Appendix D-1

Geotechnical Exploration and Infiltration Testing, Proposed Crescent Gardens Cemetery, East of the Intersection of Garfield Street and Roberts Street
March 11, 2024
prepared by Leighton, Inc

GEOTECHNICAL EXPLORATION AND INFILTRATION TESTING, PROPOSED CRESCENT GARDENS CEMETERY EAST OF THE INTERSECTION OF GARFIELD STREET AND ROBERTS STREET ASSESSOR'S PARCEL NUMBERS (APN'S) 349-170-003 THROUGH -013, -035, 349-180-002, -004 THROUGH -008, -016, -017, -020, AND 349-190-006 PERRIS AREA OF UNINCORPORATED RIVERSIDE COUNTY, CALIFORNIA

Prepared For

MUSLIM MORTUARY & CEMETERY COMMITTEE
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Project Number 038.0000019894

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March 11, 2024

Project No. 038.00000019894

Muslim Mortuary & Cemetery Committee
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Attention: Hamid Haque, President

Subject: Geotechnical Exploration and Infiltration Testing
Proposed Crescent Gardens Cemetery
East of the Intersection of Garfield Street and Roberts Street
Assessor's Parcel Numbers (APN's) 349-170-003 through -013, -035, 349-180-002, -004 through -008, -016, -017, -020, and 349-190-006
Perris Area of Unincorporated Riverside County, California

In accordance with your request, Leighton Consulting, Inc. (Leighton) has prepared this geotechnical exploration report for the Crescent Gardens Cemetery development to be located within the combined 84.4-acre site of Assessor's Parcel Numbers (APNs) 349-170-003 though -013, -035, 349-180-002, -004 through -008, -016, -017, -020, 349-190-006, in the Perris area of unincorporated Riverside County, California. The purpose of this exploration has been to evaluate the general geotechnical conditions of the site with respect to the proposed development and to provide geotechnical recommendations for design and construction of the project.

Based on our geotechnical exploration, construction of the proposed Crescent Gardens Cemetery 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, compressible soils, and shallow, resistant bedrock. Good

planning and design of the project can limit the impact of these constraints. This report presents our findings, conclusions, and geotechnical recommendations for this project.

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

Respectfully submitted,

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1.0 INTRODUCTION

1.1 Site Location and Description

This 84.4-acre project site is made up of Assessor's Parcel Numbers (APN's) 349-170-003 through -013, -035, 349-180-002, -004, through -008, -016, -017, -020, and 349-190-006 in the Perris area of unincorporated Riverside County, California. This project site is located east of the intersection of Garfield Street and Roberts Street. The site is bounded by residential homes to the west and the southwest, to the north by Pechanga Tribe property, and to the south and east by undeveloped and vacant land.

Currently, the site is undeveloped with scattered outcrops of bedrock within and surrounding the site. Based on our review of historic aerial imagery, the site has been historically vacant and undeveloped since the earliest available aerial imagery of the site dated 1967 (NETR, 2024). Based on review of available topographic maps and the elevation model from Google Earth, elevations within the project site range from approximately 1,550 feet above mean sea level (msl) to the north to approximately 1,669 feet msl to the southeast. The site generally drains toward the north and west from the base of the hillsides separating the site from the San Jacinto River. We also understand based on the *Grading Study September 18, 2023 vs. Exist. Topo* prepared by Hunsaker and Associates, Inc., annotated by you to indicate the scope of our exploration, that several surface artifacts associated with past usage of the site from the Pechanga Band of Indians have been mapped by others within the site.

1.2 Proposed Development

Based on the provided *Cresent Gardens – Conceptual Site Plan Exhibit* prepared by Hunsaker and Associates dated September 29, 2023. The project will consist of the construction of four new structures ranging in footprint size from approximately 1,340 to 8,983 square feet located near the entry of the proposed cemetery, which will be near the intersection of Garfield Street and Roberts Street. These proposed structures will consist of a chapel, a mortuary, a caretakers residence, and an equipment shed. Plans also call for grading of the site to accommodate large relatively flat pads for future cemetery plots, interior private roads, parking lots, and several water quality detention basins. Based on the annotated *Grading Study*, we understand that design cuts will be as deep as

approximately 20 feet from the current surface, and design fills will be on the order of approximately 15 feet thick. The deeper design cuts are planned in the southern portion of the site and the thicker design fills are planned in the central portion of the site.

1.3 Previous Work

Based on the project documents provided by you, we understand that two previous geotechnical investigations (EEI, 2006 and Soil Exploration Company Inc., 2021) and one environmental site assessment (EEI, 2005) were conducted for this project and site. These past investigations and site evaluations are summarized below:

EEI (2005), *Phase I Environmental Site Assessment*

EEI previously prepared a *Phase I Environmental Site Assessment* in 2005 (EEI, 2005) for portions of the site that were previously planned for a residential development referred to as "Dowling Estates/Meadow Brook." As part of their assessment, EEI conducted a site reconnaissance and office studies to gather available background information of the site to identify potential recognized environmental concerns. Based on their assessment, EEI did not find any Recognized Environmental Conditions within the property. That study did, however, identify numerous discarded tires and miscellaneous debris throughout the site, and EEI recommended that this debris be properly evaluated by an environmental consultant and discarded, especially within natural drainages within the site.

EEI (2006), *Geotechnical Feasibility Study*

EEI previously prepared a *Geotechnical Feasibility Study* (EEI, 2006) for portions of the site that were previously planned for a residential development referred to as "Dowling Estates/Meadow Brook." EEI's *Geotechnical Feasibility Study* focused on evaluating regional and site-specific information regarding potential geotechnical constraints associated with development of the 130-acre study area, which contained the currently proposed cemetery development. EEI's study included drilling of six (6) exploratory hollow-stem auger borings and 31 test pits. Their borings were advanced to depths ranging from 5 to 22 feet below the ground surface (bgs) and their test pits were excavated throughout the site to depths ranging from 1.5 feet to 6 feet bgs. EEI's feasibility study did not identify

any significant geotechnical issues that would preclude the project at that time from being developed. Bedrock was identified as shallow as ½ foot in hillside areas and between 3 to 10 feet bgs within the lower elevations on that site. Groundwater was encountered within one boring B-2 within a natural drainage area as shallow as 13 feet bgs.

Soil Exploration Company (2021),

RGS Engineering Geology (RGS) previously prepared a *Limited Geologic Feasibility Report* on behalf of the Soil Exploration Company (SEC) in 2021 for the proposed Cresent Cardens Cemetery (SEC, 2021). That limited geologic feasibility study focused on evaluating the existing soils and geologic conditions at the site relative to the proposed development. SEC's report provided findings regarding the rippability and percolation characteristics of the onsite earth materials. The study involved the excavation of fourteen (14) test pits to maximum depths of 10 feet below ground surface (bgs) and conducting infiltration testing throughout the site. RGS concluded that the development proposed at that time was feasible from an engineering standpoint provided their findings, conclusions, and recommendations were considered during design and construction. The proposed development was found to be underlain by shallow granitic and metamorphic bedrock with excavation characteristics identified as "moderate" within the granitic bedrock and "poor" within the metamorphic bedrock. RGS indicated that excavation 6 feet deep or greater was achievable with a rubber tire backhoe in areas with "poor" excavation characteristics. Moderate to poor percolation and infiltrations rates were observed onsite with moderately decomposed bedrock yielding moderate rates. Geologic Hazards were identified to be considered low except for the potential of high seismicity, as is the case for most of southern California.

1.4 Purpose of Investigation

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

Our geotechnical exploration included excavating, logging, and sampling a total of fourteen (14) test pits, infiltration testing, laboratory testing, review of previous geotechnical exploration for this project, and geotechnical analysis to evaluate

existing geotechnical conditions and to develop the conclusions and recommendations contained in this report.

1.5 Scope of Investigation

The scope of our study 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, available online, or those provided by you.

- Utility Coordination: We contacted DigAlert (811) at least 48 hours prior to excavation of test pits to locate major utilities, underground services, and easements. We coordinated our work with you and a site representative from the Pechanga Tribe of Indians.

- Field Exploration: Our field exploration included excavating, logging, and sampling of eight (8) standard test pits (TP-1 through TP-8) at the proposed structure locations, select roadway paths, and other areas throughout the site. The test pits were excavated to depths of approximately 10 feet below ground surface (bgs). Earth materials encountered in the test pits were logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Each test pit was digitally photographed for inclusion in our report. Samples were retained in plastic bags and brought to our in-house geotechnical laboratory.

  Infiltration tests were conducted within six (6) additional test pits (IT-1 through IT-6) to evaluate general infiltration characteristics of subsurface soils at the depths and locations tested. Infiltration tests were conducted in general accordance with the Riverside County Standards. Infiltration tests were generally conducted at the bottom of infiltration test pit excavations with depths ranging from approximately 2 feet to 3.5 feet bgs.

  Our test pit logs are included in Appendix A, *Exploration Logs*. Approximate test pit locations are shown on Figure 2, *Exploration Location Map*.

  Five (5) P-wave seismic refraction survey profiles were conducted along lines SR-1 through SR-5 by a subcontracted geophysical firm, utilizing Geometrics

Geode 24-channel signal enhancement seismographs, 10 Hz vertical geophones, seismic cables with 10-foot spaced connectors, piezo hammer switches, and a 20-lb sledgehammer with an aluminum strike plate. Each seismic line consisted of 24 geophones spaced 10 feet apart for a line length of 230 feet. The purpose of these seismic refraction survey was to estimate density and ribbapility characteristics of the bedrock underlying this project. Approximate seismic refraction line locations are illustrated on Figure 2, *Exploration Location map*. A summary of seismic refraction survey results for this project is presented in Appendix D.

- Geotechnical Laboratory Testing: Geotechnical laboratory tests were conducted on selected bulk soil samples and relatively undisturbed 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:

  - Grain size distribution
  - Expansion Index
  - Maximum Dry Density and Optimum Moisture Content
  - Resistivity, sulfate content, chloride content and pH
  - R-Value Testing

  The laboratory tests results are presented in Appendix B, *Laboratory Test Results*.

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

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

2 .0 FINDINGS

2.1 Regional Geologic Conditions

The site is located within the Peninsular Ranges Geomorphic Province on the southern portion of the Perris Block. The Perris Block is bounded by the Elsinore fault zone to the west, San Jacinto fault zone to the east, the Sierra Madre fault zone to the north, and the San Felipe fault zone to the south and is a relatively unfaulted mass of Mesozoic plutonic rocks of the southern California Batholith and metasedimentary bedrock. The project site is located approximately 4.8 miles to the northeast of the Elsinore fault zone and approximately 15.5 miles to the southwest of the San Jacinto fault zone.

The site has been regionally mapped to be underlain by Cretaceous-age tonalite (granitic) and Triassic-age phyllite (metamorphic) bedrock with mantle of Pleistocene-age old axial channel deposits (alluvium) within the drainage in the southern portion of the project site (Morton & Miller, 2006). The old alluvial deposits have been described to consist of sands, silts, and gravel eroded from the surrounding mountains. Regional geologic conditions are illustrated on Figure 3, *Regional Geology Map*.

2.2 Subsurface Soil Conditions

Based upon our review of pertinent geotechnical literature and our subsurface exploration, the site is generally underlain by a thin mantle of topsoil was uncovered consisting of loose to medium dense sands with clays and silts, silty sands, and clayey sands. Granitic bedrock encountered underlying the topsoil in the northwestern portion of the site and generally consisted of white to gray, with zones of orange to orangish brown, reddish brown and brown, intensely weathered to decomposed tonalite (Kgt). Phyllite (Trmp) bedrock encountered underlying the topsoil in the southeastern portion of the site and generally consisted of gray, with fracture surfaces of orange to orangish brown, moderately to intensely weathered. Granitic and metamorphic bedrock in our test pits were considered to be slightly weathered where excavation refusal by a backhoe was met. We encountered excavation refusal at depths ranging from 1½ to 7½ feet bgs in our test pits, with shallower bedrock generally being encountered towards the eastern end of the site within the rolling hillsides and deeper bedrock being encountered towards the west.

Seismic refraction tomography from this study measured velocities that indicated fresh bedrock (identified as non-rippable with conventional earthmoving equipment) to be as shallow as approximately 20 feet bgs, and as deep as 65 feet bgs within the central portion of the property.

More detailed descriptions of the subsurface material encountered during our current investigation and past evaluations conducted by others are presented on the test pit logs in Appendix A.

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 this study, the upper portion of the thin mantle of topsoils and alluvial soils are considered moderately compressible. Removal and recompaction of compressible topsoil and alluvial soils beneath the proposed improvements are 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 Leighton's recommendation to remove potentially compressible residual soils onsite and replace with compacted fill to design grades and the presence of shallow bedrock, the hydro-collapse potential is considered negligible.

2.2.2 Expansive Soils

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

Based on our laboratory test results from our current exploration and past evaluations at the site (Leighton, 2024 and EEI, 2006), onsite near-surface soils are expected to have a very low expansion potential.

2.2.3 Sulfate Content

Water-soluble sulfates in soil can react adversely with concrete. However, concrete in contact with soil containing sulfate concentrations of less than 0.1 percent by weight is considered to have negligible sulfate exposure based on American Concrete Institute (ACI) provisions, adopted by the 2022 CBC (CBC, 2022, Chapter 19, and ACI, 2019).

A near-surface soil sample was tested for soluble sulfate content for this study. That test result and those performed for a previous study (EEI, 2006) indicated sulfate contents of less than 0.1 percent by weight, indicating negligible sulfate content (Exposure Class S0). Recommendations for concrete in contact with the soil are provided in Section 3.11.

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. In general, soil having a minimum resistivity less than 1,000 ohm-cm is considered severely corrosive. Soil with a chloride content of 500 parts-per-million (ppm) or more is considered corrosive to ferrous metals.

As a screening for potentially corrosive soil, a representative soil sample was tested during our investigation to determine minimum resistivity, chloride content, and pH. This test and those performed for a previous study (EEI, 2006) indicated a minimum resistivity of 2,850 ohm-cm, a chloride content as much as 30 ppm, and pH as acidic as 6.8. Based on these results, the onsite soil is considered moderately corrosive to ferrous metals.

2.3 Groundwater

Groundwater was not encountered in any of our test pits excavated to a maximum total depth of 7.5 feet bgs during our investigation. Groundwater data from nearby State Well Number 05S03W32L001S located approximately 3.8 miles southeast from the site located within regionally mapped alluvial soils was reviewed to include readings from April 25, 1995 through October 10, 2023.

These data indicated the shallowest groundwater was on the order of approximately 1403 feet above msl, which corelated to a depth of approximately 147 feet bgs from the lowest elevation of the site (CDWR, 2024). However, the site is underlain by shallow granitic and metamorphic bedrock, which are not considered a water-bearing earth unit.

During EEI's 2006 evaluation of the site (EEI, 2006) groundwater also was not encountered within any of their borings or test pits, with the exception of hollow stem-auger boring (B-2), which was located towards the southern end of the site within the natural drainage area. This logged groundwater was found at a depth of 13 feet bgs within that boring as was described as being perched on top of bedrock. Other areas may experience perched groundwater on top of bedrock, depending on precipitation.

2.5 Faulting and Seismicity

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

2.5.1 Surface Faulting

The State of California and Riverside County have mapped the site to be outside of any state designated fault zones (CGS, 2021 and Riverside County, 2020). Based on these maps and our research, no active faults are known to have been mapped on or trending toward the site. The closest mapped active or potentially active faults are presented in the following table.

Fault Name	Approximate Distance from Site
Elsinore fault	4.6 miles to the southwest
San Jacinto Fault	14.8 miles to the northeast

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

2.5.2 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 2022 CBC:

2022 CBC Parameters (CBC or ASCE 7-16 reference)	Value 2022 CBC
Site Latitude and Longitude (degrees): 33.7169, -117.2814	
Site Class Definition (1613.2.2, ASCE 7-16 Ch 20)	B
Mapped Spectral Response Acceleration at 0.2s Period (1613.2.1), S_s	1.570 g
Mapped Spectral Response Acceleration at 1s Period (1613.2.1), S_1	0.584 g
Short Period Site Coefficient at 0.2s Period (T1613.2.3(1)), F_a	0.900 g
Long Period Site Coefficient at 1s Period (T1613.2.3(2)), F_v	0.800 g
Adjusted Spectral Response Acceleration at 0.2s Period (1613.2.3), S_{MS}	1.413 g
Adjusted Spectral Response Acceleration at 1s Period (1613.2.3), S_{M1}	0.467 g
Design Spectral Response Acceleration at 0.2s Period (1613.2.4), S_{DS}	0.942 g
Design Spectral Response Acceleration at 1s Period (1613.2.4), S_{D1}	0.312 g
Mapped *MCE*$_G$ peak ground acceleration (11.8.3.2, Fig 22-9 to 13), ***PGA***	0.667 g
Site Coefficient for Mapped *MCE*$_G$ *PGA* (11.8.3.2), F_{PGA}	0.900
Site-Modified Peak Ground Acceleration (1803.5.12; 11.8.3.2), PGA_M	0.600 g

Hazard deaggregation was estimated using the USGS Interactive Deaggregations utility. The results of this analysis indicate that the predominant modal earthquake has a magnitude of approximately 6.5

(M_W) at a distance on the order of 11.6 kilometers for the Maximum Considered Earthquake (2% probability of exceedance in 50 years), and corresponding peak ground acceleration of 0.55 g.

2.6 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.

2.6.1 Liquefaction Potential

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

The State of California has not evaluated the site for liquefaction hazards. However, Riverside County has mapped the bottom of the natural drainage in the southern portion of the site to be within a low zone of liquefaction susceptibility. Groundwater was not encountered during any of our test pits conducted to a maximum depth of about 7.5 feet below ground surface. Previous geotechnical exploration (EEI, 2006) included borings that extended to a maximum depth of 22 feet bgs indicated perched groundwater in one boring (B-2) at a depth of 13 feet bgs in bedrock.

Based on the hazard delineation by Riverside County, the presence of shallow bedrock, and the absence of shallow groundwater within the overall site, the potential for liquefaction to occur at the site is considered to be less than significant.

2.6.2 Lateral Spreading

Liquefaction may also cause lateral spreading. For lateral spreading to occur, the liquefiable zone must be continuous, unconstrained laterally, and free to move along gently sloping ground toward an unconfined area. Due to the shallow bedrock present at the site and the absence of liquefaction hazards, the potential for lateral spreading to occur at the site is considered low.

2.6.3 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. Based on the presence of shallow bedrock encountered during our investigation, significant seismically induced settlement is unlikely to occur onsite.

2.6.4 Seismically Induced Landslides

The site has not been evaluated for seismically induced landslide potential by the State of California. No mapped limits of historic landslides have been mapped by the California Geological Survey on their Landslide Inventory Interactive website. Natural rolling hillsides towards the southeast portion of the development have gradients that are generally 2:1 (horizontal:vertical) or flatter and are composed of very dense shallow bedrock. These slopes are not expected to pose seismically induced landslides hazards.

2.6.5 Flooding

The Federal Emergency Management Agency (FEMA, 2008) has mapped a narrow drainage in the northwest portion of the site to be within a 100-year flood zone. The reminder of the site has been mapped to be outside of any other flood zones (see Figure 6, Flood Hazard Zone Map). Assuming adequate drainage is incorporated into the design and grade

changes, as currently proposed, increase the existing elevation for the proposed improvements adjacent to the identified flood zone area, the potential for flooding to occur at the site is considered very low.

2.6.6 Seismically Induced Flooding

Earthquake-induced flooding can be caused by failure of dams or other water-retaining structures as a result of an earthquake. The site is not located within a dam breach inundation zone as delineated by the Department of Safety of Dams (DSOD, 2021) see Figure 7, *Dam Breach Inundation Map*). Considering that the site is located outside State delineated dam inundation zones, the potential for inundation at the site from earthquake-induced dam failure is less than significant.

2.6.7 Seiches and Tsunamis

A tsunami, or seismically generated sea wave, is generally created by a large, distant earthquake occurring near a deep ocean trough. A seiche is an earthquake-induced wave in a confined body of water, such as a lake or reservoir. Damage from tsunamis is confined to coastal areas that are 20 feet or less above sea level. Since the project is not located near the coast and is separated by significant hillsides to the nearest confined body bodies of water, the risk of inundation from a tsunami or seiche has no impact.

2.6.8 Slope Stability and Landslides

The site is situated at the base of the adjacent rolling hillsides. No historic landslides have been mapped near the site by the State of California. Additionally, no evidence of previous landslides was found during our investigation. Therefore, slope stability and landslides are not expected to be a hazard at the site. Based on our review the provided *Geotechnical Evaluation Summary Exhibit* prepared by Hunsaker and Associates, we understand that existing rolling hillsides and proposed graded slopes are to be at configurations of 2:1 (H:V) or flatter within very dense bedrock formations.

2.6.9 Regional Subsidence

The site has been mapped outside of any areas of subsidence due to groundwater pumping, peat loss, or oil extraction as delineated by the United States Geological Survey (USGS). Due to the shallow bedrock present at the site, the chance for subsidence to occur at the site is considered nil.

2.7 Infiltration Testing

Infiltration testing was conducted within five of the six excavated test pits onsite (IT-1, IT-2, and IT-4 through IT-6) to estimate the infiltration characteristics of the onsite soils at the depths and locations tested and requested by the design team. The infiltration testing was generally conducted at a bottom test zone depth of approximately 2 feet to 3.5 feet bgs

The test consisted of excavating a test pit to the depth of the test area, and excavating a smaller infiltration test cube, generally a 1ft x 1ft x 1ft (L:W:H) at the bottom of the test pit. A layer of ¾ inch rock was placed in the excavated infiltration test pit bottom to support a temporary 6-inch-diameter perforated well casing pipe. In addition, ¾ inch rock was poured around the outside of the well casing within the test zone to prevent the test area from caving/collapsing or eroding when water is added. Water is added into the infiltration test pit to an initial water height, as water within the pit infiltrates into the soil, measurements are taken of the height of the water column within the pit at equally timed intervals (known as a falling head test). The infiltration rate as measured during intervals of the test is defined as the flow rate of water infiltrated, divided by the surface area of the infiltration interface.

Results of the infiltration testing are summarized below and are provided in Appendix A. Infiltration Recommendations are presented in section 3.10.

Infiltration Test Rates

Exploration ID	Soil Type	Approx. Test Zone (ft), bgs	Approximate Percent Fines	Unfactored Infiltration Rate (in/hr)	Proposed Application
IT-1	Weathered Bedrock	3-3.5	13	1.4	Infiltration Basin
IT-2	Weathered Bedrock	2-2.5	34	0.5	Infiltration Basin
IT-3	Unable to Test Due to Very Shallow Hard Bedrock				Infiltration Basin
IT-4	Weathered Bedrock	3-3.5	6	10	Infiltration Basin
IT-5	Weathered Bedrock	2.7-3	31	0.5	Leach Field
IT-6	Weathered Bedrock	2-2.5	25	1.7	Leach Field

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on our study, construction of the proposed development is generally feasible from a geotechnical standpoint, except for the construction of infiltration systems (see Section 3.10). No severe geologic or soil 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, potentially compressible near-surface soils, and rippability of shallow bedrock. 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 debris, which should be disposed of offsite. Any underground obstructions should be removed. 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 total and differential settlement of the proposed structures, the underlying subgrade soil should be prepared in such a manner that a uniform response to the applied loads is achieved.

We recommend that the onsite soils and highly weathered bedrock be overexcavated in areas of proposed structures (including retaining walls retaining more than 3 feet) a minimum depth of 3 feet below the existing ground surface or 2 feet below the bottom of the proposed footings, whichever is deeper. Where possible, overexcavation bottoms should

extend horizontally a minimum of 5 feet from the outside edges of the footings (including columns connected to the buildings), or a distance equal to the depth of overexcavation below the footings, whichever is farther. During overexcavation, the soil and bedrock conditions should be observed by Leighton to further evaluate these recommendations based on actual field conditions encountered. A firm removal bottom should be established across each building footprint to provide uniform foundation support for the proposed structures. Leighton should observe and test the removal bottom prior to placing fill. Deeper overexcavation and recompaction may be recommended locally to remove any loose soil at the overexcavation bottom or if the gradient of the bottom surface would yield a relatively large fill thickness differential beneath a proposed structure.

Areas outside these overexcavation limits planned for asphalt or concrete pavement, flatwork, and 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.

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 D1557 laboratory maximum density.

These recommendations should be reviewed once grading plans for the development are available.

3.1.3 Fill Placement and Compaction

Onsite soil to be used for compacted structural fill should also be free of organic material debris and oversized material (greater than 12 inches in largest dimension). Generation of oversized rock is expected during grading of this project, and handling of oversized material is explained in Section 3.1.7 of this report. 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 to near optimum moisture content, and compacted to a minimum 90 percent relative compaction. Relative compaction should be determined in accordance with ASTM Test Method D1557. Aggregate base for pavement should be compacted to a minimum of 95 percent relative compaction.

3.1.4 Import Fill Soil

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

We assume that the bedrock onsite will not shrink but estimate it could bulk (roughly 5 percent) when placed as compacted fill. The alluvial soils onsite are expected to shrink on the order of 10 ± 5 percent when placed as compacted fill.

These values can vary significantly depending on the level of fill compaction, variations in the dry density of the existing soil, and other factors that influence the amount of volume change. The preliminary estimates of shrinkage presented here are, by necessity, based on very limited data as compared to the thousands of cubic yards of earth to be moved. Adjustments to earthwork volume should be anticipated during grading of the site, and areas should be established where grades can be adjusted to accommodate variations in earthwork shrinkage or swelling.

3.1.6 Rippability

Resistant tonalite bedrock underlies the site and excavations into this unit will be made with difficulty. Five areas were identified and requested for further evaluation of feasibility by Hunsaker and Associates within the provided *Geotechnical Evaluation Summary Exhibit* as these identified areas were proposed to have the deeper cuts in order to accommodate the grading design of the large pads onsite. Cuts ranging in depth from 0.5 to 20 feet in depth were identified within these select areas, with the majority of 20 foot cuts generally being located towards the southeast of the site. The conducted Seismic Refraction Surveys were conducted on the requested areas mentioned above with proposed cuts up to 20 feet in depth. Based on the Seismic Refraction Survey conducted during our investigation, bedrock was generally considered rippable using a Caterpillar D8R, D9R, and D10R heavy rippers at depths of 0 to 20 feet bgs with other areas being considered rippable up to a depth of 65 feet. The Seismic Refraction Survey identified a portion of the southwestern corner of the site (Line 2) to contain a zone at a depth of 15 to 20 feet bgs that is anticipated to contain bedrock that is considered marginally rippable, , heavy ripping and blasting should be expected. In addition, unweathered boulders or zones may be present within otherwise rippable material.

The amount of excavation into resistant bedrock can be minimized with a grading plan that minimizes deep cuts into bedrock based on review of the provided seismic refraction survey attached in Appendix D.

Design of proposed utilities should consider the rippability aspect within the onsite bedrock. The proposed utilities should be overexcavated during the grading phase, as heavy rippers are readily available during this time and excavations into bedrock with conventional pipeline equipment might pose difficulty.

3.1.7 Oversized Material

In addition to the difficulty of excavating resistant bedrock during grading, excavation into unweathered bedrock during grading will likely generate oversized material. In general, any rock or rock fragments greater than 12

inches in largest dimension are considered oversized. Oversized material excavated during grading will require special handling and/or special placement in fills.

The primary geotechnical consideration of oversized materials is the potential for settlement of the compacted fill and negative impacts on overlying improvements associated with improperly placed oversized materials. Settlement can occur when there are significant voids or loose soil around or between rocks, occurring for instance, when water eventually works its way into these areas. Nesting oversize material can cause these voids, as can inadequate quantities of water applied during fill placement, or inadequate compaction methods / equipment.

With the presence of shallow dense bedrock onsite, the generation of oversized material during grading is anticipated and should be anticipated by the grading contractor. The amount of oversized expected to be generated during grading will depend on the methods and equipment used by the grading contractor. Oversize rock should be placed in deeper fills (deeper than 5 feet below finish grade) or removed from structural fill areas. All rocks larger than 24 inches in dimension should be placed in windrows, surrounded by sandy soils, and placed with copious amounts of water. The rock windrows should be placed such that individual rocks are not nested and sandy soil can be worked completely around the rocks. It is imperative that the contractor use copious amounts of water.

Excavations for proposed utilities can be very difficult in the presence of large (greater than 24 inches) rocks. To facilitate utility construction (but not a geotechnical requirement), removing rocks larger than 24 inches in the upper 5 feet below the rough graded surface or 1 foot below the deepest utility may be considered.

3.2 Shallow Foundation Recommendations

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

California Building Code (CBC) requirements. Local agencies, the structural engineer or the CBC may have requirements that are more stringent.

Overexcavation and recompaction of the footing subgrade soil should be performed as detailed in Section 3.1.2.

3.2.1 Minimum Embedment and Width

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

3.2.2 Allowable Bearing

An allowable bearing pressure of 2,000 pounds-per-square-foot (psf) may be used, based on an assumed embedment depth of 18 inches and minimum width described 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 higher bearing pressures are required, this should be reviewed on a case-by-case basis and may include additional overexcavation and/or soil reinforcement. These allowable bearing pressures are for total dead load and sustained live loads. Footing reinforcement should be designed by the structural engineer.

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 0.3. The passive resistance may be computed using an allowable equivalent fluid pressure of 240 pounds per cubic foot (pcf), assuming there is constant contact between the footing and undisturbed soil. The coefficient of friction and passive resistance may be combined without further reduction.

3.2.4 Increase in Bearing and Friction - Short Duration Loads

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

3.3 Recommendations for Slabs-On-Grade

Concrete slabs-on-grade should be designed by the structural engineer in accordance with the current CBC for soil with a 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 finish grade to evaluate the expansion index of near-surface subgrade soils. In addition, slabs-on-grade should have the following minimum recommended components:

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

- Moisture Retarder: A minimum of 10-mil moisture retarder should be placed below slabs where moisture-sensitive floor coverings or equipment is planned. The structural engineer should specify pertinent concrete design parameters and moisture migration prevention measures, such as whether a sand blotter layer should be placed over the vapor retarder. The moisture barrier may be placed directly on subgrade provided gravel or other protruding objects that could puncture the moisture retarder are removed from the subgrade prior to placement. A heavier vapor retarder (such as 15 mil Stego Wrap) placed directly on prepared subgrade may also be used. 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.

- 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, and rapid moisture loss due to hot, dry, and/or windy weather conditions during placement and curing. Cracking due to temperature and moisture fluctuations can also be expected. Low slump concrete can reduce the potential for shrinkage cracking. Additionally, our experience indicates that reinforcement in slabs and foundations can generally reduce the potential for concrete cracking. The structural engineer should consider these components in slab design and specifications.

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

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

3.4 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 current CBC. The CBC seismic design parameters listed in Section 2.5.2 of this

report should be considered for the seismic analysis of the subject site.

3.5 Retaining Walls

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 8 (rear of text). Using expansive soil as retaining wall backfill will result in higher lateral earth pressures exerted on the wall. Based on these recommendations, the following parameters may be used for the design of conventional retaining walls:

Static Equivalent Fluid Weight for Sand Backfill (pcf)		
Condition	Level Backfill	2:1 Sloping Backfill
Active	40	65
At-Rest	60	87
Passive (allowable)	240 (Maximum of 4,000 psf)	120 (2:1 Sloping Front)

The above values do not contain an appreciable factor of safety except the passive pressure, so the structural engineer should apply the applicable factors of safety and/or load factors during design, as specified by the California Building Code.

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

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

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

projection from the surcharging structure on the stem of the wall should be considered in the design. A third of uniform vertical surcharge-loads should be applied at the surface 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.

For walls with a retained height over 6 feet, or where otherwise required by Code or deemed appropriate by the structural engineer, we recommend that the wall designs be checked seismically using an additive seismic Equivalent Fluid Pressure (EFP) of 19 pcf for level backfill and an EFP of 32 pcf for a 2:1 (H:V) sloping backfill, which is added to the active EFP. Such walls that are to be designed in the static case assuming the at-rest condition should be checked seismically using this additive seismic EFP added to the active condition (i.e., the additive seismic EFP is not added to the at-rest EFP value shown in Table 2 above). The additive seismic EFP should be applied with a standard EFP pressure distribution (i.e., it is not an inverted triangle).

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 3,000 psf.

Plans indicating retaining wall locations and heights were not available during this study. Once plans are available they should be reviewed by Leighton to ensure the recommendations provided above remain applicable.

3.6 Pavement Design

Based on the design procedures outlined in the 2017 Caltrans Highway Design Manual, and using an assumed design R-value of 18, 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.

Asphalt Pavement Section Thickness, Type I Subgrade Soil			
Traffic Index	Asphaltic Concrete (AC) Thickness (inches)	Class 2 Aggregate Base Thickness (inches)	Total Pavement Section Thickness (inches)
5 or less	3	7.5	10.5
6	4.0	9.0	13.0
7	5.0	10.5	15.5

All pavement construction should be performed in accordance with the Standard Specifications for Public Works Construction or Caltrans Specifications. Field observations and periodic testing, as needed during placement of the base course materials, should be undertaken to ensure that the requirements of the standard specifications are fulfilled.

In areas where rigid concrete pavement is planned with auto and light truck access, we recommend 5.5 inches of Portland Cement Concrete (PCC) over 4 inches of aggregate base placed on prepared subgrade soil. For heavy truck access, we recommend a minimum of 7.0 inches of PCC over 6 inches of aggregate base. The PCC should have a 28-day compressive strength of 4,000 psi. Reinforcement should be specified by the structural engineer. The PCC pavement sections should be provided with crack-control joints spaced no more than 15 and 12.5 feet on center each way for 7 and 5.5-inch thick concrete, respectively. If sawcuts are used, they should have a minimum depth of ¼ of the slab thickness and made within 24 hours of concrete placement. We recommend that sections be as nearly square as possible.

PCC sidewalks should be at least 4 inches thick over prepared subgrade soil, with construction joints no more than 8 feet on center each way, with sections as nearly square as possible. Use of reinforcing will help reduce severity of cracking.

For concrete trash aprons, ADA stalls and ADA cross-walks, we recommend a minimum 5 inches of concrete over prepared subgrade soil.

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 95 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.

3.7 Temporary Excavations

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

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

Cantilever shoring should be designed based on an active equivalent fluid pressure of 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.

3.8 Trench Backfill

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

(1) **Granular Bedding:** Uniform, free-draining (minimum permeability of 2 in/hr) sand material with a Sand Equivalent (SE) greater than or equal to 30, passing the No. 4 U.S. Standard Sieve (or as specified by the pipe manufacturer). Open-graded material (such as crushed rock) should not be used (unless surrounded by Mirafi 140N filter fabric), as surrounding soil can erode into such material, potentially causing voids at the surface and under structures.

(2) **CLSM:** Controlled Low Strength Material (CLSM) conforming to Section 201-6 of the SPWC. CLSM bedding should be placed to 1-foot (0.3 m) over the top of the conduit, and vibrated.

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

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

3.9 Surface Drainage

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

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

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

3.10 Infiltration Recommendations

Based on our onsite observations, laboratory testing, and infiltration test results summarized in Section 2.6 and presented in Appendix A, reliance of infiltration into onsite native soils for basins or leach fields is not recommended. Due to the presence of shallow bedrock in this area, as shallow as 3 inches in IT-3 before hitting refusal at 1.5 feet, and considering separation requirements by the County of Riverside for Low Impact Development Best Management Practices, infiltration is not feasible for this project. Although infiltration testing in TP-4, located in an area with a larger layer of weathered bedrock, showed favorable results, the thin mantle and underlying impermeable bedrock will create a confining layer within the area. It is likely that water infiltrated at depths of approximately 3 feet bgs through prolonged infiltration will tend to migrate laterally rather than vertically and produce lower infiltration values. Actual infiltration rates would be anticipated to decrease as the adjacent soils saturate.

The County of Riverside requires a minimum 5-foot vertical separation from bedrock or impermeable material for infiltration BMP devices. Compacted fill should also not be relied upon for infiltration due to inconsistent rates during compaction efforts and different relative density rates. Due to the presence of shallow bedrock in this area and considering separation requirements by the

County of Riverside for Low Impact Development Best Management Practices, infiltration of storm water at the site is generally considered not feasible for this project.

3.11 Sulfate Attack and Corrosion Protection

Based on the results of laboratory testing, concrete structures in contact with the onsite soil will have negligible exposure to water-soluble sulfates in the soil (Exposure Class S0). There is no cement type restriction for Exposure Class S0 per ACI 318. Concrete should be designed in accordance with ACI 318-14, Section 19.3 (ACI, 2014), adopted by the 2022 CBC (Section 1904.2).

The onsite soil is considered to be moderately corrosive to ferrous metals. It is recommended that any buried pipe be made of non-ferrous material, or that any ferrous pipe be protected by dielectric tape, polyethylene sleeves and/or other methods, with recommendations from a corrosion engineer. Corrosion information presented in this report should be provided to your underground utility subcontractors. Additional testing and evaluation by a corrosion engineer may be warranted if metallic utilities are planned.

3.12 Additional Geotechnical Services

The geotechnical recommendations presented in this report are based on subsurface conditions as interpreted from limited subsurface explorations and limited laboratory testing. Our geotechnical recommendations provided in this report are based on information available at the time the report was prepared and may change as plans are developed. 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 Consulting, Inc. 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 necessarily based in part upon data obtained from a limited number of observances, site visits, soil samples, tests, analyses, histories of occurrences, spaced subsurface explorations and limited information on historical events and observations. Such information is necessarily incomplete. The nature of many sites is such that differing characteristics can be experienced within small distances and under various climatic conditions. Changes in subsurface conditions can and do occur over time. This investigation was performed with the understanding that the subject site is proposed for residential development. The client is referred to Appendix F regarding important information provided by the Geo-Professional Business Association (GBA) on geotechnical engineering studies and reports and their applicability.

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

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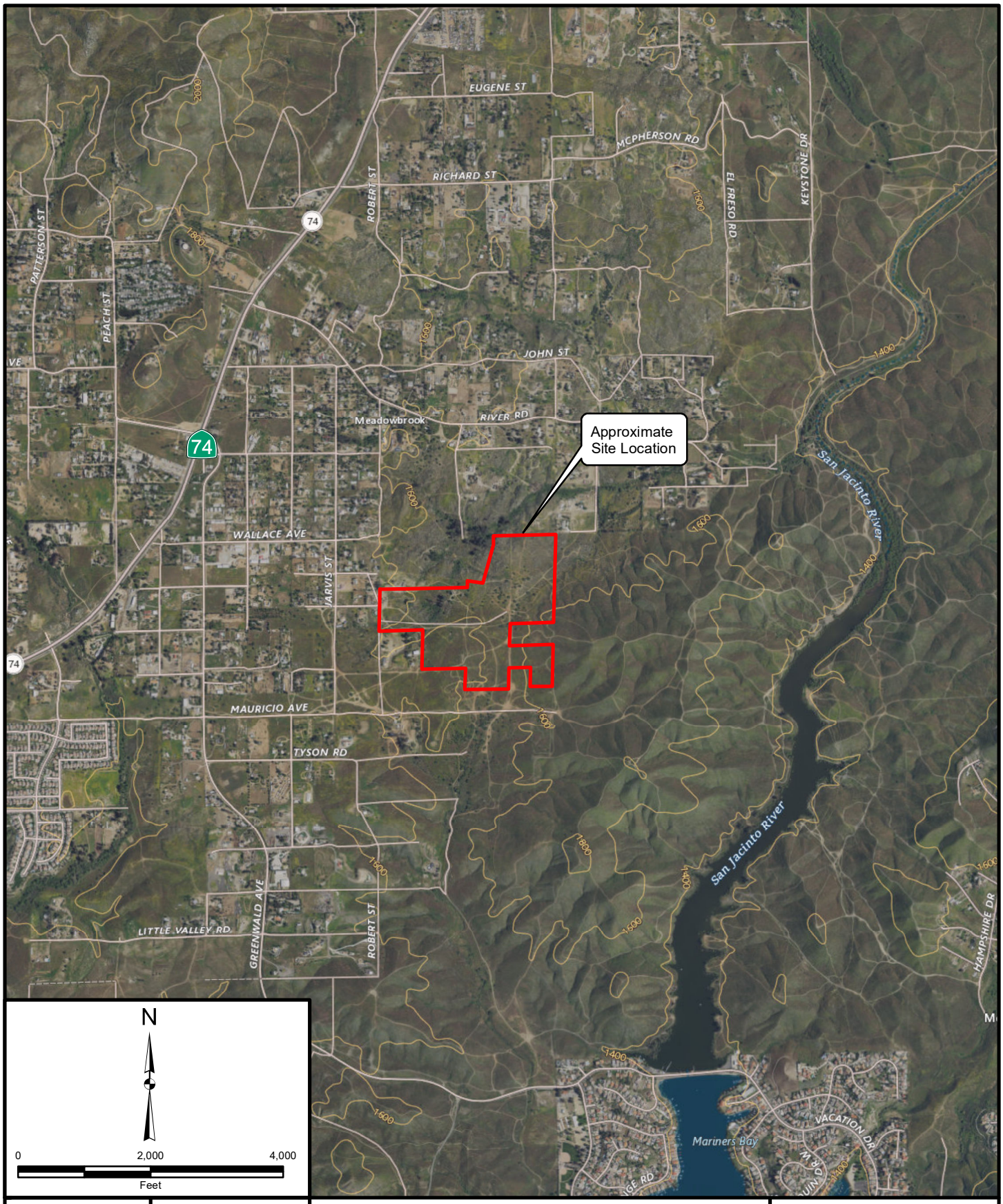


Project: 19894.038	Eng/Geol: SGO/JDH
Scale: 1 " = 2,000 '	Date: February 2024

Reference: USGS The National Map: Orthoimagery and US Topo. Data refreshed

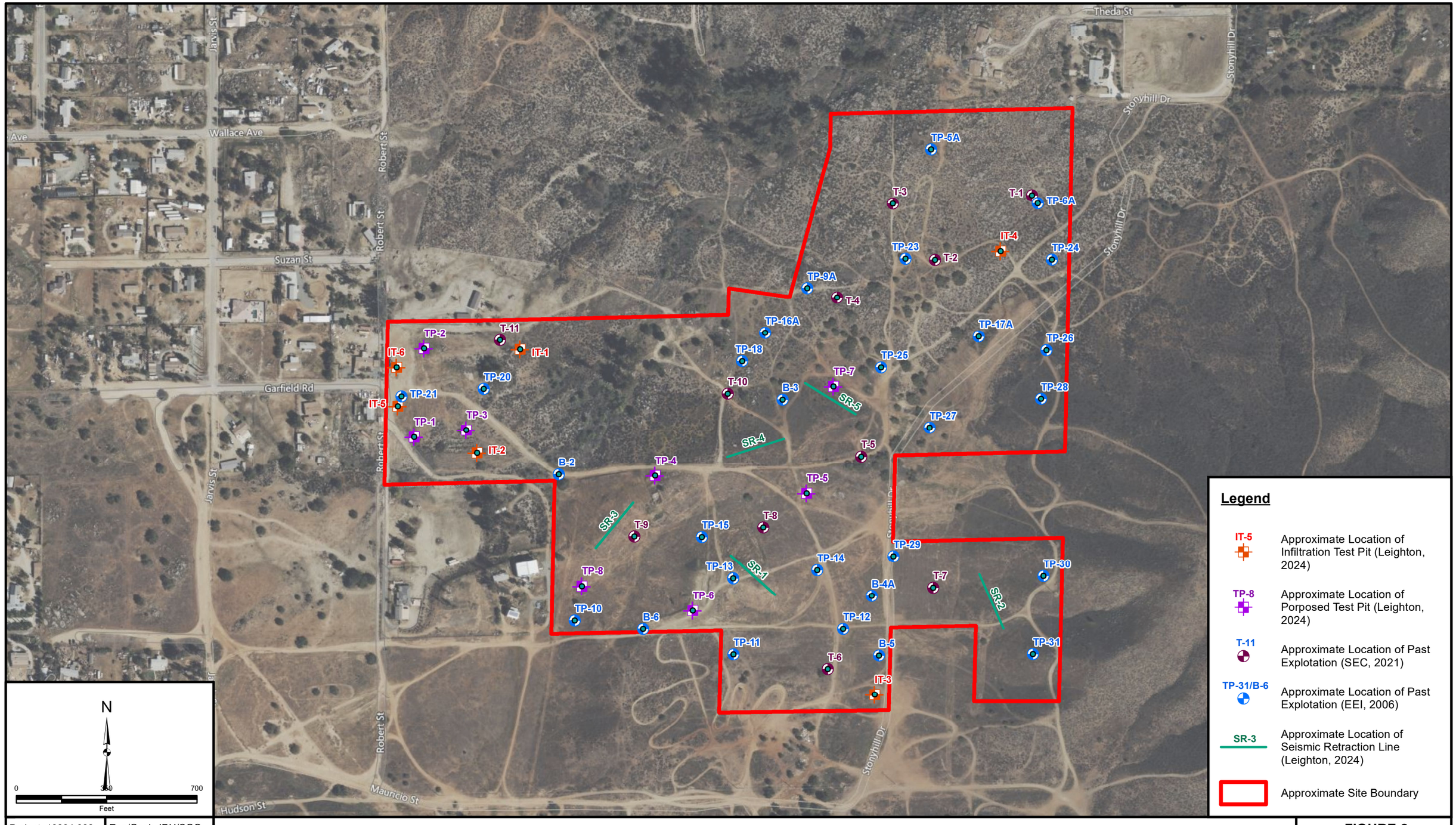
SITE LOCATION MAP

Proposed Crescent Gardens Cemetery
APNs 349-170-003 through -013, -035,
349-180-002, -004 through -008, -016, -017,
-020, 349-190-006, Perris Area,
Unincorporated Riverside County, California

FIGURE 1

Leighton

Map Saved as J:\Drafting\19894\Maps\19894_F01_SLM_2024-02-12.mxd on 2/15/2024 7:08:37 AM Author: KVM (kmanchikanti)



Legend

- IT-5 — Approximate Location of Infiltration Test Pit (Leighton, 2024)
- TP-8 — Approximate Location of Porposed Test Pit (Leighton, 2024)
- T-11 — Approximate Location of Past Explotation (SEC, 2021)
- TP-31/B-6 — Approximate Location of Past Explotation (EEI, 2006)
- SR-3 — Approximate Location of Seismic Retraction Line (Leighton, 2024)
- Approximate Site Boundary

Project: 19894.038	Eng/Geol: JDH/SGO
Scale: 1 " = 350 '	Date: February 2024

Reference: © 2024 Microsoft Corporation © 2024 Maxar ©CNES (2024) Distribution Airbus DS © 2024 TomTom, Rexford Industrial Realty, Conceptual Sie Plan

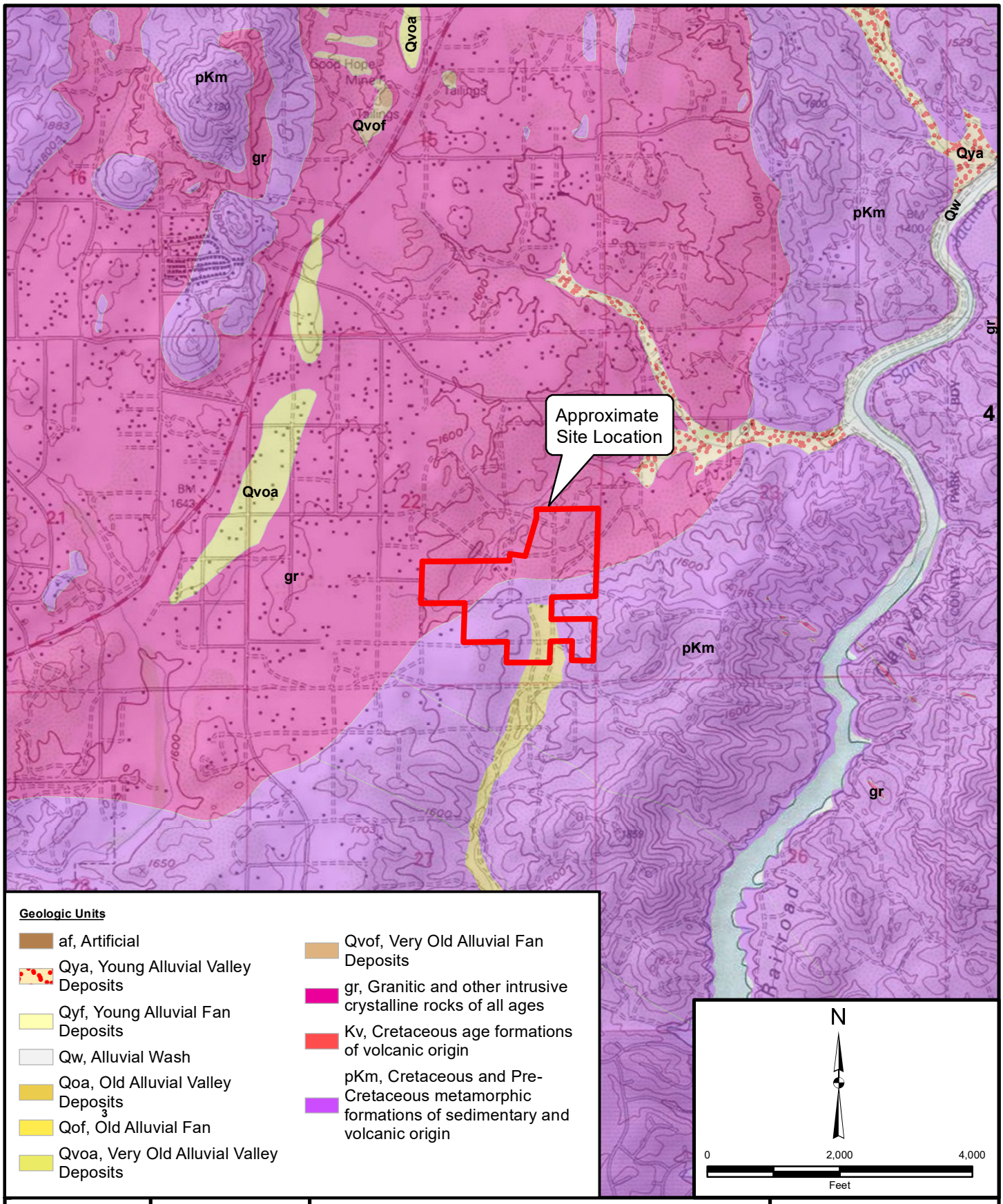
EXPLORATION LOCATION MAP

Proposed Crescent Gardens Cemetary
APNs 349-170-003 through -013, -035, 349-180-002, -004 through -008, -016, -017, -020, 349-190-006
Perris Area, Unincorporated Riverside County, California

FIGURE 2

Leighton

Map Saved as J:\Drafting\19894\Maps\19894_F02_GM_2024-02-23.mxd on 2/29/2024 10:55:09 AM Author: KVM (kmanchikanti)



Geologic Units

- af, Artificial
- Qya, Young Alluvial Valley Deposits
- Qyf, Young Alluvial Fan Deposits
- Qw, Alluvial Wash
- Qoa, Old Alluvial Valley Deposits
- Qof, Old Alluvial Fan
- Qvoa, Very Old Alluvial Valley Deposits
- Qvof, Very Old Alluvial Fan Deposits
- gr, Granitic and other intrusive crystalline rocks of all ages
- Kv, Cretaceous age formations of volcanic origin
- pKm, Cretaceous and Pre-Cretaceous metamorphic formations of sedimentary and volcanic origin

Project: 19894.038	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: February 2024

Reference: Geologic map of the San Bernardino & Santa Ana 30'x60' Quadrangles, California. Morton & Miller, 2006

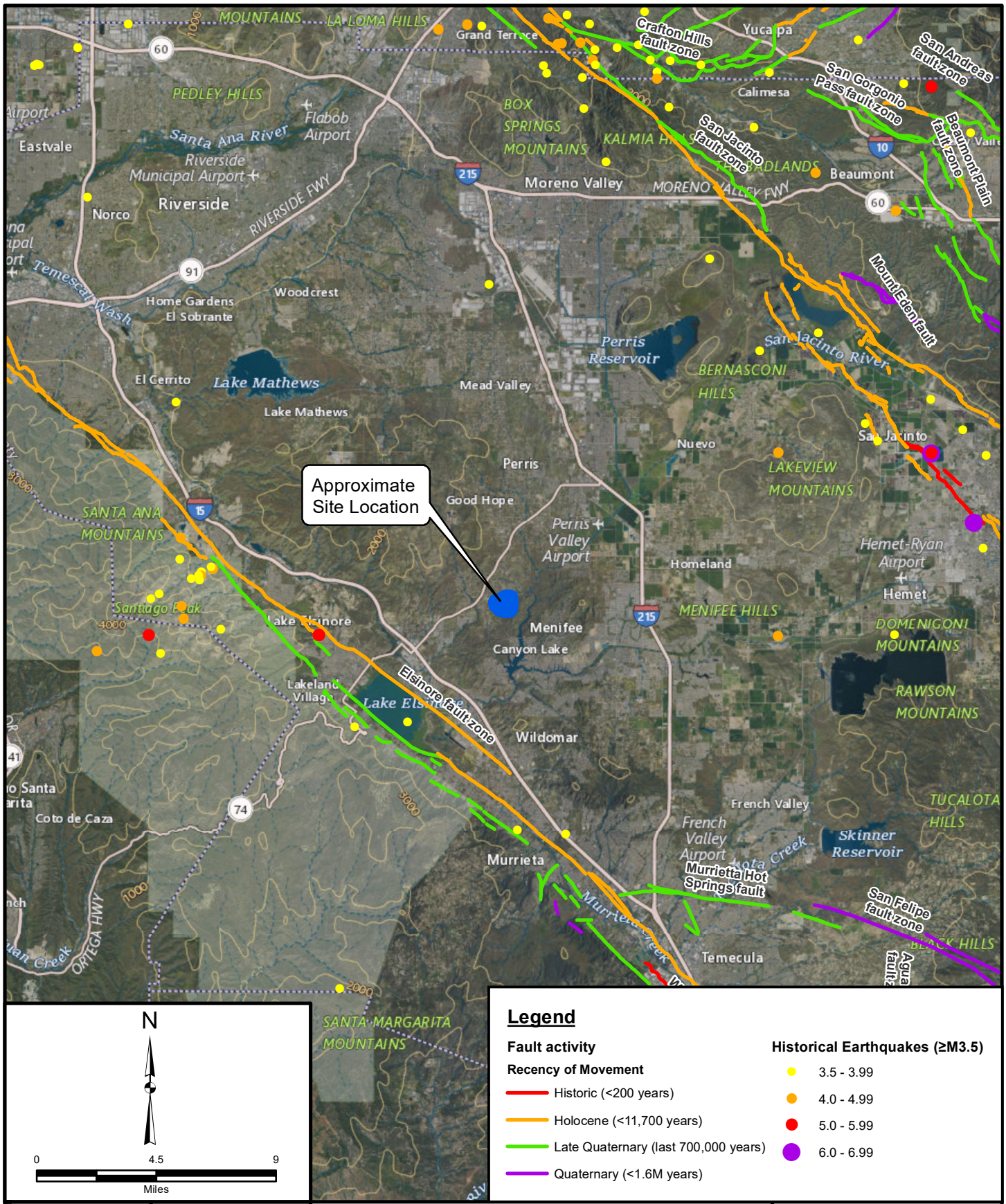
REGIONAL GEOLOGY MAP

Proposed Crescent Gardens Cemetary
APNs 349-170-003 through -013, -035,
349-180-002, -004 through -008, -016, -017,
-020, 349-190-006, Perris Area,
Unincorporated Riverside County, California

FIGURE 2

Leighton

Map Saved as J:\Drafting\19894\Maps\19894_F03_RGM_2024-02-12.mxd on 2/15/2024 7:15:13 AM Author: KVM (kmanchikanti)



Project: 19894.038	Eng/Geol: JDH/SGO
Scale: 1 " = 5 miles	Date: February 2024

Basemap Reference: USGS The National Map: Orthoimagery and US Topo. Data refreshed August, 2023.
Seismicity Data Reference: maps.conservation.ca.gov

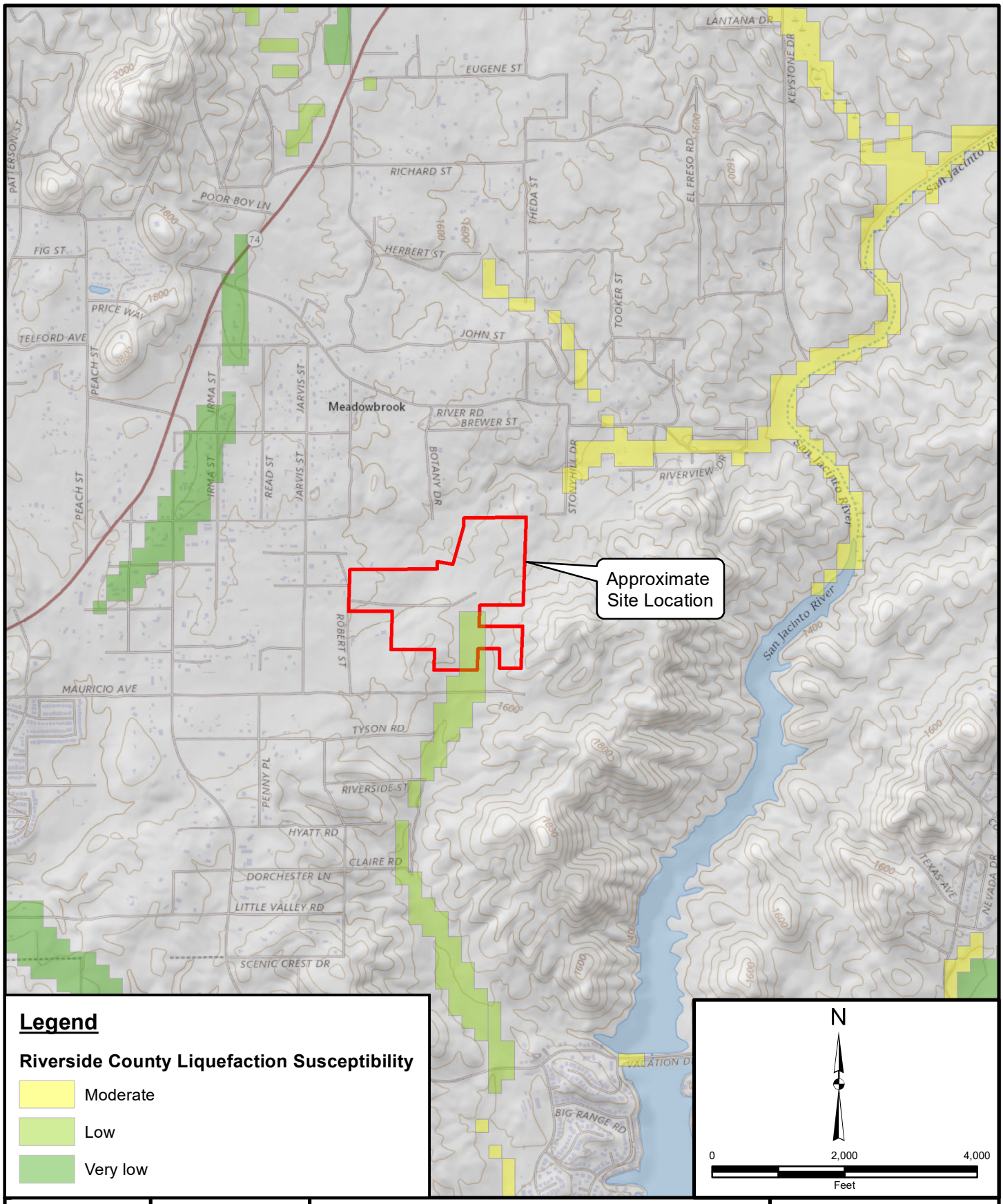
REGIONAL FAULT AND HISTORIC SEISMICITIY MAP

Proposed Crescent Gardens Cemetary
APNs 349-170-003 through -013, -035,
349-180-002, -004 through -008, -016, -017,
-020, 349-190-006, Perris Area,
Unincorporated Riverside County, California

FIGURE 4

Leighton

Map Saved as J:\Drafting\19894\Maps\19894_F04_RFHSM_2024-02-12.mxd on 2/15/2024 7:19:55 AM Author: KVM (kmanchikanti)



Project: 19894.038	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: February 2024

Service Layer Credits: USGS The National Map: National Boundaries Dataset, 3DEP Elevation Program, Geographic Names Information System, National Hydrography Dataset, National Land Cover Database, National Structures Dataset, and National Transportation Dataset; USGS Global Ecosystems; U.S. Census Bureau TIGER/Line data; USFS Road

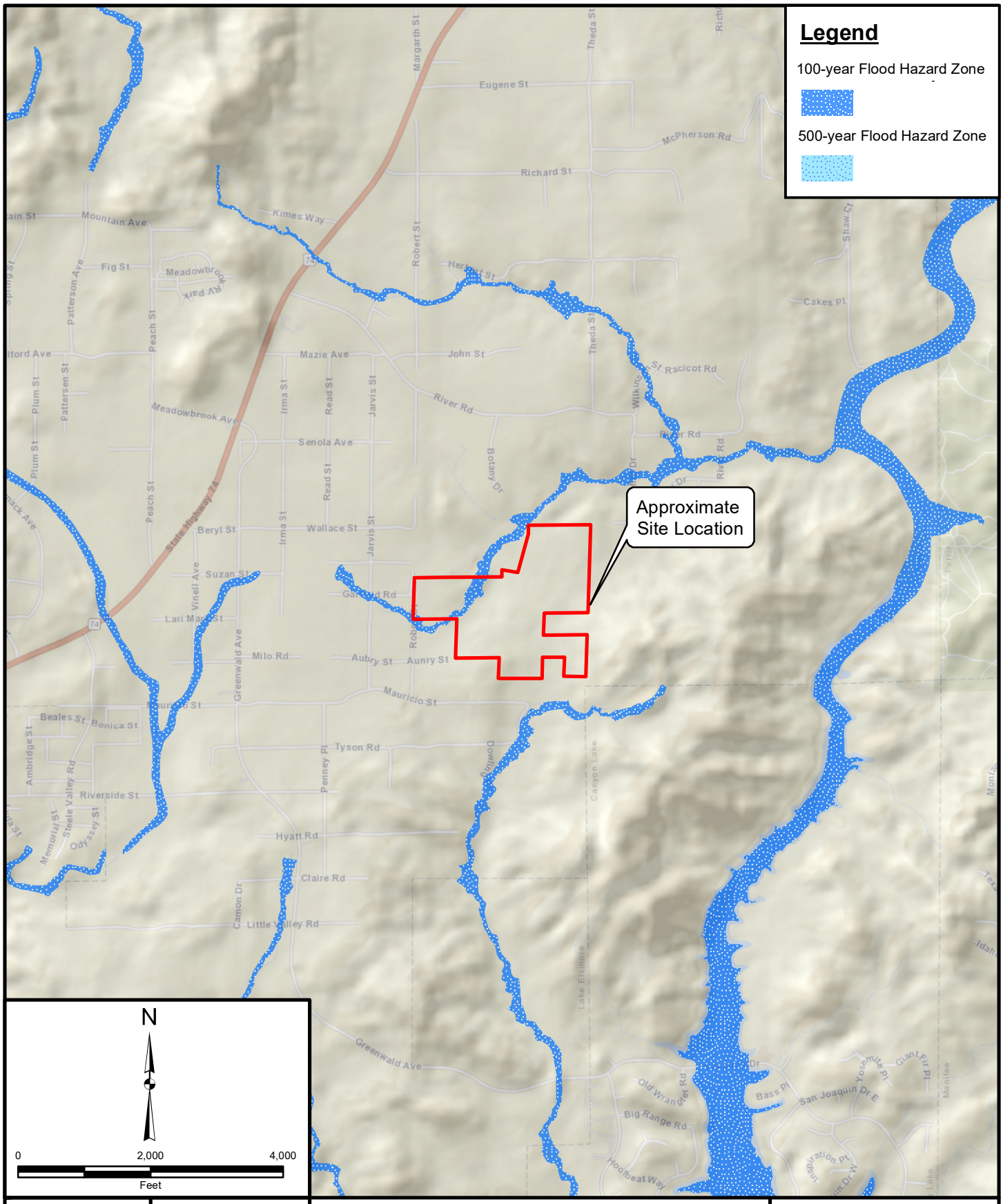
SEISMIC HAZARD MAP

Proposed Crescent Gardens Cemetary
APNs 349-170-003 through -013, -035,
349-180-002, -004 through -008, -016, -017,
-020, 349-190-006, Perris Area,
Unincorporated Riverside County, California

FIGURE 5

Leighton

Map Saved as J:\Drafting\19894\Maps\19894_F05_SHM_2024-02-12.mxd on 2/15/2024 7:14:40 AM Author: KVM (kmanchikanti)



Project: 19894.038	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: February 2024

Reference: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China (Hong Kong), Esri Korea, Esri (Thailand), NGCC, (c) OpenStreetMap contributors, and the GIS User Community
FEMA (http://www.fema.gov/index.shtm), DWR (http://www.dwr.ca.gov)

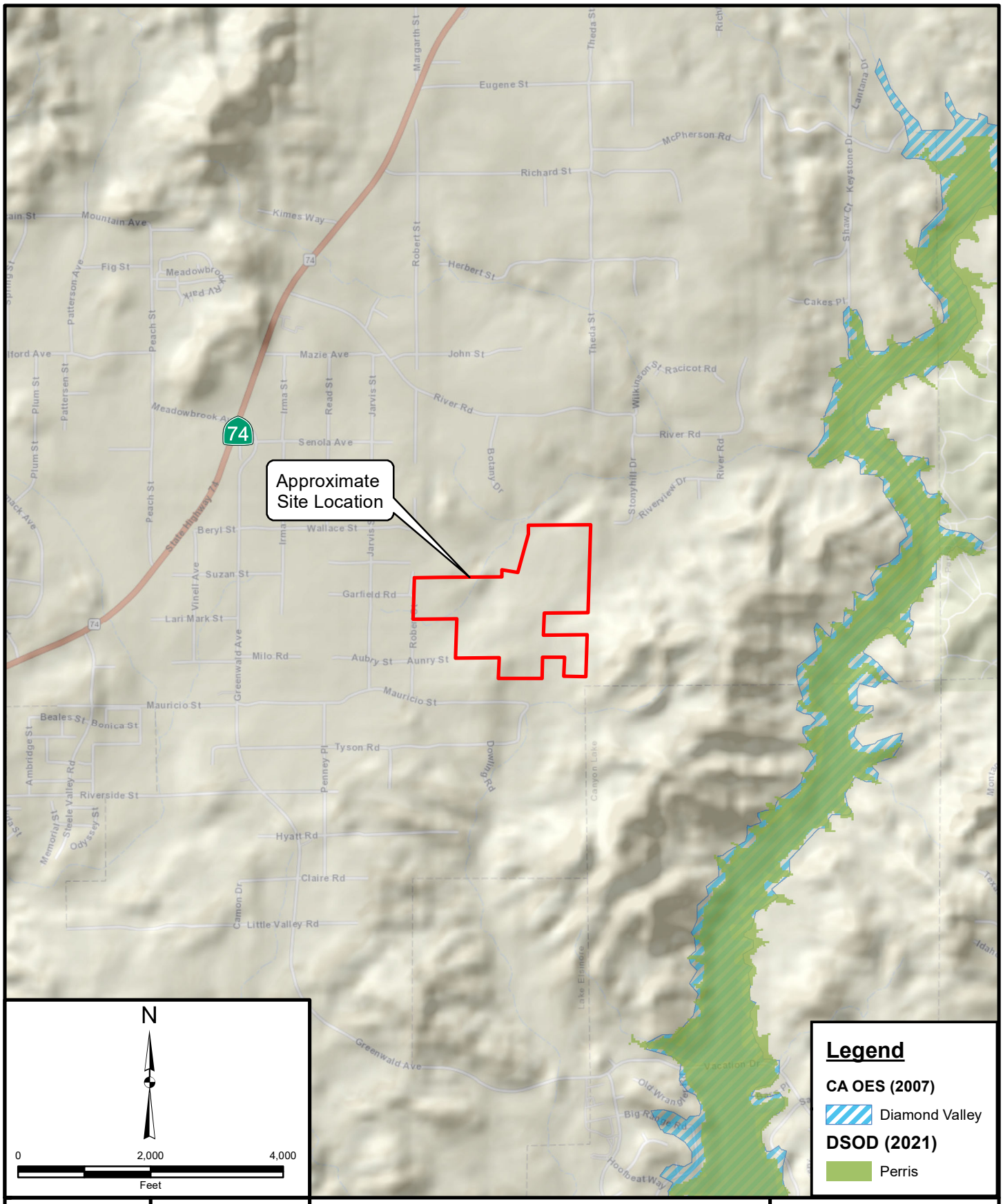
FLOOD HAZARD ZONE MAP

Proposed Crescent Gardens Cemetary
APNs 349-170-003 through -013, -035,
349-180-002, -004 through -008, -016, -017,
-020, 349-190-006, Perris Area,
Unincorporated Riverside County, California

FIGURE 6

Leighton

Map Saved as J:\Drafting\19894\Maps\19894_F06_FHM_2024-02-21.mxd on 2/21/2024 10:57:12 AM Author: KVM (kmanchikanti)



Project: 19894.038	Eng/Geol: JDH/SGO
Scale: 1 " = 2,000 '	Date: February 2024

Basemap Reference: Sources: Esri, HERE, Garmin, USGS, Intermap, INCREMENT P, NRCan, Esri Japan, METI, Esri China
Reference: Office of Emergency Services (2007),
Dept of Safety of Dams (2021)
National Inventory of Dams, Army Corps of Engrs (2021)

DAM INUNDATION MAP

Proposed Crescent Gardens Cemetary
APNs 349-170-003 through -013, -035,
349-180-002, -004 through -008, -016, -017,
-020, 349-190-006, Perris Area,
Unincorporated Riverside County, California

FIGURE 7

Leighton

Map Saved as J:\Drafting\19894\Maps\19894_F07_DIM_2024-02-21.mxd on 2/21/2024 11:01:12 AM Author: KVM (kmanchikanti)

APPENDIX A
EXPLORATION LOGS

TEST PIT TP-1

City of Rancho Cucamonga Memorial Park

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1609'

Location: **See Figure 2 - Exploration Location Map**

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	1.3	SP-SC	POORLY-GRADED SAND with CLAY, brown, moist, mostly fine sand, some medium to coarse sand, medium plasticity, slightly micaceous, rootlets	Topsoil				
1.3	3.0	IGNEOUS	IGNEOUS ROCK (granitic), orange brown, coarse-grained, intensely weathered to decomposed, dry to slightly moist, fine to coarse sand sized mineral grains, trace clay	Kgt				
3.0	7.5	IGNEOUS	IGNEOUS ROCK (granitic), white and black to gray, coarse-grained, intensely weathered, dry, fine to coarse sand sized mineral grains, friable, few fines, relatively dense, more intact with depth	Kgt				

Total Depth = 7.5 feet bgs (REFUSAL)
No groundwater encountered when excavating
Test pit backfilled with spoils and wheel-rolled at surface



TEST PIT TP-2

Muslim Mortuary Crescent Gardens Canyon

Project No. 038.0000019894

Logged By: BTM

Date Excavated: 1/22/2024

Sampled By: BTM

Elevation: 1608'

Location: **See Figure 2 - Exploration Location Map**

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.5	SW-SM	WELL-GRADED SAND with SILT, brown, moist, mostly fine sand, some medium to coarse sand, low to medium plasticity	Topsoil				
0.5	1.0	IGNEOUS	IGNEOUS ROCK (granitic), orange, coarse-grained, decomposed, moist, fine to coarse sand sized mineral grains, extensive clay development	Kgt				
1.0	3.0	IGNEOUS	IGNEOUS ROCK, orange brown, coarse-grained, intensely weathered, slightly moist, fine to coarse sand sized grains, few fines, minor clay development	Kgt				
3.0	7.0	IGNEOUS	IGNEOUS ROCK, orange brown, coarse-grained, intensely to moderately weathered, 6% fines (lab)	Kgt				

Total Depth = 7 feet bgs (REFUSAL)
No groundwater encountered when excavating
Test pit backfilled with spoils and wheel-rolled at surface



TEST PIT TP-3

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1605'

Location: **See Figure 2 - Exploration Location Map**

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.8	SC	CLAYEY SAND, dark brown, moist, loose, fine to coarse sand, low plasticity, rootlets, slightly micaceous	Topsoil				
0.8	1.5	IGNEOUS	IGNEOUS ROCK (granitic), dark orange brown, decomposed, slightly moist, fine to coarse sand sized mineral grains, slightly micaceous, few rootlets, friable	Kgt				
1.5	7.0	IGNEOUS	IGNEOUS ROCK (granitic), gray, intesely to moderately weathered, dry to slightly moist, fine to coarse sand sized grains, few fines, heavily micaceous, becomes more dense with depth	Kgt				

Total Depth = 7 feet bgs (REFUSAL)
No groundwater encountered when excavating
Test pit back-filled with spoils and wheel-rolled at surface



TEST PIT TP-4

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1612'

Location: **See Figure 2 - Exploration Location Map**

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	1.33	SC	CLAYEY SAND, dark brown, moist to wet, loose, mostly fine sand, few medium to coarse sand, low plasticity, trace rootlets	Topsoil				
1.33	2.75	SCg	CLAYEY SAND with GRAVEL, red to reddish brown, dry to slighlty moist, dense, mostly fine sand, some medium to coarse sand, low plasticity, fine to coarse subangular granitic gravel	Qoa				
2.75	5.0	IGNEOUS	IGNEOUS ROCK (Granitic), orange brown, coarse-grained, intensely weathered, friable, fine to coarse sand sized grains, few fines, nonplastic	Kgt				

Total Depth = 5 feet bgs (REFUSAL)
No groundwater encountered when excavating
Test pit back-filled with spoils and wheel rolled at surface



TEST PIT TP-5

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1598'

Location: **See Figure 2 - Exploration Location Map**

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.9	SC	SANDY CLAY, reddish brown, moist, loose, mostly fine sand, few medium to coarse sand, low to medium plasticity	Topsoil				
0.9	1.75	IGNEOUS	IGNEOUS ROCK (Granitic), orange brown, coarse-grained, decomposed, fine to coarse sand sized mineral grains, some clay development, heavily oxidized, slightly micaceous, 35% fines (lab)	Kgt				
1.75	4.0	IGNEOUS	IGNEOUS ROCK (Granitic), white and orange, intensely weathered, friable, dry to slightly moist, fine to coarse sand sized mineral grains, significant fines, heavily oxidized; less oxidized and less clay development with depth	Kgt				

Total Depth = 5 feet bgs
No groundwater encountered when excavating
Test pit back-filled with spoils and wheel-rolled at surface



TEST PIT TP-6

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1642'

Location: **See Figure 2 - Exploration Location Map**

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.75	SM	SILTY SAND with GRAVEL, brown, slightly moist to moist, fine sand, low plastiity, slightly micaceous, rootlets, 17% fines (lab)	Topsoil				
0.75	4.0	IGNEOUS	METAMORPHIC ROCK (Phyllite), gray, moderately weathered, hard, highly fractured, heavily oxidized on fracture surfaces	Trmp				

Total Depth = 4 feet bgs (REFUSAL)
No groundwater encountered when excavating
Test pit back-filled with spoils and wheel-rolled at surface



TEST PIT TP-7

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1582'

Location: (see Figure 2, *Geotechnical Map*)

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.4	SC	CLAYEY SAND, brown, moist, fine to medium sand, some coarse sand, low plasticity, rootlets	Topsoil				
0.4	2.0	IGNEOUS	IGNEOUS ROCK (Granitic), red to reddish brown, coarse-grained, decomposed, fine to coarse sand sized mineral grains, loose, low plasticity	Kgt				
2.0	3.5	IGNEOUS	IGNEOUS ROCK (Granitic), ornage and gray, decomposed to intensely weathered, fine to coarse sand sized grains, few fines, friable, more dense than above	Kgt				
3.5	5.0	IGNEOUS	IGNEOUS ROCK (Granitic), gray with orange, intensely weathered, fine to coarse sand sized mineral grains, trace to few fines, nonplastic, micaceous, oxidized	Kgt				

Total Depth = 5 feet bgs
No groundwater encountered when excavating
Test pit back-filled with spoils and wheel-rolled at surface



TEST PIT TP-8

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1622'

Location: (see Figure 2, *Geotechnical Map*)

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.5	SM	SILTY SAND with gravel, reddish brown, slighlty moist, mostly fine sand, few medium to coarse sand, few fine subangular gravel, low plasticity	Topsoil				
0.5	3.0	IGNEOUS	METAMORPHIC ROCK (Phyllite), gray, moderately weathered, hard, highly fractured, heavily oxidized on fracture surfaces (reddish color)	Trmp				

Total Depth = 3 feet bgs(REFUSAL)
No groundwater encountered when excavating
Test pit back-filled with spoils and wheel-rolled at surface



TEST PIT IT-1

Muslim Mortuary Crescent Gardens Canyon
Logged By: BTM
Sampled By: BTM
Location: (see Figure 2, *Geotechnical Map*)

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1602'

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet) Top	Depth (feet) Bottom	USCS Symbol	Soil Description	Geologic Unit	Engineering Properties: Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.5	SC	SANDY CLAY, dark brown, moist, fine sand, medium plasticity	Topsoil				
0.5	1.0	IGNEOUS	IGNEOUS ROCK (Granitic), white and orange, decomposed, slightly moist, fine to coarse sand sized mineral grains, micaceous	Kgt				
1.0	3.0	IGNEOUS	IGNEOUS ROCK (Granitic), white to gray with slight orange oxidation, intensely weathered, dry, fine to coarse sand sized mineral grains, low plasticity, micaceous, 13% fines (lab)	Kgt				

Total Depth = 3 feet bgs
No groundwater encountered when excavating
Infiltration testing performed from approximately 3 to 3.5 feet bgs
Test pit backfilled with spoils and wheel-rolled at surface



TEST PIT IT-2

Muslim Mortuary Crescent Gardens Canyon
Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/25/2024
Elevation: 1600'

Location: (see Figure 2, *Geotechnical Map*)

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	1.1	SMc	SILTY SAND with CLAY, dark brown, moist, fine to medium sand, low plasticity, slight cohesion, rootlets	Topsoil				
1.1	2.0	SC	CLAYEY SAND, light to medium brown, dense, mostly fine sand, few medium to coarse sand, low plasticity, slightly micaceous, 34% fines (lab)	Qoa				
2.0	2.5	IGNEOUS	IGNEOUS ROCK (granitic), intensely weathered at bottom of test pit, dry fine to coarse sand sized mineral grains, low plasticity, micaceous	Kgt				

Total Depth = 2 feet bgs
No groundwater encountered when excavating
Infiltration testing performed from approximately 2 to 2.5 feet bgs
Test pit backfilled with spoils and wheel-rolled at surface



TEST PIT IT-3

Muslim Mortuary Crescent Gardens Canyon
Logged By: BTM
Sampled By: BTM
Location: (see Figure 2, *Geotechnical Map*)

Project No. 038.0000019894
Date Excavated: 1/25/2024
Elevation: 1587'

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet) Top	Depth (feet) Bottom	USCS Symbol	Soil Description	Geologic Unit	Engineering Properties: Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	0.25	SMc	SILTY SAND with CLAY, reddish brown, moist, very fine sand, low plasticity, rootlets	Topsoil				
0.25	1.5	IGNEOUS	METAMORPHIC ROCK (Phyllite), gray with red weathering, very hard, moderately to slightly weathered, moderately to highly fractured, extensive oxidation on fracture surfaces, micaeous, slight sheen *Stepout test pit attempted approximately 15 feet south of orignal test pit. Refusal encountered at similar depth. (Infiltration testing not feasible)	Trmp				

Total Depth = 1.5 feet (practical refusal)
No groundwater encountered when excavating
Infiltration testing not feasible at this location
Test pit backfilled with spoils and wheel-rolled at surface



TEST PIT IT-4

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/25/2024
Elevation: 1559'

Location: (see Figure 2, *Geotechnical Map*)

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	1.0	SM	SILTY SAND, medium brown, moist, mostly fine sand, few medium to coarse sand, slightly micaceous, low plasticity	Topsoil				
1.0	3.0	IGNEOUS	IGNEOUS ROCK (Granitic), speckled black and white to gray, decomposed, disintegration of mineral grains, loose to medium dense, slightly moist, fine to coarse sand sized mineral grains, few silt, micaceous, friable, 6% fines (lab)	Kgt				

Total Depth = 3 feet bgs
No groundwater encountered when excavating
Infiltration testing performed from approximately 3 to 3.5 feet bgs
Test pit backfilled with spoils and wheel-rolled at surface



TEST PIT IT-5

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1611'

Location: (see Figure 2, *Geotechnical Map*)

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	1.3	SC	CLAYEY SAND, reddish brown, moist, mostly fine sand, some medium to coarse sand, low to medium plasticity, slightly micaceous	Afu/ Topsoil				
1.3	2.0	IGNEOUS	IGNEOUS ROCK (Granitic), orange brown, intensely weatered to decomposed, dry to slightly moist, fine to coarse sand sized mineral grains, low plasticity, micaceous, 31% fines (lab)	Kgt				

Total Depth = 2 feet bgs
No groundwater encountered when excavating
Infiltration testing performed from approximately 2 to 2.75 feet bgs
Test pit backfilled with spoils and wheel-rolled at surface



TEST PIT IT-6

Muslim Mortuary Crescent Gardens Canyon

Logged By: BTM
Sampled By: BTM

Project No. 038.0000019894
Date Excavated: 1/22/2024
Elevation: 1615'

Location: (see Figure 2, *Geotechnical Map*)

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. This soil description (below) is a simplification of actual conditions encountered. Transitions between soil type may be gradual.

Depth (feet)		USCS Symbol	Soil Description	Geologic Unit	Engineering Properties			
Top	Bottom				Depth (feet)	Moisture (%)	Dry Density (pcf)	Relative Compaction (%)*
0.0	1.3	SC	SANDY CLAY to CLAYEY SAND, reddish brown, moist, mostly fine sand, some medium to coarse sand, low to medium plasticity, slightly micaceous	Afu/ Topsoil				
1.3	3.0	IGNEOUS	IGNEOUS ROCK (Granitic), orange brown, intensely weathered to decomposed, dry to slightly moist, fine to coarse sand sized mineral grains, micaceous, 25% fines (lab)	Kgt				

Total Depth = 3 feet bgs
No groundwater encountered when excavating
Infiltration testing performed from approximately 2 to 2.75 feet bgs
Test pit backfilled with spoils and wheel-rolled at surface



Results of Falling Head Infiltration Test

Project: 19894

Exploration #/Location: IT-1

Depth Test Pit, bgs (ft): 3

Tested by: AA

USCS Soil Type in test zone: Weathered Bedrock

Weather (start to finish): Cloudy

Water Source/pH: Tap Water

Equivalent boring diameter: 13.541 in. 6.770275 in. Well Radius

Depth to GW or aquitard, bgs: 100 ft

Well Prep: Bottom of test pit at approximately 3 feet below the ground surface with 4 inch depth infiltration pit, Set 6 inch diameter perforated pipe with 3/4" gravel around anulus

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surface)	0. ft	4. in.	4
Casing stickup measured above top of auger (or ground surface) (+ is up	0. ft	8. in.	8
Depth to top of sand from top of casing	0. ft	0. in.	

Initial estimated Depth to Water Surface (in.): -7

Average depth of water in well, "h" (in.): 3

approx. h/r: 0.4

Tu (Fig. 8) (ft): 100.6

Tu>3h?: yes, OK

Open Pit Dimensions

Length 12 in

Width 12 in

Depth of well bottom below top of casing (in): 12

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity	0.5
Casing outer diameter, in.	6.5
Casing inner diameter, in.	6
Cross-sectional area, in.^2	83.7

Use of Barrels:	No
Use of Flow Meter:	No
Test Type:	Falling Head

Field Data | **Calculations**

Date	Time	Data from Flow Meter		Depth to WL in Boring (measured from top of casing)		Water Temp (deg F)	Refilled?	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)			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)
		Reading (gallons)	Interval Pulse Count				(or Comments)							from supply	from Δh	Total						
Start Date	Start time:			ft	in.																	
1/22/24	14:20	Gallons																				
1/22/24	14:25				9				5	1.0	3.0											
1/22/24	14:27				9.375			2	7	1.4	2.6	-0.375	3	0	31	31	16	941	264	0.9	4.11	3.29
1/22/24	14:30				9.75			3	10	1.8	2.3	-0.375	2	0	31	31	10	628	248	0.9	3.24	2.34
1/22/24	14:33				10.375			3	13	2.4	1.6	-0.625	2	0	52	52	17	1046	226	0.9	8.26	4.26
1/22/24	14:35				10.5			2	15	2.5	1.5	-0.125	2	0	10	10	5	314	210	0.9	2.35	1.37
							Refilled															
1/22/24	14:36				8.25				16	0.3	3.8											
1/22/24	14:38				8.5			2	18	0.5	3.5	-0.25	4	0	21	21	10	628	298	0.9	1.97	1.94
1/22/24	14:40				8.75			2	20	0.8	3.3	-0.25	3	0	21	21	10	628	288	0.9	2.13	2.01
1/22/24	14:43				9.125			3	23	1.1	2.9	-0.375	3	0	31	31	10	628	274	0.9	2.48	2.11
1/22/24	14:46				9.25			3	26	1.3	2.8	-0.125	3	0	10	10	3	209	264	0.9	0.83	0.73
1/22/24	14:49				9.75			3	29	1.8	2.3	-0.5	3	0	42	42	14	837	250	0.9	4.43	3.08
1/22/24	14:52				9.825			3	32	1.8	2.2	-0.075	2	0	6	6	2	126	238	0.9	0.63	0.49
1/22/24	14:55				10.125			3	35	2.1	1.9	-0.3	2	0	25	25	8	502	230	0.9	3.11	2.01
1/22/24	14:58				10.5			3	38	2.5	1.5	-0.375	2	0	31	31	10	628	216	0.9	5.07	2.68
							Refilled															
1/22/24	14:59				7.875				39	-0.1	4.1											
1/22/24	15:02				8.25			3	42	0.3	3.8	-0.375	4	0	31	31	10	628	311	0.9	1.86	1.86
1/22/24	15:05				8.5			3	45	0.5	3.5	-0.25	4	0	21	21	7	418	298	0.9	1.31	1.29
1/22/24	15:10				9.125			5	50	1.1	2.9	-0.625	3	0	52	52	10	628	280	0.9	2.58	2.07
1/22/24	15:15				9.5			5	55	1.5	2.5	-0.375	3	0	31	31	6	377	258	0.9	1.73	1.34
1/22/24	15:20				9.75			5	60	1.8	2.3	-0.25	2	0	21	21	4	251	245	0.9	1.26	0.94
1/22/24	15:25				10.125			5	65	2.1	1.9	-0.375	2	0	31	31	6	377	232	0.9	2.38	1.50
							Refilled															
1/22/24	15:29				8.5				69	0.5	3.5											
1/22/24	15:32				8.875			3	72	0.9	3.1	-0.375	3	0	31	31	10	628	285	0.9	2.26	2.03
1/22/24	15:38				9.25			6	78	1.3	2.8	-0.375	3	0	31	31	5	314	269	0.9	1.30	1.08
1/22/24	15:42				9.5			4	82	1.5	2.5	-0.25	3	0	21	21	5	314	256	0.9	1.41	1.13
1/22/24	15:46				9.75			4	86	1.8	2.3	-0.25	2	0	21	21	5	314	245	0.9	1.58	1.18
1/22/24	15:49				10			3	89	2.0	2.0	-0.25	2	0	21	21	7	418	234	0.9	2.39	1.65
																				Minimum Rate:		0.49
														Raw Rate for design, prior to application of adjustment factors:								**1.46**

Results of Falling Head Infiltration Test

Project: 19894

Exploration #/Location: IT-2

Depth Test Pit, bgs (ft): 2

Tested by: AA

USCS Soil Type in test zone: Weathered Bedrock

Weather (start to finish): Cloudy

Water Source/pH: Tap Water

Equivalent boring diameter: 19.182 in. 9.591223 in. Well Radius

Depth to GW or aquitard, bgs: 100 ft

Well Prep: Bottom of test pit at approximately 2 feet below the ground surface with 6 inch depth infiltration pit, Set 6 inch diameter perforated pipe with 3/4" gravel around anulus

Initial estimated Depth to Water Surface (in.): -4

Average depth of water in well, "h" (in.): 3

approx. h/r: 0.4

Tu (Fig. 8) (ft): 100.4

Tu>3h?: yes, OK

Open Pit Dimensions

Length 17 in

Width 17 in

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity 0.5

Casing outer diameter, in. 6.5

Casing inner diameter, in. 6

Cross-sectional area, in.^2 156.2

Use of Barrels: No

Use of Flow Meter: No

Test Type: Falling Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surface)	0. ft	5.5 in.	5.5
Casing stickup measured above top of auger (or ground surface) (+ is up	0. ft	6.5 in.	6.5
Depth to top of sand from top of casing	0. ft	0. in.	

Depth of well bottom below top of casing (in): 12

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of casing)	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	Vol Change (in.^3): from Δh	Vol Change (in.^3): 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:	Gallons		ft / in.																	
1/25/2024	12:00																				
1/25/24	12:08			6.875				8	0.4	5.1											
1/25/24	12:10			7.25			2	10	0.8	4.8	-0.375	5	0	59	59	29	1757	587	0.9	2.88	2.76
1/25/24	12:12			7.5			2	12	1.0	4.5	-0.25	5	0	39	39	20	1171	568	0.9	2.01	1.90
1/25/24	12:15			7.75			3	15	1.3	4.3	-0.25	4	0	39	39	13	781	553	0.9	1.42	1.30
1/25/24	12:20			8			5	20	1.5	4.0	-0.25	4	0	39	39	8	469	538	0.9	0.91	0.80
1/25/24	12:25			8.375			5	25	1.9	3.6	-0.375	4	0	59	59	12	703	519	0.9	1.54	1.25
1/25/24	12:30			8.5			5	30	2.0	3.5	-0.125	4	0	20	20	4	234	504	0.9	0.52	0.43
1/25/24	12:40			8.75			10	40	2.3	3.3	-0.25	3	0	39	39	4	234	492	0.9	0.57	0.44
1/25/24	12:50			9.125			10	50	2.6	2.9	-0.375	3	0	59	59	6	351	474	0.9	0.99	0.68
1/25/24	13:00			9.25			10	60	2.8	2.8	-0.125	3	0	20	20	2	117	458	0.9	0.33	0.24
1/25/24	13:10			9.375			10	70	2.9	2.6	-0.125	3	0	20	20	2	117	451	0.9	0.35	0.24
1/25/24	13:20			9.675			10	80	3.2	2.3	-0.3	2	0	47	47	5	281	438	0.9	0.98	0.59
1/25/24	13:30			9.875			10	90	3.4	2.1	-0.2	2	0	31	31	3	187	423	0.9	0.70	0.41
1/25/24	13:40			10			10	100	3.5	2.0	-0.125	2	0	20	20	2	117	413	0.9	0.46	0.26
					Poor Rates Concluded Test																
																		Minimum Rate:			0.24
																		Raw Rate for design, prior to application of adjustment factors:			**0.50**

Results of Falling Head Infiltration Test

Project: 19894

Exploration #/Location: IT-4

Depth Test Pit, bgs (ft): 3

Tested by: AA

USCS Soil Type in test zone: SP-SM

Weather (start to finish): Rain

Water Source/pH: Tap Water

Equivalent boring diameter: 16.116 in. — 8.058239 in. Well Radius

Depth to GW or aquitard, bgs: 100 ft

Well Prep: Bottom of test pit at approximately 3 feet below the ground surface with 7 inch depth infiltration pit, Set 6 inch diameter perforated pipe with 3/4" gravel around anulus

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surface)	0. ft	7. in.	7
Casing stickup measured above top of auger (or ground surface) (+ is up	0. ft	5. in.	5
Depth to top of sand from top of casing	0. ft	0. in.	

Initial estimated Depth to Water Surface (in.): -2

Average depth of water in well, "h" (in.): 4

approx. h/r: 0.5

Tu (Fig. 8) (ft): 100.2

Tu>3h?: yes, OK

Open Pit Dimensions

Length 12 in

Width 17 in

Depth of well bottom below top of casing (in): 12

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity	0.5
Casing outer diameter, in.	6.5
Casing inner diameter, in.	6
Cross-sectional area, in.^2	113.7

Use of Barrels:	No
Use of Flow Meter:	No
Test Type:	Falling Head

Field Data | **Calculations**

Date	Time	Data from Flow Meter		Depth to WL in Boring (measured from top of casing)		Water Temp (deg F)	Refilled?	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)			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)
		Reading (gallons)	Interval Pulse Count				(or Comments)							from supply	from Δh	Total						
Start Date	Start time:																					
1/25/2024	13:50	Gallons		ft	in.																	
1/25/24	13:56				6				6	1.0	6.0											
1/25/24	13:58				6.375			2	8	1.4	5.6	-0.375	6	0	43	43	21	1279	498	0.9	2.06	2.37
1/25/24	14:00				9.875			2	10	4.9	2.1	-3.5	4	0	398	398	199	11937	400	0.9	91.57	27.50
1/25/24	14:02				11.25			2	12	6.3	0.8	-1.375	1	0	156	156	78	4689	277	0.9	108.81	15.62
							Refilled															
1/25/24	14:03				4.75				13	-0.3	7.3											
1/25/24	14:05				7			2	15	2.0	5.0	-2.25	6	0	256	256	128	7674	514	0.9	16.41	13.76
1/25/24	14:07				8.5			2	17	3.5	3.5	-1.5	4	0	171	171	85	5116	419	0.9	15.81	11.25
1/25/24	14:09				9.75			2	19	4.8	2.3	-1.25	3	0	142	142	71	4263	350	0.9	21.81	11.24
1/25/24	14:11				11.125			2	21	6.1	0.9	-1.375	2	0	156	156	78	4689	283	0.9	86.85	15.27
							Refilled															
1/25/24	14:13				4.5				23	-0.5	7.5											
1/25/24	14:15				5.75			2	25	0.8	6.3	-1.25	7	0	142	142	71	4263	552	0.9	6.49	7.12
1/25/24	14:18				8			3	28	3.0	4.0	-2.25	5	0	256	256	85	5116	463	0.9	14.47	10.18
1/25/24	14:21				10			3	31	5.0	2.0	-2	3	0	227	227	76	4547	356	0.9	30.70	11.78
1/25/24	14:23				11.25			2	33	6.3	0.8	-1.25	1	0	142	142	71	4263	274	0.9	94.64	14.36
							Refilled															
1/25/24	14:26				4.375				36	-0.6	7.6											
1/25/24	14:29				6.5			3	39	1.5	5.5	-2.125	7	0	242	242	81	4832	536	0.9	9.10	8.31
1/25/24	14:32				8.675			3	42	3.7	3.3	-2.175	4	0	247	247	82	4945	427	0.9	17.55	10.67
1/25/24	14:35				10.375			3	45	5.4	1.6	-1.7	2	0	193	193	64	3865	329	0.9	32.73	10.82
1/25/24	14:37				11.5			2	47	6.5	0.5	-1.125	1	0	128	128	64	3837	258	0.9	148.23	13.72
							Refilled															
1/25/24	14:41				4.675				51	-0.3	7.3											
1/25/24	14:44				7			3	54	2.0	5.0	-2.325	6	0	264	264	88	5286	516	0.9	11.37	9.44
1/25/24	14:47				9			3	57	4.0	3.0	-2	4	0	227	227	76	4547	407	0.9	18.05	10.31
1/25/24	14:50				10.75			3	60	5.8	1.3	-1.75	2	0	199	199	66	3979	312	0.9	48.98	11.77
							Refilled															
1/25/24	14:53				4.75				63	-0.3	7.3											
1/25/24	14:56				6.75			3	66	1.8	5.3	-2	6	0	227	227	76	4547	520	0.9	8.98	8.05
1/25/24	14:59				8.675			3	69	3.7	3.3	-1.925	4	0	219	219	73	4377	421	0.9	15.12	9.58
1/25/24	15:02				10.375			3	72	5.4	1.6	-1.7	2	0	193	193	64	3865	329	0.9	32.73	10.82
1/25/24	15:04				11.375			2	74	6.4	0.6	-1	1	0	114	114	57	3410	261	0.9	89.27	12.05
							Refilled															
1/25/24	15:05				5				75	0.0	7.0											
1/25/24	15:08				7.125			3	78	2.1	4.9	-2.125	6	0	242	242	81	4832	505	0.9	10.56	8.83
1/25/24	15:11				8.875			3	81	3.9	3.1	-1.75	4	0	199	199	66	3979	407	0.9	14.55	9.02
1/25/24	15:14				10.75			3	84	5.8	1.3	-1.875	2	0	213	213	71	4263	315	0.9	54.01	12.49
							Refilled															
1/25/24	15:20				4.875				90	-0.1	7.1											
1/25/24	15:23				7.875			3	93	2.9	4.1	-3	6	0	341	341	114	6821	489	0.9	19.79	12.86
1/25/24	15:26				9.25			3	96	4.3	2.8	-1.375	3	0	156	156	52	3126	378	0.9	12.75	7.62
																			Minimum Rate:			2.37
														Raw Rate for design, prior to application of adjustment factors:								**10.40**

Results of Falling Head Infiltration Test

Project:	19894	Initial estimated Depth to Water Surface (in.):	-5
Exploration #/Location:	IT-5	Average depth of water in well, "h" (in.):	4
Depth Test Pit, bgs (ft):	2.75	approx. h/r:	0.5
Tested by:	AA	Tu (Fig. 8) (ft):	100.4
USCS Soil Type in test zone:	Weathered Bedrock Recovered as Clayey Sand	Tu>3h?:	yes, OK
Weather (start to finish):	Rain		
Water Source/pH:	Tap Water	Open Pit Dimensions	
Equivalent boring diameter:	13.541 in.	6.770275 in. Well Radius	Length 12 in
Depth to GW or aquitard, bgs:	100 ft		Width 12 in

Well Prep: Bottom of test pit at approximately 2.75 feet below the ground surface with 5.5 inch depth infiltration pit, Set 6 inch diameter perforated pipe with 3/4" gravel around anulus

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surface)	0. ft	5.5 in.	5.5
Casing stickup measured above top of auger (or ground surface) (+ is up	0. ft	6.5 in.	6.5
Depth to top of sand from top of casing	0. ft	0. in.	

Depth of well bottom below top of casing (in): 12

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity	0.5
Casing outer diameter, in.	6.5
Casing inner diameter, in.	6
Cross-sectional area, in.^2	83.7

Use of Barrels:	No
Use of Flow Meter:	No
Test Type:	Falling Head

Field Data / **Calculations**

Date	Time	Data from Flow Meter		Depth to WL in Boring (measured from top of casing)		Water Temp (deg F)	Refilled?	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)			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:	Reading (gallons)	Interval Pulse Count	ft	in.		(or Comments)							from supply	from Δh	Total						
1/22/24	9:00	Gallons																				
1/22/24	9:26				7.25				26	0.8	4.8											
1/22/24	9:28				7.5			2	28	1.0	4.5	-0.25	5	0	21	21	10	628	341	0.9	1.50	1.70
1/22/24	9:30				7.75			2	30	1.3	4.3	-0.25	4	0	21	21	10	628	330	0.9	1.60	1.75
1/22/24	9:32				8.25			2	32	1.8	3.8	-0.5	4	0	42	42	21	1255	314	0.9	3.77	3.68
1/22/24	9:34				8.25			2	34	1.8	3.8	0	4	0	0	0	0	0	304	0.9	0.00	0.00
1/22/24	9:38				8.5			4	38	2.0	3.5	-0.25	4	0	21	21	5	314	298	0.9	0.98	0.97
1/22/24	9:42				8.75			4	42	2.3	3.3	-0.25	3	0	21	21	5	314	288	0.9	1.07	1.01
1/22/24	9:46				9			4	46	2.5	3.0	-0.25	3	0	21	21	5	314	277	0.9	1.16	1.04
1/22/24	9:50				9.125			4	50	2.6	2.9	-0.125	3	0	10	10	3	157	269	0.9	0.60	0.54
1/22/24	9:55				9.5			5	55	3.0	2.5	-0.375	3	0	31	31	6	377	258	0.9	1.73	1.34
1/22/24	10:00				9.5			5	60	3.0	2.5	0	3	0	0	0	0	0	250	0.9	0.00	0.00
1/22/24	10:10				9.75			10	70	3.3	2.3	-0.25	2	0	21	21	2	126	245	0.9	0.63	0.47
1/22/24	10:20				9.875			10	80	3.4	2.1	-0.125	2	0	10	10	1	63	237	0.9	0.33	0.24
1/22/24	10:33				10.5			13	93	4.0	1.5	-0.625	2	0	52	52	4	241	221	0.9	2.09	1.01
							Refilled															
1/22/24	11:01				6.25				121	-0.3	5.8											
1/22/24	11:06				6.75			5	126	0.3	5.3	-0.5	6	0	42	42	8	502	378	0.9	1.04	1.22
1/22/24	11:11				6.875			5	131	0.4	5.1	-0.125	5	0	10	10	2	126	365	0.9	0.26	0.32
1/22/24	11:16				7.125			5	136	0.6	4.9	-0.25	5	0	21	21	4	251	357	0.9	0.55	0.65
1/22/24	11:30				7.75			14	150	1.3	4.3	-0.625	5	0	52	52	4	224	338	0.9	0.59	0.61
1/22/24	11:45				8.375			15	165	1.9	3.6	-0.625	4	0	52	52	3	209	311	0.9	0.66	0.62
1/22/24	11:53				8.5			8	173	2.0	3.5	-0.125	4	0	10	10	1	78	296	0.9	0.24	0.24
1/22/24	12:07				8.875			14	187	2.4	3.1	-0.375	3	0	31	31	2	134	285	0.9	0.49	0.44
							Poor Infiltration Concluded Test															
																			Minimum Rate:			0.00
																			Raw Rate for design, prior to application of adjustment factors:			**0.53**

Results of Falling Head Infiltration Test

Project: 19894

Exploration #/Location: IT-6

Depth Test Pit, bgs (ft): 2

Tested by: AA

USCS Soil Type in test zone: Weathered Bedrock

Weather (start to finish): Rain

Water Source/pH: Tap Water

Equivalent boring diameter: 13.541 in. 6.770275 in. Well Radius

Depth to GW or aquitard, bgs: 100 ft

Well Prep: Bottom of test pit at approximately 2 feet below the ground surface with 5 inch depth infiltration pit, Set 6 inch diameter perforated pipe with 3/4" gravel around anulus

Initial estimated Depth to Water Surface (in.): -5

Average depth of water in well, "h" (in.): 3

approx. h/r: 0.4

Tu (Fig. 8) (ft): 100.4

Tu>3h?: yes, OK

Open Pit Dimensions

Length 12 in

Width 12 in

Cross-sectional area for flow calcs based on Δh

Well pack sand porosity 0.5

Casing outer diameter, in. 6.5

Casing inner diameter, in. 6

Cross-sectional area, in.^2 83.7

Use of Barrels: No

Use of Flow Meter: No

Test Type: Falling Head

	ft	in.	Total (in.)
Depth to bottom of well measured from top of auger (or ground surface)	0. ft	5. in.	5
Casing stickup measured above top of auger (or ground surface) (+ is up	0. ft	7. in.	7
Depth to top of sand from top of casing	0. ft	0. in.	

Depth of well bottom below top of casing (in): 12

Field Data / **Calculations**

Date	Time	Data from Flow Meter: Reading (gallons)	Data from Flow Meter: Interval Pulse Count	Depth to WL in Boring (measured from top of casing) ft	Depth to WL in Boring (measured from top of casing) 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	Vol Change (in.^3): from Δh	Vol Change (in.^3): 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:	Gallons		ft	in.																	
1/22/24	8:30																					
1/22/24	8:48			10					18	3.0	2.0											
1/22/24	8:49			10				1	19	3.0	2.0	0	2	0	0	0	0	0	229	0.9	0.00	0.00
1/22/24	8:50			10.375				1	20	3.4	1.6	-0.375	2	0	31	31	31	1883	221	0.9	13.92	7.85
1/22/24	8:52			10.75				2	22	3.8	1.3	-0.375	1	0	31	31	16	941	205	0.9	9.35	4.23
							Refilled															
1/22/24	11:21			7.5					171	0.5	4.5											
1/22/24	11:22			8.5				1	172	1.5	3.5	-1	4	0	84	84	84	5021	314	0.9	17.30	14.73
1/22/24	11:24			9.25				2	174	2.3	2.8	-0.75	3	0	63	63	31	1883	277	0.9	8.29	6.27
1/22/24	11:25			9.5				1	175	2.5	2.5	-0.25	3	0	21	21	21	1255	256	0.9	5.64	4.53
1/22/24	11:26			9.875				1	176	2.9	2.1	-0.375	2	0	31	31	31	1883	242	0.9	10.35	7.16
1/22/24	11:28			10				2	178	3.0	2.0	-0.125	2	0	10	10	5	314	232	0.9	1.74	1.25
							Refilled															
1/22/24	11:33			7.5					183	0.5	4.5											
1/22/24	11:35			8				2	185	1.0	4.0	-0.5	4	0	42	42	21	1255	325	0.9	3.51	3.56
1/22/24	11:37			8				2	187	1.0	4.0	0	4	0	0	0	0	0	314	0.9	0.00	0.00
1/22/24	11:40			9.125				3	190	2.1	2.9	-1.125	3	0	94	94	31	1883	290	0.9	8.32	5.98
1/22/24	11:43			9.375				3	193	2.4	2.6	-0.25	3	0	21	21	7	418	261	0.9	1.79	1.48
1/22/24	11:45			9.875				2	195	2.9	2.1	-0.5	2	0	42	42	21	1255	245	0.9	7.08	4.72
1/22/24	11:50			10.25				5	200	3.3	1.8	-0.375	2	0	31	31	6	377	226	0.9	2.56	1.53
							Refilled															
1/22/24	11:56			7.5					206	0.5	4.5											
1/22/24	11:58			8				2	208	1.0	4.0	-0.5	4	0	42	42	21	1255	325	0.9	3.51	3.56
1/22/24	12:00			8.125				2	210	1.1	3.9	-0.125	4	0	10	10	5	314	311	0.9	0.87	0.93
1/22/24	12:05			8.875				5	215	1.9	3.1	-0.75	4	0	63	63	13	753	293	0.9	2.86	2.37
1/22/24	12:10			9.5				5	220	2.5	2.5	-0.625	3	0	52	52	10	628	264	0.9	3.02	2.19
1/22/24	12:15			9.875				5	225	2.9	2.1	-0.375	2	0	31	31	6	377	242	0.9	2.07	1.43
1/22/24	12:20			10.5				5	230	3.5	1.5	-0.625	2	0	52	52	10	628	221	0.9	5.45	2.62
							Refilled															
1/22/24	12:54			7.5					264	0.5	4.5											
1/22/24	12:56			7.75				2	266	0.8	4.3	-0.25	4	0	21	21	10	628	330	0.9	1.60	1.75
1/22/24	13:00			8.125				4	270	1.1	3.9	-0.375	4	0	31	31	8	471	317	0.9	1.34	1.37
1/22/24	13:06			9				6	276	2.0	3.0	-0.875	3	0	73	73	12	732	290	0.9	2.97	2.33
1/22/24	13:10			9.25				4	280	2.3	2.8	-0.25	3	0	21	21	5	314	266	0.9	1.27	1.09
1/22/24	13:15			9.75				5	285	2.8	2.3	-0.5	3	0	42	42	8	502	250	0.9	2.66	1.85
1/22/24	13:22			10.375				7	292	3.4	1.6	-0.625	2	0	52	52	7	448	226	0.9	3.54	1.83
							Refilled															
1/22/24	13:45			7.25					315	0.3	4.8											
1/22/24	13:47			7.5				2	317	0.5	4.5	-0.25	5	0	21	21	10	628	341	0.9	1.50	1.70
1/22/24	13:50			7.75				3	320	0.8	4.3	-0.25	4	0	21	21	7	418	330	0.9	1.06	1.17
1/22/24	13:57			8.5				7	327	1.5	3.5	-0.75	4	0	63	63	9	538	309	0.9	1.80	1.61
1/22/24	14:02			8.875				5	332	1.9	3.1	-0.375	3	0	31	31	6	377	285	0.9	1.36	1.22
1/22/24	14:10			9.875				8	340	2.9	2.1	-1	3	0	84	84	10	628	256	0.9	3.91	2.26
1/22/24	14:13			10				3	343	3.0	2.0	-0.125	2	0	10	10	3	209	232	0.9	1.16	0.83

EEI 2006 LOGS

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BOREHOLE LOG

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Number: B-2

Client: Shopoff Group

Sheet: 1 of 1

Location: Meadowbrook Subdivision
Lake Elsinore/Perris, Riverside County, California

Date Started: 4/11/06

Project Number: P051109-1

EEI Representative: E.L.

Drill Rig/Sampling Method:
CalPac Drilling, HSA, 140 lb Auto-hammer with 30" drop,
California Modified Sampler

Borehole Diameter: 8"

SAMPLE LOG					BOREHOLE LOG			
Bulk	Undisturbed	Blows Per 6 Inches	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					0	SM		**0' - +/- 4' - <u>ALLUVIUM</u>** – Silty-**SAND**, dark brown, moist, loose, porous, common organics.
					1			
					2			
	MOD	2/1	114.0	5.9	3			**3'** – Silty-**SAND**, dark brown, moist, loose, porous, common organics.
					4			
	MOD	9/24	132.0	5.9	5			**5'** – Weathered **<u>BEDROCK</u> (Kgr)** – Decomposed granitic **ROCK (DG)**, gray and black, moist, dense, micaceous, easy excavating creates medium to coarse grain sand.
					6			
					7			
					8			
					9			
	MOD	23/40	137.8	5.2	10	Kgr		**10'** – Decomposed granitic **ROCK (DG)**, black and gray, slightly moist, dense, highly mafic, micaceous.
					11			
					12			**+/- 13'** – Groundwater Encountered
					13			
					14			
	MOD	50 for 5"	125.2	10.8	15			**15'** – Decomposed granitic **ROCK (DG)**, black and white, coarse-grained, moist, dense, highly mafic, massive.
					16			
					17			
					18			
					19			
					20			
					21			
					22			
					23			
					24			
					25			
					26			
					27			
					28			
					29			
					30			
					31			
					32			
					33			
					34			
					35			
					36			
					37			

Total Depth of 15½'
Groundwater Encountered +/- 13' (Stabilized to 9')
Hole Backfilled with Native Soils
MOD = California Modified Sampler
NA = Not Available

BOREHOLE LOG

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Number: B-3

Client: Shopoff Group

Sheet: 1 of 1

Location: Meadowbrook Subdivision
Lake Elsinore/Perris, Riverside County, California

Date Started: 4/11/06

Project Number: P051109-1

EEI Representative: E.L.

Drill Rig/Sampling Method:
CalPac Drilling, HSA, 140 lb Auto-hammer with 30" drop,
California Modified Sampler

Borehole Diameter: 8"

SAMPLE LOG					BOREHOLE LOG			
Bulk	Undisturbed	Blows Per 6 Inches	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					0 1 2	SM		**0'-1'** - **TOPSOIL** – Silty-**SAND**, brown, moist, loose.
	MOD	20/32	119.6	7.5	3 4			**2'** – **BEDROCK (Kgr)** **3'** – Weathered **BEDROCK** – Decomposed granitic **ROCK (DG)**, orange and brown, dry, dense, mafic, friable, excavates to sand.
	MOD	21/44	119.1	3.2	5 6 7 8 9			**5'** – Weathered granitic **ROCK**, orange and brown, dry, dense, mafic, friable, massive, easy excavating.
	MOD	20/45	94.5	4.3	10 11 12 13 14	Kgr		**10'** – Weathered granitic **ROCK (DG)**, brownish-gray, dry, dense, mafic, massive.
	MOD	30/50 for 4"	115.0	2.8	15 16 17 18 19			**15'** – Weathered granitic **ROCK (DG)**, gray, dry, dense, mafic, moderately easy drilling.
					20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37			Total Depth of 20' No Groundwater Encountered Hole Backfilled with Native Soils MOD = California Modified Sampler NA = Not Available

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BOREHOLE LOG

Number: B-4A

Client: Shopoff Group

Sheet: 1 of 1

Location: Meadowbrook Subdivision
Lake Elsinore/Perris, Riverside County, California

Date Started: 4/11/06

Project Number: P051109-1

EEI Representative: E.L.

Drill Rig/Sampling Method:
CalPac Drilling, HSA, 140 lb Auto-hammer with 30" drop,
California Modified Sampler

Borehole Diameter: 8"

SAMPLE LOG						BOREHOLE LOG		
Bulk	Undisturbed	Blows Per 6 Inches	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					0	SM		**0'- +/-2' - TOPSOIL** – Clayey-**SAND**, brown, moist, pliable.
					1			
					2			**2'-10' – ALLUVIUM (Qal)**
	MOD	22/50 for 4"	115.4	12.5	3			**3'** – Silty-**SAND**, brown and orange, with small gravel, slightly moist, medium dense, porous.
					4			
	MOD	12/21	123.0	10.3	5			**5'** – Silty-**SAND**, gray-brown, with small to coarse gravel, slightly moist to moist, hard, brittle, porous.
					6			
					7			
					8			
					9			
	MOD	16/50 for 5"	118.2	12.5	10	Kgr/Jm		**+/-10'- 12' - BEDROCK**
					11			
					12			**10'** – Weathered metamorphic rock, excavates to sandy-gravel and gravelly-sand light brownish-gray, slightly moist, hard, brittle.
					13			
					14			**10½' – BEDROCK (Jm)**, gray and brown, gravel, dry, dense, weathered metamorphic rock, excavates to sandy-silt.
					15			
					16			
					17			
					18			
					19			
					20			Practical Refusal at 12'
					21			No Groundwater Encountered
					22			Hole Backfilled with Native Soils
					23			MOD = California Modified Sampler
					24			NA = Not Available
					25			
					26			
					27			
					28			
					29			
					30			
					31			
					32			
					33			
					34			
					35			
					36			
					37			

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BOREHOLE LOG

Number: B-5

Client: Shopoff Group

Sheet: 1 of 1

Location: Meadowbrook Subdivision
Lake Elsinore/Perris, Riverside County, California

Date Started: 4/11/06

Project Number: P051109-1

EEI Representative: E.L.

Drill Rig/Sampling Method:
CalPac Drilling, HSA, 140 lb Auto-hammer with 30" drop,
California Modified Sampler

Borehole Diameter: 8"

SAMPLE LOG						BOREHOLE LOG		
Bulk	Undisturbed	Blows Per 6 Inches	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					0			**0'- +/-2' - <u>TOPSOIL</u>** – Silty-**SAND**, orange-brown, occasional gravel, dry, loose.
					1			
					2	SM		
					3			**2'-5' – <u>BEDROCK</u> (Jm)**
	MOD	50 for 5"	NA	NA	NR	Jm		**3'** – Weathered metamorphic **<u>BEDROCK</u> (Jm)**, gray and brown, gravel, dry, dense, platy cleavage, excavates to sandy gravel (no recovery).
					4			
					5			**5'** – Practical Refusal. Bedrock excavates to gravelly sand, dry.
					6			
					7			
					8			
					9			
					10			
					11			
					12			
					13			
					14			
					15			
					16			
					17			
					18			
					19			
					20			
					21			
					22			
					23			
					24			
					25			
					26			
					27			
					28			
					29			
					30			
					31			
					32			
					33			
					34			
					35			
					36			
					37			

Practical Refusal at 5'
No Groundwater Encountered
Hole Backfilled with Native Soils
NR = No Recovery
MOD = California Modified Sampler
NA = Not Available

BOREHOLE LOG

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Number: B-6

Client: Shopoff Group

Sheet: 1 of 1

Location: Meadowbrook Subdivision
Lake Elsinore/Perris, Riverside County, California

Date Started: 4/11/06

Project Number: P051109-1

EEI Representative: E.L.

Drill Rig/Sampling Method: CalPac Drilling, HSA, 140 lb Auto-hammer with 30" drop, California Modified Sampler

Borehole Diameter: 8"

SAMPLE LOG						BOREHOLE LOG		
Bulk	Undisturbed	Blows Per 6 Inches	Dry Unit Wt. (pcf)	Moisture (%)	Depth In Feet	USCS Symbol	Graphic Log	Geologic Description (Soil Type, Color, Grain, Minor Soil Component, Moisture, Density, Odor, Etc.)
					0			**0'-22'** – Weathered **BEDROCK (Jm)**
					1			
					2			
	MOD	30/50 for 4"	131.6	2.3	3			**3'** – Weathered **BEDROCK (Jm)**, gray with orange-brown, dry, dense, fractured, weathered metamorphic rock, oxidized, moderately difficult drilling creates gravelly sand.
					4			
	MOD	35/50 for 2"	NA	3.7	5	PR		**5'** – Weathered metamorphic **ROCK (Jm)**, gray with orange-brown, dry, dense, fractured, oxidized (poor recovery).
					6			
					7			
					8			Stopped sampling, but drilled to find practical refusal on ridge.
					9	Jm		
					10			
					11			**5'-22'** – Difficult drilling, generates sandy-gravel and gravelly sand, with minor clay, dry, dense.
					12			
					13			
					14			
					15			
					16			
					17			
					18			
					19			
					20			
					21			
					22			
					23			Practical Refusal at 22'
					24			No Groundwater Encountered
					25			Hole Backfilled with Native Soils
					26			PR = Poor Recovery
					27			MOD = California Modified Sampler
					28			NA = Not Available
					29			
					30			
					31			
					32			
					33			
					34			
					35			
					36			
					37			

TP-5A

0' - ½' **Topsoil**- brown, silty-**SAND** (SM), loose, porous, slightly organic, moist

½' - 1' **Highly Weathered Bedrock (Kgr)** - orange brown, highly decomposed GRANITIC rock (D.G.), hard, friable, porous, excavates to clayey-**SAND** (SC), slightly moist.

1' - 5' **Weathered Bedrock (Kgr)** - orange brown and gray and black, decomposed GRANITE rock(D.G.) medium dense, massive, moderately friable, mafic, micaceous, slightly moist.

T.D. @ 5'
Test Pit Backfilled
N 33° 43.228'
W 117° 16.626

TP-6A

0' - ½' **Highly Weathered Bedrock (Kgr)**- orange brown, highly decomposed GRANITE rock (D.G.), hard, friable, porous, excavates to clayey-**SAND** (SC), slightly moist.

½' - 5½' **Weathered Bedrock (Kgr)**- orange brown and gray and black, decomposed GRANITIC rock(D.G.) medium dense, massive, moderately friable, mafic, micaceous, slightly moist.

T.D. @ 5½'
Test Pit Backfilled
N 33° 43.228'
W 117° 16.626

TP-7

0' - 1' **Topsoil** - brown silty sand(SM), loose, porous, brittle to friable, common rootlets, slightly moist.

1' - 3' **Colluvium** - brown to gray with depth, clayey- **SAND** (SC), hard, brittle, porous, blocks, relict rootlets, slightly moist, relict granitic structure 0' to 3'.

3' - 4' **Weathered Bedrock (Kgr)** - orange-black-olive gray, decomposed GRANITIC rock (D.G.), dense, massive, highly mafic, slightly friable, excavates to gravely SAND (SW), slightly moist.

T.D. @ 4'
Test Pit Backfilled
N 33° 42.897'
W117° 17.105'

TP-8

0' - 1½' **Topsoil**- brown, silty-**SAND** and sandy-**SILT** (ML/SM), loose, brittle, blocky, porous, occasional rootlets and roots, moist.

1½' - 3' **Weathered Bedrock (Kgr)** - gray and brown, decomposed GRANITIC rock (D.G.), dense, slightly friable, moisture, excavated to medium to coarse grain **SAND** with gravel (SP), mafic, occasional hard rock fragments floating in D.G. mass, slightly moist.

T.D. @ 3'
Test Pit Backfilled
N 33° 42.901'
W 117° 17.058'

TP-9A

0' - ½' **Topsoil** - orange brown, silty-**SAND** (SM), loose, porous, slightly moist.

½' - 3½' **Weathered Bedrock (Kgr)** - light brown and light gray, decomposed GRANITIC rock (D.G.), medium dense, moderately friable, massive, mafic, slightly moist, excavates to coarse grain **SAND** (SP).

T.D. @ 3½'
Test Pit Backfilled
N 33° 43.136'
W 17° 16.706'

TP-10

0' - 1½' **Topsoil** - red brown, silty-**SAND** (SM), loose, porous, friable, organic, moist.

1½' - 3' **Slopewash** - orange brown, silty-**SAND** with gravel (SM), loose to slightly dense, porous, occasional rootlets, angular rock fragments up to + 6" - in one dimension, slightly moist.

3' - 4' **Weathered Bedrock (Jm)** - gray and orange brown, weathered METAMORPHIC rock, medium dense to dense, difficult to excavate, blocky cleavage, excavated to rock fragments ranging from ½" to 4" in one dimension, oxidized, slightly moist.

T.D. @ 4'
Test Pit Backfilled
N 33° 42.916'
W 117° 16.902'

TP-11

0' - ½' **Topsoil** - orange brown, silty-**SAND** (SM), loose, porous, friable, organic, with occasional rootlets, occasional gravel, slightly moist.

½' - 3' **Weathered Bedrock (Jm)** - brown and gray, weathered METAMORPHIC rock, dense, brittle, blocky cleavage, difficult to excavate, angular rock fragments up to 8" in one dimension, oxidized fractured surface, dry.

Practical Refusal @ 3½'
Test Pit Backfilled
N 33° 42.901'
W 117° 16.780'

TP-12

0' - ½' **Topsoil** - orange brown, silty-**SAND** (SM), loose, porous, friable, organic, with occasional rootlets, occasional gravel, slightly moist.

½' - 3' **Slopewash** - silty-**SAND** and sandy-**SILT** (SM/ML), loose, porous, occasional rootlets and rock fragments, slightly moist.

3' - 4' **Weathered Bedrock (Jm)**- orange brown and gray, weathered METAMORPHIC rock, dense, fragmented, oxidized, difficult excavating creates angular rock fragments up to 4" in one dimension, dry, highly decomposed from 3' - 3½' with minor clayey-**GRAVEL** (GW).

Practical Refusal @ 4'
Test Pit Backfilled
N 33° 42.913'
W 117° 16.733'

TP-13

0' - ½' **Topsoil** - orange brown, silty-**SAND** (SM), loose, porous, friable, organic, with occasional rootlets, occasional gravel, slightly moist.

½' - 3½' **Weathered Bedrock(Jm)**- brown and gray, weathered METAMORPHIC rock, dense, brittle, blocky cleavage, difficult to excavate, angular rock fragments up to 8" in one dimension, oxidized fractured surface, dry.

T.D. @ 3½'
Test Pit Backfilled
N 33° 42.946'
W 117° 16.787'

TP-14

0' - 1½' **Topsoil**- brown, sandy-**SILT** (ML), loose, porous, organic, moist.

1½' - 3' **Weathered Bedrock (Jm)** - gray and orange brown on fracture surface, weathered METAMORPHIC rock, dense, hard and blocky, excavates to angular rock fragments up to 4" in one dimension, dry.

T.D. @ 3'
Test Pit Backfilled
N 33° 42.904'
W 117° 16.704'

TP-15

0' - ½' **Topsoil**- orange brown, silty-**SAND** (SM), loose, porous, organic, moist.

½' - 1' **Colluvium**- orange brown, clayey-**SAND** (SC) with gravel, brittle, hard, porous, organic, moist.

1' - 2½' **Weathered Bedrock (Kgr)** - orangish brown and gray, weathered GRANITE and METAMORPHIC rock, dense, excavates to sandy (D.G.) gravel (SW),metamorphic, friable, dry.

Practical Refusal @ 2½'
Test Pit Backfilled
N 33° 42.998'
W 117° 16.794'

TP-16A

0' - ½' **Topsoil-** orange brown, silty- **SAND** (SM), loose, porous, slightly moist.

½' -3½' **Weathered Bedrock (Kgr)** - light brown and light gray, decomposed, GRANITE rock, medium dense, moderately friable, massive, mafic, slightly moist, excavated to coarse grain sand.

T.D. @ 4'
Test Pit Backfilled
N 33° 43.12'
W 117° 16.730'

TP-17A

0' - ½' **Topsoil** - brown silty-**SAND** (SM), loose, porous, occasional roots, occasional animal burrows, moist.

½' - 5' **Weathered Bedrock (Kgr)** - orange brown and gray, weathered bedrock (D.G.), medium dense, friable, massive, easy excavating creates medium to coarse grained **SAND** (SP), occasional relief fractures, colored white, slightly moist.

T.D. @ 5'
Test Pit Backfilled
N 33° 43.107'
W 117° 16.586'

TP-18

0' - ½' **Topsoil** - brown silty-**SAND** (SM), loose, porous, moist.

1½' - 4' **Colluvium**- orange brown, silty-**SAND** and clayey-**SAND** (SM/SC), porous, brittle, blocky, occasional rock fragments, slightly moist.

4' - 4½' **Weathered Bedrock (Kgr)** - orange brown, decomposed GRANITIC rock (D.G.), medium dense, sightly friable, massive, mafic, oxidized, slightly moist

T.D. @ 4½'
Test Pit Backfilled
N 33° 43. 101'
W 117° 16.768'

TP-19

0' - 5' **Alluvium**- dark gray, and brown, silty-**SAND** (SM), loose, porous, occasional roots, moist, minor sloughing.

5' - 5½' **Weathered Bedrock (kgr)** - gray and white, decomposed GRANITIC rock (D.G.), medium dense, friable, mafic, excavates to clayey-**SAND** (SC), moist.

T.D. @ 5½'
Test Pit Backfilled
N 33° 43.123'
W117° 16.838'

TP-20

0' - ½' **Topsoil** - dark brown, sandy-**SILT** and silty-**SAND** (SM/ML) with minor clay, loose, porous, occasional rootlets, very moist.

½' - 3' **Weathered Bedrock (Kgr)** - orange brown and gray black, decomposed GRANITIC rock (D.G.), dense, friable massive, mafic, excavates to coarse grain **SAND** (SP), slightly moist to dry with depth.

T.D. @ 3'
Test Pit backfill
N 33° 43.007'
W 117° 16.972'

TP-21

0' - ½' **Topsoil** - dark brown, sandy-**SILT** (ML) and silty-**SAND** (SM) with minor clay, loose, porous, occasional rootlets, very moist.

½' - 1' **Colluvium** - dark brown, sandy-**CLAY** (CL), brittle, porous, blocky, common root hairs, moist.

1' - 2½' **Highly Weathered Bedrock (Kgr)** - orange brown, highly decomposed GRANITIC rock (D.G.), excavated to sandy-**CLAY** and clayey-**SAND** (CL/SC), massive, slightly moist.

2½' - 4' **Weathered Bedrock**- gray, change to brown decomposed GRANITIC rock (D.G.), medium dense to dense, massive, slightly friable, mafic, excavated to coarse grain **SAND** (SP), dry.

T.D. @ 4'
Test Pit Backfilled
N 33° 43. 057'
W 117° 17.024'

TP-22

0' - ½' **Topsoil** - orange brown, silty-**SAND** (SM), loose, porous, moist.

½' - 4' **Weathered Bedrock (Kgr)** - gray and black & white & orange, decomposed GRANITE rock (D.G.), very friable, massive, excavates to coarse-grain **SAND** (SP), moist.

T.D. @ 4'
Test Pit Backfilled
N 33° 43.231'
W 117° 17.045'

TP-23

0' - ½' **Topsoil** - brown, silty-**SAND** (SM), loose, porous, moist.

½' - 3½' **Weathered Bedrock (Kgr)** - orange brown and gray, decomposed GRANITIC rock (D.G.), medium dense, friable massive, oxidized, slightly moist.

3½' - 6½' Dark grey & white & brown, decomposed GRANITIC rock (D.G.), medium dense, friable, massive, mafic, excavates to medium to coarse grain **SAND** (SP), slightly moist.

T.D. @ 6'
Test Pit Backfilled
N 33° 43.181'
W117° 16.646'

TP-24

0' - ½' **Topsoil** - brown, silty-**SAND** (SM), loose, porous, slightly organic, moist.

½' - 5' **Weathered Bedrock (Kgr)** - orange brown and gray and black, decomposed GRANITE rock (D.G.), medium dense, massive, moderately friable, mafic, easy excavating creates medium to coarse grain **SAND** (SP), micaceous, slightly moist.

T.D. @ 5'
Test Pit Backfilled
N33° 43.153'
W117° 16. 586'

TP-25

0' - 1' **Topsoil** - red brown, silty-**SAND** (SM), loose, porous, organic, friable, moist.

1' - 6' **Weathered Bedrock (Kgr)** - orange brown and gray brown, decomposed GRANITIC rock (D.G.), friable, massive, medium dense, occasional relict fractures are white & crumbly, oxidized, medium to coarse grain **SAND** (SP) mafic, dry.

T.D. @ 6'
Test Pit Backfilled
N 33° 43. 087'
W 117° 16.500'

TP-26

0' - ½' **Topsoil** - brown, silty-**SAND** (SM), loose, porous, occasional roots, occasional animal burrows, moist.

½' - 4' **Weathered Bedrock (Kgr)** - orange brown and gray, decomposed GRANITE rock (D.G.), dense, slightly friable, micaceous, mafic, moderately difficult excavating creates **SAND** (SP), slightly moist.

T.D. @ 4'
Test Pit Backfilled
N 33° 43. 104'
W 117° 16.527'

TP-27

0' - 1' **Topsoil** - red brown, silty-SAND (SM), loose, porous, organic, friable. moist.

1' - 5' **Weathered Bedrock (Kgr)** - orange brown & gray brown, weathered GRANITIC rock (D.G.), friable massive, medium dense, micaceous, mafic, dry.

T.D. @ 5'
Test Pit Backfilled
N 33° 43. 047'
W 117° 16.634'

TP-28

0' - 1½' **Weathered Bedrock (Jm)** - brown, orange and gray, weathered METAMORPHIC rock, dense, fractured, oxidized, difficult excavating creates sandy-**GRAVEL** (GW) with angular fragments up to 6" in one direction, slightly moist.

Practical Refusal @ 1½'
Test Pit Backfilled
N 33° 43.045'
W 117° 16.535'

TP-29

0' - 1' **Topsoil** - brown, silty-**SAND** (SM), loose, porous, organic, moist.

1' - 2' **Slopewash** - medium brown, sandy-**CLAY** (CL), brittle, porous, blocky cleavage, slightly moist, well cemented, poorly sorted.

2' - 4' **Weathered Bedrock (Kgr)**- gray brown, weathered metamorphic rock, medium dense to dense, moderately difficult excavating creates sandy gravel (GP), with fragments up to 5" in one direction, dry.

T.D. @ 4'
Test Pit Backfilled
N 33° 42. 968'
W 117° 16. 656'

TP-30

0' - 2' **Weathered Bedrock (Jm)** - brown, orange and gray, weathered METAMORPHIC rock, dense, fractured, oxidized, difficult excavating creates sandy-**GRAVEL** (GW), angular fragments up to + 6" in one direction, slightly moist.

T.D. @ 2'
Test Pit Backfilled
N 33° 42.971'
W 117° 16. 539'

TP-31

0' - ½' **Topsoil** - orange brown, silty-**SAND** (SM), loose, porous, organic, moist.

½' - 4½' **Slopewash** - orange brown, silty-**SAND** (SM) with angular gravel, loose to slightly dense, porous, occasional rootlets, moist.

4½' - 5½' **Weathered Bedrock (Jm)** - orange brown & gray, weathered METAMORPHIC rock, dense, fractured, oxidized, difficult excavating creates sandy-GRAVEL (GW), angular fragments up to 6" in one dimension, slightly moist to dry.

Practical Refusal @ 5½'
Test Pit Backfilled
N 33° 42.872'
W 117° 16. 534'

SOIL EXPLORATION COMPANY 2021 LOGS

Leighton
a verdantas company

EXPLORATORY TRENCH LOG

PROJECT NAME Cemetery ELEVATION _____ TRENCH NO. T-1

PROJECT No. 1789-01 EQUIPMENT Case 580 w/ 24-inch bucket

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY CK DATE 12-17-20** **SAMPLED BY CK**
5							**BEDROCK- Decomposed Granite (Kgr)** Dark grey, medium to coarse grained, dense, Damp, generally massive structure, few fractures Excavates into coarse sand that is friable and Non-cohesive, trench walls are stable, Practical refusal at 10 ft.
10							Trench Backfilled No Groundwater Practical Refusal @ 10 ft.
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1566’	TRENCH NO.	T-2
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							TOPSOIL – Silty Clayey Sand Medium grey-brown, fine to medium grained, Loose, dry, porous, easily crumbles in hand, Rootlets & organic material to 3%-5%.
5							**BEDROCK- Granite (Kgr)** Very hard (fresh), non-excavatable generally massive structure Practical refusal at 3 ft.
10							Trench Backfilled No Groundwater Practical Refusal @ 3 ft.
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1566 ft	TRENCH NO.	T-3
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							BEDROCK- Decomposed Granite (Kgr)
							Light grey to tan brown, coarse grained, dense,
							Dry, massive structure, few fractures
							Excavates into coarse sand that is friable and
5							Non-cohesive, Practical refusal at 5 ft.
							Trench Backfilled
							No Groundwater
							No Caving
							Practical Refusal @ 5 ft
10							
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Cemetery	ELEVATION		TRENCH NO.	T-4
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

GEOTECHNICAL DESCRIPTION

LOGGED BY CK **DATE** 12-17-20
SAMPLED BY CK

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	
0							
5							**BEDROCK- Decomposed Granite (Kgr)** Light grey to tan brown, coarse grained, dense, Dry, massive structure, few fractures Excavates into coarse sand that is friable and Non-cohesive, vertical fractures @ 24" spacing With calcium carbonate infill
10							Practical Refusal @ 10 ft. No Groundwater No Caving Trench Backfilled
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1593’	TRENCH NO.	T-5
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							TOPSOIL – Silty Clayey Sand Medium grey-brown, fine to medium grained, Loose, dry, porous, easily crumbles in hand, Rootlets & organic material to 3%-5%, 18” thick
5							**BEDROCK- Quartzite & Phyllite Metamorphics** Very hard, well fractured, minor foliation, Fine to medium grained, very difficult to excavate Practical refusal at 3.5 ft.
10							Trench Backfilled No Groundwater No Caving Practical Refusal @ 3.5 ft.
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1582’	TRENCH NO.	T-6
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							BEDROCK- Quartzite & Phyllite Metamorphics Very hard, well fractured, minor foliation, Fine to medium grained, very difficult to excavate Practical refusal at 1.5 ft.
5							Trench Backfilled No Groundwater No Caving Practical Refusal @ 1.5 ft.
10							
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1599’	TRENCH NO.	T-7
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							BEDROCK- Quartzite & Phyllite Metamorphics Very hard, well fractured, minor foliation, Fine to medium grained, very difficult to excavate Practical refusal at 1.5 ft.
5							Trench Backfilled No Groundwater No Caving Practical Refusal @ 1.5 ft.
10							
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1598’	TRENCH NO.	T-8
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							TOPSOIL – Silty Clayey Sand Medium grey-brown, fine to medium grained, Loose, dry, porous, easily crumbles in hand, Rootlets & organic material to 3%-5%, 18” thick
5							**BEDROCK- Quartzite & Phyllite Metamorphics** Very hard, difficult to excavate Excavates as medium to coarse sand with gravel Both decomposed granite & metamorphics Visible, Practical refusal at 7 ft.
10							Trench Backfilled No Groundwater No Caving Practical Refusal @ 7 ft.
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1611’	TRENCH NO.	T-9
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							BEDROCK- Quartzite & Phyllite Metamorphics Very hard, well fractured, minor foliation, Fine to medium grained, very difficult to excavate Practical refusal at 2 ft.
5							Trench Backfilled No Groundwater No Caving Practical Refusal @ 2 ft.
10							
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1587’	TRENCH NO.	T-10
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

GEOTECHNICAL DESCRIPTION

LOGGED BY CK **DATE** 12-17-20
SAMPLED BY CK

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	
0							
							TOPSOIL – Silty Clayey Sand (18”) Medium grey-brown, fine to medium grained, Loose, dry, porous, easily crumbles in hand, Rootlets & organic material present.
5							**BEDROCK- Granite (Kgr)** Dark grey, medium to coarse grained, dense Damp, massive structure, few fractures Excavates as coarse sand that is friable, And Non-cohesive, Practical refusal at 10 ft.
10							Trench Backfilled No Groundwater Practical Refusal @ 10 ft. No Caving
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

EXPLORATORY TRENCH LOG

PROJECT NAME	Crescent Gardens	ELEVATION	1601 ft	TRENCH NO.	T-11
PROJECT No.	1789-01	EQUIPMENT	Case 580 w/ 24-inch bucket		

DEPTH (FEET)	TYPE OF TEST*	SAMPLE DEPTH	DRY DENSITY (PCF)	MOISTURE CONTENT %	(USCS) SOIL CLASSIFICATION	EARTH MATERIAL	GEOTECHNICAL DESCRIPTION
0							**LOGGED BY** CK **DATE** 12-17-20 **SAMPLED BY** CK
							TOPSOIL – Silty Clayey Sand (18") Medium grey-brown, fine to medium grained, Loose, dry, porous,
5							**BEDROCK- Granite (Kgr)** Dark grey, medium to coarse grained, dense Damp, massive structure, few fractures Very hard at 5 ft
10							Trench Backfilled No Groundwater Practical Refusal @ 6 ft. No Caving
15							

GRAPHIC LOG Trend: Scale: 1" = 5'

*TEST SYMBOLS

B - BULK SAMPLE
R - RING SAMPLE
SC - SANDCONE
MD - MAXIMUM DENSITY
GS - GRAIN SIZE
SE - SAND EQUIVALENT
NG - NUCLEAR GAUGE
(90) - RELATIVE COMPACTION

RGS Engineering Geology

APPENDIX B
LABORATORY TEST RESULTS

MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Muslim Mortuary Crescent Gardens Canyon	Tested By:	PM/NP	Date:	01/31/24
Project No.:	038.0000019894	Checked By:	A. Santos	Date:	02/08/24
Boring No.:	TP-2	Depth (ft.):	0-1		
Sample No.:	B-1				
Soil Identification:	Dark brown well-graded sand with silt (SW-SM)				

Preparation Method: [X] Moist [ ] Dry

[X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03320 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3906	3976	3950			
Weight of Mold (g)	1808	1808	1808			
Net Weight of Soil (g)	2098	2168	2142			
Wet Weight of Soil + Cont. (g)	970.8	953.8	904.9			
Dry Weight of Soil + Cont. (g)	909.4	875.6	814.6			
Weight of Container (g)	75.4	89.2	72.4			
Moisture Content (%)	7.36	9.94	12.17			
Wet Density (pcf)	139.3	144.0	142.2			
Dry Density (pcf)	129.8	130.9	126.8			

Maximum Dry Density (pcf) **131.3** **Optimum Moisture Content (%)** **9.1**

PROCEDURE USED

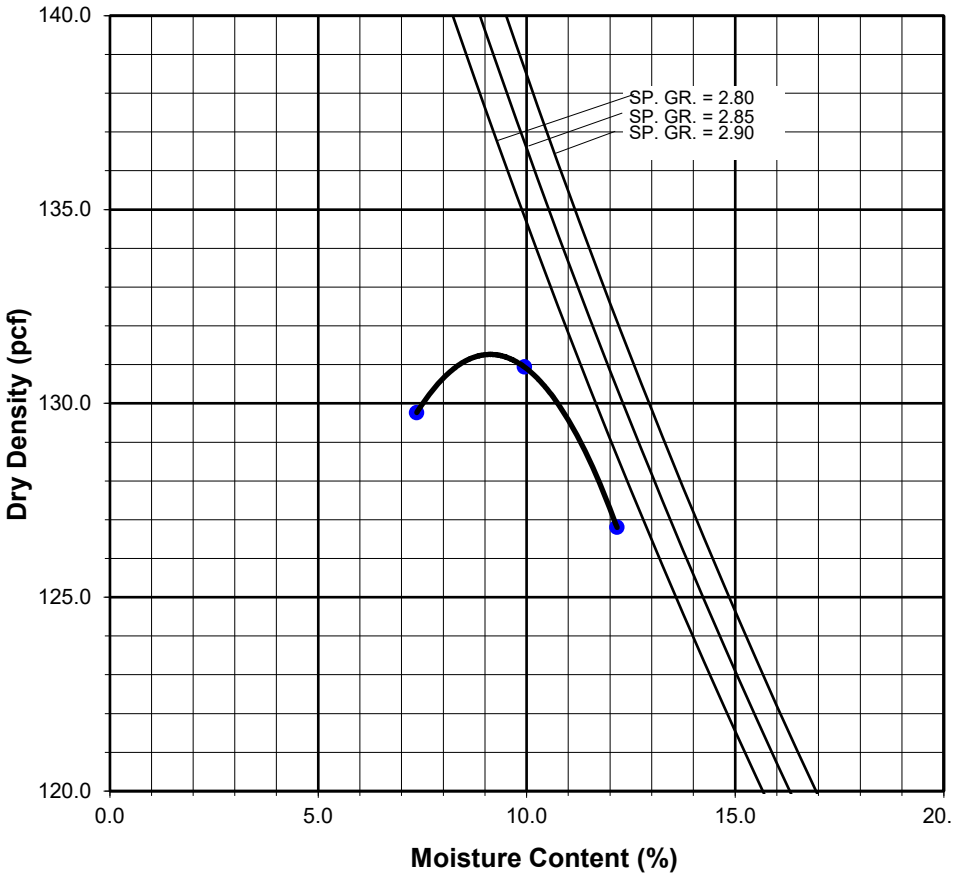
[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **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



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Muslim Mortuary Crescent Gardens Canyon	Tested By:	PM/NP	Date:	02/01/24
Project No.:	038.0000019894	Checked By:	A. Santos	Date:	02/08/24
Boring No.:	TP-5	Depth (ft.):	0-1		
Sample No.:	B-1				
Soil Identification:	Strong brown clayey sand (SC)				

Preparation Method: [X] Moist [ ] Dry

[X] Mechanical Ram [ ] Manual Ram

Mold Volume (ft³) 0.03320 *Ram Weight = 10 lb.; Drop = 18 in.*

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3805	3894	3880			
Weight of Mold (g)	1808	1808	1808			
Net Weight of Soil (g)	1997	2086	2072			
Wet Weight of Soil + Cont. (g)	893.1	883.0	974.2			
Dry Weight of Soil + Cont. (g)	844.8	818.5	882.6			
Weight of Container (g)	75.3	72.4	88.5			
Moisture Content (%)	6.28	8.64	11.54			
Wet Density (pcf)	132.6	138.5	137.6			
Dry Density (pcf)	124.8	127.5	123.4			

Maximum Dry Density (pcf) **127.5** **Optimum Moisture Content (%)** **8.6**

PROCEDURE USED

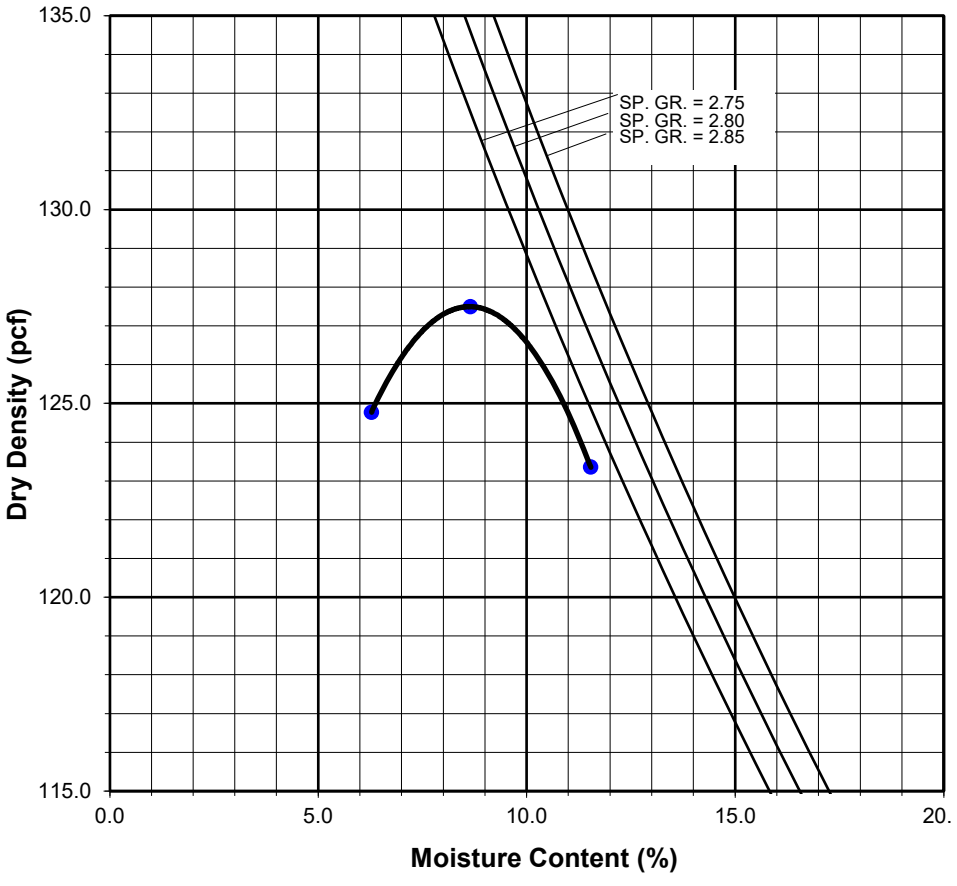
[X] **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

[ ] **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



MODIFIED PROCTOR COMPACTION TEST

ASTM D 1557

Project Name:	Muslim Mortuary Crescent Gardens Canyon	Tested By:	PM/NP	Date:	02/02/24
Project No.:	038.0000019894	Input By:	A. Santos	Date:	02/08/24
Boring No.:	TP-6	Depth (ft.):	0-1		
Sample No.:	B-1				
Soil Identification:	Dark yellowish brown silty sand with gravel (SM)g				

Note: Corrected dry density calculation assumes specific gravity of 2.70 and moisture content of 1.0% for oversize particles

Preparation Method:	X	Moist
		Dry
Compaction Method	X	Mechanical Ram
		Manual Ram

Scalp Fraction (%)	
#3/4	
#3/8	26.7
#4	

Rammer Weight (lb.) = 10.0

Height of Drop (in.) = 18.0

Mold Volume (ft³) 0.03320

TEST NO.	1	2	3	4	5	6
Wt. Compacted Soil + Mold (g)	3804	3899	3906			
Weight of Mold (g)	1808	1808	1808			
Net Weight of Soil (g)	1996	2091	2098			
Wet Weight of Soil + Cont. (g)	815.1	968.8	872.0			
Dry Weight of Soil + Cont. (g)	781.4	908.3	798.5			
Weight of Container (g)	88.7	88.9	74.6			
Moisture Content (%)	4.87	7.38	10.15			
Wet Density (pcf)	132.5	138.8	139.3			
Dry Density (pcf)	126.4	129.3	126.5			

Maximum Dry Density (pcf) 129.3

Corrected Dry Density (pcf) 137.9

Optimum Moisture Content (%) 7.6

Corrected Moisture Content (%) 5.8

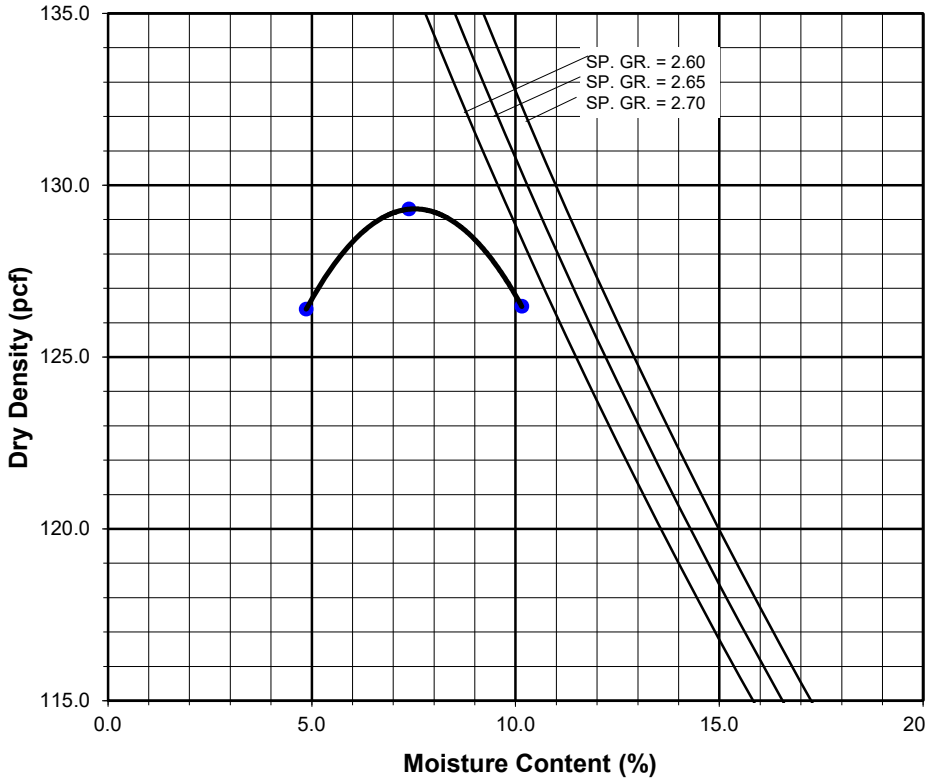
☐ **Procedure A**
Soil Passing No. 4 (4.75 mm) Sieve
Mold : 4 in. (101.6 mm) diameter
Layers : 5 (Five)
Blows per layer : 25 (twenty-five)
May be used if +#4 is 20% or less

☒ **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.	IT-1							
Sample No.	B-1							
Depth (ft.)	1-3							
Sample Type	Bulk							
Soil Identification	Olive gray silty sand (SM)							
No Moisture Correction; ASTM D 1140 modified to include splitting the sample on the #4 sieve								
Total Sample Dry Weight Determination								
Dry Weight of Soil + Container (g)	2717.30							
Weight of Container (g)	277.13							
Dry Weight of Soil (g)	2440.17							
Sample Dry Weight Determination, Retained on Sieve #4								
Dry Weight of Sample + Cont. (g)	248.83							
Weight of Container (g)	75.53							
Weight of Dry Sample (g)	173.30							
Sample Dry Weight Determination, Passing Sieve #4								
Dry Weight of Sample + Cont. (g)	479.08							
Weight of Container (g)	110.88							
Weight of Dry Sample (g)	368.20							
After Wash								
Method (A or B)	A							
Dry Weight of Sample + Cont. (g)	427.98							
Weight of Container (g)	110.88							
Weight of Dry Sample (g)	317.10							
% Passing No. 4 Sieve	92.9							
% Retained No. 4 Sieve	7.1							
% Passing No. 200 Sieve	**12.9**							

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Muslim Mortuary Crescent Gardens Canyon
Project No.: 038.0000019894

Tested By: G. Bathala Date: 02/01/24

Boring No.	IT-2	IT-4	IT-5	IT-6				
Sample No.	B-1	B-1	B-1	B-1				
Depth (ft.)	1-3	2	3	0-2				
Sample Type	Bulk	Bulk	Bulk	Bulk				
Soil Identification	Dark brown clayey sand (SC)	Gray well-graded sand with silt (SW-SM)	Brown clayey sand (SC)	Yellowish brown clayey sand (SC)				
Moisture Correction								
Wet Weight of Soil + Container (g)	0.00	0.00	0.00	0.00				
Dry Weight of Soil + Container (g)	0.00	0.00	0.00	0.00				
Weight of Container (g)	1.00	1.00	1.00	1.00				
Moisture Content (%)	0.00	0.00	0.00	0.00				
Sample Dry Weight Determination								
Weight of Sample + Container (g)	474.27	481.30	427.78	460.68				
Weight of Container (g)	107.37	108.40	109.37	110.67				
Weight of Dry Sample (g)	366.90	372.90	318.41	350.01				
Container No.:								
After Wash								
Method (A or B)	A	A	A	A				
Dry Weight of Sample + Cont. (g)	351.16	457.32	328.73	372.76				
Weight of Container (g)	107.37	108.40	109.37	110.67				
Dry Weight of Sample (g)	243.79	348.92	219.36	262.09				
% Passing No. 200 Sieve	**33.6**	**6.4**	**31.1**	**25.1**				
% Retained No. 200 Sieve	66.4	93.6	68.9	74.9				

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PERCENT PASSING
No. 200 SIEVE
ASTM D 1140

Project Name: Muslim Mortuary Crescent Gardens Canyon

Project No.: 038.0000019894

Tested By: G. Bathala Date: 02/01/24

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

ASTM D6913

Project Name: Muslim Mortuary Crescent Gardens Canyon
Project No.: 038.0000019894
Boring No.: TP-2
Sample No.: B-1
Soil Identification: Dark brown well-graded sand with silt (SW-SM)

Tested By: KJ/GB Date: 02/01/24
Checked By: A. Santos Date: 02/05/24
Depth (feet): 4-5

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	1F	R-2	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	2463.7	616.1	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	223.0	107.2	Wt. of Container No._____(g)	1.0	1.0
Dry Wt. of Soil (g)	2240.7	508.9	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	R-2
	Wt. of Dry Soil + Container (g)	587.7
	Wt. of Container (g)	107.2
	Dry Wt. of Soil Retained on # 200 Sieve (g)	480.5

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.0			
1 1/2"	37.5			
1"	25.0			
3/4"	19.0			
1/2"	12.5	0.0		100.0
3/8"	9.5	1.3		99.9
#4	4.75	12.1		99.5
#8	2.36		127.0	74.7
#16	1.18		272.0	46.3
#30	0.600		362.0	28.7
#50	0.300		421.8	17.0
#100	0.150		457.6	10.0
#200	0.075		478.5	5.9
PAN				

GRAVEL: **1 %**
SAND: **93 %**
FINES: **6 %**
GROUP SYMBOL: **SW-SM**

Cu = D60/D10 = 10.63
Cc = $(D30)^2/(D60*D10)$ = 1.46

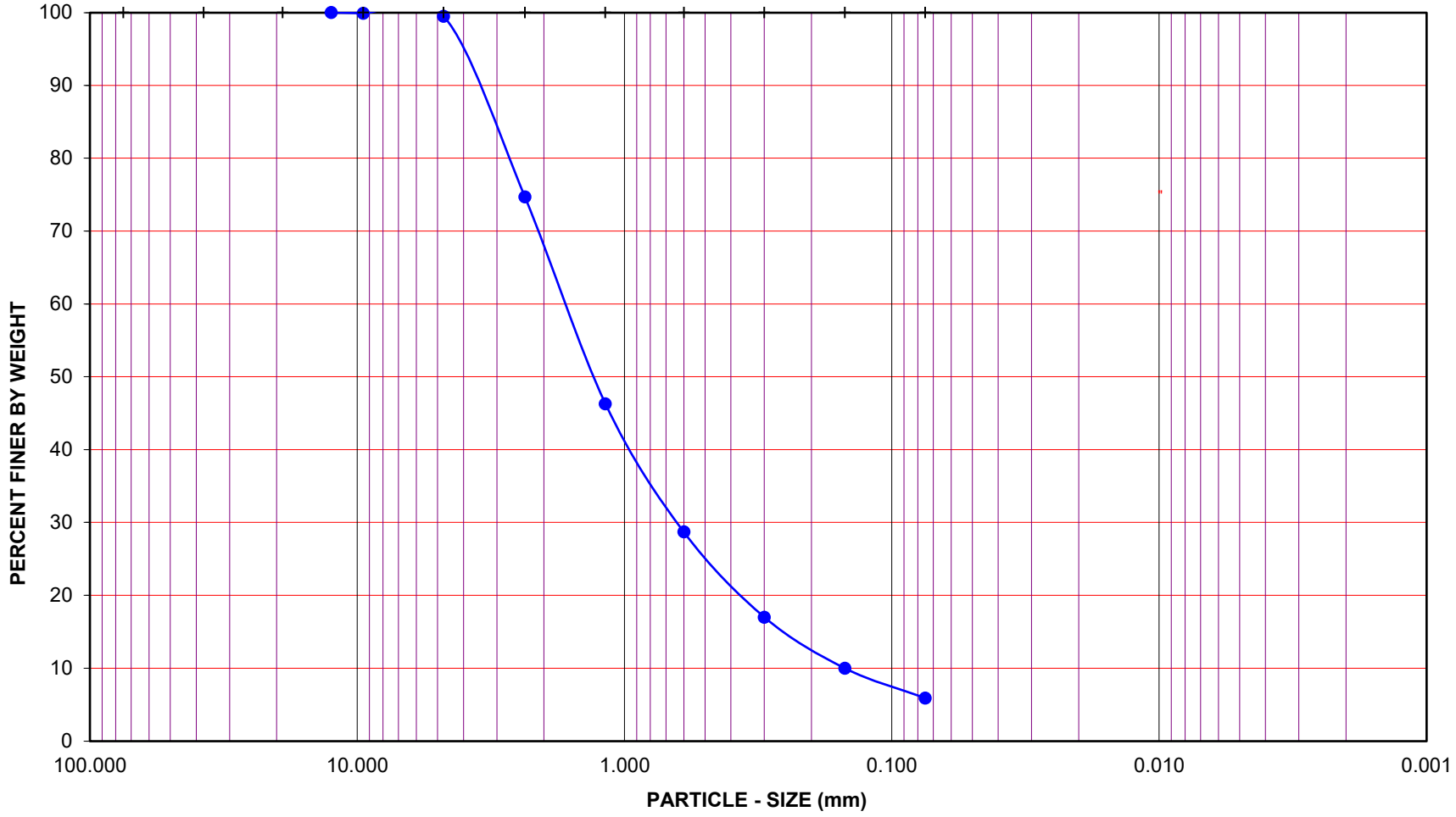
Remarks:

GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

U.S. STANDARD SIEVE OPENING: 3.0" 1 1/2" 3/4" 3/8"

U.S. STANDARD SIEVE NUMBER: #4 #8 #16 #30 #50 #100 #200

HYDROMETER



Project Name: Muslim Mortuary Crescent Gardens Canyon

Project No.: 038.0000019894

Boring No.: TP-2 Sample No.: B-1

Depth (feet): 4-5 Soil Type : SW-SM

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

GR:SA:FI : (%) 1 : 93 : 6

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Feb-24

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

ASTM D6913

Project Name: Muslim Mortuary Crescent Gardens Canyon
Project No.: 038.0000019894
Boring No.: TP-5
Sample No.: B-1
Soil Identification: Strong brown clayey sand (SC)

Tested By: KJ/GB Date: 02/01/24
Checked By: A. Santos Date: 02/08/24
Depth (feet): 0-1

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	ES-4	D-81	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	2244.3	611.6	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	228.9	108.0	Wt. of Container No._____(g)	1.0	1.0
Dry Wt. of Soil (g)	2015.4	503.6	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	D-81
	Wt. of Dry Soil + Container (g)	435.9
	Wt. of Container (g)	108.0
	Dry Wt. of Soil Retained on # 200 Sieve (g)	327.9

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.0			
1 1/2"	37.5			
1"	25.0	0.0		100.0
3/4"	19.0	14.1		99.3
1/2"	12.5	22.8		98.9
3/8"	9.5	26.1		98.7
#4	4.75	43.7		97.8
#8	2.36		36.1	90.8
#16	1.18		101.8	78.0
#30	0.600		165.7	65.6
#50	0.300		227.5	53.6
#100	0.150		280.4	43.3
#200	0.075		323.5	35.0
PAN				

GRAVEL: **2 %**
SAND: **63 %**
FINES: **35 %**
GROUP SYMBOL: **SC**

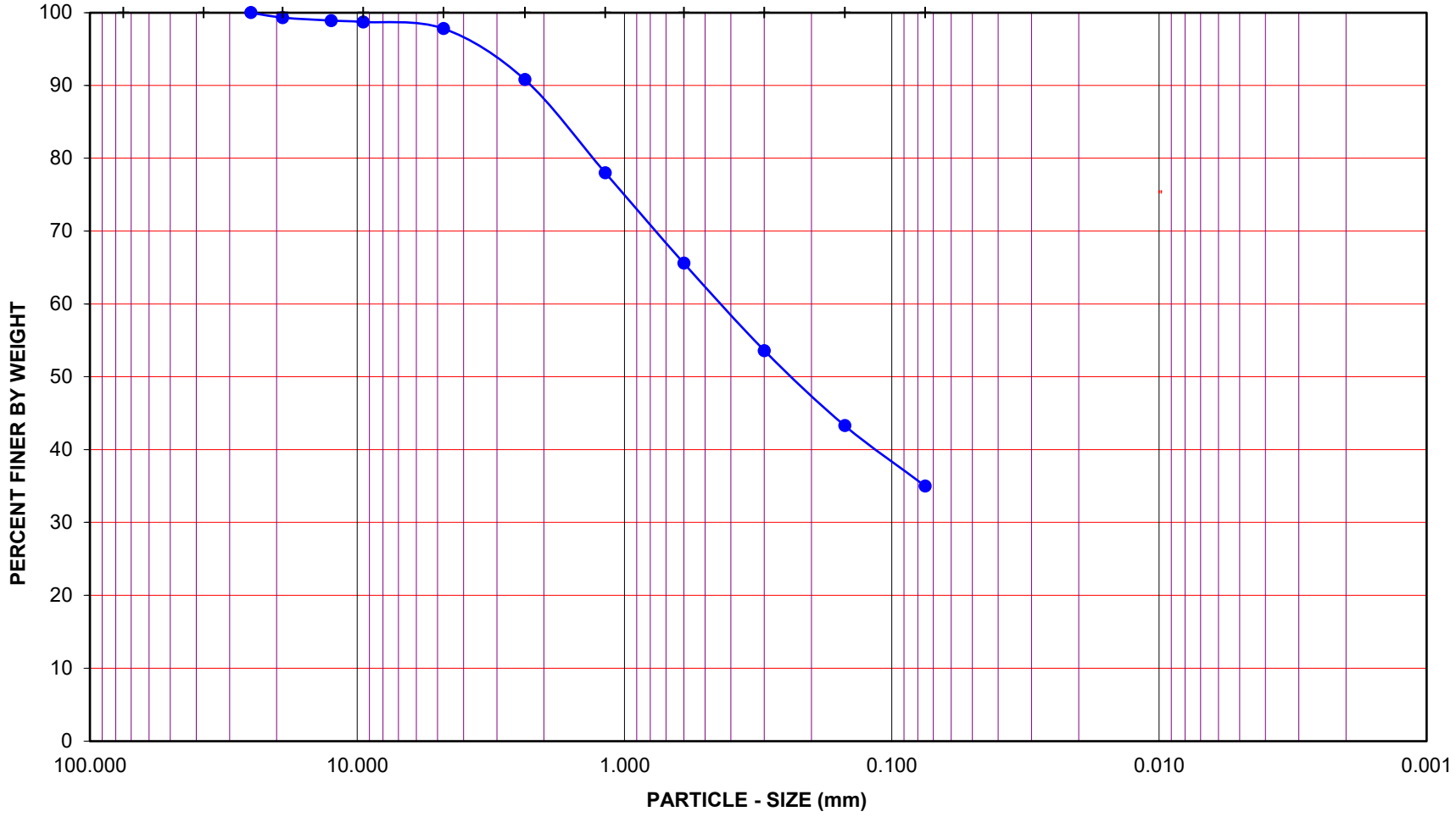
Cu = D60/D10 = 10.63
Cc = $(D30)^2/(D60*D10)$ = 1.46

Remarks:

GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

U.S. STANDARD SIEVE OPENING — U.S. STANDARD SIEVE NUMBER — HYDROMETER

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



Project Name: Muslim Mortuary Crescent Gardens Canyon

Project No.: 038.0000019894

Boring No.: TP-5 Sample No.: B-1

Depth (feet): 0-1 Soil Type : SC

Soil Identification: Strong brown clayey sand (SC)

GR:SA:FI : (%) 2 : 63 : 35

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Feb-24

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

ASTM D6913

Project Name: Muslim Mortuary Crescent Gardens Canyon
Tested By: KJ/GB Date: 02/01/24
Project No.: 038.0000019894
Checked By: A. Santos Date: 02/08/24
Boring No.: TP-6
Depth (feet): 0-1
Sample No.: B-1
Soil Identification: Dark yellowish brown silty sand with gravel (SM)g, noted asphalt & organics

Calculation of Dry Weights	Whole Sample	Sample Passing #4	Moisture Contents	Whole Sample	Sample passing #4
Container No.:	AAA	57	Wt. of Air-Dry Soil + Cont.(g)	0.0	0.0
Wt. Air-Dried Soil + Cont.(g)	5043.6	627.2	Wt. of Dry Soil + Cont. (g)	0.0	0.0
Wt. of Container (g)	461.1	107.2	Wt. of Container No._____(g)	1.0	1.0
Dry Wt. of Soil (g)	4582.5	520.0	Moisture Content (%)	0.0	0.0

Passing #4 Material After Wet Sieve		
	Container No.	57
	Wt. of Dry Soil + Container (g)	505.2
	Wt. of Container (g)	107.2
	Dry Wt. of Soil Retained on # 200 Sieve (g)	398.0

U. S. Sieve Size		Cumulative Weight of Dry Soil Retained (g)		Percent Passing (%)
	(mm.)	Whole Sample	Sample Passing #4	
3"	75.0			
1 1/2"	37.5	0.0		100.0
1"	25.0	470.5		89.7
3/4"	19.0	691.3		84.9
1/2"	12.5	1007.1		78.0
3/8"	9.5	1221.4		73.3
#4	4.75	1656.8		63.8
#8	2.36		66.8	55.6
#16	1.18		125.9	48.4
#30	0.600		181.1	41.6
#50	0.300		244.7	33.8
#100	0.150		314.1	25.3
#200	0.075		382.6	16.9
PAN				

GRAVEL: **36 %**
SAND: **47 %**
FINES: **17 %**
GROUP SYMBOL: **(SM)g**

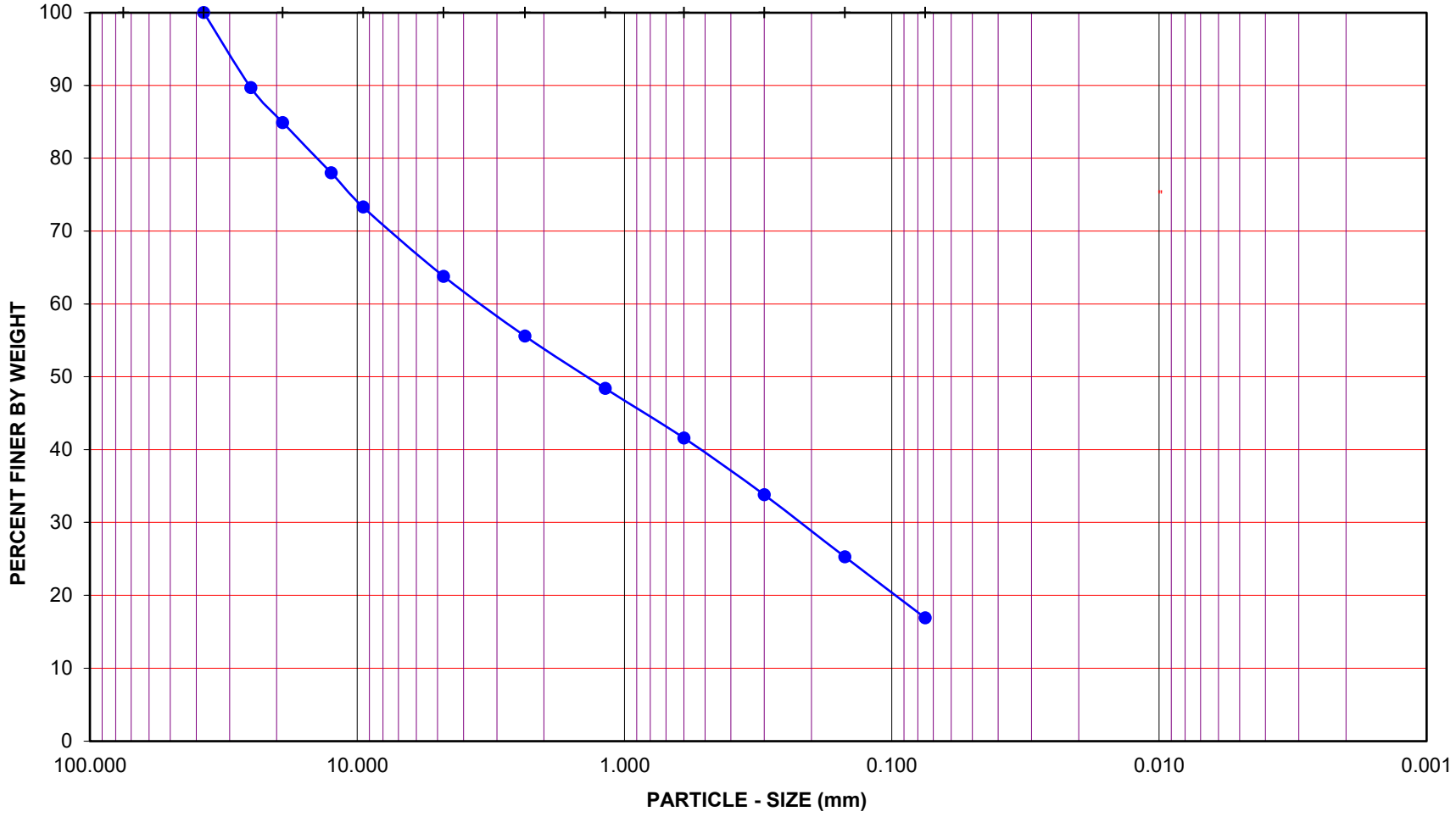
Cu = D60/D10 = 10.63
Cc = $(D30)^2/(D60*D10)$ = 1.46

Remarks:

GRAVEL		SAND			FINES	
COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY

U.S. STANDARD SIEVE OPENING | U.S. STANDARD SIEVE NUMBER | HYDROMETER

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



Project Name: Muslim Mortuary Crescent Gardens Canyon

Project No.: 038.0000019894

Boring No.: TP-6 Sample No.: B-1

Depth (feet): 0-1 Soil Type : (SM)g

Soil Identification: Dark yellowish brown silty sand with gravel (SM)g, noted asphalt & organics

GR:SA:FI : (%) 36 : 47 : 17

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PARTICLE - SIZE DISTRIBUTION ASTM D 6913

Feb-24

Sieve TP-6, B-1 @ 0-1

EXPANSION INDEX of SOILS
ASTM D 4829

Project Name:	Muslim Mortuary Crescent Gardens Canyon	Tested By:	J. Domingo	Date:	02/02/24
Project No.:	038.0000019894	Checked By:	A. Santos	Date:	02/08/24
Boring No.:	TP-1	Depth (ft.):	1-2		
Sample No.:	B-1				
Soil Identification:	Yellowish brown poorly-graded sand with clay (SP-SC)				

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

MOLDED SPECIMEN	Before Test	After Test
Specimen Diameter (in.)	4.01	4.01
Specimen Height (in.)	1.0000	1.0005
Wt. Comp. Soil + Mold (g)	613.50	433.44
Wt. of Mold (g)	208.10	0.00
Specific Gravity (Assumed)	2.70	2.70
Container No.	O	O
Wet Wt. of Soil + Cont. (g)	808.60	641.54
Dry Wt. of Soil + Cont. (g)	738.50	578.31
Wt. of Container (g)	0.00	208.10
Moisture Content (%)	9.49	17.08
Wet Density (pcf)	122.3	130.7
Dry Density (pcf)	111.7	111.6
Void Ratio	0.509	0.510
Total Porosity	0.338	0.338
Pore Volume (cc)	69.9	70.0
Degree of Saturation (%) [S_{meas}]	**50.3**	90.4

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.)
02/02/24	14:00	1.0	0	0.5385
02/02/24	14:10	1.0	10	0.5370
Add Distilled Water to the Specimen				
02/02/24	15:00	1.0	50	0.5375
02/03/24	7:22	1.0	1032	0.5390
02/03/24	11:53	1.0	1303	0.5390

Expansion Index (EI_{meas}) = ((Final Rdg - Initial Rdg) / Initial Thick.) x 1000	**2**

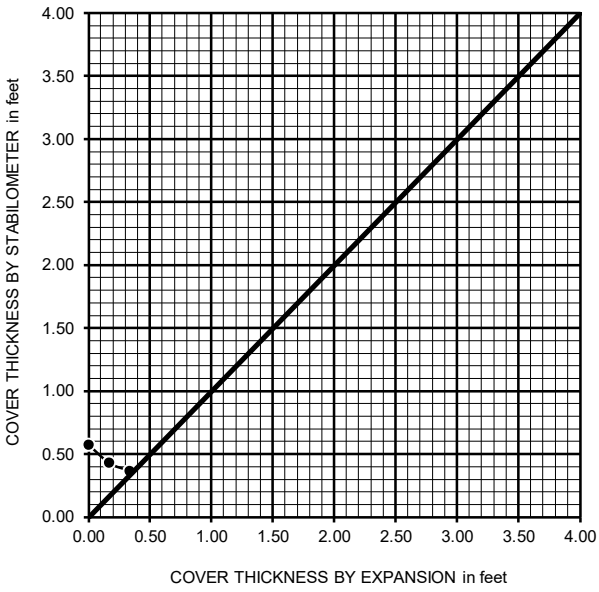
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Muslim Mortuary Crescent Gardens Canyon	PROJECT NUMBER:	038.0000019894
BORING NUMBER:	TP-1	DEPTH (FT.):	1-2
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Yellowish brown poorly-graded sand w/ clay (SP-SC)	DATE COMPLETED:	1/31/2024

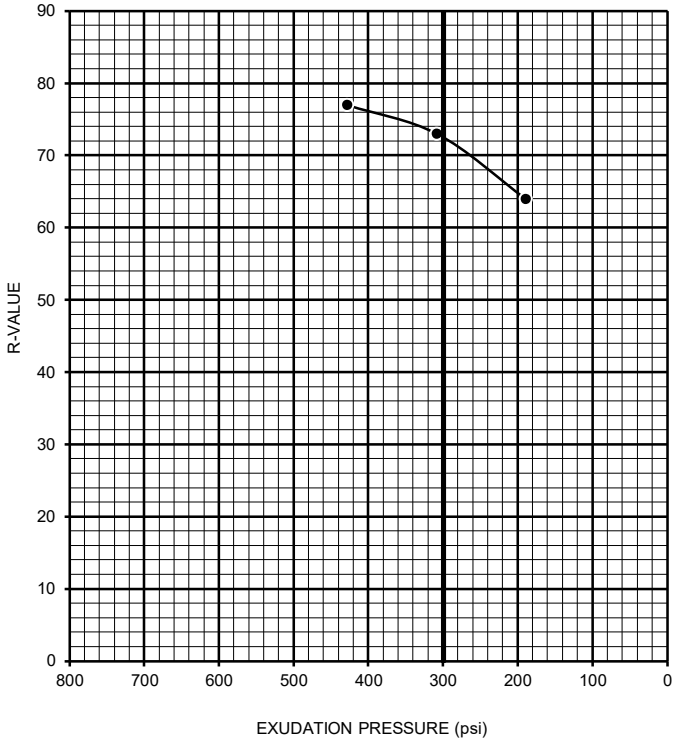
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	12.5	13.4	14.5
HEIGHT OF SAMPLE, Inches	2.49	2.52	2.54
DRY DENSITY, pcf	119.8	119.9	118.8
COMPACTOR PRESSURE, psi	275	220	150
EXUDATION PRESSURE, psi	428	308	189
EXPANSION, Inches x 10exp-4	10	5	0
STABILITY Ph 2,000 lbs (160 psi)	21	25	35
TURNS DISPLACEMENT	4.88	4.98	5.10
R-VALUE UNCORRECTED	77	73	64
R-VALUE CORRECTED	77	73	64

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.37	0.43	0.58
EXPANSION PRESSURE THICKNESS, ft.	0.33	0.17	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	75
R-VALUE BY EXUDATION:	72
EQUILIBRIUM R-VALUE:	72

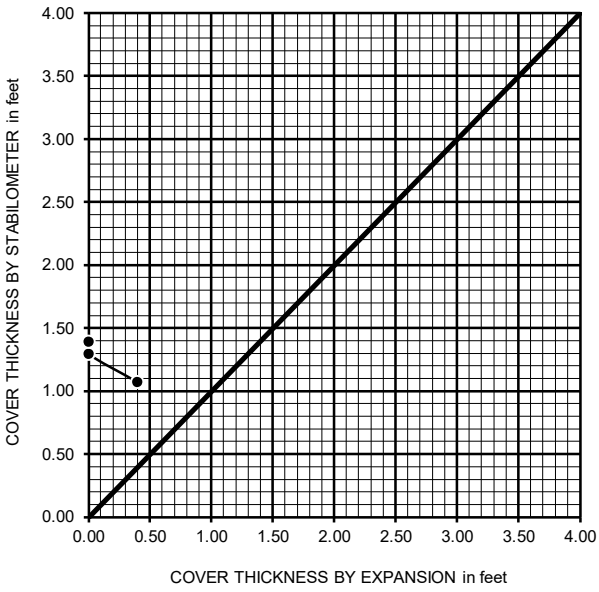
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Muslim Mortuary Crescent Gardens Canyon	PROJECT NUMBER:	038.0000019894
BORING NUMBER:	TP-5	DEPTH (FT.):	0-1
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Strong brown clayey sand (SC)	DATE COMPLETED:	2/1/2024

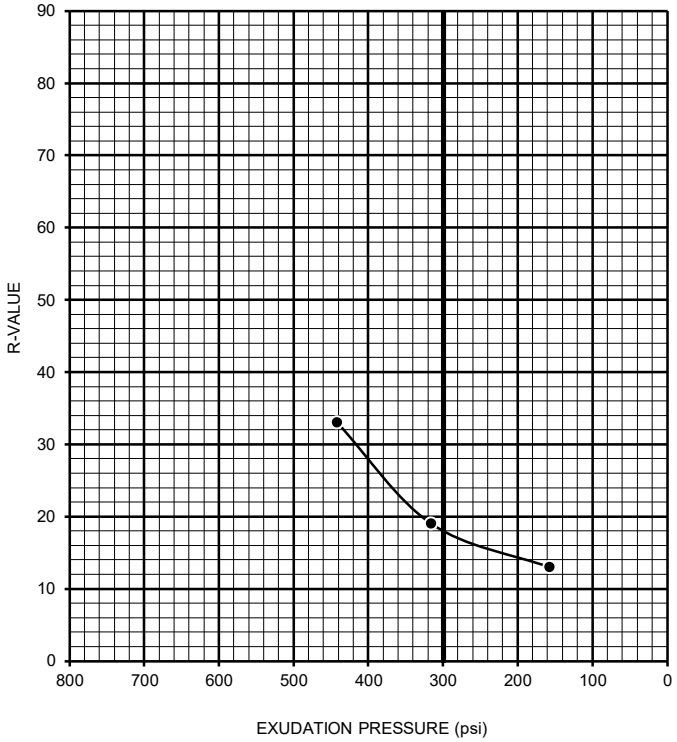
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	11.7	12.7	14.0
HEIGHT OF SAMPLE, Inches	2.46	2.53	2.48
DRY DENSITY, pcf	126.3	123.4	122.0
COMPACTOR PRESSURE, psi	100	60	50
EXUDATION PRESSURE, psi	442	316	158
EXPANSION, Inches x 10exp-4	12	0	0
STABILITY Ph 2,000 lbs (160 psi)	88	114	127
TURNS DISPLACEMENT	4.15	4.30	4.50
R-VALUE UNCORRECTED	33	19	13
R-VALUE CORRECTED	33	19	13

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	1.07	1.30	1.39
EXPANSION PRESSURE THICKNESS, ft.	0.40	0.00	0.00

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	38
R-VALUE BY EXUDATION:	18
EQUILIBRIUM R-VALUE:	18

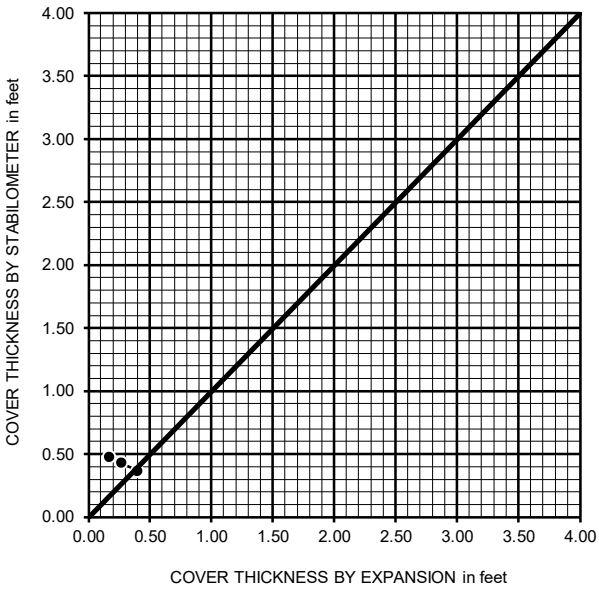
R-VALUE TEST RESULTS
DOT CA Test 301

PROJECT NAME:	Muslim Mortuary Crescent Gardens Canyon	PROJECT NUMBER:	038.0000019894
BORING NUMBER:	TP-6	DEPTH (FT.):	0-1
SAMPLE NUMBER:	B-1	TECHNICIAN:	O. Figueroa
SAMPLE DESCRIPTION:	Dark yellowish brown silty sand with gravel (SM)g	DATE COMPLETED:	2/1/2024

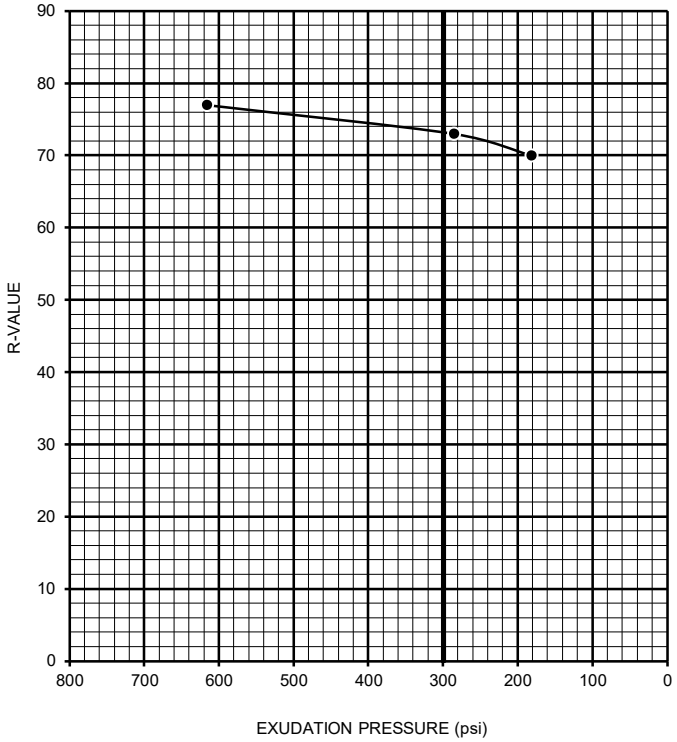
TEST SPECIMEN	a	b	c
MOISTURE AT COMPACTION %	9.4	9.8	10.2
HEIGHT OF SAMPLE, Inches	2.54	2.52	2.55
DRY DENSITY, pcf	126.7	125.5	124.5
COMPACTOR PRESSURE, psi	275	225	200
EXUDATION PRESSURE, psi	616	285	181
EXPANSION, Inches x 10exp-4	12	8	5
STABILITY Ph 2,000 lbs (160 psi)	20	24	26
TURNS DISPLACEMENT	5.13	5.28	5.45
R-VALUE UNCORRECTED	77	73	70
R-VALUE CORRECTED	77	73	70

DESIGN CALCULATION DATA	a	b	c
GRAVEL EQUIVALENT FACTOR	1.0	1.0	1.0
TRAFFIC INDEX	5.0	5.0	5.0
STABILOMETER THICKNESS, ft.	0.37	0.43	0.48
EXPANSION PRESSURE THICKNESS, ft.	0.40	0.27	0.17

EXPANSION PRESSURE CHART



EXUDATION PRESSURE CHART



R-VALUE BY EXPANSION:	78
R-VALUE BY EXUDATION:	73
EQUILIBRIUM R-VALUE:	73

TESTS for SULFATE CONTENT
CHLORIDE CONTENT and pH of SOILS

Project Name: Muslim Mortuary Crescent Gardens Canyon Tested By : K. Jumig Date: 02/01/24

Project No. : 038.0000019894 Checked By: A. Santos Date: 02/08/24

Boring No.	TP-1			
Sample No.	B-1			
Sample Depth (ft)	1-2			
Soil Identification:	Yellowish brown (SP-SC)			
Wet Weight of Soil + Container (g)	0.00			
Dry Weight of Soil + Container (g)	0.00			
Weight of Container (g)	1.00			
Moisture Content (%)	0.00			
Weight of Soaked Soil (g)	100.67			

SULFATE CONTENT, DOT California Test 417, Part II

Beaker No.	404			
Crucible No.	K			
Furnace Temperature (°C)	860			
Time In / Time Out	8:50/9:35			
Duration of Combustion (min)	45			
Wt. of Crucible + Residue (g)	11.2860			
Wt. of Crucible (g)	11.2846			
Wt. of Residue (g) (A)	0.0014			
PPM of Sulfate (A) x 41150	57.61			
PPM of Sulfate, Dry Weight Basis	**58**			

CHLORIDE CONTENT, DOT California Test 422

ml of Extract For Titration (B)	30			
ml of AgNO3 Soln. Used in Titration (C)	0.5			
PPM of Chloride (C -0.2) * 100 * 30 / B	30			
PPM of Chloride, Dry Wt. Basis	**30**			

pH TEST, DOT California Test 643

pH Value	**7.67**			
Temperature °C	20.7			

SOIL RESISTIVITY TEST

DOT CA TEST 643

Project Name:	Muslim Mortuary Crescent Gardens Canyon		Tested By :	K. Jumig	Date: 02/05/24
Project No. :	038.0000019894		Checked By:	A. Santos	Date: 02/08/24
Boring No.:	TP-1		Depth (ft.) :	1-2	
Sample No. :	B-1				

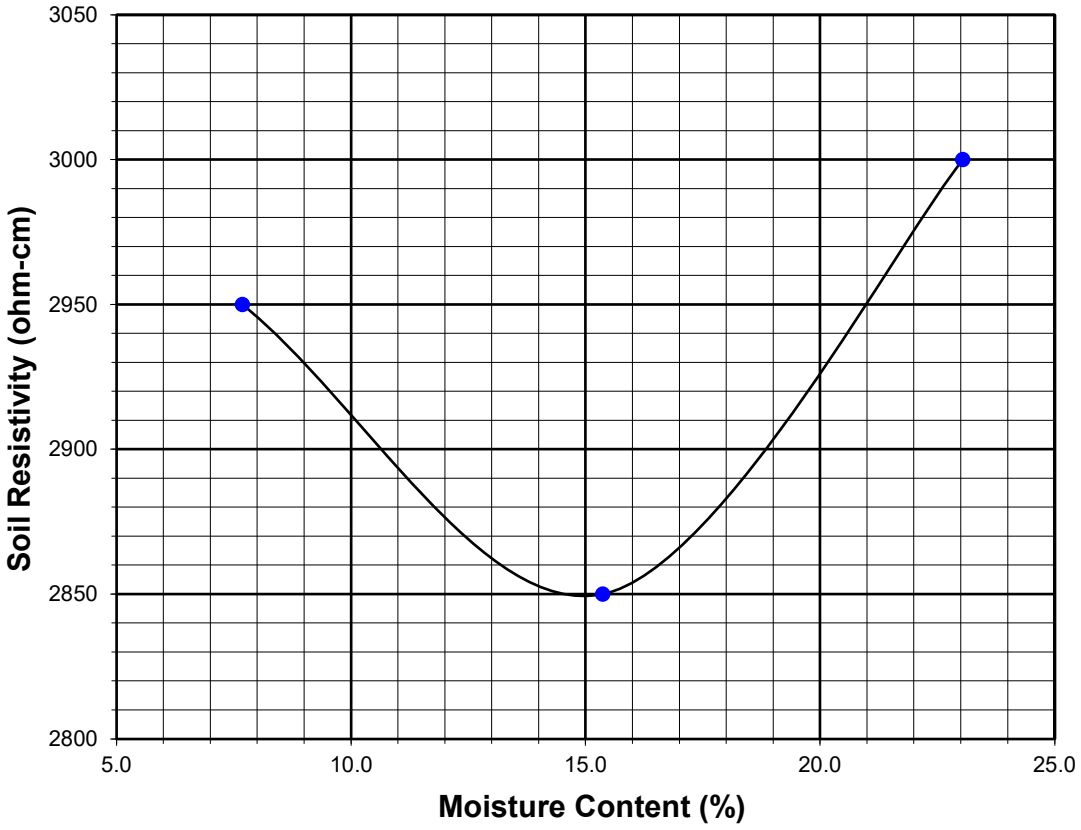
Soil Identification:* Yellowish brown (SP-SC)

*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	10	7.68	2950	2950
2	20	15.36	2850	2850
3	30	23.04	3000	3000
4				
5				

Moisture Content (%) (MCi)	0.00
Wet Wt. of Soil + Cont. (g)	0.00
Dry Wt. of Soil + Cont. (g)	0.00
Wt. of Container (g)	1.00
Container No.	
Initial Soil Wt. (g) (Wt)	130.19
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	
2850	**15.0**	**58**	**30**	**7.67**	**20.7**



EEI 2006 LABORATORY

Leighton
a verdantas company

CONSOLIDATION-SWELL TEST ASTM D 2435

Job Data
Job No.: P051109-1
Client: Lowe's HIW
Project Name: Shopoff Group, Meadowbrook
Date: 4/28/06
Sample Data
Sample: B2 10-11'
Sample Type: In-place California Modified
Remarks:
Soil Description: Decomposed Granite

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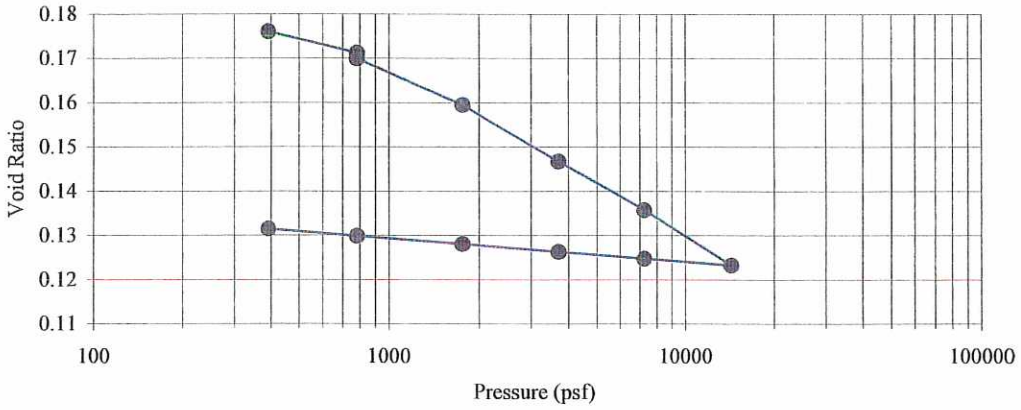

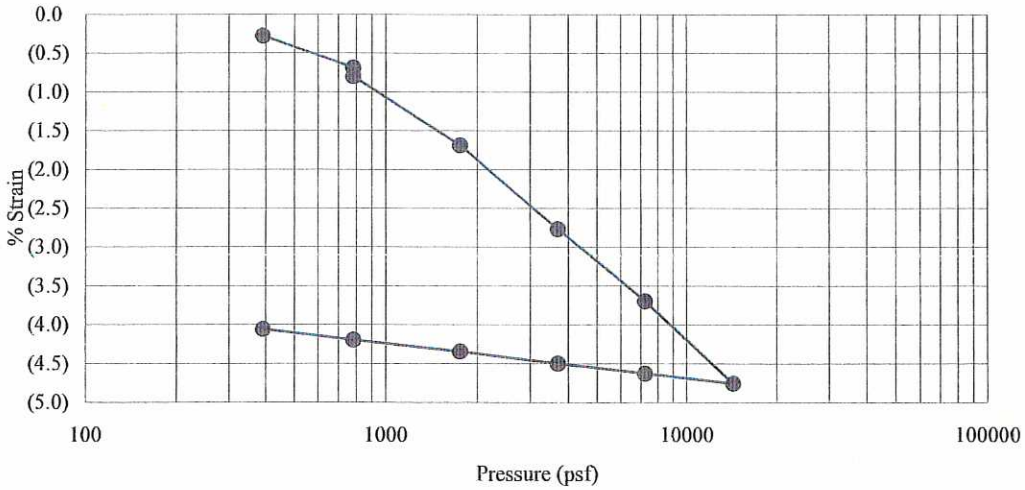

Swell Pressure:	786	psf
Percent Swell:	-0.1%	
Comp. Index (Cc):	0.042	
Consol. Index (Cr):	0.005	

Specimen Diameter:	2.42	in.
Specimen Height:	1.00	in.
Overburden Pressure (Po):	1275	psf
Preconsol. Pressure (Pp):	n/a	psf

	Initial	Final
Moisture Content:	5.2%	8.4%
Void Ratio:	0.176	0.131
Saturation:	n/a	180%
Dry Density (pcf):	142.9	150.0

CONSOLIDATION-SWELL TEST ASTM D 2435

Job Data
Job No.: P051109-1
Client: Lowe's HIW
Project Name: Shopoff Group, Meadowbrook
Date: 4/28/06
Sample Data
Sample: B4A 5-6'
Sample Type: In-place California Modified
Remarks:
Soil Description: Clayey Sand

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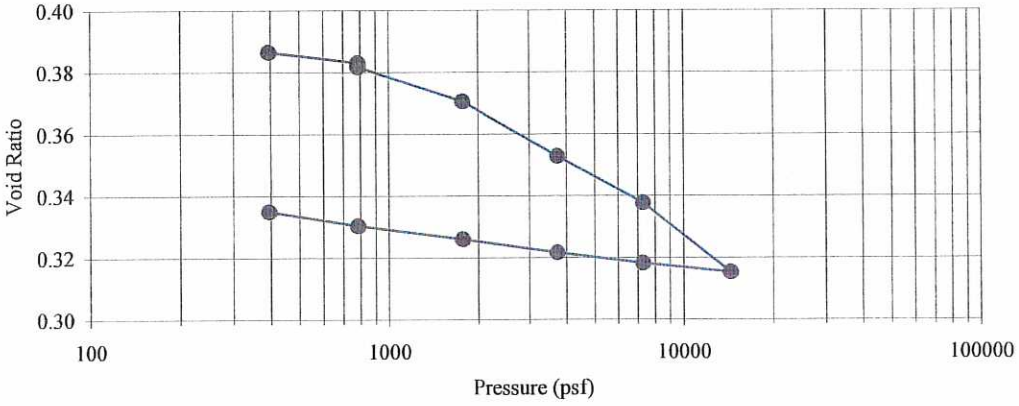

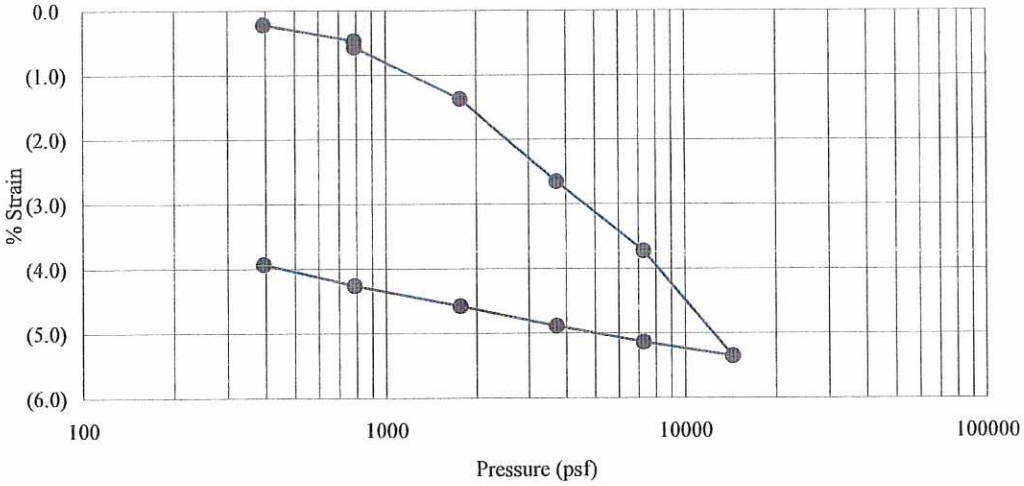

Swell Pressure:	786	psf
Percent Swell:	-0.1%	
Comp. Index (Cc):	0.076	
Consol. Index (Cr):	0.013	

Specimen Diameter:	2.42	in.
Specimen Height:	1.00	in.
Overburden Pressure (Po):	575	psf
Preconsol. Pressure (Pp):	n/a	psf

	Initial	Final
Moisture Content:	10.3%	15.0%
Void Ratio:	0.387	0.335
Saturation:	n/a	125%
Dry Density (pcf):	121.2	127.5

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	P051109-1
Client:	Shopoff Group, Meadowbrook
Date:	4/19/06
Sample Data	
Sample:	B1 5-6'
Sample Type:	Natural
Remarks:	Sample Innundated Prior to Testing
Soil Description:	DG

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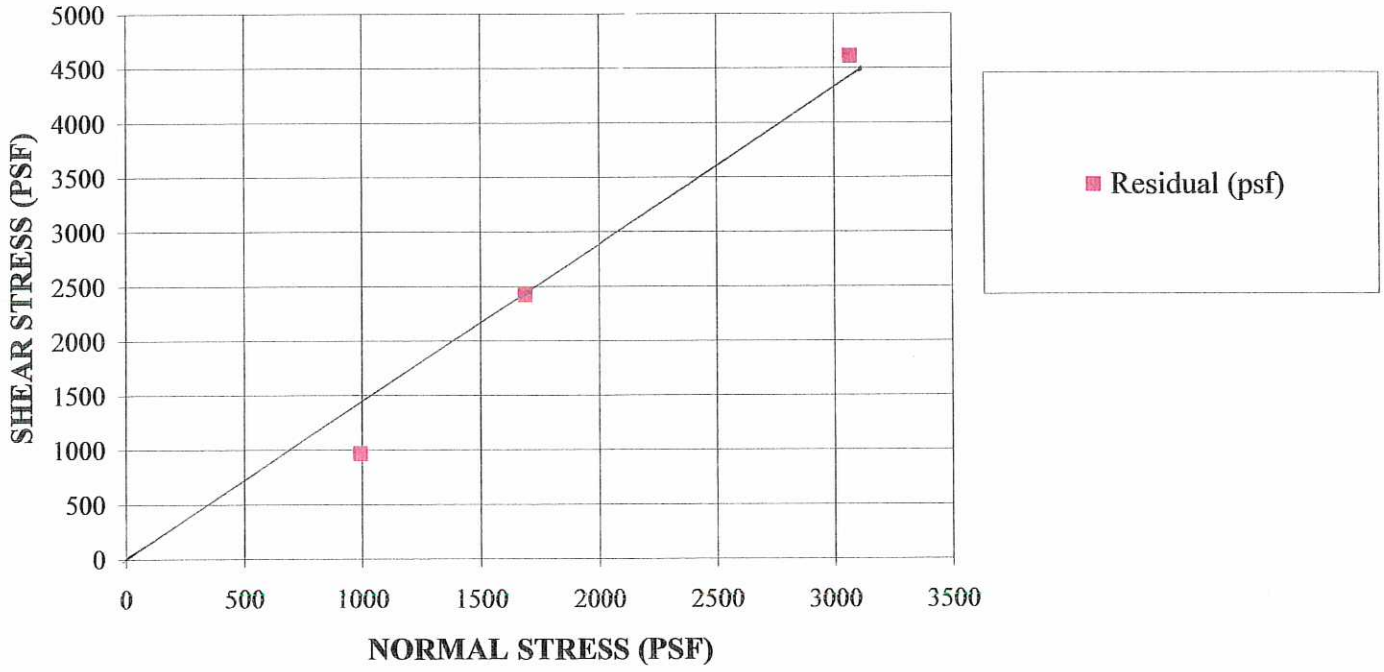
SHEAR TEST DIAGRAM



	Test Results Phi		Cohesion	
Residual (psf)	60	degrees	0	psf

Average Initial Moisture	3.9%
Average Dry Density	109.0 pcf
Average Final Moisture	10.8%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	P051109-1
Client:	Shopoff Group, Meadowbrook
Date:	4/19/06
Sample Data	
Sample:	B2 5-6'
Sample Type:	Natural
Remarks:	Sample Innundated Prior to Testing
Soil Description:	SW-SM

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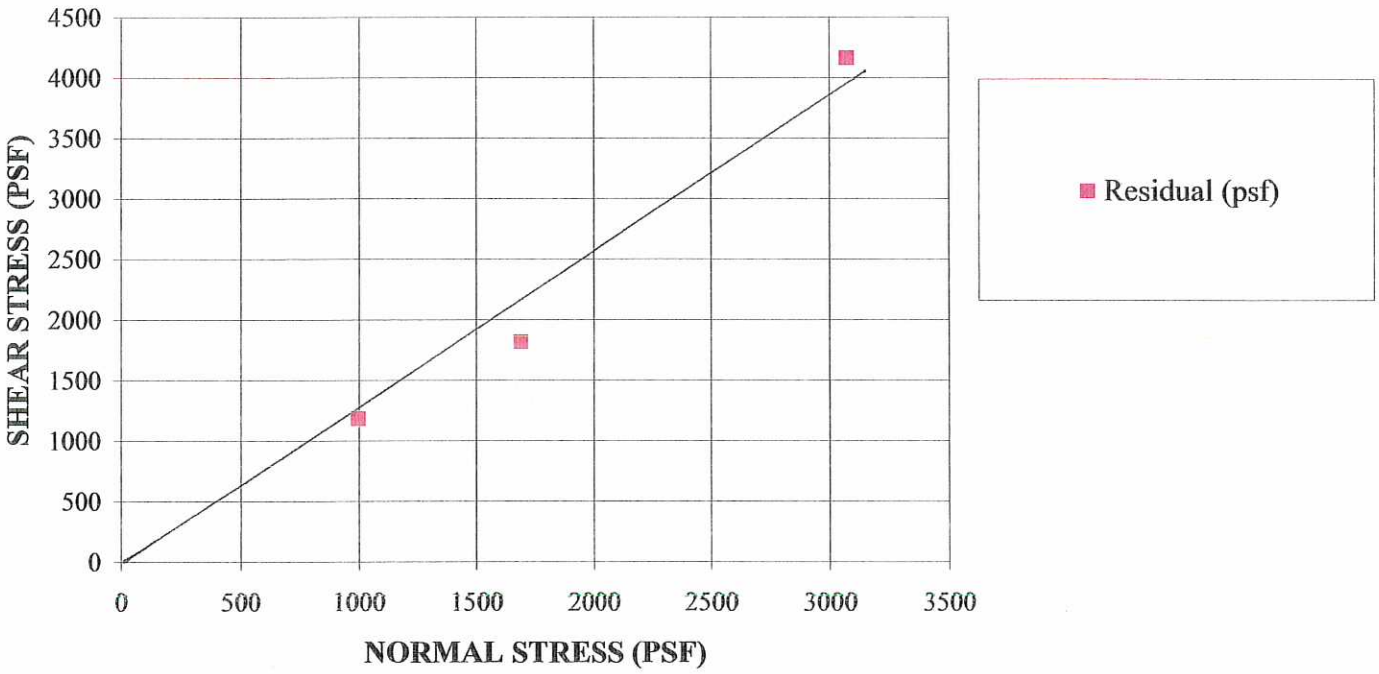
SHEAR TEST DIAGRAM



	Test Results Phi		Cohesion	
Residual (psf)	56	degrees	0	psf

Average Initial Moisture	5.9%
Average Dry Density	106.3 pcf
Average Final Moisture	12.3%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	P051109-1
Client:	Shopoff Group, Meadowbrook
Date:	4/19/06
Sample Data	
Sample:	B3 5-6'
Sample Type:	Natural
Remarks:	Sample Innundated Prior to Testing
Soil Description:	DG

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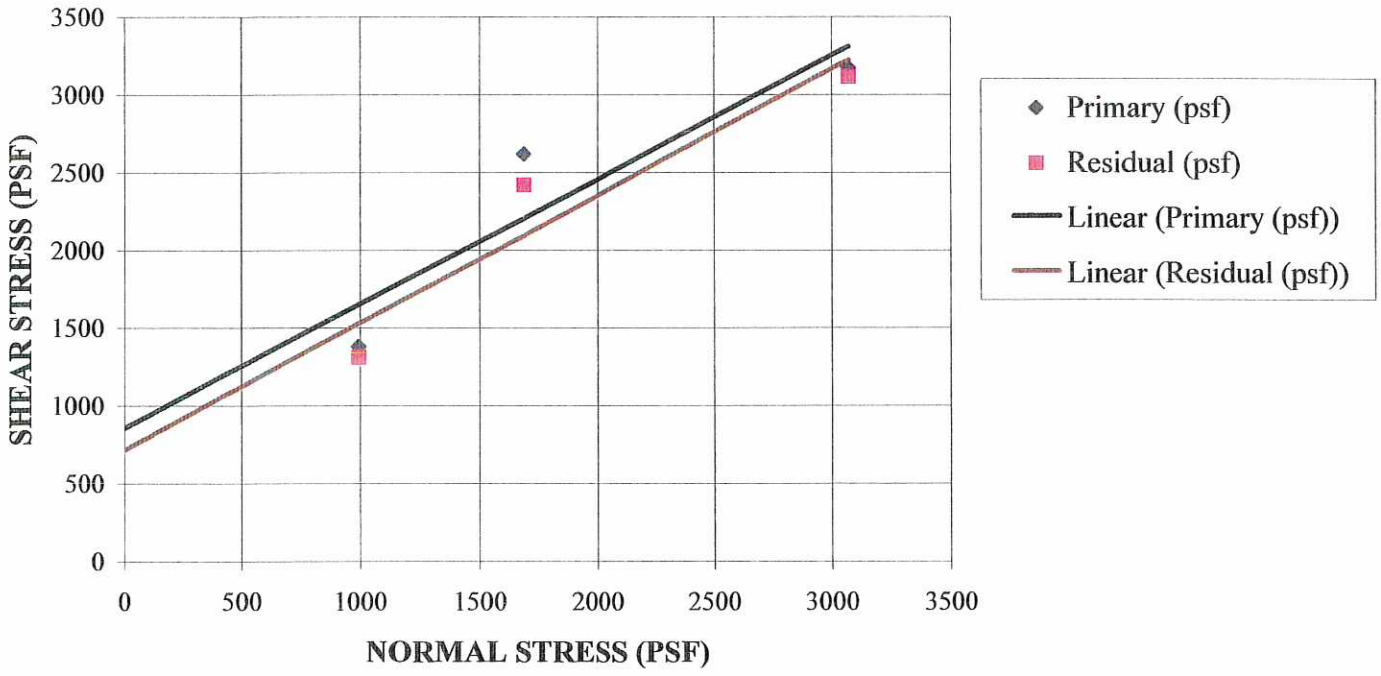
SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Primary (psf)	39	degrees	860	psf
Residual (psf)	39	degrees	716	psf

Average Initial Moisture	3.2%
Average Dry Density	105.1 pcf
Average Final Moisture	12.6%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	P051109-1
Client:	Shopoff Group - Meadowbrook
Date:	4/14/06
Sample Data	
Sample:	TP-22 2-4'
Sample Type:	Remolding to 92% of Max. Proctor
Remarks:	Sample Innundated Prior to Testing
Soil Description:	SW-SM, Well Graded Sand with Silt

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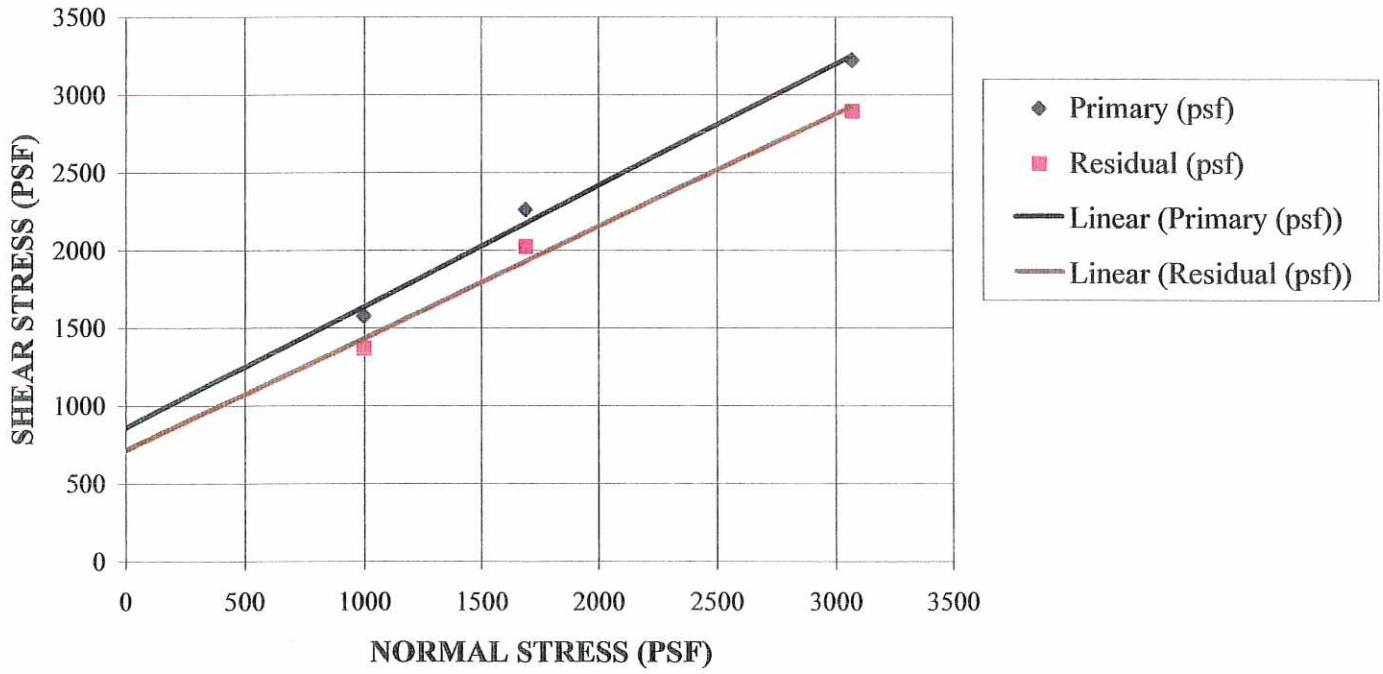
SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Primary (psf)	38	degrees	861	psf
Residual (psf)	36	degrees	715	psf

Average Initial Moisture	8.5%
Average Dry Density	120.4 pcf
Average Final Moisture	10.8%

DIRECT SHEAR TEST ASTM D 3080

Job Data	
Job No.:	P051109-1
Client:	Shopoff Group - Meadowbrook
Date:	4/18/06
Sample Data	
Sample:	TP-31 0-4'
Sample Type:	Remolding to 92% of Max. Proctor
Remarks:	Sample Innundated Prior to Testing
Soil Description:	SC-SM, Silty, Clayey Sand with Gravel

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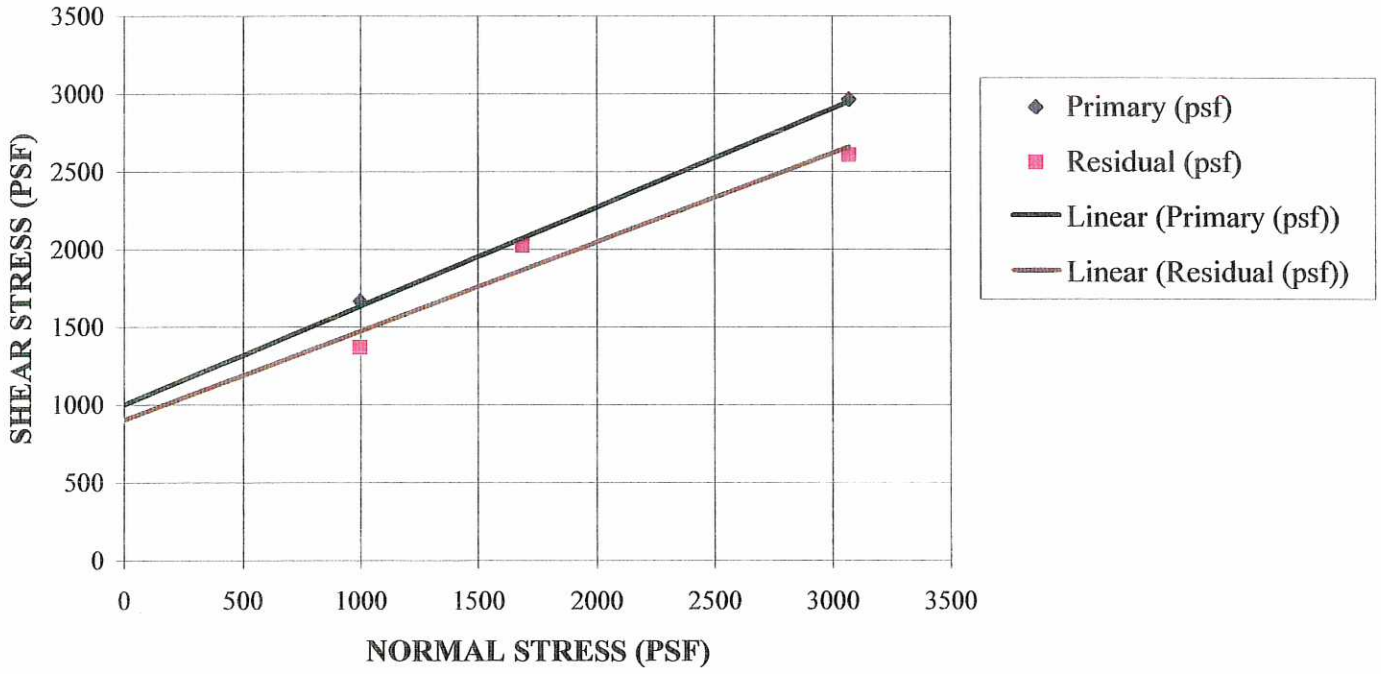
SHEAR TEST DIAGRAM



Test Results

	Phi		Cohesion	
Primary (psf)	32	degrees	1005	psf
Residual (psf)	30	degrees	904	psf

Average Initial Moisture	9.0%
Average Dry Density	126.6 pcf
Average Final Moisture	13.2%

EXPANSION INDEX TEST

ASTM METHOD D 4829

Sample : TP-3 0-4'

Moisture Content of Initial Sample		% Saturation of Re-molded Sample		Moisture Content of Final Sample	
Tare No. -		Wt. of Soil and Ring (g) -	624.5	Wt. of Soil and Ring (g) -	657
Wet Weight and Tare (g) -	200.0	Ring Weight (g) -	207.3	Ring Weight (g) -	207.3
Dry Weight and Tare (g) -	185.3	Wet Weight of Soil (g) -	417.2	Wet Weight of Soil (g) -	449.7
Tare Weight (g) -	0.0	Dry Weight of Soil (g) -	386.5	Dry Weight of Soil (g) -	386.5
Water Loss (g) -	14.7	Volume of Ring (ft^3) -	0.0073	Weight of Water (g) -	63.2
Dry Weight (g) -	185.3	Dry Density (pcf) -	116.7	*Final Moisture (%)*	*16.3*
Initial Moisture (%) -	*7.9*	*Initital Saturation (%) -*	*48.3*	*Final Saturation (%) -*	*99.5*

	Expansion Test - UBC (144 PSF)			
	Date	Time	Reading	
Add Weight	4/13/2006	16:18	0.340	
Add Water 10 Minutes		16:28	0.345	Initial Reading
		16:54	0.349	
	4/14/2006	9:19	0.349	
		14:02	0.349	Final Reading

EImeasured	=	4
EI_{50}	=	3

Expansion Index, EI_{50}	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4-13-06

Boring Number: TP-3

Location: 0-4'

Soil Description: Silty, Clayey Sand, SC-SM

Tested by: AW

EXPANSION INDEX TEST

ASTM METHOD D 4829

Sample : TP-31 0-4'

Moisture Content of Initial Sample		% Saturation of Re-molded Sample		Moisture Content of Final Sample	
Tare No. -		Wt. of Soil and Ring (g) -	635	Wt. of Soil and Ring (g) -	652
Wet Weight and Tare (g) -	200.0	Ring Weight (g) -	207.3	Ring Weight (g) -	207.3
Dry Weight and Tare (g) -	184.4	Wet Weight of Soil (g) -	427.7	Wet Weight of Soil (g) -	444.7
Tare Weight (g) -	0.0	Dry Weight of Soil (g) -	394.3	Dry Weight of Soil (g) -	394.3
Water Loss (g) -	15.6	Volume of Ring (ft^3) -	0.0073	Weight of Water (g) -	50.4
Dry Weight (g) -	184.4	Dry Density (pcf) -	119.1	*Final Moisture (%)*	*12.8*
Initial Moisture (%) -	*8.5*	*Initital Saturation (%) -*	*55.1*	*Final Saturation (%) -*	*83.1*

Expansion Test - UBC (144 PSF)				
	Date	Time	Reading	
Add Weight	4/14/2006	14:40	0.377	
Add Water 10 Minutes		14:50	0.377	Initial Reading
		15:55	0.380	
	4/15/2006	12:00	0.380	
		17:30	0.381	Final Reading

EImeasured	=	4
EI_{50}	=	**6**

Expansion Index, EI_{50}	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
>130	Very High

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4-13-06

Boring Number: TP-31

Location: 0-4'

Soil Description: Silty, Clayey Sand with Gravel, SC-SM

Tested by: AW

Table 1 - Laboratory Tests on Soil Samples

E.E.I.
Shopoff Group
Your #P051109-1, MJS&A #06-0589LAB
7-Apr-06

Sample ID			TP-3 @ 0'-4' Bulk	TP-22 @ 2'-4' DG	TP-31 @ 0'-4' QSW
Resistivity		**Units**			
as-received		ohm-cm	160,000	500,000	370,000
saturated		ohm-cm	5,400	33,000	4,700
pH			6.8	7.2	6.8
Electrical Conductivity		mS/cm	0.07	0.02	0.10
Chemical Analyses					
Cations					
calcium	Ca^{2+}	mg/kg	56	20	88
magnesium	Mg^{2+}	mg/kg	70	7	61
sodium	Na^{1+}	mg/kg	ND	ND	ND
Anions					
carbonate	CO_3^{2-}	mg/kg	ND	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	153	31	265
chloride	Cl^{1-}	mg/kg	ND	ND	15
sulfate	SO_4^{2-}	mg/kg	39	27	32
Other Tests					
ammonium	NH_4^{1+}	mg/kg	na	na	na
nitrate	NO_3^{1-}	mg/kg	na	na	na
sulfide	S^{2-}	qual	na	na	na
Redox		mV	na	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

R - VALUE DATA SHEET

P.N. P051109-1
Meadow Brook

PROJECT NUMBER 33240 BORING NUMBER: TP-3 @ 0'-4'

SAMPLE DESCRIPTION: Brown Silty Sand

Item	SPECIMEN		
	a	b	c
Mold Number	10	11	12
Water added, grams	55	75	40
Initial Test Water, %	10.8	12.5	9.4
Compact Gage Pressure,psi	105	50	350
Exudation Pressure, psi	277	164	543
Height Sample, Inches	2.63	2.62	2.53
Gross Weight Mold, grams	3346	3287	3326
Tare Weight Mold, grams	2123	2101	2117
Sample Wet Weight, grams	1223	1186	1209
Expansion, Inches x 10exp-4	0	0	6
Stability 2,000 lbs (160psi)	28 / 64	36 / 84	19 / 39
Turns Displacement	4.82	5.33	4.23
R-Value Uncorrected	44	30	65
R-Value Corrected	48	33	65
Dry Density, pcf	127.2	121.9	132.3
DESIGN CALCULATION DATA			
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.53	0.69	0.36
G. E. by Expansion	0.00	0.00	0.20

Equilibrium R-Value **50** by EXUDATION

Examined & Checked: 4 /12/ 06

Gf = 1.25

REMARKS:

Steven R. Marvin, RCE 30659

The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

LaBelle • Marvin

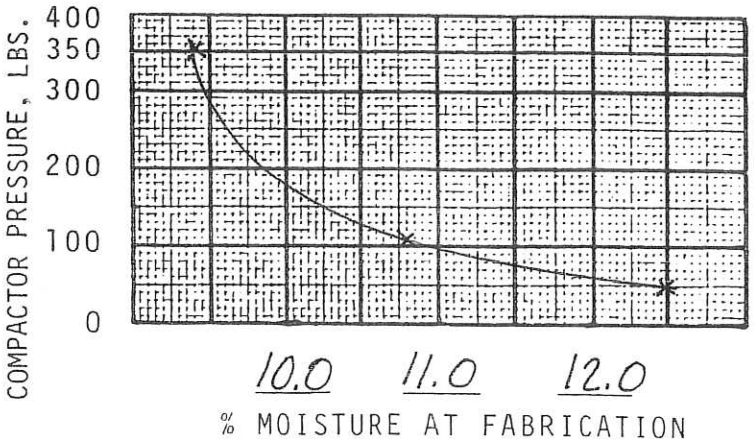
R-VALUE GRAPHICAL PRESENTATION

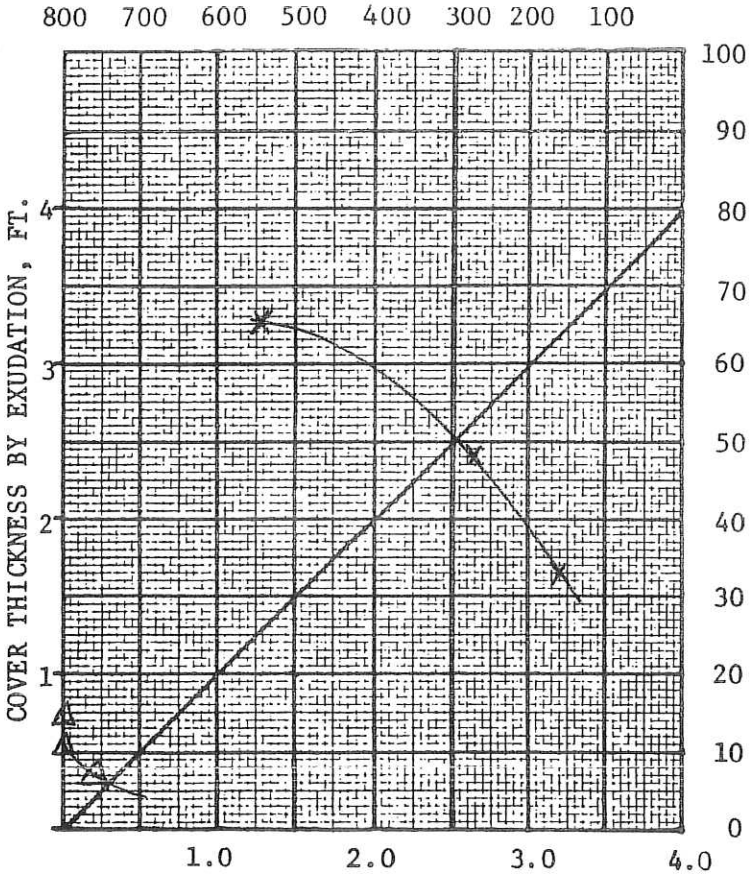
PROJECT NO. 33240
P.N. P051109-1 TP-3@0-4'
BORING NO. Meadowbrook
DATE 4/12/06

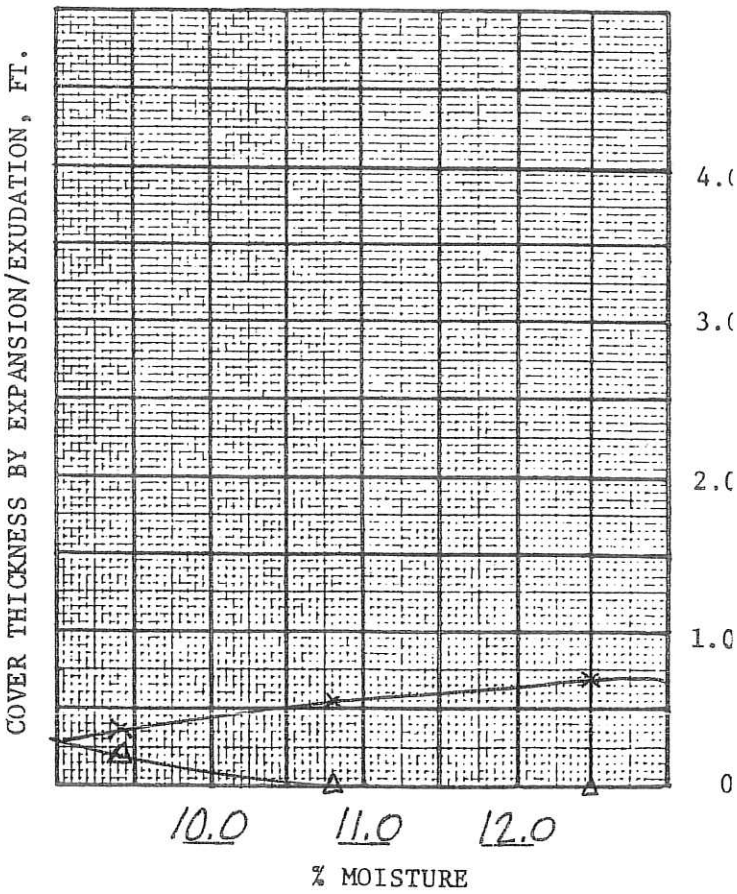
TRAFFIC INDEX Assumed 4.0

R-VALUE BY EXUDATION 50

R-VALUE BY EXPANSION —







R-VALUE vs. EXUD. PRES.

T by EXUDATION

EXUD. T vs. EXPAN. T

T by EXPANSION

REMARKS Gf = 1.25

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

R - VALUE DATA SHEET

P.N. P051109-1
Meadow Brook

PROJECT NUMBER 33240 BORING NUMBER: TP-22 @ 2'-4'

SAMPLE DESCRIPTION: Brown Medium to Coarse Sand

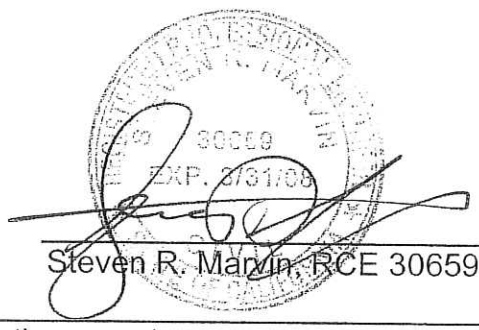
Item	SPECIMEN		
	a	b	c
Mold Number	7	8	9
Water added, grams	80	95	71
Initial Test Water, %	10.0	11.3	9.2
Compact Gage Pressure,psi	350	350	350
Exudation Pressure, psi	302	176	493
Height Sample, Inches	2.53	2.61	2.53
Gross Weight Mold, grams	3297	3328	3275
Tare Weight Mold, grams	2125	2116	2113
Sample Wet Weight, grams	1172	1212	1162
Expansion, Inches x 10exp-4	0	0	6
Stability 2,000 lbs (160psi)	15 / 29	17 / 34	11 / 20
Turns Displacement	5.36	5.41	5.31
R-Value Uncorrected	68	63	77
R-Value Corrected	68	66	77
Dry Density, pcf	127.6	126.5	127.4
DESIGN CALCULATION DATA			
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.33	0.35	0.24
G. E. by Expansion	0.00	0.00	0.20

Equilibrium R-Value 68 by EXUDATION

Examined & Checked: 4 /12/ 06

Gf = 1.25

REMARKS:



The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

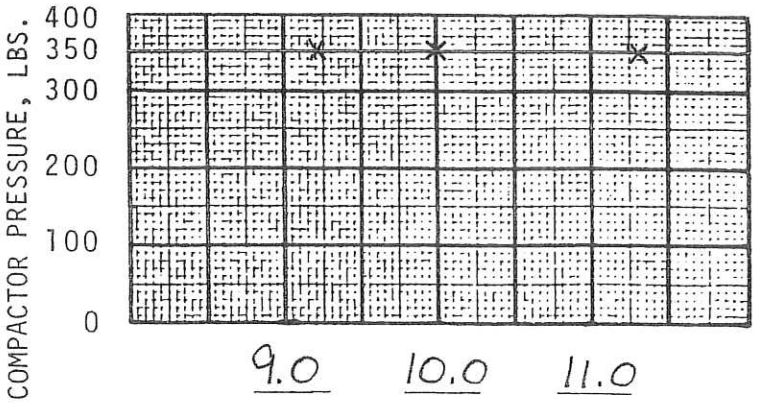
R-VALUE GRAPHICAL PRESENTATION

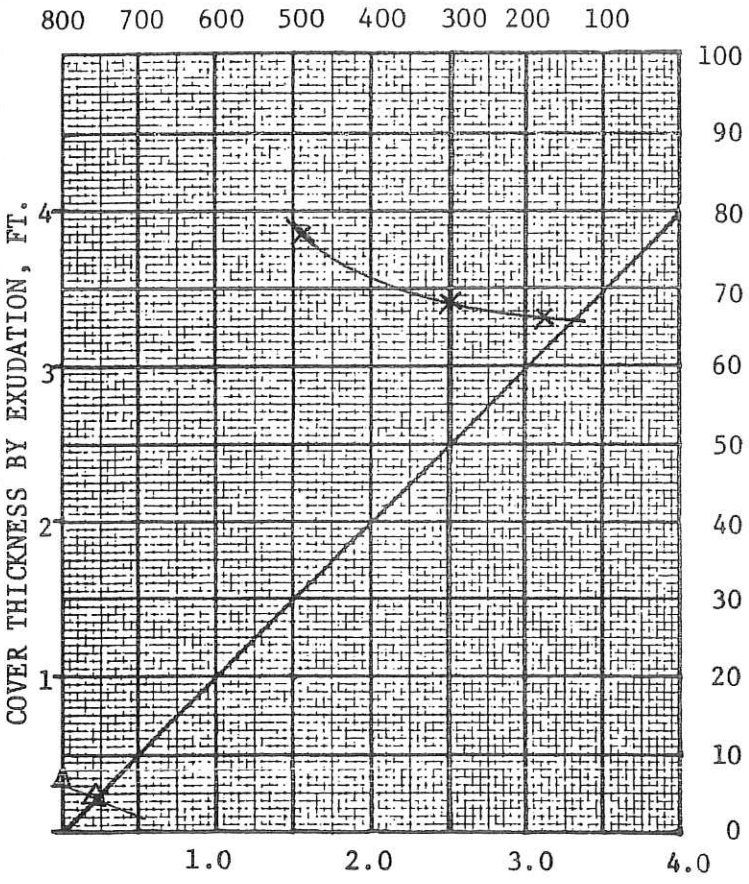
PROJECT NO. 33240
P.N. P051109-1 TP-22@2-4'
BORING NO. MeadowBrook
DATE 4/12/06

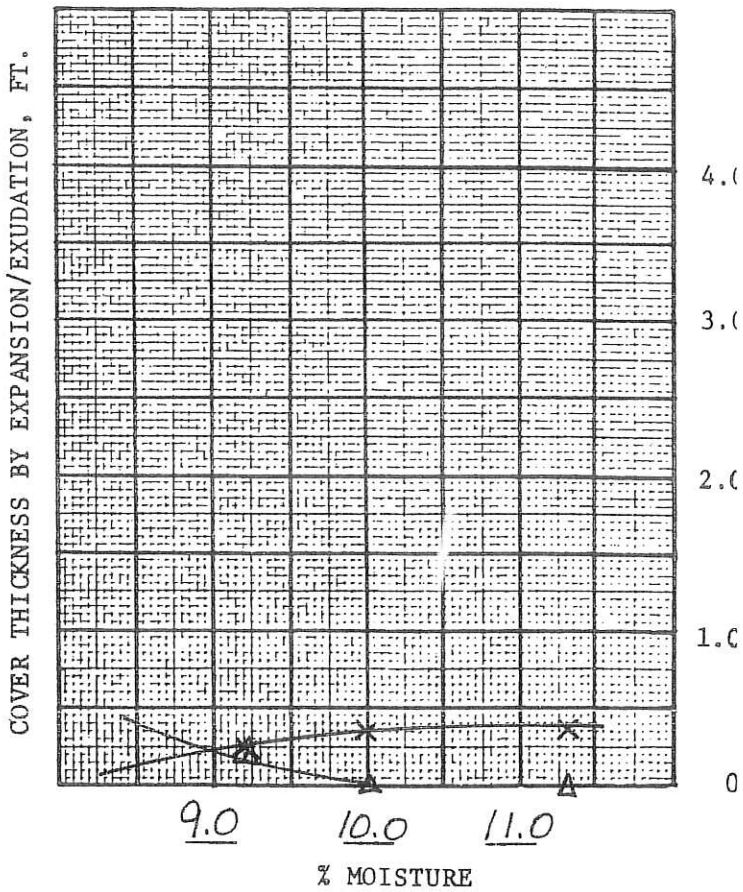
TRAFFIC INDEX Assumed 4.0

R-VALUE BY EXUDATION 68

R-VALUE BY EXPANSION —







R-VALUE vs. EXUD. PRES.
T by EXUDATION
EXUD. T vs. EXPAN. T
T by EXPANSION

REMARKS Gf=1.25

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

R - VALUE DATA SHEET

P.N. P051109-1
Meadow Brook

PROJECT NUMBER 33240 BORING NUMBER: TP-31 @ 2'-4'

SAMPLE DESCRIPTION: Brown Silty Sand

Item	SPECIMEN		
	a	b	c
Mold Number	4	5	6
Water added, grams	50	64	45
Initial Test Water, %	9.9	11.2	9.5
Compact Gage Pressure,psi	220	90	350
Exudation Pressure, psi	357	241	475
Height Sample, Inches	2.60	2.64	2.60
Gross Weight Mold, grams	3324	3322	3346
Tare Weight Mold, grams	2120	2117	2122
Sample Wet Weight, grams	1204	1205	1224
Expansion, Inches x 10exp-4	16	5	19
Stability 2,000 lbs (160psi)	27 / 53	33 / 71	26 / 49
Turns Displacement	5.13	5.73	4.82
R-Value Uncorrected	50	35	54
R-Value Corrected	53	38	57
Dry Density, pcf	127.6	124.4	130.3
DESIGN CALCULATION DATA			
Traffic Index Assumed:	4.0	4.0	4.0
G.E. by Stability	0.48	0.63	0.44
G. E. by Expansion	0.53	0.17	0.63

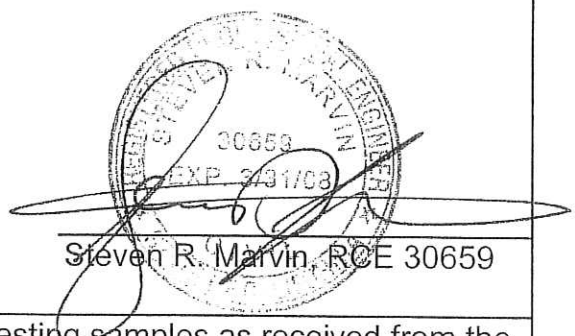
Equilibrium R-Value 46 by EXUDATION

Examined & Checked: 4 /12/ 06

Gf = 1.25

2.3% Ret. On 3/4" sieve.

REMARKS:



Steven R. Marvin, RCE 30659

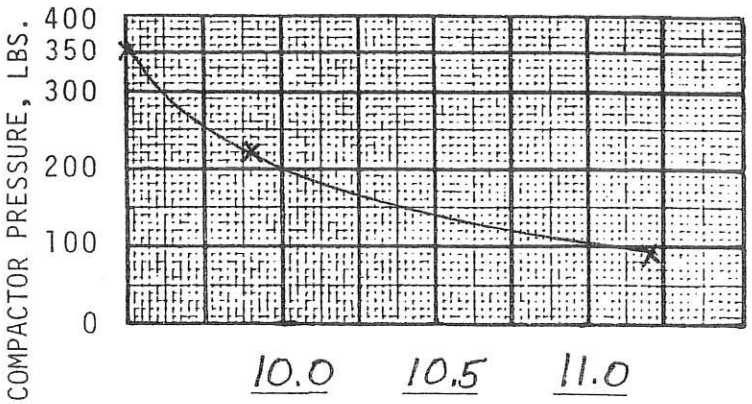
The data above is based upon processing and testing samples as received from the field. Test procedures in accordance with latest revisions to Department of Transportation, State of California, Materials & Research Test Method No. 301.

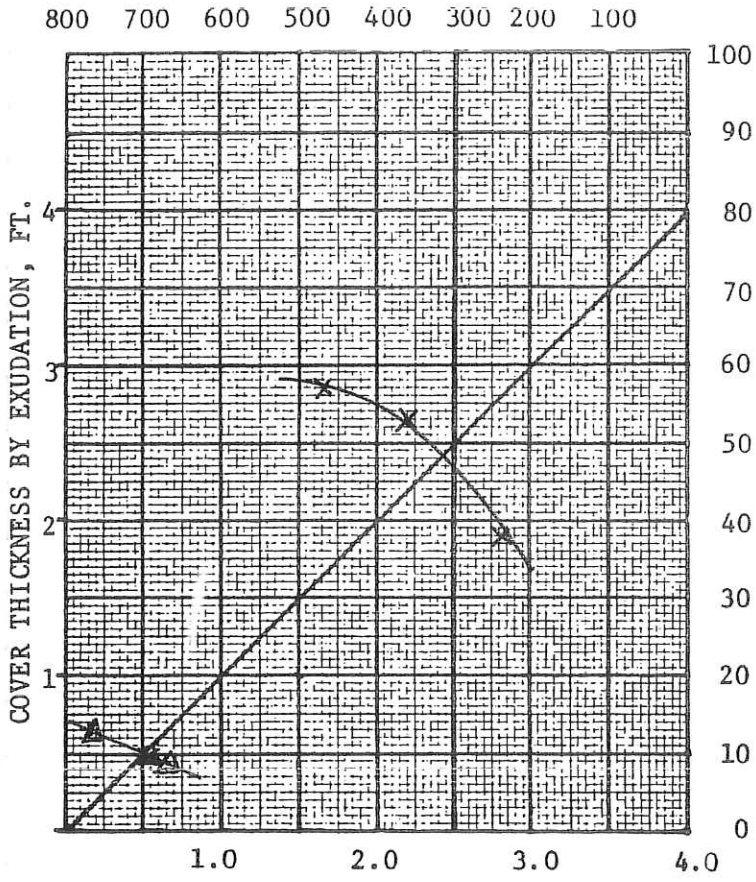
LaBelle • Marvin

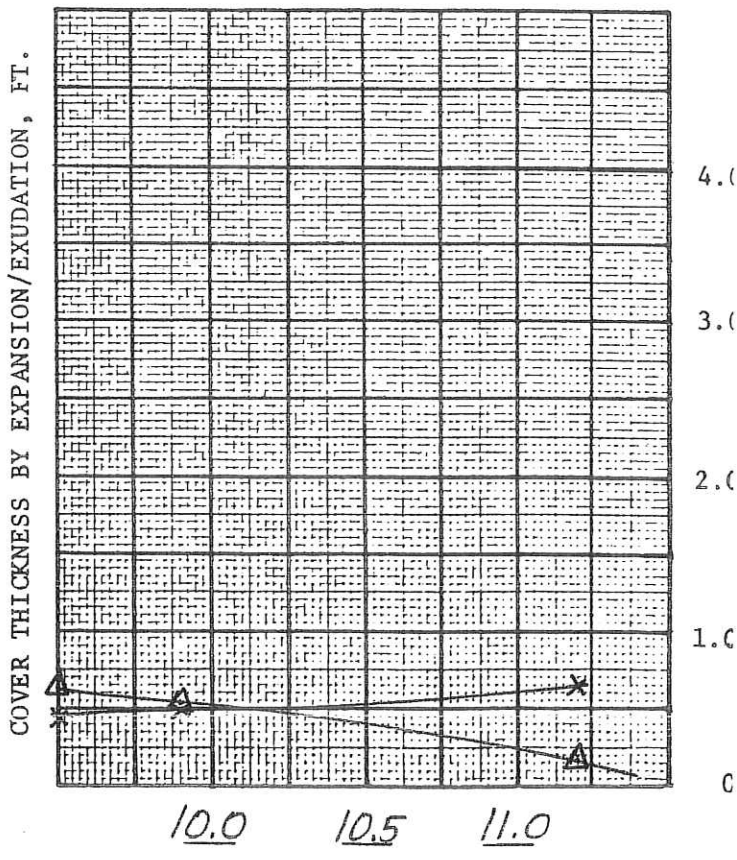
R-VALUE GRAPHICAL PRESENTATION

PROJECT NO. 33240
P.N. P051109-1 TP-31@2'-4'
BORING NO. Meadow Brook
DATE 4/12/06

TRAFFIC INDEX Assumed 4.0
R-VALUE BY EXUDATION 46
R-VALUE BY EXPANSION —







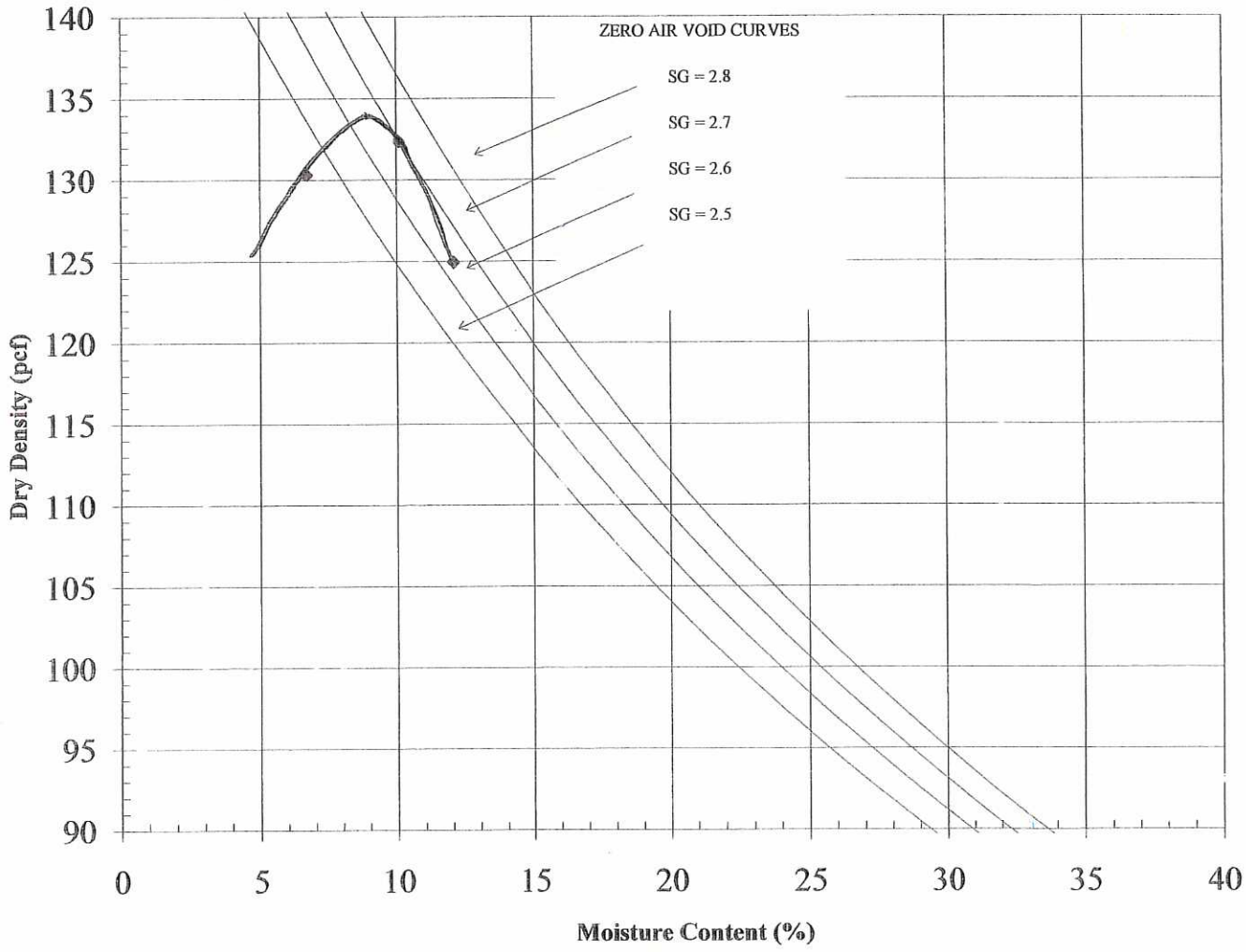
R-VALUE vs. EXUD. PRES.
T by EXUDATION
EXUD. T vs. EXPAN. T
T by EXPANSION

REMARKS Gf=1.25

LaBelle • Marvin
PROFESSIONAL PAVEMENT ENGINEERING

LABORATORY COMPACTION ASTM D 1557

Sample	1	2	3	
Mold and wet soil (lbs.)	13.882	14.104	13.918	
Mold (lbs.)	9.250	9.250	9.250	
Wet Soil (lbs.)	4.632	4.854	4.668	
Wet Density (pcf)	139.10	145.77	140.05	
Moisture (%)	6.7	10.1	12.1	
Dry Density (pcf)	130.3	132.4	125.0	



Maximum density 134 pcf @ 9.0% moisture

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadow Brook

Procedure: Method A

Job Number: P051109-1

Date: 4/04/06

Boring Number: TP-3

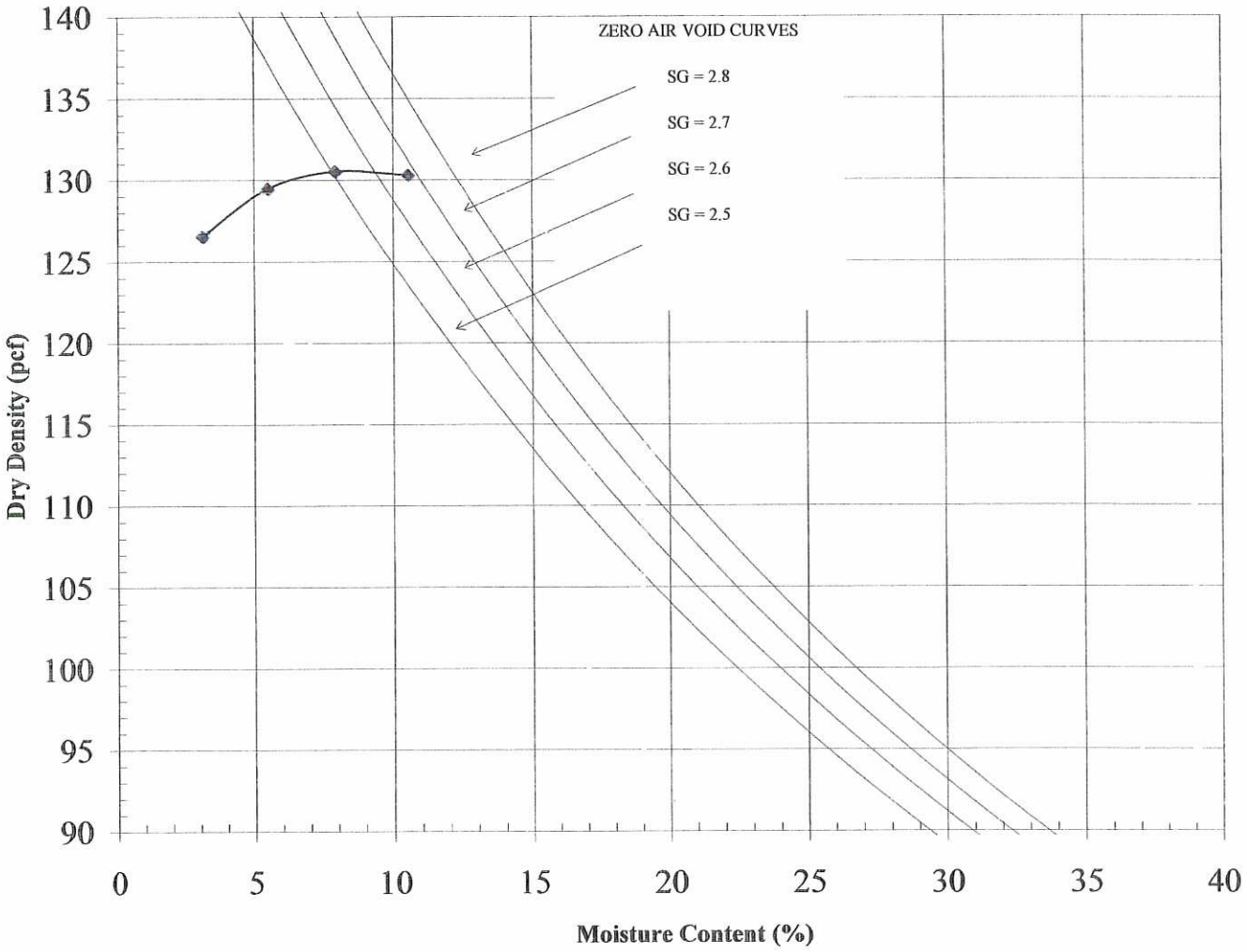
Location: 0-4 Feet

Soil Description: Silty, Clayey Sand, SC-SM

Tested by: AW

LABORATORY COMPACTION ASTM D 1557

Sample	1	2	3	4
Mold and wet soil (lbs.)	13.594	13.798	13.946	14.050
Mold (lbs.)	9.250	9.250	9.250	9.250
Wet Soil (lbs.)	4.344	4.548	4.696	4.800
Wet Density (pcf)	130.45	136.58	140.89	144.01
Moisture (%)	3.1	5.5	7.9	10.5
Dry Density (pcf)	126.6	129.5	130.6	130.3



Maximum density 131 pcf @ 9% moisture

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadow Brook

Procedure: Method A

Job Number: P051109-1

Date: 4/04/06

Boring Number: TP-22

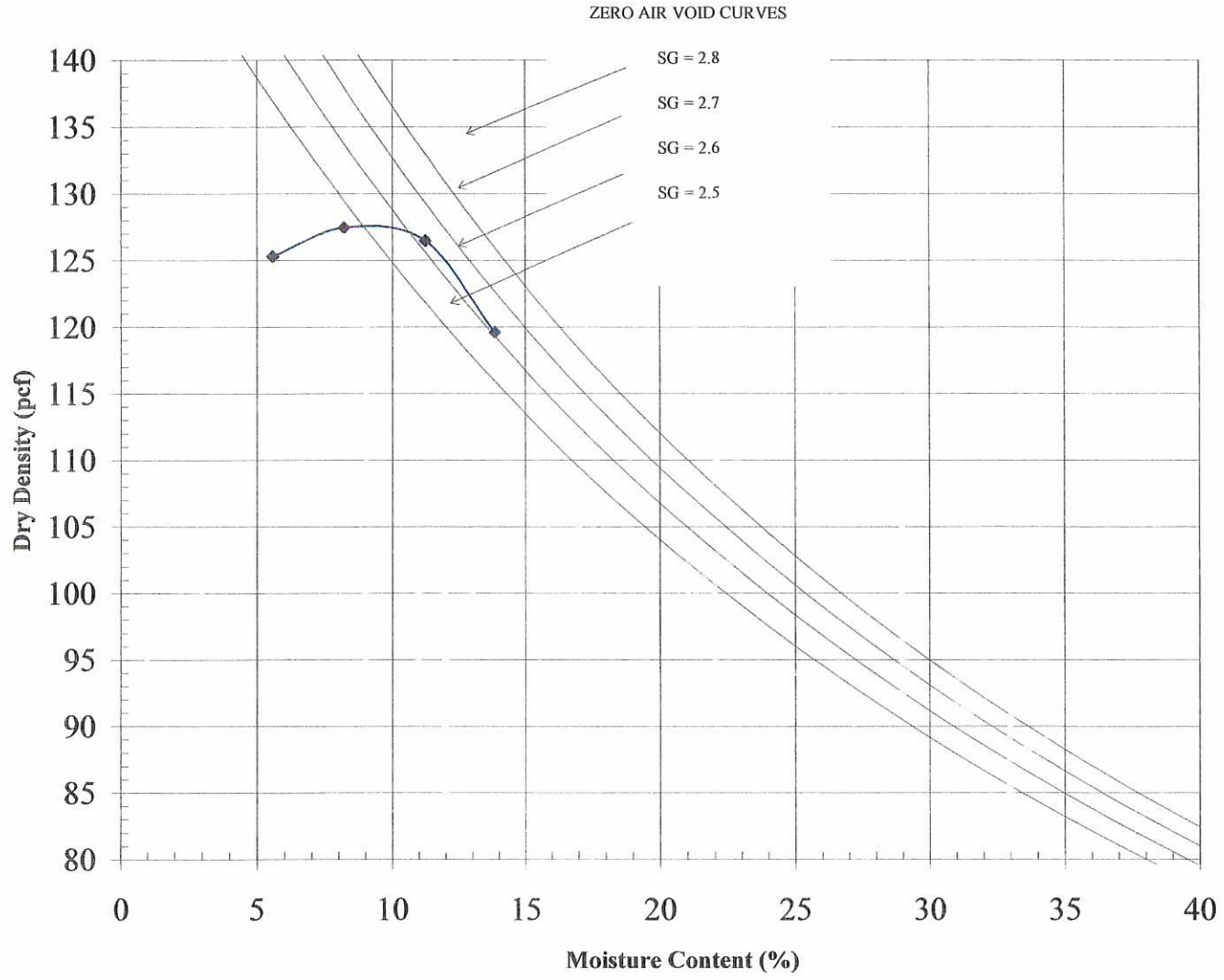
Location: 2-4 Feet

Soil Description: Well Graded Sand with Silt, SW-SM

Tested by: AW

LABORATORY COMPACTION ASTM D 1557

Sample	1	2	3	4
Mold and wet soil (lbs.)	16.078	16.500	16.708	16.368
Mold (lbs.)	6.154	6.154	6.154	6.154
Wet Soil (lbs.)	9.924	10.346	10.554	10.214
Wet Density (pcf)	132.32	137.95	140.72	136.19
Moisture (%)	5.6	8.2	11.2	13.9
Dry Density (pcf)	125.3	127.5	126.5	119.6



Maximum density 138 pcf @ 9% moisture

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Lowe's HIW, Inc.

Project Name: Meadow Brook

Procedure: Method C

Job Number: P051109-1

Date: 4/12/06

Boring Number: TP-31

Location: 0-4' Feet

Soil Description: Silty Clayey Sand with Gravel, SC-SM

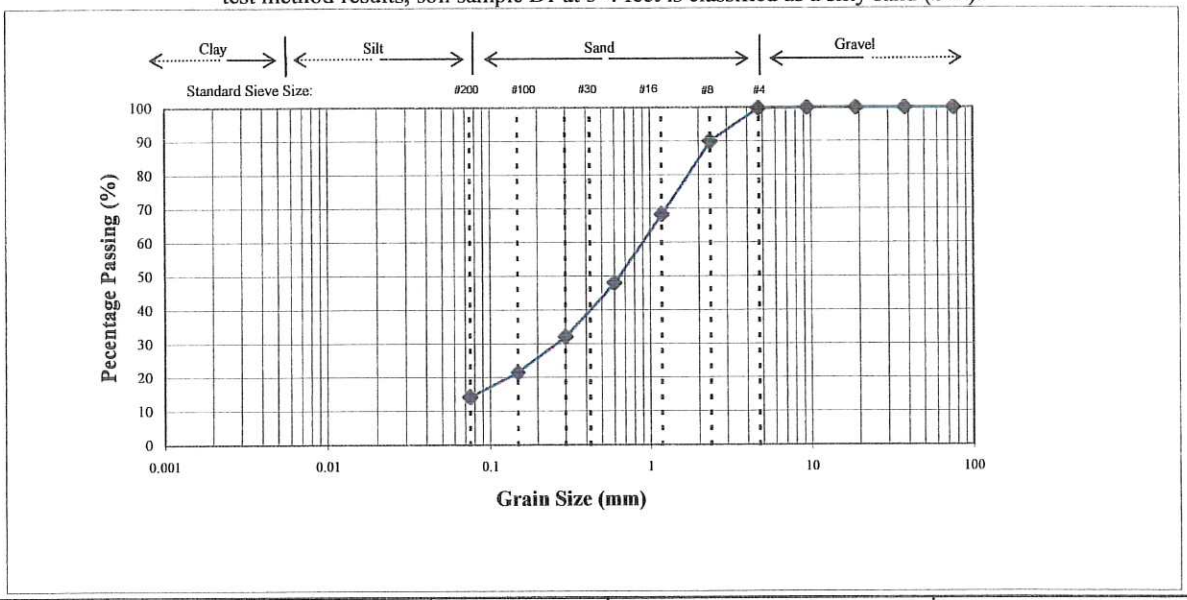
Tested by: AW

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B1 3-4'
Total Weight (g)	340.2
Dry Weight (g)	327.3
Wet Sieve Weight (g)	285.6
Initial Moisture (%)	3.9

D10 (mm)	NA
D30 (mm)	0.25
D60 (mm)	0.90
Cu	NA
Cc	NA

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B1 at 3-4 feet is classified as a silty sand (SM).



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.6	0.2	99.8
#8	2.36	32.5	9.9	90.1
#16	1.18	103.3	31.6	68.4
#30	0.6	169.8	51.9	48.1
#50	0.3	221.9	67.8	32.2
#100	0.15	256.7	78.4	21.6
#200	0.075	280.8	85.8	14.2

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4/20/06

Boring Number: B1

Location: 3-4 Feet

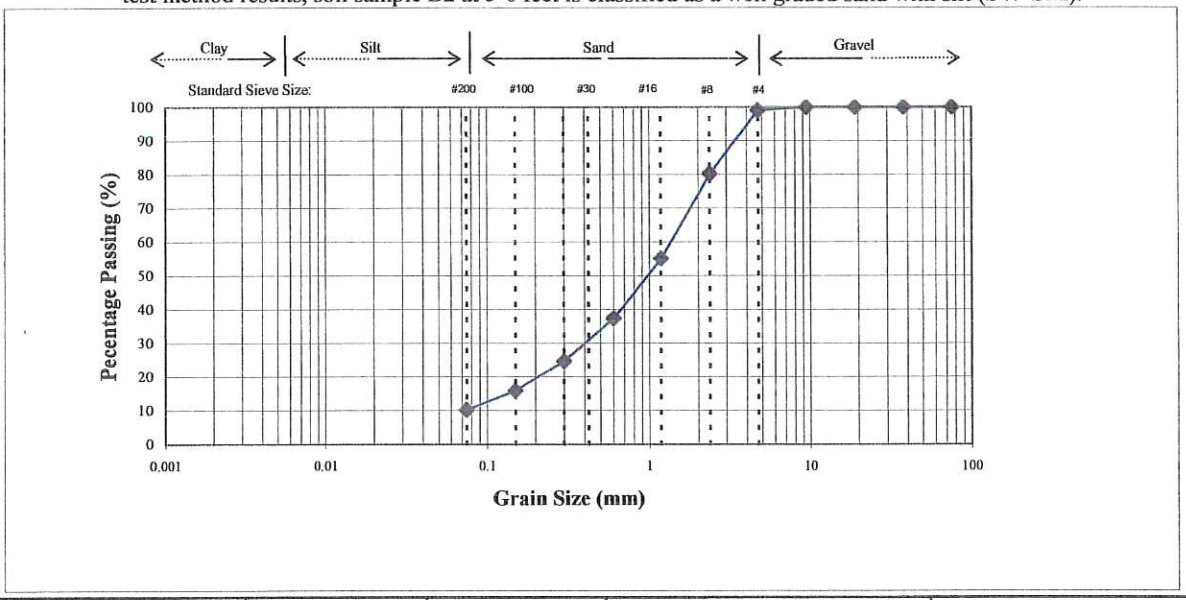
Soil Description: Silty Sand, SM

Tested by: AW

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B2 5-6'		D10 (mm)	0.08
Total Weight (g)	225.0		D30 (mm)	0.40
Dry Weight (g)	212.4		D60 (mm)	1.50
Wet Sieve Weight (g)	193.5		Cu	20.00
Initial Moisture (%)	5.9		Cc	1.42

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B2 at 5-6 feet is classified as a well graded sand with silt (**SW-SM**).



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	2.0	0.9	99.1
#8	2.36	41.7	19.6	80.4
#16	1.18	95.3	44.9	55.1
#30	0.6	132.9	62.6	37.4
#50	0.3	159.9	75.3	24.7
#100	0.15	178.6	84.1	15.9
#200	0.075	190.7	89.8	10.2

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4/20/06

Boring Number: B2

Location: 5-6 Feet

Soil Description: Well Graded Sand with Silt, SW-SM

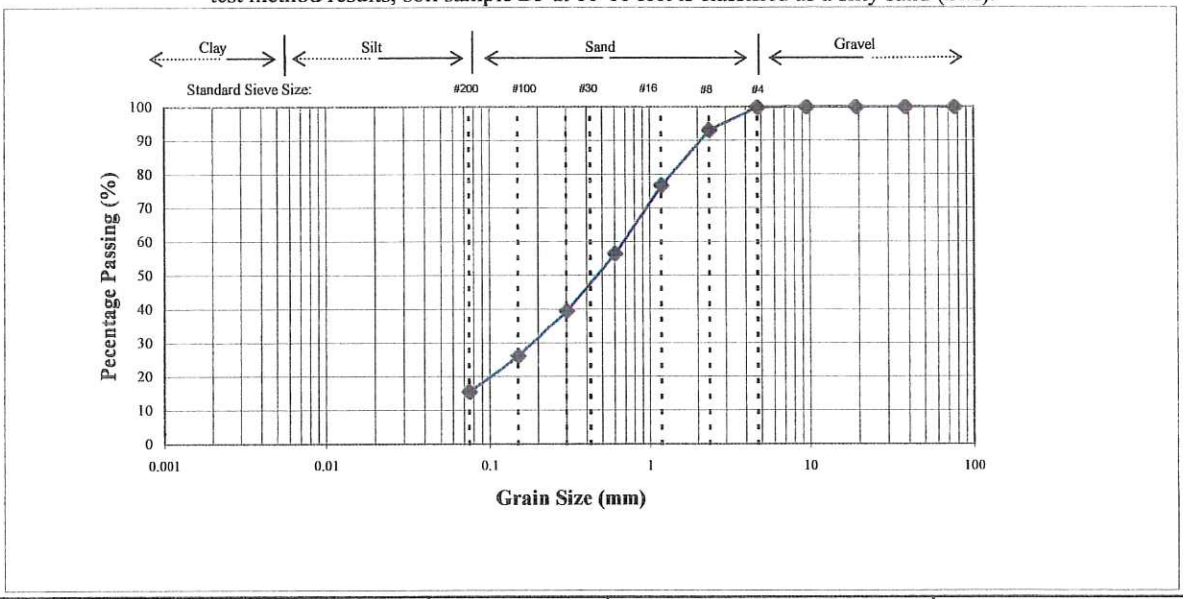
Tested by: AW

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	**B3 10-11'**
Total Weight (g)	**283.9**
Dry Weight (g)	**272.2**
Wet Sieve Weight (g)	**238.0**
Initial Moisture (%)	4.3

D_{10} (mm)	**NA**
D_{30} (mm)	**0.20**
D_{60} (mm)	**0.70**
C_u	NA
C_c	NA

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B3 at 10-11 feet is classified as a silty sand (SM).



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.3	0.1	99.9
#8	2.36	18.6	6.8	93.2
#16	1.18	63.0	23.1	76.9
#30	0.6	118.3	43.5	56.5
#50	0.3	164.4	60.4	39.6
#100	0.15	200.9	73.8	26.2
#200	0.075	229.9	84.5	15.5

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4/20/06

Boring Number: B3

Location: 10-11 Feet

Soil Description: Silty Sand, SM

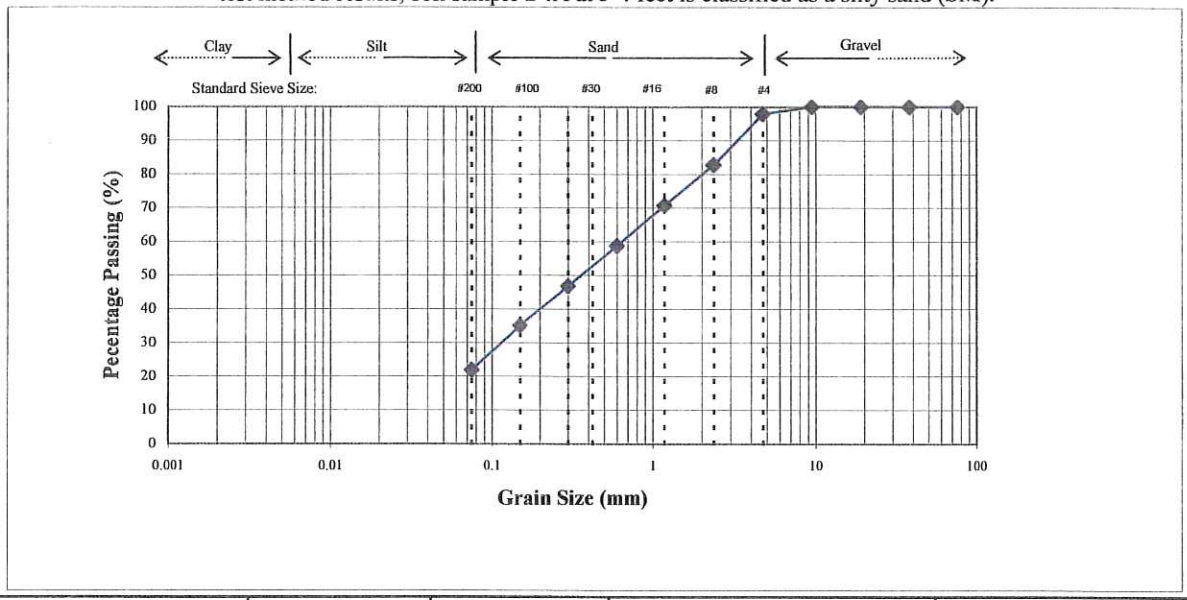
Tested by: AW

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	B4A 3-4'
Total Weight (g)	186.3
Dry Weight (g)	165.6
Wet Sieve Weight (g)	129.2
Initial Moisture (%)	12.5

D10 (mm)	NA
D30 (mm)	0.10
D60 (mm)	0.60
Cu	NA
Cc	NA

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample B4A at 3-4 feet is classified as a silty sand (**SM**).



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	3.4	2.1	97.9
#8	2.36	28.5	17.2	82.8
#16	1.18	48.3	29.2	70.8
#30	0.6	68.3	41.2	58.8
#50	0.3	88.0	53.1	46.9
#100	0.15	107.4	64.9	35.1
#200	0.075	129.2	78.0	22.0

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4/20/06

Boring Number: B4A

Location: 3-4 Feet

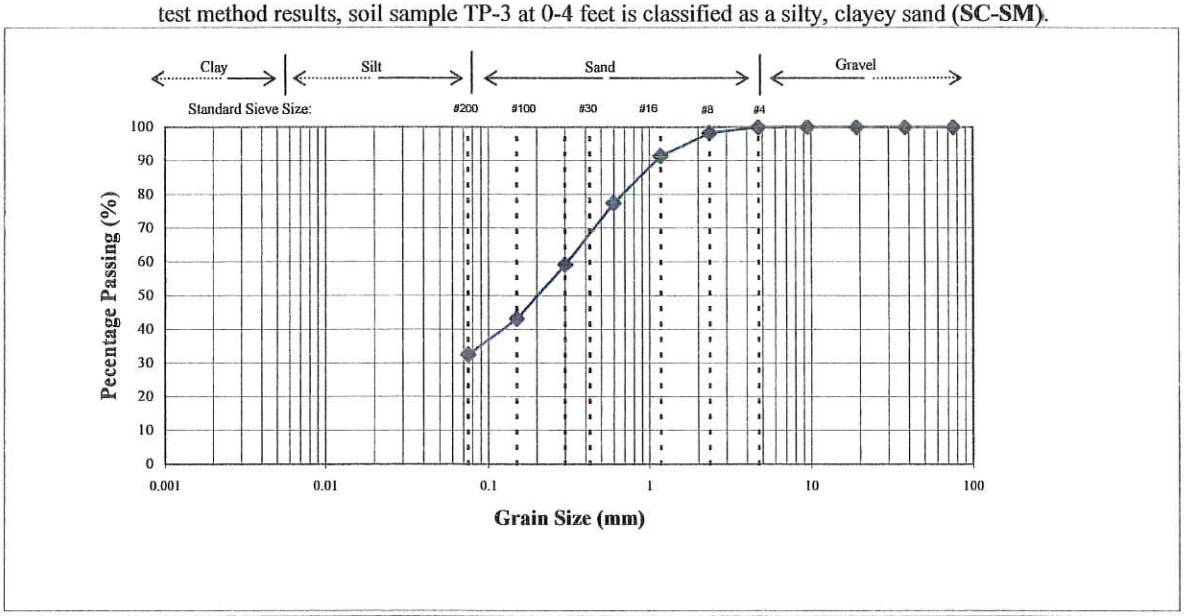
Soil Description: Silty Sand, SM

Tested by: AW

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	TP-3		D10 (mm)	NA
Total Weight (g)	220.3		D30 (mm)	NA
Dry Weight (g)	205.8		D60 (mm)	0.30
Wet Sieve Weight (g)	146.4		Cu	NA
Initial Moisture (%)	7.0		Cc	NA

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample TP-3 at 0-4 feet is classified as a silty, clayey sand **(SC-SM)**.



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.2	0.1	99.9
#8	2.36	3.8	1.8	98.2
#16	1.18	17.6	8.6	91.4
#30	0.6	46.4	22.5	77.5
#50	0.3	83.9	40.8	59.2
#100	0.15	116.9	56.8	43.2
#200	0.075	138.9	67.5	32.5

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4/13/06

Boring Number: TP-3

Location: 0-4 Feet

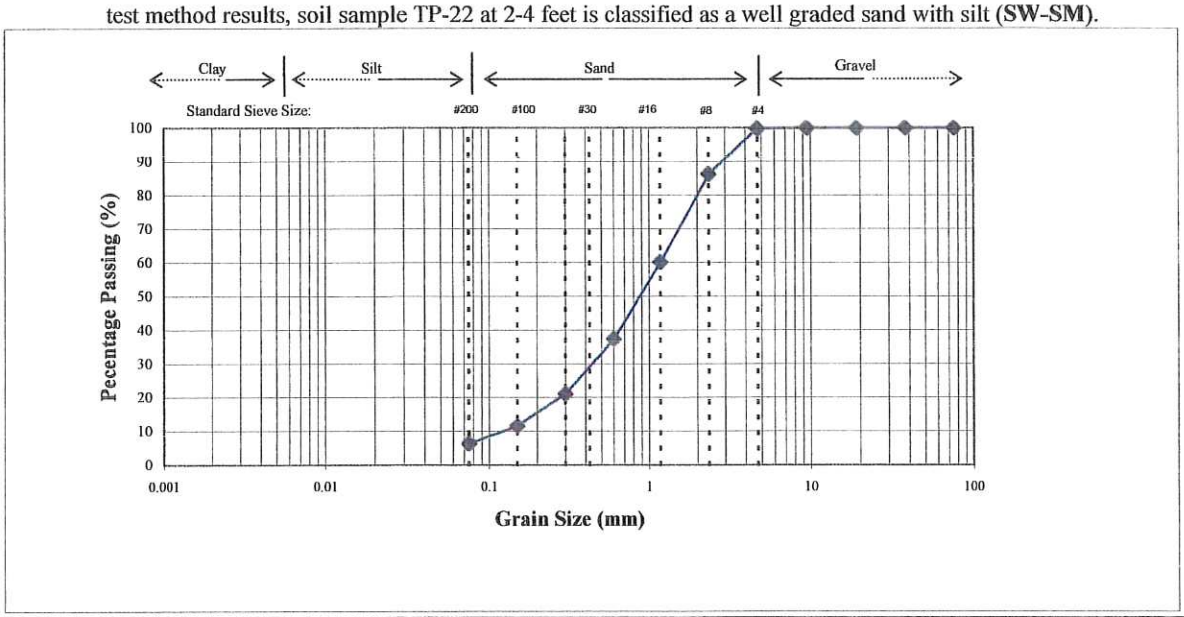
Soil Description: Silty, Clayey Sand SC-SM

Tested by: AW

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	TP-22		D10 (mm)	0.10
Total Weight (g)	289.8		D30 (mm)	0.40
Dry Weight (g)	280.1		D60 (mm)	1.25
Wet Sieve Weight (g)	265.6		Cu	12.50
Initial Moisture (%)	3.5		Cc	1.28

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample TP-22 at 2-4 feet is classified as a well graded sand with silt (**SW-SM**).



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
3"	76.2	0.0	0.0	100.0
1.5"	38.1	0.0	0.0	100.0
3/4"	19.05	0.0	0.0	100.0
3/8"	9.53	0.0	0.0	100.0
#4	4.75	0.5	0.2	99.8
#8	2.36	38.3	13.7	86.3
#16	1.18	111.3	39.7	60.3
#30	0.6	175.0	62.5	37.5
#50	0.3	220.8	78.8	21.2
#100	0.15	247.6	88.4	11.6
#200	0.075	262.2	93.6	6.4

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4/13/06

Boring Number: TP-22

Location: 2-4 Feet

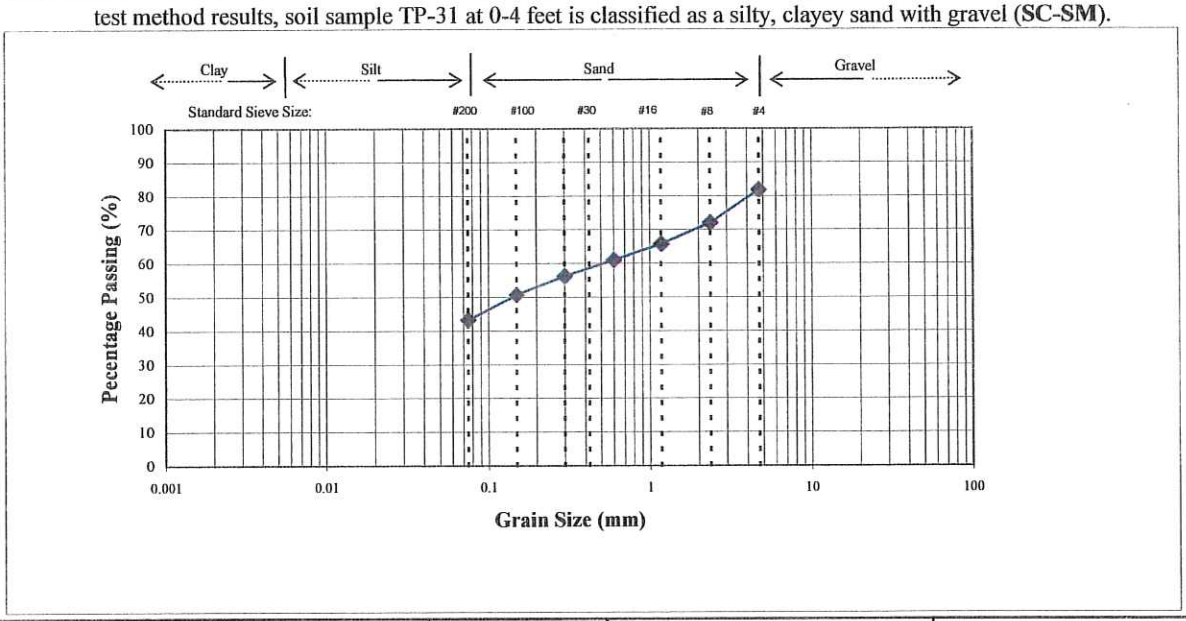
Soil Description: Well Graded Sand with Silt, SW-SM

Tested by: AW

PARTICLE-SIZE ANALYSIS OF SOILS
ASTM METHOD D 422 (SIEVE ANALYSIS)

Sample :	TP-31 0-4'		D10 (mm)	NA
Total Weight (g)	205.5		D30 (mm)	NA
Dry Weight (g)	194.3		D60 (mm)	0.50
Wet Sieve Weight (g)	119.7		Cu	NA
Initial Moisture (%)	5.8		Cc	NA

According to ASTM D 2487 Unified Soil Classification System (USCS) and ASTM D 422 (Standard Test Method for Particle-Size Analysis) test method results, soil sample TP-31 at 0-4 feet is classified as a silty, clayey sand with gravel (SC-SM).



Sieve Size (in)	Sieve Size (mm)	Cumulative Weight of dry soil (gm)	Percent Retained (%)	Percent Passing (%)
#4	4.75	35.2	18.1	81.9
#8	2.36	54.1	27.8	72.2
#16	1.18	66.3	34.1	65.9
#30	0.6	75.7	39.0	61.0
#50	0.3	84.9	43.7	56.3
#100	0.15	95.7	49.3	50.7
#200	0.075	110.1	56.7	43.3

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5845 Avenida Encinas, Ste. 128 Carlsbad, CA 92008

Client: Shopoff Group

Project Name: Meadowbrook

Job Number: P051109-1

Date: 4/13/06

Boring Number: TP-31

Location: 0-4 Feet

Soil Description: Silty, Clayey Sand with Gravel SC-SM

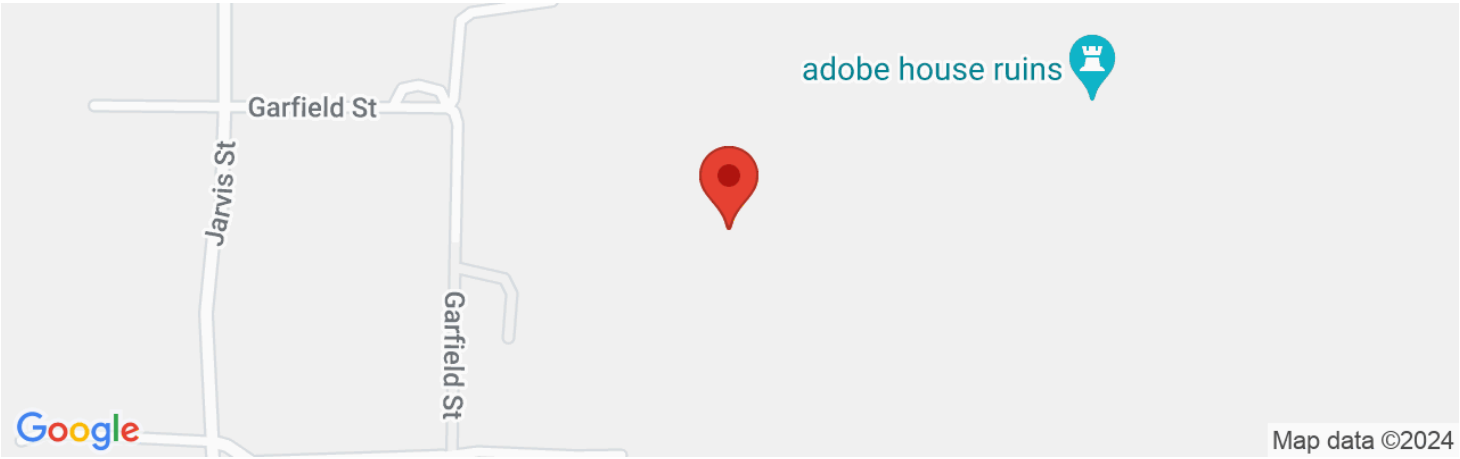
Tested by: AW

APPENDIX C
SEISMIC ANALYSIS

Leighton
a verdantas company

USGS web services were down for some period of time and as a result this tool wasn't operational, resulting in *timeout* error.
USGS web services are now operational so this tool should work as expected.

Latitude, Longitude: 33.7169, -117.2814



Date	2/23/2024, 4:59:50 PM
Design Code Reference Document	ASCE7-16
Risk Category	II
Site Class	B - Rock

Type	Value	Description
S_S	1.57	MCE_R ground motion. (for 0.2 second period)
S_1	0.584	MCE_R ground motion. (for 1.0s period)
S_{MS}	1.413	Site-modified spectral acceleration value
S_{M1}	0.467	Site-modified spectral acceleration value
S_{DS}	0.942	Numeric seismic design value at 0.2 second SA
S_{D1}	0.312	Numeric seismic design value at 1.0 second SA

Type	Value	Description
SDC	D	Seismic design category
F_a	0.9	Site amplification factor at 0.2 second
F_v	0.8	Site amplification factor at 1.0 second
PGA	0.667	MCE_G peak ground acceleration
F_{PGA}	0.9	Site amplification factor at PGA
PGA_M	0.6	Site modified peak ground acceleration
T_L	8	Long-period transition period in seconds
SsRT	1.614	Probabilistic risk-targeted ground motion. (0.2 second)
SsUH	1.746	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration
SsD	1.57	Factored deterministic acceleration value. (0.2 second)
S1RT	0.584	Probabilistic risk-targeted ground motion. (1.0 second)
S1UH	0.64	Factored uniform-hazard (2% probability of exceedance in 50 years) spectral acceleration.
S1D	0.607	Factored deterministic acceleration value. (1.0 second)
PGAd	0.667	Factored deterministic acceleration value. (Peak Ground Acceleration)
PGA_{UH}	0.674	Uniform-hazard (2% probability of exceedance in 50 years) Peak Ground Acceleration
C_{RS}	0.925	Mapped value of the risk coefficient at short periods
C_{R1}	0.913	Mapped value of the risk coefficient at a period of 1 s
C_V	0.9	Vertical coefficient

DISCLAIMER

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U.S. Geological Survey - Earthquake Hazards Program

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.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (4.2.0)

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

33.7169

Time Horizon

Return period in years

2475

Longitude

Decimal degrees, negative values for western longitudes

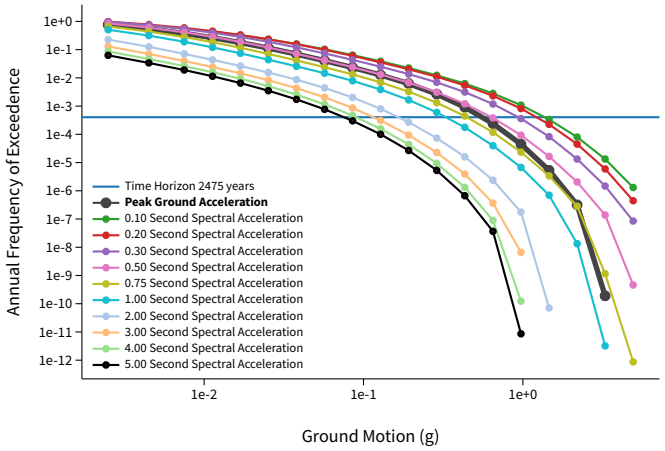
-117.2814

Site Class

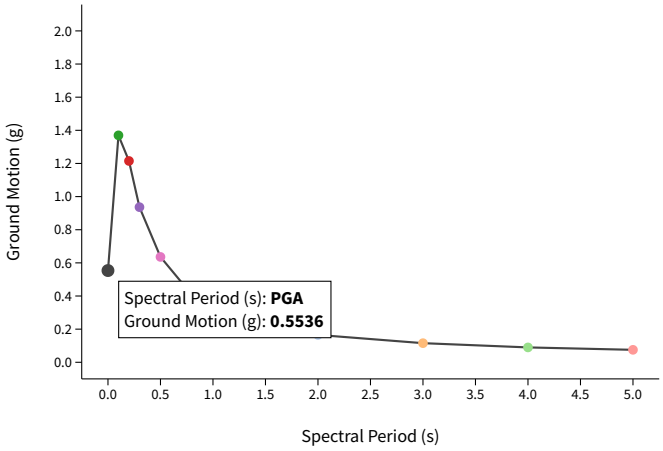
1150 m/s (Site class B)

Hazard Curve

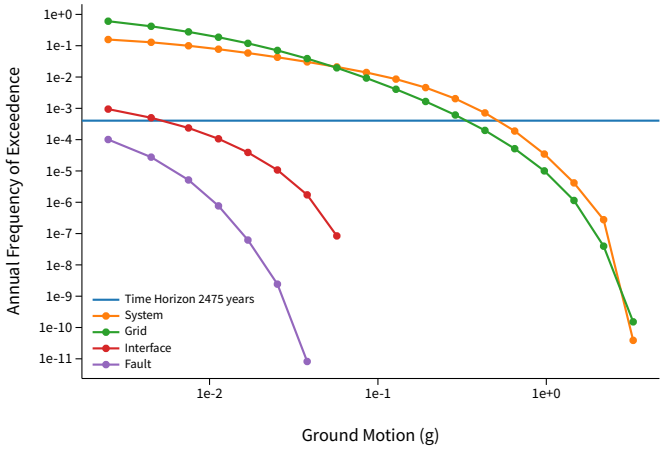
Hazard Curves



Uniform Hazard Response Spectrum



Component Curves for Peak Ground Acceleration

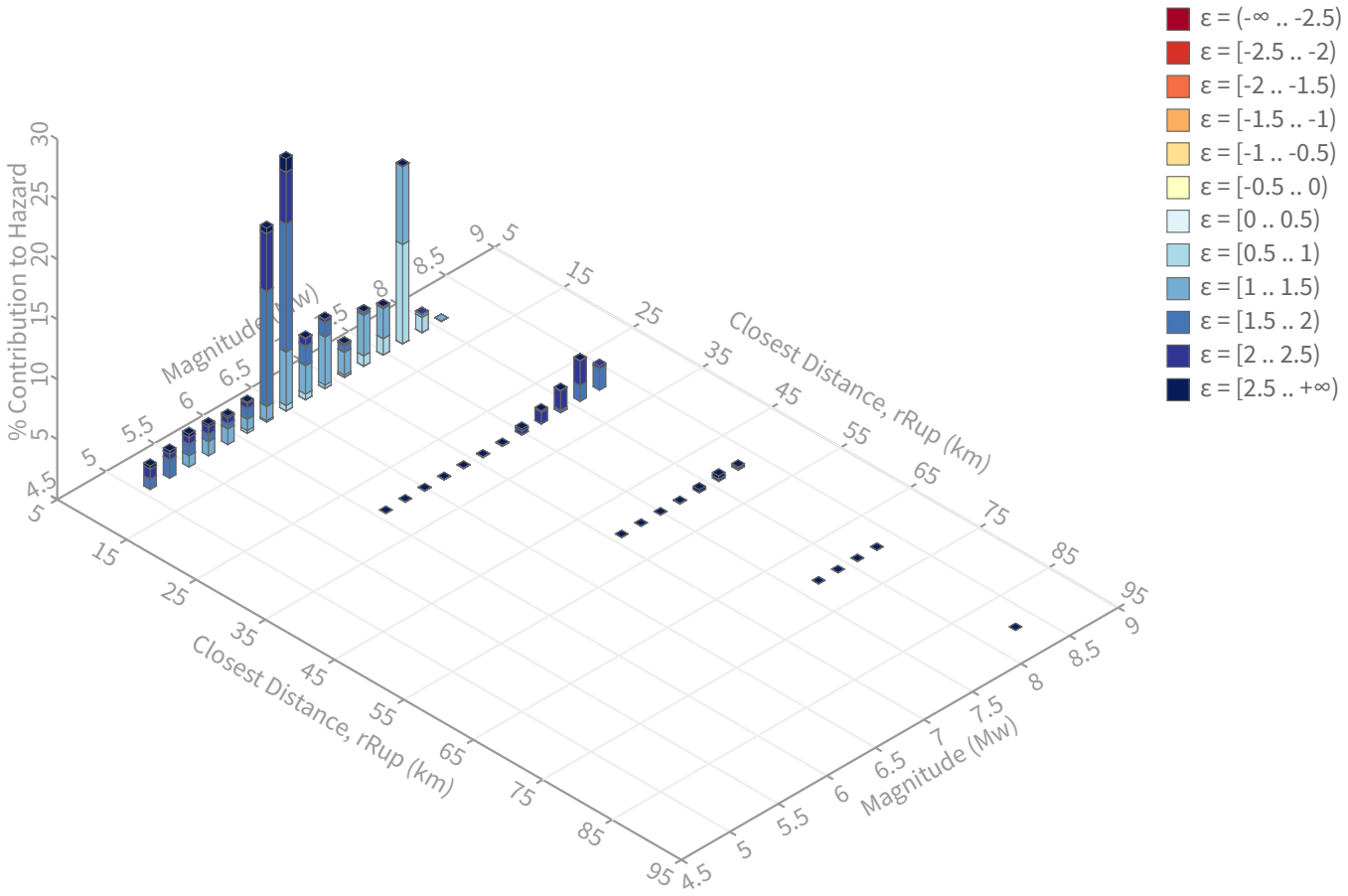


View Raw Data

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.55363796 g

Recovered targets

Return period: 2817.0191 yrs
Exceedance rate: 0.00035498517 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.06 %

Mean (over all sources)

m: 6.81
r: 11.44 km
ε_0: 1.59 σ

Mode (largest m-r bin)

m: 6.5
r: 11.57 km
ε_0: 1.83 σ
Contribution: 20.99 %

Mode (largest m-r-ε_0 bin)

m: 6.5
r: 11.82 km
ε_0: 1.84 σ
Contribution: 10.66 %

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							39.79
Elsinore (Stepovers Combined) [0]		8.51	7.42	1.11	117.342°W	33.661°N	221.96	16.34
Elsinore (Glen Ivy) rev [3]		9.25	6.41	1.67	117.373°W	33.685°N	247.39	10.80
Elsinore (Glen Ivy) rev [2]		13.62	6.51	2.08	117.428°W	33.721°N	271.89	6.77
San Jacinto (Stepovers Combined) [0]		27.07	8.04	2.05	117.058°W	33.875°N	49.58	4.08
UC33brAvg_FM31	System							39.12
Elsinore (Stepovers Combined) [0]		8.51	7.39	1.12	117.342°W	33.661°N	221.96	16.14
Elsinore (Glen Ivy) rev [3]		9.25	6.41	1.67	117.373°W	33.685°N	247.39	10.51
Elsinore (Glen Ivy) rev [2]		13.62	6.48	2.10	117.428°W	33.721°N	271.89	6.58
San Jacinto (Stepovers Combined) [0]		27.07	8.04	2.05	117.058°W	33.875°N	49.58	4.11
UC33brAvg_FM31 (opt)	Grid							10.55
PointSourceFinite: -117.281, 33.739		5.75	5.56	1.52	117.281°W	33.739°N	0.00	2.25
PointSourceFinite: -117.281, 33.739		5.75	5.56	1.52	117.281°W	33.739°N	0.00	2.25
PointSourceFinite: -117.281, 33.784		7.95	6.04	1.60	117.281°W	33.784°N	0.00	1.03
PointSourceFinite: -117.281, 33.784		7.95	6.04	1.60	117.281°W	33.784°N	0.00	1.03
UC33brAvg_FM32 (opt)	Grid							10.54
PointSourceFinite: -117.281, 33.739		5.75	5.56	1.52	117.281°W	33.739°N	0.00	2.25
PointSourceFinite: -117.281, 33.739		5.75	5.56	1.52	117.281°W	33.739°N	0.00	2.25
PointSourceFinite: -117.281, 33.784		7.95	6.04	1.60	117.281°W	33.784°N	0.00	1.02
PointSourceFinite: -117.281, 33.784		7.95	6.04	1.60	117.281°W	33.784°N	0.00	1.02

APPENDIX D
SEISMIC REFRACTION SURVEY

Leighton
a verdantas company

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geophysical services

REPORT

SEISMIC REFRACTION SURVEY

Proposed Cresent Cemetery Project
Riverside County, California

GEO*Vision* Project No. 24019

Prepared for

Leighton Group, Inc.
10532 Acacia Street
Rancho Cucamonga, California 91730
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Prepared by

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Report 24019-01 Rev 0

February 2, 2024

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2 METHODOLOGY **3**
3 EQUIPMENT AND FIELD PROCEDURES **5**
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Figure 6 Line 5 P-wave Seismic Refraction Model 12

1 INTRODUCTION

A P-wave seismic refraction survey was conducted near Garfield Road and Robert Street in Perris, California on January 19th, 2024. The purpose of this investigation was to determine the rippability of the Jurassic sedimentary (Lines 1-2) and Mesozoic granitic rock (Lines 3-5) at the site. P-wave seismic data was acquired along five profiles, designated as Lines 1, 2, 3, 4 and 5 (Figure 1).

Depending on the degree of weathering, jointing, etc., granitic rock may broadly be characterized as rippable using a Caterpillar D8R, D9R, and D10R ripper at P-wave velocities less than 5,800, 6,800, and 7,200 ft/s, respectively (Caterpillar, 2018). Granitic rock is expected to be marginally rippable by a D8R, D9R, and D10R between a P-wave velocity of 5,800 and 8,000 ft/s, 6,800 and 8,000 ft/s, and 7,200 and 8,500 ft/s, respectively, and non-rippable at higher velocities (Caterpillar, 2018). A metasedimentary interface can be characterized as rippable using a Caterpillar D8R, D9R, and D10R ripper at P-wave velocities 6,300, 7,200, and 8,000 ft/s respectively (Caterpillar, 2018). As well as marginally rippable D8R, D9R, and D10R between a P-wave velocity of 6,300 and 8,400 ft/s, 7,200 and 9,200 ft/s, and 8,000 and 9,900 ft/s, respectively, and non-rippable at higher velocities. (Caterpillar, 2018).

The following sections include a discussion of equipment and field procedures, methodology, data processing, and results of the geophysical survey.



Seismic Refraction Line

Notes:
Coordinate System: NAD 1983 StatePlane California VI FIPS 0406 Feet
Base map source: Nearmap (9-2023)

GEOVision geophysical services

Date:	1/26/2024
GV Project:	24019
Developed by:	C Xiong
Drawn by:	T Rodriguez
Approved by:	A Martin
File Name:	GV_24019

FIGURE 1
SITE MAP

PROPOSED CRESENT CEMETERY PROJECT
RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR
LEIGHTON GROUP, INC.

2 METHODOLOGY

Detailed discussions of the seismic refraction method can be found in Telford et al. (1990), Dobrin and Savit (1988), and Redpath (1973).

When conducting a seismic survey, acoustic energy is input to the subsurface by an energy source such as a sledgehammer impacting a metallic plate, weight drop, vibratory source, or explosive charge. The acoustic waves propagate into the subsurface at a velocity dependent upon the elastic properties of the material through which they travel. When the waves reach an interface where the density or velocity changes significantly, a portion of the energy is reflected to the surface and the remainder is transmitted into the lower layer. Where the velocity of the lower layer is higher than that of the upper layer, a portion of the energy is also critically refracted along the interface. Critically refracted waves travel along the interface at the velocity of the lower layer and continually refract energy back to the surface. Receivers (geophones) laid out in linear array on the surface record the incoming refracted and reflected waves. The seismic refraction method involves analysis of the travel times of the first energy to arrive at the geophones. These seismic first arrivals are from either the direct wave (at geophones close to the source) or critically refracted waves (at geophones further from the source).

Analysis of seismic refraction data depends upon the complexity of the subsurface velocity structure. If the subsurface target is planar then the slope intercept method (Telford et al., 1990) can be used to model multiple horizontal or dipping planar layers. A minimum of one end shot is required to model horizontal layers and reverse end shots are required to model dipping planar layers. If the subsurface target is undulating (i.e. bedrock valley) then layer based analysis routines such as the generalized reciprocal method (Palmer, 1980 and 1981, Lankston and Lankston, 1986 and Lankston, 1990); reciprocal method (Hawkins, 1961) also referred to as the ABC method; Hales' method (Hales, 1958); delay time method (Wyrobek, 1956 and Gardner, 1967); time-term inversion (Scheidegger and Willmore, 1959); plus-minus method (Hagedoorn, 1959); and wavefront method (Rockwell, 1967) are preferred to model subsurface velocity structure. These methods generally require a minimum of 5 shot points per spread (end shots, off end shots and a center shot). If subsurface velocity structure is complex and cannot be adequately modeled using layer-based modeling techniques (e.g., complex weathering profile in bedrock, numerous lateral velocity variations), then Monte Carlo or tomographic inversion techniques (Zhang and Toksoz, 1998; Schuster and Quintus-Bosz, 1993) are required to model the seismic refraction data. These techniques require a high shot density; typically, every 3 to 6 stations/geophones.

Errors in seismic refraction models not associated with errors in first arrival data can be caused by blind zones, hidden layers, and lateral velocity variability. A blind zone is a geologic layer with a lower seismic velocity than the overlying layer and, therefore, does not give rise to a seismic refraction. This type of layer, therefore, cannot be recognized or modeled and depths to underlying layers would be overestimated. The presence of blind zones will cause errors in depth averaged seismic velocity or slowness.

A hidden layer is a layer with a velocity increase, but of sufficiently small thickness relative to the velocities of overlying and underlying layers, that refracted arrivals do not arrive at the geophones before those from the deeper, higher velocity layer. Because the seismic refraction method generally only involves the interpretation of first arrivals, a hidden layer cannot be

recognized or modeled and depths to underlying layers would be underestimated. However, it can be demonstrated that the presence of hidden layers does not cause significant errors in depth averaged seismic velocity or slowness, such as the average velocity of the upper 100 ft. A subsurface velocity structure that increases as a function of depth rather than as discrete layers, will also cause depths to subsurface refractors to be underestimated, in a manner very similar to that of the hidden layer problem.

Lateral velocity variability within a layer that is not characterized by the modeling scheme utilized for analysis will also result in depth errors to underlying layers. Additionally, at sites with steeply dipping or highly irregular bedrock surfaces, out-of-plane refractions (refractions from structures to the side of the line rather than from beneath the line) may severely complicate modeling. Tomographic inversion techniques can often resolve the complex velocity structures associated with hidden layers, velocity gradients and lateral velocity variations. However, in the event of an abrupt increase in velocity at a geologic horizon, the velocity model generated using tomographic inversion routines will smooth the horizon with velocity being underestimated at the interface and possibly overestimated at depth.

3 EQUIPMENT AND FIELD PROCEDURES

Seismic refraction equipment used during this investigation consisted of a Geometrics Geode 24-channel signal enhancement seismographs, 10 Hz vertical geophones, seismic cables with 10-foot spaced connectors, piezo hammer switches, and a 20-lb sledgehammer with an aluminum strike plate.

The locations of the seismic refraction profiles were established by Leighton Group Inc. Each seismic line consisted of 24 geophones spaced 10 feet apart for a line length of 230 feet. All geophone and shot point locations along each line were measured using a 300-foot tape measure. Elevations along each seismic refraction line were surveyed using a Trimble R10 GPS system with CenterPoint RTX real-time differential corrections.

Source locations included end shots at each end geophone, multiple off-end shot locations with a maximum offset of 100 ft, and interior shot locations at nominal 30 ft intervals for a total of 15 source locations per line. The 20-lb sledgehammer was used as the energy source for all source locations. A hammer switch mounted on the hammer was used to trigger the seismograph upon impact with the aluminum plate. The final seismic record at each shot point was the result of stacking 7 to 14 shots to increase the signal to noise ratio. All seismic records were stored on a laptop computer. Data acquisition parameters, file names, and other observations were recorded on a field log, which is retained in project files.

4 DATA REDUCTION AND MODELING

The first step in data processing consisted of picking the arrival time of the first energy (first arrival) received at each geophone for each shot point. The first arrivals on each seismic record are either a direct arrival from a compressional (P) wave traveling in the uppermost layer or a refracted arrival from a subsurface interface where there is a velocity increase. First-arrival times were selected using the manual picking routines in the SeisImager™ software suite (Geometrics, Inc.). These first-arrival times were saved in an ASCII file containing shot location, geophone locations, and associated first-arrival time. Errors in the first-arrival times were variable with error generally increasing with distance from the shot point. Elevations for each geophone location were calculated from the GPS data.

Seismic refraction data were then modeled using the tomographic analysis technique available in the SeisImager™ Plotrefa software package. Refraction tomography techniques are often able to resolve complex velocity structure (e.g., velocity gradients) that can be observed in bedrock weathering profiles. Layer-based modeling techniques such as the GRM are more applicable to characterize geologic structure that exhibits layering (e.g., low velocity sediments over high velocity rock). It should be noted, however, that tomographic modeling techniques will generate a velocity model with a gradual increase in velocity with depth even though an abrupt velocity increase may be present.

Tomographic analysis was conducted as outlined in the following steps. A smooth velocity gradient initial model was developed covering the expected velocity range in weathered rock at the site. The initial model had 18 layers with the top of the bottom layer at a depth related to the effective depth of investigation of the model. Velocity ranges were also set to values outside of the starting model minimum and maximum. The velocity models were extended to permit the use of off-end shot points during the inversion, as applicable. A minimum of 10 iterations of non-linear raypath inversion were then implemented to model the seismic data. The starting model parameters were then adjusted, as necessary, and the modeling process repeated until an acceptable fit between observed and calculated first arrival data was achieved.

The final tomographic velocity models for the seismic line were exported as ASCII files and imported into the Golden Software Surfer® mapping system where the velocity model was gridded, contoured, and annotated for presentation.

5 DISCUSSION OF RESULTS

The P-wave seismic refraction models for Lines 1 to 5 are presented as Figures 2, 3, 4, 5, and 6, respectively. A color scheme with blue-cyan, green-orange, and red-magenta representing low, intermediate, and high P-wave velocities, respectively, and velocity contours at 1,000 ft/s intervals are used to display the seismic velocity model. The depth of investigation is about 65 feet for Line 1 and 3, about 55 feet for Line 2, 60 feet for Line 4, and around 90 feet for Line 5.

Tomographic inversion techniques will typically model a gradual increase in seismic velocity with depth even if an abrupt velocity contact is present. Velocity gradients are, however, common in weathered rock geologic environments, such as the project site.

For discussion purposes, we assume that a Caterpillar D8R Ripper, or equivalent, will be used on site. Using higher performing rippers, such as the Caterpillar D9R Ripper, can increase the depth of rippable and marginally rippable zones. It may be more cost-effective to blast marginally rippable rock. Granitic rock with P-wave velocity of less than about 5,800 ft/s is considered rippable by a D8R Ripper. Granitic rock with P-wave velocity between about 5,800 and 8,000 ft/s is considered marginally rippable by a D8R. Granitic rock with P-wave velocity greater than 8,000 ft/s is considered non-rippable by a D8R. Metasedimentary rock with P-wave velocity between about 6,300 and 8,400 ft/s is considered marginally rippable by a D8R. Metasedimentary rock with P-wave velocity greater than 8,400 ft/s is considered non-rippable by a D8R.

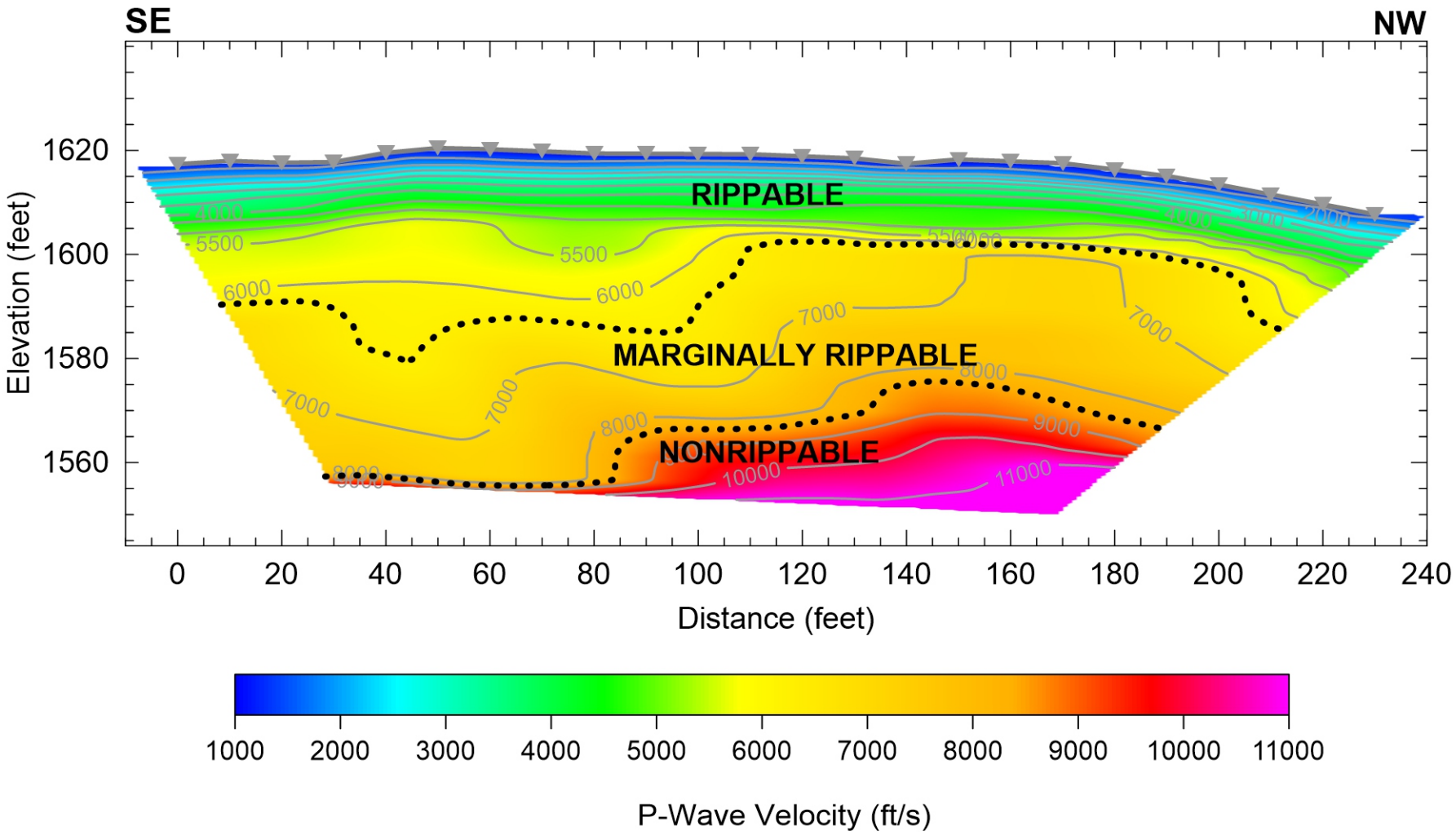
Line 1 (Figure 2) shows P-wave velocity increasing with depth from about 1,000 to 1,500 ft/s at the surface to over 11,000 ft/s at a depth of about 55 to 65 feet. The rock is expected to be rippable to a depth of between 15 and 40 ft and marginally rippable to a depth of about 40 to 65 ft. The metasedimentary rock is considered non-rippable at greater depths.

Beneath Line 2 (Figure 3), P-wave velocity increases with depth from about 1,000 to 2,000 ft/s at the surface to over 11,000 ft/s at a depth between about 30 and 40 ft. The rock is expected to be rippable to a depth of about 9 to 20 ft and marginally rippable to a depth of about 15 to 45 ft. The metasedimentary rock is considered non-rippable below a depth of 15 to 45 ft.

P-wave velocity increases with depth beneath Line 3 (Figure 4) from about 1,000 to 2,000 ft/s at the surface to over 11,000 ft/s at a depth between about 55 and 65 ft. The rock is expected to be rippable to a depth of about 20 to 35 ft and marginally rippable to a depth of about 40 to 50 ft.

Line 4 (Figure 5) shows P-wave velocity increasing with depth from about 1,000 to 2,000 ft/s at the surface to over 11,000 ft/s at a depth of about 60 ft. The rock is expected to be rippable to a depth of about 15 to 25 feet and marginally rippable to a depth of 35 to 50 feet. The granitic rock is expected to be non-rippable below a depth of 35 to 50 feet.

Line 5 (Figure 6) is modeled to a depth of about 90 ft with P-wave velocity increasing with depth from about 1,000 to 2,500 ft/s at the surface to over 11,000 at a depth of about 60 to 70 ft. The granitic rock is expected to be rippable to a depth of about 50 to 55 ft, marginally rippable to a depth of about 55 to 60 ft, and non-rippable at greater depths.



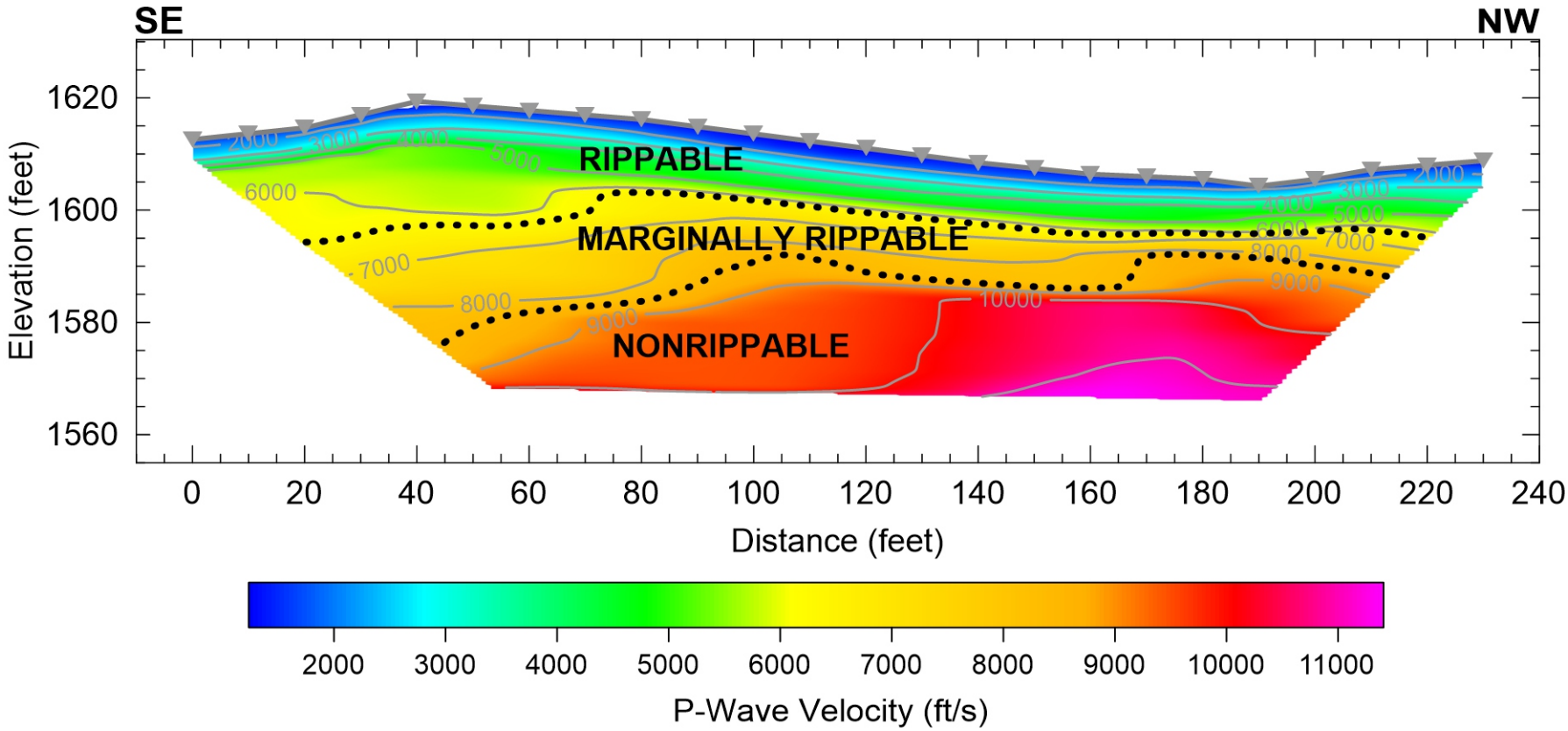
GEOVision geophysical services

Project No: 24019
Date: JAN 26, 2024
Drawn By: C. GARCIA
Approved By:
R:\GVProjects\2024\24008 ENGEO\Report\Figure 2.cdr

FIGURE 2
LINE 1 P-WAVE SEISMIC REFRACTION MODEL

CRESENT CEMETERY
RIVERSIDE COUNTY, CALIFORNIA

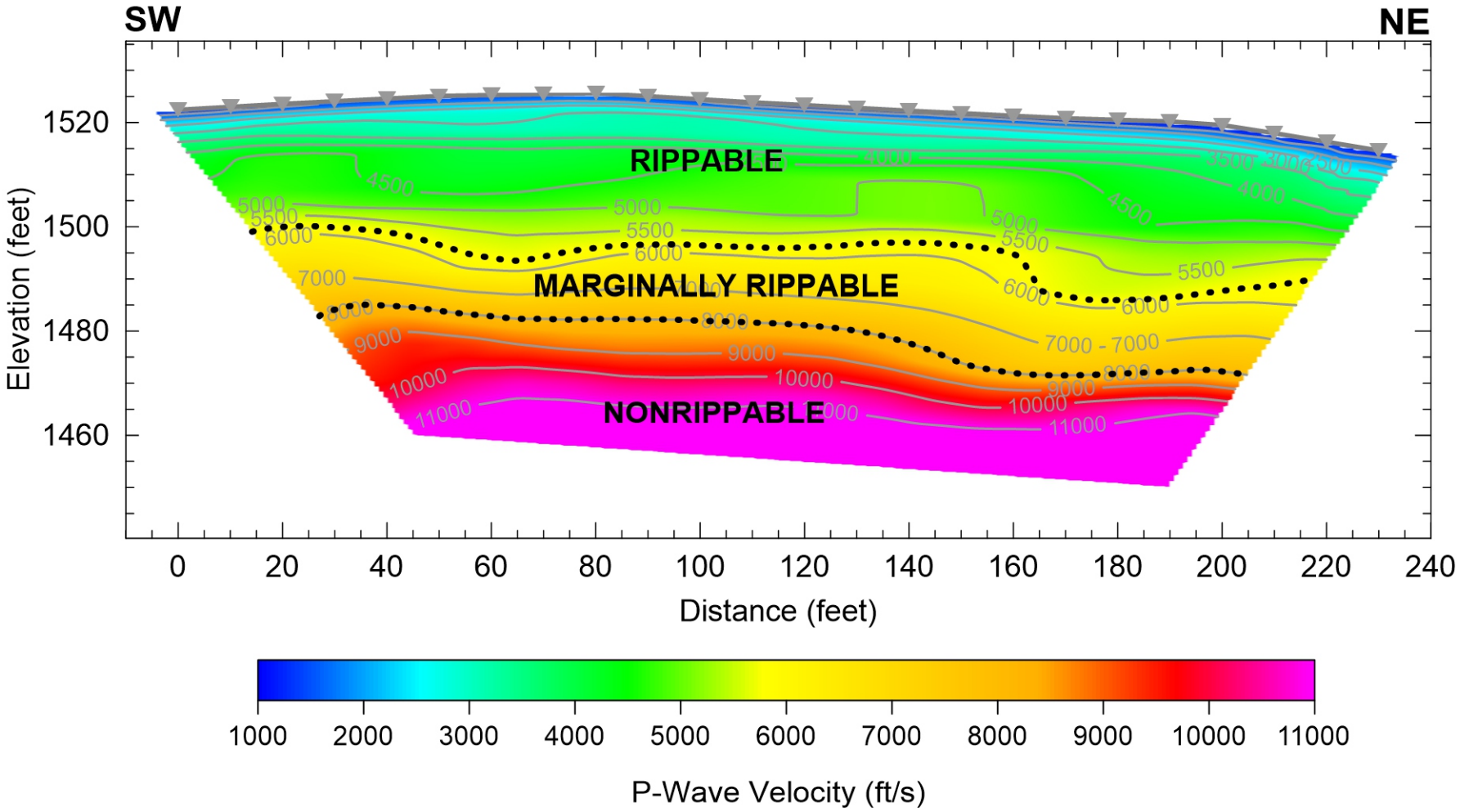
PREPARED FOR
LEIGHTON GROUP, INC.



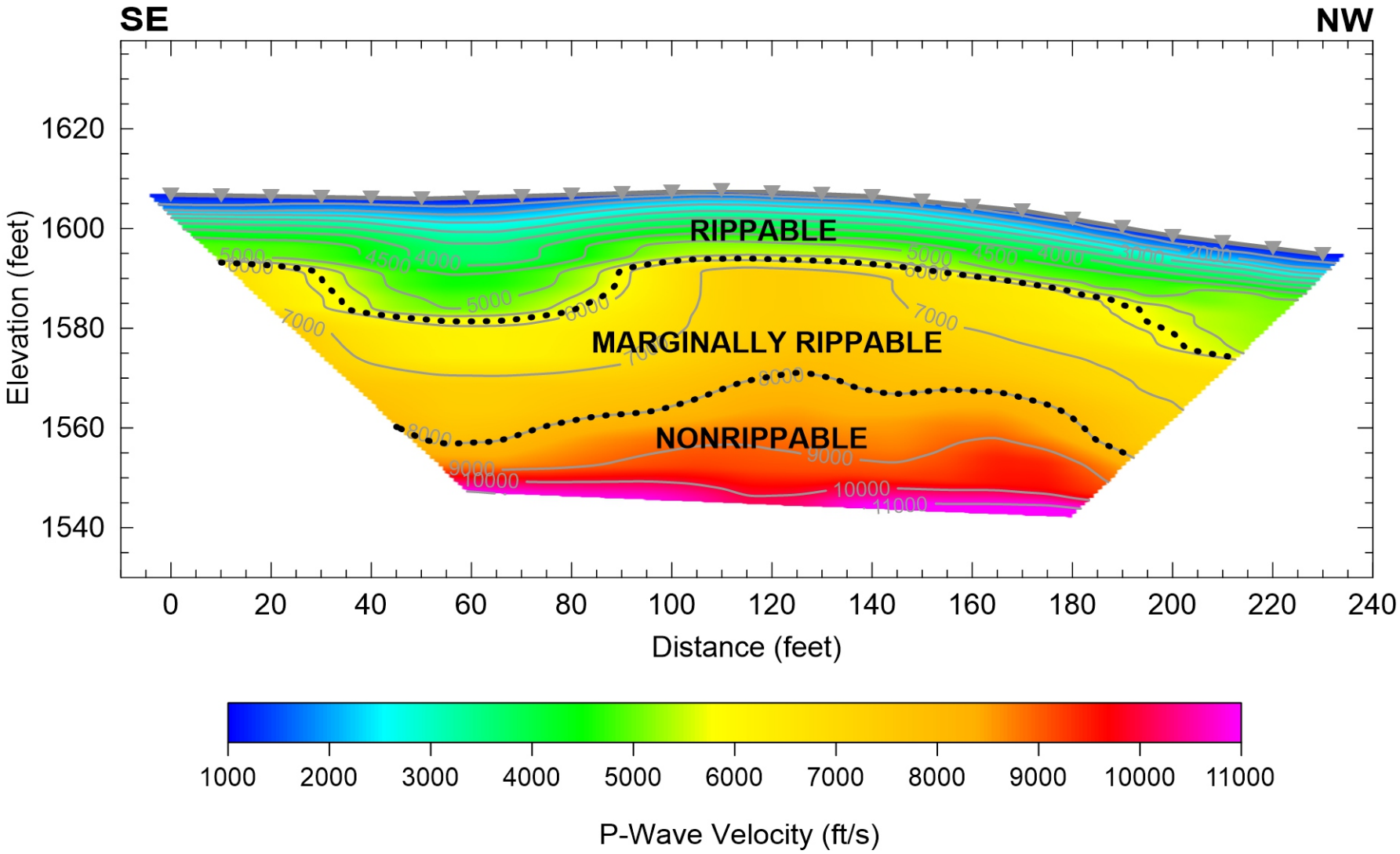
GEOVision geophysical services

Project No: 24019	FIGURE 3 LINE 2 P-WAVE SEISMIC REFRACTION MODEL
Date: JAN 25, 2024	CRESENT CEMETERY RIVERSIDE COUNTY, CALIFORNIA
Drawn By: C. GARCIA	PREPARED FOR LEIGHTON GROUP, INC.
Approved By:	

R:\GVProjects\2024\24008 ENGEO\Report\Figure 2.cdr



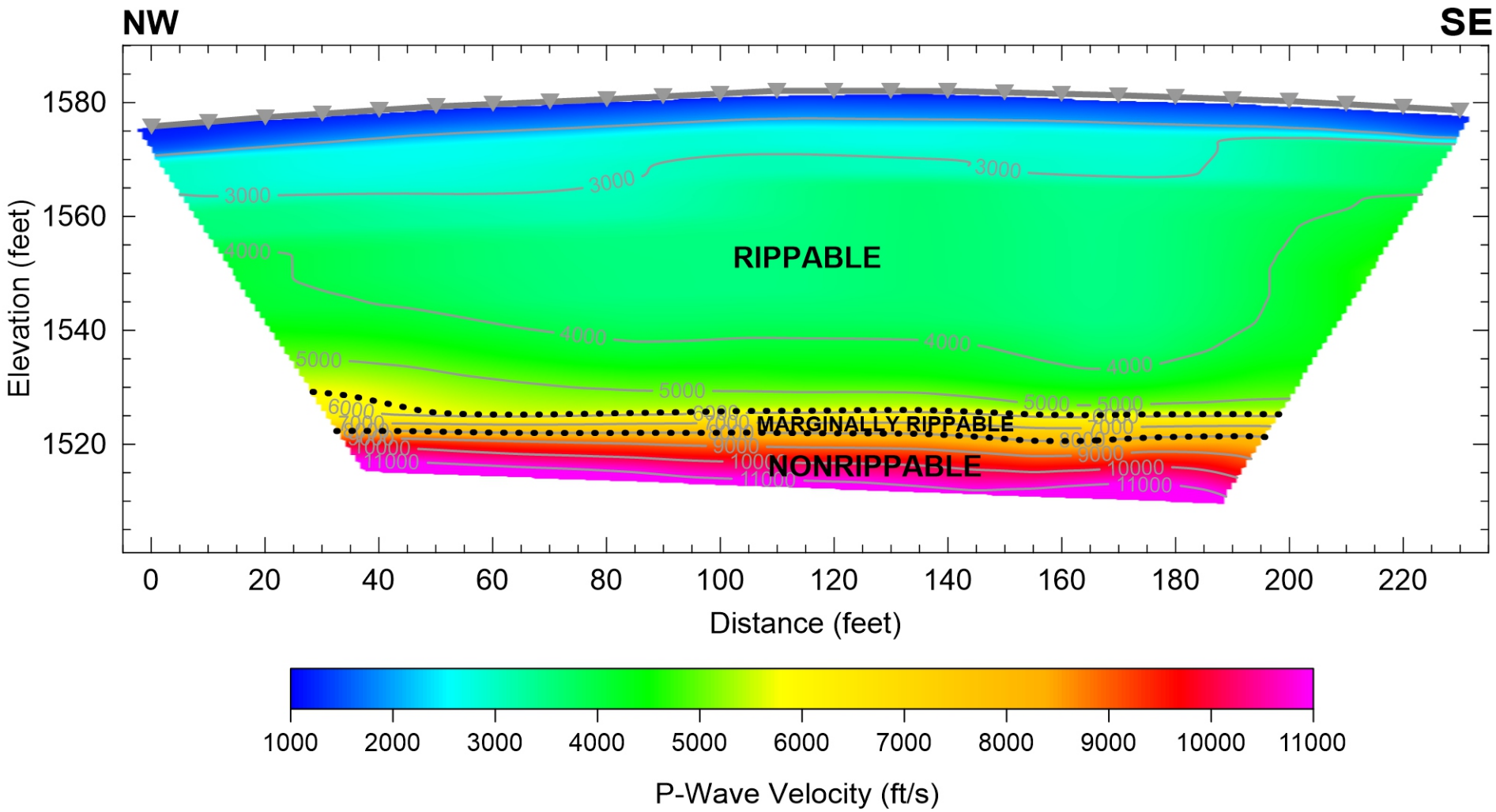
GEOVision geophysical services	FIGURE 4 LINE 3 P-WAVE SEISMIC REFRACTION MODEL
Project No: 24019 Date: JAN 25, 2024 Drawn By: E TRIPLETT Approved By:	CRESENT CEMETARY RIVERSIDE COUNTY, CALIFORNIA
R:\GVProjects\2024\24008 ENGEO\Report\Figure 2.cdr	PREPARED FOR LEIGHTON GROUP, INC.



GEOVision geophysical services

Project No: 24019	FIGURE 5 LINE 4 P-WAVE SEISMIC REFRACTION MODEL
Date: JAN 25, 2024	CRESENT CEMETARY RIVERSIDE COUNTY, CALIFORNIA
Drawn By: E TRIPLETT	PREPARED FOR LEIGHTON GROUP, INC.
Approved By:	

R:\GVProjects\2024\24008 ENGEO\Report\Figure 2.cdr



GEOVision geophysical services	FIGURE 6 LINE 5 P-WAVE SEISMIC REFRACTION MODEL
Project No: 24019 Date: JAN 26, 2024 Drawn By: C. GARCIA Approved By:	CRESENT CEMETERY RIVERSIDE COUNTY, CALIFORNIA
R:\GVProjects\2024\24008 ENGEO\Report\Figure 2.cdr	PREPARED FOR LEIGHTON GROUP, INC.

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7 CERTIFICATION

All geophysical data, analysis, interpretations, conclusions, and recommendations in this document have been prepared under the supervision of and reviewed by a **GEO***Vision* California Professional Geophysicist.

Prepared by,

2/2/2024

Eric Triplett — Date
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2/2/2024

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California Professional Geophysicist, P. Gp. 989
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* This geophysical investigation was conducted under the supervision of a California Professional Geophysicist using industry standard methods and equipment. A high degree of professionalism was maintained during all aspects of the project from the field investigation and data acquisition, through data processing, interpretation, and reporting. All original field data files, field notes, observations, and other pertinent information are maintained in the project files and are available for the client to review for a period of at least one year.

  A professional geophysicist's certification of interpreted geophysical conditions comprises a declaration of his/her professional judgment. It does not constitute a warranty or guarantee, expressed or implied, nor does it relieve any other party of its responsibility to abide by contract documents, applicable codes, standards, regulations, or ordinances

APPENDIX E
GENERAL EARTHWORK AND GRADING SPECIFICATIONS

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LEIGHTON CONSULTING, INC.

GENERAL EARTHWORK AND GRADING SPECIFICATIONS FOR ROUGH GRADING

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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.

APPENDIX F

GEO-PROFESSIONAL BUSINESS ASSOCIATION (GBA)

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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

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