REPORT OF GEOTECHNICAL EXPLORATION

PROPOSED 66-LOT RESIDENTIAL DEVELOPMENT
VTTM NO. 20544, APN 3071-111-01
SOUTHEAST OF BEAR VALLEY ROAD AND VERBENA ROAD
VICTORVILLE, SAN BERNARDINO COUNTY, CALIFORNIA

Prepared For

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Project No. 13526.001

June 24, 2022

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To: Charles F. Paine and Judith S. Paine, Trustees of the Pain Family Trust dated 12/14/1978
Jack Lee Herron and Deborah L. Herron, Trustees of the Jack and Deborah Herron Trust dated 02/01/08
2472 Chambers Road, Suite 150
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Attention: Mr. Jack Herron

Subject: Report of Geotechnical Investigation, Proposed 66-Lot Residential Development, VTTM No. 20544, APN 3071-111-01
Southeast of Bear Valley Road and Verbena Road
Victorville, San Bernardino County, California

In response to your request, Leighton and Associates, Inc. has conducted a preliminary geotechnical investigation for the proposed 66-Lot residential development located at Vesting Tentative Tract Map (VTTM) No. 20544, Assessor's Parcel Number (APN) 3071-111-01, in the City of Victorville California. The property is located southeast of Bear Valley Road and Verbena Road. The purpose of this investigation was to evaluate the general geotechnical conditions at the site with respect to the proposed development and to provide preliminary geotechnical recommendations for design and construction.

Based on this investigation, construction of the proposed residential development is feasible from a geotechnical standpoint. The most significant geotechnical issues with respect to the project are those related to the potential for strong seismic shaking and potentially compressible soil. Good planning and design of the project can limit the impact of these constraints. This report presents our preliminary findings, conclusions, and geotechnical recommendations for the project.

We appreciate the opportunity to work with you on the development of this project. If you have any questions regarding this report, please call us at your convenience.

Respectfully submitted,

LEIGHTON AND ASSOCIATES, INC.

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1.0 INTRODUCTION

1.1 Site Location and Description

The site is approximately 20 acres and located southeast of the intersection of Bear Valley Road and Verbena Road, in the southwestern portion of the City of Victorville, San Bernardino County, California (see Figure 1, *Site Location Map*). The site is bounded to the north Bear Valley Road, to the west by Verbena Road, to the south by Sierra Road, and to the east by vacant land. The surrounding areas around the site are currently undeveloped, with the exception of existing single-family homes on the southwest corner of Sierra Road and Verbena Road and the previous home that was removed sometime in 2016 adjacent to the northeast corner of the site.

The site and surroundings are relatively flat and generally drain to the north, with site elevations ranging from approximately 3,385 feet above mean sea level on the south to an approximate elevation of 3,360 feet above msl on the northern end of the site.

Based on a review of available historical aerial photographs from 1952 to present, the site has been undeveloped.

1.2 Proposed Development

Our understanding of the project is based on the provided *Vesting Tentative Tract Map No. 20544* prepared by Madole and Associates, Inc., plotted March 11, 2022. The plan shows the proposed development of approximately 66 lots that will accommodate single family homes, each with a minimum 7,200-square-foot lot area. Also planned are three lots (A, B, C) planned for landscape, and one lot (D) proposed to be utilized as a park and detention basin. Additional development will include interior streets and sidewalks, and presumably landscape and buried utility improvements.

Although no grading plans were available for our review, the site has low relief, and relatively shallow cuts and fills (generally 5 feet or less) have been assumed to ultimately achieve design grades.

1.3 Purpose of Investigation

The purpose of this study has been to evaluate the general geotechnical conditions at the site with respect to the proposed development and to provide preliminary geotechnical recommendations for design and construction.

Our geotechnical exploration included hollow-stem auger soil borings, infiltration testing, laboratory testing and geotechnical analysis to evaluate existing conditions and develop the recommendations contained in this report.

1.4 Scope of Work

Our scope of work has included the following tasks:

- Background Review: We reviewed available, relevant geotechnical/ geologic maps and reports and aerial photographs available from our in-house library.

- Utility Coordination: We contacted Underground Services Alert (USA) prior to excavating borings so that utility companies could mark utilities onsite in the areas of our proposed borings.

- Field Exploration: Our field exploration included drilling of hollow-stem auger borings and infiltration testing. Logs of the geotechnical borings and infiltration testing are presented in Appendix B.

  Seven (7) hollow-stem auger borings (LB-1 through LB-5 and IT-1 through IT-2) were drilled, logged, and sampled in representative locations onsite to evaluate the subsurface conditions. The borings were excavated by a subcontracted drill rig operator to depths ranging from 20 to 51.5 feet below the existing ground surface (bgs). Each boring was logged by a member of our technical staff. Relatively undisturbed soil samples were obtained at selected depth intervals within each boring using a modified California ring sampler. Standard Penetration Tests (SPT) were conducted at selected depths within the borings and samples were obtained. Bulk samples of representative soil types were also obtained from the borings.

Well permeameter tests were conducted within two of these borings (IT-1 and IT-2) to evaluate general infiltration rates of subsurface soils at the depths and locations tested based on the anticipated location of the proposed basin within Lot D located on the northeast portion of the site. Well permeameter tests were conducted based on the USBR-7300-89 method. Tests were conducted at depths of approximately 17 to 20 feet bgs to estimate the tested soil's infiltration characteristics.

Borings drilled at the site were backfilled and tamped with soil cuttings at the surface. Logs of the geotechnical borings are presented in Appendix B. Approximate boring locations are shown on the accompanying Figure 2, *Boring and Infiltration Test Location Map*.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected relatively undisturbed and bulk soil samples obtained during our field investigation. This laboratory testing program was designed to evaluate engineering characteristics of site soils. Laboratory tests conducted during this investigation include:

  - In situ moisture content and dry density
  - Maximum dry density, optimum moisture content
  - Grain size distribution and percent fines
  - Expansion index
  - Swell and collapse potential
  - Water-soluble sulfate concentration
  - Resistivity, chloride content and pH

  The laboratory test results are presented in Appendix C.

- Engineering Analysis: Data obtained from our background review, field exploration and geotechnical laboratory testing was evaluated and analyzed to develop geotechnical conclusions and to provide preliminary recommendations presented in this report.

- Report Preparation: Results of our preliminary geotechnical investigation have been summarized in this report, presenting our findings, conclusions and preliminary geotechnical recommendations for design and construction of the proposed residential development.

2.0 FINDINGS

2.1 Regional Geologic Setting

The site is located in the western Mojave Desert, in San Bernardino County California, and is part of the Mojave Desert geomorphic province, a wide interior region of isolated mountain ranges separated by broad desert plains and deep alluvial valleys. The Mojave province is a structural block wedged in a sharp angle between the Garlock fault (southern boundary of the Sierra Nevada) and the San Andreas fault, where the San Andreas fault bends north from its northwest trend. The northern boundary of the Mojave province is separated from the prominent Basin and Range to the north by the eastern extension of the Garlock fault. The San Andreas fault, at its nearest point, is approximately 12.7 miles southwest of the project site.

The geology of the region consists of three main rock groups, crystalline rocks of Pre-Tertiary age; volcanic and sedimentary rocks of the Tertiary age; and alluvial sedimentary rocks of the Quaternary age. The Pre-Tertiary and Tertiary rocks are hard, consolidated materials forming the surrounding mountains and rocky buttes that rise from the valley floors and underlie the alluvium at depths. The valley soil profile consists of up to several thousand feet of fine to coarse-grained alluvial fill underlain by consolidated rocks. The alluvial fill consists of Pliocene to Holocene age (5 million years old to recent) fine to coarse-grained soil layers formed as a result of uplift and erosion of the surrounding mountains. Figure 3, *Regional Geologic Map* shows the site in relation to the predominate geologic material (alluvium) of the area.

Quaternary Young Alluvial Fan Deposits (Map Symbol Qyf): On a local site-specific scale, the site is mapped as being underlain by Holocene age (recent) young alluvial fan deposits (see Figure 3, *Regional Geology Map*) consisting of fine to coarse sand and gravel eroded and transported from the surrounding mountain areas.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is underlain by alluvial deposits. The alluvial soil encountered within the borings generally consisted of sand and silty sands with varying amounts of gravel within the matrix. The soil was generally moist and medium dense to dense. The in-situ moisture content within the upper approximately 10 feet generally ranged from 1 to 4 percent and was typically described as dry to moist, while the in situ dry density within the upper 10 feet generally ranged from 107 to 120 pound per cubic feet (pcf). More detailed descriptions of the subsurface soil are presented on the boring logs in Appendix B.

2.2.1 Compressible and Collapsible Soil

Soil compressibility refers to a soil's potential for settlement when subjected to increased loads as from a fill surcharge. Based on our investigation, the near surface alluvial soil encountered is generally considered moderately compressible. Partial removal and recompaction of this material under shallow foundations is recommended to reduce the potential for adverse total and differential settlement of the proposed improvements.

Collapse potential refers to the potential settlement of a soil under existing stresses upon being wetted. Based on the results of or laboratory testing, the relatively shallow onsite alluvial soil is anticipated to have a low collapse potential.

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 subject to large uplifting forces caused by the swelling. Without proper measures taken, heaving and cracking of both building foundations and slabs-on-grade could result.

A sample of the subsurface soil was tested for expansion potential. The test result indicates an Expansion Index of 0. Based on our field sampling and laboratory test results, soils with very low expansion potential are expected onsite.

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 the American Concrete Institute (ACI) provisions, adopted by the 2016 CBC (CBC, 2016, Chapter 19, and ACI, 2005, Chapter 4).

A near-surface soil sample was tested during this investigation for soluble sulfate content. The results of this test indicate a sulfate content of approximately 0.015 percent by weight, indicating negligible sulfate exposure.

2.2.4 Resistivity, Chloride and pH

Soil corrosivity to ferrous metals can be estimated by the soil's electrical resistivity, chloride content, and pH level. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive, soil having a minimum resistivity of 1,000 to 2,000 is considered corrosive, soil having a minimum resistivity of 2,000 to 5,000 is considered moderately corrosive, and soil having a minimum resistivity of 5,000 to 10,000 is considered mildly corrosive Soil with a chloride content of 500 ppm or greater is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested during this investigation to estimate minimum resistivity, chloride content, and pH. The test indicates a minimum resistivity 9,410 ohm-cm, chloride content of 40 ppm, and pH of 6.8. Based on these results, the onsite soil is considered mildly corrosive to ferrous metals.

2.3 Groundwater

Groundwater was not encountered in any of our borings to a max explored depth of 51 ½ feet below ground surface.

Based on groundwater data from a nearby well (State Well No. 05N05W22E002S) located approximately 3 miles northeast of the site with groundwater readings from 1960 to 2006, the shallowest groundwater reading identified was measured on March 1961, which was at an elevation of 2,823 feet above mean sea level (msl). This elevation correlates to a groundwater depth of 302 feet bgs based on the lowest elevation at the project site (CDWR, 2022).

More recent groundwater readings from another nearby groundwater well (State Well No. 04N05W09R001S) managed by the Mojave Water Agency, located approximately 2.1 miles southeast of the site, from readings measured from October 2010 to March 2020 indicated the shallowest groundwater recorded was at an elevation of 2,881 feet msl on May 2015 (CDWR, 2022). This groundwater elevation correlates to a groundwater depth of approximately 589 feet bgs based on the lowest elevation at the project site.

2.4 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.

2.4.1 Surface Faulting

Our review of available in-house literature indicates that there are no known active faults traversing the site. The closest known active or potentially active fault is the Cleghorn fault, located approximately 11.1 miles south of the site. Based on our review of readily available resources, no active faults have been previously mapped through or trending towards the project site.

Based on our understanding of the current geologic framework, the potential for future surface rupture of active faults onsite is considered very low.

2.4.2 Seismic Design Parameters

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: 34.4685, -117.4247	
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	1.423 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.553 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	*1.747**
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.423 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	*0.966** g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	0.949 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	*0.644** g
Mapped MCE_G peak ground acceleration (11.8.3.2, Fig 22-9 to 13), **PGA**	0.500 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	0.550 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., Fv 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 (Cs), and Fv is only used for calculation of Ts. 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.

2.5 Secondary Seismic Hazards

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

2.5.1 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.

San Bernardino County has mapped the site to not be within an area that is susceptible to liquefaction hazards based on the Land Use Plan Geologic Hazards Overlay Map EHFH C (SBC, 2010) The State of California has not prepared liquefaction hazard maps for this area.

Based on the site not having shallow groundwater historically and the relatively dense nature of the underlying soils the potential for liquefaction to occur onsite is considered very low.

2.5.2 Seismically Induced Settlement

During a strong seismic event, seismically induced settlement can occur within loose to moderately dense, dry or saturated granular soil. Settlement caused by ground shaking is often nonuniformly distributed, which can result in differential settlement.

We have performed analyses to estimate the potential for seismically induced settlement using the method of Tokimatsu and Seed (1987), and based on Martin and Lew (1999), considering the maximum considered earthquake (MCE) peak ground acceleration (PGA_M). The results of our analyses suggest that the onsite soils are susceptible to 1 inch or less of seismic settlement based on the MCE. Differential settlement due to seismic loading is assumed to be ½ inch or less over a horizontal distance of 40 feet based on the MCE. A summary of seismic settlement analysis is included in Appendix D.

2.5.3 Seismically Induced Landslides

The site is generally level without significant slopes. This site is not considered susceptible to static slope instability or seismically induced landslides.

2.6 Infiltration Testing

Two well permeameter tests were conducted onsite (IT-1 and IT-2) to evaluate infiltration rates of native soils. The encountered soils were generally composed of sand and silty sand in the upper 20 feet. Well permeameter tests IT-1 and IT-2 were performed within sand and silty sand material at test zone depths of about 17 to 20 feet bgs.

Well permeameter tests are useful for field measurements of soil infiltration rates, and are suited for testing when the design depth of the basin or chamber is deeper than current existing grades. It should be noted that this is a clean-water, small-scale test, and that correction factors need to be applied. The test consisted of excavating a boring to the depth of the test. A layer of clean sand was placed in the boring bottom to support temporary perforated well casing pipe. In addition, No. 3 Monterrey Sand was poured around the outside of the well casing within the test zone to prevent the boring from caving/collapsing or eroding when water was added. Water was added into the boring to the predetermined test zone depth from a supplied water source and measured at select time intervals as a falling head test method. The test is repeated several times until a constant rate is achieved. The incremental infiltration rate as measured during intervals of the test is defined as the incremental flow rate of water infiltrated, divided by the surface area of the infiltration interface. The test was conducted based on the USBR 7300-89 test method.

Field infiltration tests indicated small scale unfactored infiltration rates of 14.9 in/hr to 35 in/hr at the depths tested. See Section 3.8 for infiltration recommendations. The results of infiltration tests are provided in Appendix B.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on this investigation, construction of the proposed 66-Lot residential development is feasible from a geotechnical standpoint. No severe geologic or soils related issues were identified that would preclude development of the site for the proposed improvements. The most significant geotechnical issues at the site are those related to the potential for strong seismic shaking, and potentially compressible near surface soils. Good planning and design of the project can limit the impact of these constraints. Remedial recommendations for these and other geotechnical issues are provided in the following sections.

3.1 General Earthwork and Grading

All grading should be performed in accordance with the General Earthwork and Grading Specifications presented in Appendix E, unless specifically revised or amended below or by future recommendations based on final development plans.

3.1.1 Site Preparation

Prior to construction, the site should be cleared of vegetation, trash and debris, which should be disposed of offsite. Any underground obstructions should be removed, as should large trees and their root systems. Resulting cavities should be properly backfilled and compacted. Efforts should be made to locate existing utility lines. Those lines should be removed or rerouted if they interfere with the proposed construction, and the resulting cavities should be properly backfilled and compacted.

3.1.2 Overexcavation and Recompaction

To reduce the potential for adverse differential settlement of the proposed improvements, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved. For structures with shallow foundations, we recommend that onsite alluvial soils be overexcavated and recompacted to a minimum depth of 3 feet below the bottom of the proposed footings or 4.5 feet below existing grade, whichever is deeper. Overexcavation and recompaction should extend a minimum horizontal distance of 5 feet from perimeter edges of the proposed footings, where feasible.

Local conditions may require that deeper overexcavation be performed; such areas should be evaluated by Leighton during grading.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, site walls, and areas to receive fill should be overexcavated to a minimum depth of 24 inches below the existing ground surface or 12 inches below the proposed subgrade, whichever is deeper. In addition, any undocumented artificial fill should be overexcavated.

After completion of the overexcavation, and prior to fill placement, the exposed surfaces 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.

3.1.3 <u>Fill Placement and Compaction</u>

Onsite soil to be used for compacted structural fill should also be free of debris, organic material and oversized material (greater than 8 inches in largest dimension). Any soil to be placed as fill, whether onsite or imported material, should be reviewed and possibly tested by Leighton.

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 <u>Import Fill Soil</u>

Import soil to be placed as fill should be geotechnically accepted by Leighton. 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. 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 Shrinkage and Subsidence

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. Subsidence occurs as in-place soil (e.g., natural ground) is moisture-conditioned and densified to receive fill, such as in processing an overexcavation 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 +/- 5 percent
Subsidence (overexcavation bottom processing)	Approximately 0.20 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

Oversized material (rock or rock fragments greater than 8 inches in dimension) was not observed during our investigation. Oversized material should not be used within structural fill areas.

3.2 Recommendations for Foundations

Based on our study, conventional shallow foundations may be used to support the loads of the proposed single-family wood-frame structures. Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1. The following design parameters are based on soils with a very low expansion potential. Additional testing of expansion potential should be conducted at the conclusion of site grading.

3.2.1 Minimum Embedment and Width

Footings for the proposed single-family structures should have a minimum embedment depth in accordance with California Building Code (CBC) requirements, with a minimum width of 24 and 15 inches for isolated and continuous footings, respectively.

3.2.2 Allowable Bearing

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used, based on the minimum embedment depth and width above. This allowable bearing value may be increased by 250 psf per foot increase in depth or width to a maximum allowable bearing pressure of 4,000 psf. If additional allowable bearing pressure is needed, this should be evaluated on a case-by-case basis. These allowable bearing pressures are for total dead load and sustained live loads, and include a factor of safety of 3 which factor of safety does not consider excessive settlement. Footing reinforcement should be designed by the structural engineer, but as a minimum, footings should have one No. 4 rebar top and bottom.

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 0.35. The passive resistance may be computed using an unfactored equivalent fluid pressure of 375 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. Friction and passive pressure may be combined without reduction, provided the footings can move laterally sufficiently to develop passive pressure (approximately ¼ inch); otherwise, friction alone should be assumed.

3.2.4 Settlement Estimates

The recommended allowable bearing pressure is generally based on a total allowable, post-construction settlement of 1 inch. Differential settlement due to static loading is estimated at ½ inch over a horizontal distance of 40 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.

3.3 Recommendations for Slabs-On-Grade

Slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soils with a very low expansion potential. 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 the end of rough grading to evaluate the expansion index of near-surface subgrade soils. Slabs-on-grade should have the following minimum recommended components:

- Subgrade Moisture Conditioning: The subgrade soil should be moisture conditioned to 2 percentage points above optimum moisture content to a minimum depth of 12 inches prior to placing the moisture barrier, steel or concrete.

- Concrete and Structural Design Thickness: Slabs-on-grade should be designed by the structural engineer, but 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 slabs) should be No. 3 rebar placed at 18 inches on center, each direction, mid-depth in the slab.

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, aggregate that is not sufficiently clean, 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, reinforcement in slabs and foundations can generally reduce the potential for shrinkage cracking. The structural engineer should consider these and other pertinent concrete design and construction considerations in slab design and specifications.

3.3.1 Slab Underlayment for Moisture Vapor Retarding

Because moisture vapor from the underlying soils will be transmitted through slabs-on-grade without preventive measures, slab underlayment for moisture vapor retarding should be designed by qualified professionals (such as the structural engineer and/or architect) where control of moisture vapor transmission through slabs is considered important to this project (such as where moisture-sensitive floor coverings or equipment are planned). Slab underlayment typically includes a moisture vapor retarder membrane (such as 10-mil thick or greater), underlain by a capillary break and provisions for protection of the vapor retarder during construction. The structural engineer and/or architect should specify pertinent slab and concrete design parameters, such as whether a sand blotter layer should be placed over the vapor retarder.

Moisture retarders can reduce, but not eliminate moisture vapor rise from the underlying soils up through the slab. Moisture retarders should be designed and constructed in accordance with applicable American Concrete Institute, Portland Cement Association, Post-Tensioning Institute, ASTM International, and California Building Code requirements and guidelines.

Leighton does not practice in the field of moisture vapor transmission evaluation/mitigation, since this does not fall under the geotechnical discipline. Therefore, we recommend that a qualified person, such as the flooring subcontractor, structural engineer, and/or architect, be consulted to evaluate the general and specific moisture vapor transmission paths and any impact on the proposed construction. That person (or persons) should provide recommendations for mitigation of potential adverse impact of moisture vapor transmission on various components of the structures as deemed appropriate. In addition, the recommendations in this report and our services in general are not intended to address mold prevention, since we, along with geotechnical consultants in general, do not practice in the area of mold prevention. If specific recommendations are desired, a professional mold prevention consultant should be contacted.

3.4 Seismic Design Parameters

Seismic parameters presented in this report should be considered during project design. In order to reduce the effects of ground shaking produced by regional seismic events, seismic design should be performed in accordance with the most recent edition of the California Building Code (CBC). Seismic parameters based on the 2016 CBC are included in Section 2.4.2. The seismic parameters should be updated if design will be per the 2019 CBC.

3.5 Lateral Earth Pressures

We recommend that retaining walls be backfilled with very low expansive soil and constructed with a backdrain in accordance with the recommendations provided on Figure 3, *Retaining Wall Backfill and Subdrain Detail*. Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall and are, therefore, not recommended. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Lateral Earth Pressures

Condition	Level Backfill
Active	38
At-Rest	59
Passive (allowable)	250 (Maximum of 4,000 psi)

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 0.35 may be used at the concrete and soil interface. The lateral passive resistance should be taken into account only if it is ensured that soil providing passive resistance, embedded against the foundation elements, will remain intact with time. A soil unit weight of 120 pcf may be assumed for calculating the actual weight of the soil over the wall footing.

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. A third of uniform vertical surcharge-loads should be applied as a horizontal pressure on cantilever (active) retaining walls, while half of uniform vertical surcharge-loads should be applied as a horizontal pressure on braced (at-rest) retaining walls. To account for automobile parking surcharge, we suggest that a uniform horizontal pressure of 100 psf (for restrained walls) or 70 psf (for cantilever walls) be added for design, where autos are parked within a horizontal distance behind the retaining wall less than the height of the retaining wall stem.

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 20 pcf, which is added to the EFP. The additive seismic EFP should be applied at the retained midpoint.

Conventional retaining wall footings should have a minimum width of 24 inches and a minimum embedment of 12 inches below the lowest adjacent grade. An allowable bearing pressure of 2,000 psf may be used for retaining wall footing design, based on the minimum footing width and depth. This bearing value may be increased by 250 psf per foot increase in width or depth to a maximum allowable bearing pressure of 4,000 psf.

3.6 Cement Type and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with onsite soil will have negligible exposure to water-soluble sulfates in the soil. Therefore, common Type II cement may be used for concrete construction. Concrete should be designed in accordance with ACI 318-14, Section 19.3 (ACI, 2014), adopted by the 2019 CBC (Section 1904.2).

Based on our laboratory testing, the onsite soil is considered mildly corrosive to ferrous metals. Corrosion information presented in this report should be provided to your underground utility contractors and consultation with a Corrosion Engineer should be considered.

3.7 Pavement Design

Based on the design procedures outlined in the current Caltrans Highway Design Manual, and using an assumed design R-value of 50, flexible pavement sections may consist of the following for the Traffic Indices indicated. Final pavement design should be based on the Traffic Index determined by the project civil engineer and R-value testing provided near the end of grading.

Table 1 - Asphalt Pavement Section Thickness

Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)*
5.5 or less	3.0	8.0
6	3.5	8.0
7	4.0	8.0

* Pavement section to include a minimum 8" thick of base materials for local, collector, and arterial streets in accordance with the City of Victorville Street Design Standards drawing S-25

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction. 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 recompacted to a minimum of 90 percent relative compaction. Aggregate base should be moisture conditioned, as necessary, and compacted to a minimum of 95 percent relative compaction.

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.

3.8 <u>Infiltration Recommendations</u>

<u>Infiltration Rate:</u>
The onsite silty sandy soils and soils with a relatively low fines content encountered within approximately the upper 15 to 20 feet are anticipated to have infiltration rates of 15 inches per hour or more. We recommend an unfactored (small-scale) incremental infiltration rate of 15 inches per hour at a depth of 15 to 20 feet. Infiltration rates may vary significantly at various depths or locations across the site. It should be confirmed during infiltration facility excavation that the excavation penetrates sufficiently into very granular soils.

We recommend that a correction factor/safety factor be applied to the infiltration rate in conformance with San Bernardino County guidelines, since monitoring of actual facility performance has shown that actual infiltration rates are lower than for small-scale tests. The small-scale infiltration rate should be divided by a correction factor of at least 3, but the correction/safety factor may be higher based on project-specific aspects. If open basins are planned, we recommend that a low-flow infiltration trench with minimum depth of 10 feet be constructed in the bottom of the basin; this low-flow trench should be backfilled with clean washed concrete sand with maximum fines content (passing the No. 200 sieve) of 2 percent by weight.

The infiltration rates described herein are for a clean, unsilted infiltration surface in native, sandy alluvial soil. These values may be reduced over time as silting of the infiltration facility occurs. Furthermore, if the basin or chamber bottom is allowed to be compacted by heavy equipment, this value is expected to be significantly reduced. Infiltration of water through soil is highly dependent on such factors as grain size distribution of the soil particles, particle shape, fines content, clay content, and density. Small changes in soil conditions, including density, can cause large differences in observed infiltration rates. Infiltration is not suitable in compacted fill.

It should be noted that during periods of prolonged precipitation, the underlying soils tend to become saturated to greater and greater depths/extents. Therefore, infiltration rates tend to decrease with prolonged rainfall. It is difficult to extrapolate longer-term, full-scale infiltration rates from small-scale tests, and as such, this is a significant source of uncertainty in infiltration rates.

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 may be needed based on the design of infiltration facilities, particularly considering their type, depth and location.

General Design Considerations:
The periodic flow of water carrying sediments into the infiltration facility, plus the introduction of wind-blown sediments and sediments from erosion of basin side

walls, can eventually cause the bottom of the facility to accumulate a layer of silt, which has the potential of significantly reducing the overall infiltration rate of the facility. Therefore, we recommend that significant amounts of silt/sediment not be allowed to flow into the facility within stormwater, especially during construction of the project and prior to achieving a mature landscape on site. We recommend that an easily maintained, robust silt/sediment removal system be installed to pretreat storm water before it enters the infiltration facility.

As infiltrating water can seep within the soil strata nearly horizontally for long distances, it is important to consider the impact that infiltration facilities can have on nearby subterranean structures, such as basement walls or open excavations, whether onsite or offsite, and whether existing or planned. We understand that some of the infiltration facilities will be located between structures and that water could be present in the system for several days, and saturating the subsurface soils. In this condition, we recommend that a 15-mil stego wrap be placed on the sides of the exaction to limit later movement horizontally and reduce saturation of soil immediately below structures. The seams of the stego wrap should be taped and care taken to limit damage during installation. Setbacks should be discussed with Leighton during the planning process.

Infiltration facilities should be constructed with spillways or other appropriate means that would cause overfilling to not be a concern to the facility or nearby improvements.

For buried chambers, control/access manhole covers should not contain holes or should be screened to prevent mosquitos from entering the chambers.

Additional Design Considerations (Particularly for Open Basins):
If open basins are planned, additional observation of the soils exposed at the bottom of the basin should be conducted, as these soils are critical to the basin's success. Soils at the bottom of buried chambers are also important, but not as critical to their success, provided the infiltration chamber cuts through sufficiently granular soils.

In general, the rate of infiltration reduces as the head of water in the infiltration facility reduces, and it also reduces with prolonged periods of infiltration. As such, water typically infiltrates much faster near the beginning of and/or immediately after storm events than at times well after a storm when the water level in the facility has receded, since the infiltration rate is then slower due to both lower head and longer overall duration of infiltration. In open basins with compacted or silty bottoms, this could be problematic, in that, even if the basin had already infiltrated significant amounts of storm water, the lower several inches or feet of water could remain in the basin for an extended period of time, creating a prolonged open-water safety concern and potential for mosquitos. In a buried/covered infiltration chamber without direct access to the open atmosphere, these conditions would be of less concern.

For open basins and swales, vegetation within the basin bottoms and sides is expected to help reduce erosion and help maintain infiltration rates.

Estimating infiltration rates, especially based on small-scale testing, is inexact and indefinite, and often involves known and unknown soil complexities, potentially resulting in a condition where actual infiltration rates of the completed facility are significantly less than design rates. In open infiltration basins, this could create nuisance water in the basin. As such, enhancements may be needed after completion of the basin if prolonged or frequent standing water is experienced. A potential basin enhancement, if needed, might be to install additional infiltration trenches or infiltration borings in the basin bottom to capture and infiltrate low flows and to help speed infiltration during/after storms; specific recommendations, such as minimum trench/boring depth, would be developed based on conditions observed.

Construction Considerations:

We recommend that Leighton evaluate the infiltration facility excavations, to confirm that granular, undisturbed alluvium is exposed in the bottoms and sides. Additional excavation or evaluation may be required if silty or clayey soils are exposed.

It is critical to infiltration that the basin or chamber bottom not be allowed to be compacted during construction or maintenance; rubber-tired equipment and vehicles should not be allowed to operate on the bottom. We recommend that at least the bottom 3 feet of the basins or chambers be excavated with an excavator or similar.

If fill material is needed to be placed in the basin, such as due to removal of uncontrolled artificial fill, the fill material should be select and free-draining sand, and should be observed and evaluated by Leighton.

Maintenance Considerations:
The infiltration facilities should be routinely monitored, especially before and during the rainy season, and corrective measures should be implemented as/when needed. Things to check for include proper upkeep, proper infiltration, absence of accumulated silt, and that de-silting filters/features are clean and functioning. Pretreatment desilting features should be cleaned and maintained per manufacturers' recommendations. Even with measures to prevent silt from flowing into the infiltration facility, accumulated silt may need to be removed occasionally as part of maintenance.

3.9 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 40 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 geotechnical engineer should be maintained to facilitate construction while providing safe excavations.

3.10 Trench Backfill

Utility-type trenches onsite can be backfilled with the onsite material, provided it is free of debris, significant organic material and oversized material. Prior to backfilling the trench, pipes should be bedded and shaded in a granular material that has a sand equivalent of 30 or greater. The sand should extend 12 inches above the top of the pipe. The bedding/shading sand should be densified. The native backfill should be placed in loose layers, moisture conditioned, as necessary, and mechanically compacted using a minimum standard of 90 percent relative compaction. The thickness of layers should be based on the compaction equipment used in accordance with the Standard Specifications for Public Works Construction (Greenbook).

3.11 Surface Drainage

Inadequate control of runoff water and/or poorly controlled irrigation can cause the onsite soils to expand and/or shrink, producing heaving and/or 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 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 areas.

3.12 Additional Geotechnical Services

The preliminary geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our preliminary 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. Additional geotechnical investigation and analysis may be required based on final improvement plans. Leighton should review the site and grading 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 grading operations. Our conclusions and preliminary recommendations should be reviewed and verified by Leighton during construction and revised accordingly if geotechnical conditions encountered vary from our preliminary findings and interpretations.

Geotechnical observation and testing should be provided:

- After completion of site clearing.
- During overexcavation of compressible soil.
- 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 based in part on data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soil or geologic conditions can be present within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, our findings, conclusions, and recommendations presented in this report are based on the assumption that Leighton and Associates, Inc. will provide geotechnical observation and testing during construction.

This report was prepared for the sole use of Charles F. Paine and Judith S. Paine, Trustees of the Pain Family Trust and Jack Lee Herron and Deborah L. Herron, Trustees of the Jack and Deborah Herron Trust for application to the design of the proposed 66-Lot residential development in accordance with generally accepted geotechnical engineering practices at this time in California.

See the GBA insert on the following page for important information about this geotechnical engineering report.

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, clients can benefit from a lowered exposure to the subsurface problems 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 below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a 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. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *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.*

This Report May Not Be Reliable

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

- for a different client;
- for a different project;
- 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, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been 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 your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-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 from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

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 judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed 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 full-time 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 on hand quickly 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 observation.

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 informational 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, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. 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. 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

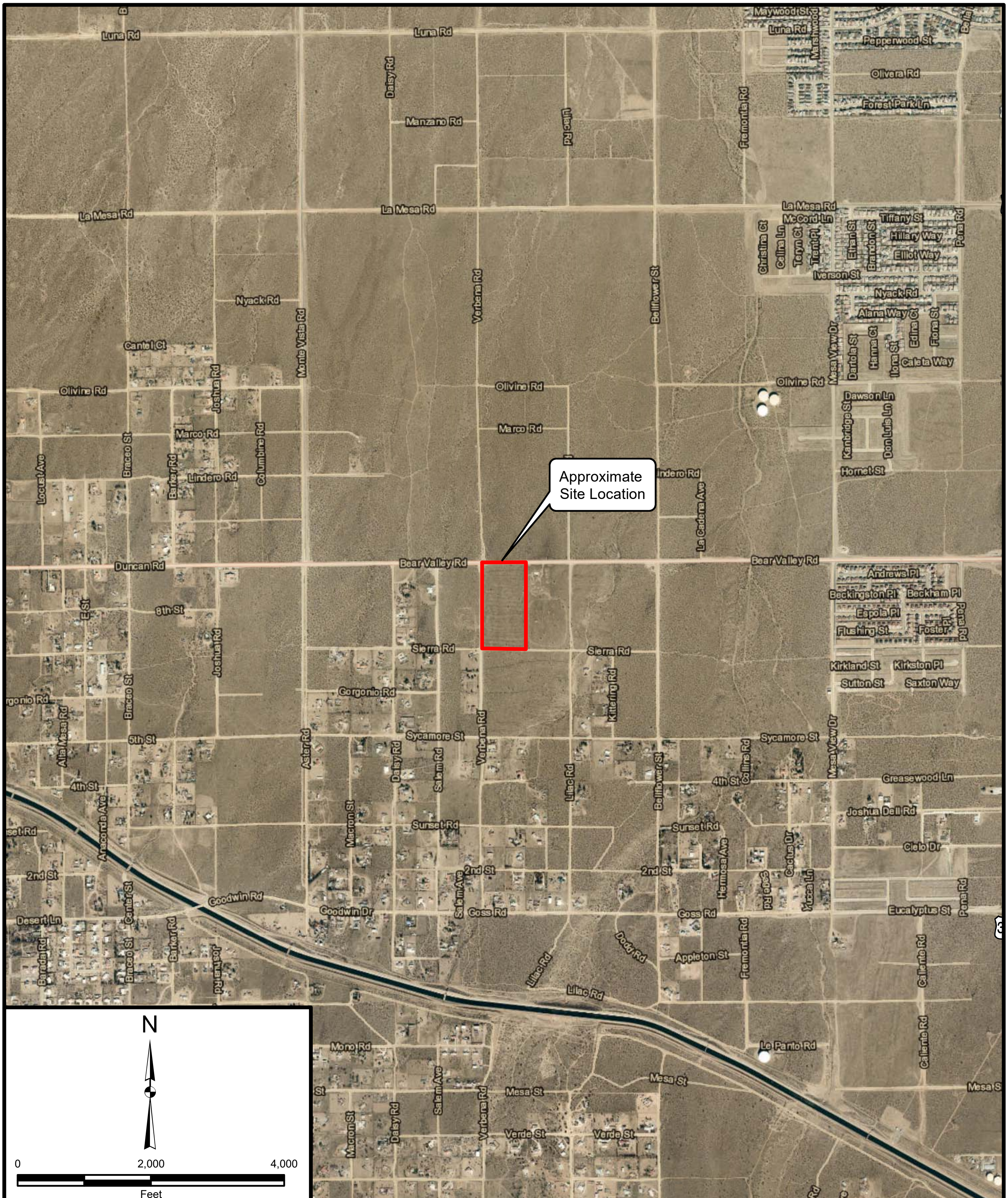
The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any 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 yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled 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

Telephone: 301/565-2733
e-mail: info@geoprofessional.org   www.geoprofessional.org



Project: 13526.001	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: June 2022

Reference: Source: Esri, Maxar, GeoEye, Earthstar Geographics, CNES/Airbus DS,

SITE LOCATION MAP

Proposed 66-Lot Residential Development, Tract No. 20544
Southeast of Bear Valley Road and Verbena Road
City of Victorville, San Bernardino County, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\13526\001\Maps\13526-001_F01_SLM_2022-05-12.mxd on 5/12/2022 9:47:43 AM Author: KVM (btran)

VESTING TENTATIVE MAP, TRACT NO. 20544

IN THE CITY OF VICTORVILLE, COUNTY OF SAN BERNARDINO, STATE OF CALIFORNIA

BEING A PROPOSED SUBDIVISION OF A PORTION OF SECTION 5, TOWNSHIP 4 NORTH, RANGE 5 WEST, S.B.M.

APN NO. 3071-111-01

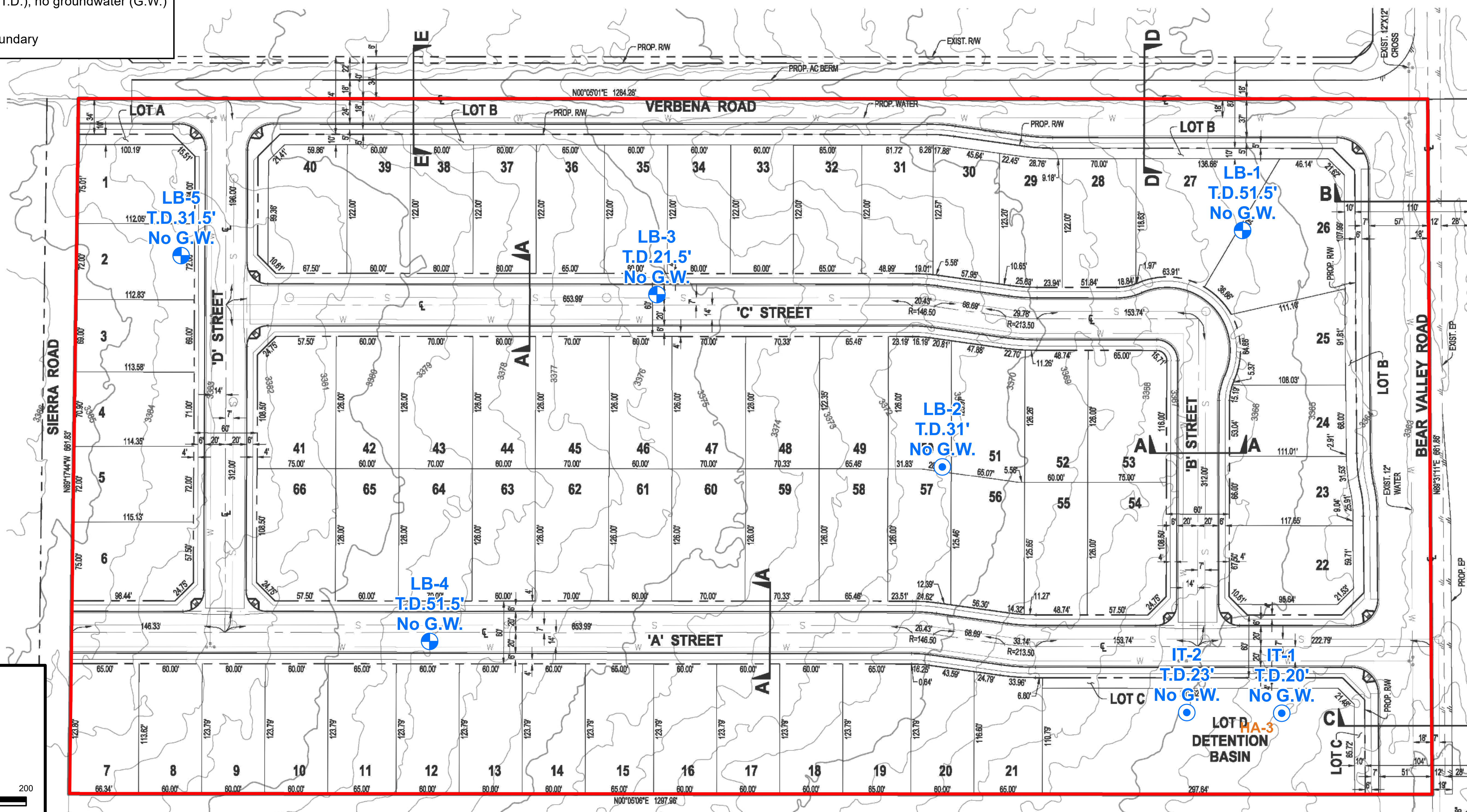
LEGEND

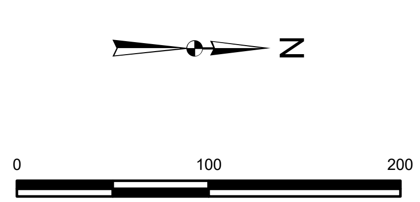
LB-5 — Approximate location of hollow-stem auger boring showing total depth (T.D.), no groundwater (G.W.)

IT-2 — Approximate location of infiltration test showing total depth (T.D.), no groundwater (G.W.)

Approximate Site Boundary

3	7,811
4	8,092
5	8,261
6	8,512
7	8,130
8	7,428
9	7,428
10	7,428
11	8,047
12	7,428
13	7,428
14	7,428
15	8,047
16	7,428
17	7,429
18	7,429
19	8,048
20	7,243
21	7,330
22	8,707
23	7,573
24	7,514
25	8,237
26	13,546
27	11,831
28	8,523
29	7,378
30	7,799
31	8,302
32	7,930
33	7,320
34	7,320
35	7,320
36	7,930
37	7,320
38	7,320
39	7,320
40	9,007





Project: 13526.001	Eng/Geol: JDH/SGO
Scale: 1 " = 100 '	Date: June 2022

Base Map:
Vesting Tentative Tract Map, Tract No. 20544
Sheet 1 of 1 by Madole & Associates, Inc.

EXPLORATION LOCATION MAP

Proposed 66-Lot Residential Development, Tract No. 20544
Southeast of Bear Valley Road and Verbena Road
City of Victorville, San Bernardino County, California

FIGURE 2

Leighton

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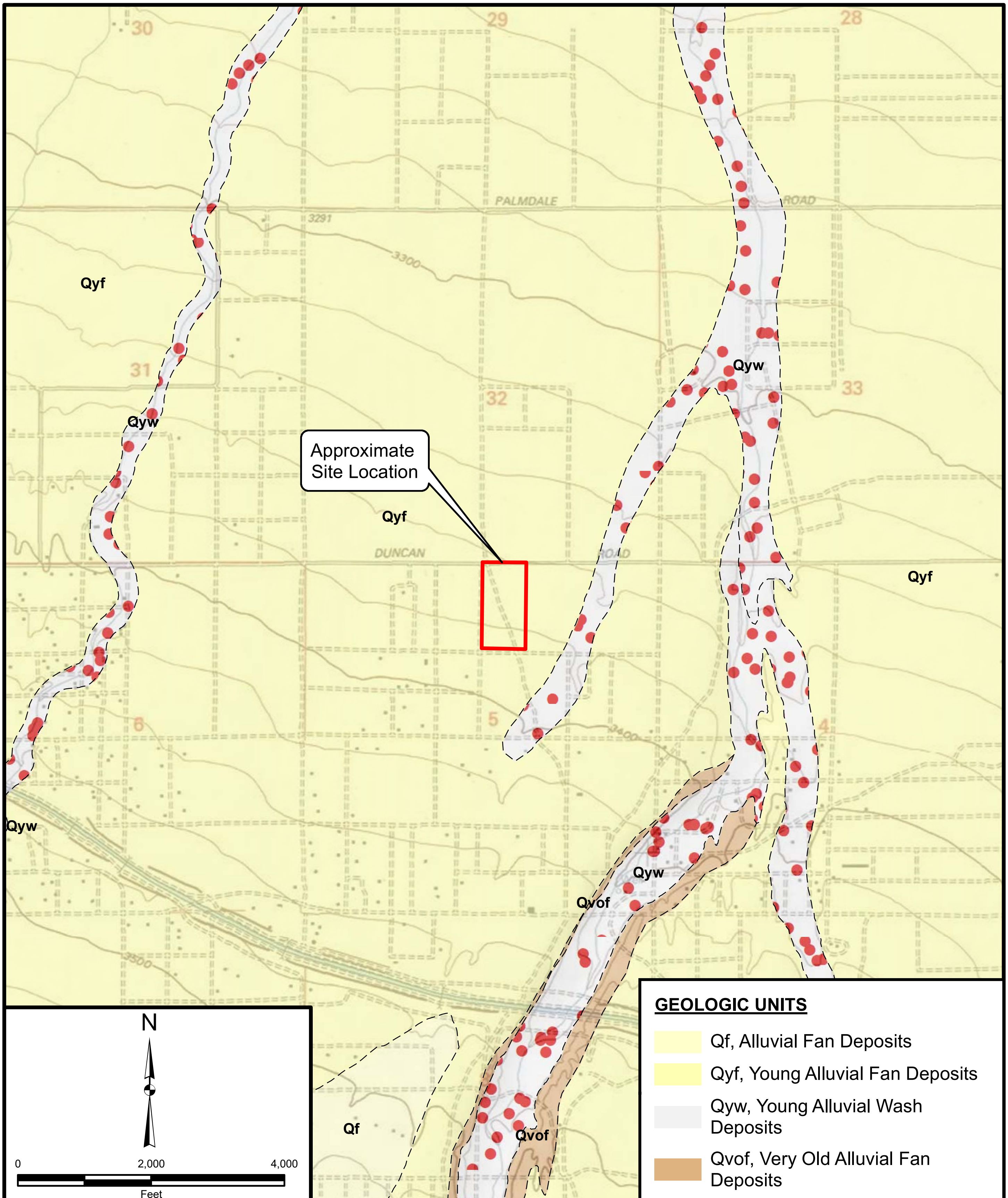


Project: 13526.001	Eng/Geol: JDH/SGO
Scale:1 " = 2,000 '	Date: June 2022

Reference: Geologic Compilation of Quaternary Surficial Deposits in Southern California, Orange County, Bedrossian et. al., 2010

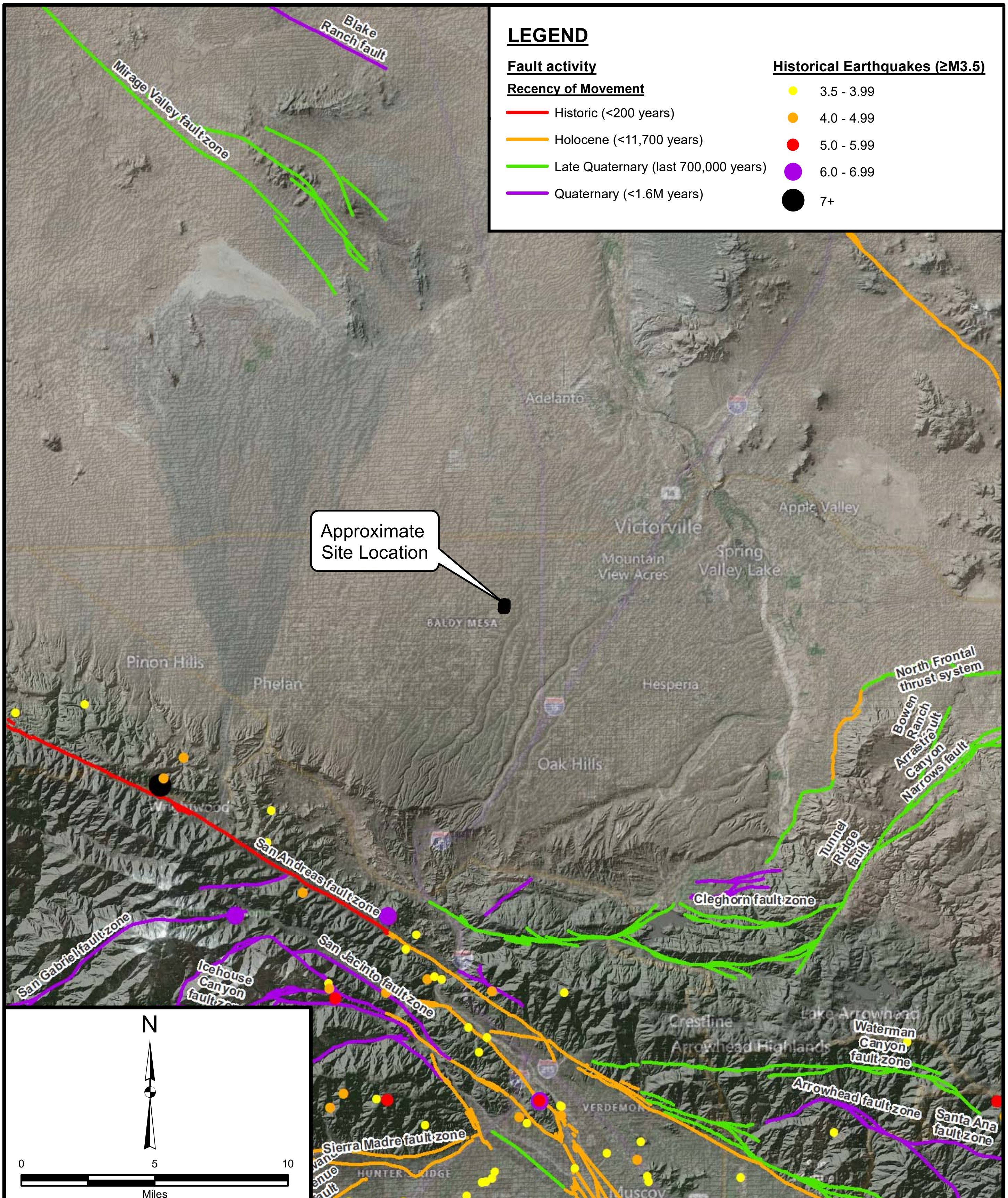
REGIONAL GEOLOGY MAP

Proposed 66-Lot Residential Development, Tract No. 20544
Southeast of Bear Valley Road and Verbena Road
City of Victorville, San Bernardino County, California

FIGURE 3

Leighton

Map Saved as J:\Drafting\13526\001\Maps\13526-001_F03_RGM_2022-05-12.mxd on 5/12/2022 10:01:36 AM Author: KVM (btran)



Project: 13526.001	Eng/Geol: JDH/SGO
Scale: 1 " = 5 miles	Date: June 2022

Basemap Reference: © 2022 Microsoft Corporation
Earthstar Geographics SIO © 2022 TomTom
Seismicity Data Reference: maps.conservation.ca.gov

REGIONAL FAULTS AND HISTORIC SEISMICITY MAP

Proposed 66-Lot Residential Development, Tract No. 20544
Southeast of Bear Valley Road and Verbena Road
City of Victorville, San Bernardino County, California

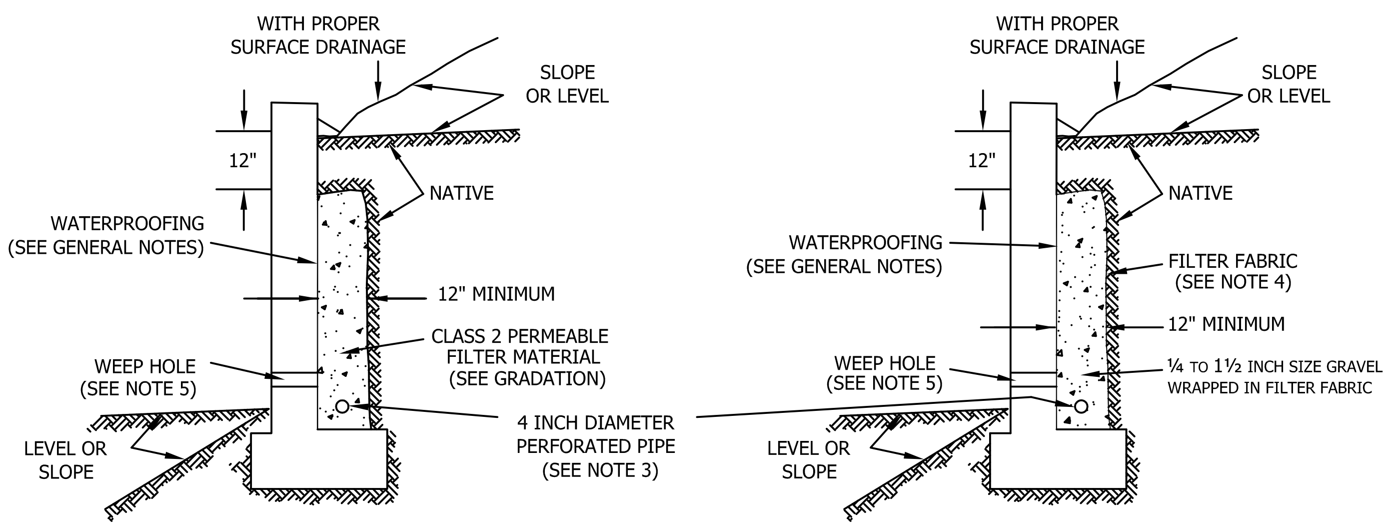
FIGURE 4

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

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

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

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Figure 5

P:Drafting\templates\details\retain-wall-backfill-and subdrain.dwg (7/00)

APPENDIX A

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APPENDIX A

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APPENDIX B

GEOTECHNICAL BORING LOGS
AND INFILTRATION TEST RESULTS

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APPENDIX B

FIELD EXPLORATION

Encountered soils were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D 2488). Relatively undisturbed soil samples were obtained at selected intervals within these borings using both a California ring-lined and Standard Penetration Test (SPT) split-spoon sampler. Standard Penetration Test (SPT) resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch outside diameter split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D 1586). In addition, 2.4-inch inside diameter brass ring samples were obtained using a Modified California sampler driven into the soil with the 140-pound hammer. Borings were backfilled with soil cuttings obtained during the exploration. Representative earth-material samples obtained from these subsurface explorations were transported to our geotechnical laboratory for evaluation and appropriate testing.

The attached subsurface exploration logs and related information depict subsurface conditions only at the locations indicated and at the particular date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

GEOTECHNICAL BORING LOG LB-1

Project No.	13526.001	**Date Drilled**	5-4-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3366'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

SOIL DESCRIPTION

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.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	@Surface: brush, over SILTY SAND (SM)	
3365										
				R-1	6 17 24	113	3	SM	@2.5': SILTY SAND (SM), medium dense, light brown, dry to slightly moist, fine sand, fine to coarse gravel, micaceous, non-plastic	
3360	5			R-2	11 24 33	119	4	SM	@5': SILTY SAND (SM), dense, light brown, dry to slightly moist, fine to medium sand, fine to coarse gravel, micaceous, non-plastic	
				R-3	3 5 8	114	1	SM	@7.5': SILTY SAND (SM), loose, light brown, dry to slightly moist, fine sand, fine to coarse gravel, micaceous, non-plastic	CO
3355	10			R-4	5 11 14	109	2	SP-SM	@10': SAND with silt (SP-SM), medium dense, brown, slightly moist to moist, fine to coarse, trace coarse gravel, 10% fines (lab)	-200
3350	15			R-5	20 46 50/6"			SM	@15': SILTY SAND (SM), dense, light brown, dry to slightly moist, fine sand, fine to coarse gravel, micaceous, non-plastic	
3345	20			S-1	11 24 50/3"			SP-SM	@20': SAND with silt (SP-SM), dense, brown, slightly moist to moist, fine to coarse, trace coarse gravel	
3340	25			R-6	13 29 30			SM	@25': SILTY SAND (SM), dense, light brown, dry to slightly moist, fine to medium sand, trace fine gravel, micaceous, non-plastic	
	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

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GEOTECHNICAL BORING LOG LB-1

Project No.	13526.001	**Date Drilled**	5-4-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3366'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

SOIL DESCRIPTION

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.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
3335	30			S-2	6 9 12			SM	@30': SILTY SAND (SM), medium dense, light brown, dry to slightly moist, fine to medium sand, trace fine gravel, micaceous, non-plastic	
3330	35			R-7	6 11 16			SM	@35': SILTY SAND (SM), medium dense, light brown, dry to slightly moist, fine to medium sand, trace fine gravel, micaceous, non-plastic	
3325	40			S-3	5 18 18			SM	@40': SILTY SAND (SM), dense, light brown, dry to slightly moist, fine to medium sand, trace fine gravel, micaceous, non-plastic	
									@42.5': Spoils: light olive brown color	
3320	45			R-8	15 24 26			SP-SM	@45': SAND with silt (SP-SM), dense, light olive brown, slightly moist to moist, fine to coarse sand, trace fine gravel	
3315	50			S-4	7 8 11			SM	@50': SILTY SAND (SM), medium dense, light olive brown, slightly moist to moist, fine to medium sand, trace fine gravel	
									TOTAL DEPTH = 51.5 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3310	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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- PP POCKET PENETROMETER
- RV R VALUE
- SA SIEVE ANALYSIS
- SE SAND EQUIVALENT
- SG SPECIFIC GRAVITY
- UC UNCONFINED COMPRESSIVE STRENGTH

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GEOTECHNICAL BORING LOG LB-2

Project No.	13526.001	**Date Drilled**	5-4-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3371'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

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.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	0			B-1				SM	@Surface: brush, over SILTY SAND (SM)	CR
3370				R-1	7 19 20			SM	@2.5': SILTY SAND (SM), medium dense, light brown, dry to slightly moist, fine sand, trace medium, trace fine gravel, micaceous, non-plastic	
3365	5			R-2	7 9 12			SP-SM	@5': SAND with silt (SP-SM), medium dense, brown, slightly moist to moist, fine to coarse sand, fine gravel, trace coarse, 12% fines (lab)	-200
				R-3	10 19 26			SM	@7.5': SILTY SAND (SM), medium dense, brown, moist, fine to medium sand, trace fine gravel, micaceous, non-plastic	
3360	10			R-4	6 10 20			SP-SM	@10': SAND with silt (SP-SM), medium dense, brown, slightly moist to moist, fine to coarse sand, fine gravel, trace coarse	
3355	15			S-1	10 18 19			SP-SM	@15': SAND with silt (SP-SM), dense, brown, slightly moist to moist, fine to coarse sand, fine gravel, trace coarse	
3350	20			R-5	5 25 50/3"			SP-SM	@20': SAND with silt (SP-SM), dense, brown, slightly moist to moist, fine to coarse sand, fine gravel, trace coarse	
3345	25			S-2	7 19 23			SP-SM	@25': SAND with silt (SP-SM), dense, brown, slightly moist to moist, fine to coarse sand, fine gravel, trace coarse	
	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

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GEOTECHNICAL BORING LOG LB-2

Project No.	13526.001	**Date Drilled**	5-4-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3371'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-6	28 50/6"			SP-SM	@30': SAND with silt (SP-SM), dense, brown, slightly moist to moist, fine to coarse sand, fine gravel, trace coarse	
3340									**TOTAL DEPTH = 31 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3335	35									
3330	40									
3325	45									
3320	50									
3315	55									
	60									

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.

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

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GEOTECHNICAL BORING LOG LB-3

Project No.	13526.001	**Date Drilled**	5-4-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3375'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

SOIL DESCRIPTION

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.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
3375	0			B-1				SM	@Surface: brush, over SILTY SAND (SM)	
				R-1	10 14 16	120	2	SM	@2.5': SILTY SAND (SM), medium dense, light brown, dry to slightly moist, fine sand, trace medium, trace fine gravel, micecous, non-plastic, 28% fines (lab)	MD, SA, EI
3370	5			R-2	9 16 24			SM	@5': SILTY SAND (SM), medium dense, light brown, dry to slightly moist, fine sand, trace medium, trace fine gravel, micecous, non-plastic	
				R-3	10 20 26	120	2	SM	@7.5': SILTY SAND (SM), medium dense, light brown, dry to slightly moist, fine sand, trace medium, trace fine gravel, micecous, non-plastic	
3365	10			R-4	7 21 31			SP-SM	@10': SAND with silt (SP-SM), dense, brown, slightly to moist, fine to coarse sand, fine gravel	
3360	15			R-5	8 24 50/5"	117	3	SP-SM	@15': SAND with silt (SP-SM), dense, brown, slightly to moist, medium to coarse sand, trace fine, fine to coarse gravel	
3355	20			S-1	5 10 20			SP-SM	@20': SAND with silt (SP-SM), dense, brown, slightly to moist, medium to coarse sand, trace fine, fine to coarse gravel	
									TOTAL DEPTH = 21.5 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3350	25									
3345	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

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GEOTECHNICAL BORING LOG LB-4

Project No.	13526.001	**Date Drilled**	5-5-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3378'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

SOIL DESCRIPTION

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.

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	Soil Description	Type of Tests
	0			B-1				SM	@Surface: brush, over SILTY SAND (SM)	
3375				R-1	5 17 21	119	2	SM	@2.5': SILTY SAND (SM), medium dense, light brown, dry, fine to coarse gravel, ~30% fines (field estimate), trace rootlets, slightly porous, micaceous	
	5								@5': SAND with silt (SP-SM), loose, brown, dry to slightly moist, fine to coarse sand, ~10% fines (field estimate), trace rootlets	
3370				R-2	10 20 26	119	3	SP-SM	@7.5': SAND with silt (SP-SM), medium dense, brown, dry to slightly moist, fine to coarse sand, ~10% fines (field estimate), trace rootlets	
	10			R-3	9 15 19	117	2	SP-SM	@10': SAND (SP), medium dense, brown, dry to slightly moist, fine to coarse sand, ~5% fines (field estimate), trace rootlets	
3365										
	15			S-1	25 50/6"			SP-SM	@15': SAND (SP), dense, brown, dry to slightly moist, fine to coarse sand, ~5% fines (field estimate), trace rootlets	
3360										
	20			R-4	38 50/4"	118	6	SM	@20': SILTY SAND (SM), dense, light brown, dry to moist, fine to medium sand, trace coarse gravel, ~20% fines (field estimate)	
3355										
	25			S-2	5 13 19			SM	@25': SILTY SAND (SM), dense, light brown, dry to moist, fine sand, ~35% fines (field estimate)	
3350										
	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

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GEOTECHNICAL BORING LOG LB-4

Project No.	13526.001	**Date Drilled**	5-5-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3378'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

SOIL DESCRIPTION

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.

Elevation Feet	Depth Feet	Graphic Log (N / S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			R-5	16 20 31	108	2	SM	@30': SILTY SAND (SM), dense, light brown, dry to moist, fine sand, trace medium, trace fine gravel, ~35% fines (field estimate)	
3345										
	35			S-3	27 50			SM	@35': SILTY SAND (SM), dense, light brown, dry to moist, fine sand, trace medium, trace fine gravel, ~30% fines (field estimate)	
3340										
	40			R-6	10 12 18			SM	@40': SILTY SAND (SM), medium dense, light brown, dry to moist, fine sand, trace medium, trace fine gravel, ~45% fines (field estimate)	
3335										
	45			S-4	9 12 18			SM	@45': SILTY SAND (SM), dense, light brown, dry to moist, fine sand, trace medium, trace fine gravel, ~45% fines (field estimate)	
3330										
	50			R-7	13 19 24			SM	@50': SILTY SAND (SM), medium dense, light brown, dry to moist, fine sand, trace medium, trace fine gravel, ~45% fines (field estimate)	
3325									**TOTAL DEPTH = 51.5 FEET** **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
	55									
3320										
	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
- 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

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GEOTECHNICAL BORING LOG LB-5

Project No.	13526.001	**Date Drilled**	5-5-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3384'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	**SOIL DESCRIPTION** *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.*	Type of Tests
	0			B-1				SM	@Surface: brush, over SILTY SAND (SM)	
				R-1	10 9 8	114	2	SM	@2.5': SILTY SAND (SM), loose, brown, dry to slightly moist, fine sand, trace fine gravel, micaceous, non-plastic, trace rootlets, 25% fines (lab)	-200
3380	5			R-2	6 7 9			SM	@5': SILTY SAND (SM), loose, brown, dry to slightly moist, fine sand, trace fine gravel, micaceous, non-plastic, trace rootlets, ~20% fines (field estimate)	CO
3375				R-3	4 5 7	107	2	SM	@7.5': SILTY SAND (SM), loose, brown, dry to slightly moist, fine sand, trace fine gravel, micaceous, non-plastic, trace rootlets, ~20% fines (field estimate)	
	10			R-4	5 10 15			SP-SM	@10': SAND with silt (SP-SM), medium dense, brown, dry to slightly moist, fine to medium sand, trace fine gravel, trace coarse, ~10% fines (field estimate)	
3370	15			R-5	9 16 32	115	2	SP-SM	@15': SAND with silt (SP-SM), medium dense, brown, dry to slightly moist, medium to coarse sand, trace fine to coarse gravel, ~10% fines (field estimate)	
3365	20			S-1	7 31 39			SM	@20': SILTY SAND (SM), dense, brown, slightly moist to moist, fine sand, trace coarse, trace fine gravel, ~25% fines (field estimate)	
3360	25			R-6	12 50/6"			SM	@25': SILTY SAND (SM), dense, brown, slightly moist to moist, fine sand, trace coarse, trace fine gravel, ~25% fines (field estimate)	
3355	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

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GEOTECHNICAL BORING LOG LB-5

Project No.	13526.001	**Date Drilled**	5-5-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3384'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
	30			S-2	9 11 22			SM	@30': SILTY SAND (SM), dense, brown, slightly moist to moist, fine sand, trace coarse, trace fine gravel, ~25% fines (field estimate)	
									TOTAL DEPTH = 31.5 FEET **NO GROUNDWATER ENCOUNTERED** **BACKFILLED WITH SOIL CUTTINGS TO SURFACE**	
3350	35									
3345	40									
3340	45									
3335	50									
3330	55									
3325	60									

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.

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

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*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG IT-1

Project No.	13526.001	**Date Drilled**	5-5-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3363'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									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.	
	0								@Surface: minor brush, over SILTY SAND (SM)	
3360										
	5			R-1	13 20 26			SM	@5': SILTY SAND (SM), medium dense, light brown, dry, fine sand, trace medium, trace fine gravel, trace rootlets, micaceous, ~35% fines (field estimate)	
3355										
	10									
3350				R-2	10 24 50/4"			SM	@12': SILTY SAND (SM), dense, light brown, dry, fine sand, trace medium, trace fine gravel, trace rootlets, micaceous, ~35% fines (field estimate)	
				R-3	18 50/5"			SM	@13.5': SILTY SAND (SM), dense, light brown, dry, fine to medium sand,trace coarse gravel, trace rootlets, micaceous, ~35% fines (field estimate)	
	15									
3345				R-4	12 28 45			SW-SM	@17': SAND with silt (SW-SM), dense, brown, slightly moist to moist, medium to coarse sand, trace fine, fine to coarse gravel, ~10% fines (field estimate)	
				R-5	10 14 19			SW-SM	@18.5': SAND with silt (SW-SM), medium dense, brown, slightly moist to moist, medium to coarse sand, trace fine, fine to coarse gravel, 10% fines (lab)	SA
	20								**TOTAL DEPTH = 20 FEET** **NO GROUNDWATER ENCOUNTERED** **CONVERTED TO INFILTRATION BORING** **SET WELL @ 20 FT**	
3340										
	25									
3335										
	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

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*** * * This log is a part of a report by Leighton and should not be used as a stand-alone document. * * ***

GEOTECHNICAL BORING LOG IT-2

Project No.	13526.001	**Date Drilled**	5-5-22
Project	Herron Residential Development	**Logged By**	JP
Drilling Co.	BC2 Drilling	**Hole Diameter**	8"
Drilling Method	Autohammer - 140lb - Hollow Stem Auger - 30" Drop	**Ground Elevation**	3365'
Location	See Figure 2- Exploration Location Map	**Sampled By**	JP

Elevation Feet	Depth Feet	Graphic Log (N S)	Attitudes	Sample No.	Blows Per 6 Inches	Dry Density pcf	Moisture Content, %	Soil Class. (U.S.C.S.)	SOIL DESCRIPTION	Type of Tests
									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.	
3365	0								@Surface: minor brush, over SILTY SAND (SM)	
3360	5									
3355	10									
3350	15			R-1	17 50/5"			SM	@15': SILTY SAND (SM), dense, brown, dry, to slightly moist, fine to medium sand, trace coarse, ~25% fines (field estimate)	
				R-2	16 30 50/5"			SP-SM	@17': SAND with silt (SP-SM), dense, brown, dry to slightly moist, medium to coarse sand, trace fine, fine gravel, ~10% fines (field estimate)	
3345	20			R-3	15 22 31			SM	@20': SILTY SAND (SM), dense, brown, dry to slightly moist, fine to medium sand, trace coarse, 16% fines (lab)	SA
				R-4	28 50/4"			SM	@21.5': SILTY SAND (SM), dense, brown, dry to slightly moist, fine to medium sand, trace coarse, 25-30% fines (field estimate)	
									TOTAL DEPTH = 23 FEET **NO GROUNDWATER ENCOUNTERED** **CONVERTED TO INFILTRATION BORING** **SET WELL @ 20 FT**	
3340	25									
3335	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

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Results of Well Permeameter, from USBR 7300-89 Method

Leighton 60 Anniversary

Project: 13526

Exploration #/Location:	IT-1	Initial estimated Depth to Water Surface (in.):	222
Depth Boring drilled, bgs (ft):	20	Average depth of water in well, "h" (in.):	18
Tested by:	JP	approx. h/r:	4.6
USCS Soil Type in test zone:	SW-SM	Tu (Fig. 8) (ft):	283.5
Weather (start to finish):	Sunny, Windy	Tu>3h?:	yes, OK
Water Source/pH:	H2O		
Measured boring diameter:	8 in.	4 in. Well Radius	
Depth to GW or aquitard, bgs:	302 ft		

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity	0.3
Casing outer diameter, in.	2.3
Casing inner diameter, in.	2.1
Cross-sectional area, in.^2	17.3

Well Prep: Drilled to 20ft, set 2in well with #3 sand around anulus to 14.5ft bgs. Screened from 15 to 20ft

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or gr	20. ft	0. in.	240
Casing stickup measured above top of auger (or ground su	0. ft	0. in.	0
Depth to top of sand from top of casing	14.5 ft	0. in.	

Depth of well bottom below top of casing (in): 240

Use of Barrels:	Yes
Use of Flow Meter:	No
Test Type:	Constant Head

Barrel Data: Diameter (in.): 22.5   # of Supply barrels: 1   Total Area of barrels (in.^2): 397

Field Data / **Calculations**

Date	Time	Water Level in Supply Barrel (in.)	Depth to WL in Boring (measured from top of casing) ft	in.	Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	from Δh	Total	Flow (in^3/ min)	q, Flow (in^3/ hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	K20, Coef. Of Perme-ability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date	Start time:																				
5/6/2022	10:30																				
5/6/22	10:57	27.5	19.3					27	231.6	8.4											
5/6/22	10:59	27	19.4				2	29	232.8	7.2	-1.2	8	199	21	219	110	6584	246	0.9	15.09	24.64
5/6/22	11:01	26.75	19.4				2	31	232.8	7.2	0	7	99	0	99	50	2981	231	0.9	6.43	11.88
5/6/22	11:06	25.625	19.4				5	36	232.8	7.2	0	7	447	0	447	89	5365	231	0.9	11.57	21.39
5/6/22	11:11	24.375	19.5				5	41	234.0	6.0	-1.2	7	497	21	518	104	6210	216	0.9	18.02	26.49
5/6/22	11:16	23.125	19.4				5	46	232.8	7.2	1.2	7	497	-21	476	95	5712	216	0.9	11.51	24.36
5/6/22	11:21	21.875	19.26				5	51	231.1	8.9	1.68	8	497	-29	468	94	5612	252	0.9	8.65	20.50
5/6/22	11:26	20.5	19.25				5	56	231.0	9.0	0.12	9	546	-2	544	109	6532	275	0.9	10.60	21.90
5/6/22	11:31	19.125	19.25				5	61	231.0	9.0	0	9	546	0	546	109	6557	276	0.9	10.69	21.87
5/6/22	11:36	17.815	19.15				5	66	229.8	10.2	1.2	10	521	-21	500	100	5998	292	0.9	7.97	18.97
5/6/22	11:41	16.5	18				5	71	216.0	24.0	13.8	17	523	-239	284	57	3407	480	0.9	1.19	6.54
5/6/22	11:46	14.75	17.9				5	76	214.8	25.2	1.2	25	695	-21	675	135	8096	669	0.9	3.12	11.16
5/6/22	11:51	13.25	18				5	81	216.0	24.0	-1.2	25	596	21	617	123	7402	669	0.9	3.14	10.21
5/6/22	11:56	11.5	18.05				5	86	216.6	23.4	-0.6	24	695	10	706	141	8470	646	0.9	3.71	12.09
5/6/22	12:01	10	18.1				5	91	217.2	22.8	-0.6	23	596	10	606	121	7278	631	0.9	3.31	10.64
5/6/22	12:11	7.5	18.05				10	101	216.6	23.4	0.6	23	994	-10	983	98	5899	631	0.9	2.55	8.62
5/6/22	12:21	4.25	17.9				10	111	214.8	25.2	1.8	24	1292	-31	1260	126	7563	661	0.9	2.89	10.55
						Swtich drums															
5/6/22	12:25	28.125	16.6					115	199.2	40.8											
5/6/22	12:27	26	16.61				2	117	199.3	40.7	-0.12	41	844	2	847	423	25397	1074	0.9	4.75	21.80
						Adjusted flow															
5/6/22	12:35	19.375	16.65					125	199.8	40.2											
5/6/22	12:40	15.625	16.7				5	130	200.4	39.6	-0.6	40	1490	10	1501	300	18008	1053	0.9	3.52	15.76
5/6/22	12:50	7	16.8				10	140	201.6	38.4	-1.2	39	3428	21	3448	345	20690	1030	0.9	4.26	18.51
																			Minimum Rate:		6.54
													Raw Rate for design, prior to application of adjustment factors:								**14.9**

Results of Well Permeameter, from USBR 7300-89 Method

Project: 13526
Exploration #/Location: IT-2
Depth Boring drilled, bgs (ft): 23
Tested by: JP
USCS Soil Type in test zone: SM
Weather (start to finish): Sunny, Windy
Water Source/pH: H2O
Measured boring diameter: 8 in. 4 in. Well Radius
Depth to GW or aquitard, bgs: 302 ft
Well Prep: Drilled to 23 ft bgs backfilled and set well at 20ft bgs. 2" screen pipe bottom 5 feet, #3 sand around anulus to 13'bgs

Initial estimated Depth to Water Surface (in.): 201
Average depth of water in well, "h" (in.): 39
approx. h/r: 9.9
Tu (Fig. 8) (ft): 285.3
Tu>3h?: yes, OK

Cross-sectional area for flow calcs based on Δh
Well pack sand porosity 0.3
Casing outer diameter, in. 2.3
Casing inner diameter, in. 2.1
Cross-sectional area, in.^2 17.3

Use of Barrels: Yes
Use of Flow Meter: No
Test Type: Constant Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or gr	20. ft	0. in.	240
Casing stickup measured above top of auger (or ground su	0. ft	0. in.	0
Depth to top of sand from top of casing	13. ft	0. in.	

Depth of well bottom below top of casing (in): 240

Barrel Data: Diameter (in.): 22.5 # of Supply barrels: 1 Total Area of barrels (in.^2): 397

Field Data | Calculations

Date	Time	Water Level in Supply Barrel (in.)	Depth to WL in Boring (measured from top of casing) ft	in.	Water Temp (deg F)	Refilled? (or Comments)	Δt (min)	Total Elapsed Time (min)	Depth to WL in well (in.)	h, Height of Water in Well (in.)	Δh (in.)	Avg. h	Vol Change (in.^3) from supply	from Δh	Total	Flow (in^3/min)	q, Flow (in^3/hr)	Average Infiltration Surface Area, (in^2)	V (Fig 9)	K20, Coef. Of Permeability at 20 deg C (in./hr)	Infiltration Rate [flow/surf area] (in./hr) (FS=1)
Start Date 5/6/2022	Start time: 12:00																				
5/6/22	12:04	19.625	17.5					4	210.0	30.0											
5/6/22	12:06	14.875	17.4				2	6	208.8	31.2	1.2	31	1888	-21	1867	933	56008	819	0.9	15.65	63.02
5/6/22	12:08	10.125	17.25				2	8	207.0	33.0	1.8	32	1888	-31	1857	928	55696	857	0.9	14.23	59.91
5/6/22	12:10	6.5	17.2				2	10	206.4	33.6	0.6	33	1441	-10	1430	715	42907	887	0.9	10.76	44.58
						Switched Drums															
5/6/22	12:16	26.125	18.6					16	223.2	16.8											
5/6/22	12:18	25	19.25				2	18	231.0	9.0	-7.8	13	447	135	582	291	17460	374	0.9	36.38	42.98
5/6/22	12:20	24.625	19.6				2	20	235.2	4.8	-4.2	7	149	73	222	111	6650	224	0.9	31.22	27.41
						Adjusted flow															
5/6/22	12:33	13.5	13.5					33	162.0	78.0											
5/6/22	12:37	6.375	6.375				4	37	76.5	163.5	85.5	121	2832	-1479	1353	338	20290	3085	0.9	0.35	6.06
																				Minimum Rate:	6.06
													Raw Rate for design, prior to application of adjustment factors:								**35.00**

APPENDIX C

LABORATORY TEST RESULTS

APPENDIX C

GEOTECHNICAL LABORATORY TESTING

The geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of physical and mechanical properties of soils underlying the site and to aid in soil classification.

In Situ Moisture and Density: The natural water content (ASTM D 2216) and in-situ dry density (ASTM D 2937) was determined for recovered relatively undisturbed ring-lined barrel drive samples, from our subsurface explorations. Results of these tests are shown on the logs at the appropriate sample depths, in Appendix B.

Modified Proctor Compaction Curve (MD): In accordance with ASTM Test Method D1557, a modified Proctor laboratory compaction curve was established for a shallow bulk soil sample, to determine the laboratory maximum dry density and optimum moisture content compaction curve. Results are plotted on the following "*Modified Proctor Compaction Test*" sheet in this appendix.

Collapse Potential: Collapse potential tests were performed on selected soil samples in general accordance with ASTM Standard Test Method D 4546. Test results are presented on the "*One Dimensional Swell or Settlement*" figures.

Expansion Index: Expansion Index of a representative bulk sample was determined by the ASTM D 4829 standard test method to identify expansion potential. The expansion index is presented in this appendix.

Percent Fines (Percentage Passing No. 200 Sieve, -200): Selected soil samples were wet-washed through a No. 200 U.S. Standard brass sieve in accordance with ASTM Test Methods D1140 to measure percent fines (silts and clays). This data was used to refine the Unified Soil Classification for tested soil samples. Test results are tabulated in this appendix and listed on boring logs in Appendix A.

Atterberg Limits (AL): Liquid Limit (LL), Plastic Limit (PL) and Plasticity Index (PI) were determined for soil samples in accordance with ASTM D4318 Standard Test Method. Soil samples were air-dried and passed through a No. 40 sieve, then re-moisturized. Liquid Limit and Plastic Limit tests were performed on fraction of soil samples passing the No. 40 sieve. Results of these tests are presented on the "*Atterberg Limits*" sheet in this appendix.

Corrosivity Tests: To evaluate the corrosion potential of the subsurface soils at the site, we tested a representative bulk sample collected during our subsurface investigation for pH, resistivity and soluble sulfate and chloride content testing. Results of these tests are presented at the end of this appendix.

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Leighton

Project Name:	Herron Victorville	Tested By:	J. Gonzalez	Date:	05/25/22
Project No.:	13526.001	Checked By:	A. Santos	Date:	06/02/22
Boring No.:	LB-3	Depth (ft.):	0-5		
Sample No.:	B-1				
Soil Identification:	Brown silty sand (SM)				

Preparation Method: [X] Moist [ ] Dry   [X] 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)	3883	4002	3951			
Weight of Mold (g)	1826	1826	1826			
Net Weight of Soil (g)	2057	2176	2125			
Wet Weight of Soil + Cont. (g)	484.0	431.2	540.6			
Dry Weight of Soil + Cont. (g)	464.7	404.3	497.2			
Weight of Container (g)	37.3	37.8	39.2			
Moisture Content (%)	4.52	7.34	9.48			
Wet Density (pcf)	136.2	144.1	140.7			
Dry Density (pcf)	130.3	134.2	128.5			

Maximum Dry Density (pcf) 134.5   **Optimum Moisture Content (%)** 6.8

PROCEDURE USED

[X] **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

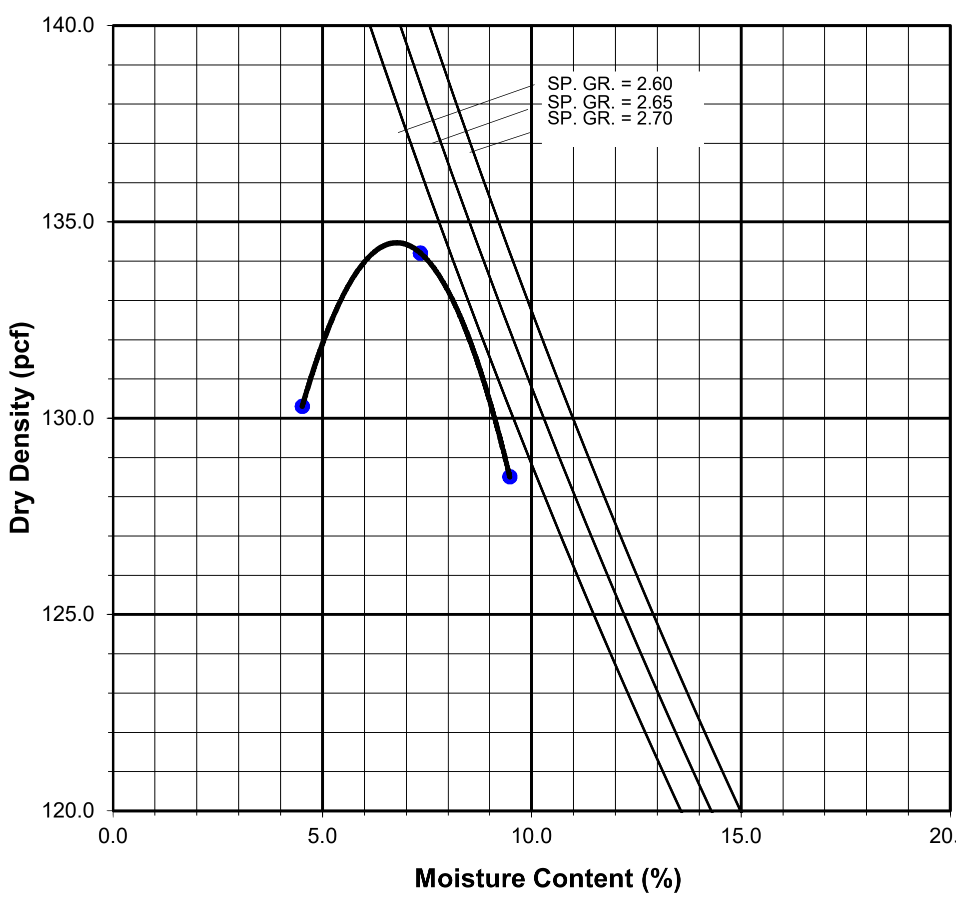
[ ] **Procedure C**
Soil Passing 3/4 in. (19.0 mm) Sieve
Mold : 6 in. (152.4 mm) diameter
Layers : 5 (Five)
Blows per layer : 56 (fifty-six)
Use if +3/8 in. is >20% and +¾ in. is <30%

Particle-Size Distribution:

GR:SA:FI

Atterberg Limits:

LL,PL,PI



Boring No.	LB-1	LB-2	LB-5						
Sample No.	R-4	R-2	R-1						
Depth (ft.)	10.0	5.0	2.5						
Sample Type	Ring	Ring	Ring						
Soil Identification	Brown poorly-graded sand with silt (SP-SM)	Brown poorly-graded sand with silt and gravel (SP-SM)g	Brown silty sand (SM)						
Moisture Correction									
Wet Weight of Soil + Container (g)	0.00	0.00	0.00						
Dry Weight of Soil + Container (g)	0.00	0.00	0.00						
Weight of Container (g)	1.00	1.00	1.00						
Moisture Content (%)	0.00	0.00	0.00						
Sample Dry Weight Determination									
Weight of Sample + Container (g)	922.30	1007.70	804.70						
Weight of Container (g)	110.00	159.60	108.50						
Weight of Dry Sample (g)	812.30	848.10	696.20						
Container No.:									
After Wash									
Method (A or B)	A	A	A						
Dry Weight of Sample + Cont. (g)	844.10	908.80	629.70						
Weight of Container (g)	110.00	159.60	108.50						
Dry Weight of Sample (g)	734.10	749.20	521.20						
% Passing No. 200 Sieve	**9.6**	**11.7**	**25.1**						
% Retained No. 200 Sieve	90.4	88.3	74.9						

Leighton

PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Herron Victorville
Project No.: 13526.001
Tested By: S. Felter Date: 05/26/22

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Project Name: Herron Victorville
Project No.: 13526.001
Boring No.: IT-1
Sample No.: R-5
Soil Identification: Brown well-graded sand with silt (SW-SM)

Tested By: S. Felter Date: 05/26/22
Checked By: A. Santos Date: 06/01/22
Depth (feet): 18.5

		Moisture Content of Total Air - Dry Soil	
Container No.:	9554	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	936.2	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	107.9	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	828.3	Moisture Content (%)	0.0

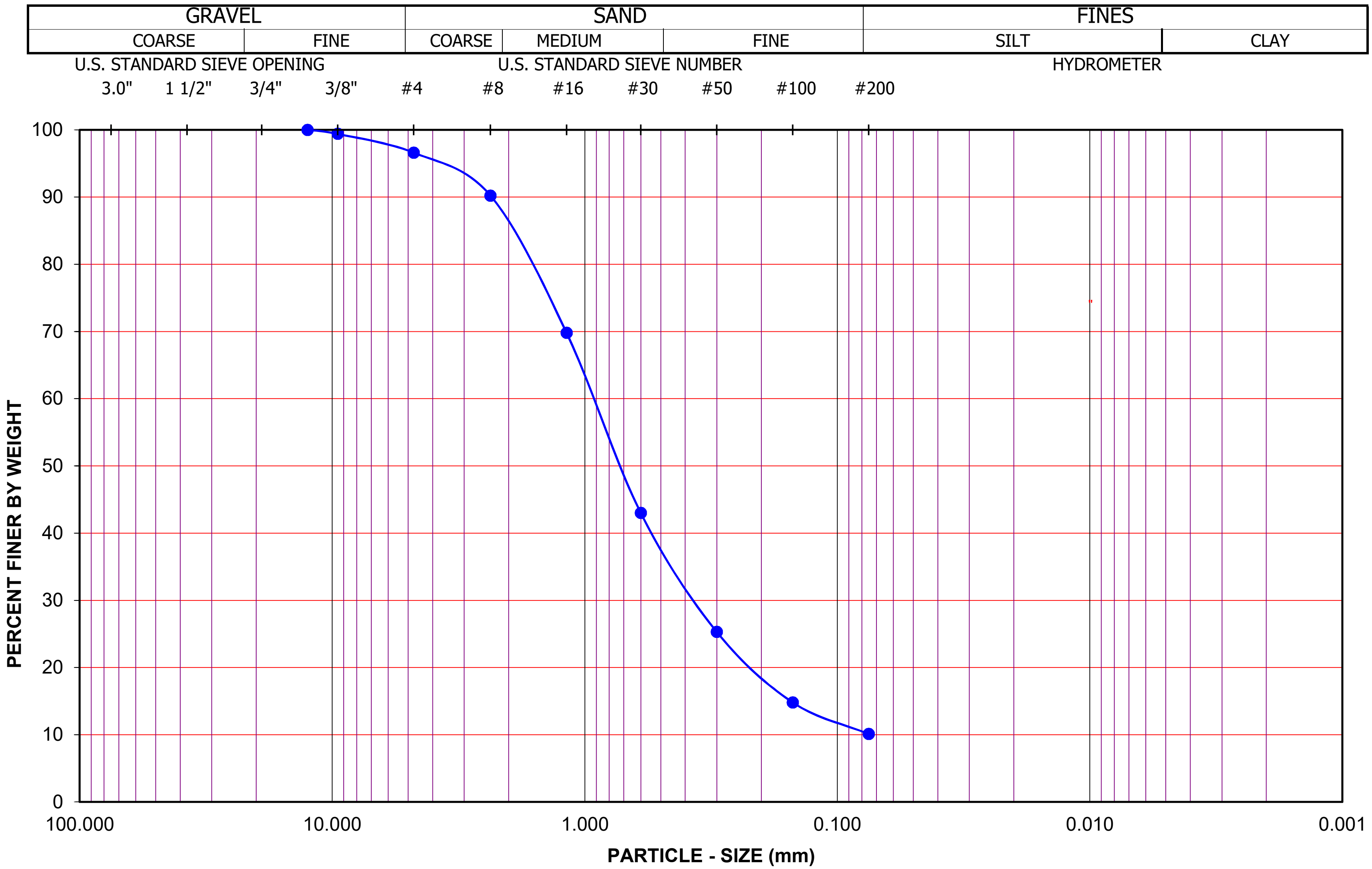
After Wet Sieve	Container No.	9554
	Wt. of Dry Soil + Container (g)	856.1
	Wt. of Container (g)	107.9
	Dry Wt. of Soil Retained on # 200 Sieve (g)	748.2

U. S. Sieve Size		Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
(in.)	(mm.)		
1 1/2"	37.5		
1"	25.0		
3/4"	19.0		
1/2"	12.5	0.0	100.0
3/8"	9.5	4.9	99.4
#4	4.75	28.1	96.6
#8	2.36	81.1	90.2
#16	1.18	250.2	69.8
#30	0.600	472.3	43.0
#50	0.300	619.0	25.3
#100	0.150	705.3	14.8
#200	0.075	744.7	10.1
PAN			

GRAVEL: **3 %**
SAND: **87 %**
FINES: **10 %**
GROUP SYMBOL: **SW-SM**

$Cu = D60/D10 =$ 13.14
$Cc = (D30)^2/(D60*D10) =$ 2.13

Remarks:



Project Name: Herron Victorville

Project No.: 13526.001

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: IT-1 Sample No.: R-5

Depth (feet): 18.5 Soil Type : SW-SM

Soil Identification: Brown well-graded sand with silt (SW-SM)

GR:SA:FI : (%) 3 : 87 : 10

Jun-22

Leighton

PARTICLE-SIZE DISTRIBUTION (GRADATION) of SOILS USING SIEVE ANALYSIS

ASTM D 6913

Project Name:	Herron Victorville	Tested By:	S. Felter Date: 05/26/22
Project No.:	13526.001	Checked By:	A. Santos Date: 06/01/22
Boring No.:	IT-2	Depth (feet):	20.0
Sample No.:	R-3		
Soil Identification:	Brown silty sand (SM)		

		Moisture Content of Total Air - Dry Soil	
Container No.:	979	Wt. of Air-Dry Soil + Cont. (g)	0.0
Wt. of Air-Dried Soil + Cont.(g)	959.2	Wt. of Dry Soil + Cont. (g)	0.0
Wt. of Container (g)	111.0	Wt. of Container No.______ (g)	1.0
Dry Wt. of Soil (g)	848.2	Moisture Content (%)	0.0

After Wet Sieve	Container No.	979
	Wt. of Dry Soil + Container (g)	832.3
	Wt. of Container (g)	111.0
	Dry Wt. of Soil Retained on # 200 Sieve (g)	721.3

U. S. Sieve Size (in.)	U. S. Sieve Size (mm.)	Cumulative Weight Dry Soil Retained (g)	Percent Passing (%)
1 1/2"	37.5		
1"	25.0	0.0	100.0
3/4"	19.0	19.1	97.7
1/2"	12.5	51.3	94.0
3/8"	9.5	67.4	92.1
#4	4.75	96.0	88.7
#8	2.36	143.2	83.1
#16	1.18	219.9	74.1
#30	0.600	346.2	59.2
#50	0.300	506.7	40.3
#100	0.150	641.7	24.3
#200	0.075	716.9	15.5
PAN			

GRAVEL: **11 %**

SAND: **73 %**

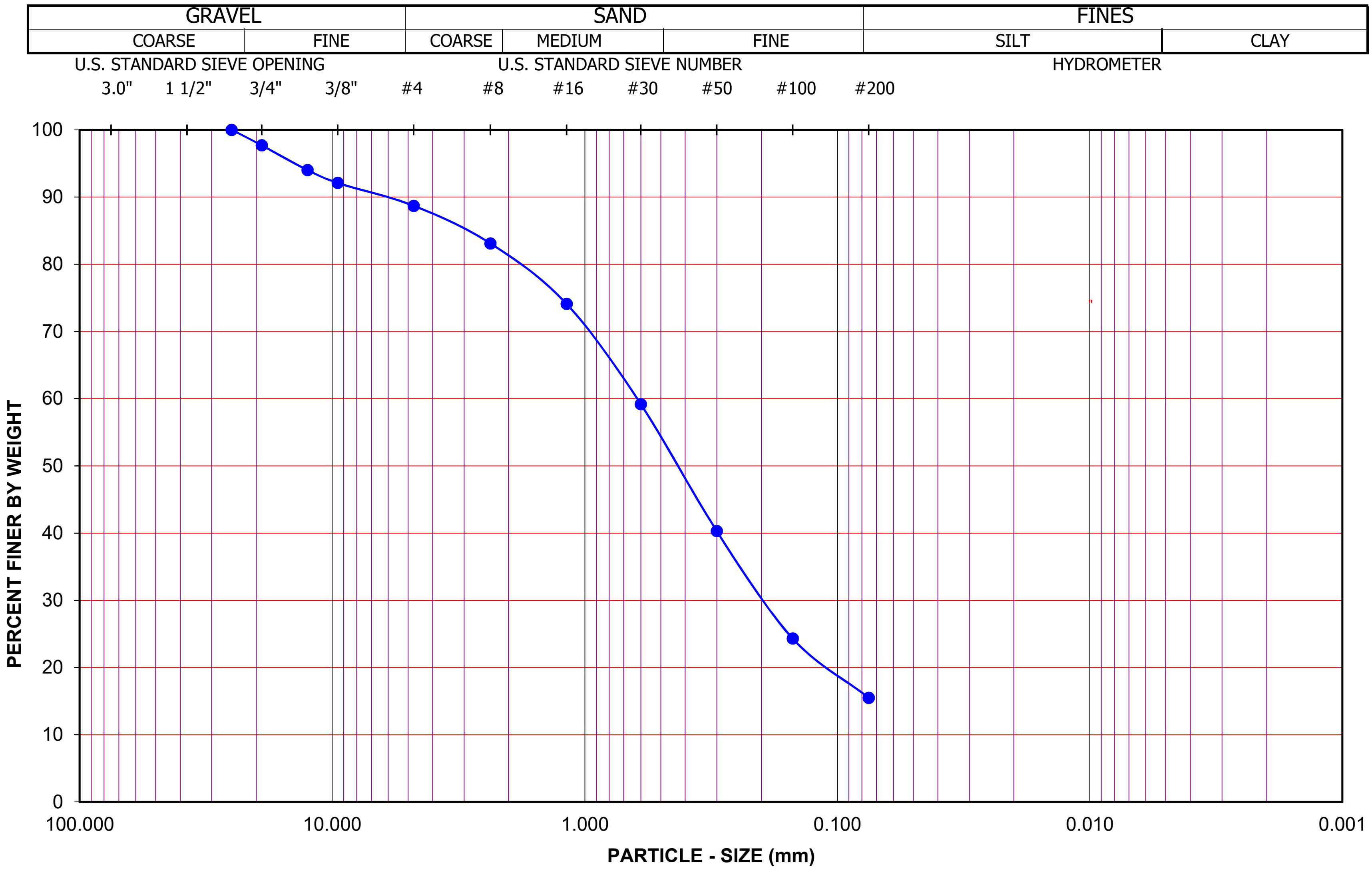
FINES: **16 %**

GROUP SYMBOL: **SM**

Cu = D60/D10 = ______

Cc = (D30)²/(D60*D10) = ______

Remarks: ______



Project Name: Herron Victorville

Project No.: 13526.001

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Boring No.: IT-2

Sample No.: R-3

Depth (feet): 20.0

Soil Type : SM

Soil Identification: Brown silty sand (SM)

GR:SA:FI : (%) 11 : 73 : 16

Jun-22

PARTICLE-SIZE ANALYSIS OF SOILS

ASTM D 7928 & D 6913

Leighton

Project Name: Herron Victorville
Project No.: 13526.001
Boring No.: LB-3
Sample No.: B-1
Soil Identification: Brown silty sand (SM)

Tested By: J. Domingo — Date: 05/27/22
Checked By: A. Santos — Date: 06/05/22
Depth (feet): 0-5

		Soil Type
% Gravel	**4**	**SM**
% Sand	**68**	
% Fines	**28**	

			Moisture Content of Total Air-Dry Soil	Moisture Content of Air-Dry Soil Passing #10	After Hydrometer & Wet Sieve ret. in #200 Sieve
Specific Gravity (Assumed)	2.70	Wt.of Air-Dry Soil + Cont.(g)	0.00	70.55	
Correction for Specific Gravity	0.99	Dry Wt. of Soil + Cont. (g)	0.00	70.45	149.69
Wt.of Air-Dry Soil + Cont. (g)	2519.26	Wt. of Container No.___ (g)	1.00	1.00	74.74
Wt. of Container	226.29	Moisture Content (%)	0.00	0.14	
Dry Wt. of Soil (g)	2292.97	Wt. of Dry Soil (g)			74.95

Coarse Sieve

U.S. Sieve	Cumulative Wt. Of Dry Soil Retained (g)	% Passing
3"	0.00	100.0
1½"	0.00	100.0
3/4"	0.00	100.0
3/8"	30.62	98.7
No. 4	100.05	95.6
No. 10	272.38	88.1
Pan		

Sieve after Hydrometer & Wet Sieve

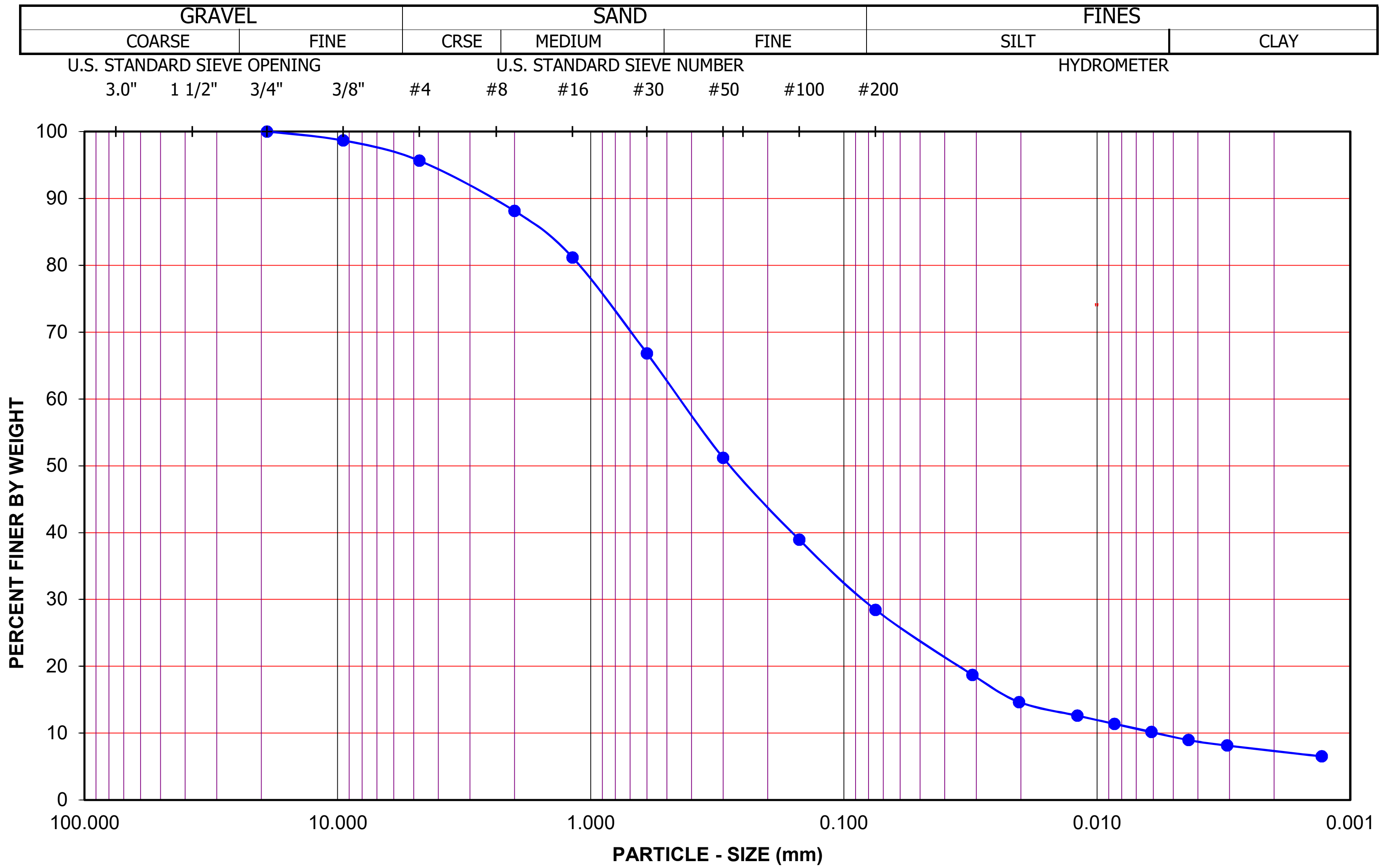
U.S. Sieve Size	Cumulative Wt. Of Dry Soil Retained (g)	% Passing	% Total Sample
No. 10	0.00	100.0	88.1
No. 16	8.48	92.1	81.2
No. 30	25.97	75.8	66.8
No. 50	45.02	58.1	51.2
No. 100	59.96	44.2	38.9
No. 200	72.75	32.3	28.4
Pan			

Hydrometer

Wt. of Air-Dry Soil (g): 107.58 — Wt. of Dry Soil (g): 107.43

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)
31-May-22	7:18	0		8.0			
	7:20	2	22.5	8.0	31.0	18.7	0.0311
	7:23	5	22.5	8.0	26.0	14.6	0.0203
	7:33	15	22.5	8.0	23.5	12.6	0.0120
	7:48	30	22.4	8.0	22.0	11.4	0.0085
	8:18	60	22.3	8.0	20.5	10.2	0.0061
	9:18	120	22.3	8.0	19.0	9.0	0.0044
	11:28	250	21.9	8.0	18.0	8.1	0.0031
01-Jun-22	7:18	1440	21.4	8.0	16.0	6.5	0.0013



Project Name: Herron Victorville

Project No.: 13526.001

Boring No.: LB-3 Sample No.: B-1

Depth (feet): 0-5 Soil Type : SM

Soil Identification: Brown silty sand (SM)

Leighton

PARTICLE - SIZE DISTRIBUTION ASTM D 7928 & D 6913

GR:SA:FI : (%) 4 : 68 : 28

Jun-22

S&H LB-3, B-1 @ 0-5

Leighton

EXPANSION INDEX of SOILS

ASTM D 4829

Project Name:	Herron Victorville	Tested By:	G. Berdy	Date:	06/01/22
Project No.:	13526.001	Checked By:	A. Santos	Date:	06/05/22
Boring No.:	LB-3	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	0.9995
Wt. Comp. Soil + Mold (g)	647.10	462.90
Wt. of Mold (g)	202.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	889.00	665.00
Dry Wt. of Soil + Cont. (g)	837.10	621.12
Wt. of Container (g)	0.00	202.10
Moisture Content (%)	6.20	10.47
Wet Density (pcf)	134.2	139.7
Dry Density (pcf)	126.4	126.5
Void Ratio	0.334	0.333
Total Porosity	0.250	0.250
Pore Volume (cc)	51.8	51.7
Degree of Saturation (%) [S_{meas}]	**50.2**	84.9

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.)
06/01/22	14:00	1.0	0	0.5630
06/01/22	14:10	1.0	10	0.5625
Add Distilled Water to the Specimen				
06/01/22	14:30	1.0	20	0.5625
06/02/22	5:36	1.0	926	0.5625
06/02/22	8:36	1.0	1106	0.5625

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**0**

ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

ASTM D 4546

Project Name: Herron Victorville
Project No.: 13526.001
Boring No.: LB-1
Sample No.: R-3
Sample Description: Light olive brown silty sand (SM)

Tested By: G. Bathala Date: 05/27/22
Checked By: A. Santos Date: 06/01/22
Sample Type: Ring
Depth (ft.) 7.5

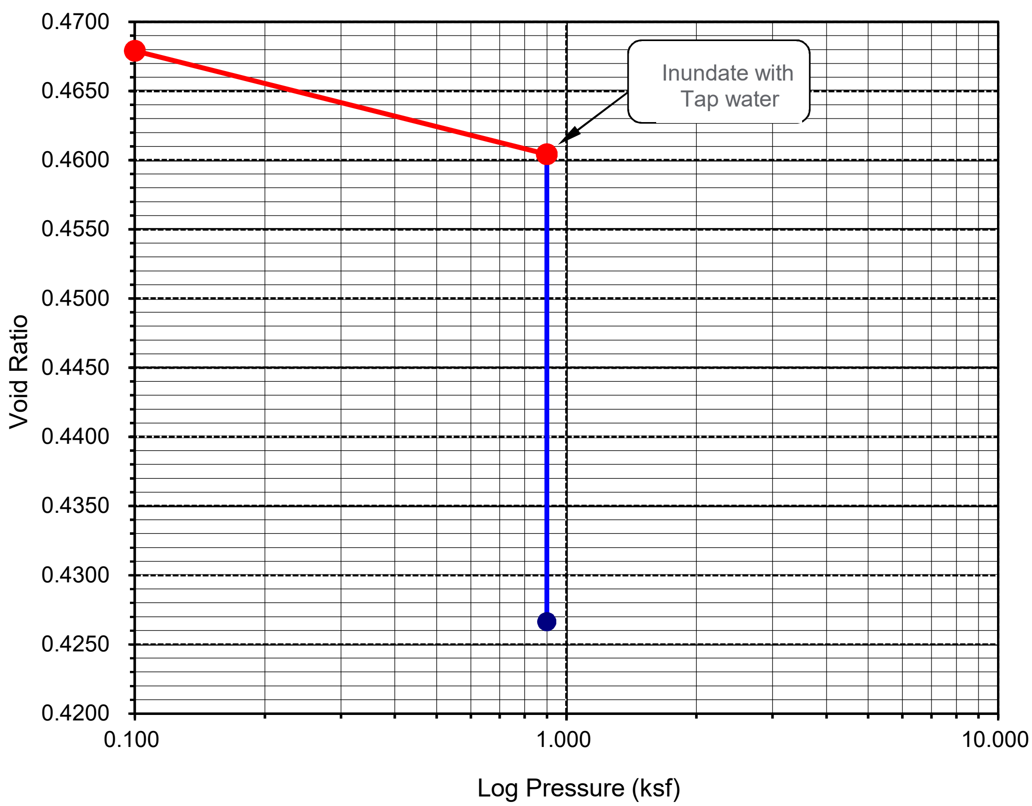
Initial Dry Density (pcf):	114.8
Initial Moisture (%):	1.46
Initial Length (in.):	1.0000
Initial Dial Reading:	0.0837
Diameter(in):	2.415

Final Dry Density (pcf):	119.1
Final Moisture (%) :	11.4
Initial Void ratio:	0.4684
Specific Gravity(assumed):	2.70
Initial Saturation (%)	8.4

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.0840	0.9997	0.00	-0.03	0.4679	-0.03
0.900	0.0930	0.9907	0.39	-0.93	0.4604	-0.54
H_2O	0.1160	0.9677	0.39	-3.23	0.4267	-2.84

Percent Swell (+) / Settlement (-) After Inundation = -2.31

Void Ratio - Log Pressure Curve



ONE-DIMENSIONAL SWELL OR SETTLEMENT POTENTIAL OF COHESIVE SOILS

ASTM D 4546

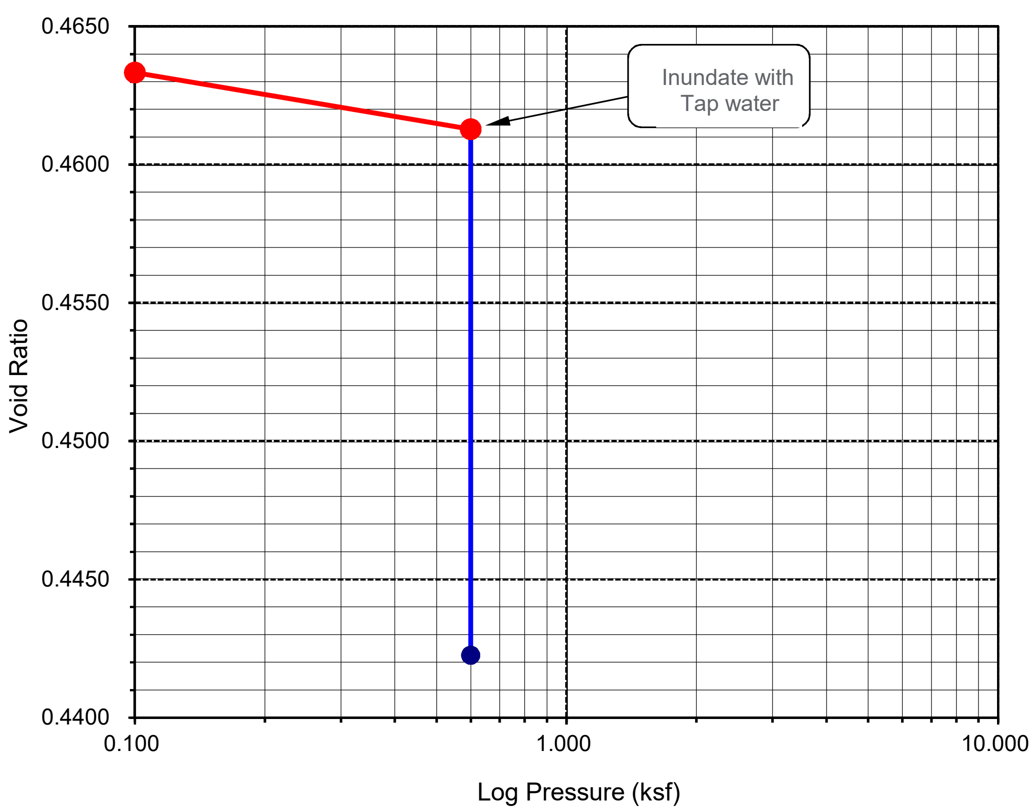
Project Name:	Herron Victorville	Tested By:	G. Bathala	Date:	05/27/22
Project No.:	13526.001	Checked By:	A. Santos	Date:	06/01/22
Boring No.:	LB-5	Sample Type:	Ring		
Sample No.:	R-2	Depth (ft.)	5.0		
Sample Description:	Yellowish brown silty sand (SM)				

Initial Dry Density (pcf):	115.2	Final Dry Density (pcf):	117.3
Initial Moisture (%):	2.21	Final Moisture (%) :	12.5
Initial Length (in.):	1.0000	Initial Void ratio:	0.4638
Initial Dial Reading:	0.0970	Specific Gravity(assumed):	2.70
Diameter(in):	2.415	Initial Saturation (%)	12.8

Pressure (p) (ksf)	Final Reading (in)	Apparent Thickness (in)	Load Compliance (%)	Swell (+) Settlement (-) % of Sample Thickness	Void Ratio	Corrected Deformation (%)
0.100	0.0973	0.9997	0.00	-0.03	0.4633	-0.03
0.600	0.1003	0.9967	0.16	-0.33	0.4613	-0.17
H2O	0.1133	0.9837	0.16	-1.63	0.4423	-1.47

Percent Swell (+) / Settlement (-) After Inundation = -1.30

Void Ratio - Log Pressure Curve



TESTS for SULFATE CONTENT CHLORIDE CONTENT and pH of SOILS

Project Name: Herron Victorville — Tested By : G. Berdy — Date: 05/26/22

Project No. : 13526.001 — Checked By: A. Santos — Date: 06/05/22

Boring No.	LB-2			
Sample No.	B-1			
Sample Depth (ft)	0-5			
Soil Identification:	Yellowish brown (SM)			
Wet Weight of Soil + Container (g)	203.58			
Dry Weight of Soil + Container (g)	203.34			
Weight of Container (g)	62.97			
Moisture Content (%)	0.17			
Weight of Soaked Soil (g)	100.27			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	402			
Crucible No.	2			
Furnace Temperature (°C)	860			
Time In / Time Out	7:00/7:45			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	22.4966			
Wt. of Crucible (g)	22.4929			
Wt. of Residue (g) (A)	0.0037			
PPM of Sulfate (A) x 41150	152.26			
PPM of Sulfate, Dry Weight Basis	**153**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	15			
ml of AgNO3 Soln. Used in Titration (C)	0.4			
PPM of Chloride (C -0.2) * 100 * 30 / B	40			
PPM of Chloride, Dry Wt. Basis	**40**			

pH TEST, DOT California Test 643

pH Value	**6.78**			
Temperature °C	21.2			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Herron Victorville	Tested By :	J. Domingo Date: 06/02/22
Project No. :	13526.001	Checked By:	A. Santos Date: 06/05/22
Boring No.:	LB-2	Depth (ft.) :	0-5
Sample No. :	B-1		

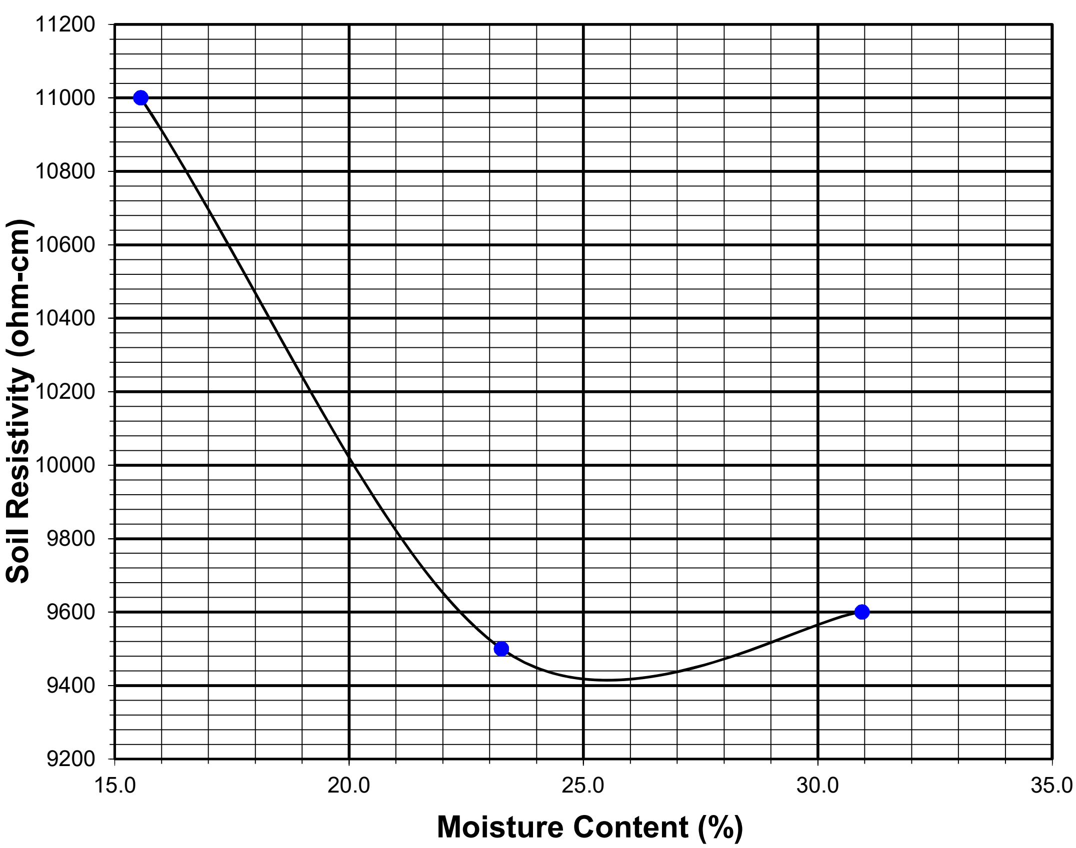
Soil Identification:* Yellowish brown (SM)

*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	11000	11000
2	30	23.25	9500	9500
3	40	30.95	9600	9600
4				
5				

Moisture Content (%) (MCi)	0.17
Wet Wt. of Soil + Cont. (g)	203.58
Dry Wt. of Soil + Cont. (g)	203.34
Wt. of Container (g)	62.97
Container No.	
Initial Soil Wt. (g) (Wt)	130.20
Box Constant	1.000
MC =(((1+Mci/100)x(Wa/Wt+1))-1)x100	

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	
9410	**25.5**	**153**	**40**	**6.78**	**21.2**



APPENDIX D

SEISMIC

Liquefaction Susceptibility Analysis: SPT Method

Leighton

Youd and Idriss (2001), Martin and Lew (1999)

Description: Herron Victorville; Case 1; PGAm 0.55; design GW 302; No overex 0
Project No.: 13526.001
May 2022

General Boring Information:

Boring No.	Existing GW Depth (ft)	Design GW Depth (ft)	Design Fill Height (ft)	Overex. depth bgs (ft)	Ground Surface Elev (ft)	design gw elve
LB-1	118	302		0	3366	3064
LB-2	118	302		0	3371	3069
LB-3	118	302		0	3375.5	3073.5
LB-4	118	302		0	3378	3076
LB-5	118	302		0	3383.5	3081.5
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0
						0

Boring Location Coordinates X (ft)	Y (ft)
128.59	-147.9
350.53	-432.9
176.6	-708.3
512.93	-925.3
142.59	-1161

General Parameters:

a_{max} = 0.55g
M_W = 8.1
MSF eq: 1
MSF = 0.82
Hammer Efficiency = 84
C_E = 1.40
C_B = 1
C_S for SPT? TRUE
Unlined, but room for liner
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).

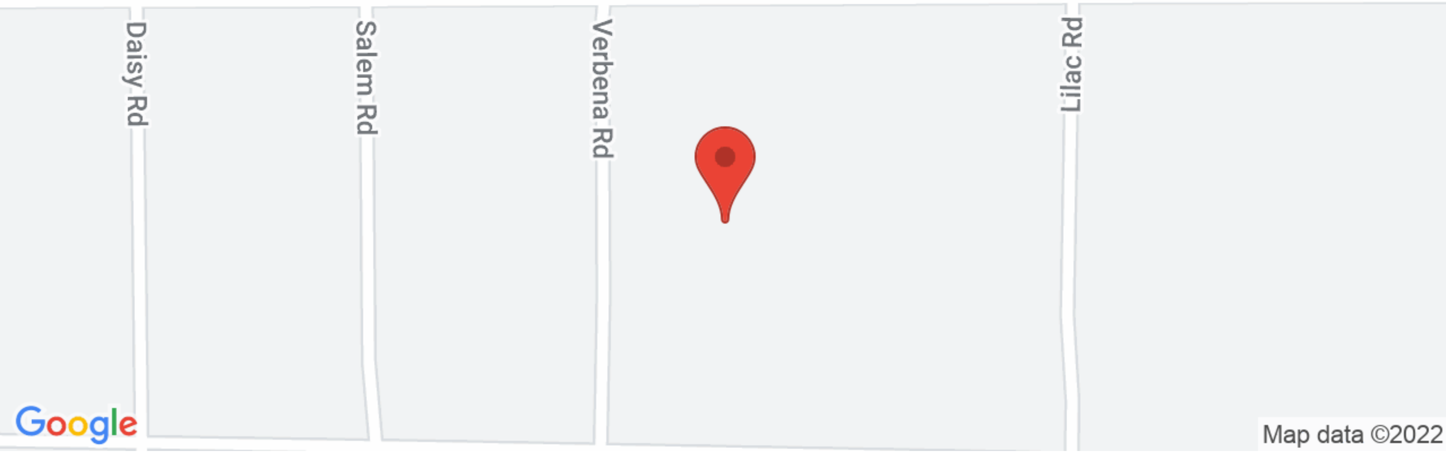
Project: Herron Victorville; Case 1; PGAm 0.55; design GW 302; No overex 0

Project No.: 13526.001

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (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)	Cs	N_m (corrected for Cs 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 Settle-ment) (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 3.8	2.5	3.8		25	120	41	2	1	26.7	300	47.6	57.3	>Range	300	0.36	0.43	**NonLiq**	57.3	0.01		0.00	0.7
LB-1	3.8 to 6.3	5	2.5		25	120	57	2	1	37.1	600	66.1	78.0	>Range	600	0.35	0.43	**NonLiq**	78.0	0.01		0.00	0.6
LB-1	6.3 to 8.8	7.5	2.5		25	120	13	2	1	8.5	900	14.4	20.4	0.220	900	0.35	0.43	**NonLiq**	20.4	0.14		0.04	0.6
LB-1	8.8 to 12.5	10	3.8		**10**	120	25	2	1	16.3	1200	25.5	26.9	0.336	1200	0.35	0.43	**NonLiq**	26.9	0.16		0.07	0.6
LB-1	12.5 to 17.5	15	5.0		10	120	96	2	1	62.4	1800	80.0	82.6	>Range	1800	0.34	0.42	**NonLiq**	82.6	0.01		0.01	0.5
LB-1	17.5 to 22.5	20	5.0		10	120	100	1	1.3	130.0	2400	161.3	165.6	>Range	2400	0.34	0.42	**NonLiq**	165.6	0.01		0.01	0.5
LB-1	22.5 to 27.5	25	5.0		10	120	59	2	1	38.4	3000	42.6	44.3	>Range	3000	0.34	0.41	**NonLiq**	44.3	0.03		0.02	0.5
LB-1	27.5 to 32.5	30	5.0		10	120	21	1	1.29	27.0	3600	28.8	30.3	>Range	3600	0.33	0.41	**NonLiq**	30.3	0.08		0.05	0.5
LB-1	32.5 to 37.5	35	5.0		10	120	27	2	1	17.6	4200	17.3	18.6	0.198	4200	0.32	0.39	**NonLiq**	18.6	0.38		0.23	0.4
LB-1	37.5 to 42.5	40	5.0		10	120	36	1	1.3	46.8	4800	43.2	45.0	>Range	4800	0.30	0.37	**NonLiq**	45.0	0.02		0.01	0.2
LB-1	42.5 to 47.5	45	5.0		10	120	50	2	1	32.5	5400	28.3	29.8	0.453	5400	0.29	0.35	**NonLiq**	29.8	0.17		0.10	0.2
LB-1	47.5 to 52.0	50	4.5		25	120	19	1	1.19	22.5	6000	18.6	25.0	0.293	6000	0.27	0.33	**NonLiq**	25.0	0.20		0.11	0.1
LB-2	0 to 3.8	2.5	3.8		25	120	39	2	1	25.4	300	45.2	54.7	>Range	300	0.36	0.43	**NonLiq**	54.7	0.01		0.00	0.1
LB-2	3.8 to 6.3	5	2.5		**12**	120	21	2	1	13.7	600	24.4	26.7	0.330	600	0.35	0.43	**NonLiq**	26.7	0.15		0.04	0.1
LB-2	6.3 to 8.8	7.5	2.5		25	120	45	2	1	29.3	900	49.9	59.9	>Range	900	0.35	0.43	**NonLiq**	59.9	0.01		0.00	0.1
LB-2	8.8 to 12.5	10	3.8		10	120	30	2	1	19.5	1200	30.6	32.1	>Range	1200	0.35	0.43	**NonLiq**	32.1	0.08		0.03	0.1
LB-2	12.5 to 17.5	15	5.0		10	120	37	1	1.3	48.1	1800	61.6	63.9	>Range	1800	0.34	0.42	**NonLiq**	63.9	0.01		0.01	0.0
LB-2	17.5 to 22.5	20	5.0		10	120	100	2	1	65.0	2400	80.6	83.2	>Range	2400	0.34	0.42	**NonLiq**	83.2	0.01		0.01	0.0
LB-2	22.5 to 27.5	25	5.0		10	120	42	1	1.3	54.6	3000	60.6	62.8	>Range	3000	0.34	0.41	**NonLiq**	62.8	0.02		0.01	0.0
LB-2	27.5 to 32.0	30	4.5		10	120	100	2	1	65.0	3600	69.3	71.7	>Range	3600	0.33	0.41	**NonLiq**	71.7	0.02		0.01	0.0
LB-3	0 to 3.8	2.5	3.8		**28**	120	30	2	1	19.5	300	34.8	44.2	>Range	300	0.36	0.43	**NonLiq**	44.2	0.01		0.00	0.0
LB-3	3.8 to 6.3	5	2.5		28	120	40	2	1	26.0	600	46.4	57.4	>Range	600	0.35	0.43	**NonLiq**	57.4	0.01		0.00	0.0
LB-3	6.3 to 8.8	7.5	2.5		28	120	46	2	1	29.9	900	51.0	62.6	>Range	900	0.35	0.43	**NonLiq**	62.6	0.01		0.00	0.0
LB-3	8.8 to 12.5	10	3.8		10	120	52	2	1	33.8	1200	53.1	55.1	>Range	1200	0.35	0.43	**NonLiq**	55.1	0.02		0.01	0.0
LB-3	12.5 to 17.5	15	5.0		10	120	84	2	1	54.6	1800	70.0	72.4	>Range	1800	0.34	0.42	**NonLiq**	72.4	0.01		0.01	0.0
LB-3	17.5 to 22.0	20	4.5		10	120	30	1	1.3	39.0	2400	48.4	50.3	>Range	2400	0.34	0.42	**NonLiq**	50.3	0.02		0.01	0.0
LB-4	0 to 3.8	2.5	3.8		30	120	38	2	1	24.7	300	44.1	55.6	>Range	300	0.36	0.43	**NonLiq**	55.6	0.01		0.00	0.3
LB-4	3.8 to 6.3	5	2.5		10	120	17	2	1	11.1	600	19.7	21.0	0.229	600	0.35	0.43	**NonLiq**	21.0	0.20		0.06	0.3
LB-4	6.3 to 8.8	7.5	2.5		10	120	46	2	1	29.9	900	51.0	53.0	>Range	900	0.35	0.43	**NonLiq**	53.0	0.01		0.00	0.3
LB-4	8.8 to 12.5	10	3.8		5	120	34	2	1	22.1	1200	34.7	34.7	>Range	1200	0.35	0.43	**NonLiq**	34.7	0.07		0.03	0.3
LB-4	12.5 to 17.5	15	5.0		5	120	100	1	1.3	130.0	1800	166.6	166.6	>Range	1800	0.34	0.42	**NonLiq**	166.6	0.01		0.00	0.2
LB-4	17.5 to 22.5	20	5.0		20	120	100	2	1	65.0	2400	80.6	90.7	>Range	2400	0.34	0.42	**NonLiq**	90.7	0.01		0.01	0.2
LB-4	22.5 to 27.5	25	5.0		35	120	32	1	1.3	41.6	3000	46.2	60.4	>Range	3000	0.34	0.41	**NonLiq**	60.4	0.02		0.01	0.2
LB-4	27.5 to 32.5	30	5.0		35	120	51	2	1	33.2	3600	35.3	47.4	>Range	3600	0.33	0.41	**NonLiq**	47.4	0.02		0.01	0.2

Boring No.	Approx. Layer Depth (ft)	SPT Depth (ft)	Approx Layer Thick-ness (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)	Cs	N_m (corrected for Cs 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 Settle-ment) (blows/ft)	Dry Sand Strain (%) (Tok/ Seed 87) (%)	Sat Sand Strain (%) (Tok/ Seed 87) (%)	Seismic Sett. of Layer (in.)	**Cummulative Seismic Settlement** (in.)
LB-4	32.5 to 37.5	35	5.0		30	120	100	1	1.3	130.0	4200	128.3	152.8	>Range	4200	0.32	0.39	**NonLiq**	152.8	0.01		0.01	0.2
LB-4	37.5 to 42.5	40	5.0		45	120	30	2	1	19.5	4800	18.0	26.6	0.328	4800	0.30	0.37	**NonLiq**	26.6	0.18		0.11	0.2
LB-4	42.5 to 47.5	45	5.0		45	120	30	1	1.3	39.0	5400	34.0	45.7	>Range	5400	0.29	0.35	**NonLiq**	45.7	0.02		0.01	0.1
LB-4	47.5 to 52.0	50	4.5		45	120	43	2	1	28.0	6000	23.1	32.7	>Range	6000	0.27	0.33	**NonLiq**	32.7	0.09		0.05	0.0
LB-5	0 to 3.8	2.5	3.8		**25**	120	17	2	1	11.1	300	19.7	26.3	0.320	300	0.36	0.43	**NonLiq**	26.3	0.05		0.02	0.3
LB-5	3.8 to 6.3	5	2.5		20	120	16	2	1	10.4	600	18.6	23.7	0.267	600	0.35	0.43	**NonLiq**	23.7	0.17		0.05	0.2
LB-5	6.3 to 8.8	7.5	2.5		20	120	12	2	1	7.8	900	13.3	18.0	0.192	900	0.35	0.43	**NonLiq**	18.0	0.26		0.08	0.2
LB-5	8.8 to 12.5	10	3.8		10	120	25	2	1	16.3	1200	25.5	26.9	0.336	1200	0.35	0.43	**NonLiq**	26.9	0.16		0.07	0.1
LB-5	12.5 to 17.5	15	5.0		10	120	48	2	1	31.2	1800	40.0	41.7	>Range	1800	0.34	0.42	**NonLiq**	41.7	0.02		0.01	0.0
LB-5	17.5 to 22.5	20	5.0		25	120	70	1	1.3	91.0	2400	112.9	130.2	>Range	2400	0.34	0.42	**NonLiq**	130.2	0.01		0.01	0.0
LB-5	22.5 to 27.5	25	5.0		25	120	100	2	1	65.0	3000	72.1	84.7	>Range	3000	0.34	0.41	**NonLiq**	84.7	0.02		0.01	0.0
LB-5	27.5 to 32.0	30	4.5		25	120	33	1	1.3	42.9	3600	45.7	55.3	>Range	3600	0.33	0.41	**NonLiq**	55.3	0.02		0.01	0.0

Latitude, Longitude: 34.4685, -117.4247



Date	5/11/2022, 4:17:52 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	D - Stiff Soil

Type	Value	Description
S_S	1.423	MCE_R ground motion. (for 0.2 second period)
S_1	0.553	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.423	Site-modified spectral acceleration value
S_{M1}	null -See Section 11.4.8	Site-modified spectral acceleration value
S_{DS}	0.949	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	0.5	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.55	Site modified peak ground acceleration
T_L	12	Long-period transition period in seconds
SsRT	1.423	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.533	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.5	Factored deterministic acceleration value. (0.2 second)
S1RT	0.553	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.609	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.6	Factored deterministic acceleration value. (1.0 second)
PGAd	0.5	Factored deterministic acceleration value. (Peak Ground Acceleration)
C_{RS}	0.928	Mapped value of the risk coefficient at short periods
C_{R1}	0.908	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 (u...

Latitude

Decimal degrees

34.4685

Longitude

Decimal degrees, negative values for western longitudes

-117.4247

Site Class

259 m/s (Site class D)

Spectral Period

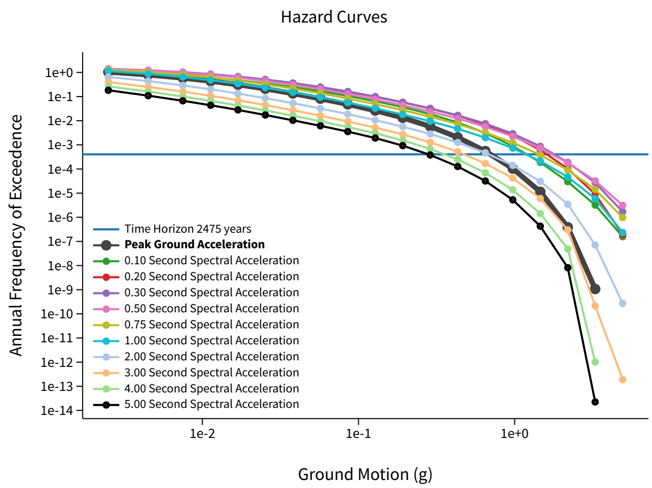
Peak Ground Acceleration

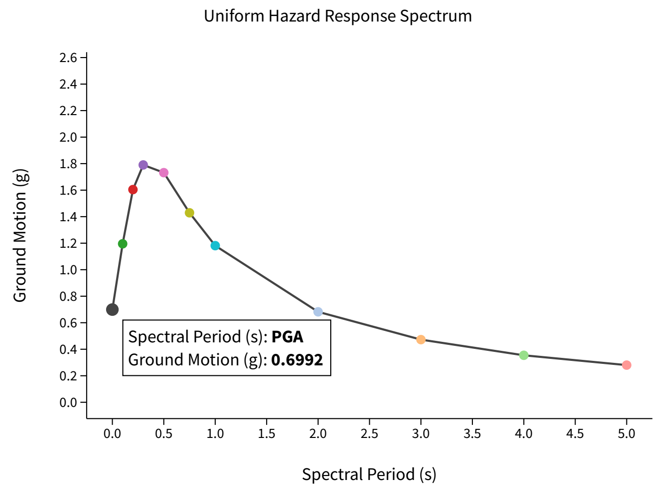
Time Horizon

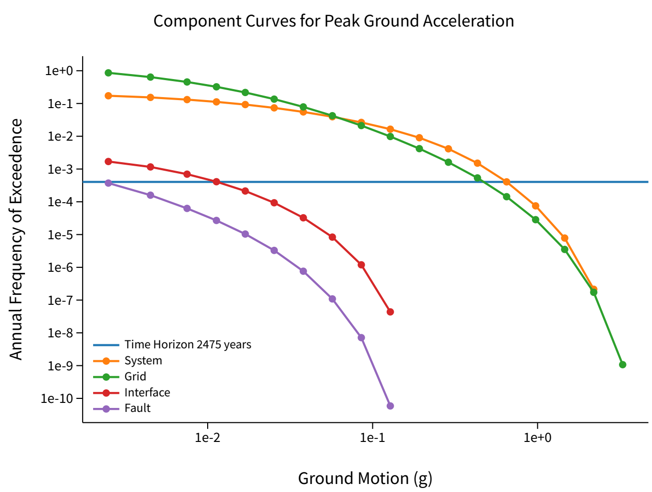
Return period in years

2475

Hazard Curve





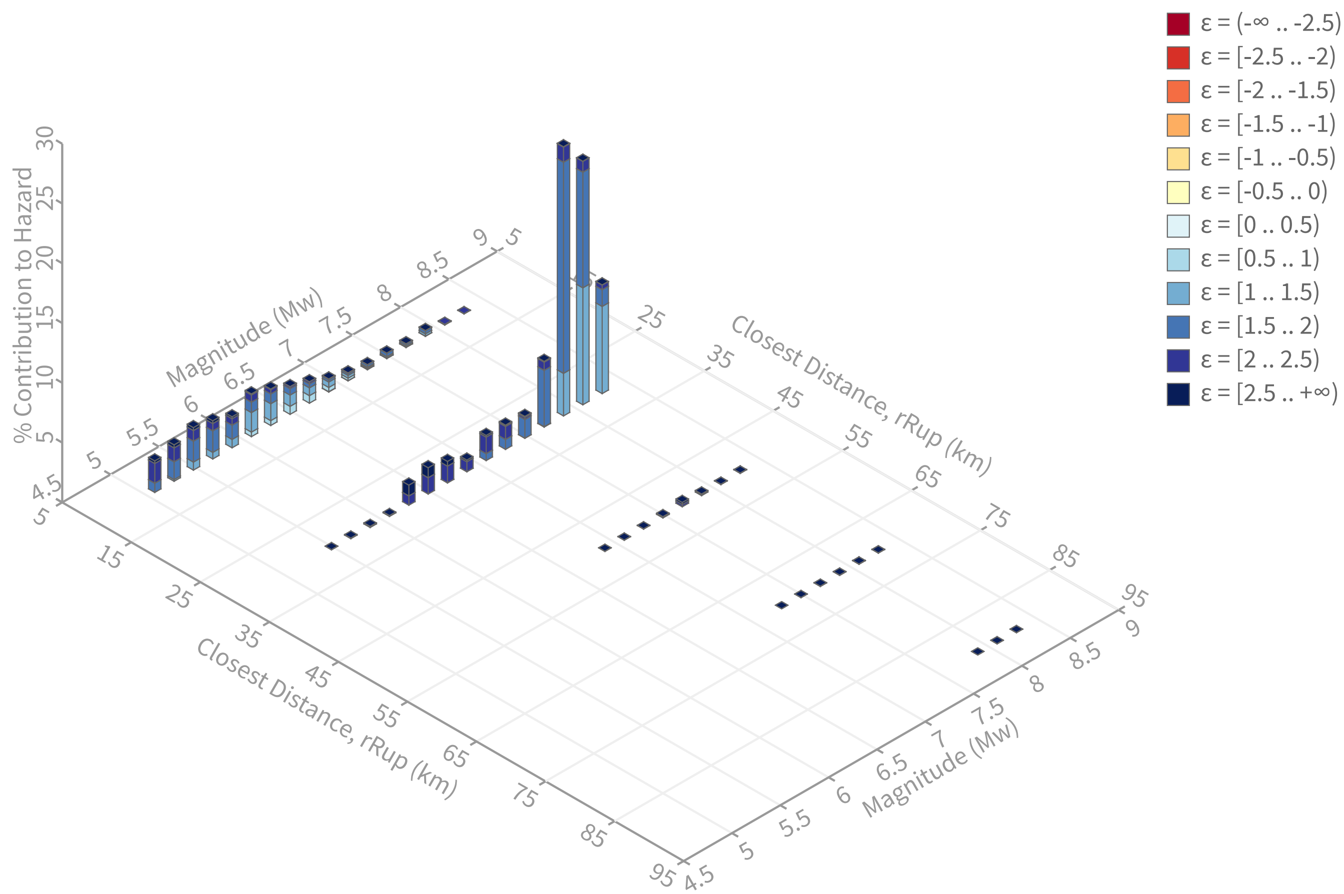


[View Raw Data]()

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.69917613 g

Recovered targets

Return period: 3110.3139 yrs
Exceedance rate: 0.00032151095 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.08 %

Mean (over all sources)

m: 7.29
r: 17.85 km
ε_0: 1.7 σ

Mode (largest m-r bin)

m: 7.9
r: 20.99 km
ε_0: 1.66 σ
Contribution: 22.51 %

Mode (largest m-r-ε_0 bin)

m: 7.89
r: 20.86 km
ε_0: 1.65 σ
Contribution: 17.66 %

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							36.11
San Andreas (San Bernardino N) [0]		20.34	7.96	1.61	117.530°W	34.308°N	208.49	27.22
San Andreas (Mojave S) [14]		20.41	7.11	2.10	117.549°W	34.316°N	214.01	2.14
Cucamonga [0]		23.74	7.83	1.69	117.445°W	34.192°N	183.41	1.22
San Andreas (Mojave S) [13]		22.14	7.11	2.18	117.612°W	34.343°N	231.00	1.14
San Andreas (San Bernardino N) [1]		20.77	7.25	2.02	117.493°W	34.290°N	197.60	1.04
UC33brAvg_FM31	System							36.04
San Andreas (San Bernardino N) [0]		20.34	7.96	1.61	117.530°W	34.308°N	208.49	27.10
San Andreas (Mojave S) [14]		20.41	7.09	2.12	117.549°W	34.316°N	214.01	2.18
San Andreas (Mojave S) [13]		22.14	7.09	2.19	117.612°W	34.343°N	231.00	1.16
Cucamonga [0]		23.74	7.82	1.69	117.445°W	34.192°N	183.41	1.13
San Andreas (San Bernardino N) [1]		20.77	7.25	2.01	117.493°W	34.290°N	197.60	1.08
UC33brAvg_FM31 (opt)	Grid							13.93
PointSourceFinite: -117.425, 34.536		8.44	5.85	1.63	117.425°W	34.536°N	0.00	2.55
PointSourceFinite: -117.425, 34.536		8.44	5.85	1.63	117.425°W	34.536°N	0.00	2.55
PointSourceFinite: -117.425, 34.509		6.52	5.81	1.36	117.425°W	34.509°N	0.00	2.11
PointSourceFinite: -117.425, 34.509		6.52	5.81	1.36	117.425°W	34.509°N	0.00	2.11
UC33brAvg_FM32 (opt)	Grid							13.92
PointSourceFinite: -117.425, 34.536		8.44	5.85	1.63	117.425°W	34.536°N	0.00	2.54
PointSourceFinite: -117.425, 34.536		8.44	5.85	1.63	117.425°W	34.536°N	0.00	2.54
PointSourceFinite: -117.425, 34.509		6.52	5.81	1.36	117.425°W	34.509°N	0.00	2.11
PointSourceFinite: -117.425, 34.509		6.52	5.81	1.36	117.425°W	34.509°N	0.00	2.10

APPENDIX E

GENERAL EARTHWORK AND GRADING SPECIFICATIONS

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

Table of Contents

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 D1557-91).

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 D1557-91). 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 sheepsfoot 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 D1557-91.

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 OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 Bedding and Backfill: All bedding and backfill of utility trenches shall be done 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 maximum 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.