

DESIGN-LEVEL GEOTECHNICAL EVALUATION
PROPOSED CANYON RANCH RESIDENTIAL
DEVELOPMENT (REDLANDS 27)
NORTHEAST CORNER OF
SAN TIMOTEO CANYON ROAD AND NEVADA STREET
CITY OF REDLANDS, CALIFORNIA

Prepared for:

HIGHPOINTE CANYON RANCH, LLC

530 Technology, Suite 100
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Project No. 12828.005

April 27, 2021



Leighton and Associates, Inc.

A LEIGHTON GROUP COMPANY



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Highpointe Canyon Ranch, LLC
530 Technology, Suite 100
Irvine, California 92618

Attention: Mr. Tim England

**Subject: Design-Level Geotechnical Evaluation
Proposed Canyon Ranch Residential Development (Redlands 27),
Northeast Corner of San Timoteo Canyon Road and Nevada Street,
City of Redlands, California**

In accordance with your request and authorization, Leighton and Associates, Inc. (Leighton) has prepared this design-level geotechnical evaluation report for the single-family residential development, proposed at the northeast corner of San Timoteo Canyon Road and Nevada Street in the City of Redlands, California (the site).

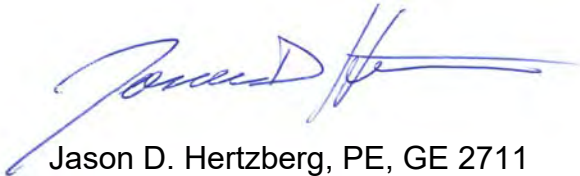
Based on the findings of our investigation, construction of the proposed residential development is considered feasible from a geotechnical standpoint. The most significant geotechnical issues at the site include strong seismic shaking, and the potential for seismic settlement, and compressible shallow soils. Sound project planning and design can limit the impact of these constraints. This report presents our findings, conclusions, and geotechnical recommendations for the project.

We appreciate this opportunity to be of service. If you have any questions regarding this report or if we can be of further service, please call us at your convenience at (866) LEIGHTON, directly at the phone extensions or e-mail addresses listed below.



Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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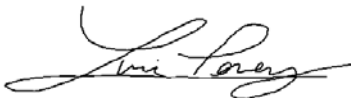
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1.0 INTRODUCTION

1.1 Site Location and Description

The site lies within a broad alluvial plain of low relief, in the City of Redlands, California. It encompasses approximately 21 acres fronting San Timoteo Canyon Road on the north and east. The overall surface gradient has an indistinct descent toward the northwest. The eastern site boundary borders an offsite ascending natural slope representing the eastern geographic margin of the San Timoteo Valley. The slope in this area is moderately inclined with a height of approximately 60 feet. Scattered custom homes occupy pads along the top of the slope. Two narrow easterly ascending tributary canyons interrupt the slope, oriented generally perpendicular to the property boundary. The property boundaries extend eastward into the southern-most tributary canyon. A pole-mounted transmission line transects the southern portion of the east site margin, situated on an inferred lower intermediate stream terrace. The site boundaries immediate vicinity are shown on Figure 1, *Site Location Map*.

For the purposes of this report, the geographic location of the site is designated as: Latitude 34.0437°, Longitude -117.2160°.

Our review of aerial photographs, dated from circa 1938 to 2016, reveals the site was operated as a citrus farm throughout its history, occupied by a dense orchard. Presently, the site is undeveloped and transected by a closely spaced network of windrows/furrows associated with former citrus production. Associated irrigation wells and piping likely remain buried beneath the site. Existing vegetation includes intermittent trees and tumbleweed shrubs.

1.2 Proposed Development

Our understanding of the development concept is based on an undated 60-scale Tentative Tract Map prepared by Proactive Engineering Consultants, 2021. The plan indicates 27 lots will be constructed, along with a network of internal streets and an infiltration basin on a lot at the northwest property corner.

We assume the new homes will be supported on conventional spread footings with slab-on-grade floors and wood-framing. We anticipate appurtenant elements will include concrete driveways and flatwork, asphalt-paved streets, and on- and off-site underground utilities. We expect no subterranean elements are planned as part of the development.

We understand all existing site improvements will be demolished, and building pad and street construction will be accomplished through conventional remedial earthwork (grading), involving minor cuts and fills to establish finish surfaces at or near existing grades. Structural loads for buildings are presently unknown. We assume a maximum column load of 45 kips and maximum wall load of 2.5 kips per lineal foot are generally applicable for the relatively light residential structural loads.

1.3 **Purpose of Study**

The purpose of our study has been to supplement existing site geotechnical information obtained during our recent due diligence study (Leighton, 2020), which did not include exploratory borings. The general intent of our services has been to verify the preliminary findings and conclusions of our prior due diligence report through direct subsurface exploration, sampling and laboratory testing. Our purpose has also been to provide recommendations for design and construction based on the 2019 California Building Code (CBC).

1.4 **Scope of Investigation**

The scope of our study has included the following tasks:

- **Background Review:** Review readily available geotechnical and geologic literature pertinent to the property, including maps, reports and aerial photographs contained within our in-house technical library, online, and/or provided by you. Principal references used in the preparation of this report are listed at the end of this report.
- **Field Exploration:** Advancement of four (4) 8-inch-diameter hollow-stem auger borings to depths ranging between 25 to 50 feet below ground surface (bgs), designated as LB-1 through LB-4. Approximate boring locations are depicted on Figure 2, *Geotechnical Map*. The borings were excavated to evaluate the engineering characteristics and distribution of subsurface soils, and collect representative bulk and relatively undisturbed samples for laboratory testing. Earth materials encountered during drilling were inspected by a member of our technical staff, who collected the samples and transported them to our laboratory for analysis. Upon completion the borings were backfilled with excavated soil cuttings. Logs of the borings describing the encountered soil conditions at each location are included in Appendix A.

- Field Percolation Testing: Advance one (1) boring in the area of the proposed WQMP infiltration basin. The boring was converted to a percolation test well and testing was performed at a depth of approximately 10 feet bgs. The test was performed in general conformance with San Bernardino County guidelines. Test results are included in Appendix B, *Percolation Testing*. Measured rates of infiltration are also noted on Figure 2, *Geotechnical Map*.
- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This testing program was designed to evaluate engineering characteristics of the onsite soils. Laboratory tests conducted during this investigation include:
 - In situ moisture content and dry density
 - Sieve analysis
 - Expansion Index
 - Consolidation / Swell
 - Direct Shear
 - R-Value
 - Max density and optimum moisture content
 - Resistivity, chloride content and pH

The in situ moisture content and dry density test results are shown on the boring logs in Appendix A. The other laboratory test results are presented in Appendix C, *Geotechnical Laboratory Testing*.

- Engineering Analysis: Data obtained from our background review, along with data from field exploration and geotechnical laboratory testing, was evaluated and analyzed to develop geotechnical conclusions and provide geotechnical recommendations presented in this report.
- Report Preparation: Results of our geotechnical study have been summarized in this report, presenting a summary of our findings, conclusions and preliminary geotechnical recommendations for design and construction of the residential development as currently proposed.

2.0 FINDINGS

2.1 Regional Geology

The site is located within the southeasterly area of the San Bernardino Valley (SBV), a large regional-scale valley representing a small portion of the greater Peninsular Ranges Geomorphic Province of Southern California. Major structural features surrounding the region include the SBV, including the San Gabriel (SG) Mountains on the northwest, San Bernardino (SB) Mountains to the north and east, and the San Timoteo Badlands to the south. The location of the site relative to the above features is depicted on Figure 3, *Site Geology Map*.

Development of the SB and SG mountain ranges, and the SBV basin, result from global-scale opposing/relative movement between the North American and Pacific tectonic plates. This plate boundary is manifest as the San Andreas Fault Zone (SAFZ), which follows a generally narrow northwesterly transect along the foot of the SB Mountains. The currently recognized active strand of the SAFZ lies approximately 6.2 miles northeast of the project site. Right-lateral offset accommodated by the SAFZ has resulted in the juxtaposition of significantly different rock types within the SB Mountains northeast of the fault zone and the SG Mountains southwest of the fault zone. An overview of these and other fault zones within the SBV area are shown on Figure 4, *Regional Fault and Historic Seismicity Map*.

The range of low-lying hills south of the site represent the general northwest contiguous extension of the San Jacinto Mountains. The hills are moderately elevated, smoothly to deeply eroded, and locally referred to as The Badlands. The Badlands have been uplifted by dextral right-lateral offset and along the San Bernardino Valley Section of the San Jacinto Fault Zone (SJFZ). The SJFZ is similar to the SAFZ in earthquake history, movement, and seismic potential. It is one of a handful of major fault zones in southern California, comprising the overall San Andreas system of faults. The nearest strand of the SJFZ lies approximately 2.4 miles southwest of the site. The presence of the SBV, between the SAFZ and SJFZ, represents a major tectonically down-dropped structural depression or sediment-sink, referred to as the Perris Structural Block.

Several thousand feet of sediments have infilled central areas of the SBV basin, derived from erosion of the adjacent uplifted mountain ranges. In general, the sediments are relatively thinner along valley margins. The transport of sediments into the valley occurs via large alluvial fans associated with major past and present

rivers. These include Lytle and Cajon Creek washes at the northwest margin of the SBV. Another significant sediment source is from the presently active Santa Ana River (SAR), which exits the SB Mountains near the northeastern edge of the SBV, and flows in a west-southwesterly direction across the SBV. The SAR is located approximately 3.2 miles north of the site at its nearest location.

2.2 Subsurface Soil Conditions

The findings of our literature review and subsurface exploration indicate the site surface is underlain by middle-Holocene age fluvial sediments, assigned to a unit of young axial-valley alluvium (map-unit Qya3). We interpret the configuration of this deposit is that of a westerly-thickening wedge beneath the site, on the order of 5 feet thick at the eastern site margin, and 50 feet thick along the west. As encountered, the unit is composed of loose to medium dense silty sands and sand with varying amounts of silt and gravel. The Qya3 unit is underlain by a deposit of very old axial-valley alluvium (map-unit Qvoa3), composed of fluvial sediments similar in lithology to the Qya3 unit, but higher density due to its age. The Qvoa3 deposit outcrop within the slope offsite to the east. A more detailed description of subsurface units is presented within our boring logs (Appendix A). The subsurface distribution of the units are depicted on Plate 1, *Cross Section A-A'*.

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to the settlement potential of a soil when subjected to increased loads, such as that resulting from the addition of fill to an area, referred to as a fill surcharge. Although no artificial fill deposits were observed in any of our exploratory borings, the upper zone of surface soils are disturbed to variable depths due to the effects of long term farming activities. Near surface in-situ soils are loose and considered to be slightly compressible based on our findings. Remedial grading (removal and recompaction) of the disturbed and compressible alluvium is recommended to mitigate potential total and differential settlement, and adverse effects on proposed improvements.

Soil collapse refers to the potential of a soil to settle under stresses due to wetting. One (1) laboratory collapse test was performed on a representative soil sample from LB-1 and LB-2 at 10 feet below ground surface, which yielded low collapse results. Additionally, near-surface soils exhibited a generally high content of fines. Based on this and results of our laboratory

collapse tests, the onsite soils are anticipated to have a low collapse potential when inundated with water.

2.2.2 Expansive Soils

Expansive soils contain significant amounts of clay particles that swell considerably when wetted and shrink when dried. Foundations constructed on these soils are subjected to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of building foundations and slabs-on-grade could result.

Expansion index testing was performed on a representative sample obtained from near-surface fine-grained soils at Boring LB-1. Laboratory tests resulted in an expansion index of 1. Based on our exploratory borings, field observations, and laboratory testing, the encountered near-surface clayey soils are expected to have a “very low” expansion potential.

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on American Concrete Institute (ACI) provisions, adopted by the 2019 CBC (CBC, 2019, Chapter 19, and ACI, 2014).

Sulfate tests performed during this study from near-surface fine-grained samples resulted in a sulfate content of 82 ppm (less than 0.1 percent by weight), indicating negligible sulfate exposure (Exposure Class S0).

Recommendations for concrete in contact with the soil are provided in Section 3.6.

2.2.4 Resistivity, Chloride and pH

In general, soil resistivity, which is a measure of how easily electrical current flows through soils, is the most influential factor for ferrous corrosivity. Based on findings of studies presented in ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” (February 1989), an approximate relationship between soil resistivity and soil corrosiveness was developed as shown in the table below.

Soil Corrosivity as a Function of Resistivity

Soil Resistivity (ohm-cm)	Classification of Soil Corrosiveness
0 to 900	Very severe corrosion
900 to 2,300	Severely corrosive
2,300 to 5,000	Moderately corrosive
5,000 to 10,000	Mildly corrosive
10,000 to >100,000	Very mildly corrosive

Additionally, soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative sample collected during this study for near-surface soils at boring location LB-2 was tested to determine minimum resistivity. The test result indicated a minimum resistivity of 2,399 ohm-cm, which indicated that the soil sample was moderately corrosive to buried ferrous metals per ASTM STP 1013.

2.3 Groundwater

While the SAR serves as the most significant source of aquifer recharge within the SBV, San Timoteo Wash serves as the main source of groundwater in the site area. The aquifer nearest the surface within the SBV is likely unconfined within deposits of alluvium. A historical groundwater map reviewed as part of our study shows interpreted groundwater depths beneath the site between the dates of 1973 and 1975 (Carson & Matti, 1985). An excerpt of this map is presented as Figure 6, Depth to Groundwater Map 1975-1979. The map indicates the depth to groundwater beneath the property was on the order of 100 feet during that period.

Based on our review of nearby wells with data from 1931 to 2008, groundwater near the site has been recorded at depths of approximately 74 feet bgs and deeper (CDWR, 2021). Groundwater was also not encountered in any of the borings excavated to a maximum depth of 51.5 feet bgs (Appendix A).

Based on the above, we do not anticipate that groundwater would pose constraints to earthwork grading or long-term performance of site improvements.

Our review of the State-maintained GeoTracker website (<https://geotracker.waterboards.ca.gov/map>) revealed no case files for the subject site, nor any sites within the surrounding area.

2.4 Infiltration Testing

On Lot A in the northwestern corner of the site is a planned infiltration basin (pers. comm., Proactive 04-23-2021). To evaluate an infiltration rate for use in preliminary basin design, we advanced one soil boring (LB-4) to a depth of 26.5 feet, and a second nearby boring (test well LP-1) to a depth of 10 feet bgs within the boundaries of Lot A. The location of the borings and boundaries of Lot A (basin area) are shown on attached Figure 2, *Geotechnical Map*.

Boring LP-1 was converted to a temporary percolation test well by installing a 2-inch-diameter perforated PVC pipe within the borehole, including slotted casing within the test zone between 5 and 10 feet bgs, and blank casing between 5 feet and the ground surface. The borehole annulus was infilled with a clean sand (#3 Monterey Sand) to approximately 1 foot above the screen zone. The percolation test well was pre-soaked prior to the testing in an attempt to model the behavior of the stormwater quality control measure during a design storm event. After the conclusion of the percolation testing, the PVC pipe was removed and the test hole backfilled with soil cuttings.

A constant head method of field percolation testing was employed to conduct the test, based on a preliminary evaluation of rates in the field. This test type records the approximate volume of water delivered to the test zone while maintaining a relatively constant height of water in the well throughout the duration of the test. Water was introduced into each well at a relatively constant rate concurrently with a recording of water height in the well. A measured infiltration rate was calculated by dividing the total volume of water infiltrated by the total duration of the test and dividing by the percolation surface area. A porosity reduction factor was utilized to account for the filter pack material.

A summary of the test is included in Appendix B. See Section 3.11 for infiltration recommendations.

2.5 Faulting and Seismicity

In general, the primary seismic hazards for sites in the region include surface rupture along active faults and strong ground shaking. The potential for fault rupture and seismic shaking are discussed below.

2.5.1 Regional Faulting

The state of California passed the Alquist-Priolo Earthquake Fault Zoning (AP) Act into law following the destructive February 9, 1971, $M_w=6.6$ San Fernando earthquake. The AP Act provides a mechanism for reducing potential losses from surface fault rupture on a statewide basis. The intent of the AP Act is to enhance public safety by prohibiting the siting of most structures for human occupancy across known traces of active faults that constitute a potential hazard to structures from active surface faulting. The fault classification criteria adopted by the California Geological Survey (CGS) defines Earthquake Fault Zones along active faults. An active fault is defined as one that has ruptured during Holocene time (the last 11,700 years).

No State of California established Earthquake Fault Zones have been mapped within or projecting towards the site. However, given the close proximity of southern California to the North American / Pacific plate boundary, the seismic effects of moderate to significant earthquakes are generally widespread across the area. Hundreds of minor earthquakes (magnitude 1.0 to 2.9) occur annually within the San Bernardino Valley, along major active fault zones. In the site vicinity, these earthquakes occur along the northwest-trending SAF and SJFZ (see Figures 3 and 4). These fault systems currently accommodate up to approximately 55 millimeters per year (mm/yr) of slip, ranking among the most active in all of southern California. Since the recording of seismic events in the mid-18th century, at least 6 major earthquakes have occurred along the San Jacinto Fault Zone (Blake, 2000a).

The on-line Interactive Fault Map published by the United States Geological Survey (USGS, 2006), indicates the San Bernardino Valley Section of the San Jacinto Fault Zone is the nearest active fault, located approximately 2.4 miles southwest of the site. The SJFZ is zoned under the Alquist-Priolo Act, and contains several northwest oriented paralleling strands. Movement

exhibits a dextral right-lateral sense with a common right-stepping pattern of offset. The last rupture/offset along this fault section is considered to have occurred during latest Quaternary time, or sometime during the past 15 thousand years.

Other major faults in the region include the Sierra Madre Fault zone along the southern foot of the SG Mountains, the Elsinore Fault bordering the north edge of the Santa Ana Mountains, and the Homestead Valley Fault Zone within the Eastern California Shear Zone, approximately 15 miles northwest, 25 miles southwest, and 46 miles east-northeast of the site, respectively. A map depicting the location of fault and historic seismic activity is presented as Figure 4, *Regional Fault and Historic Seismicity Map*.

2.5.2 Local Faulting

No active faults mapped in the near site vicinity in references we reviewed, there are however several mapped faults in the area northeast of the SJFZ, exhibiting orientations sub-parallel and parallel to the SJFZ (USGS, 2003; CGS, 2003). While these faults are not AP-zoned faults, and are generally considered less active than the SJFZ, we opine that they are still capable of accommodating a degree of co-seismic offset during major earthquakes along the SJFZ, if not their own earthquakes. One of these “secondary” faults is the Crafton Hills Fault Zone, situated approximately 1.3 miles southwest of the site (USGS, 2003). The same zone is referred to on CGS maps as the Live Oak Canyon Fault Zone (CGS, 2003). Another is the inferred buried Banning Fault mapped approximately 0.4 miles northeast of the site (CGS, 2003). A listing of active faults within a 60-mile search radius is presented in Appendix D. Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered very low. Major regional faults with surface expression in proximity to the site are shown on Figure 4, *Regional Fault and Historical Seismicity Map*.

2.5.3 Ground Shaking

The site will experience strong ground shaking after the proposed project is developed resulting from an earthquake occurring along one or more of the major active or potentially active faults in southern California. Accordingly,

the project should be designed in accordance with all applicable current codes and standards utilizing the appropriate seismic design parameters to reduce seismic risk as defined by California Geological Survey (CGS) Chapter 2 of Special Publication 117a (CGS, 2008). Through compliance with these regulatory requirements and the utilization of appropriate seismic design parameters selected by the design professionals, potential effects relating to seismic shaking can be reduced.

The following parameters should be considered for design under the 2019 CBC:

2019 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2019 CBC
Site Latitude and Longitude (degrees): 34.0437, -117.2159	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	D
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	2.103 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.836 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	1.000
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	1.700*
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	2.103 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	1.421* g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	1.402 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.947* g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), PGA	0.887 g
Site Coefficient for Mapped MCE_G PGA (11.8.3.2), F_{PGA}	1.100
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.976 g
* Per Table 11.4-2 of Supplement 1 of ASCE 7-16, this value of F_v may only be used to calculate T_s [that note is not included in Table 1613A.2.3(2)]; note that S_{D1} and S_{M1} are functions of F_v. In addition, per Exception 2 of 11.4.8 of ASCE 7-16, special equations for C_s are required. This is in lieu of a site-specific ground motion hazard analysis per ASCE 7-16 Chapter 21.2. ** Site Class D, and all of the resulting parameters in this table, may only be used for structures without seismic isolation or seismic damping systems.	

Based on the 2019 CBC Table 1613.2.3(2) footnote c., F_v should be determined in accordance with Section 11.4.8 of ASCE 7-16, since the mapped spectral response acceleration at 1 second is greater than 0.2g for Site Class D; in accordance with Section 11.4.8 of ASCE 7-16, a site-specific seismic analysis is

required. However, the values provided in the table above may be utilized if design is performed in accordance with Exception (2) in Section 11.4.8 of ASCE 7-16, with special requirements for the seismic response coefficient (C_s), and F_v is only used for calculation of T_s . This exception does not apply (and the values in the table above would not be applicable) for proposed structures with seismic isolation or seismic damping systems. The project structural engineer should review the seismic parameters. A site-specific seismic ground motion analysis can be performed upon request.

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 8.1 (M_w) at a distance on the order of approximately 5.7 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years).

2.6 Secondary Seismic Hazards

In general, secondary seismic hazards for sites in the region could include ground shaking, soil liquefaction, earthquake-induced settlement, lateral displacement, landsliding, and earthquake-induced flooding. The potential for secondary seismic hazards at the site is discussed below.

2.6.1 Liquefaction Potential

Liquefaction is the loss of soil strength or stiffness due to a buildup of pore-water pressure during severe ground shaking. Liquefaction is associated primarily with loose (low density), saturated, fine-to-medium grained, cohesionless soils. As the shaking action of an earthquake progresses, the soil grains are rearranged and the soil densifies within a short period of time. Rapid densification of the soil results in a buildup of pore-water pressure. When the pore-water pressure approaches the total overburden pressure, the soil reduces greatly in strength and temporarily behaves similarly to a fluid. Effects of liquefaction can include sand boils, settlement, and bearing capacity failures below structural foundations.

The State of California has not prepared a map delineating potential liquefaction zones for the quadrangle that contains the site. The San Bernardino County Geologic Hazard Overlay Map EHFH C indicates the site to have a low liquefaction susceptibility (SB County, 2009). The most

recent available groundwater data pertinent to the site is from 1979, indicating a depth of around 100 feet.

Based on the absence of shallow groundwater, the subsurface soils are not considered susceptible to liquefaction.

2.6.2 Lateral Spreading

Deformation due to lateral spreading generally occurs on sloping ground that is underlain by potentially liquefiable soil layers. Due to the absence of potentially liquefiable layers, the lateral spreading hazard on this site during the design earthquake event is considered very low.

2.6.3 Seismically Induced Settlement

Seismically induced settlement consists of dry dynamic settlement (above groundwater) and liquefaction-induced settlement (below groundwater). These settlements occur primarily within loose to moderately dense sandy soil due to reduction in volume during and shortly after an earthquake event. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement.

We performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed, and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). Historic high groundwater is at a depth that will not affect the seismic settlement analysis. The results of our analyses indicate the Qvya3 unit is susceptible to seismic settlement on the order of approximately 6 inches, based on the PGA_M of 0.98g and MCE of 8.1 (M_w).

If we estimate the potential differential settlement is half of the total seismic settlement over a horizontal distance of 30 feet, this would result in approximately 3 inches in 30 feet, or angular distortion of 0.008L. This would be within the differential settlement threshold value of 0.015L for “other single-story structures” and 0.010L for “other multistory structures” of Risk Category I or II, as listed in Table 12.13-3 of ASCE 7 16. “Other” buildings are those not constructed with concrete or masonry wall systems

(i.e. wood-framed). A summary of seismic settlement analysis is included in Appendix D, Seismic Analysis.

2.6.4 Seismically Induced Landsliding

The potential for seismically induced landsliding to occur at the site is not considered to be a hazard at the site. The CGS on-line landslide inventory map shows no specific landslides on the site or in adjacent offsite areas (CGS, 2020). They indicate the slopes abutting the east site margin have a moderate to high landslide susceptibility, based on rock strength. The SB County Land Use Plan indicates these offsite slopes have a low to moderate landslide susceptibility (SB County, 2009). Leighton observed no evidence of significant landslides in this area during our field reconnaissance. Nor did we observe such conditions on any historical aerial photographs. Additionally, the County of San Bernardino has mapped the site to be outside of a zone of Generalized Landslide Susceptibility. Given these considerations, the potential for seismically induced landsliding to affect the site is low.

2.6.5 Surface Fault Rupture

The proposed development is not located within the boundaries of an active Earthquake Fault Zone, as designated by the State of California or County of San Bernardino. Nor did our review identify the mapped trace(s) of any inactive faults on the site, or trending toward the site. Given the above, the surface fault rupture potential on the site is considered to be very low.

2.6.6 Flooding and Dam Inundation

According to a Federal Emergency Management Agency (FEMA) flood insurance rate map (FEMA, 2008), the project site is located within a flood hazard area identified as “Zone X”, defined as an area of minimal flood hazard. As shown on Figure 7, Flood Hazard Zone Map, the site is not located within a 100-year or 500-year flood hazard zone.

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of earthquakes. As shown on Figure 8, Dam Inundation Map, the site is not mapped within a dam inundation zone. Therefore, the risk of seismically-induced flooding due to dam failure is considered low.

2.6.7 Seiches and Tsunamis

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Tsunamis are waves generated in large bodies of water by fault displacement or major ground movement. Based on the absence of an enclosed water body near the site and the inland site location, seiche and tsunami risks are considered negligible.

2.7 Erosion

Climate, topography, soil types and vegetation are key factors to erosion, runoff, and sedimentation processes. The project site is situated in an area that no longer receives sediments due to rechanneling of the tributary creeks that flow northerly from the mountains to the south. The site is relatively flat and will be landscaped upon completion, therefore loss of site material to erosion (water and wind) upon completion of site improvements is considered low. However, there is a very high potential for erosion prior to completion of site improvements.

2.8 Slope Stability

The eastern edge of the site abuts a natural ascending slope on the order of approximately 60 feet in height at a ratio of 1.5H:1V. The slope profile is depicted on attached Plate 1, *Geotechnical Cross Section A-A'*. As shown on Figure 3, *Site Geology Map*, the slope is underlain by Pleistocene age very old axial-valley deposits (Qvoa), which are massive, clast-supported and well consolidated.

The above slope exists on private property beyond the boundaries of the site. Although no exploration or stability analyses were performed, outcrops reveal it is underlain entirely by the Qvoa unit, which we encountered below the Qya3 unit within our borings LB1 and LB-2. Based on our review of outcrops within the slope, and historical aerial photographs, we find no evidence of any prior deep or shallow instability. Also, based on the massive clast-supported structure of the Qvoa unit, and strengths revealed by blow-counts, we anticipate that the slope is grossly stable. Based on the above, we do not anticipate the slope will pose any adverse effect on development of this site.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this study, construction of the proposed residential development is considered feasible from a geotechnical viewpoint provided the recommendations presented in this report are implemented during design and construction. No severe geologic or soils related constraints were identified that would preclude development of the site according to the current concept plan. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, and potentially compressible soils.

The recommendations below are based upon the exhibited geotechnical engineering properties of the soils and their anticipated response both during and after construction. The recommendations are also based upon proper field observation and testing during construction. The project geotechnical engineer should be notified of suspected variances in field conditions to determine the effect upon the recommendations subsequently presented. These recommendations are considered minimal and may be superseded by more restrictive requirements of the civil and structural engineers, the City of Redlands, and other governing agencies.

3.1 General Earthwork and Grading

All site grading should be performed in accordance with the applicable local codes and in accordance with the project specifications that are prepared by the appropriate design professional. Overexcavation and recompaction recommendations are presented in the following paragraphs. The General Earthwork and Grading Recommendations are included in Appendix F. In case of conflict, the following recommendations shall supersede those provided in Appendix F.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of any vegetation, trash, and/or debris within the area of proposed grading. Any underground obstructions onsite interfering with the proposed construction should be removed or rerouted to preserve their function. Resulting cavities should be properly backfilled and compacted. After the site is cleared, the soils should be carefully observed for the removal of all unsuitable dry fill deposits by a representative of the geotechnical engineer. Random buried concrete irrigation piping associated with past orchard use should be anticipated.

3.1.2 Overexcavation and Recompaction

To reduce the potential for adverse total and differential settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved.

All man-made fill deposits, and any soils consisting of low-density and potentially compressible material, including but not limited to topsoil, residual soil and recent near surface (upper 5 feet) alluvium, should be removed to a minimum depth of 5 feet below current grades, until competent alluvium is encountered, and a minimum of 3 feet below the bottoms of the proposed footings, whichever is deeper, from areas to receive new compacted fill. The limits of the recommended remedial grading would encompass all buildings, and appurtenant improvements such as areas of proposed new asphalt or concrete pavement and flatwork, and site walls, low retaining walls, and other improvements

Deeper overexcavation may be recommended, depending on exposed conditions during grading. Removal bottoms should extend horizontally a minimum of 5 feet beyond the outside edges of footings (including columns connected to buildings), or a distance equal to the depth of overexcavation below the footings, whichever is greater. In-place alluvial soils should be deemed suitable for new fill placement if possessing a minimum in-situ relative compaction of 85 percent (ASTM Test Method D1557). Suitability of all removal bottoms should be reviewed and evaluated by an engineering geologist or a representative of the geotechnical engineer.

Given the long-term agricultural use of the property, it is likely certain man-made items remain buried in the shallow subsurface. Such may include but not be limited to underground septic tanks, irrigation wells, surface ditches, and water-conveyance pipelines. The presence of deep tree roots should also be anticipated.

Upon completion of overexcavation and prior to fill placement, exposed surfaces should be evaluated for suitability. Once determined geotechnically acceptable, the subgrade should be scarified to a minimum depth of 6 inches, moisture conditioned to or slightly above optimum moisture content, and recompacted to a minimum 90 percent relative compaction, relative to the ASTM D 1557 laboratory maximum density.

All fills should be observed and tested in the field by a Leighton representative prior to fill placement or foundation or pavement construction to ensure adequate moisture conditioning and compactive requirements are met. Fill soil should be compacted to a minimum of 90 percent relative compaction per ASTM 1557. Aggregate base should be compacted to a minimum of 95 percent relative compaction.

3.1.3 Fill Placement and Compaction

Onsite soil may be used for compacted structural fill provided it is free of debris, organic material and oversized material (greater than 6 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and tested by Leighton as needed or required.

All fill soil should be placed in thin, loose lifts, moisture conditioned, as necessary, and compacted to a minimum 90 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.4 Import Fill Soil

The geotechnical parameters of any import soil should be evaluated and accepted by Leighton prior to use as fill on the site. Preferably at least 3 working days prior to proposed import to the site, the contractor should provide Leighton pertinent information of the proposed import soil, such as location of the soil, whether stockpiled or native in place, and pertinent geotechnical reports if available. We recommend that a Leighton representative visit the proposed import site to observe the soil conditions and obtain representative soil samples for geotechnical and analytical testing for potential chemicals of concern. Potential issues may include soil that is more expansive than onsite soil, soil that is too wet, soil that is too rocky or too dissimilar to onsite soils, oversize material, organics, debris, etc.

3.1.5 Shrinking and Bulking

The change in volume of excavated and recompacted soil varies according to soil type and location. This volume change is represented as a percentage increase (bulking) or decrease (shrinkage) in volume of fill after

removal and recompaction. This value does not factor in removal of debris or other materials. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as what occurs during processing an overexcavation (subgrade) bottom. Subsidence is in addition to shrinkage due to recompaction of fill soil. Field and laboratory data used in our calculations included laboratory-measured maximum dry densities for soil types encountered at the subject site, the measured in-place densities of soils encountered and our experience. We preliminarily estimate the following earth volume changes will occur during grading:

Shrinkage	Approximately 15% +/- 4%
Subsidence (overexcavation bottom processing)	Approximately 0.2 foot

The level of fill compaction, variations in the dry density of the existing soils and other factors influence the amount of volume change. Some adjustments to earthwork volume should be anticipated during grading of the site.

3.1.6 Rippability and Oversized Material

Although not anticipated, any oversized material (rock or rock fragments greater than 8 inches in dimension) should be placed outside the limits of structural fills (i.e. not in building areas). Such materials should be placed in deeper removal bottoms on the site, at the direction of the project geotechnical engineer.

3.2 Shallow Foundation Recommendations

The proposed residential buildings can be supported on shallow foundations. Maximum column loading and wall loading is not available at the time of this report. We have anticipated that the proposed residential buildings will be wood-framed and lightly loaded. We assume a maximum column load of 45 kips and maximum wall load of 2.5 kips per lineal foot are generally applicable for the relatively light residential structural loads. Structural loading information should be provided to us when available for review.

Due to the potential for seismic settlement, we recommend that the proposed residential buildings be supported on a stiffened (post-tension) foundation system.

Overexcavation and recompaction of the footing subgrade should be performed as detailed in Section 3.1. The following recommendations are based on the onsite soil conditions and soils with a “very low” expansion potential.

3.2.1 Minimum Embedment and Width

Based on our preliminary investigation, footings should have a minimum embedment per code requirements, with a minimum width of 24 and 12 inches for isolated and continuous footings, respectively.

3.2.2 Post-Tension Foundation Design Parameters

Post-tensioned foundations should be designed by a qualified structural engineer in accordance with the 2019 CBC using the minimum geotechnical parameters provided below for soils with a “very low” Expansion Index. Expansion index should be confirmed upon completion of grading. The following table presents parameters for design of post-tension foundations based on the Post-Tensioning Institute methodology. Alternatively, proposed residential buildings may be supported with a post-tension “California slab” foundation designed based on the spanability method for very low expansive soils.

Post-tensioned Foundation Design Recommendations		
Low Expansion		
Edge Moisture Variation, e_m	Center Lift	9.0 feet
	Edge Lift	5.0 feet
Differential Swell Y_m	Center Lift	0.5 inch
	Edge Lift	0.7 inch
Modulus of Subgrade Reaction		150 pci

For post-tension slab foundations, exterior footings (thickened edges) should have a minimum depth of 12 inches below the lowest adjacent soil grade and a minimum width of 12 inches. These footings may be designed for a maximum allowable bearing pressure of 2,000 pounds per square foot. The allowable bearing pressure may be increased by one-third for short-term loading. The structural engineer should provide the slab with adequate stiffness to minimize potential cracking. The design of post-tensioned slab foundations should follow the procedures described in the latest edition of

the Design of Post-Tensioned Slabs-on-Ground by the Post-Tensioning Institute.

To provide more uniform moisture in the subgrade, the top 18 inches of the prepared subgrade should be pre-saturated to 120 percent of the optimum moisture prior to placement of concrete.

The soil-moisture around the immediate perimeter of the slab should be maintained to near-optimum moisture content (or above) during construction and up to occupancy of the homes.

The Post-Tensioning Institute (PTI) has recommended the following guidelines for residential development:

- Initial landscaping should be done on all sides adjacent to the foundation. Positive drainage away from the foundation should be implemented and maintained.
- Irrigation watering should be done in a uniform manner as equally as possible on all sides of the foundation to maintain constant soil moisture content. Ponding of irrigation or rainfall water adjacent to the foundation slab can cause differential soil moisture levels potentially causing differential movements.

Planting trees closer to the structure than a distance equal to one-half the mature height of the tree could allow the root system to enter under the foundation. The root system could alter the soil moisture content within the root system and cause soil shrinkage which may result in differential movements of the foundation. A landscape architect should be consulted and made aware of these recommendations.

3.2.3 Lateral Load Resistance

Soil resistance available to withstand lateral loads on a shallow foundation is a function of the frictional resistance along the base of the footing and the passive resistance that may develop as the face of the structure tends to move into the soil. The frictional resistance between the base of the foundation and the subgrade soil may be computed using a coefficient of friction of 0.30. The passive resistance may be computed using an allowable equivalent fluid pressure of 250 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed

soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

The allowable bearing pressure and coefficient of friction values may be increased by one-third when considering loads of short duration, such as those imposed by wind and seismic forces.

3.2.5 Settlement Estimates

The recommended allowable bearing pressure is generally based on a total allowable, post-construction static settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 30 feet. Since settlement is a function of footing sustained load, size and contact bearing pressure, differential settlement can be expected between adjacent columns or walls where a large differential loading condition exists.

Considering the design PGA of 0.98g, differential seismic settlement is estimated at 3 inches over a horizontal distance of 30 feet, or angular distortion of 0.008L.

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a low expansion potential and considering the potential for seismically induced settlement. Where conventional light floor loading conditions exist, the following minimum recommendations should be used. More stringent requirements may be required by local agencies, the structural engineer, the architect, or the CBC. Laboratory testing should be conducted at finish grade to evaluate the expansion index of near-surface subgrade soils. In addition, slabs-on-grade should have the following minimum recommended components:

- **Subgrade Moisture Conditioning:** The subgrade soil should be moisture conditioned to at least 2 percentage points above optimum moisture content to a minimum depth of 18 inches prior to placing the moisture vapor retarder, steel or concrete.

- Moisture Retarder: A minimum 10-mil thick polyethylene moisture retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a sand blotter layer should be placed over the vapor retarder, as this is a concrete design and construction issue as opposed to a geotechnical issue. Gravel or other protruding objects that could puncture the moisture retarder should be removed from the subgrade prior to placing the vapor retarder.
- Concrete Thickness: Slabs-on-grade should be at least 4 inches thick (this is referring to the actual minimum thickness, not the nominal thickness). Reinforcing steel should be designed by the structural engineer, but as a minimum (for conventionally reinforced, 4-inch-thick slabs) should be No. 4 rebar placed at 18 inches on center, each direction, mid-depth in the slab. Crack control joints should be provided at a maximum spacing of 15 feet on center.

Minor cracking of the concrete as it cures, due to drying and shrinkage, is normal and should be expected. However, cracking is often aggravated by a high water/cement ratio, high concrete temperature at the time of placement, small nominal aggregate size, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Floor covering manufacturers should be consulted for specific recommendations.

Leighton does not practice in the field of moisture vapor transmission evaluation, since this is not specifically a geotechnical issue. Therefore, we recommend that a qualified person, such as the flooring subcontractor and/or structural engineer, be consulted with to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person should provide recommendations for mitigation of

potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate.

3.4 **Exterior Concrete Flat Work**

Exterior concrete slabs-on-grade should have a minimum thickness of 4 inches. Common Type II cement should be adequate for concrete flatwork not exposed to recycled water. Concrete flatwork should be placed on previously compacted fill. If this material has been disturbed or become dry, the subgrade soil to a depth of 18 inches should be moisture conditioned to 2 percentage points above the optimum moisture content and recompact to minimum 90 percent relative compaction.

Exterior concrete driveways, ramps, curbs, gutters, sidewalks often crack. Inclusion of joints at frequent intervals and reinforcement will help control the locations of the cracks, and thus reduce the unsightly appearance. Appropriate joints or saw cuts should be constructed in accordance with either Portland Cement Association (PCA) or American Concrete Institute (ACI) guidelines. If cracking occurs, repairs may be needed to mitigate the trip hazard and/or improve the appearance.

3.5 **Retaining Wall Recommendations**

Areas planned for retaining walls should be over-excavated in accordance with the recommendations provided in Section 3.1. Retaining walls should be backfilled with *very low* expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 9, *Retaining Wall Detail*. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls up to 6 feet tall:

Static Equivalent Fluid Weight (pcf)	
Condition	Level Backfill
Active	38
At-Rest	58
Passive	250 (Maximum of 3,000 psf)

The above values do not contain an appreciable factor of safety, so the structural engineer should apply the applicable factors of safety and/or load factors during design.

Cantilever walls that are designed to yield at least $0.001H$, where H is equal to the wall height, may be designed using the active condition. Rigid walls and walls braced at the top should be designed using the at-rest condition.

Passive pressure is used to compute soil resistance to lateral structural movement. In addition, for sliding resistance, a frictional resistance coefficient of 0.30 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that the soil providing passive resistance, embedded against the foundation elements, will remain intact with time.

In addition to the above lateral forces due to retained earth, surcharge due to improvements, such as an adjacent structure or traffic loading, should be considered in the design of the retaining wall. Loads applied within a 1:1 projection from the surcharging structure on the stem of the wall should be considered in the design.

We recommend that the wall designs for walls 6 feet tall or taller be checked seismically using an additive seismic Equivalent Fluid Pressure (EFP) of 52 pcf of level backfill, which is added to the equivalent fluid pressure.

A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

Retaining wall footings should have a minimum width of 24 inches and a minimum embedment of 18 inches below the lowest adjacent grade. An allowable bearing capacity of 2,000 psf may be used for retaining wall footing design, based on the minimum footing width and depth.

3.6 Geochemical and Resistivity Characteristics

In general, soil environments that are detrimental to concrete have high concentrations of soluble sulfates and/or pH values of less than 5.5. Section 4.3 of ACI 318 (ACI, 2014). ACI provides specific guidelines for the concrete mix-design when the soluble sulfate content of the soil exceeds 0.1 percent by weight or 1,000 parts per million (ppm).

Laboratory test results indicated that onsite soils at shallow depth have “negligible” soluble sulfate content (per Section 4.3 of ACI 318). Concrete structures in contact with the on-site soils may be designed for **negligible** sulfate exposure in accordance with ACI 318 (ACI, 2014). If the concrete is expected to be in contact with reclaimed water, Type V cement and a water/cement ratio of 0.45 should be used.

Resistivity: The results of the resistivity test indicated that the underlying soil is **moderately corrosive** to buried ferrous metals per ASTM STP 1013.

As a general mitigation measure, ferrous pipe buried in moist to wet site earth materials should be avoided by using high-density polyethylene (HDPE), polyvinyl chloride (PVC) and/or other non-ferrous pipe when possible. Ferrous pipe can also be protected by polyethylene bags, tape or coatings, di-electric fittings or other means to separate the pipe from on-site soils. Once plans are developed and the type of pipe is known, i.e. steel piping, iron piping, copper tubing etc. additional recommendations can be provided as necessary. Plastic and vitrified clay piping do not warrant any special precautions. Standard concrete cover over reinforcing steel may be used for concrete structures and pipe in contact with site soils possessing negligible sulfate reactions.

3.7 Temporary Excavations

All temporary excavations, including utility trenches, retaining wall excavations and other excavations should be performed in accordance with project plans, specifications and all OSHA requirements.

No surcharge loads should be permitted within a horizontal distance equal to the height of cut or 5 feet, whichever is greater from the top of the slope, unless the cut is shored appropriately. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site foundation should be properly shored to maintain support of the adjacent structures.

Cantilever shoring should be designed based on an active equivalent fluid pressure of 35 pcf. If excavations are braced at the top and at specific design intervals, the active pressure may then be approximated by a rectangular soil pressure distribution with the pressure per foot of width equal to $25H$, where H is equal to the depth of the excavation being shored.

During construction, the soil conditions should be regularly evaluated to verify that conditions are as anticipated. The contractor should be responsible for providing the "competent person" required by OSHA, standards to evaluate soil conditions. Close coordination between the competent person and the project Geotechnical Engineer or Certified Engineering Geologist should be maintained to facilitate construction while providing safe excavations.

3.8 **Trench Backfill**

Utility trenches should be backfilled with compacted fill in accordance with Sections 306-1.2 and 306-1.3 of the *Standard Specifications for Public Works Construction*, (SSPWC, "Greenbook"), 2018 Edition. Utility trenches may be backfilled with onsite material, provided it is free of rubble, debris, organic and oversized material up to 3 inches in largest dimension. Prior to backfilling trenches, pipes should be bedded in and covered with either:

- (1) **Granular Bedding:** Uniform, free-draining (minimum permeability of 2 in/hr) sand material with a Sand Equivalent (SE) greater than or equal to 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer).
- (2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the SPWC. CLSM bedding should be placed to 1-foot (0.3 m) over the top of the conduit, and vibrated.

Pipe bedding should extend at least 4 inches below the pipeline invert and at least 12 inches over the top of the pipeline. The bedding and shading sand should be densified in place in accordance with the Greenbook (current edition).

Trench backfill over the pipe bedding zone may consist of native and clean fill soils. All backfill should be placed in thin lifts (appropriate for the type of compaction equipment), moisture conditioned at least 2 percent above optimum, and mechanically compacted to at least 90 percent relative compaction, relative to the ASTM D 1557 laboratory (Proctor) maximum density.

3.9 **Surface Drainage**

Inadequate control of runoff water and/or poorly controlled irrigation can lead to settlement of foundations, flatwork, walls, and other improvements. Maintaining adequate surface drainage, proper disposal of runoff water, and control of irrigation should help reduce the potential for future soil moisture problems.

Positive surface drainage should be designed in accordance with the CBC to be directed away from foundations and toward approved drainage devices, such as gutters, paved drainage swales, or watertight area drains and collector pipes.

Surface drainage should be provided to prevent ponding of water adjacent to the structures. In general, the area around the buildings should slope away from the building. We recommend that unpaved landscaped areas adjacent to the buildings be avoided. Roof runoff should be carried to suitable drainage outlets by watertight drain pipes or over paved surfaces.

3.10 **Pavement Design Parameters**

Flexible Pavements: Based on the City of Redlands standards, the design procedures outlined in the current Caltrans Highway Design Manual, and using the laboratory R-value of 50, flexible pavement sections may consist of the following for the Traffic Index indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided upon completion of grading. The City of Redlands pavement design standard is a minimum 3 inches AC over 4 inches of aggregate base for local and collector streets.

Asphalt Pavement Sections

Traffic Index	Asphalt Concrete	Base Course (inches)
	(inches)	CAB
5.0	3.0	6.0
6.0	3.5	6.0
7.0	4.0	6.0
Notes: CAB – Crushed Aggregate Base Course; Caltrans Class 2, Section 26 or SSPWC Section 200-2.2		

If the pavement is to be constructed prior to construction of the structures, we recommend that the full depth of the pavement section be placed in order to support heavy construction traffic.

Rigid Pavements: For onsite Portland Cement Concrete (PCC) pavement in truck drive aisles and truck parking areas, we recommend a minimum of 7-inch-thick concrete, placed on compacted fill subgrade, with the upper 8 inches compacted to a minimum of 95 percent relative compaction. In areas with car traffic only, we

recommend a minimum of 5-inch-thick concrete, placed on compacted fill subgrade with the upper 8 inches compacted to a minimum of 95 percent relative compaction.

The PCC pavement sections should be provided with crack-control joints spaced no more than 15 feet on center each way. If sawcuts are used, they should have a minimum depth of $\frac{1}{4}$ of the slab thickness and made within 24 hours of concrete placement.

Other Pavement Recommendations: If pavement areas are adjacent to heavily watered landscape areas, some deterioration of the subgrade load bearing capacity may result. Moisture control measures such as deepened curbs or other moisture barrier materials may be used to prevent the subgrade soils from becoming saturated. The use of concrete cutoff or edge barriers should be considered when pavement is planned adjacent to either open (unfinished) or irrigated landscaped areas. All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

Prior to placement of aggregate base, the subgrade soil should be processed to a minimum depth of 6 inches, moisture-conditioned, as necessary, and recompact to a minimum of 95 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

3.11 Infiltration Recommendations

Soil types within the shallow subsurface on the northwestern portion of the site were found to predominately consist of sandy silt and silty sand. The results of sieve analysis tests within the percolation test zone indicate soils consist of sandy silt at 5 feet bgs (58% fines) and silty sands at 9 feet bgs (12% fines). Soils were observed to be granular with low amounts of fines down to approximately 20 feet based on the nearby boring LB-4, see Figure 2. Silt to silty clay was encountered below 20 feet in boring LB-4. Infiltration BMP design should consider this less permeable layer. The site is shown to have variable soil conditions within the upper 30 feet, ranging from sandy to silty soils.

The results of our shallow percolation test at this location yielded an unfactored, or measured infiltration rate of 5 in/hr. We recommend a safety factor of a least 3 be applied for design of infiltration systems. Such would result in a preliminary factored rate of 1.7 in/hr. However, due to variability of soils in this area, additional infiltration testing should be performed as infiltration system design progresses.

Additional Review and Evaluation: Infiltration rates are anticipated to vary significantly based on the location and depth. Infiltration concepts should be discussed with Leighton as infiltration plans are being developed. Leighton should review all infiltration plans, including specific locations and depths of proposed facilities. Further testing should be conducted based on the design of infiltration facilities, particularly considering their type, depth and location.

3.12 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from this limited subsurface explorations and limited laboratory testing. Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. Leighton and Associates, Inc. should review the site foundation, grading, retaining wall and landscape plans when available and comment further on the geotechnical aspects of the project. Geotechnical observation and testing should be conducted during excavation and all phases of lot reconditioning operations and asphalt and base placement up to final asphalt capping. Our conclusions and recommendations should be reviewed and verified by Leighton and Associates Inc. during construction and revised accordingly if geotechnical conditions encountered vary from our findings and interpretations.

Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During over excavation of site soils
- During compaction of all fill materials.
- After excavation of all footings and prior to placement of concrete.
- During utility trench backfilling and compaction.
- During pavement subgrade and base preparation.
- When any unusual conditions are encountered.

4.0 LIMITATIONS

This report was necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This investigation was performed with the understanding that the subject site is proposed for residential development. The client is referred to Appendix E regarding important information provided by the Geoprofessional Business Association (GBA) on geotechnical engineering studies and reports and their applicability.

This report was prepared for Highpointe Canyon Ranch, LLC, based on their needs, directions, and requirements at the time of our investigation. This report is not authorized for use by and is not to be relied upon by any party except Highpointe Canyon Ranch, LLC, and its successors and assignees as owner of the property, with whom Leighton and Associates, Inc. has contracted for the work. Use of or reliance on this report by any other party is at that party's risk. Unauthorized use of or reliance on this report constitutes an agreement to defend and indemnify Leighton and Associates, Inc. from and against any liability which may arise as a result of such use or reliance, regardless of any fault, negligence, or strict liability of Leighton and Associates, Inc.

REFERENCES

- American Concrete Institute (ACI), 2014, Building Code Requirements for Structural Concrete (ACI-318-14) and Commentary (ACI 318-14), and ACI Standard.
- Bryant, W.A. and Hart, E.W., Interim Revision 2007, Fault Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps: California Geological Survey, Special Publications 42, 42p.
- California Building Standards Commission, 2019, 2019 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2, Based on 2018 International Building Code, effective January 1, 2020.
- California Department of Water Resources (CDWR), 2021, Water Data Library (WDL) Station Map, State Well Number 01S03W33C001S, accessed April 8, 2021.
- California Geologic Survey, 2003, Geologic Map of the Sunnymead / South ½ of Redlands Quadrangles, San Bernardino and Riverside County, California, Scale 1: 24,000, dated December 2003.
- _____, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, Revised and Re-Adopted on September 11, 2008, Laguna Beach, California.
- _____, 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers and Geoscience Practitioners for Assessing Fault Hazards In California, Special Publication 42, Revised 2018.
- California State Water Resources Control Board, GeoTracker website (<https://geotracker.waterboards.ca.gov/map>), accessed April, 2021.
- Carson, Scott E., and Jonathan C. Matti, 1985, Contour Map Showing Minimum Depth to Ground Water, Upper Santa Ana River Valley, California, 1973-1979.
- Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas, California Division of Mines and Geology, Geologic Data Map Series, No. 6, Scale 1:750,000.
- Leighton and Associates, Inc., 2020, Geotechnical Due Diligence Report, Proposed 87-Acre 146-Lot Canyon Ranch Residential Development, South of Barton Road and West of San Timoteo Creek, Cities of Redlands and Loma Linda, San Bernardino County, California, PN 12828.001, dated August 14, 2020.

Mayer Creative, 2020, Canyon Ranch Conceptual Site Plan, dated June 29, 2020.

Nationwide Environmental Title Research (NETR), 2019, NETR Online, Historic Aerials, website: <https://www.historicaerials.com>, accessed April, 2021.

Public Works Standard, Inc., 2018, Greenbook, Standard Specifications for Public Works Construction: 2018 Edition, BNI Building News, Anaheim, California.

Proactive Engineering, Inc., 2021, Canyon Ranch Tentative Tract Map, APN 0293-141-38 & 0293-132-01, City of Redlands; 60-Scale, Sheet 1

San Bernardino County Development Code, 2009, Geologic Hazard (GH) Overlay, Chapter 82.15, Amended Ordinance 4067, updated 2009.

_____, 2010, San Bernardino County Land Use Plan, General Plan, Geologic Hazard Overlays, Redlands FH31-C, Scale 1:14,400, plot date March 9, 2010.

Southern California Earthquake Center (SCEC), 2018, San Jacinto Fault Zone, www.data.scec.org/fault_index/sanjacin.html, website viewed on April, 2021.

Tokimatsu, K., and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, ASCE Journal of Geotechnical Engineering, Vol. 113, No. 8, dated August 1987.

Topozada, T.R., Borchardt, G., Hallstrom, C., Johnson, C., Per, R., and Lagario, H. 1993, Planning scenario for a major earthquake on the San Jacinto fault Riverside and San Bernardino Counties, California: California Geological Survey, Special Publication 102, 219 p.

United States Geological Survey (USGS), 2006, Geologic Map of the San Bernardino and Santa Ana 30'x60' quadrangles, California, Version 1.0, Open File Report 2006-1217.

California Geological Survey (CGS), 2006, Quaternary fault and fold database for the United States, accessed March 30, 2019, from USGS web site: <https://earthquake.usgs.gov/hazards/qfaults>.

_____, 2003, Geologic Map of the Redlands 7.5' Quadrangle and Riverside Counties, California, Version 1.0, dated 2003.

_____, 2008b, National Seismic Hazard Maps – Fault Parameters, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm

_____, 2015a, Interactive Fault Map, <http://earthquake.usgs.gov/hazards/qfaults/map/>

_____, 2015b, Interactive Geologic Map, <http://ngmdb.usgs.gov/maps/MapView/>

_____, 2020, Earthquake Hazards Program, Unified Hazard Tool, <<https://earthquake.usgs.gov/hazards/interactive/>>.

WGCEP - Working Group on California Earthquake Probabilities, 1995, Seismic Hazards in Southern California: Probable Earthquake Probabilities, Bull. Seismic. Soc. Amer., Vol. 85, No. 2, pp. 379-439.



Project: 12828.005	Eng/Geol: JLH
Scale: 1" = 2,000'	Date: April 2021
Base Map: ESRI ArcGIS Online 2021 Thematic Information: Leighton Author: Leighton Geomatics (btran)	

SITE LOCATION MAP

Proposed Canyon Ranch Development (Redlands 27)
Northeast Corner of San Timoteo Canyon Road and Nevada Street
City of Redlands, California

Figure 1

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Legend

Qya3

Young Axial-Valley Deposits, Unit 3

Qvoa3

Very Old Axial-Valley Deposits, Unit 3

Approximate Location of Exploratory Boring
Showing Total Depth (T.D.) in Feet Below Ground Surface.

Approximate Location of Percolation Test Boring
Showing Total Depth (T.D.) and Test Depth in Parentheses.
Infiltration Rate Reported as Inches per Hour (in/hr).

A

A'

Geotechnical Cross Section

Approximate Site Boundary

Geologic Contact (dashed where approximate)

Legend

- Qya3** Young Axial-Valley Deposits, Unit 3
- Qvoa3** Very Old Axial-Valley Deposits, Unit 3
- Approximate Location of Exploratory Boring Showing Total Depth (T.D.) in Feet Below Ground Surface.
- Approximate Location of Percolation Test Boring Showing Total Depth (T.D.) and Test Depth in Parentheses. Infiltration Rate Reported as Inches per Hour (in/hr).
- Geotechnical Cross Section
- Approximate Site Boundary
- Geologic Contact (dashed where approximate)

Map Details:

- Tract Boundary:** Red line
- Lot Dimensions:** Various lot sizes and dimensions are shown.
- Geologic Units:** Qya3 (Young Axial-Valley Deposits, Unit 3) and Qvoa3 (Very Old Axial-Valley Deposits, Unit 3).
- Boring Locations:** LB-1, LB-2, LB-3, LP-1. T.D. values are provided for each.
- Percolation Test:** ITEM 13, T.D. 26.5', (5'-10') 5.0 in/hr.
- Scale:** 1" = 60'
- North Arrow:** Points North.
- City of Redlands:** Canyon Ranch Tentative Tract Map.

Project Information:

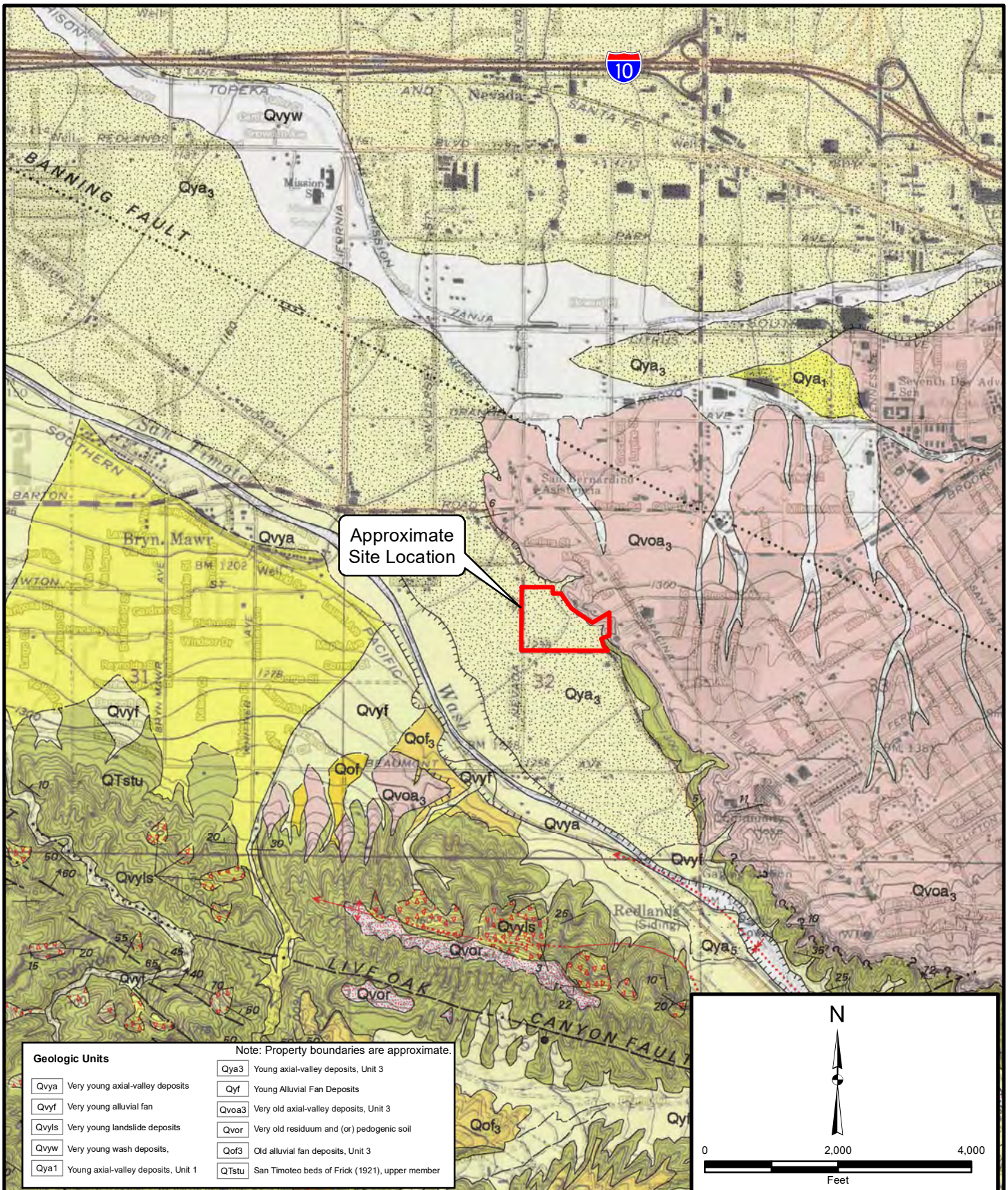
- Project: 12828.005
- Eng/Geol: JLH
- Scale: 1" = 150'
- Date: April 2021
- Base Map: Canyon Ranch Tentative Tract Map, 20,000 SQ FT Min Lots, Sheet 1 of 1
- Thematic Information: Leighton
- Author: Leighton Geomatics (btran)

Map Title: GEOTECHNICAL MAP
Proposed Canyon Ranch Development (Redlands 27)
Northeast Corner of San Timoteo Canyon Road and Nevada Street
City of Redlands, California

Project: 12828.005	Eng/Geol: JLH
Scale: 1" = 150'	Date: April 2021
Base Map: Canyon Ranch Tentative Tract Map 20,000 SQ FT Min Lots, Sheet 1 of 1 Thematic Information: Leighton Author: Leighton Geomatics (btran)	

Map Saved as P:\Drafting\12828\005\Maps\12828.005_F02_GM_2021-04-15.mxd on 4/15/2021 3:05:27 PM

Figure 2



Project: 12828.005

Eng/Geol: JLH

Scale: 1" = 2,000'

Date: April 2021

Base Map: ESRI ArcGIS Online 2021

Thematic Information: Leighton; USGS Geology 2003

Author: (btran)

SITE GEOLOGY MAP Proposed Canyon Ranch Development (Redlands 27) Northeast Corner of San Timoteo Canyon Road and Nevada Street City of Redlands, California

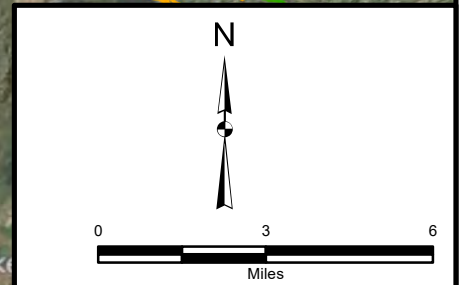
Figure 3



Leighton



Faults	Historic Seismicity
AGE	Magnitude_Value
— Historic (<200 years)	● 4.0 - 5.0
— Holocene (<10K years)	● 5.0 - 6.0
— Quaternary (<1.6M years)	● 6.0 - 7.0
— Pre-Quaternary (>1.6M years)	● 7.0 - 8.0



Project: 12828.005	Eng/Geol: JLH
Scale: 1" = 3 miles	Date: April 2021
Base Map: ESRI ArcGIS Online 2021 Thematic Information: Leighton; CGS, USGS Author: (btran)	

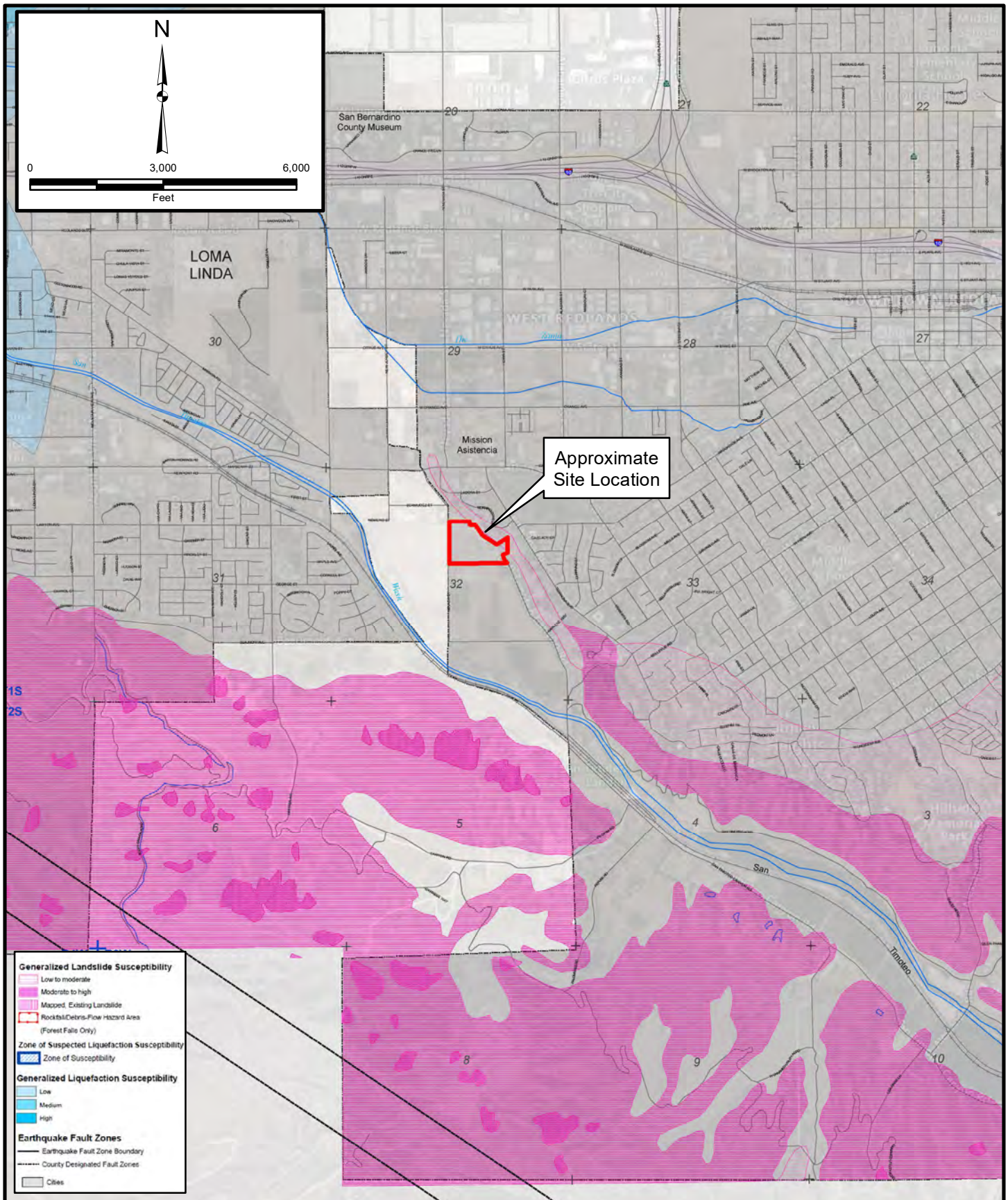
REGIONAL FAULT AND HISTORIC SEISMICITY MAP

Proposed Canyon Ranch Development (Redlands 27)
Northeast Corner of San Timoteo Canyon Road and Nevada Street
City of Redlands, California

Figure 4



Leighton



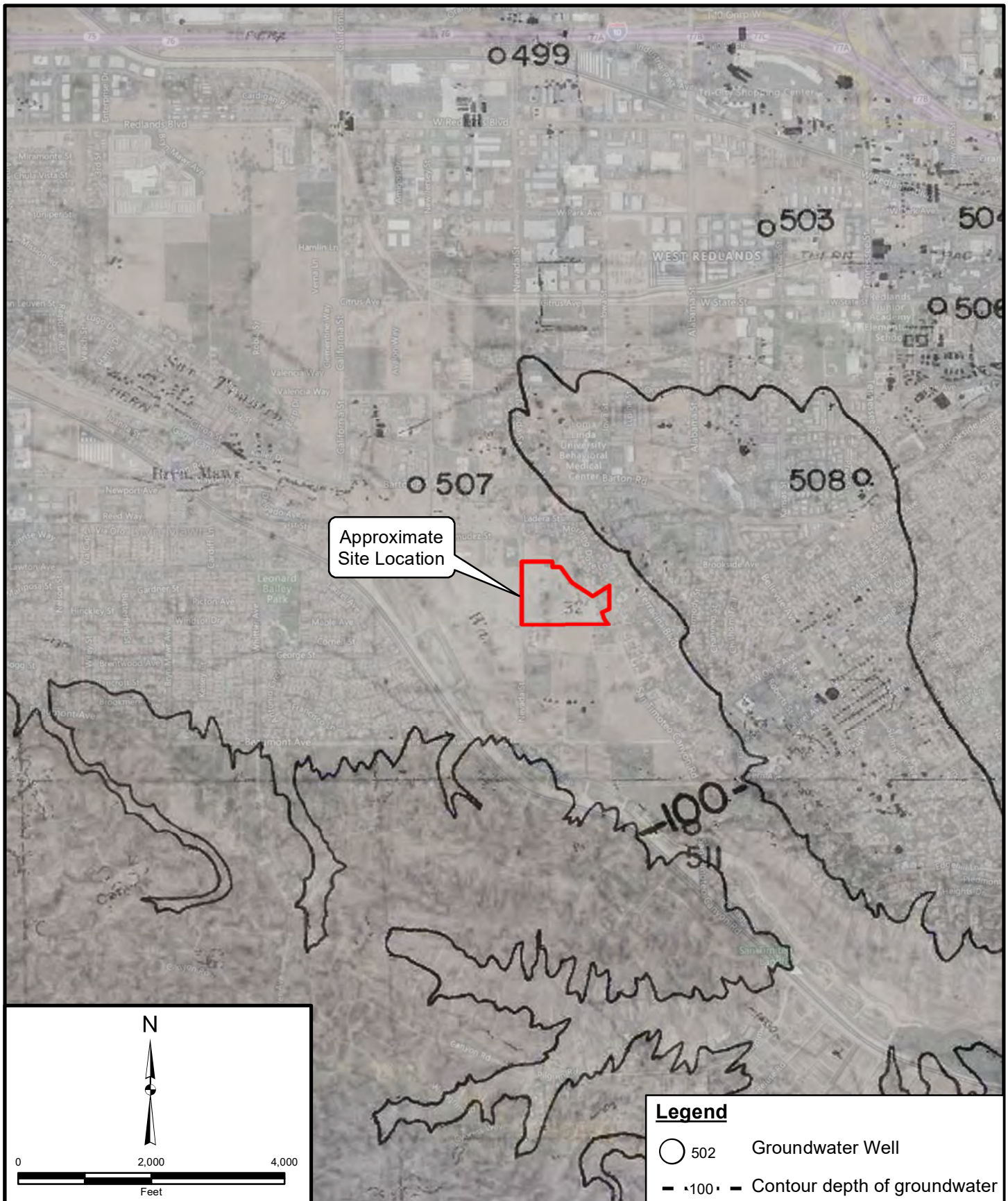
Project: 12828.005	Eng/Geol: JLH
Scale: 1" = 3,000'	Date: April 2021
Base Map: ESRI ArcGIS Online 2021, Thematic Information: Leighton, CGS Author: Leighton Geomatics (btran)	

GEOLOGIC HAZARD MAP

Proposed Canyon Ranch Development (Redlands 27)
Northeast Corner of San Timoteo Canyon Road and Nevada Street
City of Redlands, California

Figure 5

Leighton



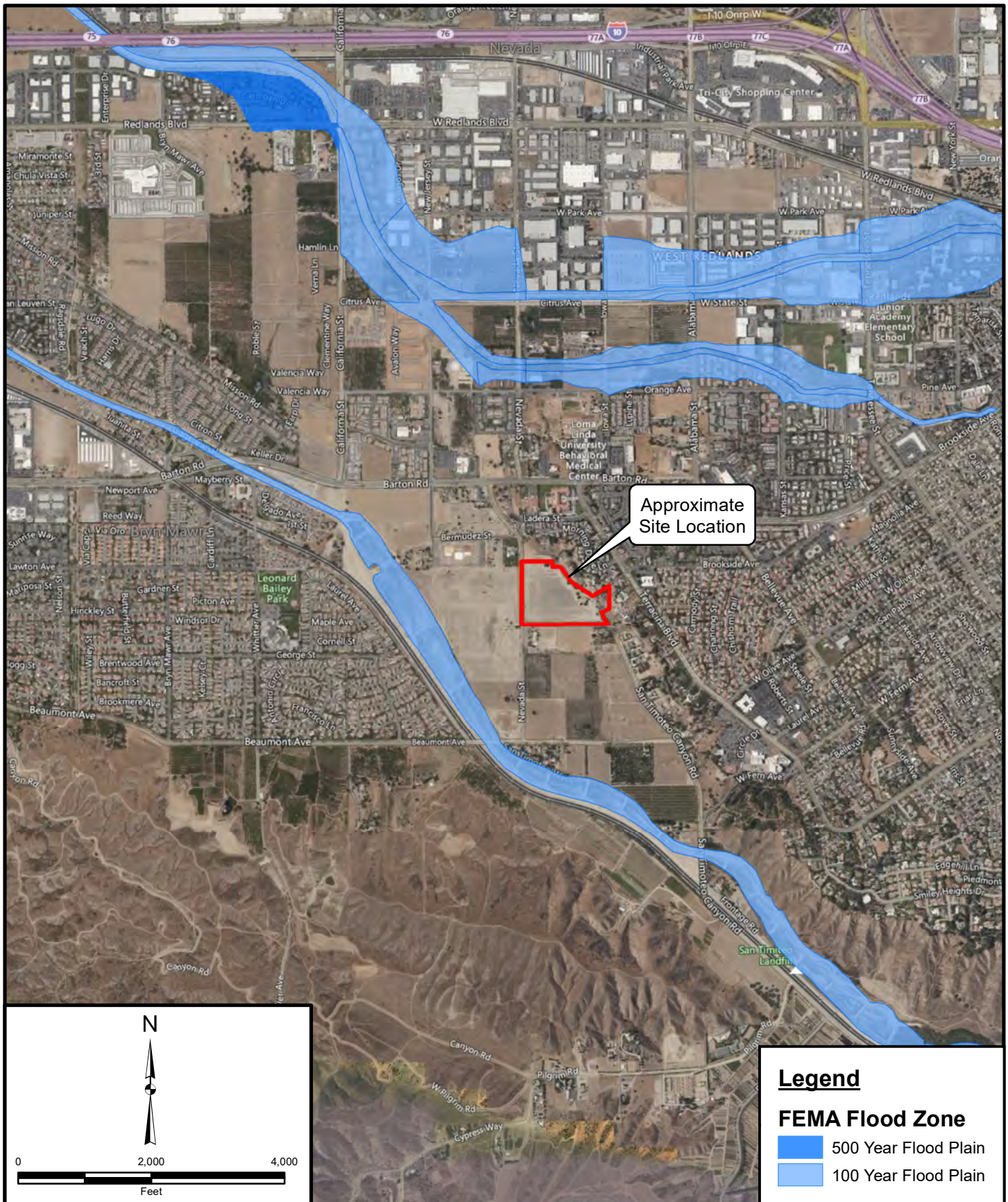
Project: 12828.005	Eng/Geol: JLH
Scale: 1" = 2,000'	Date: April 2021
Base Map: ESRI ArcGIS Online 2021 Thematic Information: Leighton, Carson and Matti, 1985 Author: Leighton Geomatics (btran)	

DEPTH TO GROUNDWATER MAP 1975-1979

Proposed Canyon Ranch Development (Redlands 27)
Northeast Corner of San Timoteo Canyon Road and Nevada Street
City of Redlands, California

Figure 6





Project: 12828.005	Eng/Geol: JLH
Scale: 1" = 2,000'	Date: April 2021
Base Map: ESRI ArcGIS Online 2021 Thematic Information: Leighton, FEMA Author: Leighton Geomatics (btran)	

FLOOD HAZARD ZONE MAP

Proposed Canyon Ranch Development (Redlands 27)
Northeast Corner of San Timoteo Canyon Road and Nevada Street
City of Redlands, California

Figure 7

Leighton



Project: 12828.005	Eng/Geol: JLH
Scale: 1" = 4,000'	Date: April 2021
Base Map: ESRI ArcGIS Online 2021 Thematic Information: Leighton, CA DWR, FEMA Author: Leighton Geomatics (btran)	

DAM INUNDATION MAP

Proposed Canyon Ranch Development (Redlands 27)
Northeast Corner of San Timoteo Canyon Road and Nevada Street
City of Redlands, California

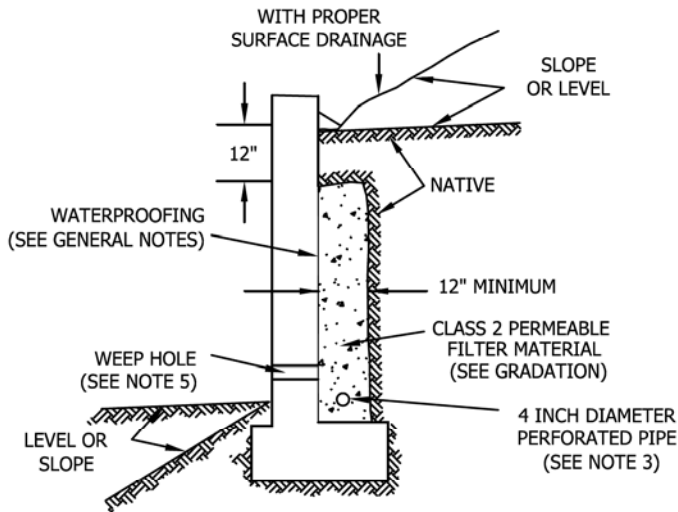
Figure 8



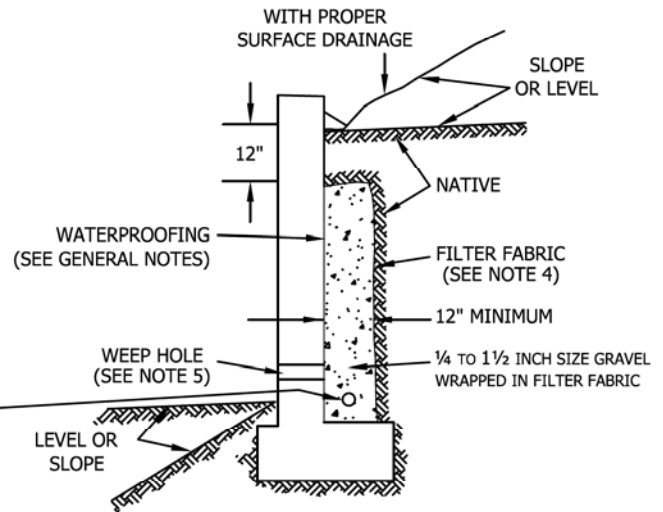
Leighton

SUBDRAIN OPTIONS AND BACKFILL WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤ 50

OPTION 1: PIPE SURROUNDED WITH
CLASS 2 PERMEABLE MATERIAL



OPTION 2: GRAVEL WRAPPED
IN FILTER FABRIC



Class 2 Filter Permeable Material Gradation
Per Caltrans Specifications

<u>Sieve Size</u>	<u>Percent Passing</u>
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

GENERAL NOTES:

- * Waterproofing should be provided where moisture nuisance problem through the wall is undesirable.
- * Water proofing of the walls is not under purview of the geotechnical engineer
- * All drains should have a gradient of 1 percent minimum
- * Outlet portion of the subdrain should have a 4-inch diameter solid pipe discharged into a suitable disposal area designed by the project engineer. The subdrain pipe should be accessible for maintenance (rodding)
- * Other subdrain backfill options are subject to the review by the geotechnical engineer and modification of design parameters.

Notes:

- 1) Sand should have a sand equivalent of 30 or greater and may be densified by water jetting.
- 2) 1 Cu. ft. per ft. of 1/4- to 1 1/2-inch size gravel wrapped in filter fabric
- 3) Pipe type should be ASTM D1527 Acrylonitrile Butadiene Styrene (ABS) SDR35 or ASTM D1785 Polyvinyl Chloride plastic (PVC), Schedule 40, Armco A2000 PVC, or approved equivalent. Pipe should be installed with perforations down. Perforations should be 3/8 inch in diameter placed at the ends of a 120-degree arc in two rows at 3-inch on center (staggered)
- 4) Filter fabric should be Mirafi 140NC or approved equivalent.
- 5) Weephole should be 3-inch minimum diameter and provided at 10-foot maximum intervals. If exposure is permitted, weepholes should be located 12 inches above finished grade. If exposure is not permitted such as for a wall adjacent to a sidewalk/curb, a pipe under the sidewalk to be discharged through the curb face or equivalent should be provided. For a basement-type wall, a proper subdrain outlet system should be provided.
- 6) Retaining wall plans should be reviewed and approved by the geotechnical engineer.
- 7) Walls over six feet in height are subject to a special review by the geotechnical engineer and modifications to the above requirements.

**RETAINING WALL BACKFILL AND SUBDRAIN DETAIL
FOR WALLS 6 FEET OR LESS IN HEIGHT**
WHEN NATIVE MATERIAL HAS EXPANSION INDEX OF ≤ 50



Figure 9

APPENDIX A

EXPLORATION LOGS



Leighton

GEOTECHNICAL BORING LOG LB-1

Project No. 12828.005
Project Highpointe Canyon Ranch LLC - 27-Lots
Drilling Co. Baja Exploration
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Geotechnical Map

Date Drilled 2-18-21
Logged By KMD
Hole Diameter 8"
Ground Elevation ~1247'
Sampled By KMD

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	Type of Tests
	0	N S		BB-1				SM	Quaternary Young Axial-Valley Deposits, Unit 3 (Qya3) @0': SILTY SAND, orangish brown, moist, predominantly fine sand, some medium to coarse sand, some fine to medium gravel	DS, MD
	5			R-1	8 11 15	101	3	(CL-ML)s	Quaternary Very Old Axial-Valley Deposits, Unit 3 (Qvoa3) @5': CLAY with SAND, light orangish brown, slightly moist to moist, stiff, predominantly fine sand, some medium to coarse sand	CN
	10			R-2	10 12 14	114	2	SM	@10': SILTY SAND, orangish brown, moist, predominantly fine sand, some medium to coarse sand, some fine gravels, especially localized in thin beds, slightly to moderately cemented	
	15			R-3	11 13 25	105	4		@15': Interbedded SAND and SILTY SAND, light greyish brown, moist, medium dense, predominantly fine sand; sand has some medium to coarse sand @16.5': Predominantly medium to coarse sand, some fine gravel	
	20			R-4	20 27 40	110	2	SPg	@20': SAND with GRAVEL to GRAVELLY SAND, light grey, moist, dense, fine to coarse sand, fine to medium gravel	
	25			S-1	20 50/6"		2		@25': GRAVELLY SAND, light grey to brownish grey, moist, very dense, fine to coarse sand, fine to medium mechanically broken gravel	
	30								Total Depth of Boring: 26.5 Feet No Groundwater encountered during drilling Boring backfilled with soil cuttings on 2-18-2021.	

SAMPLE TYPES:

B BULK SAMPLE
 C CORE SAMPLE
 G GRAB SAMPLE
 R RING SAMPLE
 S SPLIT SPOON SAMPLE
 T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-2

Project No. 12828.005
Project Highpointe Canyon Ranch LLC - 27-Lots
Drilling Co. Baja Exploration
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Geotechnical Map

Date Drilled 2-18-21
Logged By KMD
Hole Diameter 8"
Ground Elevation ~1237'
Sampled By KMD

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	Type of Tests
	0	N S		BB-1				SM	Quaternary Young Axial-Valley Deposits, Unit 3 (Qya3) @0': SILTY SAND, brown, moist, predominantly fine sand, some medium to coarse sand, fine gravel	CR, EI, RV
	5			R-1	5 5 7	108	7		@5': SILTY SAND with GRAVEL, loose	-200
	10			R-2	5 8 19	121	2		@10': SILTY SAND with GRAVEL, olive brown, moist, very stiff, fine sand, trace to few medium to coarse sand	-200
	15			R-3	15 19 34	121	15	CL-ML SM	Quaternary Very Old Axial-Valley Deposits, Unit 3 (Qvoa3) @11': SILTY GRAVELLY SAND, brown, moist, fine to coarse sand, fine to medium gravel, fine gravel, trace medium gravel @11.1': SAND with GRAVEL, grey, moist, medium dense, fine to coarse sand, fine gravel, trace medium gravel @15': SILTY CLAY, light greyish brown, moist, hard, with CaCO ₃ , grading to Sandy SILT, brown, moist, fine sand, trace medium to coarse sand, some clay @16.25': SILTY SAND, brown, moist, dense, fine sand	
	20			S-1	11 12 15		5	SP SM	@20': SAND, tan, moist, medium dense, fine sand, trace medium to coarse sand @20.75': SILTY SAND, brown, moist, fine sand, massive	
	25			S-2	16 20 24		8	SP CL-ML	@25': SAND, light grey, moist, dense, predominantly fine sand, few medium to coarse sand @26.25': SILTY CLAY, mottled lighter and darker reddish brown, moist, hard, some CaCO ₃ veins, FeO-stained zones, some fine sand	
	30								Total Depth of Boring: 26.5 Feet No Groundwater encountered during drilling Boring backfilled with soil cuttings on 2-18-2021.	

SAMPLE TYPES:

B BULK SAMPLE
 C CORE SAMPLE
 G GRAB SAMPLE
 R RING SAMPLE
 S SPLIT SPOON SAMPLE
 T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-3

Project No. 12828.005
Project Highpointe Canyon Ranch LLC - 27-Lots
Drilling Co. Baja Exploration
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Geotechnical Map

Date Drilled 2-18-21
Logged By KMD
Hole Diameter 8"
Ground Elevation ~1234'
Sampled By KMD

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	Type of Tests
	0	N S						SMg	Quaternary Young Axial-Valley Deposits, Unit 3 (Qya3) @0': GRAVELLY SILTY SAND, moist, fine to coarse sand, fine to medium gravel	
	5			R-1	10 11 12	128	2	SPg	@5': GRAVELLY SAND, greyish brown, moist, medium dense, fine to coarse sand, fine to coarse gravel (mechanically broken, subrounded, igneous and metamorphic)	
	10			R-2	5 6 10	114	2	ML	@10': SANDY SILT, olive brown, moist, stiff, predominantly fine sand, few medium to coarse sand, massive @11.25': SAND, tan, moist, medium dense, fine sand, few to some silt	
	15			R-3	10 17 21	106	3	SPg	@15': SAND with GRAVEL, reddish brown, very moist, medium dense, fine to medium dense, some coarse sand, few to some fine gravel, some clay, grading coarser to fine to coarse sand, slightly more fine gravel, no clay, tan	
	20			S-1	3 4 7		10	SP-SM	@20': Interbedded SAND and SILTY SAND, light grey brown and olive brown, respectively, moist, medium dense, very fine to fine sand, trace coarse sand, gradational contacts @20.5': Few laminations of SILT	-200
	25			S-2	4 3 5		26	(CL-ML)s SM	@25': SILTY CLAY with SAND, brown, very moist, firm, very fine sand, massive @26.33': SILTY SAND, brown, very moist, loose, predominantly medium sand, some fine sand, trace coarse sand	-200
	30									

SAMPLE TYPES:

B BULK SAMPLE
 C CORE SAMPLE
 G GRAB SAMPLE
 R RING SAMPLE
 S SPLIT SPOON SAMPLE
 T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
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 CU UNDRAINED TRIAXIAL

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SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-3

Project No.	12828.005	Date Drilled	2-18-21
Project	Highpointe Canyon Ranch LLC - 27-Lots	Logged By	KMD
Drilling Co.	Baja Exploration	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	~1234'
Location	See Figure 2 - Geotechnical Map	Sampled By	KMD

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	Type of Tests
	30	N S		S-3	4 7 15		3	SP-SM	@30': SAND with SILT, dark orangeish to reddish brown, moist, medium dense, fine to coarse sand, few fine gravel, massive	-200
	35			S-4	17 21 40			SPg	@35': SAND with GRAVEL to GRAVELLY SAND, light brown, moist, very dense, fine to coarse sand, predominantly fine gravel, trace mechanically broken medium gravel	
	40			S-5	3 4 5		4	SC-SM	@40': SILTY CLAYEY SAND, strong brown, moist to very moist, medium dense, predominantly fine to medium sand, some coarse sand, few to some fine gravel	-200
	45			S-6	6 7 11		10		@46.5': SAND, light grey, moist, medium dense, fine sand	-200
	50			S-7	10 16 25		4	SPg	Quaternary Very Old Axial-Valley Deposits, Unit 3 (Qvoa3) @50': SAND with GRAVEL, light orangish brown, moist, fine to coarse sand, predominantly fine gravel, few medium gravel, weak bedding @51.25': Bedding becomes more clear Total Depth of Boring: 51.5 Feet No Groundwater encountered during drilling Boring backfilled with soil cuttings on 2-18-2021.	
	55									
	60									

SAMPLE TYPES:

B BULK SAMPLE

C CORE SAMPLE

G GRAB SAMPLE

R RING SAMPLE

S SPLIT SPOON SAMPLE

T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING

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SE SAND EQUIVALENT

SG SPECIFIC GRAVITY

UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LB-4

Project No.	12828.005	Date Drilled	2-18-21
Project	Highpointe Canyon Ranch LLC - 27-Lots	Logged By	KMD
Drilling Co.	Baja Exploration	Hole Diameter	8"
Drilling Method	Hollow Stem Auger - 140lb - Autohammer - 30" Drop	Ground Elevation	~1231'
Location	See Figure 2 - Geotechnical Map	Sampled By	KMD

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION <i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	Type of Tests
	0	N S		BB-1				SM	Quaternary Young Axial-Valley Deposits, Unit 3 (Qya3) @0': SILTY SAND, olive brown, moist, fine to medium sand, trace fine gravels, little debris, (AC, concrete fragments)	
	5			R-1	7 7 8	116	10	SMg	@5': SILTY SAND with GRAVEL, olive brown, moist, loose, predominantly fine sand, some medium sand, few coarse sand, few fine gravel (granitic/granodiorite)	
	10			R-2	11 15 20	113	2	SP-SW	@10': SAND, with GRAVEL, light brown, moist, medium dense, fine to coarse sand, fine gravel, little silt	
	15			R-3	22 26 33	109	3		@15': GRAVELLY SAND, dark tan, moist, dense, predominantly medium to coarse sand, with some fine sand, few medium gravel, subrounded	
	20			R-4	10 13 15		3	SP	@20': SAND, light olive brown, moist, medium dense, fine sand @21.25': SANDY SILT with CLAY, olive brown, very moist, very stiff, very fine sand, laminated, low plasticity to non-plastic	
	25			S-1	4 5 7		26	ML	@25': SILT, stiff, grading to SILTY CLAY, olive brown, moist, laminated, low to moderate plasticity	-200
	30								Total Depth of Boring: 26.5 Feet No Groundwater encountered during drilling. Boring backfilled with soil cuttings on 2-18-2021.	

SAMPLE TYPES:

B BULK SAMPLE

C CORE SAMPLE

G GRAB SAMPLE

R RING SAMPLE

S SPLIT SPOON SAMPLE

T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING

AL ATTERBERG LIMITS

CN CONSOLIDATION

CO COLLAPSE

CR CORROSION

CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR

EI EXPANSION INDEX

H HYDROMETER

MD MAXIMUM DENSITY

PP POCKET PENETROMETER

RV R VALUE

SA SIEVE ANALYSIS

SE SAND EQUIVALENT

SG SPECIFIC GRAVITY

UC UNCONFINED COMPRESSIVE STRENGTH



GEOTECHNICAL BORING LOG LP-1

Project No. 12828.005
Project Highpointe Canyon Ranch LLC - 27-Lots
Drilling Co. Baja Exploration
Drilling Method Hollow Stem Auger - 140lb - Autohammer - 30" Drop
Location See Figure 2 - Geotechnical Map

Date Drilled 2-18-21
Logged By KMD
Hole Diameter 8"
Ground Elevation ~1231'
Sampled By KMD

Elevation Feet	Depth Feet	Graphic Log	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0	N S							<i>This Soil Description applies only to a location of the exploration at the time of sampling. Subsurface conditions may differ at other locations and may change with time. The description is a simplification of the actual conditions encountered. Transitions between soil types may be gradual.</i>	
	5			BB-1 S-1	3 3 3		15	s(ML)	Quaternary Young Axial-Valley Deposits, Unit 3 (Qya3) @0': SILTY SAND, olive brown, moist, fine to medium sand, some gravel @5': SANDY SILT, olive brown, moist, firm, fine sand, massive trace medium sand	
	10			S-2	7 11 18		4	(SW-SM)g	@8.5': SAND with SILT and GRAVEL, dark reddish brown, moist, medium dense, fine to coarse sand, fine to medium gravel, slightly clayey, darker red in upper 6-inches Total Depth of Boring: 10 Feet No Groundwater encountered during drilling Boring converted into Percolation Test Boring on 2-18-2021: 2-inch diameter slotted 0.020-inch Screen installed from 5 to 10 Feet; Solid PVC Riser from 12-inches above ground surface to 10 feet below ground surface. Annulus filled with No. 3 Monterey SAND to approximately 4 feet below ground surface. A "measured" infiltration rate of 5 inches per hour was calculated.	
	15									
	20									
	25									
	30									

SAMPLE TYPES:

B BULK SAMPLE
 C CORE SAMPLE
 G GRAB SAMPLE
 R RING SAMPLE
 S SPLIT SPOON SAMPLE
 T TUBE SAMPLE

TYPE OF TESTS:

-200 % FINES PASSING
 AL ATTERBERG LIMITS
 CN CONSOLIDATION
 CO COLLAPSE
 CR CORROSION
 CU UNDRAINED TRIAXIAL

DS DIRECT SHEAR
 EI EXPANSION INDEX
 H HYDROMETER
 MD MAXIMUM DENSITY
 PP POCKET PENETROMETER
 RV R VALUE

SA SIEVE ANALYSIS
 SE SAND EQUIVALENT
 SG SPECIFIC GRAVITY
 UC UNCONFINED COMPRESSIVE STRENGTH



APPENDIX B

PERCOLATION TESTING



Leighton

Boring Percolation Test Data Sheet (Constant Head)

Project Number:	12828.005	Test Hole Number:	LP-1
Project Name:	Cyn Rch 27-Lot	Date Excavated:	2/18/2021
Earth Description:	Alluvium	Date Tested:	2/23/2021
Liquid Description:	Tap water	Depth of boring (ft):	10
Tested By:	JJ	Radius of boring, r (in):	4
		Diameter of casing (in):	2
		Length of slotted of casing (ft):	5
		Depth to Initial Water Depth (ft):	3
		Porosity of Annulus Material, <i>n</i> :	0.35
		Bentonite Plug at Bottom:	No

Field Percolation Data

Reading	Time	Time Interval, Δt (minutes)	Depth to Water (feet bgs)	Water Height, H (inches)	Cumulative Water Volume Delivered (gallons)
1	11:40	-	-	-	
2	11:50	10	3.95	72.6	
3	12:00	10	4.00	72.0	
4	12:10	10	4.10	70.8	14.0
5	12:20	10	4.11	70.7	
6	12:30	10	3.88	73.4	
7	12:40	10	3.88	73.4	
8	12:50	10	3.95	72.6	28.0
9	13:00	10	4.00	72.0	
10	13:10	10	4.00	72.0	
11	13:20	10	4.00	72.0	
12	13:30	10	4.00	72.0	
13	13:40	10	4.01	71.9	82.0

High Flowrate Percolation Test Calculation

Total Volume of Water Delivered (gallons)	82.0
Total Volume of Water Delivered (cubic inches)	18942
Average Water Height (inches)	72.1
Average Percolation Surface Area (cubic Inches)	1862.8
Duration of Test (minutes)	120
Duration of Test (hours)	2.00

Measured Infiltration Rate = (Total Volume)/(Test Duration)/(Surface Area)

Measured Infiltration Rate (inches per hour) = 5.08

APPENDIX C

GEOTECHNICAL LABORATORY TESTING



Leighton

Boring No.	LB-2	LB-2	LB-3	LB-3	LB-3	LB-3	LB-3	LB-4
Sample No.	R-1	R-2	S-1	S-2	S-3	S-5	S-6	S-1
Depth (ft.)	5.0	10.0	20.0	25	30	40	45	25
Sample Type	Ring	Ring	SPT	SPT	SPT	SPT	SPT	SPT
Soil Identification	Brown silty sand with gravel (SM)g	Light olive gray silty sand with gravel (SM)g	Brown silty sand (SM)	Brown silty clay with sand (CL-ML)s	Brown poorly-graded sand with silt (SP-SM)	Brown silty, clayey sand (SC-SM)	Brown silty, clayey sand (SC-SM)	Brown silt (ML)
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Weight of Container (g)	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Moisture Content (%)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Sample Dry Weight Determination								
Weight of Sample + Container (g)	905.70	969.80	908.20	670.50	782.10	990.90	585.80	673.40
Weight of Container (g)	248.20	107.88	234.40	248.70	244.00	245.70	249.90	220.30
Weight of Dry Sample (g)	657.50	861.92	673.80	421.80	538.10	745.20	335.90	453.10
Container No.:								
After Wash								
Method (A or B)	A	A	A	A	A	A	A	A
Dry Weight of Sample + Cont. (g)	685.60	863.90	591.10	337.30	752.20	722.90	478.10	281.00
Weight of Container (g)	248.20	107.88	234.40	248.70	244.00	245.70	249.90	220.30
Dry Weight of Sample (g)	437.40	756.02	356.70	88.60	508.20	477.20	228.20	60.70
% Passing No. 200 Sieve	33.5	12.3	47.1	79.0	5.6	36.0	32.1	86.6
% Retained No. 200 Sieve	66.5	87.7	52.9	21.0	94.4	64.0	67.9	13.4
 PERCENT PASSING No. 200 SIEVE ASTM D 1140 Project Name: <u>HCR Redlands (37-Lots)</u> Project No.: <u>12828.005</u> Client Name: _____ Tested By: <u>S. Felter</u> Date: <u>03/04/21</u>								

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: HCR Redlands (37-Lots)

Tested By: Y. Nguyen

Date: 03/04/21

Project No.: 12828.005

Checked By: A. Santos

Date: 03/24/21

Boring No.: LP-1

Sample No.: S-1

Depth (feet): 5.0

Soil Identification: Brown sandy silt s(ML)

% Gravel	0	Soil Type	Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
% Sand	42	s(ML)			
% Fines	58				
Specific Gravity (Assumed)	2.70	Wt. of Air-Dry Soil + Cont. (g)	0.00	78.03	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	77.90	99.20
Wt. of Air-Dry Soil + Cont. (g)	538.39	Wt. of Container No. ____ (g)	1.00	57.38	77.22
Wt. of Container	99.95	Moisture Content (%)	0.00	0.63	
Dry Wt. of Soil (g)	438.44	Wt. of Dry Soil (g)			21.98

Coarse Sieve		
U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
¾"	0.00	100.0
⅜"	0.00	100.0
No. 4	0.00	100.0
No. 10	6.17	98.6
Pan		

Sieve after Hydrometer & Wet Sieve			
U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	98.6
No. 16	1.23	97.6	96.2
No. 30	3.62	92.8	91.5
No. 50	7.47	85.2	84.0
No. 100	12.53	75.2	74.1
No. 200	20.60	59.2	58.4
Pan			

Hydrometer

Wt. of Air-Dry Soil (g)

50.84

Wt. of Dry Soil (g)

50.52

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
05-Mar-21	9:00	0		8.0			
	9:02	2	25.4	8.0	26.0	34.8	0.0310
	9:05	5	25.4	8.0	23.0	29.0	0.0200
	9:15	15	25.2	8.0	20.0	23.2	0.0118
	9:30	30	24.8	8.0	18.0	19.4	0.0085
	10:00	60	24.1	8.0	16.5	16.5	0.0061
	11:00	120	23.4	8.0	15.0	13.6	0.0044
	13:10	250	23.2	8.0	14.5	12.6	0.0031
06-Mar-21	9:00	1440	22.3	8.0	14.0	11.6	0.0013

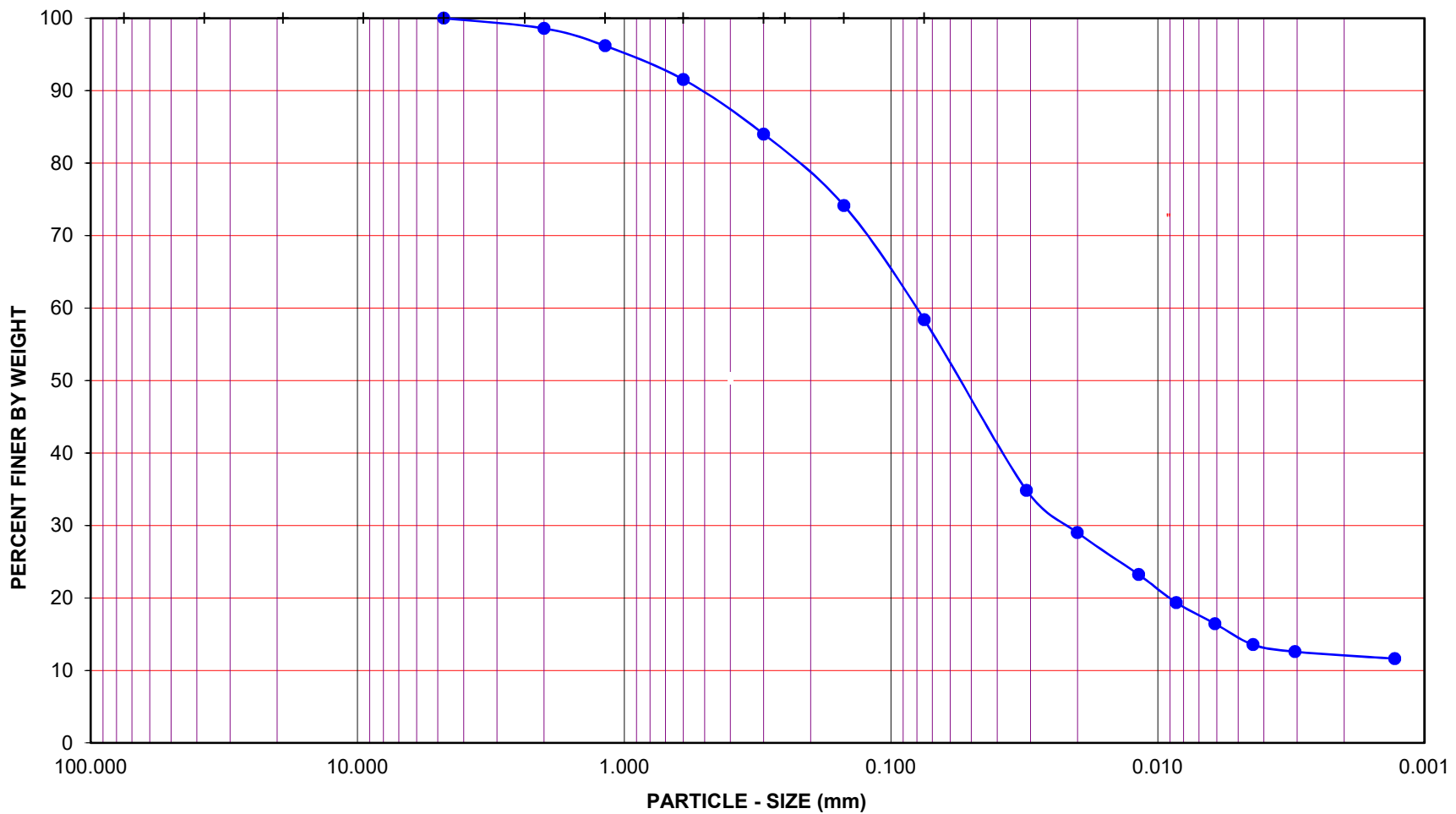
GRAVEL				SAND				FINES		
COARSE		FINE		CRSE	MEDIUM	FINE		SILT		CLAY

U.S. STANDARD SIEVE OPENING

U.S. STANDARD SIEVE NUMBER

HYDROMETER

3.0" 1 1/2" 3/4" 3/8" #4 #8 #16 #30 #50 #100 #200



Project Name: HCR Redlands (37-Lots)

Project No.: 12828.005

Boring No.: LP-1

Sample No.: S-1

Depth (feet): 5.0

Soil Type : s(ML)

Soil Identification: Brown sandy silt s(ML)

GR:SA:FI : (%) **0 : 42 : 58**



**PARTICLE - SIZE
DISTRIBUTION
ASTM D 422**

Mar-21

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 422

Project Name: HCR Redlands (37-Lots)

Tested By: Y. Nguyen

Date: 03/04/21

Project No.: 12828.005

Checked By: A. Santos

Date: 03/24/21

Boring No.: LP-1

Sample No.: S-2

Depth (feet): 8.5

Soil Identification: Light brown well-graded sand with silt and gravel (SW-SM)g

% Gravel	25	Soil Type	Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
% Sand	63	(SW-SM)g			
% Fines	12				
Specific Gravity (Assumed)	2.70	Wt. of Air-Dry Soil + Cont. (g)	0.00	105.92	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	105.83	159.57
Wt. of Air-Dry Soil + Cont. (g)	751.99	Wt. of Container No. ____ (g)	1.00	72.43	77.55
Wt. of Container	108.13	Moisture Content (%)	0.00	0.27	
Dry Wt. of Soil (g)	643.86	Wt. of Dry Soil (g)			82.02

Coarse Sieve		
U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
¾"	39.76	93.8
⅜"	106.93	83.4
No. 4	162.03	74.8
No. 10	222.55	65.4
Pan		

Sieve after Hydrometer & Wet Sieve			
U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	65.4
No. 16	13.26	86.8	56.8
No. 30	32.88	67.2	44.0
No. 50	55.30	44.8	29.3
No. 100	72.43	27.8	18.2
No. 200	81.36	18.9	12.3
Pan			

Hydrometer

Wt. of Air-Dry Soil (g)

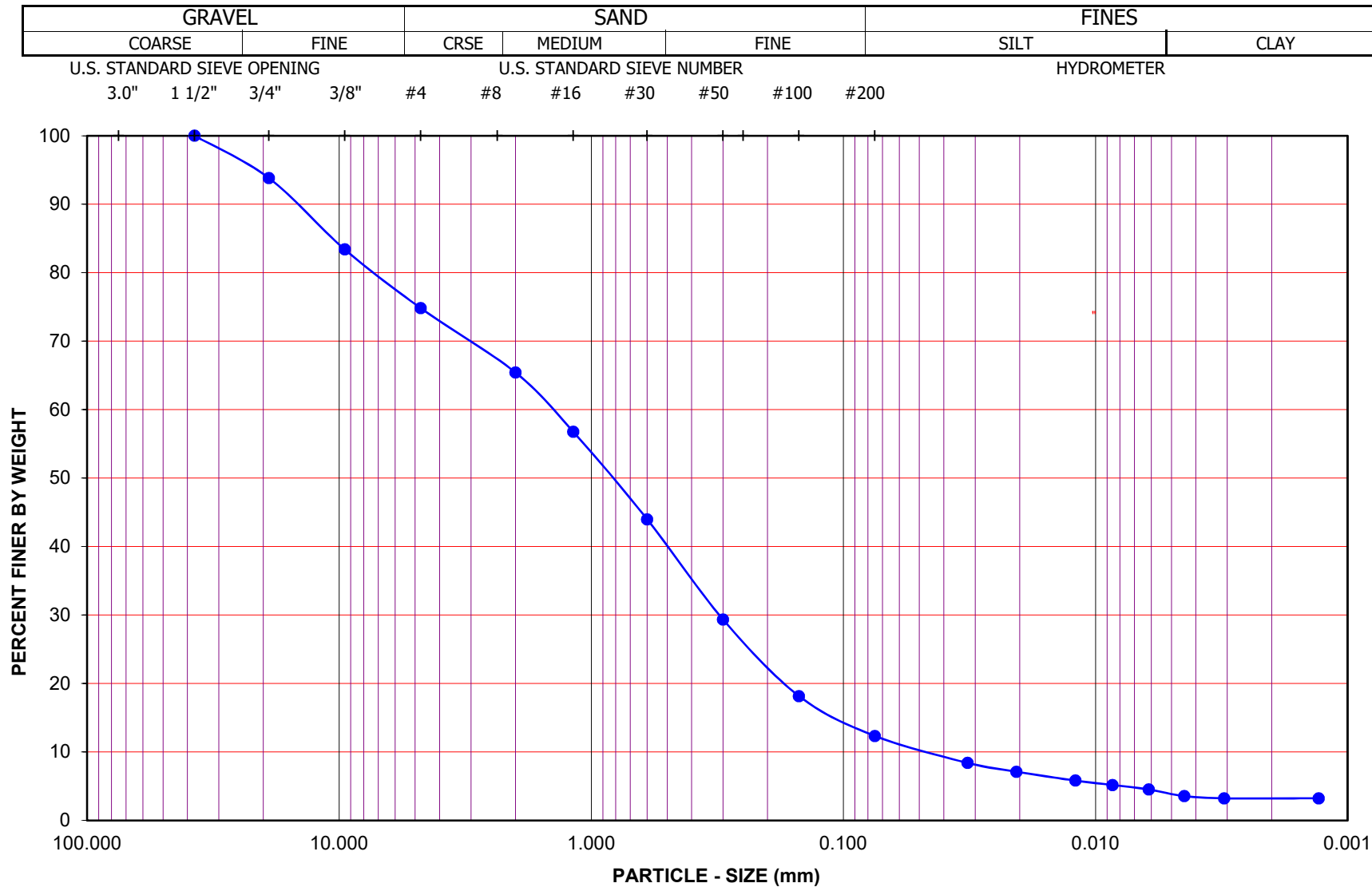
100.54

Wt. of Dry Soil (g)

100.27

Deflocculant 125 cc of 4% Solution

Date	Time	Elapsed Time (min)	Water Temperature (°C)	Composite Correction 152H	Actual Hydrometer Readings	% Total Sample (%)	Soil Particle Diameter (mm)
05-Mar-21	9:04	0		8.0			
	9:06	2	25.9	8.0	21.0	8.4	0.0322
	9:09	5	25.8	8.0	19.0	7.1	0.0206
	9:19	15	25.6	8.0	17.0	5.8	0.0120
	9:34	30	25.1	8.0	16.0	5.2	0.0086
	10:04	60	24.3	8.0	15.0	4.5	0.0061
	11:04	120	23.5	8.0	13.5	3.6	0.0044
	13:14	250	23.4	8.0	13.0	3.2	0.0031
06-Mar-21	9:04	1440	22.2	8.0	13.0	3.2	0.0013



Project Name: HCR Redlands (37-Lots)

Project No.: 12828.005

Boring No.: LP-1

Depth (feet): 8.5

Soil Identification: Light brown well-graded sand with silt and gravel (SW-SM)g

Sample No.: S-2

Soil Type : (SW-SM)g

**PARTICLE - SIZE
DISTRIBUTION
ASTM D 422**

GR:SA:FI : (%) 25 : 63 : 12

Mar-21



EXPANSION INDEX of SOILS

ASTM D 4829

Project Name: HCR Redlands (37-Lots) Tested By: S. Felter Date: 03/04/21
 Project No.: 12828.005 Checked By: A. Santos Date: 03/24/21
 Boring No.: LB-2 Depth (ft.): 0-5
 Sample No.: B-1
 Soil Identification: Olive brown silty sand (SM)

Dry Wt. of Soil + Cont.	(g)	1000.00
Wt. of Container No.	(g)	0.00
Dry Wt. of Soil	(g)	1000.00
Weight Soil Retained on #4 Sieve		0.00
Percent Passing # 4		100.00

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0010
Wt. Comp. Soil + Mold (g)	613.70	444.30
Wt. of Mold (g)	188.40	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	0	0
Wet Wt. of Soil + Cont. (g)	842.70	632.70
Dry Wt. of Soil + Cont. (g)	780.30	582.20
Wt. of Container (g)	0.00	188.40
Moisture Content (%)	8.00	12.82
Wet Density (pcf)	128.3	133.9
Dry Density (pcf)	118.8	118.7
Void Ratio	0.419	0.421
Total Porosity	0.295	0.296
Pore Volume (cc)	61.1	61.3
Degree of Saturation (%) [S _{meas}]	51.5	82.3

SPECIMEN INUNDATION in distilled water for the period of 24 h or expansion rate < 0.0002 in./h

Date	Time	Pressure (psi)	Elapsed Time (min.)	Dial Readings (in.)
03/04/21	9:35	1.0	0	0.6430
03/04/21	9:45	1.0	10	0.6430
Add Distilled Water to the Specimen				
03/04/21	12:30	1.0	165	0.6440
03/05/21	6:55	1.0	1270	0.6440
03/05/21	8:00	1.0	1335	0.6440

Expansion Index (EI _{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	1
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ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS ASTM D 2435

Project Name: HCR Redlands (37-Lots)

Project No.: 12828.005

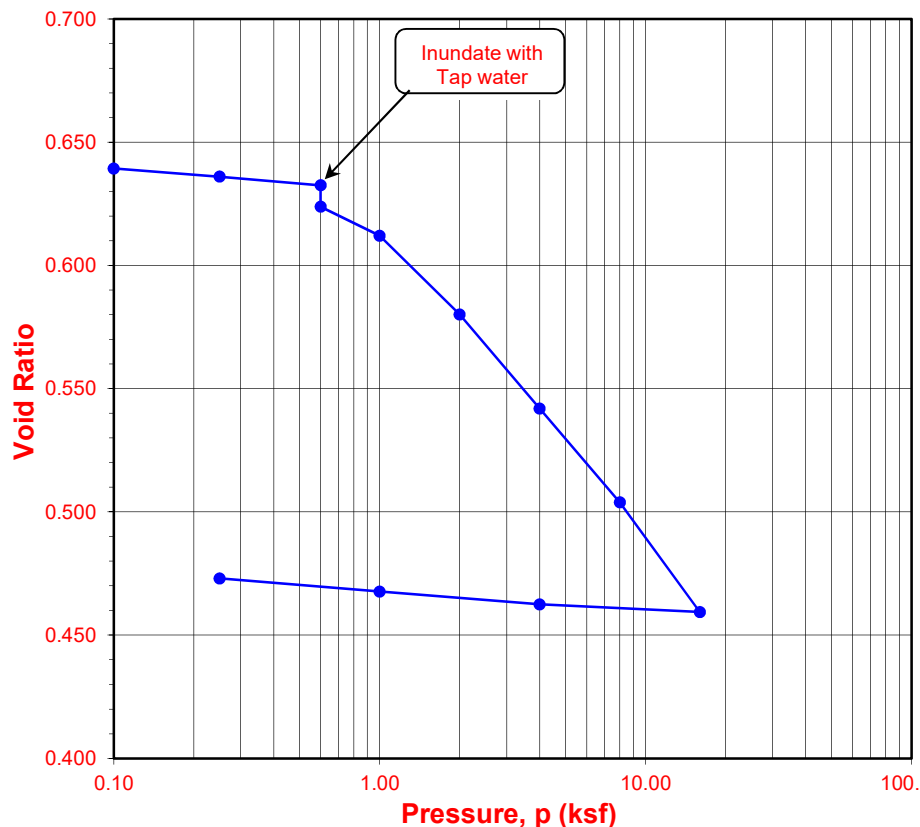
Boring No.: LB-1

Sample No.: R-1

Soil Identification: Light olive brown silty clay with sand (CL-ML)s

Tested By: G. Bathala Date: 02/28/21
 Checked By: A. Santos Date: 03/24/21
 Depth (ft.): 5.0
 Sample Type: Ring

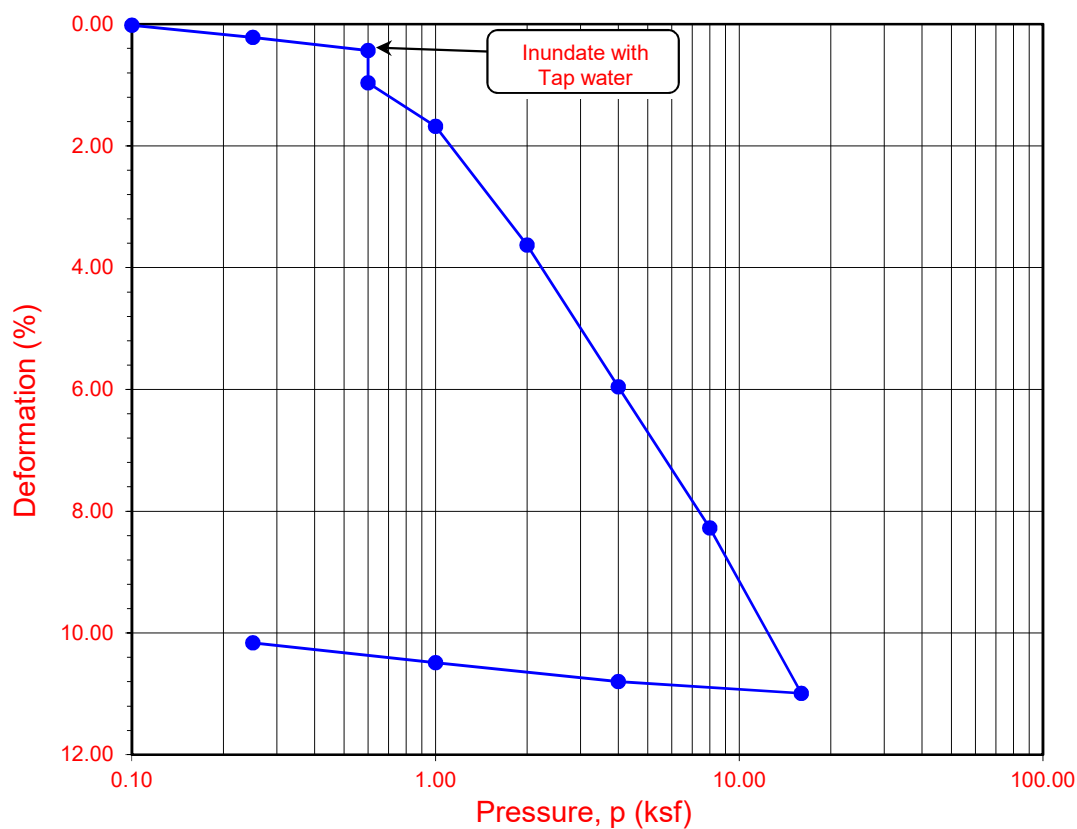
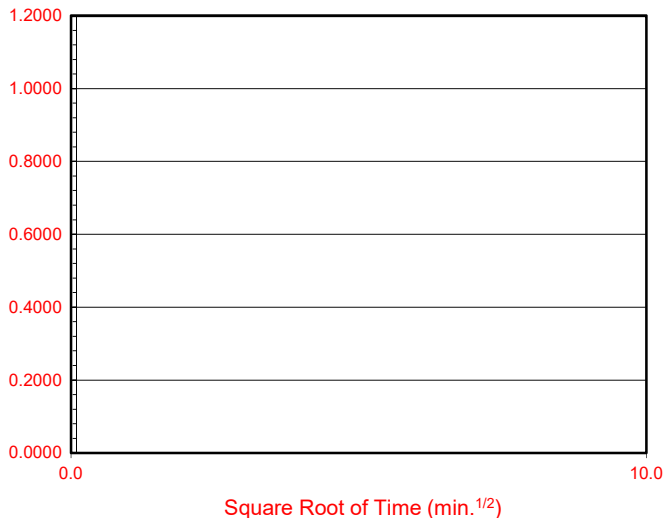
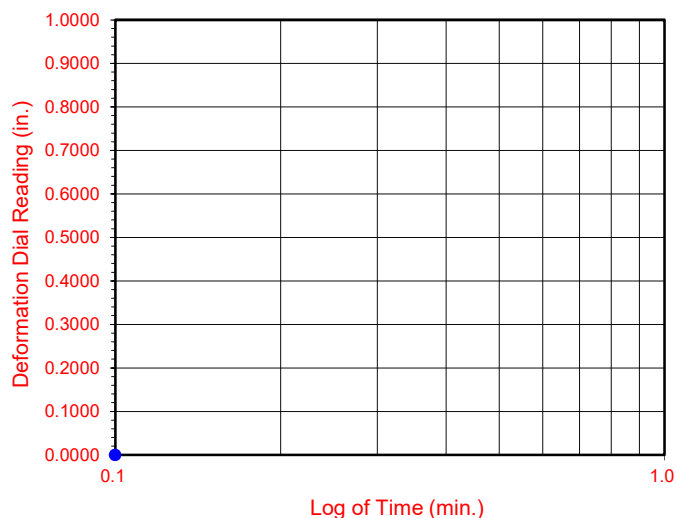
Sample Diameter (in.)	2.415
Sample Thickness (in.)	1.000
Wt. of Sample + Ring (g)	172.99
Weight of Ring (g)	45.51
Height after consol. (in.)	0.8984
Before Test	
Wt.Wet Sample+Cont. (g)	202.19
Wt.of Dry Sample+Cont. (g)	197.87
Weight of Container (g)	59.82
Initial Moisture Content (%)	3.1
Initial Dry Density (pcf)	102.8
Initial Saturation (%)	13
Initial Vertical Reading (in.)	0.3054
After Test	
Wt.of Wet Sample+Cont. (g)	252.42
Wt. of Dry Sample+Cont. (g)	232.82
Weight of Container (g)	64.70
Final Moisture Content (%)	15.99
Final Dry Density (pcf)	113.5
Final Saturation (%)	89
Final Vertical Reading (in.)	0.2021
Specific Gravity (assumed)	2.70
Water Density (pcf)	62.43



Pressure (p) (ksf)	Final Reading (in.)	Apparent Thickness (in.)	Load Compliance (%)	Deformation % of Sample Thickness	Void Ratio	Corrected Deforma- tion (%)
0.10	0.3052	0.9998	0.00	0.02	0.639	0.02
0.25	0.3031	0.9977	0.01	0.23	0.636	0.22
0.60	0.3007	0.9953	0.04	0.47	0.633	0.43
0.60	0.2954	0.9900	0.04	1.00	0.624	0.96
1.00	0.2880	0.9826	0.06	1.74	0.612	1.68
2.00	0.2680	0.9626	0.11	3.74	0.580	3.63
4.00	0.2439	0.9385	0.19	6.15	0.542	5.96
8.00	0.2194	0.9140	0.33	8.61	0.504	8.28
16.00	0.1903	0.8849	0.52	11.51	0.459	10.99
4.00	0.1938	0.8884	0.36	11.16	0.463	10.80
1.00	0.1982	0.8928	0.23	10.72	0.468	10.49
0.25	0.2021	0.8967	0.17	10.33	0.473	10.16

[illegible]

Time Readings



Boring No.	Sample No.	Depth (ft.)	Moisture Content (%)		Dry Density (pcf)		Void Ratio		Degree of Saturation (%)	
			Initial	Final	Initial	Final	Initial	Final	Initial	Final
LB-1	R-1	5.0	3.1	16.0	102.8	113.5	0.640	0.473	13	89

Soil Identification: Light olive brown silty clay with sand (CL-ML)s



Leighton

ONE-DIMENSIONAL CONSOLIDATION PROPERTIES of SOILS ASTM D 2435

Project No.: 12828.005

HCR Redlands (37-Lots)

03-21



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS ASTM D 4546

Project Name: HCR Redlands (37-Lots)
 Project No.: 12828.005
 Boring No.: LB-1
 Sample No.: R-2
 Sample Description: Light olive brown silty sand (SM)

Tested By: G. Bathala Date: 03/13/21
 Checked By: A. Santos Date: 03/24/21
 Sample Type: Ring
 Depth (ft.): 10.0

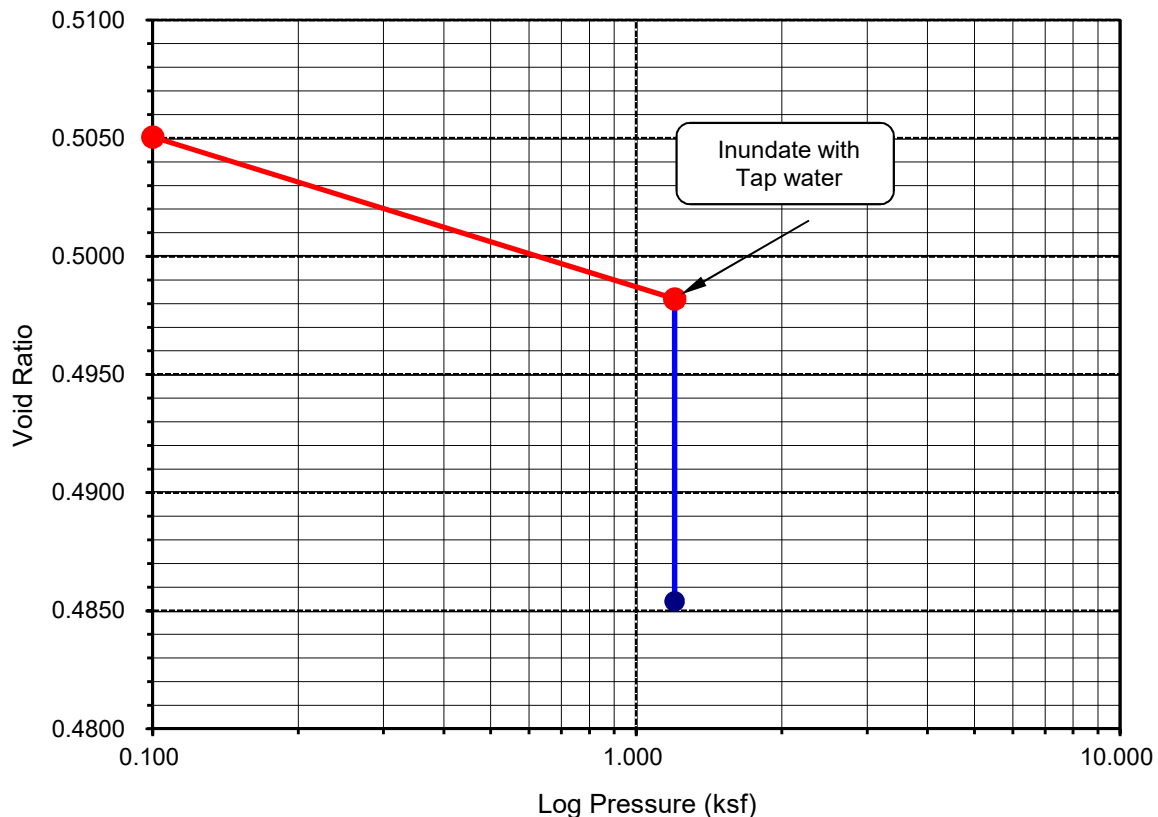
Initial Dry Density (pcf):	112.0
Initial Moisture (%):	2.13
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2805
Diameter(in):	2.415

Final Dry Density (pcf):	113.5
Final Moisture (%) :	13.7
Initial Void Ratio:	0.5054
Specific Gravity(assumed):	2.70
Initial Saturation (%)	11.4

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2803	0.9998	0.00	-0.03	0.5051	-0.03
1.200	0.2737	0.9932	0.20	-0.68	0.4982	-0.48
H2O	0.2652	0.9847	0.20	-1.53	0.4854	-1.33

Percent Swell (+) / Settlement (-) After Inundation = -0.85

Void Ratio - Log Pressure Curve





ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS ASTM D 4546

Project Name: HCR Redlands (37-Lots)
 Project No.: 12828.005
 Boring No.: LB-2
 Sample No.: R-2
 Sample Description: Light olive gray silty sand with gravel (SM)

Tested By: G. Bathala Date: 03/13/21
 Checked By: A. Santos Date: 03/24/21
 Sample Type: Ring
 Depth (ft.): 10.0

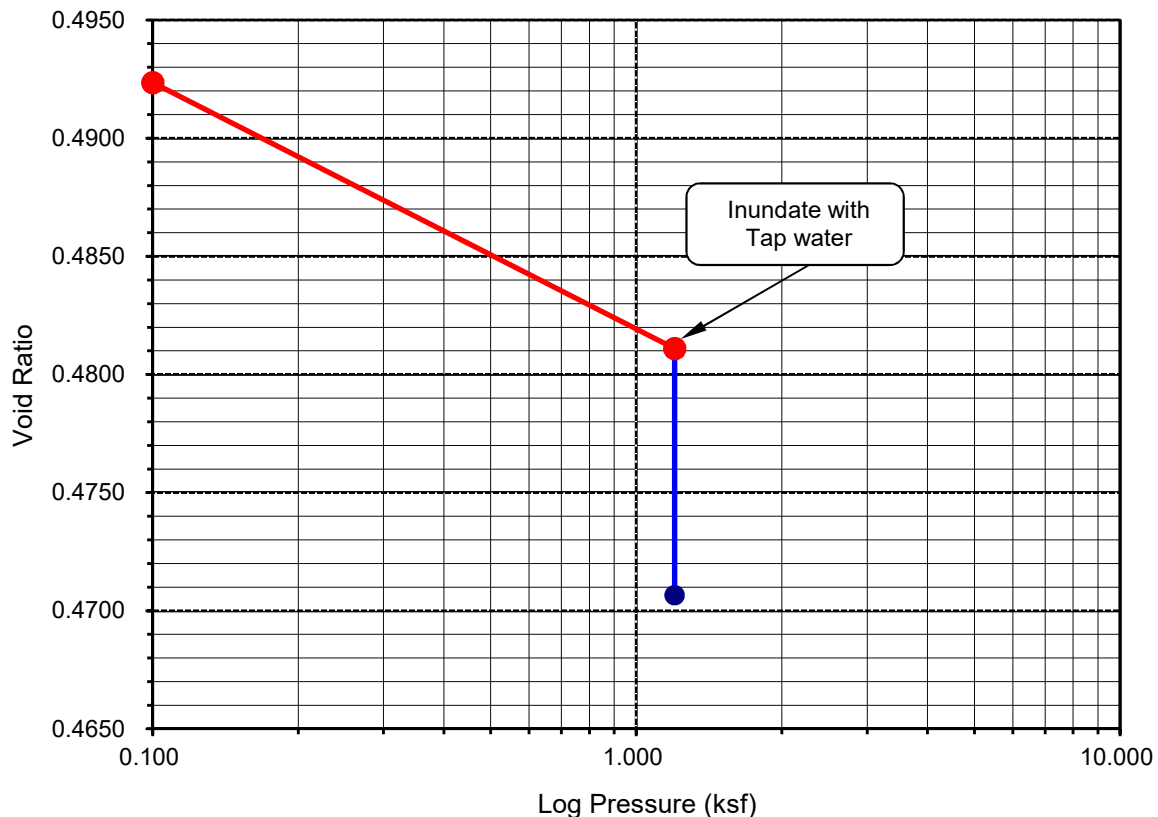
Initial Dry Density (pcf):	112.9
Initial Moisture (%):	2.25
Initial Length (in.):	1.0000
Initial Dial Reading:	0.2920
Diameter(in):	2.415

Final Dry Density (pcf):	114.6
Final Moisture (%) :	12.7
Initial Void Ratio:	0.4928
Specific Gravity(assumed):	2.70
Initial Saturation (%)	12.3

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.2917	0.9997	0.00	-0.03	0.4924	-0.03
1.200	0.2833	0.9913	0.09	-0.87	0.4811	-0.78
H2O	0.2763	0.9843	0.09	-1.57	0.4707	-1.48

Percent Swell (+) / Settlement (-) After Inundation = **-0.71**

Void Ratio - Log Pressure Curve





DIRECT SHEAR TEST

Consolidated Undrained

Project Name: [HCR Redlands \(37-Lots\)](#)

Project No.: [12828.005](#)

Boring No.: [LB-1](#)

Sample No.: [B-1](#)

Soil Identification: [Dark brown silty sand \(SM\)](#)

Tested By: [ACS/GB](#)

Checked By: [A. Santos](#)

Sample Type: [90% Remold](#)

Depth (ft.): [0-5](#)

Date: [03/18/21](#)

Date: [03/23/21](#)

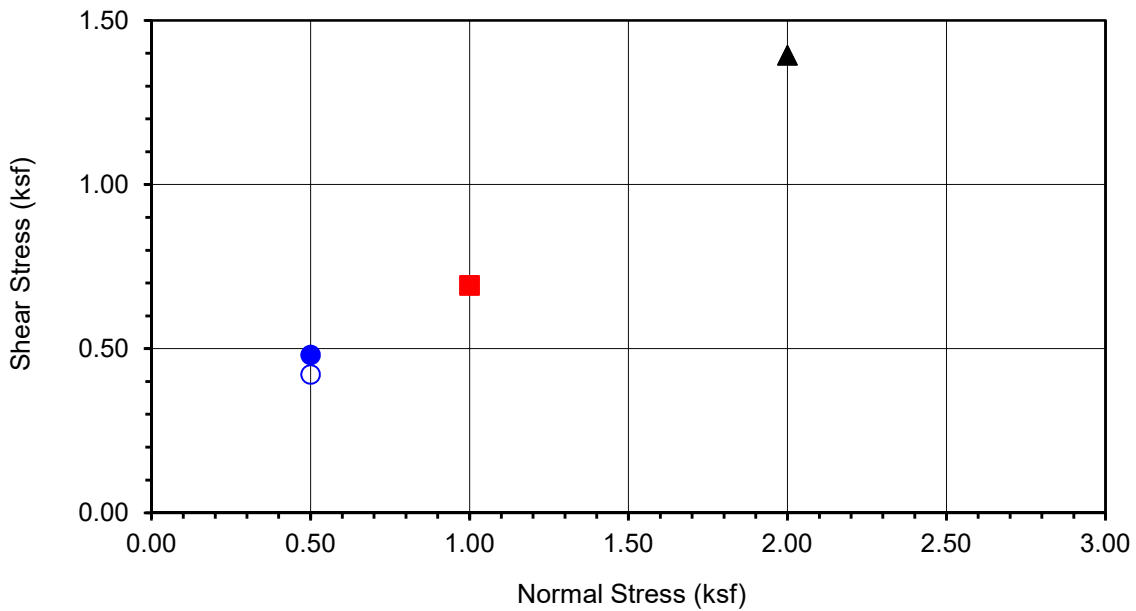
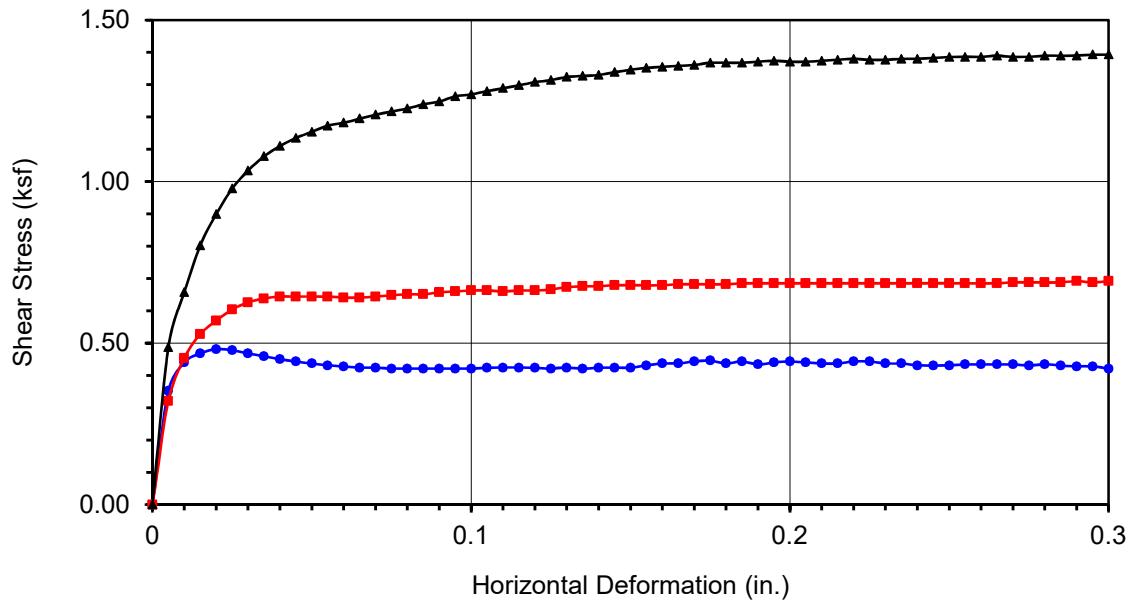
Sample Diameter(in):	2.415	2.415	2.415
Sample Thickness(in.):	1.000	1.000	1.000
Weight of Sample + ring(gm):	193.64	193.13	195.01
Weight of Ring(gm):	45.52	45.55	47.19

Before Shearing

Weight of Wet Sample+Cont.(gm):	213.25	213.25	213.25
Weight of Dry Sample+Cont.(gm):	203.49	203.49	203.49
Weight of Container(gm):	56.67	56.67	56.67
Vertical Rdg.(in): Initial	0.0000	0.2662	0.3428
Vertical Rdg.(in): Final	-0.0047	0.2748	0.3619

After Shearing

Weight of Wet Sample+Cont.(gm):	228.18	211.38	215.29
Weight of Dry Sample+Cont.(gm):	209.39	193.31	197.95
Weight of Container(gm):	72.42	57.17	60.91
Specific Gravity (Assumed):	2.70	2.70	2.70
Water Density(pcf):	62.43	62.43	62.43



Boring No.	LB-1
Sample No.	B-1
Depth (ft)	0-5
<u>Sample Type:</u>	
90% Remold	
<u>Soil Identification:</u>	
Dark brown silty sand (SM)	

Normal Stress (kip/ft ²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft ²)	● 0.481	■ 0.692	▲ 1.393
Shear Stress @ End of Test (ksf)	○ 0.421	□ 0.692	△ 1.393
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	6.65	6.65	6.65
Dry Density (pcf)	115.5	115.1	115.3
Saturation (%)	39.1	38.6	38.8
Soil Height Before Shearing (in.)	0.9953	0.9914	0.9809
Final Moisture Content (%)	13.7	13.3	12.7



Leighton

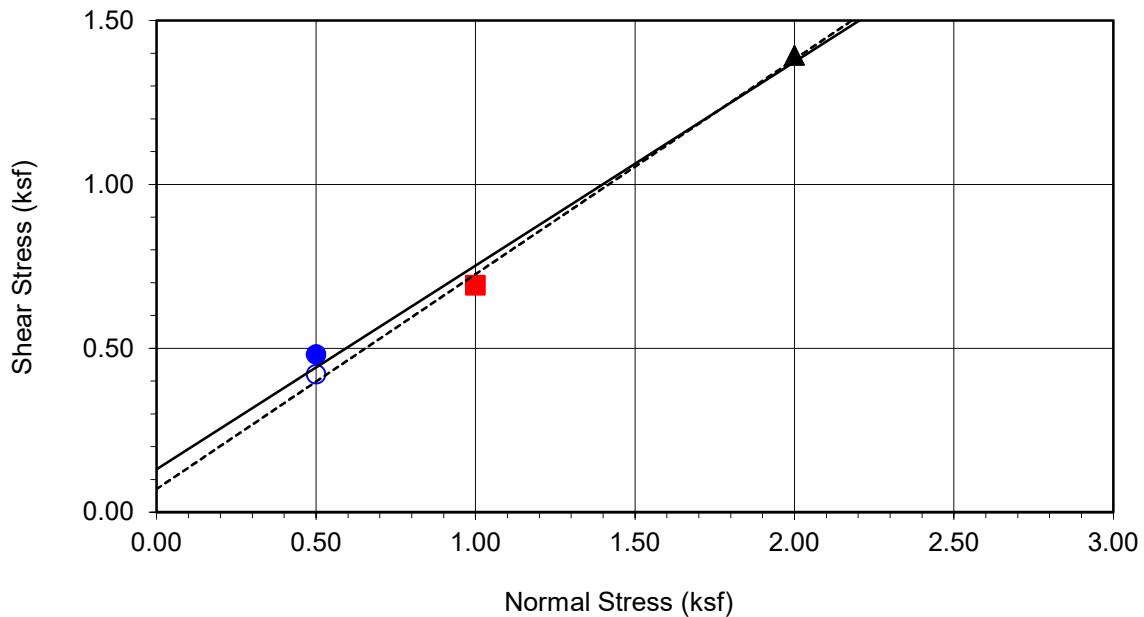
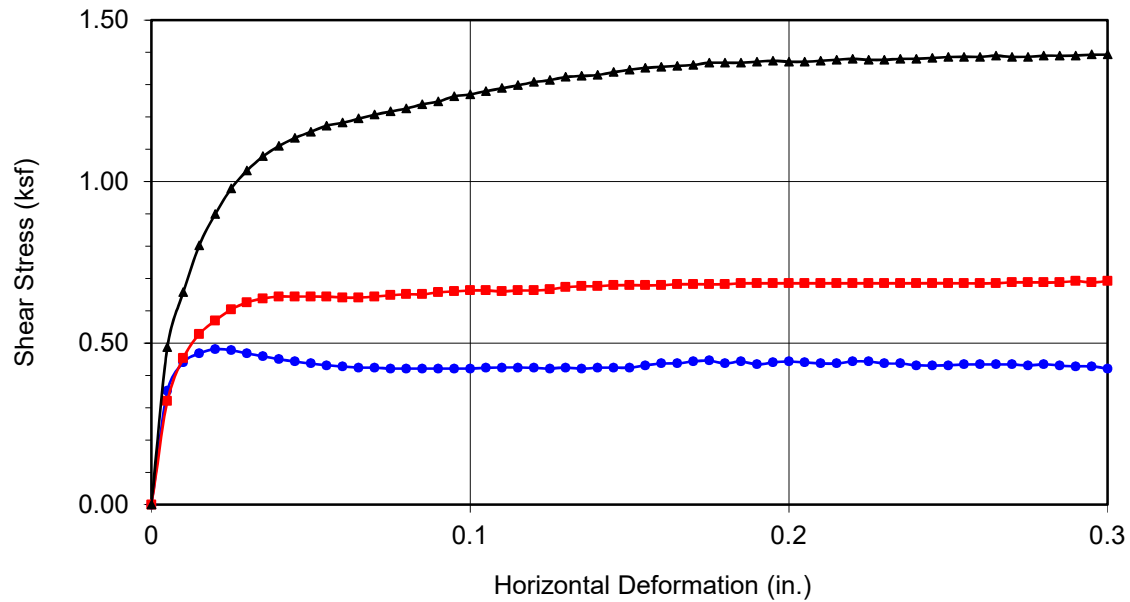
DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 12828.005

HCR Redlands (37-Lots)

03-21



Boring No.	LB-1	
Sample No.	B-1	
Depth (ft)	0-5	
<u>Sample Type:</u> 90% Remold		
<u>Soil Identification:</u> Dark brown silty sand (SM)		
<u>Strength Parameters</u>		
	C (psf)	ϕ (°)
Peak	131	32
Ultimate	71	33

Normal Stress (kip/ft ²)	0.500	1.000	2.000
Peak Shear Stress (kip/ft ²)	● 0.481	■ 0.692	▲ 1.393
Shear Stress @ End of Test (ksf)	○ 0.421	□ 0.692	△ 1.393
Deformation Rate (in./min.)	0.0500	0.0500	0.0500
Initial Sample Height (in.)	1.000	1.000	1.000
Diameter (in.)	2.415	2.415	2.415
Initial Moisture Content (%)	6.65	6.65	6.65
Dry Density (pcf)	115.5	115.1	115.3
Saturation (%)	39.1	38.6	38.8
Soil Height Before Shearing (in.)	0.9953	0.9914	0.9809
Final Moisture Content (%)	13.7	13.3	12.7



Leighton

DIRECT SHEAR TEST RESULTS

Consolidated Undrained

Project No.: 12828.005

HCR Redlands (37-Lots)

03-21



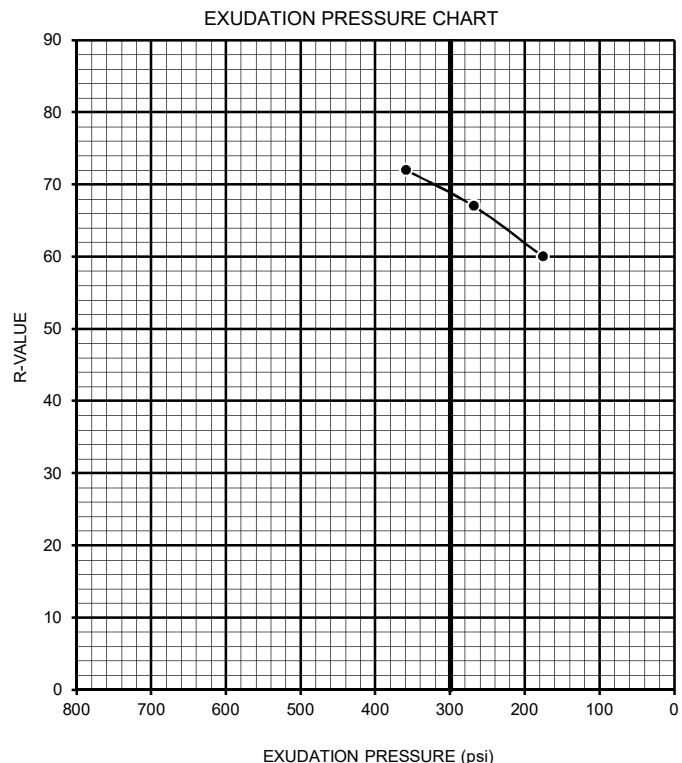
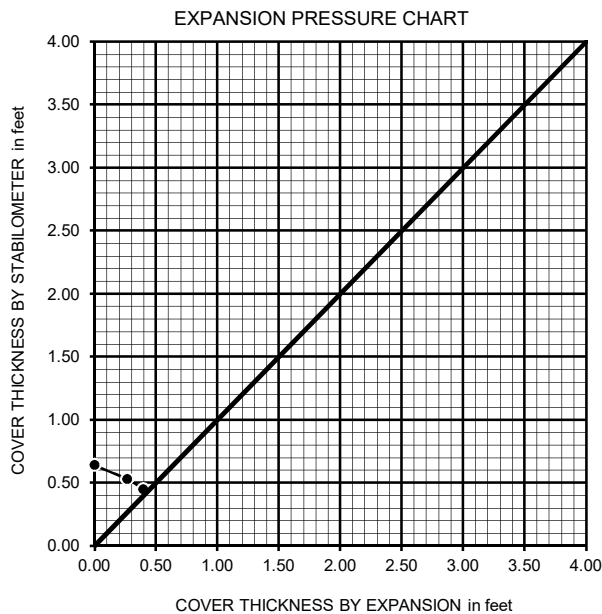
R-VALUE TEST RESULTS

DOT CA Test 301

PROJECT NAME: HCR Redlands (37-Lots) PROJECT NUMBER: 12828.005
 BORING NUMBER: LB-2 DEPTH (FT.): 0-5
 SAMPLE NUMBER: B-1 TECHNICIAN: O. Figueroa
 SAMPLE DESCRIPTION: Olive brown silty sand (SM) DATE COMPLETED: 3/16/2021

TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	9.3	9.8	10.6
HEIGHT OF SAMPLE, Inches	2.43	2.43	2.49
DRY DENSITY, pcf	130.5	127.1	127.3
COMPACTOR PRESSURE, psi	275	200	125
EXUDATION PRESSURE, psi	359	268	175
EXPANSION, Inches x 10exp-4	12	8	0
STABILITY Ph 2,000 lbs (160 psi)	25	30	40
TURNS DISPLACEMENT	4.65	4.80	5.00
R-VALUE UNCORRECTED	74	69	60
R-VALUE CORRECTED	72	67	60

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.45	0.53	0.64
EXPANSION PRESSURE THICKNESS, ft.	0.40	0.27	0.00



R-VALUE BY EXPANSION: 72
 R-VALUE BY EXUDATION: 69
 EQUILIBRIUM R-VALUE: 69

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name: <u>HCR Redlands (37-Lots)</u>	Tested By: <u>J. Gonzalez</u>	Date: <u>03/12/21</u>
Project No.: <u>12828.005</u>	Checked By: <u>A. Santos</u>	Date: <u>03/15/21</u>
Boring No.: <u>LB-1</u>	Depth (ft.): <u>0-5</u>	
Sample No.: <u>B-1</u>		
Soil Identification: <u>Dark brown silty sand (SM)</u>		

Preparation Method:

☒ Moist
☐ Dry

☒ Mechanical Ram
☐ Manual Ram

Mold Volume (ft³)

0.03330

Ram Weight = 10 lb.; Drop = 18 in.

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3794	3926	3938			
Weight of Mold (g)	1868	1868	1868			
Net Weight of Soil (g)	1926	2058	2070			
Wet Weight of Soil + Cont. (g)	348.6	268.0	333.4			
Dry Weight of Soil + Cont. (g)	338.0	254.1	309.2			
Weight of Container (g)	40.1	39.5	39.3			
Moisture Content (%)	3.56	6.48	8.97			
Wet Density (pcf)	127.5	136.2	137.0			
Dry Density (pcf)	123.1	128.0	125.8			

Maximum Dry Density (pcf)

128.0

Optimum Moisture Content (%)

6.8

PROCEDURE USED

☒ Procedure A

Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if + #4 is 20% or less

☐ Procedure B

Soil Passing 3/8 in. (9.5 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
Use if + #4 is >20% and +3/8 in. is 20% or less

☐ Procedure C

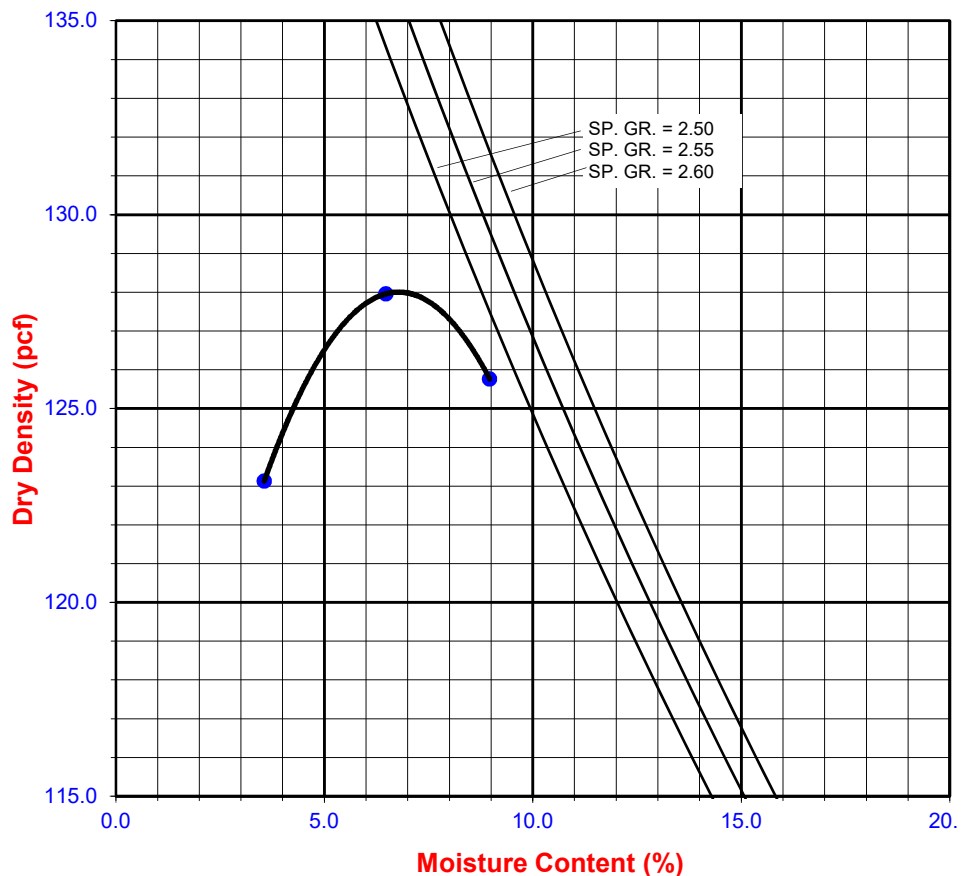
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +3/4 in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI





TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: HCR Redlands (37-Lots) Tested By : G. Bathala Date: 02/26/21
Project No. : 12828.005 Checked By: A. Santos Date: 03/24/21

Boring No.	LB-2			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Olive brown (SM)			
Wet Weight of Soil + Container (g)	127.32			
Dry Weight of Soil + Container (g)	127.22			
Weight of Container (g)	64.37			
Moisture Content (%)	0.16			
Weight of Soaked Soil (g)	100.24			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	94			
Crucible No.	16			
Furnace Temperature (°C)	860			
Time In / Time Out	13:15/14:00			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	18.4715			
Wt. of Crucible (g)	18.4695			
Wt. of Residue (g) (A)	0.0020			
PPM of Sulfate (A) x 41150	82.30			
PPM of Sulfate, Dry Weight Basis	82			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO ₃ Soln. Used in Titration (C)	1.7			
PPM of Chloride (C -0.2) * 100 * 30 / B	150			
PPM of Chloride, Dry Wt. Basis	150			

pH TEST, DOT California Test 643

pH Value	8.26			
Temperature °C	20.6			



SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name: HCR Redlands (37-Lots)
 Project No. : 12828.005
 Boring No.: LB-2
 Sample No. : B-1

Tested By : Y. Nguyen Date: 03/17/21
 Checked By: A. Santos Date: 03/24/21
 Depth (ft.) : 0-5

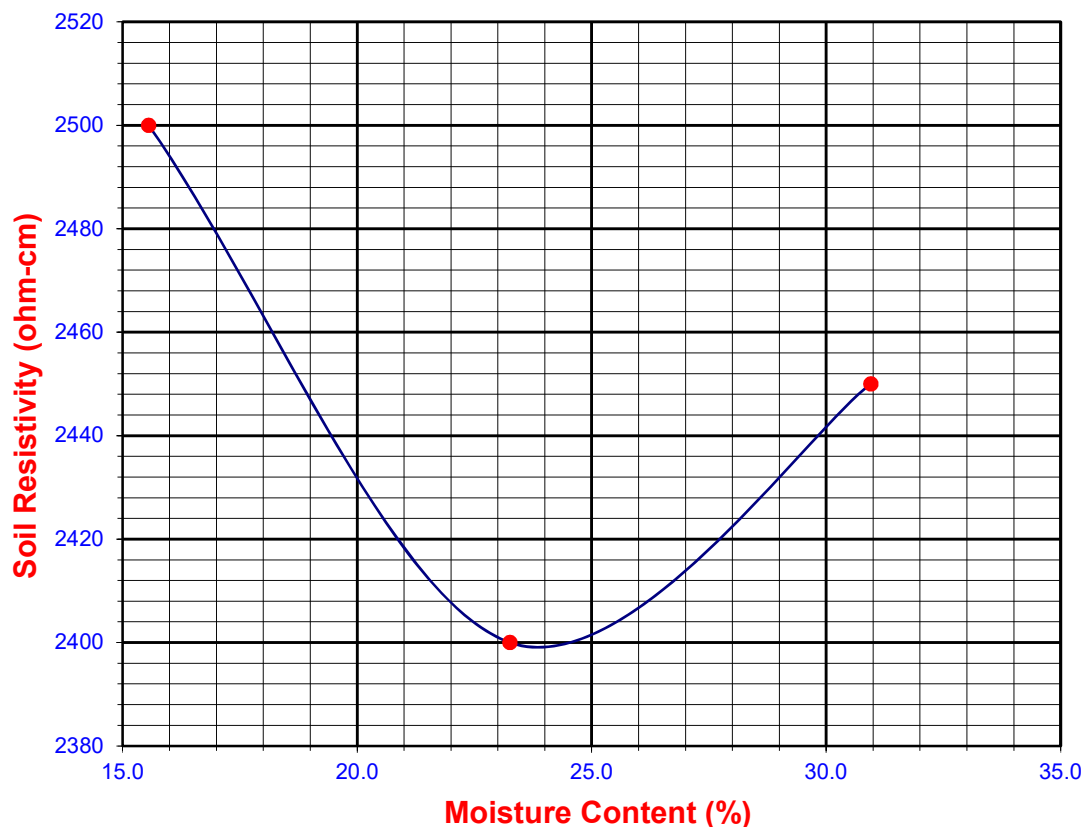
Soil Identification:* Olive brown (SM)

*California Test 643 requires soil specimens to consist only of portions of samples passing through the No. 8 US Standard Sieve before resistivity testing. Therefore, this test method may not be representative for coarser materials.

Specimen No.	Water Added (ml) (Wa)	Adjusted Moisture Content (MC)	Resistance Reading (ohm)	Soil Resistivity (ohm-cm)
1	20	15.56	2500	2500
2	30	23.25	2400	2400
3	40	30.95	2450	2450
4				
5				

Moisture Content (%) (MCi)	0.16
Wet Wt. of Soil + Cont. (g)	127.32
Dry Wt. of Soil + Cont. (g)	127.22
Wt. of Container (g)	64.37
Container No.	
Initial Soil Wt. (g) (Wt)	130.10
Box Constant	1.000
$MC = (((1 + MC_i / 100) \times (W_a / W_t + 1)) - 1) \times 100$	

Min. Resistivity (ohm-cm)	Moisture Content (%)	Sulfate Content (ppm)	Chloride Content (ppm)	Soil pH	
				pH	Temp. (°C)
DOT CA Test 643		DOT CA Test 417 Part II	DOT CA Test 422	DOT CA Test 643	
2399	24.0	82	150	8.26	20.6



APPENDIX D

SEISMIC ANALYSIS



Leighton

Liquefaction Susceptibility Analysis: SPT Method

Based on Youd and Idriss (2001), Martin and Lew (1999).

Project: Canyon Ranch Loma Linda - 89 Lots
Project No.: 12828.005

Leighton

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Ground Surface Elev (ft)
LB-1	100	100	0	-100
LB-2	100	100	0	-100
LB-3	100	100	0	-100
LB-4	100	100	0	-100
				0
				0
				0
				0

General Parameters:	
$a_{max} = 0.98g$	MCE
$M_w = 8.1$	(Idriss, 2001)
MSF eq: 1	
MSF = 0.82	%
Hammer Efficiency = 83	
$C_E = 1.38$	
$C_B = 1$	
$C_{S(SPT)} = 1.2$	
$C_{S(ring)} = 1$	
Rod Stickup (feet) = 3	
Ring sample correction = 0.65	

Summary of Liquefaction Susceptibility Analysis: SPT Method

Liquefaction Method: Youd and Idriss (2001). Seismic Settlement Method: Tokimatsu and Seed (1987) and Martin and Lew (1999).

Project: Canyon Ranch Loma Linda - 89 Lots

Project No.: 12828

Leighton

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thickness (ft)	Plasticity ("n"=non susc. to liq.)	Estimated Fines Cont (%)	γ_t (pcf)	N_m or B (blows/ft)	Sampler Type (enter 2 if mod CA Ring)	C_s	N_m (corrected for C_s and ring->SPT) (blows/ft)	Exist σ_{vo}' (psf)	$(N_1)_{60}$	$(N_1)_{60CS}$	$CRR_{7.5}$	Design σ_{vo}' (psf)	$CSR_{7.5}$	CSR_M	Liquefaction Factor of Safety	$(N_1)_{60CS}$ (for Settlement) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87)	Sat Sand Strain (%) (Tok/ Seed 87)	Seismic Sett. of Layer (in.)	Cummulative Seismic Settlement (in.)
LB-1	0 to 8	5	8		40	120	26	2	1	16.9	600	29.8	40.8	>Range	600	0.63	0.76	NonLiq	40.8	0.17		0.15	0.8
LB-1	8 to 13	10	5		30	120	26	2	1	16.9	1200	26.2	35.0	>Range	1200	0.62	0.75	NonLiq	35.0	0.50		0.30	0.7
LB-1	13 to 18	15	5		8	120	38	2	1	24.7	1800	31.3	32.0	>Range	1800	0.61	0.75	NonLiq	32.0	0.43		0.26	0.4
LB-1	18 to 23	20	5		5	120	67	2	1	43.6	2400	53.4	53.4	>Range	2400	0.60	0.74	NonLiq	53.4	0.13		0.08	0.1
LB-1	23 to 27	25	5		5	120	100	1	1.2	120.0	3000	131.6	131.6	>Range	3000	0.60	0.73	NonLiq	131.6	0.05		0.03	0.0
LB-2	0 to 5	4	5		30	120	30	1	1.2	36.0	480	63.5	78.0	>Range	480	0.63	0.77	NonLiq	78.0	0.05		0.03	1.3
LB-2	5 to 8	5	3		33	120	12	2	1	7.8	600	13.8	21.1	0.230	600	0.63	0.76	NonLiq	21.1	1.13		0.41	1.2
LB-2	8 to 13	10	5		12	120	27	2	1	17.6	1200	27.2	29.6	0.444	1200	0.62	0.75	NonLiq	29.6	1.04		0.62	0.8
LB-2	13 to 18	15	5		30	120	53	2	1	34.5	1800	43.6	55.1	>Range	1800	0.61	0.75	NonLiq	55.1	0.07		0.04	0.2
LB-2	18 to 23	20	5		20	120	27	1	1.2	32.4	2400	39.7	46.5	>Range	2400	0.60	0.74	NonLiq	46.5	0.15		0.09	0.2
LB-2	23 to 27	25	5		5	120	44	1	1.2	52.8	3000	57.9	57.9	>Range	3000	0.60	0.73	NonLiq	57.9	0.16		0.08	0.1
LB-3	0 to 5	4	5		5	120	30	1	1.2	36.0	480	63.5	63.5	>Range	480	0.63	0.77	NonLiq	63.5	0.07		0.04	6.0
LB-3	5 to 8	5	3		5	120	23	2	1	15.0	600	26.4	26.4	0.322	600	0.63	0.76	NonLiq	26.4	1.11		0.40	6.0
LB-3	8 to 13	10	5		10	120	16	2	1	10.4	1200	16.1	17.3	0.185	1200	0.62	0.75	NonLiq	17.3	1.83		1.10	5.6
LB-3	13 to 18	15	5		5	120	38	2	1	24.7	1800	31.3	31.3	>Range	1800	0.61	0.75	NonLiq	31.3	0.44		0.26	4.5
LB-3	18 to 23	20	5		47	120	11	1	1.2	13.2	2400	16.2	24.4	0.281	2400	0.60	0.74	NonLiq	24.4	1.10		0.66	4.2
LB-3	23 to 28	25	5		79	120	8	1	1.2	9.6	3000	10.5	17.6	0.188	3000	0.60	0.73	NonLiq	17.6	1.83		1.10	3.6
LB-3	28 to 33	30	5		6	120	22	1	1.2	26.4	3600	27.8	28.0	0.369	3600	0.59	0.72	NonLiq	28.0	0.99		0.59	2.5
LB-3	33 to 38	35	5		5	120	61	1	1.2	73.2	4200	71.4	71.4	>Range	4200	0.56	0.69	NonLiq	71.4	0.07		0.04	1.9
LB-3	38 to 43	40	5		36	120	9	1	1.2	10.8	4800	9.9	16.8	0.179	4800	0.54	0.66	NonLiq	16.8	1.83		1.10	1.8
LB-3	43 to 48	45	5		32	120	18	1	1.2	21.6	5400	18.6	26.6	0.327	5400	0.51	0.62	NonLiq	26.6	1.07		0.64	0.7
LB-3	48 to 52	50	5		5	120	41	1	1.2	49.2	6000	40.1	40.1	>Range	6000	0.49	0.59	NonLiq	40.1	0.16		0.09	0.1
LB-4	0 to 5	4	5		30	120	30	1	1.2	36.0	480	63.5	78.0	>Range	480	0.63	0.77	NonLiq	78.0	0.05		0.03	2.0
LB-4	5 to 8	5	3		30	120	15	2	1	9.8	600	17.2	24.6	0.283	600	0.63	0.76	NonLiq	24.6	1.11		0.40	2.0
LB-4	8 to 13	10	5		5	120	35	2	1	22.8	1200	35.3	35.3	>Range	1200	0.62	0.75	NonLiq	35.3	0.50		0.30	1.6
LB-4	13 to 18	15	5		5	120	59	2	1	38.4	1800	48.6	48.6	>Range	1800	0.61	0.75	NonLiq	48.6	0.08		0.05	1.3
LB-4	18 to 23	20	5		20	120	28	2	1	18.2	2400	22.3	27.7	0.359	2400	0.60	0.74	NonLiq	27.7	1.07		0.64	1.3
LB-4	23 to 27	25	5		87	120	12	1	1.2	14.4	3000	15.8	23.9	0.272	3000	0.60	0.73	NonLiq	23.9	1.17		0.63	0.6



Latitude, Longitude: 34.04367, -117.2159



Date	4/22/2021, 11:59:44 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	2.103	MCE_R ground motion. (for 0.2 second period)
S_1	0.836	MCE_R ground motion. (for 1.0s period)
S_{MS}	2.103	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	1.402	Numeric seismic design value at 0.2 second SA
S_{D1}	null -See Section 11.4.8	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	null -See Section 11.4.8	Seismic design category
F_a	1	Site amplification factor at 0.2 second
F_v	null -See Section 11.4.8	Site amplification factor at 1.0 second
PGA	0.887	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.976	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
S_{sRT}	2.433	Probabilistic risk-targeted ground motion. (0.2 second)
S_{sUH}	2.657	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
S_{sD}	2.103	Factored deterministic acceleration value. (0.2 second)
S_{1RT}	0.965	Probabilistic risk-targeted ground motion. (1.0 second)
S_{1UH}	1.083	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S_{1D}	0.836	Factored deterministic acceleration value. (1.0 second)
PGA_d	0.887	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.916	Mapped value of the risk coefficient at short periods
C_{R1}	0.891	Mapped value of the risk coefficient at a period of 1 s

DISCLAIMER

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Unified Hazard Tool

- Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the [U.S. Seismic Design Maps web tools](#) (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

^ Input

Edition

Dynamic: Conterminous U.S. 2014 (update... ▼

Spectral Period

Peak Ground Acceleration ▼

Latitude

Decimal degrees

34.04367

Time Horizon

Return period in years

2475

Longitude

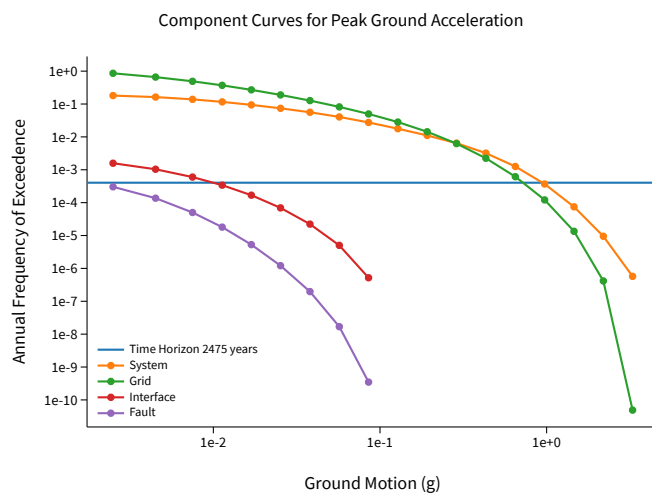
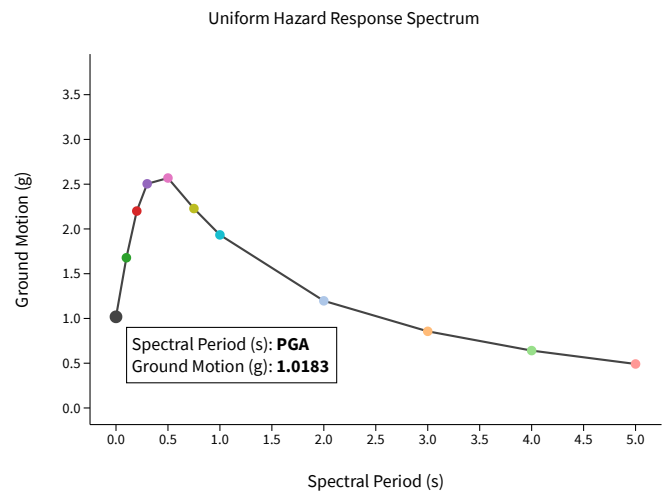
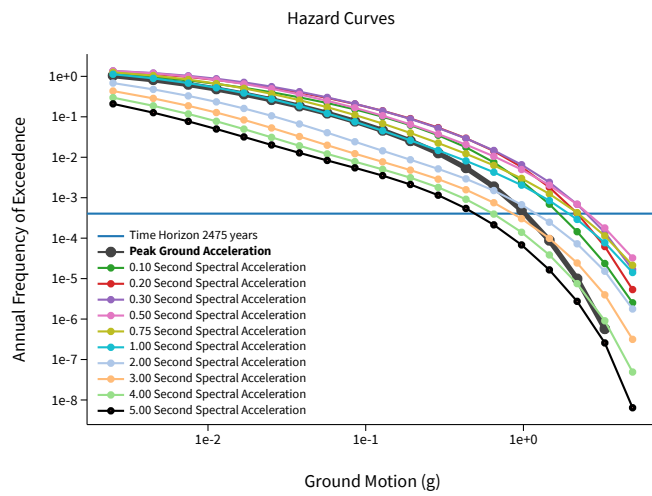
Decimal degrees, negative values for western longitudes

-117.2159

Site Class

259 m/s (Site class D) ▼

^ Hazard Curve

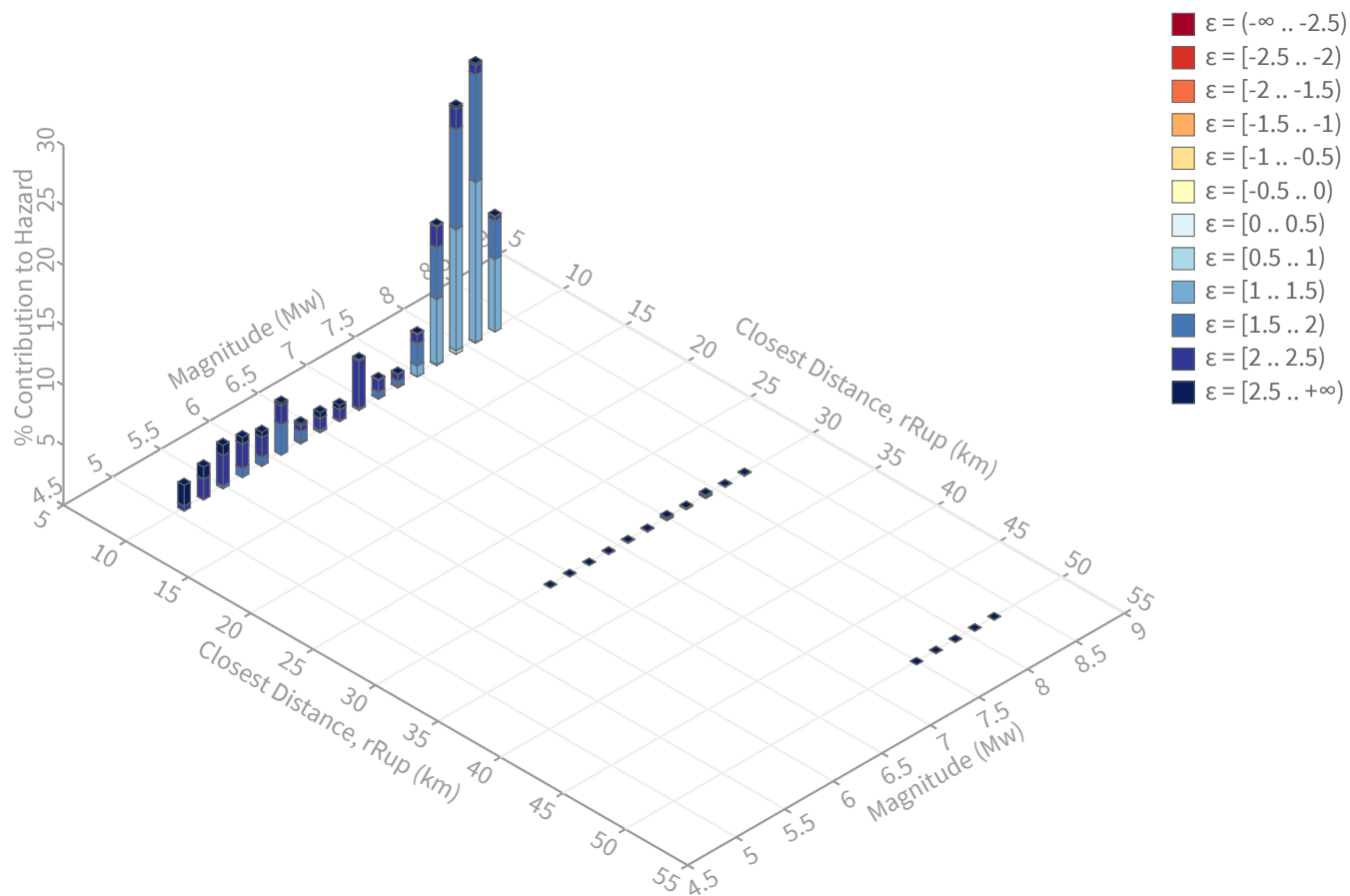


[View Raw Data](#)

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 1.0183353 g

Recovered targets

Return period: 3322.4006 yrs
Exceedance rate: 0.00030098718 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.04 %

Mean (over all sources)

m: 7.39
r: 7.23 km
ε₀: 1.7 σ

Mode (largest m-r bin)

m: 8.1
r: 5.69 km
ε₀: 1.39 σ
Contribution: 23.27 %

Mode (largest m-r-ε₀ bin)

m: 8.1
r: 3.93 km
ε₀: 1.15 σ
Contribution: 13.35 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

- ε0:** [-∞ .. -2.5)
- ε1:** [-2.5 .. -2.0)
- ε2:** [-2.0 .. -1.5)
- ε3:** [-1.5 .. -1.0)
- ε4:** [-1.0 .. -0.5)
- ε5:** [-0.5 .. 0.0)
- ε6:** [0.0 .. 0.5)
- ε7:** [0.5 .. 1.0)
- ε8:** [1.0 .. 1.5)
- ε9:** [1.5 .. 2.0)
- ε10:** [2.0 .. 2.5)
- ε11:** [2.5 .. +∞]

Deaggregation Contributors

Source Set 	Source	Type	r	m	ϵ_0	lon	lat	az	%
UC33brAvg_FM32		System							39.49
	San Jacinto (San Bernardino) [4]		3.44	8.01	1.27	117.244°W	34.023°N	228.37	20.85
	San Andreas (San Bernardino S) [0]		10.64	7.64	1.91	117.169°W	34.130°N	23.96	11.85
	San Andreas (North Branch Mill Creek) [1]		11.04	7.97	1.61	117.183°W	34.140°N	15.78	2.35
	San Jacinto (San Jacinto Valley) rev [0]		3.58	7.59	1.37	117.237°W	34.017°N	213.26	1.11
UC33brAvg_FM31		System							39.49
	San Jacinto (San Bernardino) [4]		3.44	8.02	1.27	117.244°W	34.023°N	228.37	20.92
	San Andreas (San Bernardino S) [0]		10.64	7.64	1.91	117.169°W	34.130°N	23.96	11.78
	San Andreas (North Branch Mill Creek) [1]		11.04	7.96	1.61	117.183°W	34.140°N	15.78	2.28
	San Jacinto (San Jacinto Valley) rev [0]		3.58	7.58	1.37	117.237°W	34.017°N	213.26	1.09
UC33brAvg_FM31 (opt)		Grid							10.51
	PointSourceFinite: -117.216, 34.102		8.09	5.69	2.21	117.216°W	34.102°N	0.00	2.10
	PointSourceFinite: -117.216, 34.102		8.09	5.69	2.21	117.216°W	34.102°N	0.00	2.10
	PointSourceFinite: -117.216, 34.093		7.50	5.62	2.16	117.216°W	34.093°N	0.00	1.83
	PointSourceFinite: -117.216, 34.093		7.50	5.62	2.16	117.216°W	34.093°N	0.00	1.83
	PointSourceFinite: -117.216, 34.129		9.96	5.87	2.38	117.216°W	34.129°N	0.00	1.02
	PointSourceFinite: -117.216, 34.129		9.96	5.87	2.38	117.216°W	34.129°N	0.00	1.02
UC33brAvg_FM32 (opt)		Grid							10.51
	PointSourceFinite: -117.216, 34.102		8.09	5.69	2.21	117.216°W	34.102°N	0.00	2.10
	PointSourceFinite: -117.216, 34.102		8.09	5.69	2.21	117.216°W	34.102°N	0.00	2.10
	PointSourceFinite: -117.216, 34.093		7.50	5.62	2.16	117.216°W	34.093°N	0.00	1.83
	PointSourceFinite: -117.216, 34.093		7.50	5.62	2.16	117.216°W	34.093°N	0.00	1.83
	PointSourceFinite: -117.216, 34.129		9.96	5.87	2.38	117.216°W	34.129°N	0.00	1.02
	PointSourceFinite: -117.216, 34.129		9.96	5.87	2.38	117.216°W	34.129°N	0.00	1.02

APPENDIX E

GBA IMPORTANT INFORMATION ABOUT THIS
GEOTECHNICAL ENGINEERING REPORT



Leighton

Important Information about This Geotechnical-Engineering Report

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

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation

everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed

and Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer

will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept*

responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.

Most of the “Findings” Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site’s subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual site-wide subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report’s Recommendations Are

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals’ misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals’ plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note*

conspicuously that you’ve included the material for information purposes only. To avoid misunderstanding, you may also want to note that “informational purposes” means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled “limitations,” many of these provisions indicate where geotechnical engineers’ responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a “phase-one” or “phase-two” environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer’s services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, *proper implementation of the geotechnical engineer’s recommendations will not of itself be sufficient to prevent moisture infiltration.* Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. *Geotechnical engineers are not building-envelope or mold specialists.*



**GEOPROFESSIONAL
BUSINESS
ASSOCIATION**

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e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX F

GENERAL EARTHWORK GRADING GUIDELINES



Leighton

APPENDIX F
LEIGHTON AND ASSOCIATES, INC.
GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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LEIGHTON AND ASSOCIATES, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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D - Buttress or Replacement Fill Subdrains	Rear of Text
E - Transition Lot Fills and Side Hill Fills	Rear of Text

1.0 GENERAL

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ the Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultants shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required. Subsurface areas to be geotechnically observed, mapped, elevations recorded, and/or tested include natural ground after it has been cleared for receiving fill but before fill is placed, bottoms of all "remedial removal" areas, all key bottoms, and benches made on sloping ground to receive fill.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to determine the attained level of compaction.

The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the plans and specifications.

The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "spreads" of work and the estimated quantities of daily earthwork contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate observations and tests can be planned and accomplished. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified.

2.0 PREPARATION OF AREAS TO BE FILLED

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). No fill lift shall contain more than 5 percent of organic matter. Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be overexcavated as specified in the following section. Scarification shall continue until soils are broken down and free of large clay lumps or clods and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Overexcavation

In addition to removals and overexcavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be overexcavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical

Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise overexcavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 FILL MATERIAL

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of Section 3.1. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 FILL PLACEMENT AND COMPACTION

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field-tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify

adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than 5 feet apart from potential test locations shall be provided.

5.0 SUBDRAIN INSTALLATION

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 EXCAVATION

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of

the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 TRENCH BACKFILLS

7.1 Safety

The Contractor shall follow all OSHA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill

All bedding and backfill of utility trenches shall be performed in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 ($SE > 30$). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of relative compaction from 1 foot above the top of the conduit to the surface.

The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

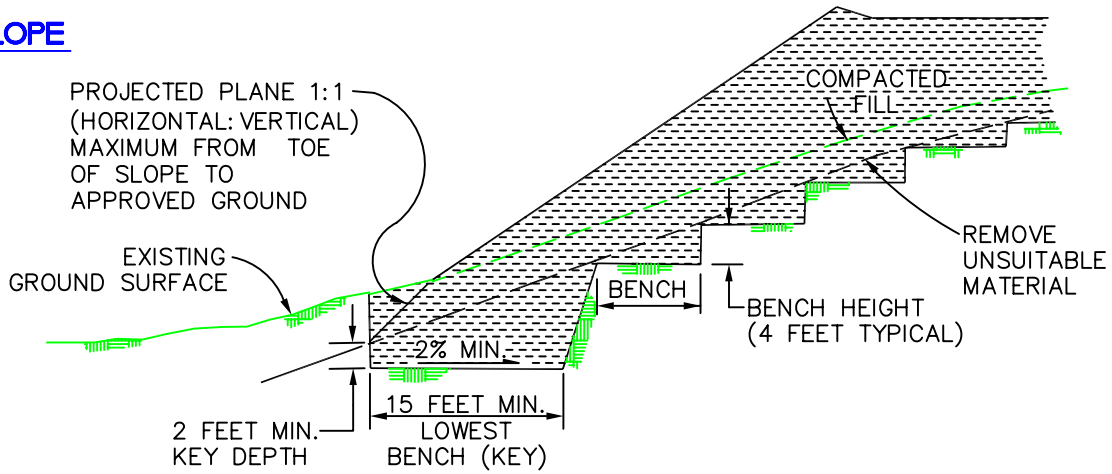
7.3 Lift Thickness

Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

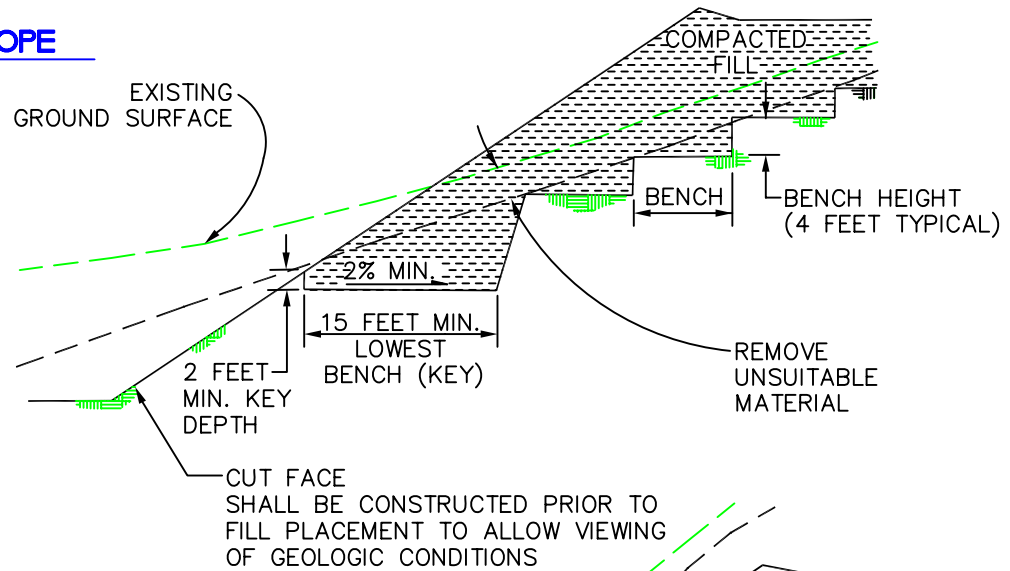
7.4 Observation and Testing

The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

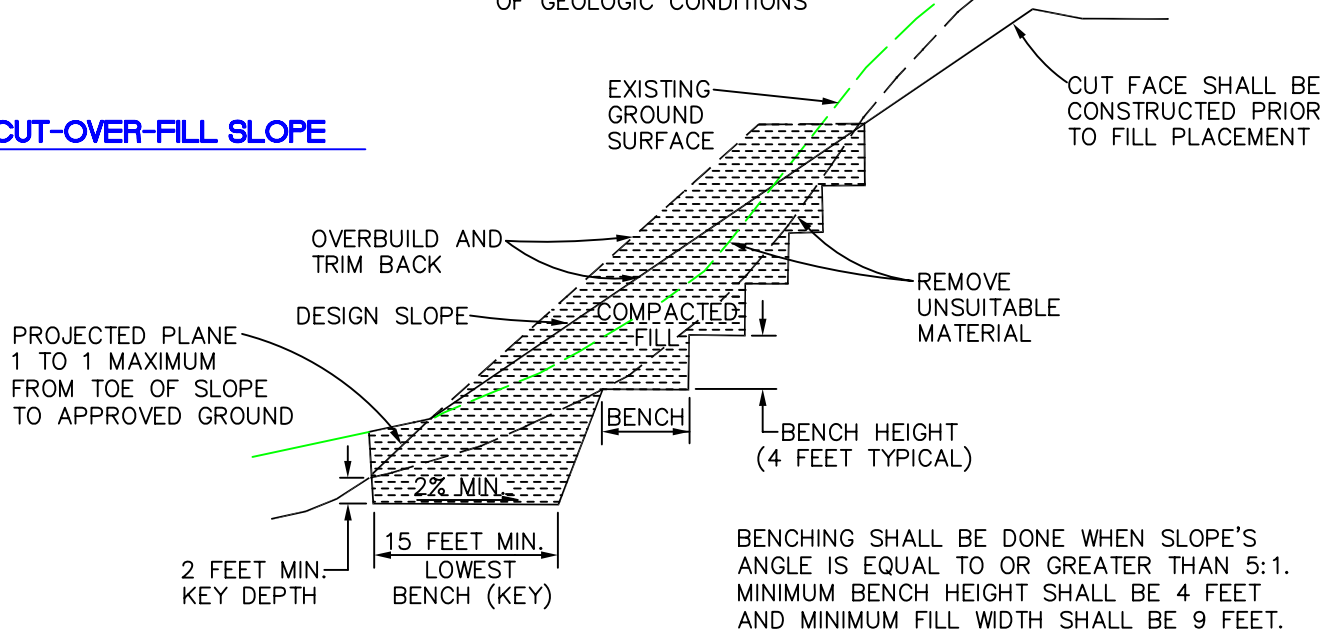
FILL SLOPE

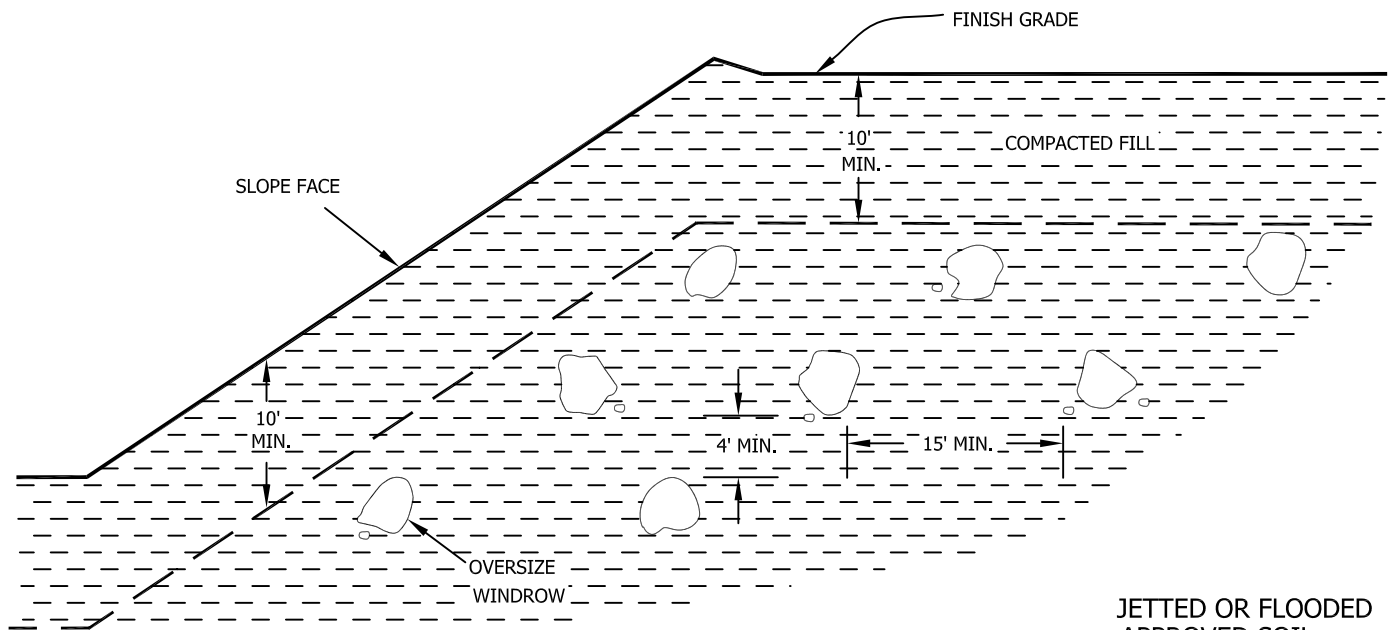


FILL-OVER-CUT SLOPE

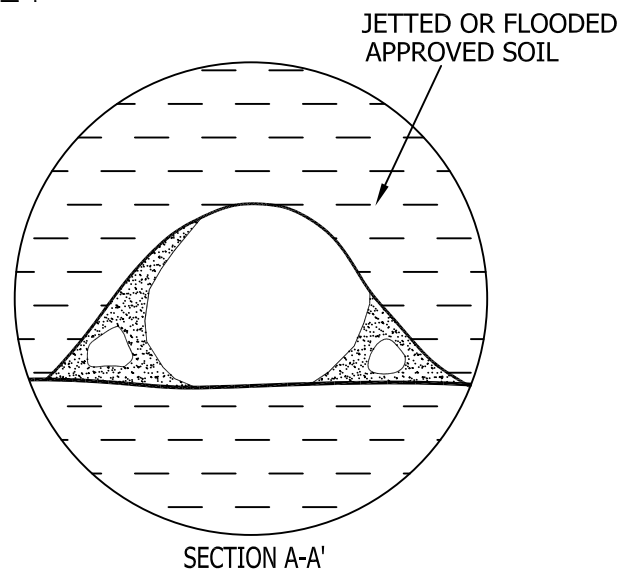


CUT-OVER-FILL SLOPE

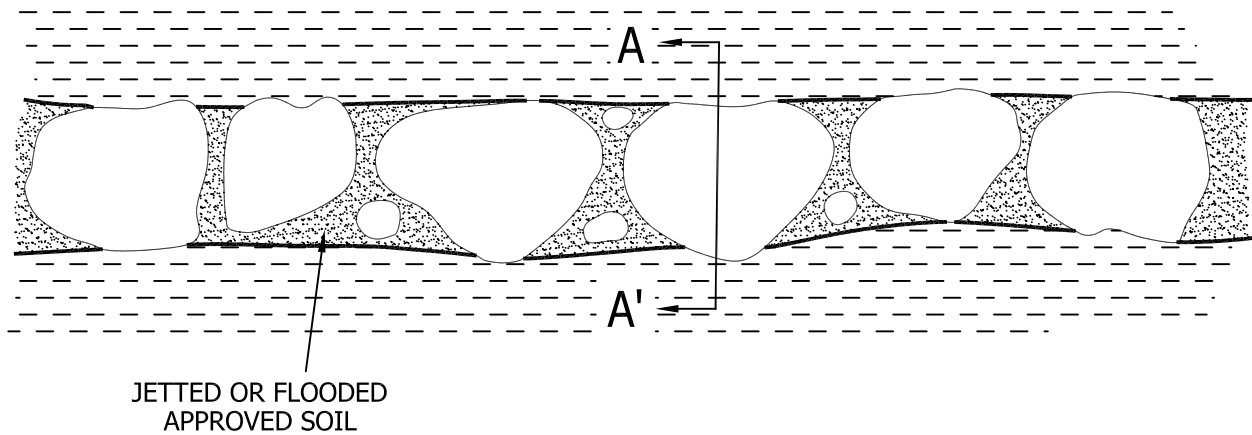




- Oversize rock is larger than 8 inches in largest dimension.
- Backfill with approved soil jetted or flooded in place to fill all the voids.
- Do not bury rock within 10 feet of finish grade.
- Windrow of buried rock shall be parallel to the finished slope face.



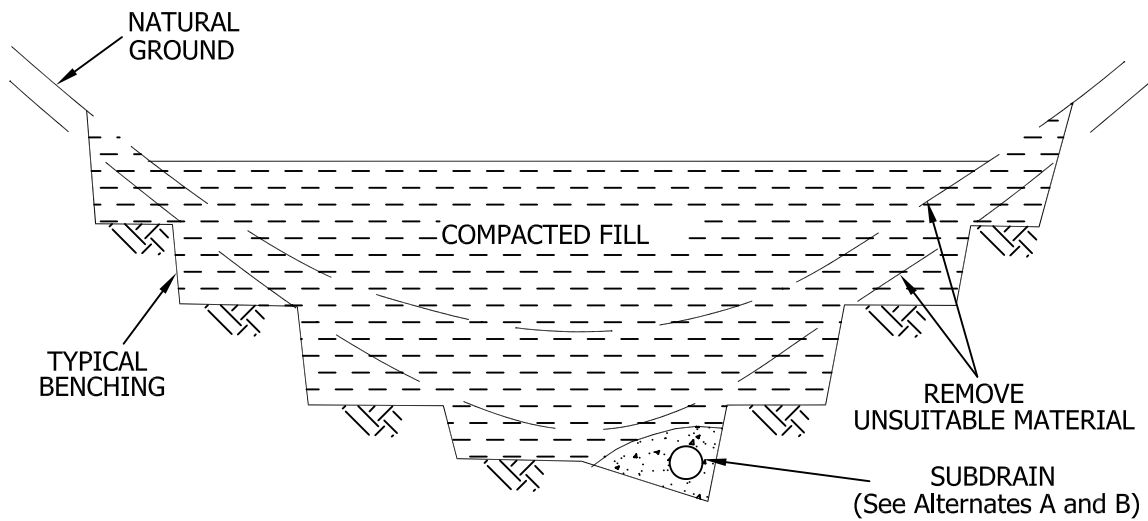
PROFILE ALONG WINDROW



OVERSIZE ROCK DISPOSAL

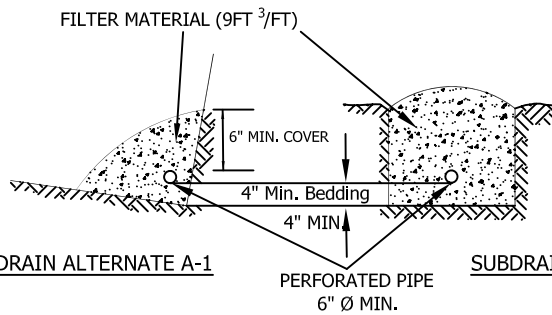
GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS B





SUBDRAIN ALTERNATE A

PERFORATED PIPE SURROUNDED
WITH FILTER MATERIAL

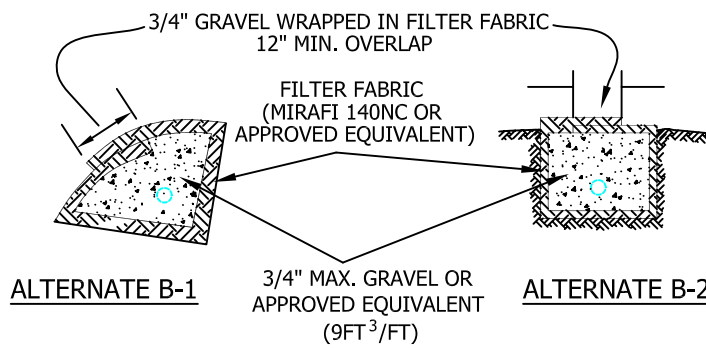


FILTER MATERIAL
FILTER MATERIAL SHALL BE CLASS 2 PERMEABLE MATERIAL PER STATE OF
CALIFORNIA STANDARD SPECIFICATION, OR APPROVED ALTERNATE.
CLASS 2 GRADING AS FOLLOWS:

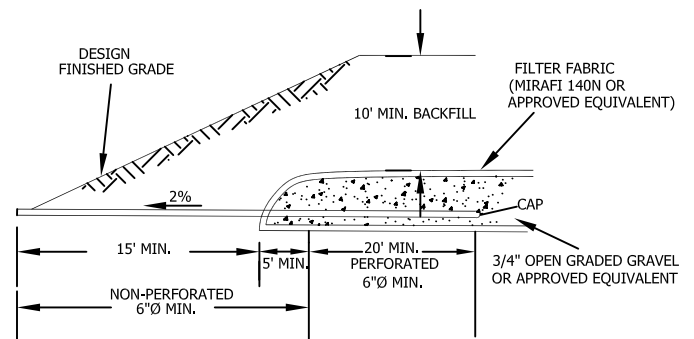
Sieve Size	Percent Passing
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SUBDRAIN ALTERNATE B

DETAIL OF CANYON SUBDRAIN TERMINAL



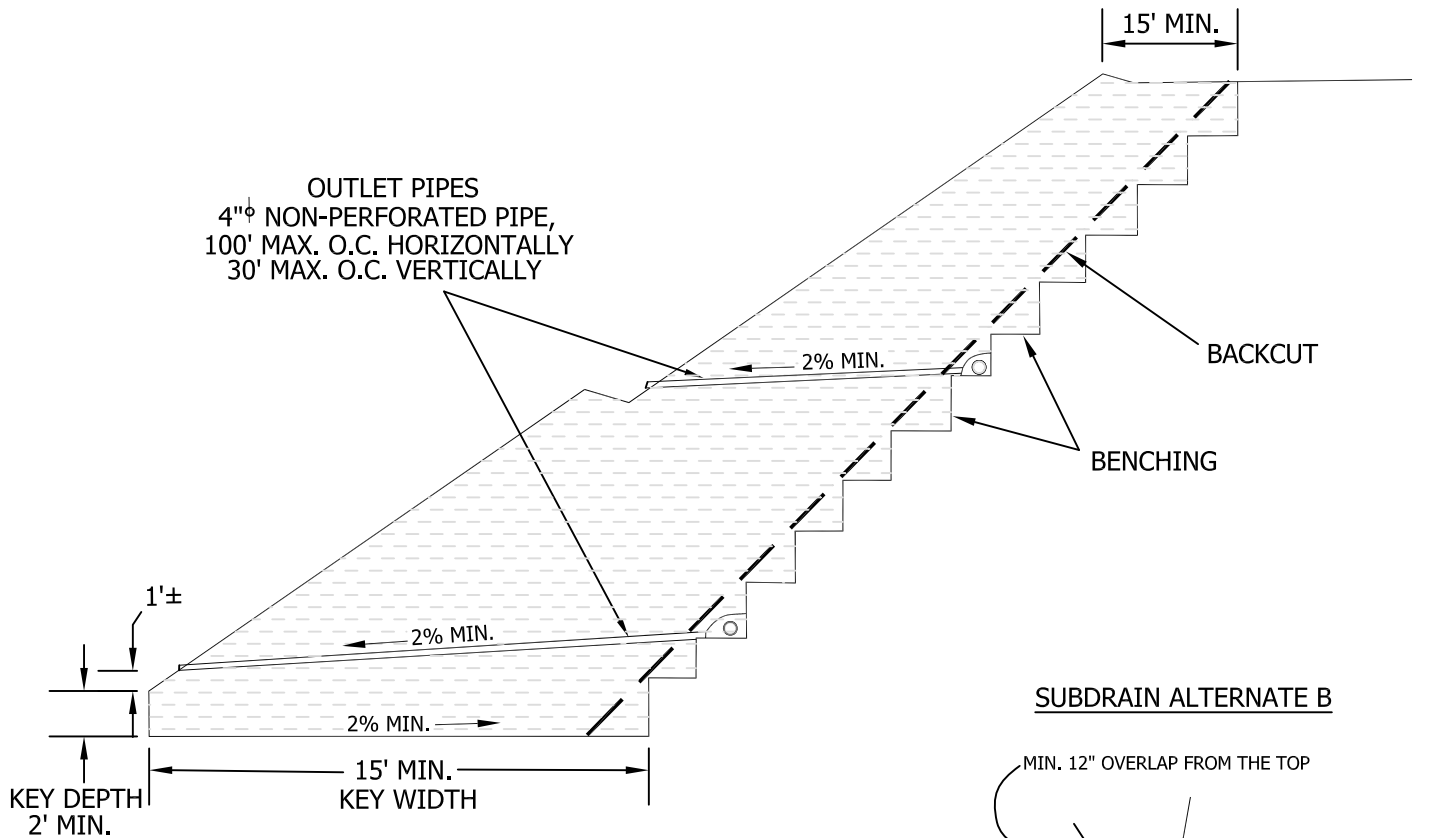
○ PERFORATED PIPE IS OPTIONAL PER
GOVERNING AGENCY'S REQUIREMENTS



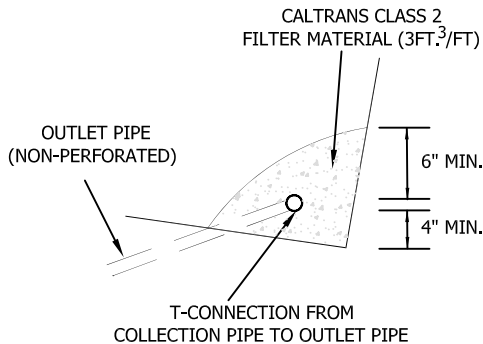
CANYON
SUBDRAIN

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS C



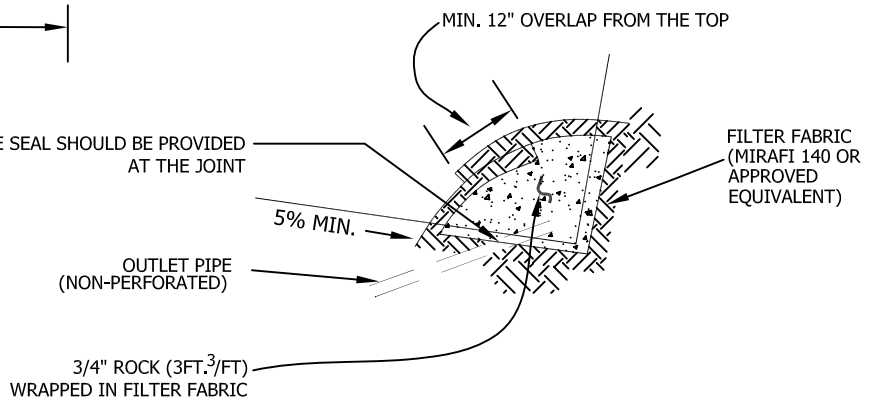


SUBDRAIN ALTERNATE A



POSITIVE SEAL SHOULD BE PROVIDED
AT THE JOINT

SUBDRAIN ALTERNATE B



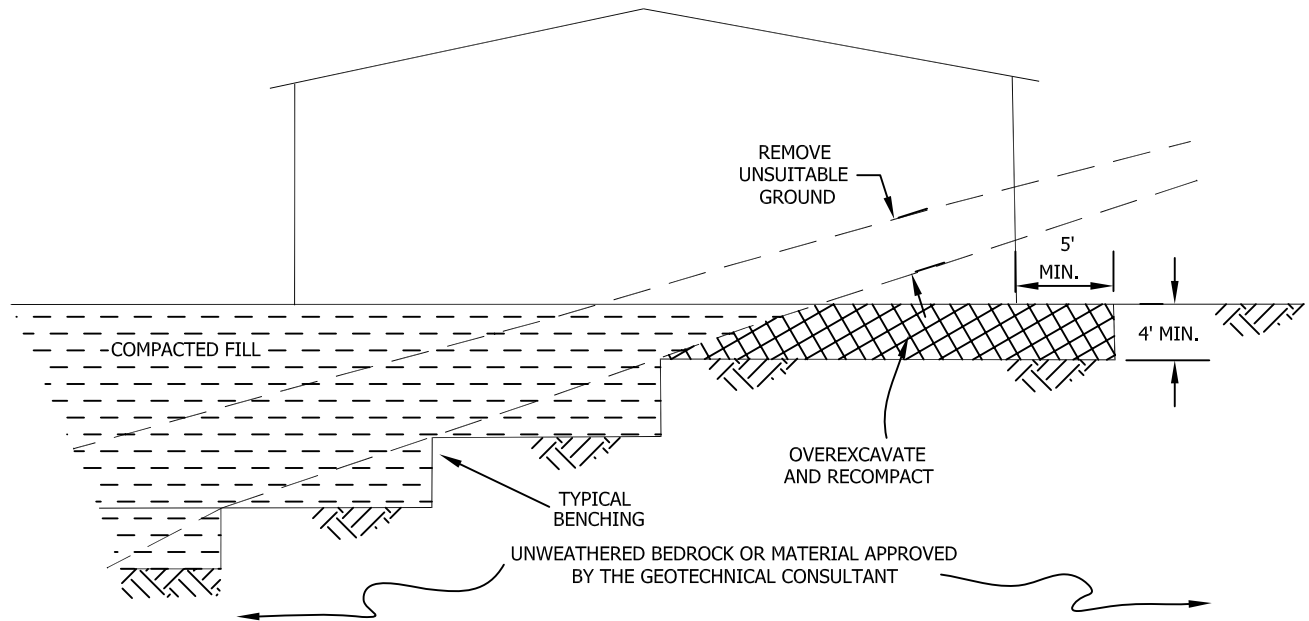
- **SUBDRAIN INSTALLATION** - Subdrain collector pipe shall be installed with perforations down or, unless otherwise designated by the geotechnical consultant. Outlet pipes shall be non-perforated pipe. The subdrain pipe shall have at least 8 perforations uniformly spaced per foot. Perforation shall be 1/4" to 1/2" if drilled holes are used. All subdrain pipes shall have a gradient at least 2% towards the outlet.
- **SUBDRAIN PIPE** - Subdrain pipe shall be ASTM D2751, ASTM D1527 (Schedule 40) or SDR 23.5 ABS pipe or ASTM D3034 (Schedule 40) or SDR 23.5 PVC pipe.
- All outlet pipe shall be placed in a trench and, after fill is placed above it, rodded to verify integrity.

**BUTTRESS OR
REPLACEMENT FILL
SUBDRAINS**

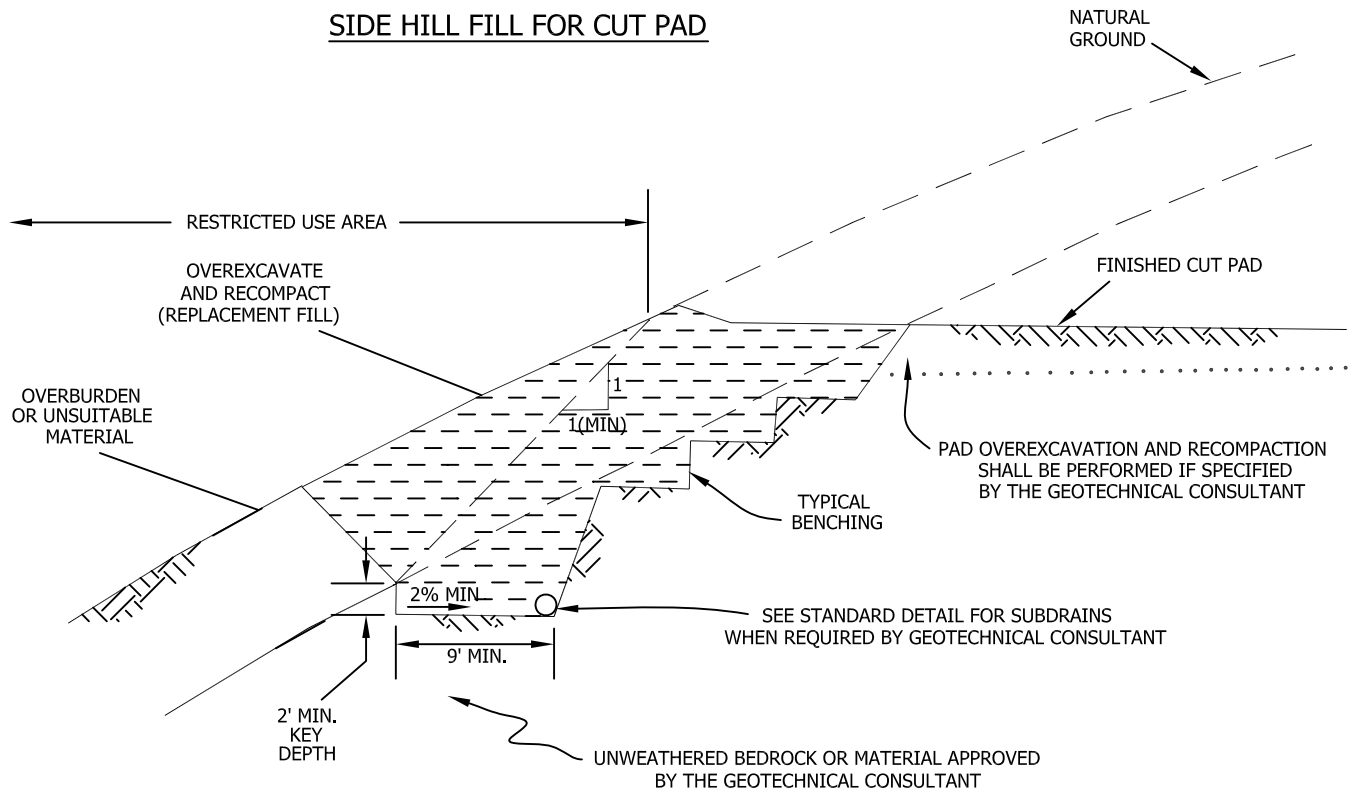
**GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS D**



CUT-FILL TRANSITION LOT OVEREXCAVATION



SIDE HILL FILL FOR CUT PAD



TRANSITION LOT FILLS
AND SIDE HILL FILLS

GENERAL EARTHWORK AND GRADING
SPECIFICATIONS
STANDARD DETAILS E

