of the probablistic (0.72g) or the deteministic (0.94g) geometric mean peak ground accelerations.

APPENDIX E –
Liquefaction Analysis

APPENDIX E

LIQUEFACTION AND SEISMIC SETTLEMENT ANALYSIS

Liquefaction and seismic settlement potential were evaluated using the GeoSuite® computer program (version 2.2.2.14). The seismic parameters included a horizontal acceleration of 0.72g and a moment magnitude of 7.78. We analyzed the soil profile logged for exploratory borings B-03/04 (combined) and B-21. Liquefaction settlement analysis was based on the simplified procedures developed by Seed and Idriss and modified by Idriss and Boulanger (2008). The potential for “dry sand” seismically-induced settlement was evaluated using Pradel’s method (1998). The GeoSuite® program calculates corrected normalized SPT N-values $(N_1)_{60}$ using the following formula (SCEC, 1999).

$(N_1)_{60} = N_M C_N C_E C_B C_R C_S$

Where; N_M = measured standard penetration resistance. Modified California sample blowcounts were converted to SPT blowcounts using Burmister’s formula (1948) prior to input in the program. The modified California sample blowcounts were also corrected to account for lined samplers, as described in the C_S factor discussion below.

C_N = depth correction factor. GeoSuite® calculates C_N for each layer in the soil profile using the relationship suggested by Idriss and Boulanger (2008)

C_E = hammer energy ratio (ER) correction factor. A C_E factor of 1.3 was applied for the automatic trip hammer used during drilling. This was calculated using the relationship suggested by Idriss and Boulanger (2008) and SPT hammer energy measurements provided by the drilling subcontractor.

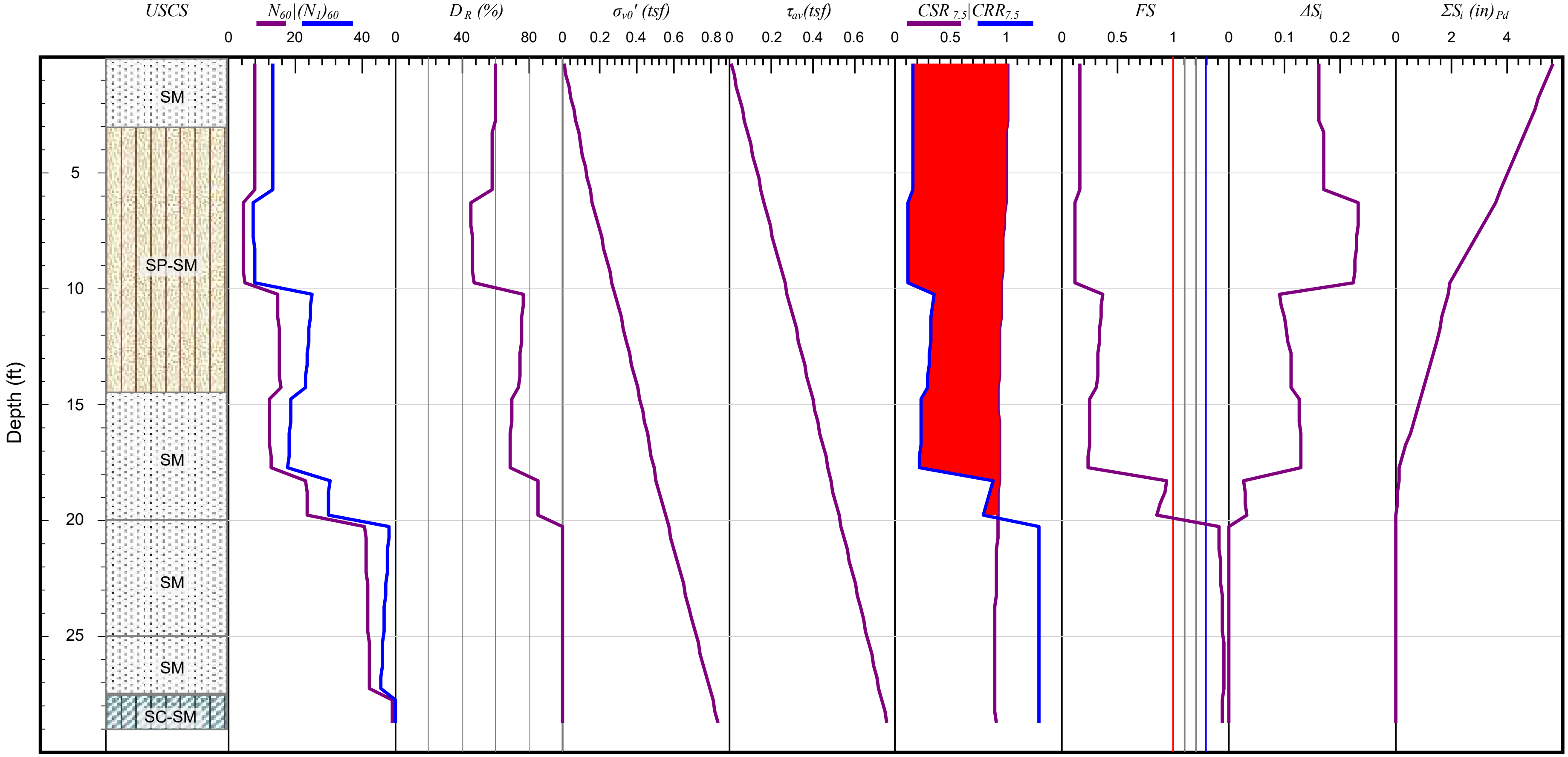
C_B = borehole diameter correction factor. A C_B factor of 1.0 was applied for the 8-inch diameter hollow-stem augers with inside diameters of four (4) inches (SCEC 1999).

C_R = rod length correction factor. GeoSuite® applies a C_R factor for each layer in the soil profile using the values in Table 5.2 of the 1999 SCEC guidelines, and assuming a rod stick up length (above the ground surface) of 3 feet.

C_S = correction factor for samplers with or without liners. SPT samplers without liners were used for this project. For SPT samplers without liners, GeoSuite® applies a C_S factor for each layer in the soil profile using the relationships from Seed et al. (1984) and suggested by Idriss and Boulanger (2008). Since GeoSuite® applies a C_S factor to all layers in the soil profile, it is necessary to adjust blowcounts for modified California samplers with liners.

This was done through an iterative process by initially dividing the modified California sampler blowcounts by an assumed C_S value of 1.2 prior to input in the program. Calculated C_S values were then checked against the assumed values and adjusted where necessary, so that the actual applied C_S value for modified California samples is 1.0.

The results of our analysis are shown on Figure E-3 and E-4.



▨ SM

▨ SC-SM

▨ SP-SM

Earthquake & Groundwater Information:
Magnitude = 7.78
Max. Acceleration = 0.72 g
Project GW = 0 ft
Maximum Settlement = 5.63 in
Settlement at Bottom of Footing = 5.63 in

Liquefaction: Idriss & Boulanger (2008)
Settl.: Pradel (1998); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

INLAND FOUNDATION ENGINEERING, INC.
Consulting Geotechnical Engineers and Geologists

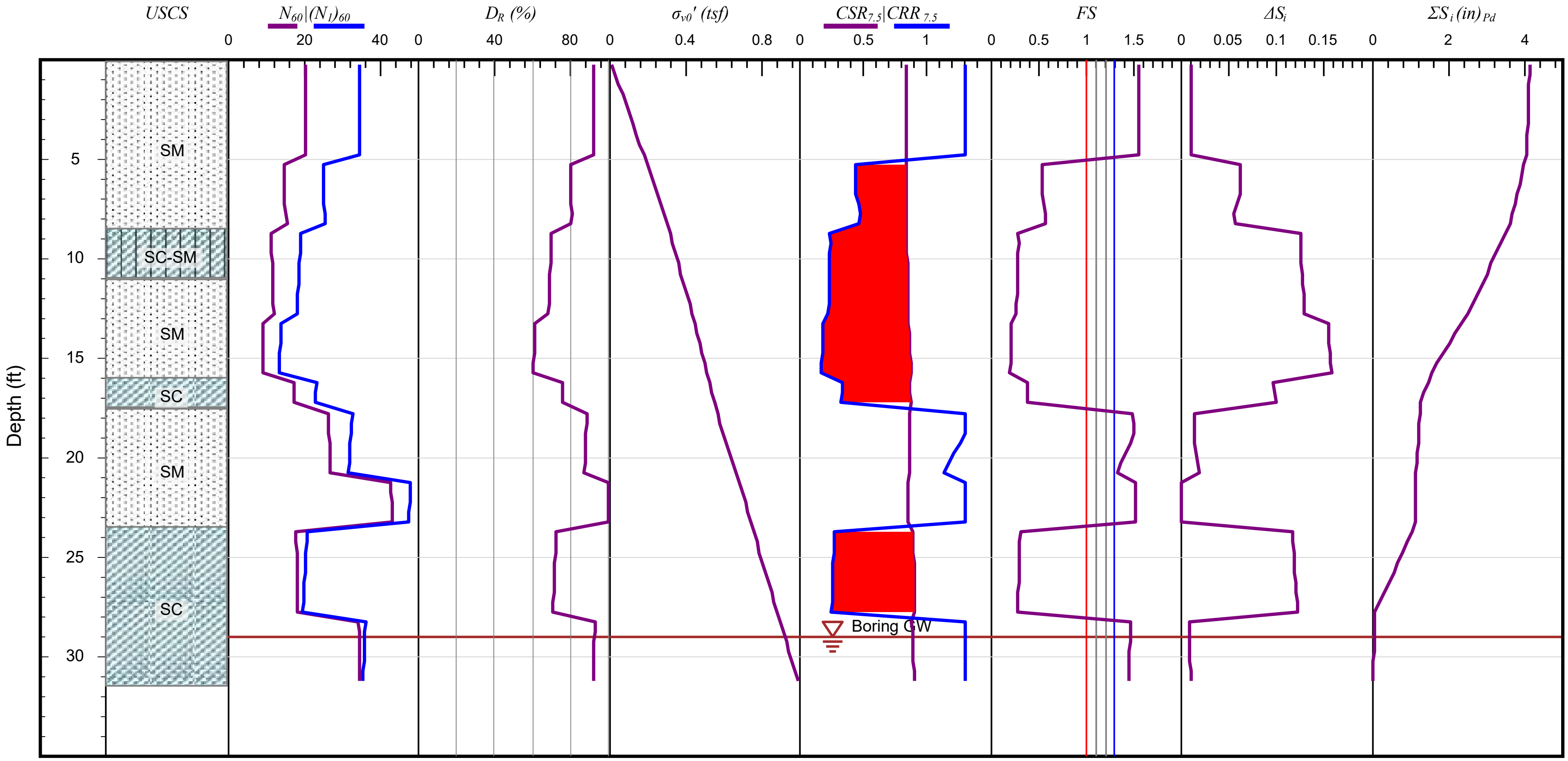
Liquefaction Potential - SPT Data

Project:	Wildomar MDP Lateral C, Stage 3		
Location:	Wildomar, California		
Project No.:	R206-025	Boring No.:	B-03, 04
Figure:	E-3		

P:\R206\R206-025 Wildomar\Liquefaction\GeoSuite\GeoSuite_R206-025_B-03.csv

GeoSuite© Version 2.2.2.14. Developed by Fred Yi, PhD, PE, GE

Prepared at 11/30/2020 10:15:58 AM



░ SM

▨ SC

▥ SC-SM

Earthquake & Groundwater Information:
Magnitude = 7.78
Max. Acceleration = 0.72 g
Project GW = 0 ft
Maximum Settlement = 4.14 in
Settlement at Bottom of Footing = 4.14 in

Liquefaction: Idriss & Boulanger (2008)
Settl.: Pradel (1998); [sat] Idriss & Boulanger (2008)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

P:\R206\R206-025 Wildomar\Liquefaction\GeoSuite\GeoSuite_R206-025_B-21.csv

INLAND FOUNDATION ENGINEERING, INC.
Consulting Geotechnical Engineers and Geologists

Liquefaction Potential - SPT Data

Project: Wildomar MDP Lateral C, Stage 3
Location: Wildomar, California
Project No.: R206-025
Boring No.: B-21
Figure: E-4

GeoSuite© Version 2.2.2.14. Developed by Fred Yi, PhD, PE, GE

Prepared at 11/30/2020 10:19:03 AM

APPENDIX F –
Slope Stability

APPENDIX F

SLOPE STABILITY ANALYSIS

Slope stability analyses were performed using the Slide 6.0 computer program (RocScience, 2013), which uses two-dimensional finite element analysis.

Permanent Slopes

The north basin slope was analyzed to be generally representative of most project slopes and consists of native alluvial soil over bedrock. The slope was analyzed for the following conditions, with the resulting factors of safety (FS) shown.

- Static stability, basin full FS = 1.7
- Static stability, basin empty FS = 1.7
- Pseudo-static stability, basin full FS = 1.1
- Pseudo-static stability, basin empty FS = 1.1
- Rapid drawdown stability FS = 0.7

Minimum factors of safety of 1.5 and 1.1 are considered acceptable for static and seismic conditions, respectively, based on current standards in Riverside County. Project slopes are expected to perform satisfactorily with routine maintenance.

The rapid drawdown factor of safety of 0.7 indicates that slope failure within the alluvial soil is likely if the basin water level drops faster than the soil pore water pressure can dissipate. Slopes should be monitored during drawdown for indications of instability.

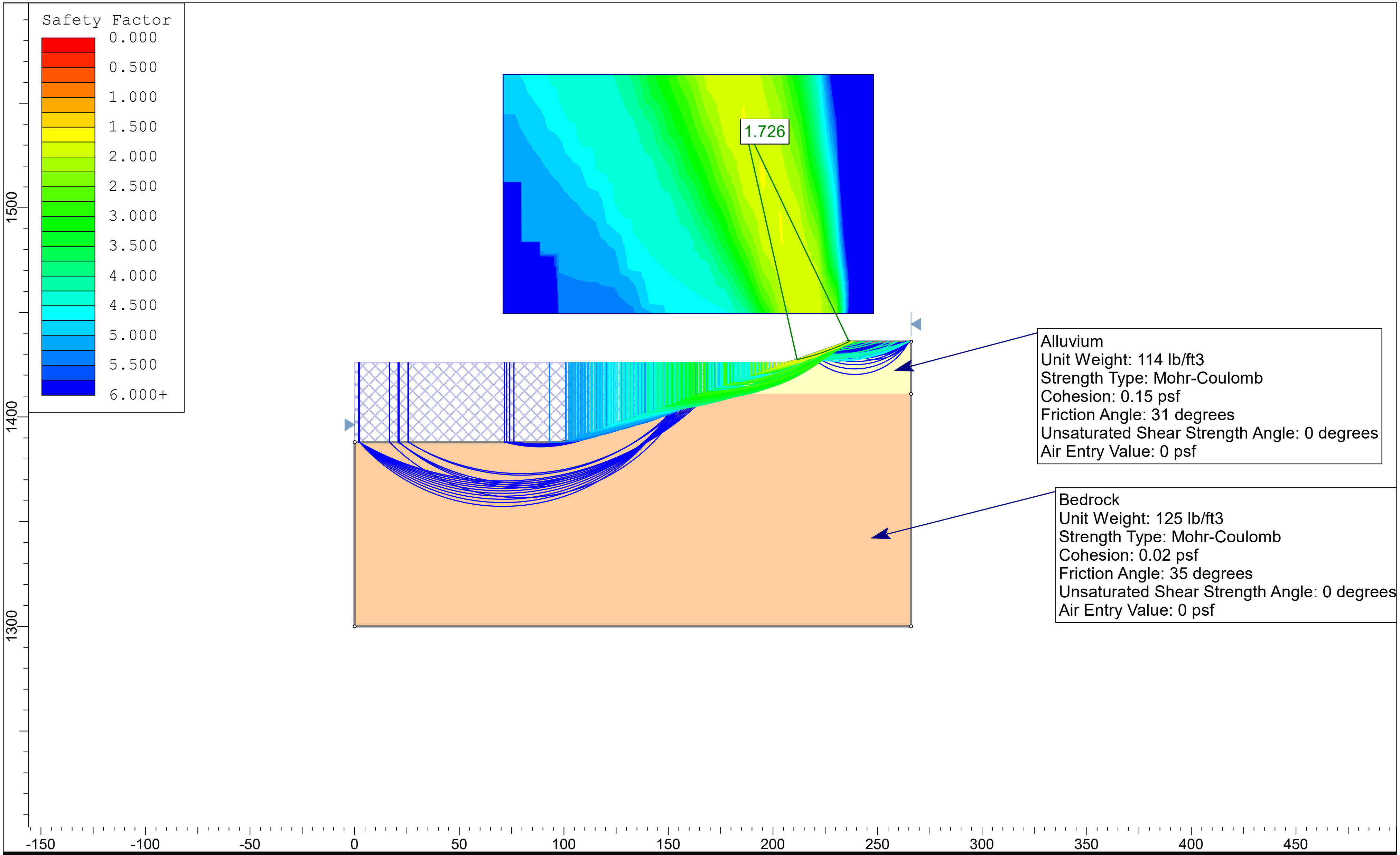
The Bishop simplified method was used to calculate the factors of safety for static and pseudo-static analysis. Rapid drawdown analysis was performed using Spencer's and B-Bar methods. Results of the slope stability analyses are included as Figure Nos. F-3 through F-7.

Temporary Slopes

Temporary excavation slopes for the proposed stilling basin were evaluated for the following conditions, with the resulting factors of safety shown.

- Near-vertical backcut within District right-of-way FS = 0.5
- 1.5:1 (h:v) backcut extending outside of District right-of-way FS = 0.9

A minimum factor of safety of 1.1 is considered acceptable for temporary slopes. Results of the temporary slope stability analyses are included as Figure Nos. F-8 and F-9.



Alluvium
Unit Weight: 114 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.15 psf
Friction Angle: 31 degrees
Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf

Bedrock
Unit Weight: 125 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.02 psf
Friction Angle: 35 degrees
Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf

INLAND FOUNDATION ENGINEERING, INC.
IF
Est. 1978

SLIDEINTERPRET 6.039

Project RCFCD - Wildomar Basin

Analysis Description Static Stability of North Basin Slope - Basin Full

Drawn By KC/AE

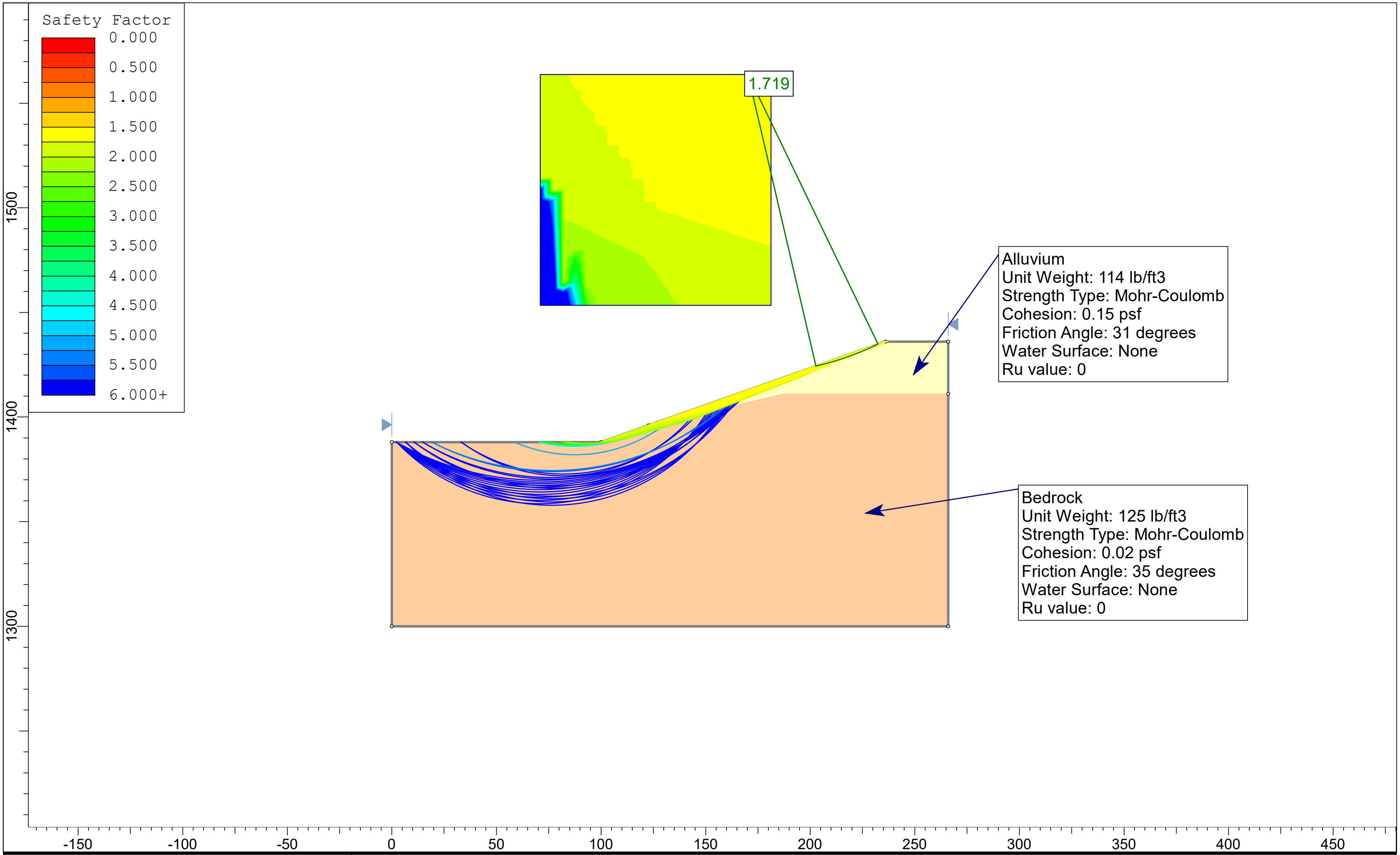
Scale 1:762

Company Inland Foundation Engineering, Inc.

Date 8/7/2020, 10:24:24 AM

File Name Static Undrained.slim

F-3

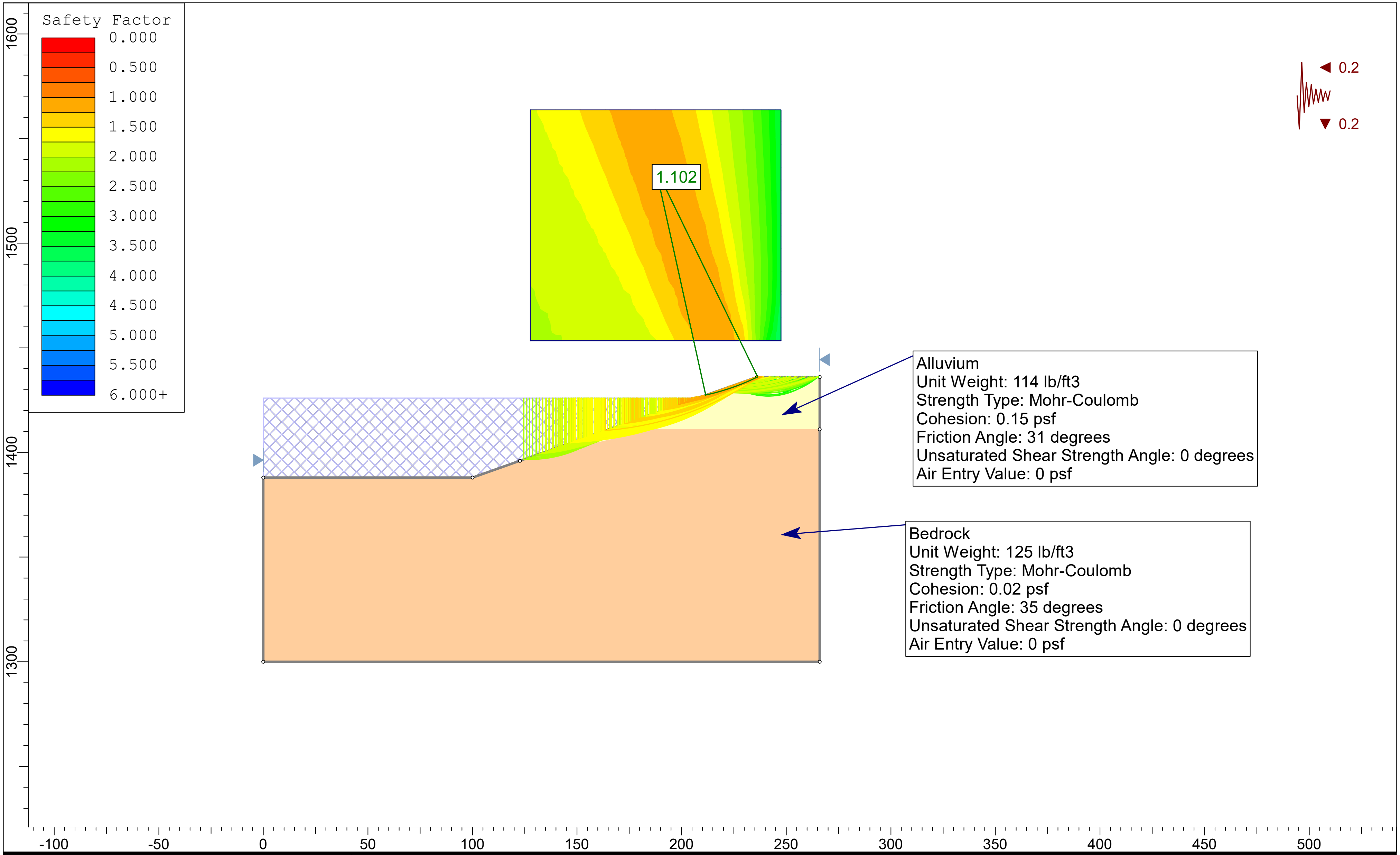


INLAND FOUNDATION ENGINEERING, INC.
Est. 1978

SLIDEINTERPRET 6.039

Project	RCFCD - Wildomar Basin
Analysis Description	Static Stability of North Basin Slope - Basin Empty
Drawn By	KC/AE
Scale	1:762
Company	Inland Foundation Engineering, Inc.
Date	8/7/2020, 10:24:24 AM
File Name	Static Drained.slim

F-4



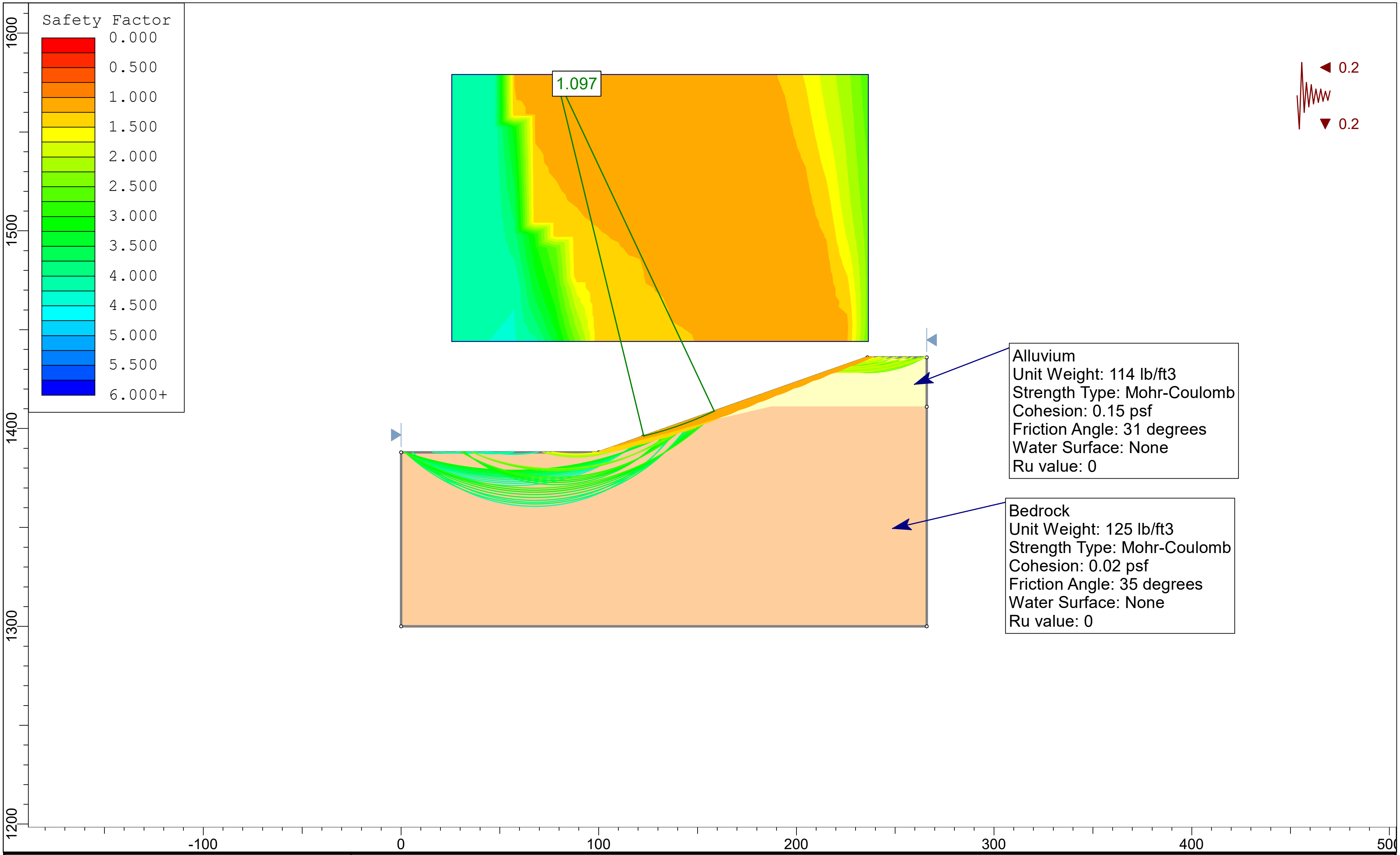
Alluvium
Unit Weight: 114 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.15 psf
Friction Angle: 31 degrees
Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf

Bedrock
Unit Weight: 125 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.02 psf
Friction Angle: 35 degrees
Unsaturated Shear Strength Angle: 0 degrees
Air Entry Value: 0 psf

INLAND FOUNDATION ENGINEERING, INC.
Est. 1978

SLIDEINTERPRET 6.039

Project RCFCD - Wildomar Basin
Analysis Description Pseudo-static Stability of North Basin Slope - Basin Full
Drawn By KC/AE
Scale 1:762
Company Inland Foundation Engineering, Inc.
Date 8/7/2020, 10:24:24 AM
File Name Psuedo-Static Undrained.slim
F-5



Alluvium
Unit Weight: 114 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.15 psf
Friction Angle: 31 degrees
Water Surface: None
Ru value: 0

Bedrock
Unit Weight: 125 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.02 psf
Friction Angle: 35 degrees
Water Surface: None
Ru value: 0

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Est. 1978

SLIDEINTERPRET 6.039

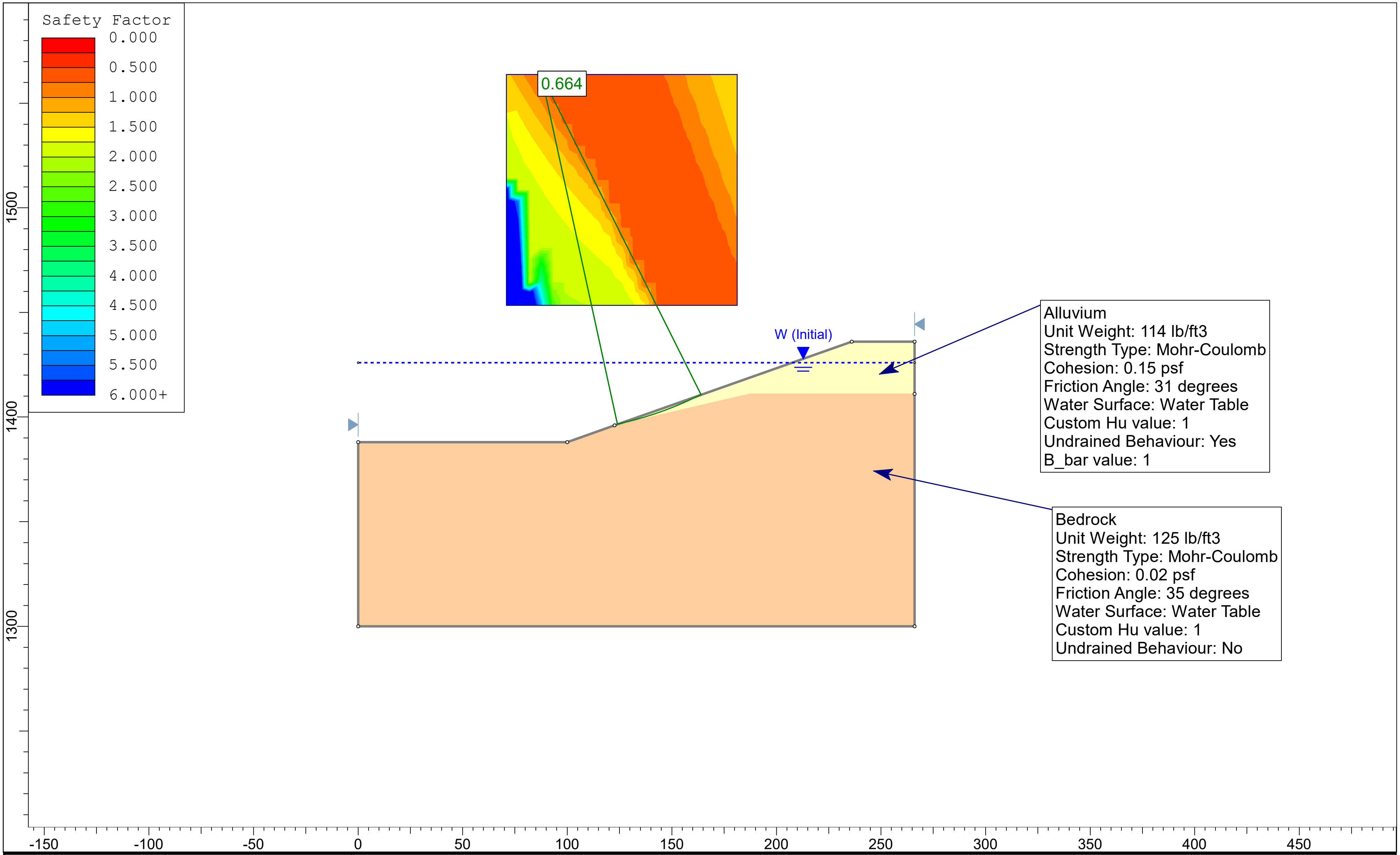
Project RCFCD - Wildomar Basin

Analysis Description Pseudo-static Stability of North Basin Slope - Basin Empty

Drawn By KC/AE
Scale 1:806
Company Inland Foundation Engineering, Inc.

Date 8/7/2020, 10:24:24 AM
File Name Psuedo-Static Drained.slim

F-6



Alluvium
Unit Weight: 114 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.15 psf
Friction Angle: 31 degrees
Water Surface: Water Table
Custom Hu value: 1
Undrained Behaviour: Yes
B_bar value: 1

Bedrock
Unit Weight: 125 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.02 psf
Friction Angle: 35 degrees
Water Surface: Water Table
Custom Hu value: 1
Undrained Behaviour: No

INLAND FOUNDATION ENGINEERING, INC.
Est. 1978

SLIDEINTERPRET 6.039

Project RCFCD - Wildomar Basin

Analysis Description Rapid Drawdown Stability of North Basin Slope

Drawn By KC/AE

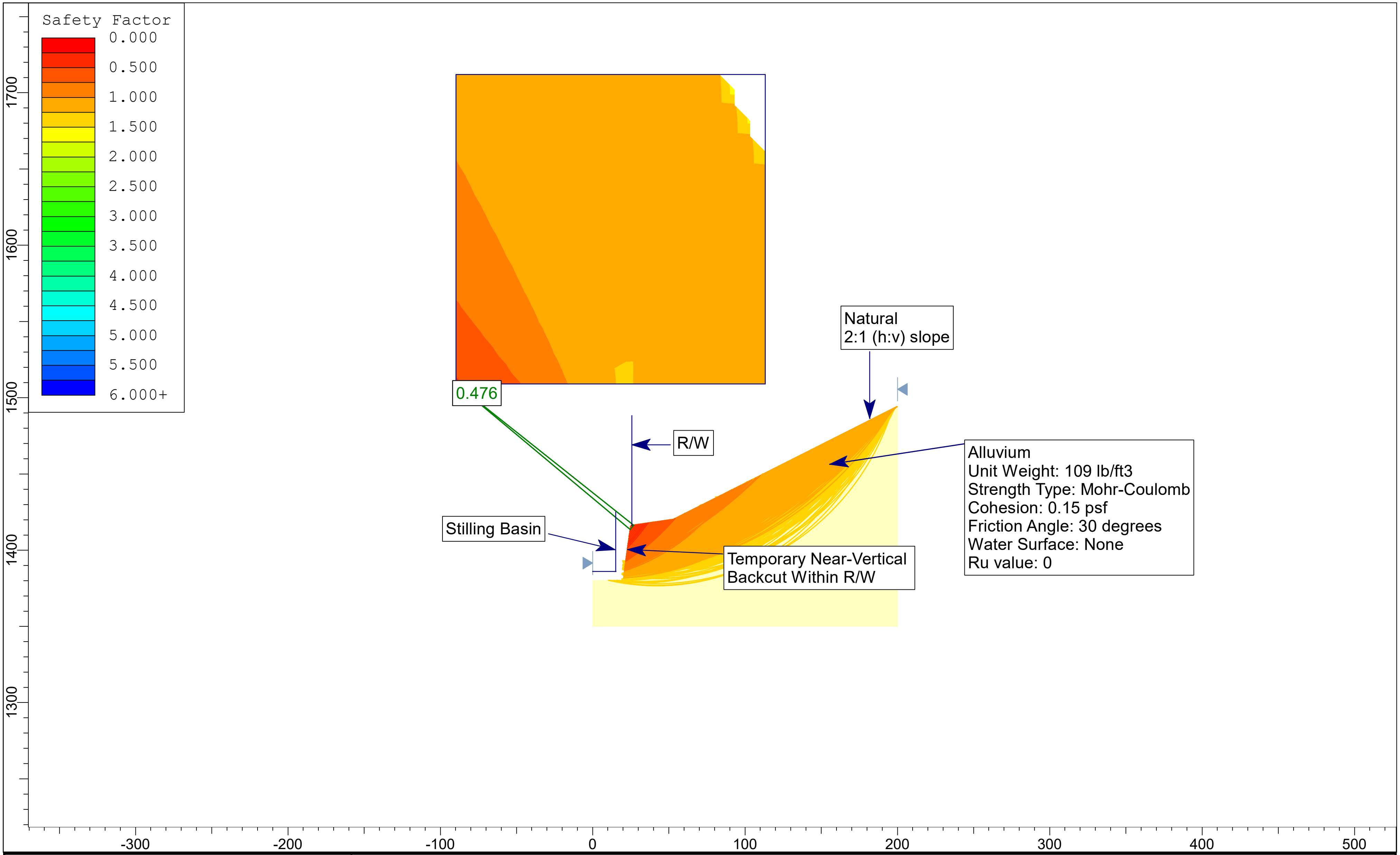
Scale 1:762

Company Inland Foundation Engineering, Inc.

Date 8/7/2020, 10:24:24 AM

File Name rapid drawdown.slim

F-7

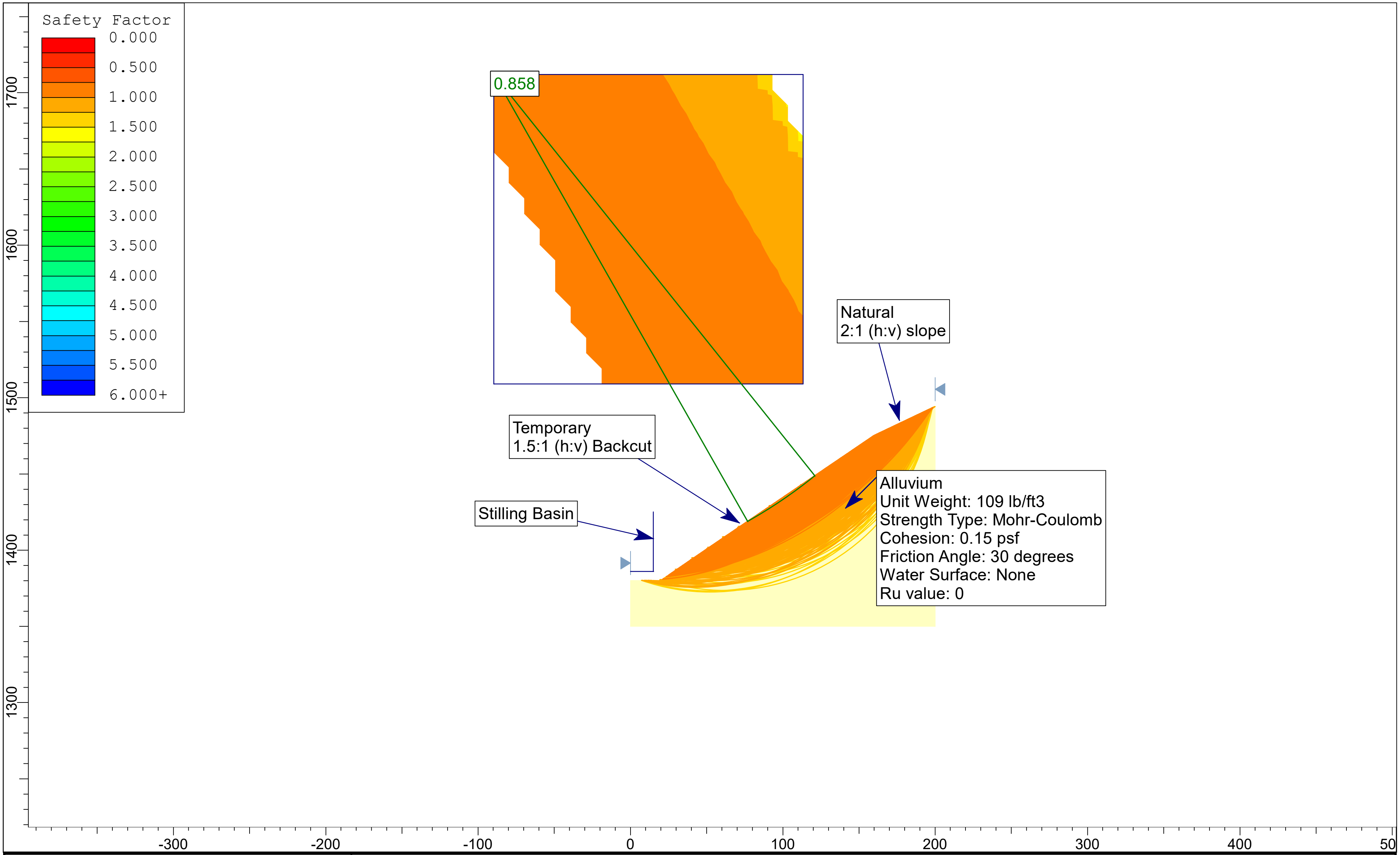


Alluvium
Unit Weight: 109 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.15 psf
Friction Angle: 30 degrees
Water Surface: None
Ru value: 0

INLAND FOUNDATION ENGINEERING, INC.
Est. 1978

SLIDEINTERPRET 6.039

Project	RCFCD - Wildomar Basin		
Analysis Description	Stability of Temporary Near-Vertical Backcut at Stilling Basin		
Drawn By KC/AE	*Scale* 1:1045	*Company*	Inland Foundation Engineering, Inc.
Date	9/2/2020, 2:44:56 PM	*File Name* SS Temporary Cut.slim	F-8



Natural
2:1 (h:v) slope

Temporary
1.5:1 (h:v) Backcut

Stilling Basin

Alluvium
Unit Weight: 109 lb/ft3
Strength Type: Mohr-Coulomb
Cohesion: 0.15 psf
Friction Angle: 30 degrees
Water Surface: None
Ru value: 0

INLAND FOUNDATION ENGINEERING, INC.
Est. 1978

SLIDEINTERPRET 6.039

Project	RCFCD - Wildomar Basin				
Analysis Description	Stability of Temporary 1.5:1 Backcut at Stilling Basin				
Drawn By	KC/AE	*Scale*	1:1045	*Company*	Inland Foundation Engineering, Inc.
Date	9/2/2020, 2:45:06 PM	*File Name*	SS Backcut.slim	F-9	

APPENDIX G –
Seepage Analysis

APPENDIX G

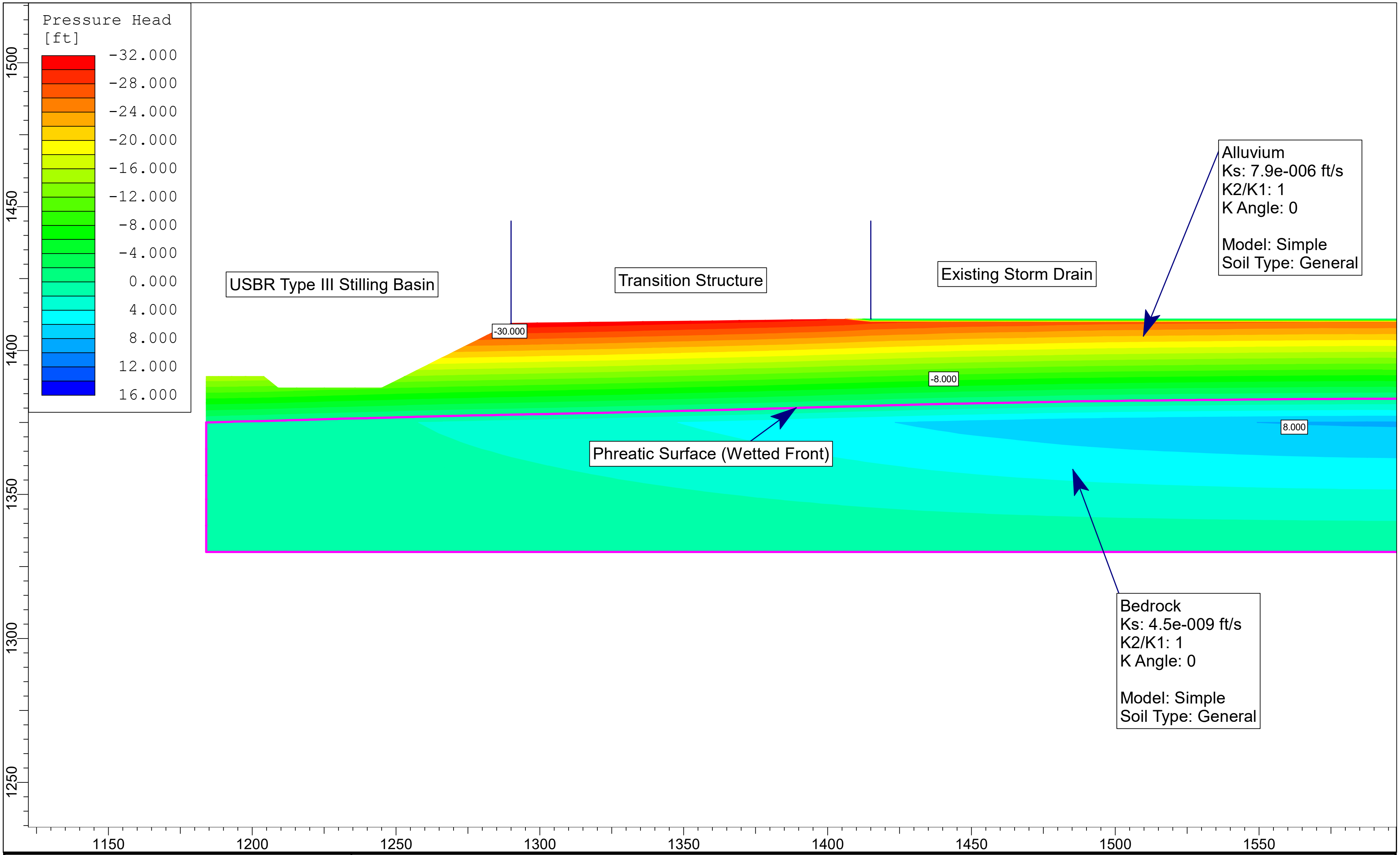
SEEPAGE ANALYSIS

Seepage analysis was performed using the Slide 6.0 computer program (RocScience, 2013). Slide 6.0 uses two-dimensional finite element analysis to evaluate saturated / unsaturated, steady state or transient flow conditions. For this project, we used a steady state analysis.

The purpose of the analysis was to evaluate seepage from the existing Bundy Canyon Channel downslope toward the proposed transition structure and stilling basin. For purposes of the analysis, we considered the area of analysis to consist of two primary soil types, the surficial alluvial soil and underlying granitic bedrock.

A schematic cross-section of the seepage model is shown on Figure G-2. The finite element seepage model is based on the following assumptions.

- There is a steady source of water in the bottom of the existing Bundy Canyon Channel at elevation 1411 until a steady-state seepage condition develops.
- The upper alluvial soil materials have a permeability coefficient of 7.9×10^{-6} ft/sec (2.4×10^{-4} cm/sec).
- The underlying site soil materials consist of granitic bedrock with a permeability coefficient of 4.5×10^{-9} ft/sec (1.4×10^{-7} cm/sec).



INLAND FOUNDATION ENGINEERING, INC.
Est. 1978

SLIDEINTERPRET 6.039

Project RCFCD - Wildomar Basin

Analysis Description Seepage Analysis - Bundy Canyon Channel

Drawn By KC/AE

Scale 1:554

Company Inland Foundation Engineering, Inc.

Date 8/6/2020, 12:40:32 PM

File Name RCFCD Pressure Head Current.slim

G-2

APPENDIX H –
Seismic Refraction Survey

TERRA
GEOSCIENCES

SEISMIC REFRACTION SURVEY
WILDOMAR MPD LATERAL C, STAGE 3 PROJECT
SEC OF BUNDY CANYON ROAD AND MONTE VISTA DRIVE
WILDOMAR AREA, RIVERSIDE COUNTY, CALIFORNIA

Project No. 203423-2

July 13, 2020

Prepared for:

Inland Foundation Engineering, Inc.
1310 South Santa Fe Avenue
San Jacinto, CA 92583

Inland Foundation Engineering, Inc.
1310 South Santa Fe Avenue
San Jacinto, CA 92583

July 13, 2020
Project No. 203423-2

Attention: Mr. Alan Evans, G.E.

Regarding: Seismic Refraction Survey
Wildomar MPD Lateral C, Stage 3 Project
SEC of Bundy Canyon Road and Monte Vista Drive
Wildomar Area, Riverside County, California
IFE Project No. R206-025

EXECUTIVE SUMMARY

As requested, this firm has performed a geophysical survey using the seismic refraction method for the above-referenced site. The purpose of this investigation was to assess the general seismic velocity characteristics of the underlying earth materials and to evaluate whether high-velocity granitic bedrock (non-rippable) may be present. This report will describe in further detail the procedures used and the results of our findings, along with presentation of representative seismic models for the survey traverses.

For this study, eight survey traverses (Seismic Lines S-1 though S-8) were performed within the proposed flood control basin project, as selected by your office, which is located along the southeast corner of Bundy Canyon Road and Monte Vista Drive, in the Wildomar area of Riverside County, California. These traverses were located in the field by use of Google™ Earth imagery (2020), along with GPS coordinates. The approximate locations of these traverses have been approximated on a captured Google™ Earth image (2020) and on a partial copy of Sheet 5 of the preliminary RCFD Wildomar MPD Lateral C, Stage 3 plans (dated January 2020), as presented on Plates 1 and 2, respectively.

This opportunity to be of service is sincerely appreciated. If you should have questions regarding this report or do not understand the limitations of this study or the data and results that are presented, please do not hesitate to contact our office.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

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

TABLE OF CONTENTS

INTRODUCTION

The subject site is located at the southeast corner of Bundy Canyon Road and Monte Vista Drive, in the Wildomar area of Riverside County, California. Geomorphically, the subject study area is situated along a southwesterly gently-sloping alluvial fan that has been created predominantly by outwash originating from Bundy Canyon to the east. Locally, as shown on Figure 1 below, surficial mapping by the California Geological Survey (2010) indicates the site to be mantled by Holocene to late Pleistocene age surficial deposits, with the northern portion of the site being mantled by young alluvial fan deposits (map symbol Qyf) and the southern portion of the site mantled by young alluvial valley deposits (map symbol Qya) and alluvial wash deposits (map symbol Qw).

The alluvial fan deposits (Qyf) are described as consisting of unconsolidated to slightly-consolidated boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon. The alluvial valley deposits (Qya) are described as consisting of unconsolidated to slightly-consolidated clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers. The alluvial wash deposits (Qw) are described as consisting of unconsolidated sandy and gravelly sediment deposited in recently active stream channels. Underlying these surficial deposits at depth are cretaceous age granitic rocks and/or other intrusive crystalline rocks (map symbol gr).

For reference, the approximate locations of the seismic traverses are indicated as the red lines in Figure 1 below, with the survey area outlined in blue.

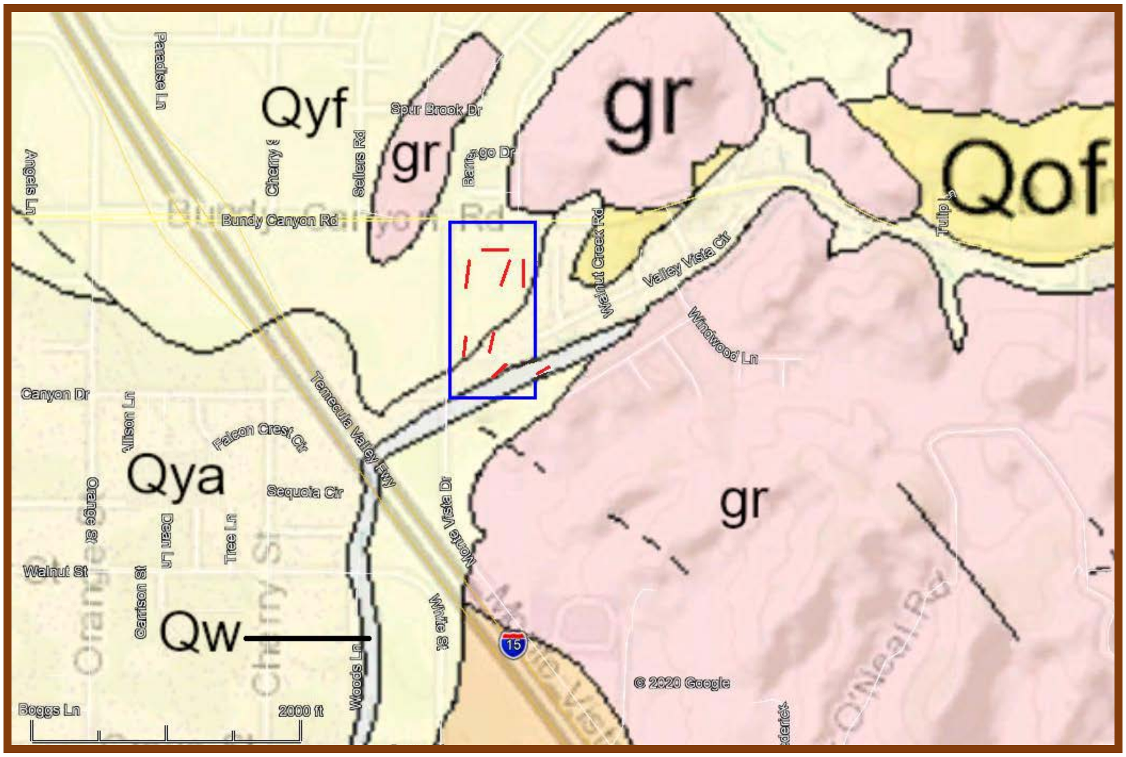


FIGURE 1- Geologic Map (CGS, 2010); Site outlined in blue, seismic traverses shown as red lines.

SEISMIC REFRACTION SURVEY

Methodology

The seismic refraction method consists of measuring (at known points along the surface of the ground) the travel times of compressional waves generated by an impulsive energy source and can be used to estimate the layering, structure, and seismic acoustic velocities of subsurface horizons. Seismic waves travel down and through the soils and rocks, and when the wave encounters a contact between two earth materials having different velocities, some of the wave's energy travels along the contact at the velocity of the lower layer. The fundamental assumption is that each successively deeper layer has a velocity greater than the layer immediately above it. As the wave travels along the contact, some of the wave's energy is refracted toward the surface where it is detected by a series of motion-sensitive transducers (geophones). The arrival time of the seismic wave at the geophone locations can be related to the relative seismic velocities of the subsurface layers in feet per second (fps), which can then be used to aid in interpreting both the depth and type of materials encountered.

Field Procedures

Eight seismic refraction survey lines (Seismic Lines S-1 through S-8 have been performed across the locations as selected by you. The traverses were located in the field by use of Google™ Earth imagery (2020), along with GPS coordinates, and have been delineated on the Seismic Line Location Map, as presented on Plates 1 and 2. These traverses ranged from 125- to 200-feet in length, which consisted of a total of twenty-four 14-Hertz geophones, spaced at regular five- to eight-foot intervals, in order to detect both the direct and refracted waves. A 16-pound sledge-hammer was used as the energy source to produce the seismic waves.

Seven shot points were utilized along each spread using forward, reverse, and several intermediate locations in order to obtain high resolution survey data for velocity analysis and depth modeling purposes. Multiple hammer impacts were utilized at each shot point location in order to increase the signal to noise ratio, which enhanced the primary seismic “P”-waves. The seismic wave arrivals were digitally recorded in SEG-2 format on a Geometrics StrataVisor™ NZXP model signal enhancement refraction seismograph. The data was acquired using a sampling rate of 0.0625 milliseconds having a record length of 0.08 to 0.12 seconds. No acquisition filters were used during data collection.

During acquisition, the seismograph displays the seismic wave arrivals on the computer screen which were used to analyze the arrival time of the primary seismic “P”-waves at each geophone station, in the form of a wiggle trace for quality control purposes in the field. If spurious “noise” was observed, the shot location was resampled during relatively quieter periods. Each geophone and seismic shot location were surveyed using a hand level and ruler for topographic correction, with “0” being the lowest point along each survey line.

Data Processing

The recorded seismic data was subsequently transferred to our office computer for processing and analyzing purposes, using the computer programs **SIPwin** (**S**eismic Refraction **I**nterpretation **P**rogram for **Win**dows) developed by Rimrock Geophysics, Inc. (2004) and **Refractor** (Geogiga, 2001-2019). All of the computer programs perform their individual analyses using exactly the same input data, which includes the first-arrival times of the "P"-waves and the survey line geometry.

- **SIPwin** is a ray-trace modeling program that evaluates the subsurface using layer assignments based on time-distance curves and is better suited for layered media, using the "Seismic Refraction Modeling by Computer" method (Scott, 1973). The first step in the modeling procedure is to compute layer velocities by least-squares techniques. Then the program uses the delay-time method to estimate depths to the top of layer-2. A forward modeling routine traces rays from the shot points to each geophone that received a first-arrival ray refracted along the top of layer-2. The travel time of each such ray is compared with the travel time recorded in the field by the seismic system. The program then adjusts the layer-2 depths so as to minimize discrepancies between the computed ray-trace travel times and the first arrival times picked from the seismic waveform record. The process of ray tracing and model adjustment is repeated a total of six times to improve the accuracy of depths to the top of layer-2. This first-arrival picks were then used to generate the Layer Velocity Model using the **SIPwin** computer program, which presents the subsurface velocities as individual layers and is presented within Appendix A for reference. In addition, the associated Time-Distance Plot, which shows the individual data picks of the first "P-wave" arrival times, also appears in Appendix A.

- **Refractor** is seismic refraction software that also evaluates the subsurface using layer assignments utilizing interactive and interchangeable analytical methods that include the Delay-Time method, the ABC method, and the Generalized Reciprocal Method (GRM). These methods are used for defining irregular non-planar refractors and are briefly described below. The Delay-Time method will measure the delay time depth to a refractor beneath each geophone rather than at shot points. Delay-time is the time spent by a wave to travel up or down through the layer (slant path) compared to the time the wave would spend if traveling along the projection of the slant path on the refractor. The ABC (intercept time) method makes use of critically refracted rays converging on a common surface position. This method involves using three surface to surface travel times between three geophones and the velocity of the first layer in an equation to calculate depth under the central geophone and is applied to all other geophones on the survey line. The GRM method is a technique for delineating undulating refractors at any depth from in-line seismic refraction data consisting of forward and reverse travel-times and is capable of resolving dips of up to 20% and does not over-smooth or average the subsurface refracting layers. In addition, the technique provides an approach for recognizing and compensating for hidden layer conditions.

The combined use of these seismic refraction computer programs provided a more thorough and comprehensive analysis of the subsurface structure and velocity characteristics. **SIPwin** and **Refractor** were primarily used for detecting generalized subsurface velocity layers providing “weighted average velocities.” The processed seismic data of these two programs were then combined and averaged to provide a final composite layer velocity model which provided a more thorough representation of the subsurface (see Appendix A).

SUMMARY OF GEOPHYSICAL INTERPRETATION

It is important to consider that the seismic velocities obtained within bedrock materials are influenced by the nature and character of the localized major structural discontinuities (foliation, fracturing, relic bedding, etc.), creating anisotropic conditions. Anisotropy (direction-dependent properties of materials) can be caused by “micro-cracks,” jointing, foliation, layered or inter-bedded rocks with unequal layer stiffness, small-scale lithologic changes, etc. (Barton, 2007). Velocity anisotropy complicates interpretation and it should be noted that the seismic velocities obtained during this survey may have been influenced by the nature and character of any localized structural discontinuities within the bedrock underlying the subject site.

Generally, it is expected that higher (truer) velocities will be obtained when the seismic waves propagate along the direction (strike) of the dominant structure, with a damping effect when the seismic waves travel in a perpendicular direction. Such variable directions can result in velocity differentials of between 2% to 40% depending upon the degree of the structural fabric (i.e., weakly-moderately-strongly jointed, respectively).

The computer programs described above (**SIPwin** and **Refractor**) that were used for the data analysis, produces the traditional layer models that are defined by seismic velocity boundaries. It should be understood that the data obtained represents an average of seismic velocities within any given layer. For example, high seismic velocity boulders, dikes, or other local lithologic inconsistencies, may be isolated within a low velocity matrix, thus yielding an average medium velocity for that layer. Therefore, in any given layer, a range of velocities could be anticipated, which can also result in a wide range of excavation characteristics.

In general, the subject site where locally surveyed, was noted to be characterized by three major subsurface layers (Layers V1, V2, and V3) with respect to seismic velocities. The following velocity layer summaries have been prepared with respect to the **SIPwin** and **Refractor** analysis, with the representative Layer Velocity Models being presented within Appendix A, along with the respective Time-Distance Plots for reference. The Time-Distance plots, also referred to as “travel-time curves” display the time it takes (in milliseconds) for the induced seismic waves (shot points) to arrive at each of the seismic receivers (geophones), with respect to their location along the survey line (distance, in feet).

☐ **Velocity Layer V1**: The surficial layer (V1) yielded a seismic velocity range of 823 to 1,382 fps, which is presumed to be comprised of variable younger (Holocene age) alluvial deposits and/or localized artificial fill. This velocity range is typical for these types of unconsolidated surficial earth materials.

☐ **Velocity Layer V2**: The second layer (V2) has a seismic velocity range of 1,594 to 2,333 fps, which is believed to be comprised of older alluvial deposits. This velocity range is typical for older (Pleistocene age) alluvial deposits that are generally more consolidated and/or indurated.

☐ **Velocity Layer V3:** The third layer (V3) indicates the presence of highly- to moderately-weathered granitic bedrock, having a seismic velocity range of 4,334 to 8,007 fps. This wide range of velocities is most likely due to the degree of the bedrock weathering at depth and possibly may be locally higher where the presence of scattered buried relatively fresher large crystalline boulders is contained within a surrounding relatively less-weathered matrix. In addition to granitic rock, seismic velocities typically ranging from 5,000 to 6,500± feet are also representative for saturated sediments, indicating the possibility of a groundwater table.

Table 1 below summarizes the results of the survey lines with respect to the "weighted average" seismic velocities for each layer, as discussed above.

TABLE 1- VELOCITY SUMMARY OF SEISMIC SURVEY LINES

Seismic Line	V1 Layer (fps)	V2 Layer (fps)	V3 Layer (fps)
S-1	**1,382**	**1,996**	**4,334**
S-2	**978**	**1,843**	**5,215**
S-3	**1,065**	**2,333**	**7,342**
S-4	**1,347**	**1,838**	**5,432**
S-5	**839**	**2,196**	**4,367**
S-6	**989**	**1,594**	**8,007**
S-7	**823**	**1,711**	**6,318**
S-8	**-----**	**1,718**	**5,698**

GENERALIZED RIPPABILITY CHARACTERISTICS OF BEDROCK

A summary of the generalized rippability characteristics of bedrock based on a compilation of rippability performance charts prepared by Caterpillar, Inc. (2018; see Figure 2, Page 8), Caltrans (Stephens, 1978), and Santi (2006), has been provided to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas surveyed. These seismic velocity ranges and rippability potentials have been tabulated below for reference.

TABLE 2- CATERPILLAR RIPPABILITY CHART (D9 Ripper)

Granitic Rock Velocity	**Rippability**
< 6,800	Rippable
6,800 – 8,000	Moderately Rippable
> 8,000	Non-Rippable

Additionally, we have provided the Caltrans Rippability Chart as presented below within Table 2 for comparison. These values are from published Caltrans studies (Stephens, 1978) that are based on their experience and which appear to be more conservative than Caterpillar's rippability chart. It should be noted that the type of bedrock was not indicated.

TABLE 3- STANDARD CALTRANS RIPPABILITY CHART

Velocity (feet/sec ±)	**Rippability**
< 3,500	Easily Ripped
3,500 – 5,000	Moderately Difficult
5,000 – 6,600	Difficult Ripping / Light Blasting
> 6,600	Blasting Required

Table 3 is partially modified from the "Engineering Behavior from Weathering Grade" as presented by Santi (2006), which also provides velocity ranges with respect to rippability potentials, along with other rock engineering properties that may be pertinent.

TABLE 4- SUMMARY OF ROCK ENGINEERING PROPERTIES

ENGINEERING PROPERTY:	Slightly Weathered	Moderately Weathered	Highly Weathered	Completely Weathered
Excavatability	Blasting necessary	Blasting to rippable	Generally rippable	Rippable
Slope Stability	½ :1 to 1:1 (H:V)	1:1 (H:V)	1:1 to 1.5:1 (H:V)	1.5:1 to 2:1 (H:V)
Schmidt Hammer Value	51 – 56	37 – 48	12 – 21	5 – 20
Seismic Velocity (fps)	8,200 – 13,125	5,000 – 10,000	3,300 – 6,600	1,650 – 3,300

The Caterpillar D9R Ripper Performance Chart (Caterpillar, 2018) has been provided on Figure 2 below for reference.

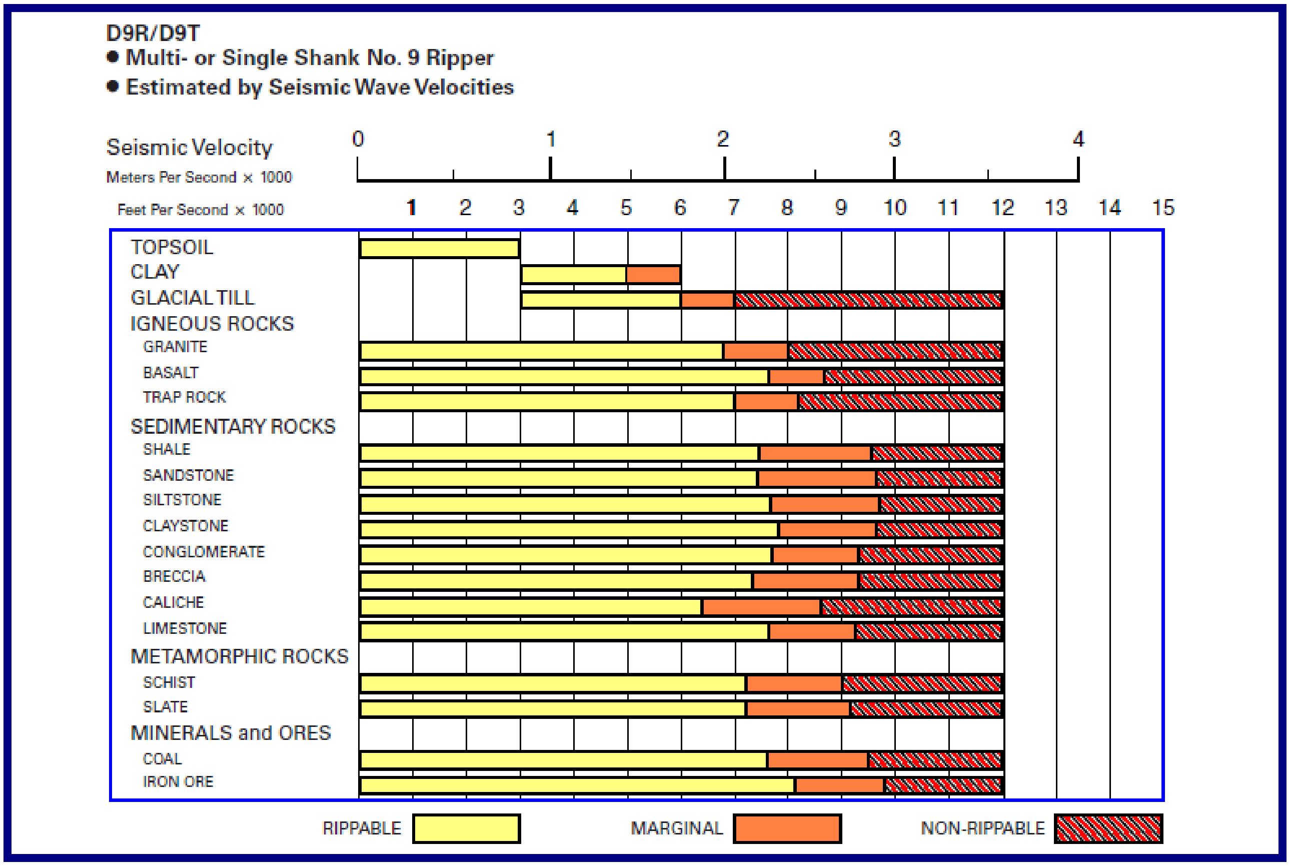


FIGURE 2- Caterpillar D9R Ripper Performance Chart (2018).

For purposes of the discussion in this report with respect to the expected bedrock rippability characteristics, we are assuming that a D9R/D9T dozer will be used as a minimum, such as discussed further below and as shown in Figure 2 above. Smaller excavating equipment will most likely result in slower production rates and possible refusal within relatively lower velocity bedrock materials. It should be noted that the decision for blasting of bedrock materials for facilitating the excavation process is sometimes made based upon economic production reasons and not solely on the rippability (velocity/hardness) characteristics of the bedrock.

A summary of the generalized rippability characteristics of granitic bedrock (such as present within the subject study area) has been provided below to aid in evaluating potential excavation difficulties with respect to the seismic velocities obtained along the local areas that were surveyed. The velocity ranges described below are general averages of Tables 2 and 3 presented in this report (see Page 7) and assume typical, good-working, heavy excavation equipment, such as D9R dozer using a single shank, as described by Caterpillar, Inc. (2000 and 2018).

However, different excavating equipment (i.e., trenching equipment) may not correlate well with these velocity ranges as the rippability performance charts are tailored for conventional bulldozer equipment and cannot be directly correlated. Trenching operations which utilize large excavator-type equipment within granitic bedrock materials, typically encounter very difficult to non-productable conditions where seismic velocities are generally greater than 4,000± fps, and less for smaller backhoe-type equipment. These average seismic velocity ranges are summarized below:

❑ **Rippable Condition (0 - 4,000 ft/sec)**:

This velocity range indicates rippable materials which may consist of alluvial-type deposits and decomposed granitic bedrock, with random hardrock floaters. These materials typically break down into silty sands (depending on parent lithologic materials), whereas floaters will require special disposal. Some areas containing numerous hardrock floaters may present utility trench problems. Large floaters exposed at or near finished grade may present problems for footing or infrastructure trenching.

Marginally Rippable Condition (4,000 - 7,000 ft/sec):

This range of seismic velocities indicates materials which may consist of moderately weathered bedrock and/or large areas of fresh bedrock materials separated by weathered fractured zones. These bedrock materials are generally rippable with difficulty by a Caterpillar D9R or equivalent. Excavations may produce material that will partially break down into a coarse, silty to clean sand, with a high percentage of very coarse sand to pebble-sized material depending on the parent bedrock lithology. Less fractured or weathered materials will probably require blasting to facilitate removal.

- **Non-Rippable Condition (7,000 ft/sec or greater)**:

  This velocity range includes non-rippable material consisting primarily of moderately fractured bedrock at lower velocities and only slightly fractured or unfractured rock at higher velocities. Materials in this velocity range may be marginally rippable, depending upon the degree of fracturing and the skill and experience of the operator. Tooth penetration is often the key to ripping success, regardless of seismic velocity. If the fractures and joints do not allow tooth penetration, the material may not be ripped effectively; however, pre-blasting or "popping" may induce sufficient fracturing to permit tooth entry. In their natural state, materials with these velocities are generally not desirable for building pad grade, due to difficulty in footing and utility trench excavation. Blasting will most likely produce oversized material, requiring special disposal.

GEOLOGIC & EARTHWORK CONSIDERATIONS

To evaluate whether a particular bedrock material can be ripped or excavated, this geophysical survey should be used in conjunction with the geologic and/or geotechnical report and/or information gathered for the subject project which may describe the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults, and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification or lamination, large grain size, moisture permeated clay, and low compressive strength. If the bedrock is foliated and/or fractured at depth, this structure could aid in excavation production. Unfavorable bedrock conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic. Use of these physical bedrock conditions along with the subsurface velocity characteristics as presented within this report should aid in properly evaluating the type of equipment that will be necessary and the production levels that can be anticipated for this project. A summary of excavation considerations is included within Appendix B in order to provide you and your grading contractor with a better understanding of the complexities of excavation in bedrock materials, so that proper planning and excavation techniques can be employed.

SUMMARY OF FINDINGS AND CONCLUSIONS

The raw field data was considered to be of moderate to good quality with variable amounts of ambient “noise” that was introduced during our survey, originating predominantly from vehicular traffic along Bundy Canyon Road to the north, Monte Vista Drive to the west, and the Interstate 15 Freeway slightly farther to the west. Analysis of the data and picking of the primary “P”-wave arrivals was therefore performed with some difficulty and minor interpolation of some data points was necessary.

Based on the results of our comparative seismic analyses of the computer programs **SIPwin** and **Refractor**, the seismic refraction survey line models appear to generally coincide with one another. The anticipated excavation potentials of the velocity layers encountered locally during our survey are as follows:

- ☐ **Velocity Layer V1**:

  No excavating difficulties are expected to be encountered within the uppermost, low-velocity V1 layer (average weighted velocity of 823 to 1,382 fps) and should excavate with conventional ripping. This surficial velocity layer is expected to be comprised of variable unconsolidated alluvial deposits and/or localized artificial fill.

- ☐ **Velocity Layer V2**:

  No excavating difficulties are expected to be encountered within the second V2 layer (average weighted velocity of 1,594 to 2,333 fps), which is believed to consist of older alluvial deposits that are generally more consolidated and/or indurated. The possibility of compacted artificial fill in this layer cannot be ruled out. The upper, lower-velocity layer encountered within Seismic Line S-8, most likely consists of overlying compacted fill (for the levee construction), underlain at depth by younger and/or older alluvium of unknown thicknesses. This is based on the Time-Distance Plot which indicate the near-surface "P"-Wave arrivals being faster, which then slow down until the deeper, high-velocity layer is encountered. This condition creates a "seismic blind zone" wherein an underlying lower-velocity layer cannot be detected, which can result in erroneous depth calculations to all contact interfaces below it and therefore, the V2/V3 contact boundary below Seismic Line S-8 may not be accurate. It should be noted that the upper seismic velocity layer for Seismic Line S-8 (V1/V2) is an overall average of artificial fill and younger and older alluvial materials.

- ☐ **Velocity Layer V3**:

  The third V3 layer is believed to consist of highly- to moderately-weathered granitic bedrock. Moderate to hard excavation difficulties within this velocity layer (average weighted velocity range of 4,334 to 8,007 fps) should be anticipated if encountered during grading. This layer may consist of relatively homogeneous bedrock with wide-spaced fracturing, or may contain higher velocity scattered corestones, dikes, and other lithologic variables, within a relatively lower velocity bedrock matrix. Although not anticipated, localized blasting may be necessary to achieve desired grade, including any infrastructure. Caterpillar (2018; see Figure 2) indicates this velocity range to be "rippable to marginally-rippable" using a D9R dozer or equivalent. Larger equipment may facilitate excavation potentials within this higher velocity layer.

  It should be noted that the V3 layer encountered within Seismic Lines S-7 and S-8 may suggest saturated sediments with the V2/V3 contact boundary possibly representing the top of the groundwater table locally.

CLOSURE

The field geophysical survey was performed on May 23 and July 11, 2020 by the undersigned using "state of the art" geophysical equipment and techniques along the selected traverse locations. The seismic data was further evaluated using recently developed computerized tomographic inversion techniques to provide a more thorough analysis and understanding of the subsurface velocity and structural conditions. It should be noted that our data presented within this report was obtained along eight specific locations therefore other areas in the local vicinity may contain different velocity layers and depths not encountered during our field survey.

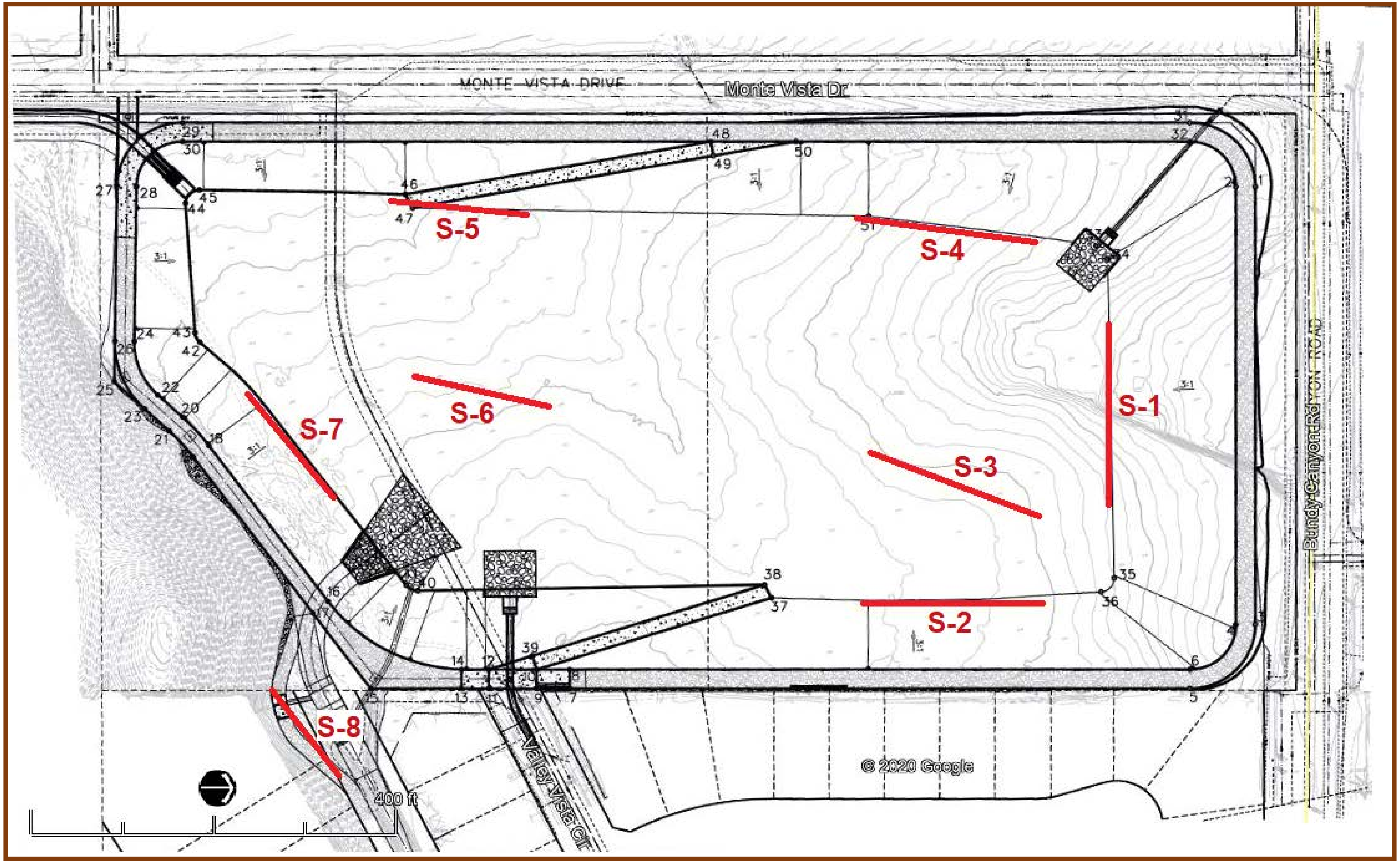
It is important to understand that the fundamental limitation for seismic refraction surveys is known as nonuniqueness, wherein a specific seismic refraction data set does not provide sufficient information to determine a single “true” earth model. Therefore, the interpretation of any seismic data set uses “best-fit” approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. Estimates of layer velocity boundaries as presented in this report are generally considered to be within 10± percent of the total depth of the contact.

Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained, and in the interpretation, and that the results of this survey may not represent actual subsurface conditions. These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied.

GOOGLE™ EARTH IMAGERY MAP

Google™ Earth (2020); Seismic survey lines (S-1 through S-7) shown as yellow lines.

SEISMIC LINE LOCATION MAP



Base Map: RCFC Wildomar MDP Lateral 3, Stage 3 (Preliminary Sheet 5, partial copy); Seismic survey lines (S-1 through S-7) shown as red lines.

PROJECT NO. 203423-2

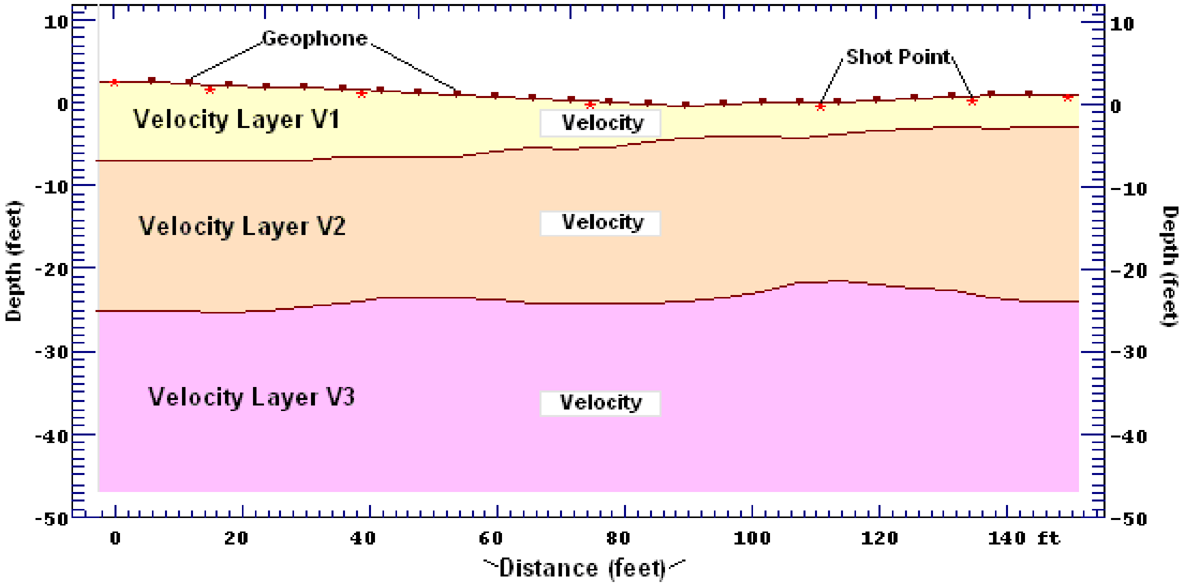
PLATE 2

APPENDIX A

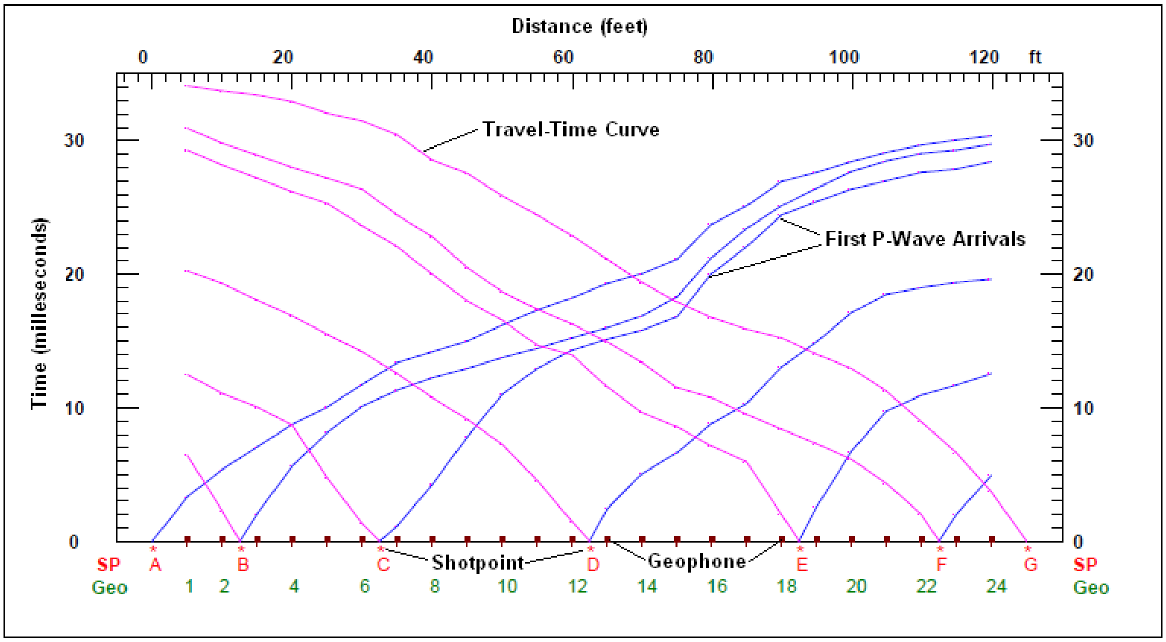
LAYER VELOCITY MODELS

LAYER VELOCITY MODEL LEGEND

LAYER VELOCITY MODEL



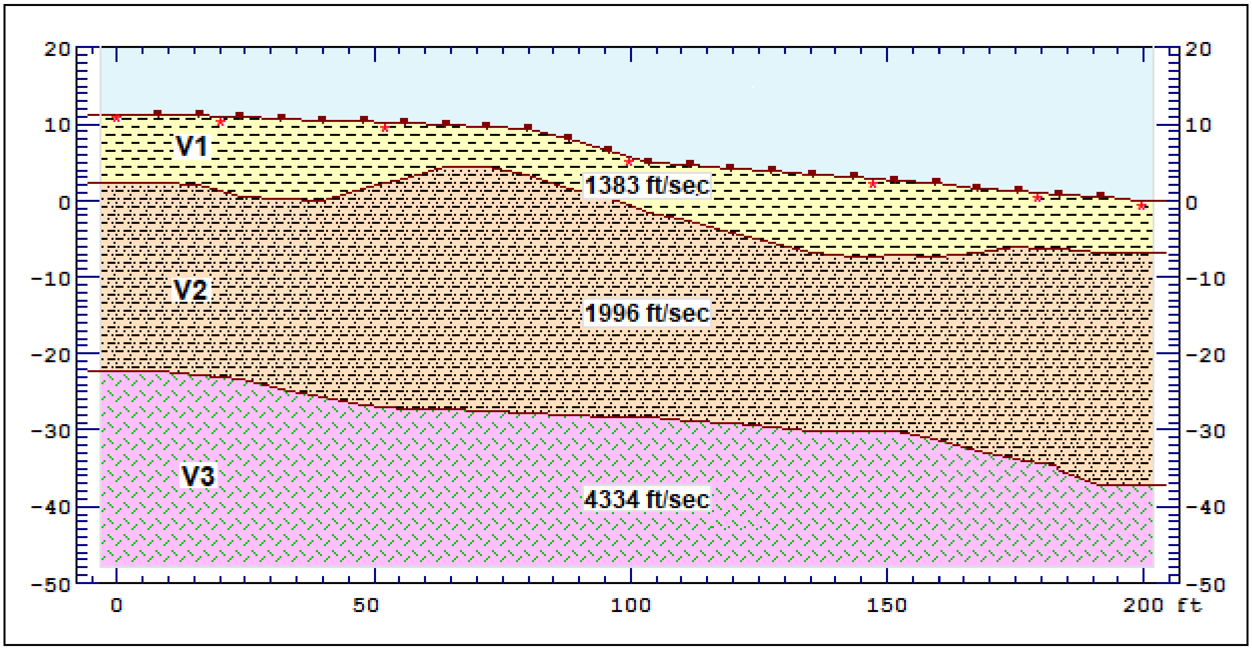
TIME-DISTANCE PLOT



SEISMIC LINE S-1

< West - East >

LAYER VELOCITY MODEL



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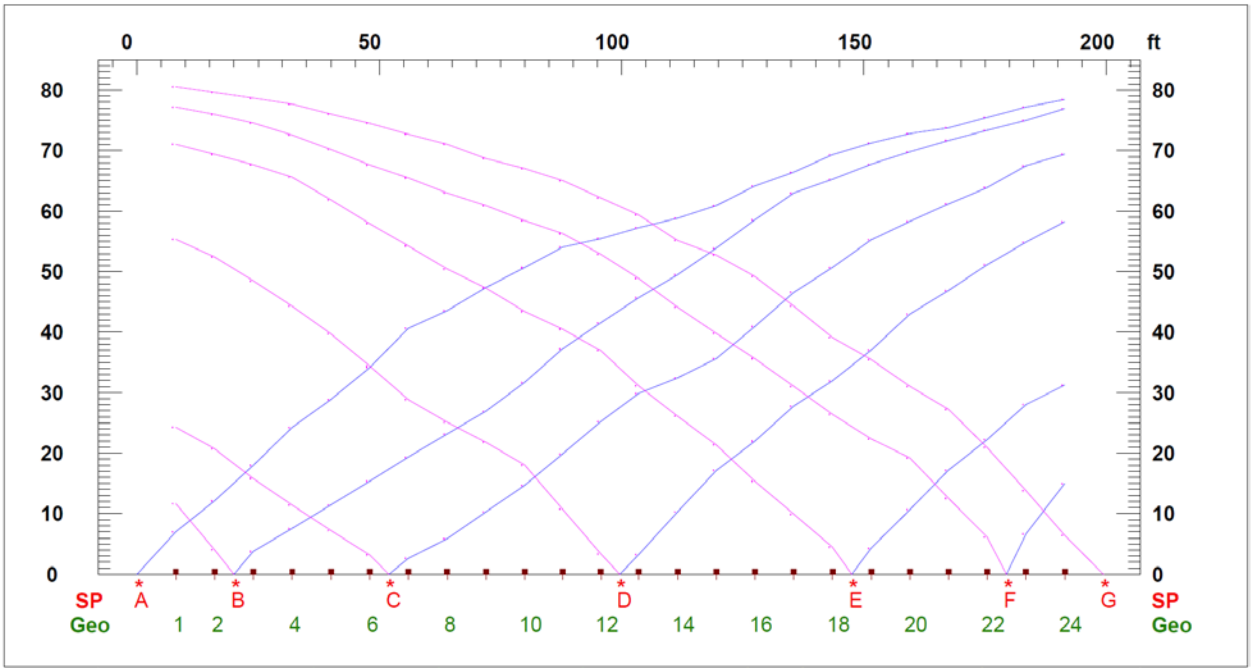
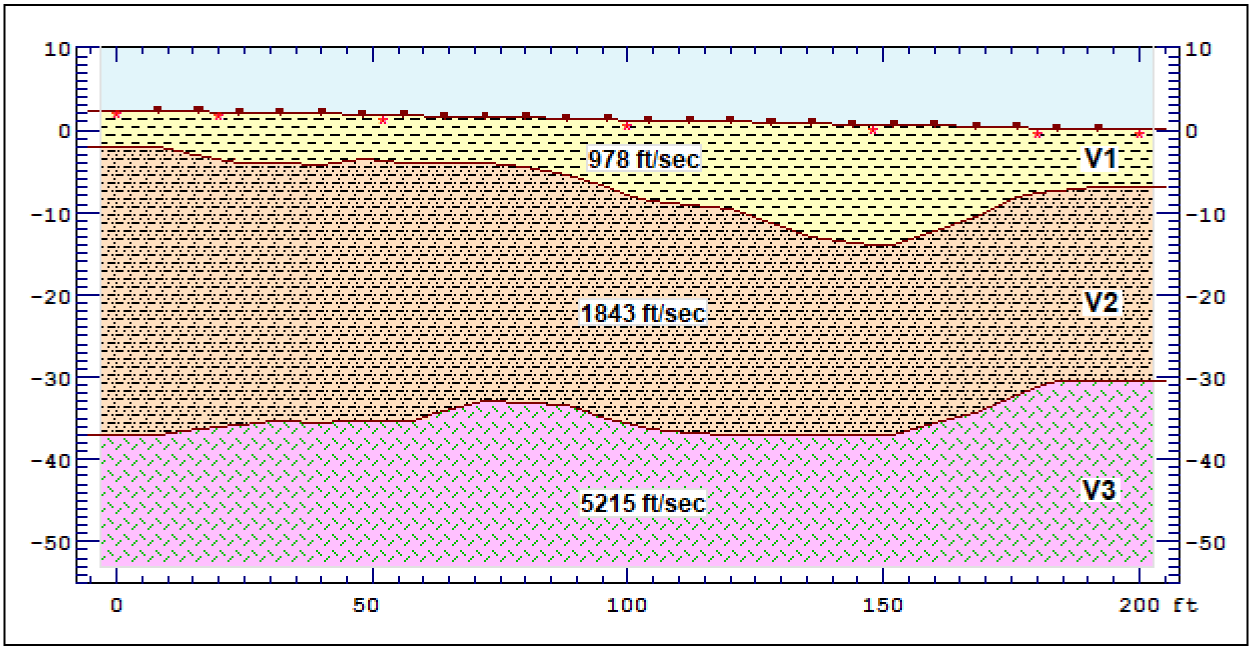


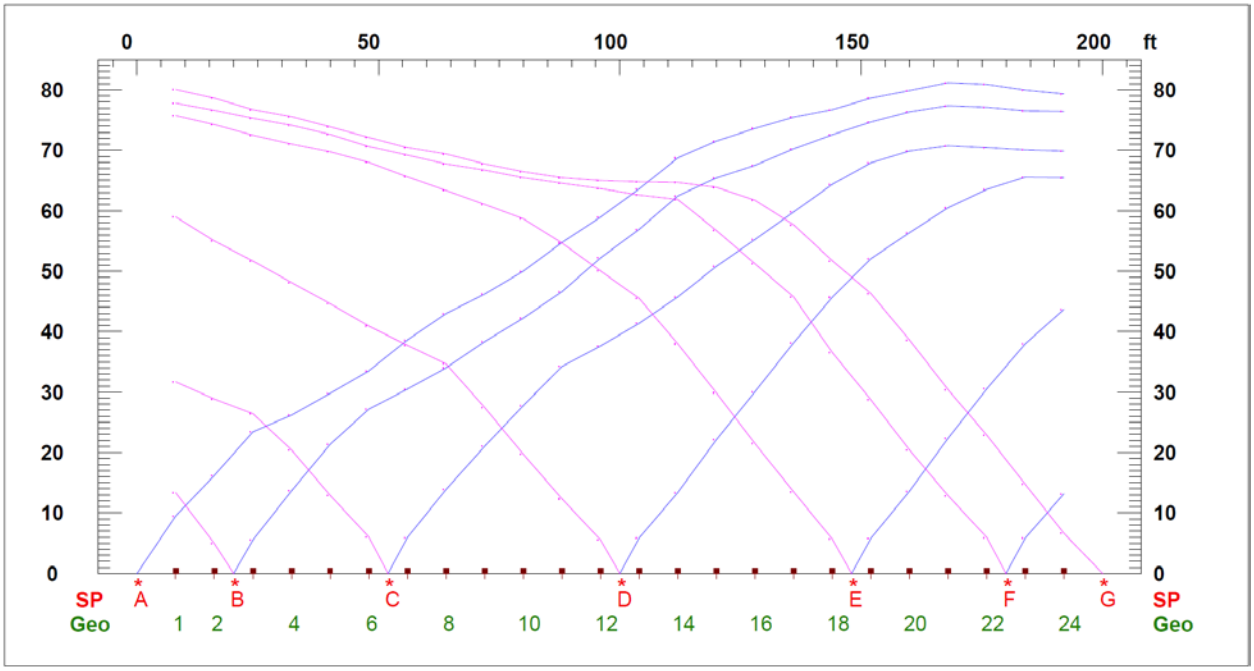
SEISMIC LINE S-2

< North - South >

LAYER VELOCITY MODEL



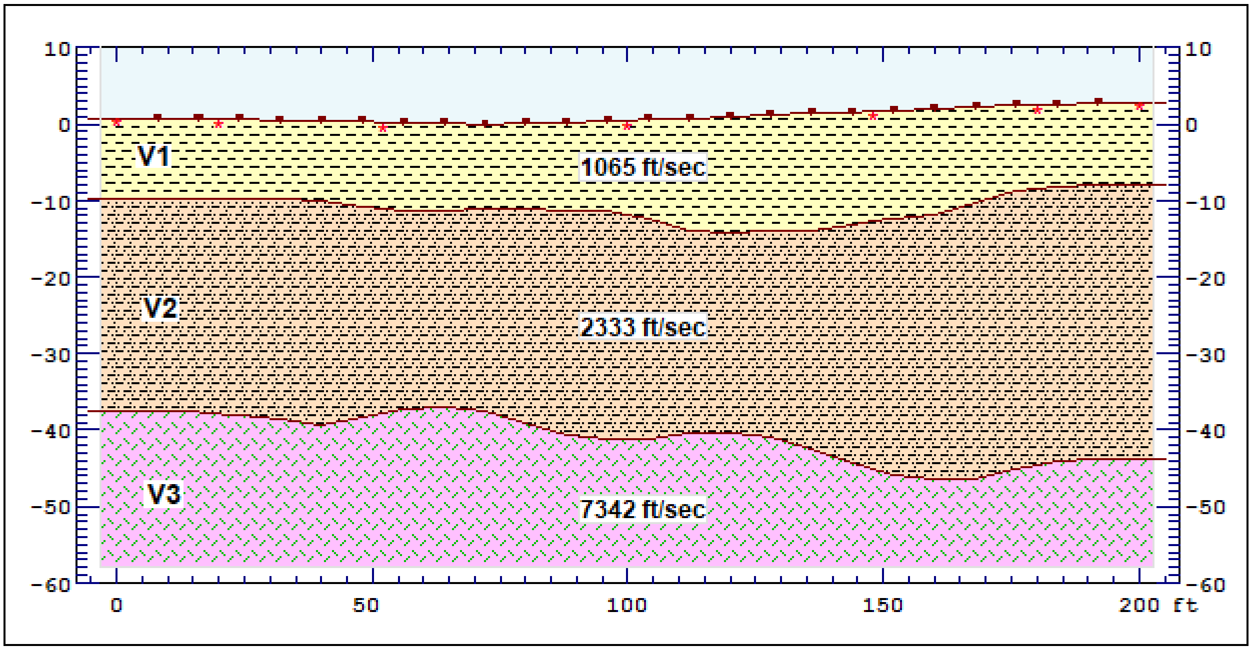
TIME-DISTANCE PLOT



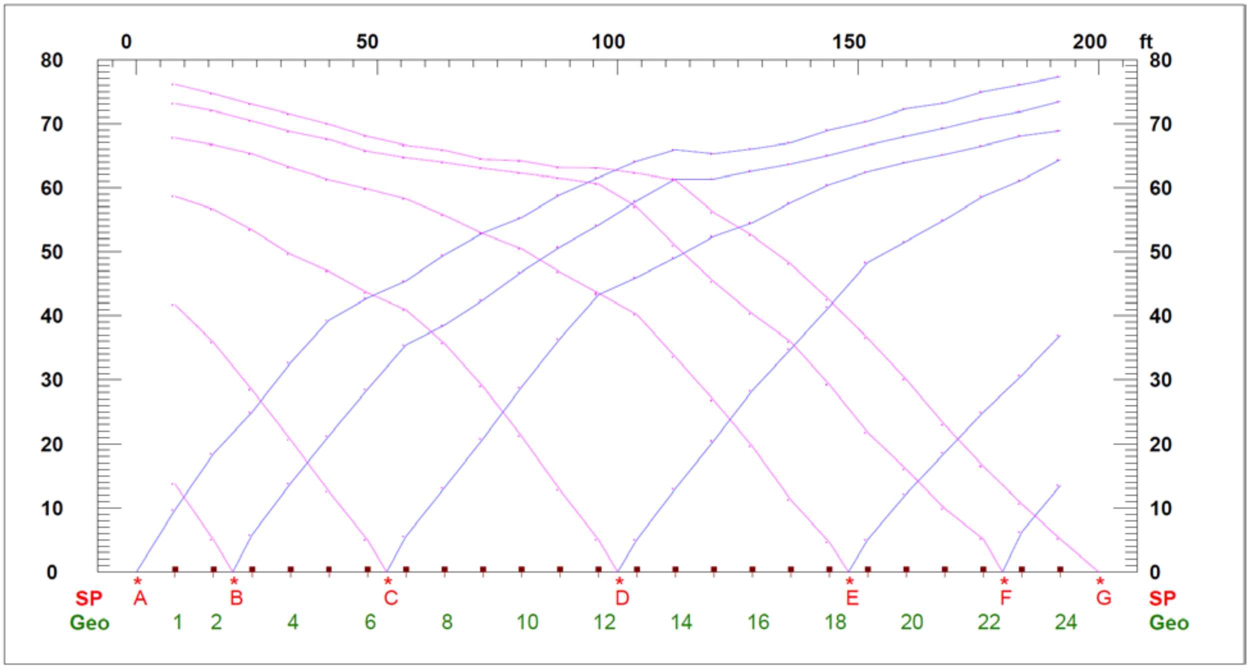
SEISMIC LINE S-3

South 20° West >

LAYER VELOCITY MODEL



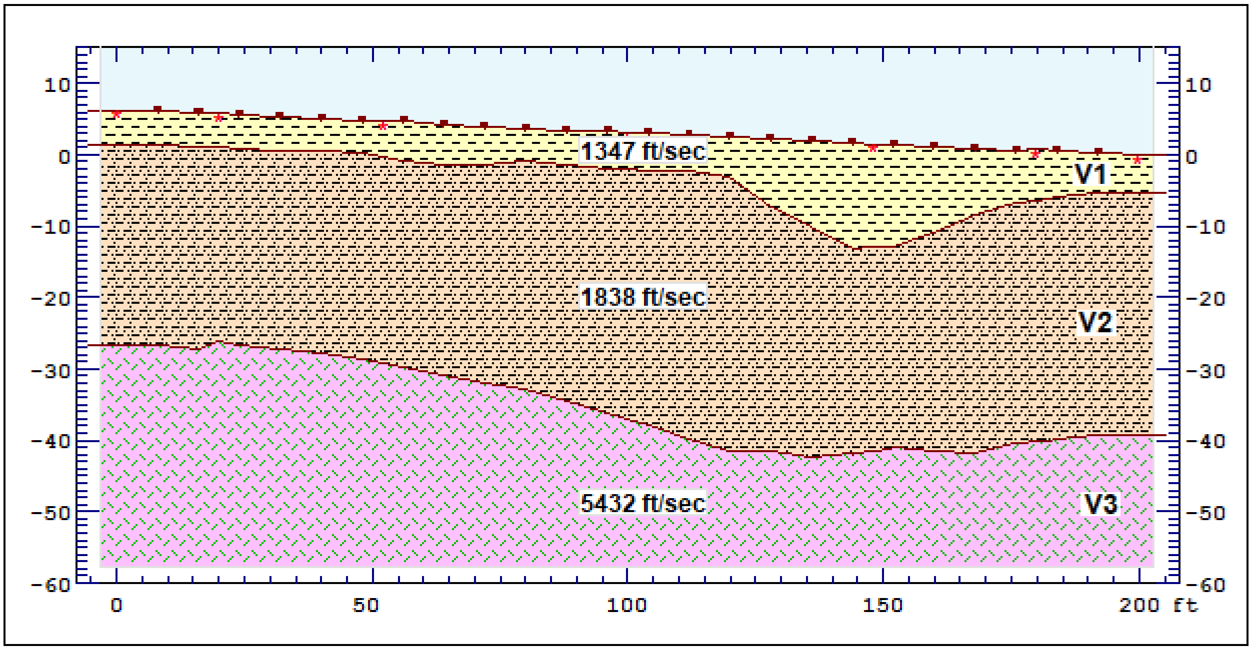
TIME-DISTANCE PLOT



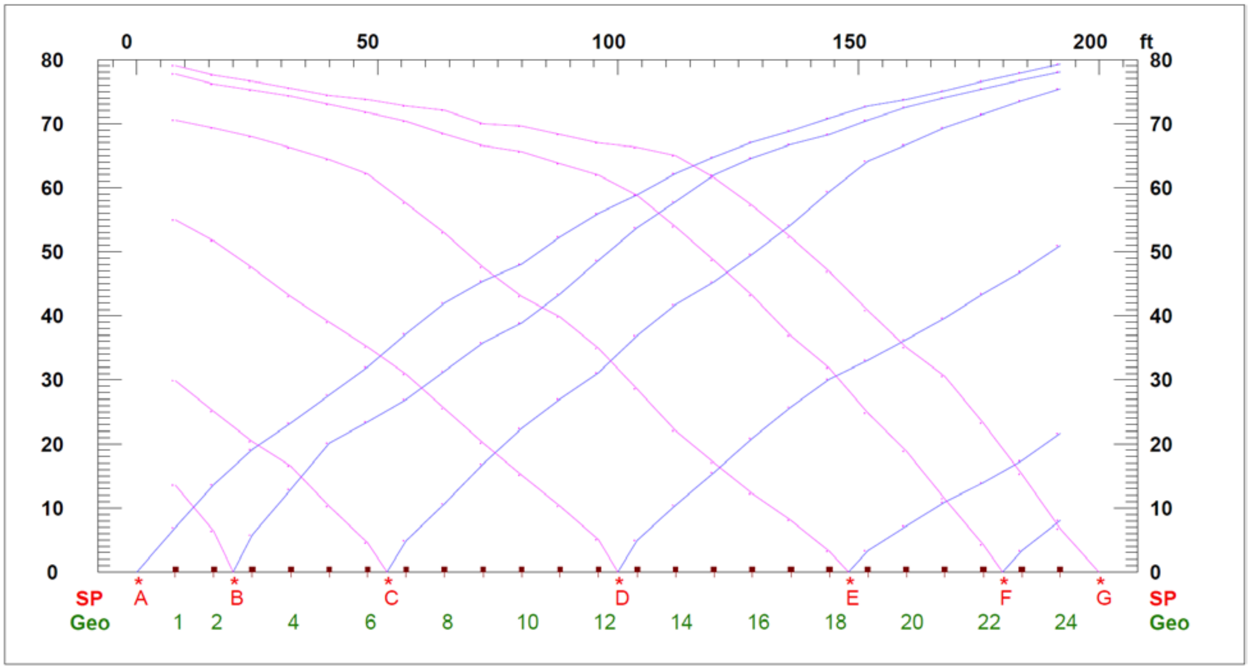
SEISMIC LINE S-4

South 8° West >

LAYER VELOCITY MODEL



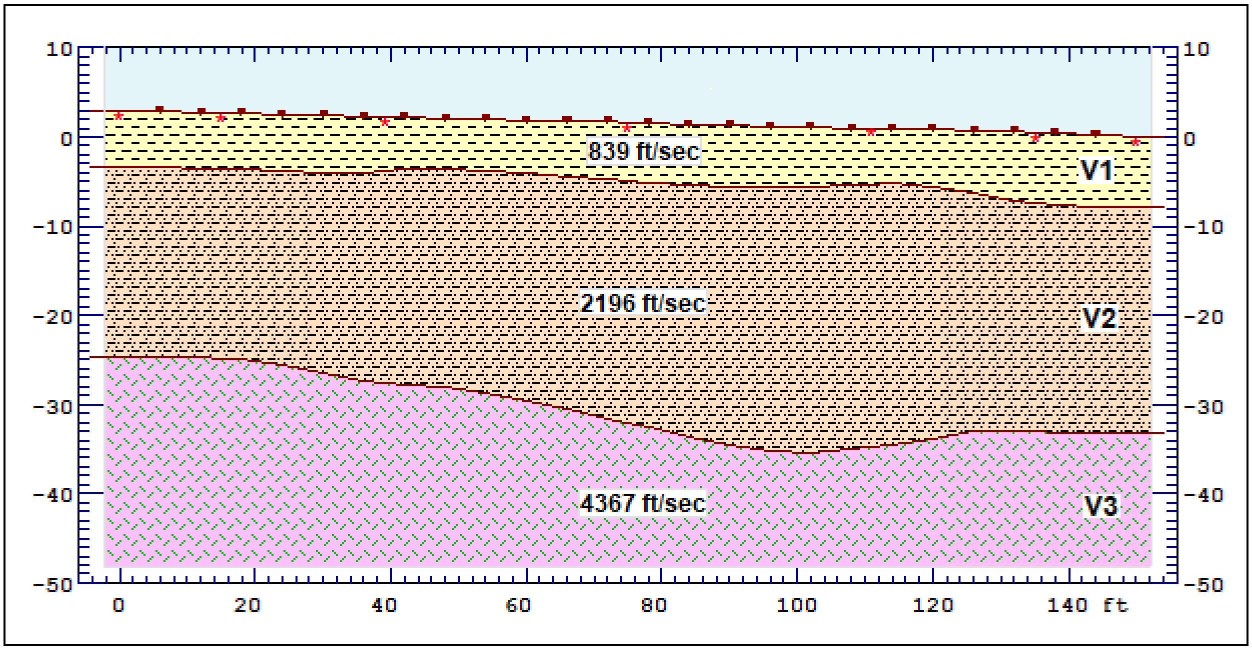
TIME-DISTANCE PLOT



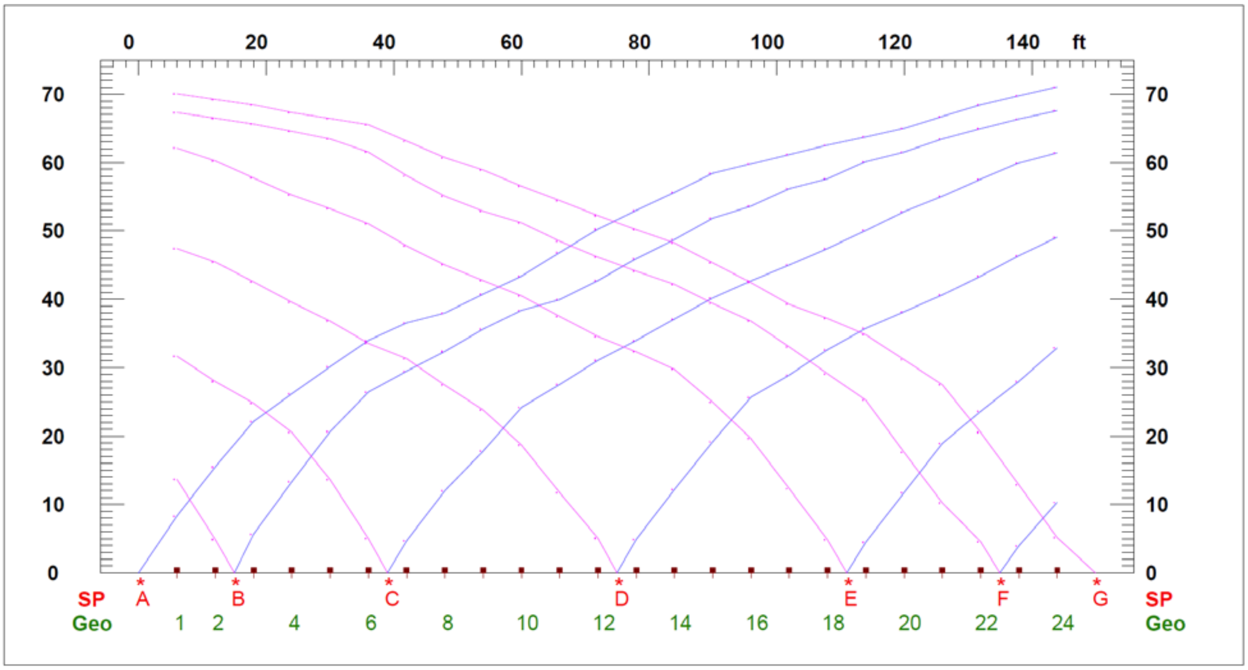
SEISMIC LINE S-5

South 6° West >

LAYER VELOCITY MODEL



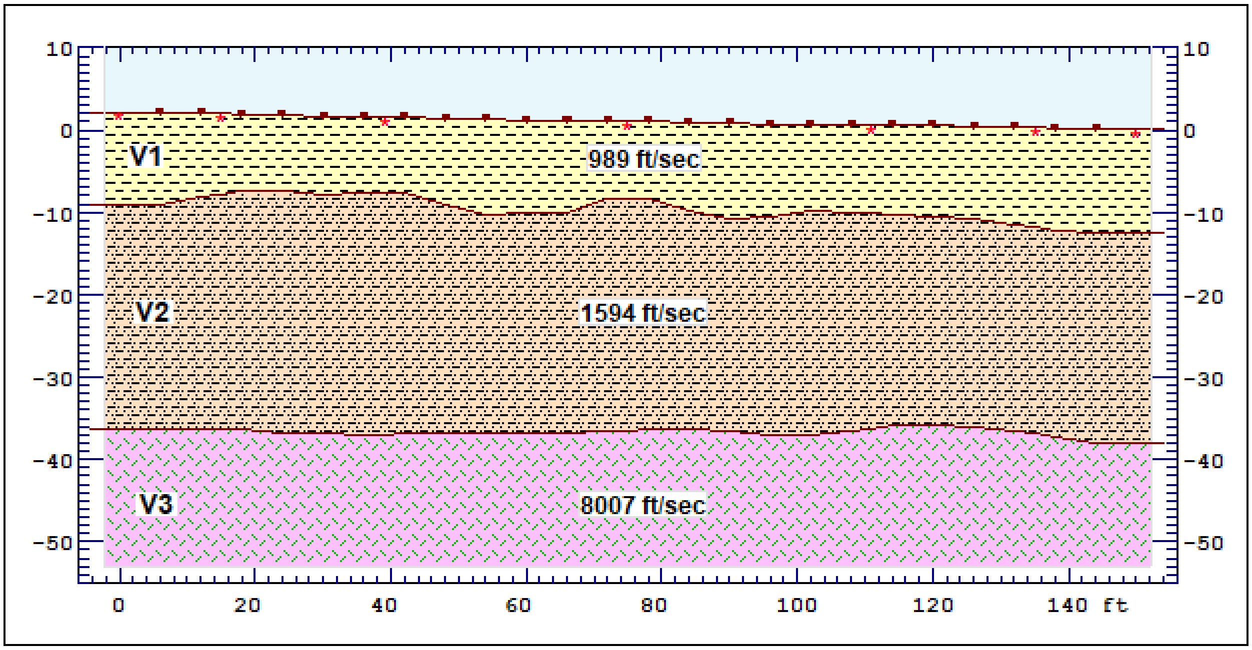
TIME-DISTANCE PLOT



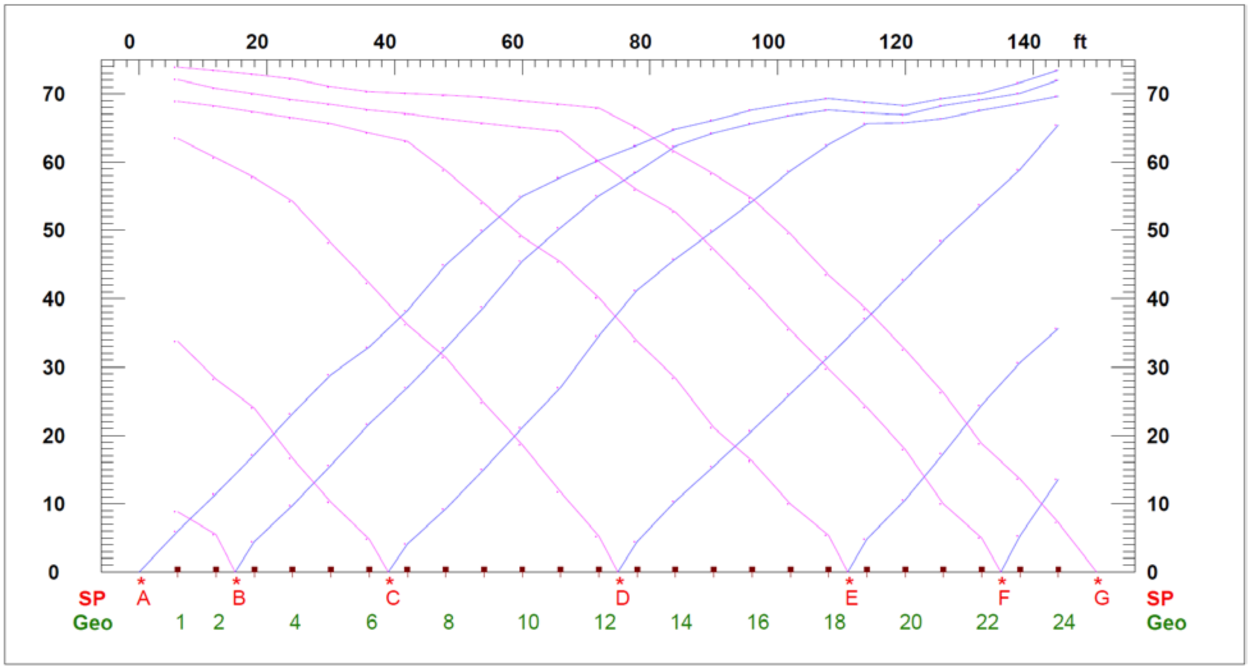
SEISMIC LINE S-6

South 13° West >

LAYER VELOCITY MODEL



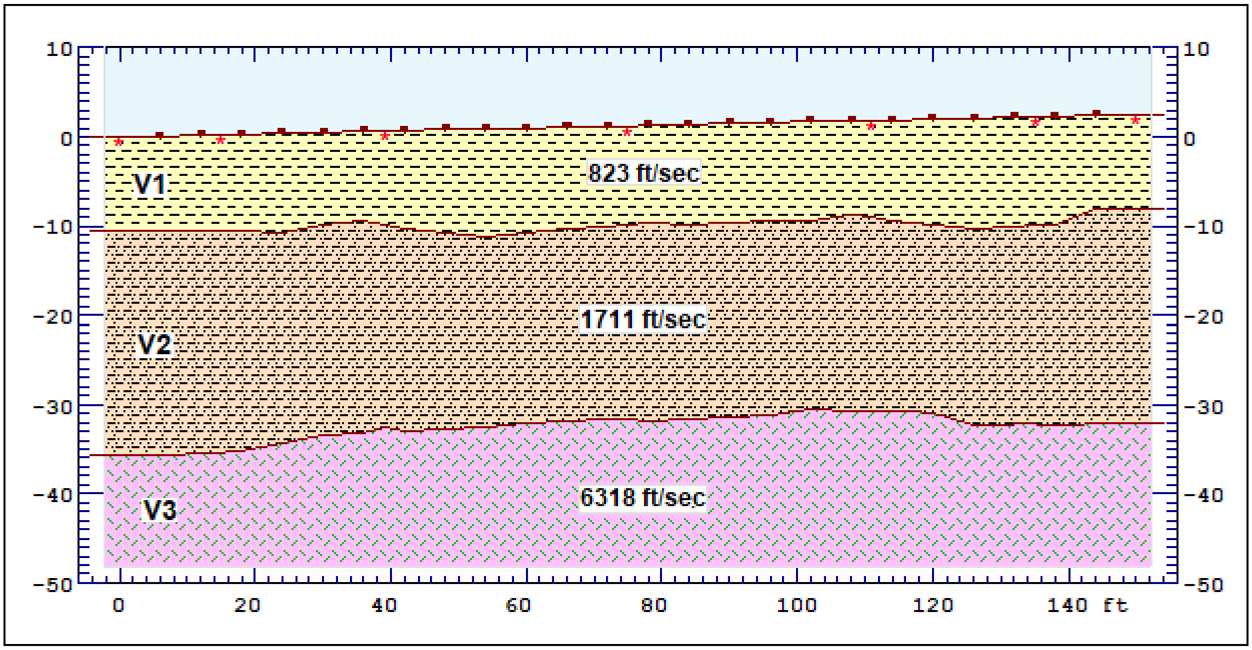
TIME-DISTANCE PLOT



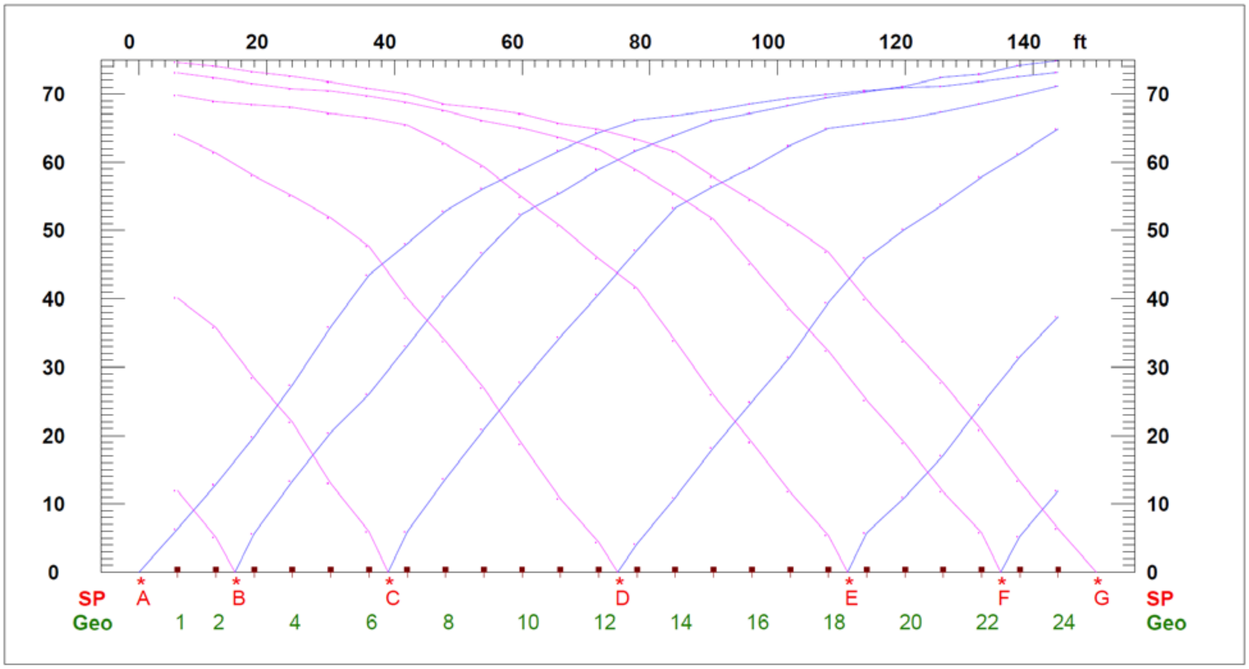
SEISMIC LINE S-7

North 50° East >

LAYER VELOCITY MODEL



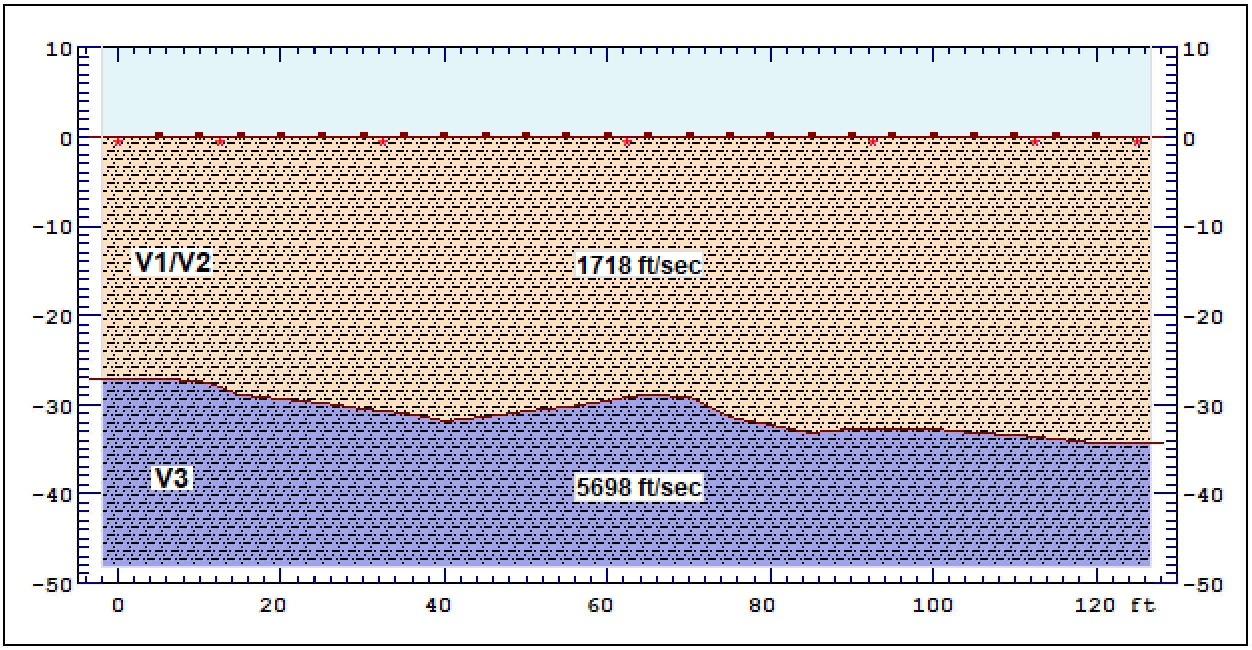
TIME-DISTANCE PLOT



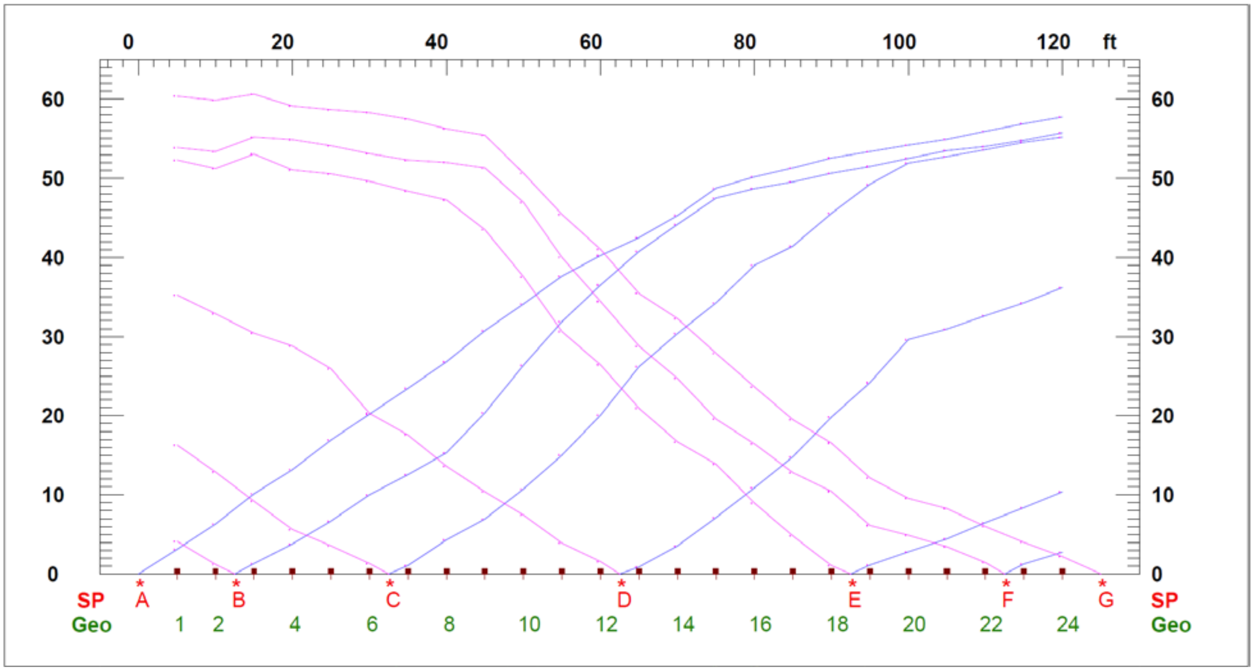
SEISMIC LINE S-8

North 53° East >

LAYER VELOCITY MODEL



TIME-DISTANCE PLOT



APPENDIX B

EXCAVATION CONSIDERATIONS

EXCAVATION CONSIDERATIONS

These excavation considerations have been included to provide the client with a brief overall summary of the general complexity of hard bedrock excavation. It is considered the clients responsibility to insure that the grading contractor they select is both properly licensed and qualified, with experience in hard-bedrock ripping processes. To evaluate whether a particular bedrock material can be ripped, this geophysical survey should be used in conjunction with the geologic or geotechnical report prepared for the project which describes the physical properties of the bedrock. The physical characteristics of bedrock materials that favor ripping generally include the presence of fractures, faults and other structural discontinuities, weathering effects, brittleness or crystalline structure, stratification of lamination, large grain size, moisture permeated clay, and low compressive strength. Unfavorable conditions can include such characteristics as massive and homogeneous formations, non-crystalline structure, absence of planes of weakness, fine-grained materials, and formations of clay origin where moisture makes the material plastic.

When assessing the potential rippability of the underlying bedrock of a given site, the above geologic characteristics along with the estimated seismic velocities can then be used to evaluate what type of equipment may be appropriate for the proposed grading. When selecting the proper ripping equipment there are three primary factors to consider, which are:

- ***Down Pressure available at the tip, which determines the ripper penetration that can be attained and maintained,***
- ***Tractor flywheel horsepower, which determines whether the tractor can advance the tip, and,***
- ***Tractor gross-weight, which determines whether the tractor will have sufficient traction to use the horsepower.***

In addition to selecting the appropriate tractor, selection of the proper ripper design is also important. There are basically three designs, being radial, parallelogram, and adjustable parallelogram, of which the contractor should be aware of when selecting the appropriate design to be used for the project. The penetration depth will depend upon the down-pressure and penetration angle, as well as the length of the shank tips (short, intermediate, and long).

Also important in the excavation process is the ripping technique used as well as the skill of the individual tractor operator. These techniques include the use of one or more ripping teeth, up- and down-hill ripping, and the direction of ripping with respect to the geologic structure of the bedrock locally. The use of two tractors (one to push the first tractor-ripper) can extend the range of materials that can be ripped. The second tractor can also be used to supply additional down-pressure on the ripper. Consideration of light blasting can also facilitate the ripper penetration and reduce the cost of moving highly consolidated rock formations.

All of the combined factors above should be considered by both the client and the grading contractor, to insure that the proper selection of equipment and ripping techniques are used for the proposed grading.

APPENDIX C

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REFERENCES

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APPENDIX I –
Soil Corrosivity Report

August 5, 2020 via email: dlind@inlandfoundation.com

INLAND FOUNDATION ENGINEERING
1310 South Santa Fe Ave.
San Jacinto, CA 92583

Attention: Mr. Dan Lind

Re: Soil Corrosivity Study
RCFCD Wildomar Sedimentation Basin
Wildomar, CA
HDR #20-0400SCS, IFE #R206-025

Introduction

Laboratory tests have been completed on six soil samples provided for the referenced project. The purpose of these tests was to determine if the soils might have deleterious effects on concrete structures and piping. HDR Engineering, Inc. (HDR) assumes that the samples provided are representative of the most corrosive soils at the site.

The proposed consists of a detention basin and related appurtenances. The site is located at the southeast corner of the intersection of Bundy Canyon Road and Monte Vista Drive in Wildomar, California, and groundwater was encountered as shallow as 26 feet deep.

The scope of this study is limited to a determination of soil corrosivity and general corrosion control recommendations for materials likely to be used for construction. HDR's recommendations do not constitute, and are not meant as a substitute for, design documents for the purpose of construction. If the architects and/or engineers desire more specific information, designs, specifications, or review of design, HDR will be happy to work with them as a separate phase of this project.

hdrinc.com

431 W. Baseline Road, Claremont, CA 91711-1608
(909) 626-0967

Laboratory Soil Corrosivity Tests

The electrical resistivity of each sample was measured in a soil box per ASTM G187 in its as-received condition and again after saturation with distilled water. Resistivities are at about their lowest value when the soil is saturated. The pH of the saturated samples was measured per ASTM G51. A 5:1 water:soil extract from each sample was chemically analyzed for the major soluble salts commonly found in soil per ASTM D4327, ASTM D6919, and Standard Method 2320-B[1]. Laboratory test results are shown in the attached Table 1.

Soil Corrosivity

A major factor in determining soil corrosivity is electrical resistivity. The electrical resistivity of a soil is a measure of its resistance to the flow of electrical current. Corrosion of buried metal is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current (DC) from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities result from higher moisture and soluble salt contents and indicate corrosive soil.

A correlation between electrical resistivity and corrosivity toward ferrous metals is:[2]

Soil Resistivity in ohm-centimeters	Corrosivity Category
Greater than 10,000	Mildly Corrosive
2,001 to 10,000	Moderately Corrosive
1,001 to 2,000	Corrosive
0 to 1,000	Severely Corrosive

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[1] *American Public Health Association (APHA). 2012. Standard Methods of Water and Wastewater. 22nd ed. American Public Health Association, American Water Works Association, Water Environment Federation publication. APHA, Washington D.C.*

[2] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, pp. 166–167.*

Other soil characteristics that may influence corrosivity towards metals are pH, soluble salt content, soil types, aeration, anaerobic conditions, and site drainage.

Electrical resistivities were in the mildly corrosive category with as-received moisture. When saturated, the resistivities were in the mildly and moderately corrosive categories. The resistivities dropped considerably with added moisture because the samples were dry as-received.

Soil pH values varied from 7.0 to 8.1. This range is neutral to moderately alkaline.[3] These values do not particularly increase soil corrosivity.

The soluble salt content of the samples was low. Chloride and sulfate were found at low concentrations.

Nitrate was detected in low concentrations. Ammonium was not detected.

Tests were not made for sulfide and oxidation-reduction (redox) potential because these samples did not exhibit characteristics typically associated with anaerobic conditions.

This soil is classified as moderately corrosive to ferrous metals.

Corrosion Control Recommendations

The life of buried materials depends on thickness, strength, loads, construction details, soil moisture, etc., in addition to soil corrosivity, and is, therefore, difficult to predict. Of more practical value are corrosion control methods that will increase the life of materials that would be subject to significant corrosion.

The following recommendations are based on the soil conditions discussed in the Soil Corrosivity section above. Unless otherwise indicated, these recommendations apply to the entire site or alignment.

Plastic and Vitrified Clay Pipe

1. No special corrosion control measures are required for plastic and vitrified clay piping placed underground.

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[3] *Romanoff, Melvin. Underground Corrosion, NBS Circular 579. Reprinted by NACE. Houston, TX, 1989, p. 8.*

2. Protect all metallic fittings and valves with wax tape per AWWA C217, or with epoxy and appropriately sized cathodic protection per NACE SP0169.

All Pipe

1. On all pipes, appurtenances, and fittings not protected by cathodic protection, coat bare metal such as valves, bolts, flange joints, joint harnesses, and flexible couplings with wax tape per AWWA C217 after assembly.

2. Where metallic pipelines penetrate concrete structures such as building floors, vault walls, and thrust blocks use plastic sleeves, rubber seals, or other dielectric material to prevent pipe contact with the concrete and reinforcing steel.

Concrete Structures and Pipe

1. From a corrosion standpoint, any type of ASTM C150 cement may be used for concrete structures and pipe because the sulfate concentration is negligible, from 0 to 0.10 percent.[4,5,6]

2. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with these soils due to the low chloride concentrations[7] found onsite. Limit the water-soluble chloride ion content in the concrete mix design to less than 0.3 percent by weight of cement.

2. Due to the high groundwater table encountered at this site, cyclical or continual wetting may be an issue. Any contact between concrete structures and groundwater should be prevented.

    a. For structures that extend below the water table, contact can be prevented with an impermeable waterproofing system. Options include a membrane such as Grace PrePrufe® products, a liquid applied barrier coating, or a waterproofing admixture such as Xypex® Admix. Visqueen, similar rolled

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[4] *2015 International Building Code (IBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[5] *2015 International Residential Code (IRC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[6] *2016 California Building Code (CBC) which refers to American Concrete Institute (ACI) 318-14 Table 19.3.2.1*

[7] *Design Manual 303: Concrete Cylinder Pipe. Ameron. p.65*

barriers, or bentonite-based membranes are not viable waterproofing systems for corrosion protection.

b. For structures above the water table, contact can be prevented with a gravel capillary break under the concrete and a vapor retarding membrane. Note that per ASTM E1643, “vapor retarders are not intended to provide a waterproofing function.” [8] Alternatively, an impermeable waterproofing system may be used.

Closure

The analysis and recommendations presented in this report are based upon data obtained from the laboratory samples. This report does not reflect variations that may occur across the site or due to the modifying effects of construction. If variations appear, HDR should be notified immediately so that further evaluation and supplemental recommendations can be provided.

HDR’s services have been performed with the usual thoroughness and competence of the engineering profession. No other warranty or representation, either expressed or implied, is included or intended.

Please call if you have any questions.

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[8] *ASTM E1643-11 (2017): Standard Practice for Selection, Design, Installation, and Inspection of Water Vapor Retarders Used in Contact with Earth or Granular Fill Under Concrete Slabs. ASTM International, 2017.*

Respectfully Submitted,
HDR Engineering, Inc.

REGISTERED PROFESSIONAL ENGINEER
AMY L. OMAE
Amy Omae
No. C76824
CIVIL
★ STATE OF CALIFORNIA ★

James Keegan

Amy Omae, PE

Enc: Table 1

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Table 1 - Laboratory Tests on Soil Samples

Inland Foundation Engineering
RCFCD Wildomar Sedimentation Basin
Your #R206-025, HDR Lab #20-0400SCS
3-Aug-20

Sample ID			B-1 @ 6-9'	B-4 @ 9-20'	B-7 @ 17.5-25'	B-13 @ 13.25-17'	B-16 @ 11-17'
Resistivity		**Units**					
as-received		ohm-cm	308,000	1,880,000	600,000	560,000	520,000
saturated		ohm-cm	10,000	16,000	8,000	13,200	4,400
pH			7.0	7.5	8.1	8.0	7.9
Electrical Conductivity		mS/cm	0.02	0.02	0.02	0.02	0.04
Chemical Analyses							
Cations							
calcium	Ca^{2+}	mg/kg	12	14	12	17	19
magnesium	Mg^{2+}	mg/kg	7.5	5.7	7.2	6.4	9.6
sodium	Na^{1+}	mg/kg	15	12	25	17	35
potassium	K^{1+}	mg/kg	4.8	5.6	5.8	5.2	6.6
Anions							
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	128	162	153	143	143
fluoride	F^{1-}	mg/kg	2.5	ND	1.3	ND	2.0
chloride	Cl^{1-}	mg/kg	4.5	3.8	3.5	4.0	18
sulfate	SO_4^{2-}	mg/kg	3.8	2.9	3.2	3.1	20
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	8.4	7.1	7.5	10	15
sulfide	S^{2-}	qual	na	na	na	na	na
Redox		mV	na	na	na	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

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

Table 1 - Laboratory Tests on Soil Samples

Inland Foundation Engineering
RCFCD Wildomar Sedimentation Basin
Your #R206-025, HDR Lab #20-0400SCS
3-Aug-20

Sample ID			B-18 @ 2.5-5'
Resistivity		**Units**	
as-received		ohm-cm	440,000
saturated		ohm-cm	17,600
pH			8.0
Electrical Conductivity		mS/cm	0.02
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	13
magnesium	Mg^{2+}	mg/kg	5.0
sodium	Na^{1+}	mg/kg	15
potassium	K^{1+}	mg/kg	5.0
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	140
fluoride	F^{1-}	mg/kg	0.5
chloride	Cl^{1-}	mg/kg	3.8
sulfate	SO_4^{2-}	mg/kg	5.2
phosphate	PO_4^{3-}	mg/kg	3.5
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	7.9
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

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

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