



FIRE STATION 46
VALENCIA, CALIFORNIA

**GEOTECHNICAL EXPLORATION AND
ENGINEERING GEOLOGY REPORT**

SUBMITTED TO
Mr. David Wong
The Newhall Land and Farming Company
25124 Springfield Court, Suite 300
Valencia, CA 91355

PREPARED BY
ENGEO Incorporated

April 14, 2025
Revised May 14, 2025

PROJECT NO.
6538.100.312

Project No.
6538.100.312

April 14, 2025
Revised May 14, 2025

Mr. David Wong
The Newhall Land and Farming Company
25124 Springfield Court, Suite 300
Valencia, CA 91355

Subject: Fire Station 46
26720 Bombero Lane
Valencia, California

GEOTECHNICAL EXPLORATION AND ENGINEERING GEOLOGY REPORT

Dear Mr. Wong:

ENGEO prepared this geotechnical exploration and engineering geology report for The Newhall Land and Farming Company as outlined in our agreement dated March 26, 2025. We characterized the subsurface conditions at the site to provide our geotechnical recommendations for planning, design, and construction. The accompanying report contains the findings of our recent exploration, published exploration data, our conclusions and recommendations regarding the geotechnical aspects of the project.


The findings of our geotechnical exploration indicate that the site is suitable for the proposed project, provided that the recommendations contained in this report are incorporated into planning, design, and implemented during construction. The main geotechnical considerations for the project include presence of moderately expansive soil and seismic shaking.

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to review the project plans and specifications and provide geotechnical testing and observation services during construction. Please let us know when working drawings are nearing completion, and we will be glad to discuss these additional services with you.

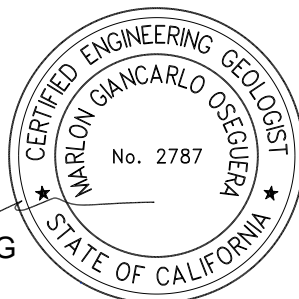
If you have any questions or comments regarding this report, please call and we will be glad to discuss them with you.

Sincerely,

ENGEO Incorporated


Marlon Oseguera, CEG

mo/tz/jjt/ca




Josef J. Tootle, GE



TABLE OF CONTENTS

LETTER OF TRANSMITTAL

1.0	INTRODUCTION	1
1.1	PURPOSE AND SCOPE	1
1.2	PROJECT LOCATION.....	1
1.3	PROJECT DESCRIPTION.....	2
2.0	FINDINGS	2
2.1	GEOLOGY AND SEISMICITY	2
2.1.1	Geology	2
2.1.2	Seismic Hazard Zone Report	3
2.1.3	Seismicity	3
2.2	FIELD EXPLORATION	3
2.3	SITE BACKGROUND	4
2.4	SURFACE CONDITIONS	4
2.5	SUBSURFACE CONDITIONS.....	4
2.6	GROUNDWATER CONDITIONS.....	5
2.7	LABORATORY TESTING	5
3.0	CONCLUSIONS	5
3.1	EXPANSIVE SOIL	5
3.2	SEISMIC HAZARDS.....	6
3.2.1	Ground Rupture	6
3.2.2	Ground Shaking	6
3.2.3	Liquefaction Analysis	7
3.2.4	Earthquake-Induced Landsliding	7
3.3	EXISTING FILL	7
3.4	SOIL CORROSION POTENTIAL	7
4.0	SITE-SPECIFIC SEISMIC-HAZARD ANALYSIS.....	8
4.1	SEISMIC SOURCE MODEL	9
4.2	GROUND-MOTION MODELS AND SITE PARAMETERS.....	9
4.3	PROBABILISTIC SEISMIC-HAZARD ANALYSIS.....	9
4.3.1	Methodology	9
4.3.2	Disaggregation of the Seismic Hazard.....	11
4.4	DETERMINISTIC SEISMIC-HAZARD ANALYSIS.....	12
4.5	RESULTING SURFACE RESPONSE SPECTRA.....	13
5.0	CONSTRUCTION MONITORING	15
6.0	EARTHWORK RECOMMENDATIONS.....	16
6.1	GENERAL SITE CLEARING.....	16
6.2	REMOVALS	16
6.3	BENCHING	16
6.4	OVER-OPTIMUM SOIL MOISTURE CONDITIONS	16
6.5	ACCEPTABLE FILL.....	17
6.6	FILL COMPACTION	17
6.6.1	Grading in Structural Areas	17
6.6.2	Underground Utility Backfill	17

TABLE OF CONTENTS (Continued)

6.6.3	Retaining Wall Backfill.....	18
6.6.4	Landscape Fill.....	18
6.7	SLOPES.....	18
6.8	SITE DRAINAGE.....	18
6.9	STORMWATER INFILTRATION.....	18
6.10	STORMWATER BIORETENTION AREAS.....	18
7.0	FOUNDATION RECOMMENDATIONS	19
7.1	CONVENTIONAL FOOTINGS WITH SLAB-ON-GRADE	20
7.1.1	Footing Dimensions and Allowable Bearing Capacity	20
7.1.2	Reinforcement	20
7.1.3	Foundation Lateral Resistance	20
7.1.4	Settlement	21
7.2	ANCILLARY STRUCTURES.....	21
8.0	SLABS-ON-GRADE.....	21
8.1	INTERIOR CONCRETE FLOOR SLABS	21
8.1.1	Minimum Design Section.....	21
8.1.2	Slab Moisture Vapor Reduction	21
8.1.3	Subgrade Moisture Conditioning for Structural Slab.....	22
8.1.4	Subgrade Modulus for Structural Slab Design	22
8.2	EXTERIOR FLATWORK.....	22
8.3	TRENCH BACKFILL.....	22
9.0	RETAINING WALLS	23
9.1	LATERAL SOIL PRESSURES.....	23
9.2	FOUNDATIONS	23
9.3	RETAINING WALL DRAINAGE	24
10.0	PAVEMENT DESIGN	24
10.1	SUBGRADE AND AGGREGATE BASE COMPACTION	25
10.2	CUTOFF CURBS	25
11.0	LIMITATIONS AND UNIFORMITY OF CONDITIONS	25
SELECTED REFERENCES		
FIGURES		
APPENDIX A – Exploration Logs		
APPENDIX B – Laboratory Test Data		
APPENDIX C – GEOVision Report		
APPENDIX D – Disaggregation Results		

1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

ENGEO prepared this geotechnical report for design of the new Fire Station 46 in Valencia, California. As outlined in our agreement dated March 26, 2025, you authorized ENGEO to conduct the following scope of services.

- Subsurface field exploration consisting of geotechnical borings
- Soil laboratory testing and data analysis
- Geotechnical report preparation summarizing site conditions, conclusions, and recommendations, written in accordance with California Geological Survey (CGS) – Note 48

For our use, we received the following.

- The Newhall Land and Farming Company. 2025. Draft Fact Sheet, Los Angeles County Fire Department, Fire Station 46, 26720 Bombero Lane, Valencia, CA 91381. March 17, 2025.
- Saiful Bouquet Structural Engineers. Undated. Minimum Requirements for Geotechnical Report, CBC 2022 – Fire Station #46. Received March 11, 2025.
- Saiful Bouquet Structural Engineers. 2025. General Soils Questionnaire Topics. March 10, 2025.
- William Loyd Jones Architect. 2025. Revised Schematic Site Plan, Fire Station 46, County of Los Angeles Fire Department, 26720 Bombero Lane, Valencia, California. March 3, 2025.
- William Loyd Jones Architect. 2025. Revised Schematic Elevations, Fire Station 46, County of Los Angeles Fire Department, 26720 Bombero Lane, Valencia, California. March 3, 2025.
- William Loyd Jones Architect. 2025. Revised Schematic Floor Plan, Fire Station 46, County of Los Angeles Fire Department, 26720 Bombero Lane, Valencia, California. March 3, 2025.

This report was prepared for the exclusive use of our client and their consultants for design of this project. In the event that any changes are made in the character, design, or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to evaluate whether modifications are recommended. This document may not be reproduced in whole or in part by any means whatsoever, nor may it be quoted or excerpted without our express written consent.

1.2 PROJECT LOCATION

Figure 1 displays a site Vicinity Map. The site is located on the southern side of Bombero Lane, east of the intersection of Bombero Lane and Westridge Parkway. Access to the site is provided through Bombero Lane. The latitude and longitude coordinates for the approximate center of the site are 34.4143, -118.6020.

Figure 2 shows proposed building and pavement areas, our exploratory locations and those by others. The project site is bound to the south, east, and north by future residential development. The site is bound by Westridge Parkway to the west.

1.3 PROJECT DESCRIPTION

Based on our discussion with you and review of the information provided, we understand that the following site improvements are proposed.

1. Minor earthwork cut and fill to achieve precise pad grades. Finished grades were not made available to us at the time of writing.
2. A new one-story building with concrete slab-on-grade floors. The proposed building is roughly 130 feet wide and 155 feet long. Structural loads have yet to be determined, but are anticipated to be typical of this type of structure.
3. A standalone reserve apparatus building. The proposed building is roughly 57 feet wide and 41 feet long. Structural loads have yet to be determined, but are anticipated to be typical of this type of structure.
4. A diesel and gasoline refueling station.
5. An emergency diesel generator.
6. Paved parking and drive lanes.
7. Site and/or retaining walls up to approximately 8 feet in height.
8. Utilities and other infrastructure improvements.
9. Concrete flatwork.
10. Landscaping.

Our report will include recommendations for the noted improvements above.

2.0 FINDINGS

2.1 GEOLOGY AND SEISMICITY

2.1.1 Geology

The site is located within the Transverse Ranges Geomorphic Province of California. The Transverse Ranges are characterized by a complex series of east-west-trending mountain ranges, valleys, and quaternary faults extending from the Santa Ynez Mountains and Channel Islands eastward through the San Bernardino Mountains. The site is in the eastern end of the Ventura Basin of Southern California and the northwestern corner of Santa Clarita Valley. The Ventura Basin is a westerly plunging depositional basin produced by tectonic downwarping initiated during the early Miocene epoch, with its axis approximately coinciding with the Santa Clara River.

Regional mapping by Dibblee and Ehrenspeck (1996; Figure 3) identifies the site to be underlain by Saugus Formation (QTs).

2.1.2 Seismic Hazard Zone Report

The Seismic Hazard Zone Report for the Newhall 7.5-Minute Quadrangle (California Division of Mines and Geology, 1997, Figure 5) includes the project site. This map indicates portions of the site lie within a Zone of Required Investigation. Specifically, a zone at the northern portion of the site is mapped as potentially susceptible to seismically induced landslides. Section 3.2.4 summarizes the rough grading that was completed to mitigate this hazard.

The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone, and no known surface expression of active faults is believed to exist within the site. No known blind-thrust faults underlie the site. The closest blind-thrust fault to the site is the Northridge Blind-Thrust fault.

2.1.3 Seismicity

The site is located in a seismically active area that contains numerous faults. Small earthquakes occur every year in the northern Los Angeles County region and larger earthquakes have been recorded and can be expected to occur in the future. Faults have been cataloged and mapped by the United States Geological Survey (USGS) in the Quaternary Fault and Fold Database of the United States. An active fault is defined by the California Geologic Survey as one that experienced surface displacement within Holocene time (about the last 11,700 years) (CGS, 2018). Figure 4 shows the approximate locations of known active faults along with other Quaternary faults based on the USGS Quaternary Fault and Fold Database, as well as significant historical earthquakes recorded within the northern Los Angeles County region. The site is not located within a currently designated Alquist-Priolo Earthquake Fault Zone, and no known surface expression of active faults is believed to exist within the site. No known blind-thrust faults underlie the site. The closest blind-thrust fault to the site is the Northridge Blind-Thrust fault.

In 2014, the Working Group on California Earthquake Probabilities estimated the 30-year likelihood of one or more moment magnitude (M_w) 6.7 or greater earthquake events in the Los Angeles County region at approximately 60 percent, considering the known seismic sources in the region. We provide a table identifying nearby active faults that are capable of generating strong seismic ground shaking at the site in Section 4.3.2 of this report.

2.2 FIELD EXPLORATION

Our field exploration included drilling three borings at various locations on the site. We performed our field exploration on March 31, 2025. The location and elevations of our explorations are approximate and were estimated by pacing from features shown in the Site Plan, Figure 2. They should be considered accurate only to the degree implied by the method used.

An ENGEO representative observed the drilling and logged the subsurface conditions at each location. We retained a rubber track-mounted Landa L10T drill rig and crew to advance the borings using an 8-inch-diameter hollow-stem auger. The borings were advanced to depths ranging from 31 to 50½ feet below existing grade. We permitted and backfilled the borings in accordance with the requirements of the County of Los Angeles.

We obtained bulk soil samples from drill cuttings and retrieved both disturbed and relatively undisturbed soil samples at various intervals in the borings using standard penetration tests (SPTs) and a 3-inch outside diameter (O.D.) split-spoon sampler fitted with 1-inch-long brass rings. The blow counts were obtained by dropping a 140-pound Auto-trip hammer through a 30-inch free fall. The standard penetrometer or 3-inch O.D. split-spoon sampler was driven

18 inches and the number of blows were recorded for each 6 inches of penetration. In addition, 2.5-inch inside diameter (I.D.) samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows to drive the last 1 foot of penetration; the blow counts have not been converted using any correction factors. When sampler driving was difficult, penetration was recorded only as inches penetrated for 50 hammer blows.

The logs depict subsurface conditions at the exploration locations during the exploration; however, subsurface conditions may vary with time. The exploration logs are included in Appendix A.

2.3 SITE BACKGROUND

Leighton and Associates (Leighton) performed a geotechnical review of the 40-scale bulk grading plan for Tract 61105 in 2012 (Leighton, 2012). Geotechnical review of the vesting tentative tract map was previously performed in 2004, 2009, and 2010 by Allan E. Seward Engineering Geology, Inc. (AES), R.T. Frankian & Associates (RTFA), and Leighton (see Selected References). These reports have been approved by the County of Los Angeles Geotechnical and Materials Engineering Division (GMED). We considered the previously collected data in developing the conclusions and recommendations included in this report. Published pertinent geotechnical logs of exploratory locations and laboratory test results from previous geotechnical studies were included in the approved bulk grading plan review report (Leighton, 2012).

Construction quality control testing and observation services were performed by Leighton, the Geotechnical Engineer-of-Record during mass grading, to confirm conformance with the design geotechnical recommendations in the previously referenced report (Leighton, 2012).

2.4 SURFACE CONDITIONS

Based on the testing and observation report (Leighton, 2023), Mission Village Phase 6 (of which the subject site is a part) was graded to gently slope from south to north. Rough grading operations within the footprint of the site consisted of civil cut into native Saugus Formation bedrock. The cut depth was approximately 60 feet deep to achieve rough finish grade. The approximate finished rough grade elevation at the subject site ranges from Elevation 1,268 feet to 1,266 feet (Leighton, 2023), sloping from south to north. There is a temporary detention basin on the northwestern corner of the site, with an approximate bottom of Elevation 1,262 feet.

During our site exploration, we observed the graded site to be generally covered in short grasses.

2.5 SUBSURFACE CONDITIONS

The site has been developed through cut and fill grading operations with all pad subgrades consisting of varying thicknesses of engineered fill. Based on the details of the rough grading operations within the site (Leighton, 2023), the site has fill thicknesses up to approximately 8 to 10 feet. Building pad zones were overexcavated, processed, and recompacted to a depth of 5 feet below finish grade. Clay beds and potentially expansive deposits were overexcavated, processed, and recompacted to a depth of 8 to 10 feet below finish grade.

Our subsurface explorations at the site encountered a generalized soil profile of silty/clayey sand or lean clay engineered fill over sandstone/siltstone/claystone bedrock of the Saugus Formation (QTs). The engineered fill generally consisted of medium dense to very dense silty/clayey fine- to medium-grained sand and stiff to hard lean clay with sand and sporadic fine gravel. The engineered fill we encountered in our explorations ranged in depth, beginning at the surface and extending to approximately 8 to 9 feet deep below ground surface (bgs). Native Saugus Formation bedrock below the engineered fill generally consisted of moderately to intensely weathered, interbedded sandstone, siltstone, and claystone. The Saugus Formation bedrock was encountered below the engineered fill to terminal depth of our borings.

Consult the Site Plan and exploration logs for specific subsurface conditions at each exploration location. We include our exploration logs in Appendix A. The logs contain the soil or rock type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System. The logs graphically depict the subsurface conditions encountered at the time of the exploration.

2.6 GROUNDWATER CONDITIONS

To the northwest of the project site along the Santa Clara River, historical high groundwater is at an approximately Elevation 970 feet, 10 feet bgs (California Statewide Groundwater Elevation Monitoring System, 2022). Groundwater was not encountered in any of our borings, nor within subsurface explorations at the site or surrounding areas by others. Groundwater was not encountered during grading activities within the site (Leighton, 2023). Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation practice, and other factors not evident at the time measurements were made.

2.7 LABORATORY TESTING

We performed laboratory tests on selected soil samples to evaluate their engineering properties. For this project, we performed particle size distribution, in situ moisture content and density, plasticity index (PI), expansion index (EI), unconfined compression, resistance value (R-value), and limited corrosion testing. Moisture contents, dry densities, Atterberg limits, and select particle size distribution results are recorded in the boring logs in Appendix A; other laboratory data is included in Appendix B.

3.0 CONCLUSIONS

From a geotechnical engineering viewpoint, in our opinion, the site is suitable for the proposed development, provided the geotechnical recommendations in this report are properly incorporated into the design plans and specifications.

The primary site-specific geotechnical conditions that could affect the proposed development are seismic shaking and moderately expansive soil. We summarize our conclusions below.

3.1 EXPANSIVE SOIL

Laboratory testing performed on two samples from our borings indicated PI values ranging between 13 and 16, which correspond to a low to moderate shrink/swell potential with variations in moisture content. The EI values for two soil samples tested ranged from 12 to 39, indicating low expansion potential with variations in moisture content.

Moderately expansive soil changes in volume with changes in moisture. It can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Successful performance of structures on expansive soil requires special attention during construction. It is imperative that exposed soil be kept moist prior to placement of concrete for foundation construction. It can be difficult to remoisturize clayey soil without excavation, moisture conditioning, and recompaction.

We have provided specific recommendations for compaction of the moderately expansive soil at the site. The purpose of these recommendations is to reduce the swell potential by compacting the soil at an increased moisture content and controlling the amount of compaction. Our recommendations for the building foundation and site improvement also consider the differential shrink/swell potential of the site soil.

3.2 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. Common secondary seismic hazards include ground shaking, soil liquefaction, and seismically induced landsliding. The following sections present a discussion of these hazards as they apply to the site. Based on previous site grading, topographic conditions, and lithologic data, the risk of regional subsidence or uplift, tsunamis, flooding or seiches is considered low to negligible at the site.

3.2.1 Ground Rupture

Since there are no known active faults crossing the property and the site is not located within an Earthquake Fault Special Study Zone, it is our opinion that ground rupture is unlikely at the subject property.

3.2.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the northern Los Angeles County region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, structures should be designed using sound engineering judgment and the 2022 California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead and live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage, but with some non-structural damage, and (3) resist major earthquakes without collapse but with some structural, as well as non-structural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1999).

3.2.3 Liquefaction Analysis

Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soil most susceptible to liquefaction is clean, loose, saturated, uniformly graded fine sand below the groundwater table. When seismic ground shaking occurs, the soil is subjected to cyclic shear stresses that can cause excess hydrostatic pressures to develop. If excess hydrostatic pressures exceed the effective confining stress from the overlying soil, the sand may undergo deformation. If the sand undergoes virtually unlimited deformation without developing significant resistance, it is said to have liquefied and ground settlement and surface deformation may occur. As described in Section 2.6, groundwater was not encountered in our exploration; furthermore, according to Leighton's report (2023), groundwater was not encountered within the site during rough grading operations. For these reasons and based upon engineering judgment, it is our opinion that the potential for liquefaction at the site is low during seismic shaking.

3.2.4 Earthquake-Induced Landsliding

As noted in Section 2.1.2, a zone within the site is mapped as areas potentially susceptible to earthquake-induced landslides (Figure 5). The As-Built Compaction report (Leighton, 2023) indicates that previously mapped landslide material within the site, as well as additional material encountered during grading, was overexcavated, processed, and recompacted as engineered fill during corrective grading operations. Furthermore, as noted in Section 2.5, the site is underlain by a minimum of 8 feet of engineered fill. For these reasons and based upon engineering judgement, it is our opinion that the potential for earthquake-induced landsliding at the site is low during seismic shaking.

3.3 EXISTING FILL

Based on our review of the As-Built Compaction report (Leighton, 2023), suitable on-site soil was spread in horizontal lifts, moisture conditioned, and compacted as engineered fill. The pad subgrades within the site consist of varying thicknesses of engineered fill, as noted in Section 2.5. In our opinion, the engineered fill material that was documented and tested is acceptable for support of future structures and improvements associated with the subject project.

3.4 SOIL CORROSION POTENTIAL

As part of this study, we obtained a representative soil sample and submitted it to a qualified analytical lab for determination of redox potential, pH, resistivity, chloride, and sulfate. The results are summarized in the table below. The results and a brief corrosion evaluation are also included in Appendix B.

TABLE 3.4-1: Corrosivity Test Results

SAMPLE LOCATION	DEPTH (feet)	REDOX (mV)	pH	RESISTIVITY (OHMS-CM)	CHLORIDE (MG/KG)	SULFATE (MG/KG)
1-B02	3.5-4	240	8.41	2,800	42	22

According to CERCO Analytical, based upon the resistivity measurements, the soil is considered "moderately corrosive" to buried ferrous metals. All buried iron, steel, cast iron, ductile iron, galvanized steel, and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping, such as ductile iron firewater pipelines, should be protected against corrosion.

The 2022 CBC references the 2019 American Concrete Institute Manual, ACI 318-19, Section 19.3.1 for concrete durability requirements. In accordance with ACI Table 19.3.1.1, this soil is categorized as within the S0 sulfate exposure class. Considering the S0 'Not Applicable' sulfate exposure, there is no requirement for cement type or water-cement ratio; however, a minimum concrete compressive strength of 2,500 pounds per square inch (psi) is specified by the building code. It should be noted, however, that the structural engineering design requirements for concrete may result in more stringent concrete specifications.

Refer to the report included in Appendix B from the analytical lab for further information. If desired to investigate this further, we recommend a corrosion consultant be retained to evaluate if specific corrosion recommendations are advised for the project. Note that ASTM Test Method D4327 was used in lieu of the ACI designated sulfate test methods as it provides more repeatable test results.

4.0 SITE-SPECIFIC SEISMIC-HAZARD ANALYSIS

We performed a site-specific seismic-hazard analysis (SHA) in accordance with the 2022 CBC, which incorporates, by reference, the seismic design criteria described in the 2016 version of the American Society of Civil Engineers document titled "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," (ASCE 7-16).

We classified the site as Site Class C in accordance with Chapter 20 of ASCE 7-16 based on nearby shear-wave velocity testing performed by GEOVision during previous phases of the Newhall Ranch development project (refer to Appendix C), as well as the subsurface conditions encountered in our subsurface explorations. We completed the following tasks to develop the risk-targeted, maximum-rotated maximum considered earthquake (MCE_R) and design earthquake (DE) response spectra and their associated seismic design parameters for this site.

- Performed probabilistic seismic-hazard analysis (PSHA) to develop a risk-targeted, maximum-rotated response spectrum corresponding to a 2-percent probability of exceedance and 1-percent probability of collapse in 50 years (2,475-year return period)
- Performed deterministic seismic-hazard analysis (DSHA) to develop an 84th-percentile maximum-rotated response spectrum
- Compared the DSHA response spectrum with the deterministic lower limit in accordance with Section 21.2.2 of ASCE 7-16, as updated in Supplement No. 1
- Compared the risk-targeted and maximum-rotated probabilistic and the maximum-rotated deterministic response spectra to obtain the MCE_R response spectrum for the site
- Compared the MCE_R response spectrum developed in the previous step with 80 percent of the general response spectrum (i.e., the code minimum) to develop the recommended site-specific MCE_R response spectrum
- Multiplied the site-specific MCE_R response spectrum by two-thirds to obtain the site-specific DE spectrum
- Developed seismic design parameters per Sections 21.4 and 21.5 of ASCE 7-16

4.1 SEISMIC SOURCE MODEL

We utilized the Third California Earthquake Rupture Forecast model or UCERF3 (Field et al. 2014 and 2015) as implemented in the OpenSHA software (<https://github.com/opensha/>). The implementation of the UCERF3 model in seismic-hazard codes considers many sources of epistemic uncertainty regarding alternate rupture scenarios, maximum magnitudes for individual faults, and alternate magnitude-recurrence relations. This uncertainty affects the mean hazard that is provided by hazard codes implementing UCERF3 and is used in typical applications, including this study.

4.2 GROUND-MOTION MODELS AND SITE PARAMETERS

We used four semi-empirical ground-motion models (GMMs) from the Next Generation Attenuation West 2 (NGA West 2) project (Ancheta et al., 2014) to estimate the ground motion for shallow crustal sources included within the UCERF3 model. These models include Abrahamson et al. (2014) [ASK], Boore et al. (2014) [BSSA], Campbell and Bozorgnia (2014) [CB], and Chiou and Youngs (2014) [CY]. We performed our analysis using all four GMMs for a spectral damping of 5 percent of critical damping. We used the logic-tree approach and assigned equal weight (0.25) to the four GMMs in our analysis.

The ground-motion models incorporate “site parameters” to estimate how subsurface soil will amplify or attenuate ground motions as they propagate from underlying bedrock. These site parameters include:

- Time-averaged shear-wave velocity (V_s) over the top 100 feet or 30 meters (V_{s30}).
- Depth at which V_s reaches 3,280 feet/sec or 1.0 kilometer/sec ($z_{1.0}$).
- Depth at which V_s reaches 8,200 feet/sec or 2.5 kilometers/sec ($z_{2.5}$).

We estimated a representative V_{s30} value for the site based on the V_s profiles measured in proximity to the site (refer to Appendix C) and with consideration of fill thickness and depth to bedrock at the site. Based on the subsurface data, the fill thickness generally ranges from approximately 8 to 9 feet across the site. Accordingly, we considered a representative V_{s30} value of 1,772 feet per second (ft/sec) (540 meters per second (m/s)). We used the Statewide California Earthquake Center Community Velocity Model version 4, as implemented in the USGS Site Data Application Software (OpenSHA), to estimate $z_{1.0}$ and $z_{2.5}$ values of approximately 1,380 feet (0.42 kilometers (km)) and 12,010 feet (3.66 km), respectively. We applied these values in our analysis.

4.3 PROBABILISTIC SEISMIC-HAZARD ANALYSIS

4.3.1 Methodology

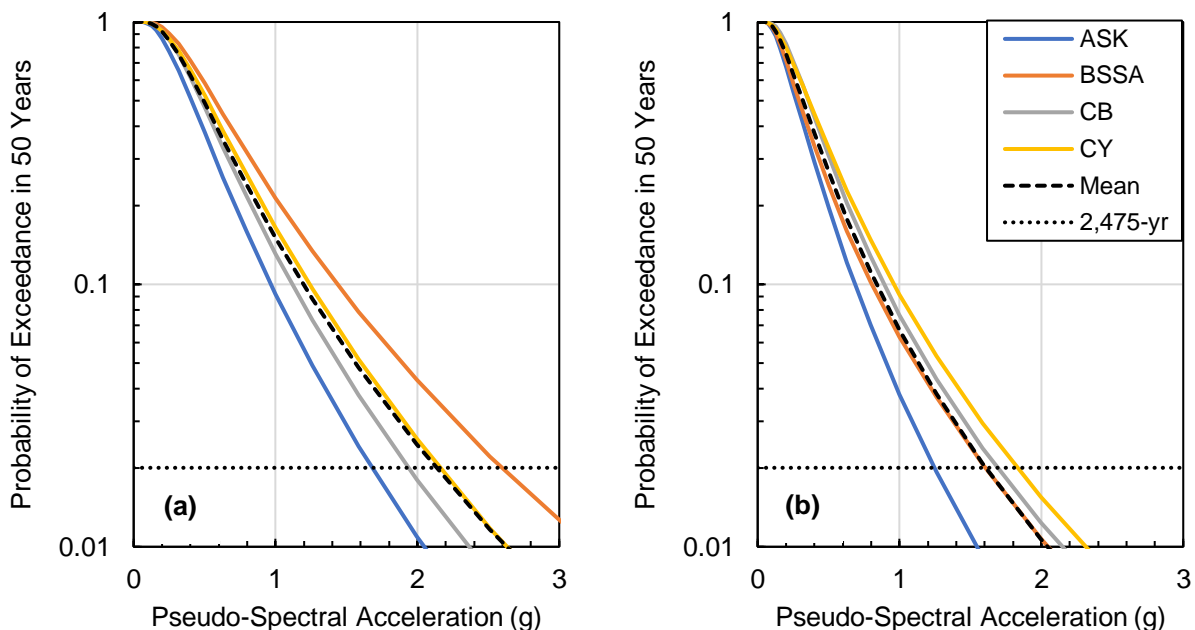
We performed a PSHA to develop a set of hazard curves and a resulting uniform hazard response spectrum (UHS) for a return period of 2,475 years. We calculated the seismic hazard using the standard methodology for hazard analysis (McGuire, 2004). The seismic-hazard calculations can be represented by the following equation, which is an application of the total-probability theorem.

$$H(a) = \sum_i^n v_i \iint P[A > a|m, r] f_{Mi}(m) f_{Ri|Mi}(r, m) dr dm$$

In this equation, the hazard $H(a)$ is the annual frequency of earthquakes that produce a ground motion amplitude A higher than a . Amplitude A may represent peak ground acceleration, velocity or pseudo-spectral acceleration (PSa) at a given frequency. The summation in the equation shown extends over all sources (i.e., over all faults and areas). In the above equation, v_i is the annual rate of earthquakes (with magnitude higher than some threshold M_i) in source i , and $f_{M_i}(m)$ and $f_{R_i|M_i}(r, m)$ are the probability density functions for magnitude and distance, respectively. $P[A > a|m, r]$ is the probability that an earthquake of magnitude m at distance r produces a ground-motion amplitude A at the site that is greater than a . Seismic sources may be either faults or background sources; the specification of source geometries and the calculation of $f_{R_i|M_i}$ are performed differently for these two types of sources.

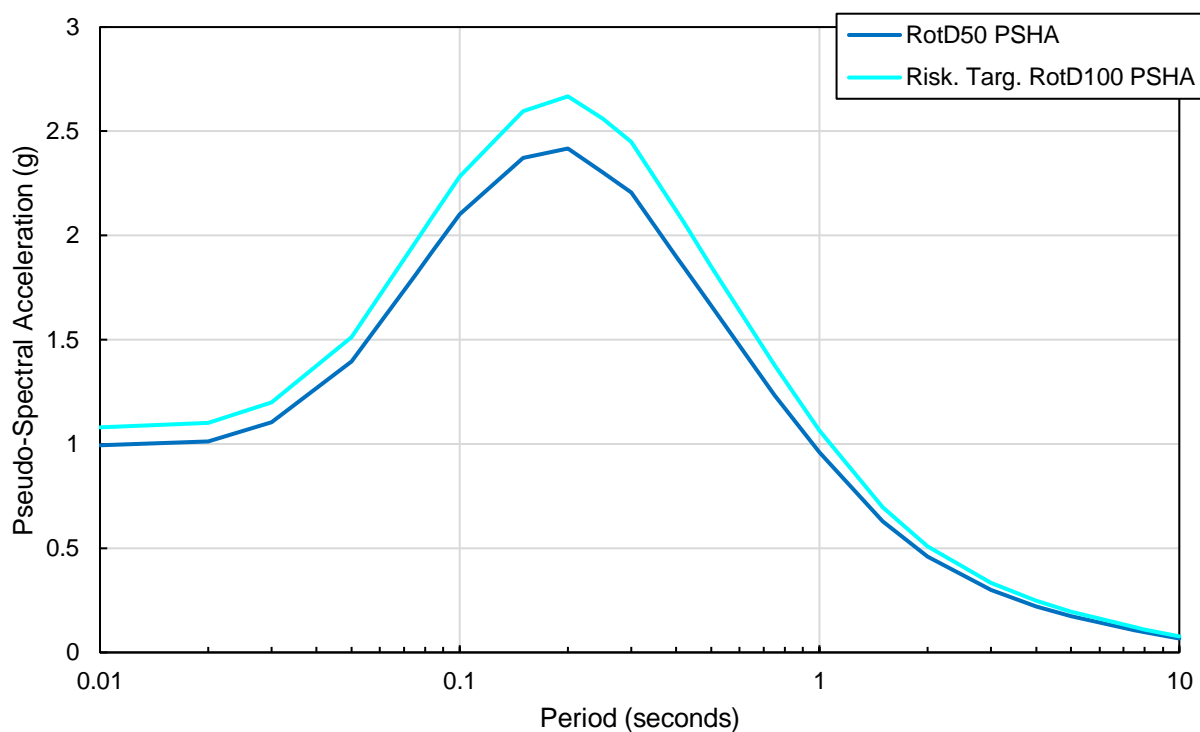
We present hazard curves at oscillator periods of 0.1 and 0.5 seconds in Exhibit 4.3.1-1. At each oscillator period, we present the hazard curves for each of the four NGA West 2 GMMs along with the weighted mean. We also show a horizontal line that represents the 2,475-year return period.

EXHIBIT 4.3.1-1: Hazard Curves at (a) 0.1 seconds and (b) 0.5 seconds



We present the median component (RotD50) 2,475-year UHS in Exhibit 4.3.1-2. This UHS was obtained by calculating the PSa value associated with a 2,475-year return period from the mean hazard curve at each oscillator period. We applied the mapped risk factors (C_{RS} of 0.912 and C_{R1} of 0.895) to the UHS in accordance with Section 21.2.1 (Method 1) of ASCE 7-16. Finally, we applied the maximum rotation factors discussed in Shahi and Baker (2014) to develop the maximum-rotated (RotD100) probabilistic response spectrum. We show the final risk-targeted, maximum-rotated probabilistic response spectrum in Exhibit 4.3.1-2.

EXHIBIT 4.3.1-2: 2,475-year Uniform Hazard Response Spectra



4.3.2 Disaggregation of the Seismic Hazard

We performed a disaggregation of the seismic hazard associated with the 2,475-year return period for oscillator periods of 0.1, 0.2, and 0.5 seconds. We anticipate that this range encompasses the period range of interest for the proposed structure. We present the resulting faults and their respective contributions to the seismic hazard at the site for each return period considered in Table 4.3.2-1. We include plots of the disaggregation results in Appendix D.

TABLE 4.3.2-1: Summary of Disaggregation Results for a 2,475-Year Return Period

FAULT SOURCE	R _{RRUP}		M _w	CONTRIBUTION (%)		
	(km)	(miles)		0.1 sec	0.2 sec	0.5 sec
Santa Susana alt 1 [1]	7.9	4.9	7.37	27.2	29.4	35.2
Holser alt 1 [1]	2.2	1.4	7.54	8.9	10.1	11.9
Del Valle [1]	6.2	3.8	7.40	7.4	8.3	9.8
Northridge [1]	9.2	5.7	7.44	5.4	6.2	7.6
Northridge Hills [1]	7.0	4.3	7.57	5.5	6.1	7.2
San Gabriel [3]	5.5	3.4	7.50	2.6	3.0	3.7
San Andreas (Mojave S) [0]	32.3	20.1	8.07	< 1.0	2.3	3.8
Mission Hills 2011 [1]	13.6	8.5	6.55	1.2	1.1	< 1.0
Santa Susana alt 2 [3]	8.1	5.0	6.69	1.4	1.4	1.4

These results represent sources contributing at least 1 percent to the seismic hazard at the site for the spectral periods considered and for the given return period. Background seismicity zones, such as gridded or areal sources, are not presented. The numbers in brackets represent unique sub-sections on a given fault. The rupture distance (R_{RUP}) and mean moment magnitude (M_W) are listed for each scenario. Note the mean M_W for each scenario varies with spectral period; thus, we show the maximum of mean M_W values from those periods where the source contributes significantly to the hazard. Note that the above fault tables are not exhaustive lists and other faults in the region may generate seismic shaking at the project site.

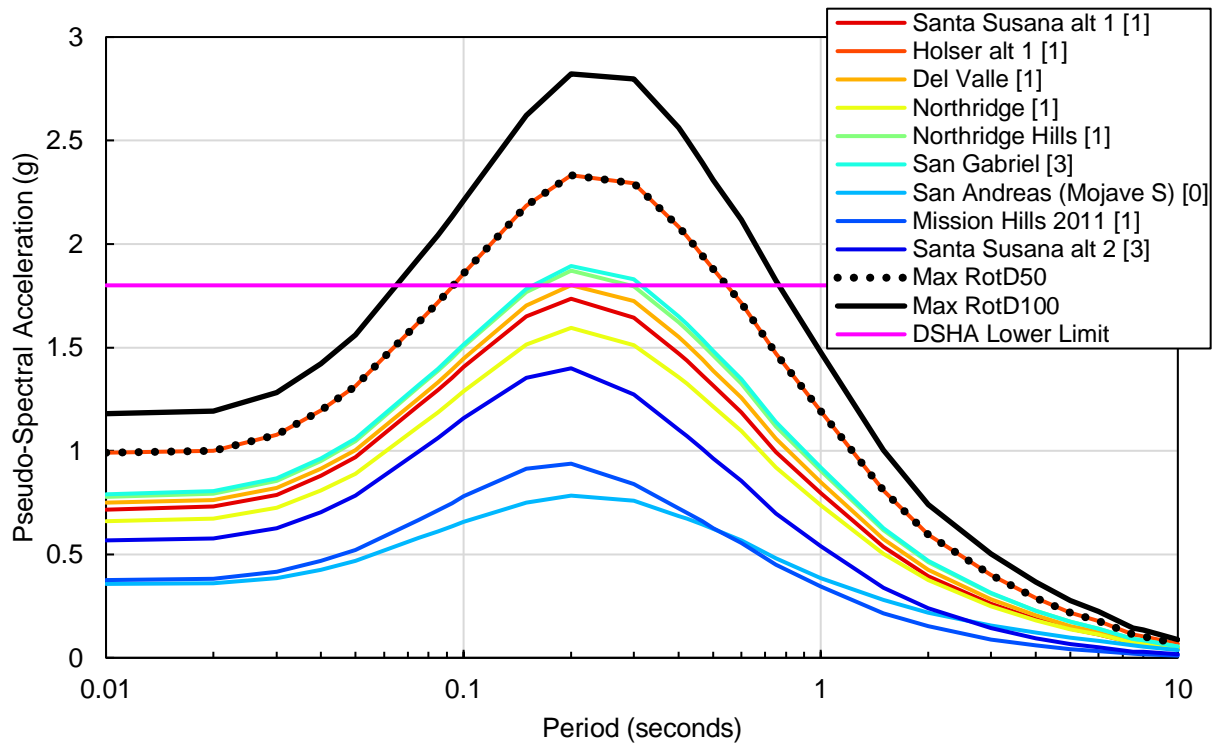
4.4 DETERMINISTIC SEISMIC-HAZARD ANALYSIS

We performed a DSHA to develop the 84th-percentile (i.e., lognormal mean plus one standard deviation) maximum-rotated response spectrum for a spectral damping of 5 percent of critical damping considering characteristic magnitudes of significant faults, without background seismicity, and utilizing the ground-motion models previously discussed. However, the definition of the characteristic magnitude is ambiguous when using the UCERF3 model due to its complexity. Based on the 2020 National Earthquake Hazards Reduction Program (NEHRP) Provisions, in deterministic analyses, “scenario” earthquakes with significant contribution to hazard should be used in lieu of “characteristic” earthquakes when using the UCERF3 model. We identified the scenario earthquakes by considering the PSHA disaggregation results previously discussed.

We considered the magnitudes in Table 4.3.2-1 and associated distance metrics (R_{RUP} , R_{JB} , and R_X) to calculate an 84th-percentile deterministic response spectrum for each source. We estimated additional ground-motion model parameters (e.g., rupture width, depth to top of rupture, etc.) based on the UCERF3 model and fault-specific information published by the United States Geological Survey (USGS). We present the median component (RotD50), 84th-percentile deterministic response spectra in Exhibit 4.4-1. Note that each response spectrum represents the weighted average of the four GMMs for each source. Similar to the probabilistic response spectra, we applied maximum rotation factors discussed in Shahi and Baker (2014) to develop the maximum-rotated, 84th-percentile deterministic response spectrum. We show the deterministic response spectra in Exhibit 4.4-1.

We compared the maximum-rotated 84th-percentile deterministic response spectrum with the lower limit defined in Section 21.2.2 of ASCE 7-16, updated in Supplement No. 1. Per Supplement No. 1, the maximum PSa of the deterministic response spectrum shall not be less than the lower limit defined as $1.5F_a$, where F_a is the short-period site coefficient corresponding to a short-period mapped acceleration (S_s) of 1.5 g (gravity). For Site Class C, the F_a is 1.2 and the lower limit is 1.8 g. Since the maximum PSa of the maximum-rotated 84th-percentile deterministic response spectrum is greater than 1.8 g, no additional scaling is required. We show the final DSHA response spectrum in Exhibit 4.4-1.

EXHIBIT 4.4-1: 84th-Percentile Deterministic Response Spectra



4.5 RESULTING SURFACE RESPONSE SPECTRA

According to Section 21.2.3 of ASCE 7-16, the MCE_R is the minimum of the risk-targeted, maximum-rotated probabilistic and the maximum-rotated 84th-percentile deterministic response spectra. Additionally, the MCE_R is not permitted to be lower than 80 percent of the associated general MCE_R response spectrum for Site Class C (i.e., the code minimum). We present the site-specific MCE_R response spectrum in Exhibit 4.5-1. In addition, we calculated the DE response spectrum, which is defined as two-thirds of the MCE_R . We provide the recommended values for the site-specific MCE_R and DE response spectra in Table 4.5-1.

EXHIBIT 4.5-1: Site-Specific MCE_R Response Spectrum

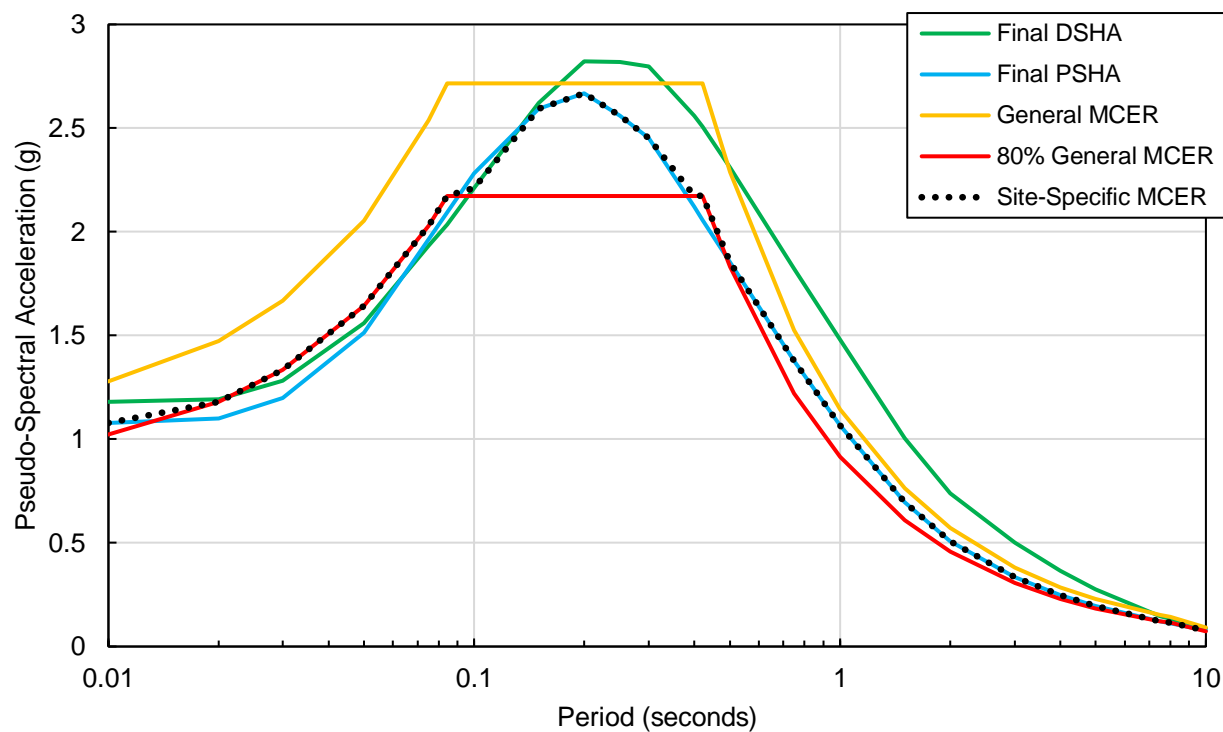


TABLE 4.5-1: Recommended Site-Specific Response Spectra

PERIOD (seconds)	PSEUDO-SPECTRAL ACCELERATION (g)	
	MCE _R	DE
0.010	1.079	0.719
0.020	1.178	0.786
0.030	1.333	0.889
0.050	1.643	1.095
0.075	2.030	1.353
0.084	2.172	1.448
0.100	2.210	1.473
0.150	2.595	1.730
0.200	2.668	1.778
0.250	2.558	1.705
0.300	2.450	1.633
0.400	2.172	1.448
0.421	2.172	1.448
0.500	1.851	1.234
0.750	1.376	0.917
1.000	1.066	0.710
1.500	0.697	0.464
2.000	0.509	0.339
3.000	0.335	0.223

PERIOD (seconds)	PSEUDO-SPECTRAL ACCELERATION (g)	
	MCE _R	DE
4.000	0.248	0.166
5.000	0.195	0.130
7.500	0.122	0.081
8.000	0.114	0.076
10.000	0.077	0.051
0.010	1.079	0.719
0.020	1.178	0.786

We provide the associated design acceleration parameters that we developed in accordance with Section 21.4 of ASCE 7-16 in Table 4.5-2. In addition, we provide the site-specific RotD50 peak ground acceleration (PGA_M) resulting from our analysis in Table 4.5-2. The PGA_M does not include risk or maximum-rotation factors and is controlled by the DSHA.

**TABLE 4.5-2: Design Acceleration Parameters based on ASCE 7-16 Sections 21.4 and 21.5
(Latitude: 34.4143 Longitude: -118.6020)**

ACCELERATION PARAMETER	VALUE (g)
Mapped MCE _R Spectral Response Acceleration at Short Periods, S _s	2.263
Mapped MCE _R Spectral Response Acceleration at 1-second Period, S ₁	0.817
MCE _R Spectral Response Acceleration at Short Periods, S _{MS}	2.401
MCE _R Spectral Response Acceleration at 1-second Period, S _{M1}	1.066
Design Spectral Response Acceleration at Short Periods, S _{DS}	1.601
Design Spectral Response Acceleration at 1-second Period, S _{D1}	0.710
MCE _G peak ground acceleration adjusted for site class effects, PGA _M	0.991

5.0 CONSTRUCTION MONITORING

Our experience and that of our profession clearly indicate that the risk of costly design, construction, and maintenance problems can be significantly lowered by retaining the design geotechnical engineering firm to:

1. Review the final precise grading and foundation plans and specifications prior to construction to evaluate whether our recommendations have been implemented, and to provide additional or modified recommendations, as needed. This also allows us to check if any changes have occurred in the nature, design, or location of the proposed improvements and provides the opportunity to prepare a written response with updated recommendations.
2. Perform construction monitoring to check the validity of the assumptions we made to prepare this report. Earthwork operations should be performed under the observation of our representative to check that the site is properly prepared, the selected fill materials are satisfactory, and that placement and compaction of the fill has been performed in accordance with our recommendations and the project specifications. Sufficient notification to us prior to earthwork is important.

If we are not retained to perform the services described above, then we are not responsible for any party's interpretation of our report (and subsequent addenda, letters, and verbal discussions).

6.0 EARTHWORK RECOMMENDATIONS

As used in this report, relative compaction refers to the in-place dry unit weight of soil expressed as a percentage of the maximum dry unit weight of the same soil, as determined by the ASTM D1557 laboratory compaction test procedure, latest edition. Compacted soil is not acceptable if it is unstable; it should exhibit only minimal flexing or pumping, as observed by an ENGEO representative. The term “moisture condition” refers to adjusting the moisture content of the soil by either drying if too wet or adding water if too dry. We define “structural areas” as any area sensitive to settlement of compacted soil. These areas include, but are not limited to building pads, sidewalks, pavement areas, and retaining walls.

6.1 GENERAL SITE CLEARING

Areas to be developed should be cleared of surface and subsurface deleterious materials, including debris, and designated trees, shrubs, and associated roots, as applicable. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 6.6. ENGEO should be retained to observe and test backfilling.

Following clearing, the site should be stripped to remove surface organic materials. Strip organics from the ground surface. Remove strippings from the site or, if considered suitable by the landscape architect and owner, use them in landscape fill.

6.2 REMOVALS

As noted in Section 2.4, we observed a temporary detention basin in the northwestern corner of the site during our site exploration. Due to the potential of saturated soil from standing water, we recommend that the upper approximately 1 to 2 feet of the basin bottom be removed and reworked prior to beginning engineered fill placement. After the reworking has been completed and a suitable firm base has been achieved, engineered fill placement may progress as outlined in Sections 6.3 and 6.6.

6.3 BENCHING

As engineered fill is placed within the footprint of the temporary detention basin, a horizontal bench or benches should be excavated on the interior basin slopes into firm soil or bedrock as the filling proceeds. The bench(es) should extend a sufficient horizontal distance into bedrock or firm competent engineered fill as to allow proper compaction of the new fill being placed and adequate bonding with the existing engineered fill, as determined by our field representative.

6.4 OVER-OPTIMUM SOIL MOISTURE CONDITIONS

The contractor should anticipate encountering excessively over-optimum (wet) soil moisture conditions during winter or spring grading, or during or following periods of rain. Wet soil can make proper compaction difficult or impossible. Wet soil conditions can be mitigated by:

1. Frequent spreading and mixing during warm dry weather;
2. Mixing with drier materials;
3. Mixing with a lime, lime-fly ash, or cement product; or
4. Stabilizing with aggregate or geotextile stabilization fabric, or both.

Options 3 and 4 should be evaluated by ENGEO prior to implementation.

6.5 ACCEPTABLE FILL

Except for organically contaminated near-surface material, the site soil and bedrock containing less than 3 percent organics are suitable for use as engineered fill. No rock fragments larger than 6 inches in diameter shall be placed in the fill. ENGEO should be consulted on the use of rock greater than 6 inches in diameter.

Pipe zone backfill (i.e. material beneath and immediately surrounding the pipe) may consist of a well-graded import or native material less than $\frac{3}{4}$ inch in maximum dimension compacted in accordance with the permitting jurisdiction's specifications. Where import material is used for pipe zone backfill, we recommend it consist of fine- to medium-grained sand or a well-graded mixture of sand and gravel, and that this material not be used within 1 foot of finish grades. In general, uniformly graded gravel should not be used for pipe or trench zone backfill due to the potential for migration of (1) soil into the relatively large void spaces present in this type of material, and (2) water along trenches backfilled with this type of material.

Imported fill material should meet the above requirements and have a PI similar to, or less than, the on-site soil material. We should be given the opportunity to sample and test proposed imported fill materials at least 5 days prior to delivery to the site.

6.6 FILL COMPACTION

6.6.1 Grading in Structural Areas

The geotechnical engineer's representative should be present during all phases of grading operations to observe site preparation and precise grading operations. Areas to receive fill should be scarified to a minimum depth of 8 inches, moisture conditioned, and recompacted to provide adequate bonding with the initial lift of fill. All fill should be placed in thin compacted lifts that do not exceed 8 inches or the depth of penetration of the compaction equipment used, whichever is less.

The following compaction recommendations should be used for the placement and compaction of fill.

TABLE 6.6.1-1: Compaction and Moisture Content Requirements

FILL LOCATION	MINIMUM RELATIVE COMPACTION (%)	MINIMUM MOISTURE CONTENT (percentage points above optimum moisture content)
General fill and Trench Backfill	90	2
Pavement Subgrade (upper 12 inches)	95	2
Import non-expansive fill	90	Optimum
Class 2 Aggregate Base (AB)	95	Optimum

6.6.2 Underground Utility Backfill

The contractor is responsible for conducting trenching and shoring in accordance with Cal/OSHA requirements. Project consultants involved in utility design should specify pipe bedding materials. Clean "washed" or open-graded rock should be avoided. Place and compact trench backfill per the General Fill recommendation in Section 6.6.1. Jetting of backfill is not an acceptable means of compaction.

6.6.3 Retaining Wall Backfill

Use light compaction equipment within 5 feet of the wall face. If heavy compaction equipment is used, the walls should be temporarily braced to avoid excessive wall movement. Place and compact retaining wall backfill per the General Fill recommendation in Section 6.6.1.

6.6.4 Landscape Fill

Process, place, and compact fill in accordance with Section 6.6, except compact to at least 85 percent relative compaction (ASTM D1557).

6.7 SLOPES

Construct final slope gradients to 2:1 (horizontal:vertical) or flatter. The contractor is responsible to construct temporary construction slopes in accordance with Cal/OSHA requirements.

6.8 SITE DRAINAGE

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we recommend that finish grades be sloped away from buildings and pavements to the maximum extent practical. The latest California Building Code Section 1804.4 specifies minimum slopes of 5 percent away from foundations. Where development conditions restrict meeting this slope requirement, we recommend that specific drainage requirements be developed. As a minimum, we recommend the following.

1. Discharge roof downspouts into closed conduits and direct away from foundations to appropriate drainage devices.
2. Do not allow water to pond near foundations, pavements, or exterior flatwork.

6.9 STORMWATER INFILTRATION

Due to the density of the site soil and fines content (percentage passing the No. 200 sieve) generally exceeding 30 percent, the near-surface site soil is expected to have a low- to moderate-permeability value for stormwater infiltration in grassy swales or permeable pavers, unless subdrains are installed. Therefore, best management practices should assume that limited stormwater infiltration will occur at the site.

6.10 STORMWATER BIORETENTION AREAS

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. When this is not practical, bioretention areas located within 5 feet of structural site improvements can either:

1. Be constructed with structural side walls capable of withstanding the loads from the adjacent improvements, or
2. Incorporate filter material compacted to between 85 and 90 percent relative compaction (ASTM D1557, latest edition) and a waterproofing system designed to reduce the potential for moisture transmission into the subgrade soil beneath the adjacent improvement.

In addition, one of the following options should be followed.

1. We recommend that bioretention design incorporate a waterproofing system lining the bioswale excavation and a subdrain, or other storm drain system, to collect and convey water to an approved outlet. The waterproofing system should cover the bioretention area excavation in such a manner as to reduce the potential for moisture transmission beneath the adjacent improvements.
2. Alternatively, and with some risk of movement of adjacent improvements, if infiltration is desired, we recommend the perimeter of the bioretention areas be lined with an HDPE tree root barrier that extends at least 1 foot below the bottom of the bioretention areas/infiltration trenches.

Site improvements located adjacent to bioretention areas that are underlain by base rock, sand, or other imported granular materials, should be designed with a deepened edge that extends to the bottom of the imported material underlying the improvement.

Where adjacent site improvements include buildings greater than three stories, streets steeper than 3 percent, or design elements subject to lateral loads (such as from impact or traffic patterns), additional design considerations may be recommended. If the surface of the bioretention area is depressed, the slope gradient should follow the slope guidelines described in earlier section(s) of this document. In addition, although not recommended, if trees are to be planted within bioretention areas, HDPE Tree Boxes that extend below the bottom of the bioretention system should be installed to reduce potential impact to subdrain systems that may be part of the bioretention area design. For this condition, the waterproofing system should be connected to the HPDE Tree Box with a waterproof seal.

Given the nature of bioretention systems and possible proximity to improvements, we recommend ENGEO be retained to review design plans and provide testing and observation services during the installation of linings, compaction of the filter material, and connection of designed drains.

It should be noted that the contractor is responsible for conducting all excavation and shoring in a manner that does not cause damage to adjacent improvements during construction and future maintenance of the bioretention areas. As with any excavation adjacent to improvements, the contractor should reduce the exposure time such that the improvements are not detrimentally impacted.

7.0 FOUNDATION RECOMMENDATIONS

We developed foundation recommendations using data obtained from our field exploration, laboratory test results, and engineering analysis. Assuming additional general fill used to achieve precise pad grades will have similar characteristics to the engineered fill used during rough grading, the planned fire station building and apparatus bay may be supported on conventional footings with slab-on-grade floors.

At the time of writing this report, we were not provided with structural plans for the proposed improvements within the site. We should review the structural details, calculations, and proposed loading when they become available. For the purposes of foundation recommendations provided below, we anticipated building loads to be light to moderate.

7.1 CONVENTIONAL FOOTINGS WITH SLAB-ON-GRADE

The proposed fire station building and apparatus bay can be supported on continuous or isolated spread footings bearing in competent native soil or compacted fill.

7.1.1 Footing Dimensions and Allowable Bearing Capacity

Provide minimum footing dimensions as follows in Table 7.1.1-1 below.

TABLE 7.1.1-1: Minimum Footing Dimensions

FOOTING TYPE	*MINIMUM DEPTH (inches)	MINIMUM WIDTH (inches)
Continuous	18	12
Isolated	18	18

*Below lowest adjacent pad grade

Minimum footing depths shown above are taken from lowest adjacent pad grade. Design foundations recommended above for a maximum allowable bearing pressure of 3,000 pounds per square foot (psf) for dead-plus-live loads. Increase this bearing capacity by one-third for the short-term effects of wind or seismic loading.

The maximum allowable bearing pressure is a net value; the weight of the footing may be neglected for design purposes. Footings located adjacent to utility trenches should have their bearing surfaces below an imaginary 1:1 (horizontal:vertical) plane projected upward from the bottom edge of the trench to the footing. The above bearing capacity should not be used for footings within 5 feet of adjacent slopes. Where footings are located on or adjacent to slopes, the footings should be deepened to provide a minimum of 8 feet of horizontal distance between the bottom edge of the footing and the adjacent slope face. As an alternative to deepening the footings, a reduced bearing capacity can be provided based on the actual footing dimensions and slope off-set desired.

7.1.2 Reinforcement

The structural engineer should design footing reinforcement to support the intended structural loads without excessive settlement. Reinforce continuous footings with top and bottom steel to provide structural continuity and to permit spanning of local irregularities.

7.1.3 Foundation Lateral Resistance

Lateral loads may be resisted by friction along the base and by passive pressure along the sides of foundations. The passive pressure is based on an equivalent fluid pressure in pounds per cubic foot (pcf). We recommend the following allowable values for design.

- Passive Lateral Pressure: 350 pcf
- Coefficient of Friction: 0.35

Increase the above values by one-third for the short-term effects of wind or seismic loading. The above values may be combined without further reduction. The above passive lateral pressure should not be used for footings within 5 feet of adjacent slopes. Where footings are located on or adjacent to slopes, the footings should be deepened to provide a minimum of 8 feet of horizontal

distance between the bottom edge of the footing and the adjacent slope face. As an alternative to deepening the footings, a reduced passive pressure can be provided based on the actual footing dimensions and slope off-set desired.

7.1.4 Settlement

Provided our report recommendations are followed and given the proposed construction (Section 1.3), we estimate total and differential foundation settlements to be less than approximately $\frac{1}{2}$ and $\frac{1}{4}$ inches over 30 feet, respectively.

7.2 ANCILLARY STRUCTURES

Ancillary structures may be support on spread footing foundations designed in accordance with the recommendations presented in Section 7.1. For support of exterior posts or columns for lightweight ancillary structures (such as overhead shade and canopy features or flag poles), the following soil parameters can be used for design.

- Passive Lateral Pressure: 350 pcf (equivalent fluid pressure; acting against 2 times the projected area of the individual pier shafts below grade)
- Active Soil Pressure: 45 pcf (equivalent fluid pressure)
- Downward Skin Friction: 550 psf
- Unit Weight of Soil: 125 pcf

Increase the above values by one-third for the short-term effects of wind or seismic loading.

8.0 SLABS-ON-GRADE

8.1 INTERIOR CONCRETE FLOOR SLABS

8.1.1 Minimum Design Section

Based on the moderately expansive site soil, we recommend the following minimum design.

1. Provide a minimum concrete thickness of 5 inches.
2. Place minimum steel reinforcing of No. 3 rebar on 18-inch centers each way within the middle third of the slab to help control the width of shrinkage cracking that inherently occurs as concrete cures.
3. Moisture condition the slab subgrade as outlined in Section 8.1.3 below.

The structural engineer should provide final design thickness and additional reinforcement, as necessary, for the intended structural loads.

8.1.2 Slab Moisture Vapor Reduction

When buildings are constructed with concrete slab-on-grade, water vapor from beneath the slab will migrate through the slab and into the building. This water vapor can be reduced but not stopped. Vapor transmission can negatively affect floor coverings and lead to increased moisture within a building. When water vapor migrating through the slab would be undesirable, we

recommend the following to reduce, but not stop, water vapor transmission upward through the slab-on-grade.

1. Construct a moisture retarder system directly beneath the slab on-grade that consists of the following.
 - a. Vapor retarder membrane sealed at all seams and pipe penetrations and connected to all footings. Vapor retarders shall conform to Class A vapor retarder in accordance with ASTM E1745, latest edition, "Standard Specification for Plastic Water Vapor Retarders used in Contact with Soil or Granular Fill under Concrete Slabs" (such as Stego Wrap Vapor Barrier 20-MIL or equivalent).
 - b. The vapor retarder should be underlain by 4 inches of clean coarse sand and overlain by 2 inches of clean coarse sand. Clean sand should have less than 3 percent passing the No. 200 Sieve.
2. Use a concrete water-cement ratio for slabs-on-grade of no more than 0.50.
3. Provide inspection and testing during concrete placement to check that the proper concrete and water-cement ratio are used.
4. Moist cure slabs for a minimum of 3 days or use other equivalent curing specified by the structural engineer.

8.1.3 Subgrade Moisture Conditioning for Structural Slab

Moisture conditioning of the building interior slab subgrade should be to a moisture content at least 3 percentage points above optimum immediately prior to slab concrete placement. The subgrade should not be allowed to dry prior to concrete placement. We also recommend that we be retained to observe the pre-pour moisture conditions to check that our report recommendations have been followed.

8.1.4 Subgrade Modulus for Structural Slab Design

Provided the site earthwork is conducted in accordance with the recommendations of this report, a subgrade modulus of 120 pounds per square inch per inch (psi/in) can be used for structural slab design.

8.2 EXTERIOR FLATWORK

Exterior flatwork includes items such as concrete sidewalks, steps, and outdoor courtyards exposed to foot traffic only. Provide a minimum section of 4 inches of concrete over compacted subgrade for pedestrian hardscape exposed to foot traffic only. Refer to Table 6.6.1-1 for fill placement specifications. Construct control and construction joints in accordance with current Portland Cement Association Guidelines.

8.3 TRENCH BACKFILL

Backfill and compact all trenches in accordance with Section 6.6.

9.0 RETAINING WALLS

9.1 LATERAL SOIL PRESSURES

Unrestrained site retaining walls up to 10 feet high may be designed for active lateral equivalent fluid pressures as follows.

TABLE 9.1-1: Lateral Soil Pressures, Unrestrained Walls

BACKFILL SLOPE CONDITION	ACTIVE PRESSURE (pcf)	AT-REST PRESSURE (pcf)	SEISMIC INCREMENT (pcf)
Level	45*	65	20
3:1	60	75	35
2:1	75	80	65

*Note: If non-expansive (plasticity index less than 12) material with a minimum Sand Equivalent of 20 is used for backfill of retaining walls, an active pressure of 36 pcf may be used for level backfill slope conditions.

Walls retaining greater than 6 feet of backfill should be designed to resist seismic pressure in addition to static pressures, as provided in Table 9.1-1.

Restrained building retaining walls, if any, should be designed as drained retaining walls using an at-rest fluid pressure of 60 pcf for level backfill conditions. In addition, unrestrained and restrained walls should be designed to resist an additional uniform pressure equivalent to one-third and one-half of any surcharge loads applied at the surface, respectively. The above lateral earth pressures assume drainage behind the walls to prevent build-up of hydrostatic pressures. If drainage is not provided, an additional equivalent fluid pressure of 40 pcf should be added to the values recommended above. Damp-proofing of the walls should be included in areas where moisture migration through walls would be problematic. Appropriate safety factors against overturning (typically FS of 2) and sliding (typically FS of 1.5) should be incorporated into the design calculations.

A minimum horizontal surcharge load equal to 100 psf should be considered for traffic loading where applicable.

9.2 FOUNDATIONS

Retaining walls may be supported on spread footings designed in accordance with the recommendations in Section 7.1. As an alternative to spread footings, retaining walls can be supported by drilled piers. The piers should have a minimum diameter of 12 inches and extend to a depth of at least 6 feet below the existing ground surface. Design piers for an allowable downward skin friction of 550 psf for combined dead-plus-live loads with a one-third increase allowed for either transient wind or seismic loading.

Lateral loads exerted on drilled piers and may be resisted by a passive resistance based on an equivalent fluid pressure of 350 pcf acting against 2 times the projected area of the individual pier shafts below grade, with a maximum of 4,000 psf at depth. Resistance to uplift loads is developed in friction along the pier shafts. We recommend that an allowable uplift frictional resistance of 250 psf be used.

The bottoms of pier excavations should be dry, reasonably clean, and free of loose soil before reinforcing steel is installed and concrete is placed. We recommend that the pier excavations be observed to establish that the piers are founded in suitable materials and constructed in accordance with the recommendations presented in this report.

9.3 RETAINING WALL DRAINAGE

Retaining walls should be provided with drainage facilities to prevent the build-up of hydrostatic pressures behind the walls. Wall drainage may be provided using a 4-inch-diameter perforated pipe (minimum SDR 35 or approved equivalent), using glued joints and end cap, embedded in either free-draining gravel surrounded by synthetic filter fabric (minimum 6-ounce) or Class 2 permeable material. The width of the drain blanket should be at least 12 inches, and the drain blanket should extend to about 1 foot below the soil subgrades. Alternatively, if pre-approved by ENGEO, prefabricated drainage strips/panels may be utilized in lieu of perforated pipe and granular drainage medium. The upper 1 foot of wall backfill and soil backfill behind drainage zones should consist of compacted site soil.

Wall subdrainage should be directed into solid pipes and flow to an outlet approved by the civil engineer, such as through the curb and into the street, or into nearby catch basins or manholes. If necessary, wall subdrains may discharge into area drains, provided the subdrain pipe discharges at least 6 inches above the invert of the area drain pipeline. Under no circumstance should area drains constructed behind the wall be tied into the wall subdrain system. Also, if necessary to achieve gravity drainage, the subdrain pipe could be placed on top of the footing at the base of the retaining wall stem, if approved by the soil engineer during construction.

10.0 PAVEMENT DESIGN

Use concrete pavement sections to resist heavy loads and turning forces. We recommend the following minimum design sections for rigid pavements using ACI 330R-08 Design Guide for Concrete Parking Lots.

TABLE 10.0-1: Rigid Pavement Sections

AVERAGE DAILY TRUCK TRAFFIC (ADTT*)	R-VALUE OF 9	
	CONCRETE (inches)	AB (inches)
1	5	6
10	5½	6
50	5½	7

*Notes: ADTT – average daily truck traffic. Trucks are defined as vehicles with at least six wheels; excludes panel trucks, pickup trucks, and other four-wheel vehicles.
AB – Caltrans Class 2 Aggregate Base (R-Value of 78 or greater)

- Jointed plane concrete pavement (JPCP) should have a minimum 28-day compressive strength of 3,500 psi for a 30-year design life
- The design assumes there is edge support provided by a curb or paving
- The joints should line up with parking stall lines wherever feasible
- Provide minimum control joint spacing in accordance with Portland Cement Association guidelines

10.1 SUBGRADE AND AGGREGATE BASE COMPACTION

Compact finish subgrade and aggregate base in accordance with Section 6.6.1. Aggregate base should meet the requirements for ¾-inch maximum Class 2 AB in accordance with Section 26-1.02B of the latest Caltrans Standard Specifications.

10.2 CUTOFF CURBS

Saturated pavement subgrade or aggregate base can cause premature failure or increased maintenance of asphalt concrete pavements. This condition often occurs where landscape areas directly abut and drain toward pavements. If desired to install pavement cutoff barriers, they should be considered where pavement areas lie downslope of any landscape areas that are to be sprinklered or irrigated, and should extend to a depth of at least 4 inches below the base rock layer. Cutoff barriers may consist of deepened concrete curbs or deep-root moisture barriers.

If reduced pavement life and greater than normal pavement maintenance are acceptable to the owner, then the cutoff barrier may be eliminated.

11.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the Fire Station 46 project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including, but not limited to, developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strive to perform our professional services in accordance with generally accepted principles and practices currently employed in the area; there is no warranty, express or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of report preparation. We developed this report with limited subsurface exploration data. We assumed that our subsurface exploration data are representative of the actual subsurface conditions across the site. Considering possible underground variability of soil and groundwater, additional costs may be required to complete the project. We recommend that the owner establish a contingency fund to cover such costs. If unexpected conditions are encountered, ENGEO must be notified immediately to review these conditions and provide additional and/or modified recommendations, as necessary.

Our services did not include excavation sloping or shoring, soil volume change factors, flood potential, or a geohazard exploration. In addition, our geotechnical exploration did not include work to determine the existence of possible hazardous materials. If any hazardous materials are encountered during construction, the proper regulatory officials must be notified immediately.

This document must not be subject to unauthorized reuse, that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time.

Actual field or other conditions will necessitate clarifications, adjustments, modifications, or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications, or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include on-site construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies, or other changes necessary to reflect changed field or other conditions.

We determined the lines designating the interface between layers on the exploration logs using visual observations. The transition between the materials may be abrupt or gradual. The exploration logs contain information concerning samples recovered, indications of the presence of various materials such as clay, sand, silt, rock, existing fill, etc., and observations of groundwater encountered. The field logs also contain our interpretation of the subsurface conditions between sample locations. Therefore, the logs contain both factual and interpretative information. Our recommendations are based on the contents of the final logs, which represent our interpretation of the field logs.

SELECTED REFERENCES

- Abrahamson, N. A., Silva, W. J., & Kamai, R. 2014. Summary of the ASK14 ground motion relation for active crustal regions. *Earthquake Spectra*, 30(3), 1025-1055.
- Allan E. Seward Engineering Geology, Inc. 2004. Review of Vesting Tentative Tract Map (dated June 14, 2004), Vesting Tentative Tract Map 61105. Job No. 04-2023-4, July 22, 2004.
- American Society of Civil Engineers. 2016. Minimum Design Loads for Buildings and other Structures (ASCE 7-16), Reston, VA: American Society of Civil Engineers.
- American Concrete Institute. 2008. Guide for the Design and Construction of Concrete Parking Lots (ACI 318R-08).
- Boore, D. M., Stewart, J. P., Seyhan, E., & Atkinson, G. M. 2014. NGA-West2 equations for predicting PGA, PGV, and 5% damped PSA for shallow crustal earthquakes. *Earthquake Spectra*, 30(3), 1057-1085.
- Bryant, W. and Hart, E. 2007. Special Publication 42, "Fault-Rupture Hazard Zones in California", Interim Revision 2007, California Department of Conservation.
- California Building Standards Commission. 2022. California Building Code, Volumes 1 and 2, Sacramento, California.
- California Division of Mines and Geology. 1995. Revised official map of Alquist-Priolo Earthquake Fault Hazard Zones, Newhall Quadrangle: California Division of Mines and Geology, scale 1:24,000.
- California Division of Mines and Geology Staff. 1997. Seismic Hazard Zone Report for the Newhall 7.5-Minute Quadrangle; Los Angeles County, California: California Division of Mines and Geology, Figure 3.5, Open –File Report 97-11, scale 1:24,000.
- California Geologic Survey. 2008. Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.
- California Geological Survey. 2018. Earthquake Fault Zones – A Guide for Government Agencies, Property Owners / Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California: Special Publication 42 (SP42). Available at https://www.conservation.ca.gov/cgs/Documents/Publications/Special-Publications/SP_042.pdf.
- California Geologic Survey. 2022. Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for Public Schools, Hospitals, and Essential Services Buildings. November 2022.
- California Geological Survey Staff. 1998. Official Seismic Hazard Zone Map, Newhall quadrangle: California Geological Survey, Official Map of Seismic Hazard Zones, scale 1:24,000.

SELECTED REFERENCES (Continued)

- California Statewide Groundwater Elevation Monitoring System. 2022. CASGEM Online System, [https://www.casgem.water.ca.gov/OSS/\(S\(swmftak4nph4mrv52ge1zt4p\)\)/GIS/PublicViewMap.aspx](https://www.casgem.water.ca.gov/OSS/(S(swmftak4nph4mrv52ge1zt4p))/GIS/PublicViewMap.aspx), accessed February 2, 2022.
- Campbell, K. W., & Bozorgnia, Y. 2014. NGA-West2 ground motion model for the average horizontal components of PGA, PGV, and 5% damped linear acceleration response spectra. *Earthquake Spectra*, 30(3), 1087-1115.
- Chiou, Brian S-J., and Robert R. Youngs. 2014. Update of the Chiou and Youngs NGA model for the average horizontal component of peak ground motion and response spectra. *Earthquake Spectra*, 30(3), 1117-1153.
- County of Los Angeles Department of Public Works. 2025. Geotechnical and Materials Engineering Division, Manual for Preparation of Geotechnical Reports. February 5, 2025.
- Dibblee, T.W., and Ehrenspeck, H.E.ed. 2004. Geologic Map of the Newhall Quadrangle, Los Angeles County, California. Dibblee Foundation Map DF-56. Map Scale 1:24,000.
- Field, E.H., Biasi, G.P., Bird, P., Dawson, T.E., Felzer, K.R., Jackson, D.D., Johnson, K.M., Jordan, T.H., Madden, C., Michael, A.J., Milner, K.R., Page, M.T., Parsons, T., Powers, P.M., Shaw, B.E., Thatcher, W.R., Weldon, R.J., II, and Zeng, Y. 2013. Uniform California earthquake rupture forecast, version 3 (UCERF3)—The time-independent model: U.S. Geological Survey Open-File Report 2013–1165, 97 p., California Geological Survey Special Report 228, and Southern California Earthquake Center Publication 1792, <http://pubs.usgs.gov/of/2013/1165/>.
- Hart, E.W. and Bryant, W.A. 1997. Fault rupture hazard in California: Alquist-Priolo earthquake fault zoning act with index to earthquake fault zone maps: California Division of Mines and Geology Special Publication 42.
- Leighton and Associates, Inc. 2012. Geotechnical Review of the 40-Scale Bulk Grading Plan, Mission Village Tract 61105, Newhall Ranch, Los Angeles County, California. Project No. 062580-100. Dated May 14, 2012.
- Leighton and Associates, Inc. 2023. Draft As-Built Compaction Report, Testing and Observation Services During Mass and Rough Grading; Phase 6 Planning Area, Tracts 61105-01, Mission Village, Newhall Ranch, Los Angeles County, California. Project No. 11688.047. Dated February 23, 2023.
- Lew, M., N. Sitar, L. Al Atik, M. Pourzanjani, and M. B. Hudson. 2010. "Seismic Earth Pressures on Deep Building Basements," in Structural Engineers Association of California, Proceedings of the Annual Convention, 2010.
- R.T. Frankian & Associates. 2009. 100-Scale Plan Review, Vesting Tentative Tract Map No. 61105, Mission Village, Newhall Ranch, Los Angeles County, California. December 21, 2009.

SELECTED REFERENCES (Continued)

- Shahi, S. K., & Baker, J. W. 2014. NGA-West2 models for ground motion directionality. *Earthquake Spectra*, 30(3), 1285-1300.
- Southern California Earthquake Center. 1999. Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Liquefaction in California.
- Wagner, N., and Sitar, N. 2016. On Seismic Response of Stiff and Flexible Retaining Structures, *Soil Dynamics and Earthquake Engineering*, 91, 284-293.

ADDENDUM #2 - APRIL 24, 2026



FIGURES

- FIGURE 1: Vicinity Map
- FIGURE 2: Site Plan
- FIGURE 3: Regional Geologic Map
- FIGURE 4: Fault & Seismicity Map
- FIGURE 5: Seismic Hazard Zone Map
- FIGURE 6: Cross Sections

COPYRIGHT © 2025 BY ENGEO INCORPORATED. THIS DOCUMENT MAY NOT BE REPRODUCED IN WHOLE OR IN PART BY ANY MEANS WHATSOEVER, NOR MAY IT BE QUOTED WITHOUT THE EXPRESS WRITTEN CONSENT OF ENGEO INCORPORATED.

ADDENDUM #2 - APRIL 24, 2026



VICINITY MAP
FIRE STATION 46
VALENCIA, CALIFORNIA

PROJECT NO.: 6538.100.312

SCALE: AS SHOWN

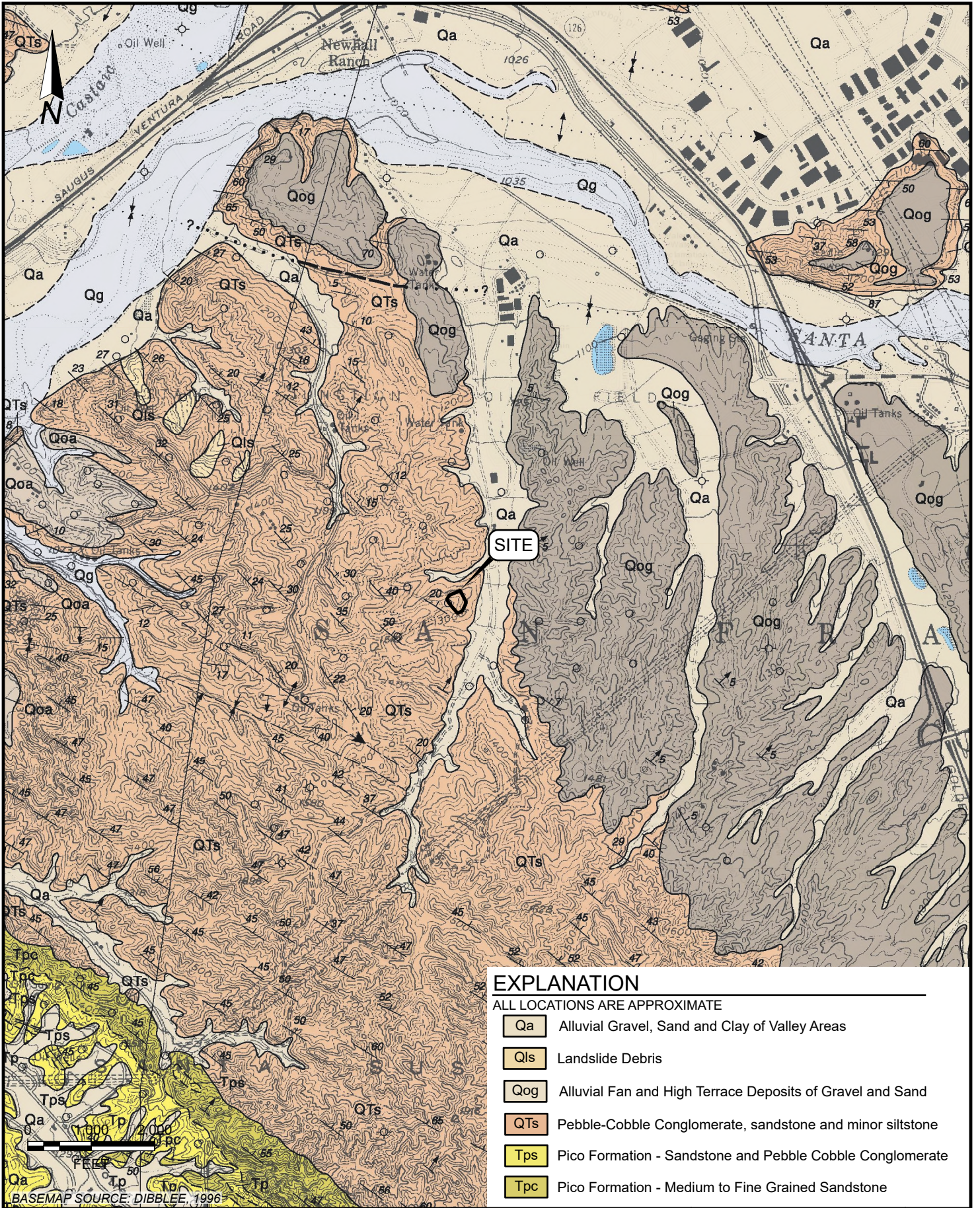
DRAWN BY: NLK

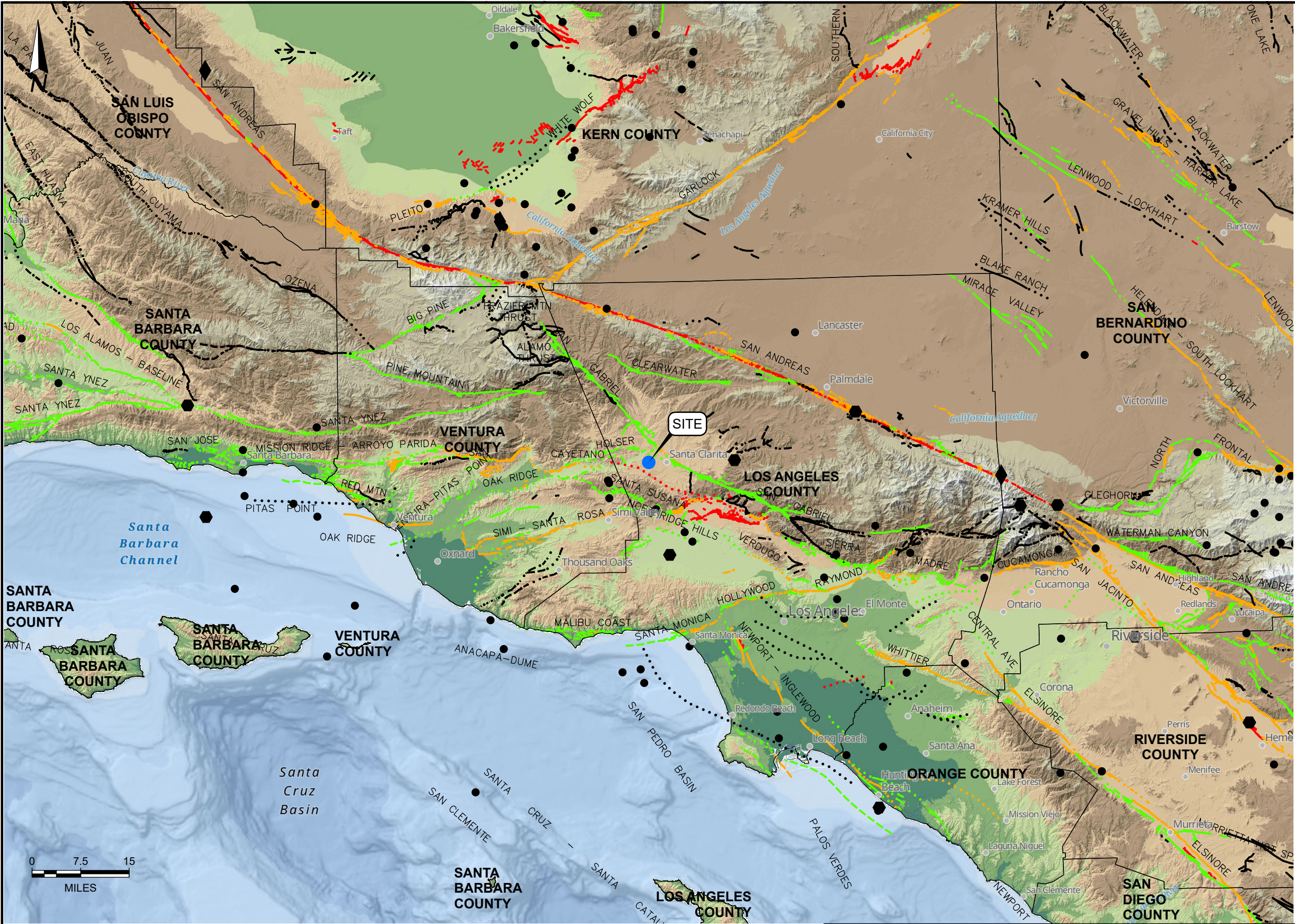
CHECKED BY: SPM

FIGURE NO.

1








EXPLANATION

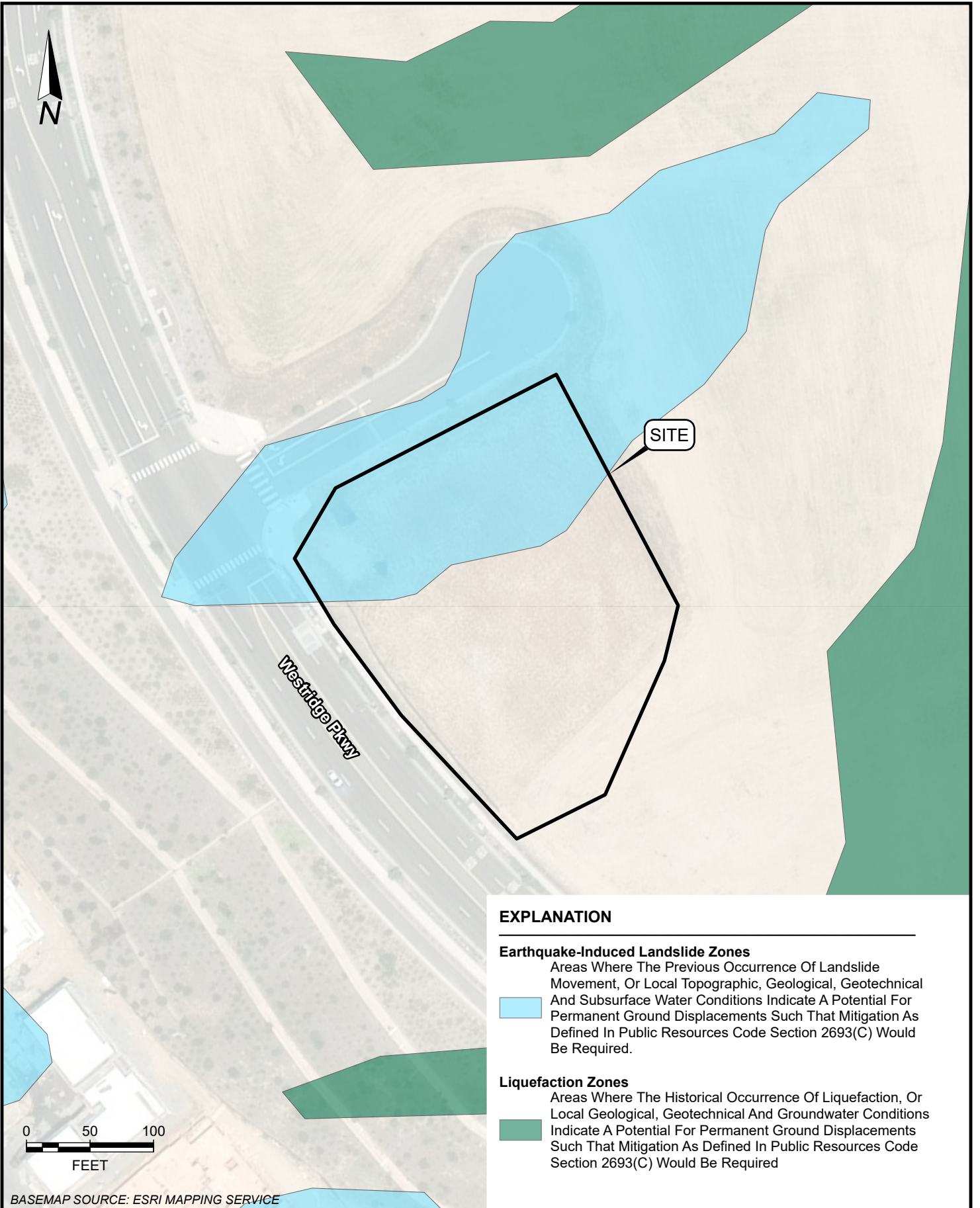
- Project Site
- HISTORIC EARTHQUAKE EPICENTERS**
 - Magnitude 5-6
 - Magnitude 6-7
 - Magnitude 7+
- Historic Blind Thrust Fault Zone
- QUATERNARY FAULTS 2020**
Based on time of most recent surface deformation
 - Historical (<150 Years), Well Constrained Location
 - Historical (<150 Years), Moderately Constrained Location
 - Historical (<150 Years), Inferred Location
 - Latest Quaternary (<15,000 Years), Well Constrained Location
 - Latest Quaternary (<15,000 Years), Moderately Constrained Location
 - Latest Quaternary (<15,000 Years), Inferred Location
 - Late Quaternary (<130,000 Years), Well Constrained Location
 - Late Quaternary (<130,000 Years), Moderately Constrained Location
 - Late Quaternary (<130,000 Years), Inferred Location
 - Middle And Late Quaternary (<750,000 Years), Well Constrained Location
 - Middle And Late Quaternary (<750,000 Years), Moderately Constrained Location
 - Middle And Late Quaternary (<750,000 Years), Inferred Location
 - Undifferentiated Quaternary(<1.6 Million Years), Well Constrained Location
 - Undifferentiated Quaternary(<1.6 Million Years), Moderately Constrained Location
 - Undifferentiated Quaternary(<1.6 Million Years), Inferred Location
 - Class B (Various Age), Well Constrained Location
 - Class B (Various Age), Moderately Constrained Location
 - Class B (Various Age), Inferred Location

BASE MAP SOURCE: ESRI, GARMIN, NATURALVUE, COUNTY OF LOS ANGELES, CALIFORNIA STATE PARKS, ESRI, TOMTOM, GARMIN, SAFEGRAPH, FAO, METI/NASA, USGS, BUREAU OF LAND MANAGEMENT, EPA, NPS, USFWS
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATA SET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, 2020
C.G.S. HISTORIC EARTHQUAKE DATABASE



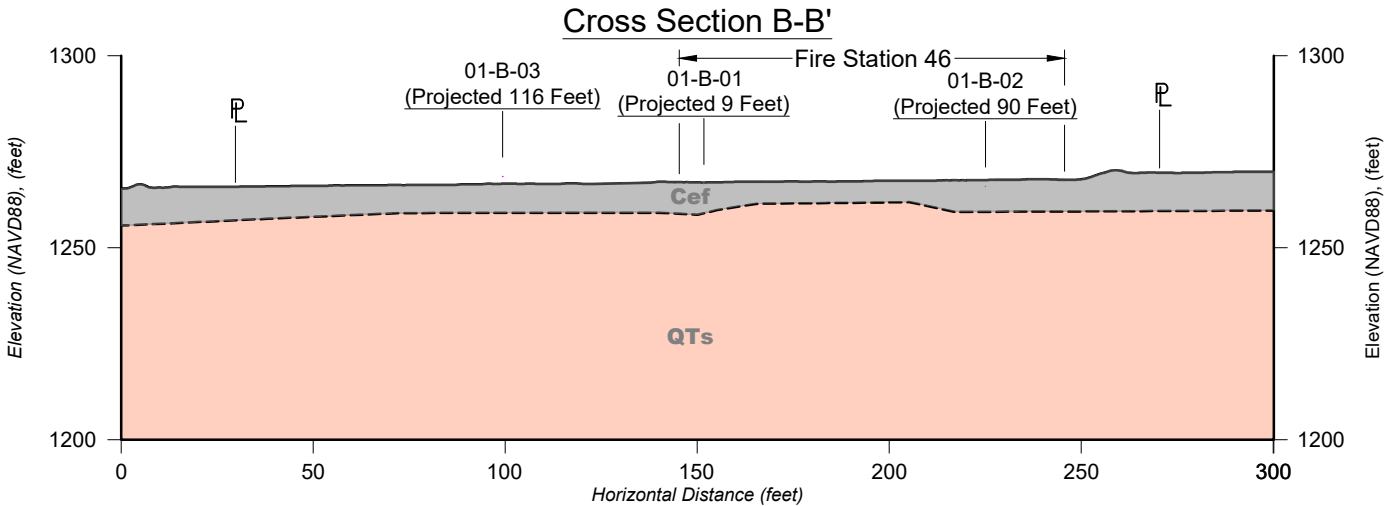
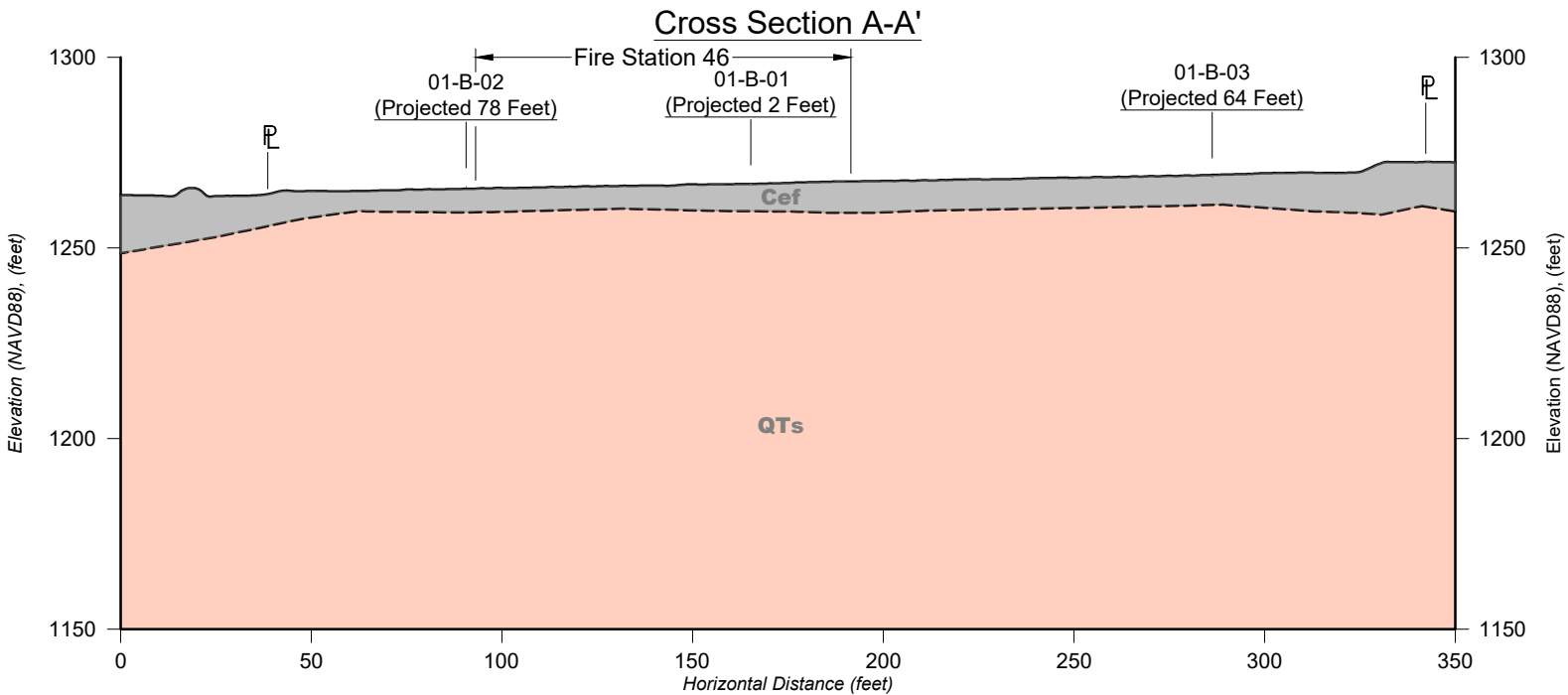
FAULT & SEISMICITY MAP
FIRE STATION 46
VALENCIA, CALIFORNIA

PROJECT NO.: 6538.100.312		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: NLK	CHECKED BY: SPM	




	SEISMIC HAZARD ZONE MAP FIRE STATION 46 VALENCIA, CALIFORNIA	PROJECT NO.: 6538.100.312	FIGURE NO. 5
		SCALE: AS SHOWN	
		DRAWN BY: NLK	
		CHECKED BY: SPM	

FILE PATH: G:\Borings\PROJECTS\6538\6538.100.312\Fire Station\CAD\6538.100.312-GE-A-B-NS-0425.dwg SWE DATE: 4/10/2025 5:48:32 PM SAVED BY: SESpinosa



Legend

- | | | | |
|---|------------------------------|--|----------------------------|
|  | Certified Engineered Fill |  | Current Ground Surface |
|  | Saugus Formation |  | Corrective Grading Surface |
|  | 01-B-03 (Projected 116 Feet) | | Boring (ENGEO, 2025) |

Disclaimer: Cross Section Is For Illustration Purposes Only. The Transition Between Materials May Be Abrupt Or Gradual. Variations Should Be Expected.

BASE MAP SOURCE: TRACEAIR, 2024 AND LEIGHTON, 2024



CROSS SECTIONS
FIRE STATION 46
VALENCIA, CALIFORNIA

PROJECT NO.:	6538.100.312
SCALE:	AS SHOWN
DRAWN BY:	SPPE
CHECKED BY:	SPM

FIGURE NO.
6

ORIGINAL FIGURE PRINTED IN COLOR

ADDENDUM #2 - APRIL 24, 2026



APPENDIX A

KEY TO BORING LOGS
EXPLORATION LOGS

KEY TO BORING LOGS

MAJOR TYPES

DESCRIPTION

COARSE-GRAINED SOILS MORE THAN HALF OF MAT'L LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LESS THAN 5% FINES		GW - Well graded gravels or gravel-sand mixtures
		GRAVELS WITH OVER 12 % FINES		GP - Poorly graded gravels or gravel-sand mixtures
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LESS THAN 5% FINES		GM - Silty gravels, gravel-sand and silt mixtures
		SANDS WITH OVER 12 % FINES		GC - Clayey gravels, gravel-sand and clay mixtures
FINE-GRAINED SOILS MORE THAN HALF OF MAT'L SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT 50 % OR LESS			SW - Well graded sands, or gravelly sand mixtures
				SP - Poorly graded sands or gravelly sand mixtures
				SM - Silty sand, sand-silt mixtures
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50 %			SC - Clayey sand, sand-clay mixtures
				ML - Inorganic silt with low to medium plasticity
				CL - Inorganic clay with low to medium plasticity
	HIGHLY ORGANIC SOILS			OL - Low plasticity organic silts and clays
				MH - Elastic silt with high plasticity
				CH - Fat clay with high plasticity
				OH - Highly plastic organic silts and clays
				PT - Peat and other highly organic soils

For fine-grained soils with 15 to 29% retained on the #200 sieve, the words "with sand" or "with gravel" (whichever is predominant) are added to the group name.

For fine-grained soil with >30% retained on the #200 sieve, the words "sandy" or "gravelly" (whichever is predominant) are added to the group name.

GRAIN SIZES

U.S. STANDARD SERIES SIEVE SIZE

CLEAR SQUARE SIEVE OPENINGS

	200	40	10	4	3/4 "	3"	12"
SILTS AND CLAYS	SAND				GRAVEL		
	FINE	MEDIUM	COARSE		FINE	COARSE	
						COBBLES	BOULDERS

RELATIVE DENSITY

SANDS AND GRAVELS

BLOWS/FOOT (S.P.T.)

VERY LOOSE	0-4
LOOSE	4-10
MEDIUM DENSE	10-30
DENSE	30-50
VERY DENSE	OVER 50

CONSISTENCY

SILTS AND CLAYS

STRENGTH*

VERY SOFT	0-1/4
SOFT	1/4-1/2
MEDIUM STIFF	1/2-1
STIFF	1-2
VERY STIFF	2-4
HARD	OVER 4

MOISTURE CONDITION

DRY	Dusty, dry to touch
MOIST	Damp but no visible water
WET	Visible freewater

LINE TYPES

—————	Solid - Layer Break
-----	Dashed - Gradational or approximate layer break

GROUNDWATER SYMBOLS

	Groundwater level during drilling
	Stabilized groundwater level

SAMPLER SYMBOLS

	Modified California (3" O.D.) sampler
	California (2.5" O.D.) sampler
	S.P.T. - Split spoon sampler
	Shelby Tube
	Dames and Moore Piston
	Continuous Core
	Bag Samples
	Grab Samples
NR	No Recovery

(S.P.T.) Number of blows of 140 lb. hammer falling 30" to drive a 2-inch O.D. (1-3/8 inch I.D.) sampler

* Unconfined compressive strength in tons/sq. ft., asterisk on log means determined by pocket penetrometer

ENGEO
Expect Excellence

ADDENDUM #2 - APRIL 24, 2026

SOIL BORING 1-B01

LATITUDE: 34.41421

LONGITUDE: -118.60193

Mission Village Fire Station
Valencia, California
6538.100.312

DATE DRILLED: 04/02/2025
HOLE DEPTH: 50.5 ft
HOLE DIAMETER: 8 in
SURFACE ELEV.: 1266 ft (NAVD88)

LOGGED BY / REVIEWED BY: MO / RTH
DRILLING CONTRACTOR: ABC Liovin
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
1245			cuttings grade to brown, less gravel present			96								
			CLAYSTONE , brown, very weak to weak, intensely to moderately weathered											
25														
1240			cuttings increase in clay content			60								
30			very sporadic calcium carbonate veins, <5% fine gravel			54								
1235														
35			fine and coarse gravel in cuttings, significant drill chatter											
1230														
			SANDSTONE , yellowish brown, weak, intensely weathered, fine- to medium-grained, trace fine gravel, oxidized											

ADDENDUM #2 - APRIL 24, 2026



SOIL BORING 1-B01

LATITUDE: 34.41421

LONGITUDE: -118.60193

Mission Village Fire Station
Valencia, California
6538.100.312

DATE DRILLED: 04/02/2025
HOLE DEPTH: 50.5 ft
HOLE DIAMETER: 8 in
SURFACE ELEV.: 1266 ft (NAVD88)

LOGGED BY / REVIEWED BY: MO / RTH
DRILLING CONTRACTOR: ABC Liovin
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
1225						50/3"								
45														
1220														
50			increase in coarse gravel			50/6"								

Bottom of boring at approximately 51½ feet below the existing ground surface. Groundwater was not encountered during drilling.

ADDENDUM #2 - APRIL 24, 2026

SOIL BORING 1-B02


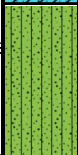

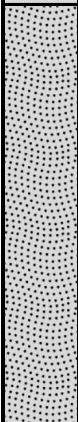
LATITUDE: 34.4145

LONGITUDE: -118.60182

Mission Village Fire Station
Valencia, California
6538.100.312

DATE DRILLED: 04/01/2025
HOLE DEPTH: 31 ft
HOLE DIAMETER: 8 in
SURFACE ELEV.: 1266 ft (NAVD88)

LOGGED BY / REVIEWED BY: MO / RTH
DRILLING CONTRACTOR: ABC Liovin
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
1265			CLAYEY SAND (SC) , reddish brown, medium dense, moist, fine- to coarse-grained sand, fine gravel, trace oxidation [FILL]											
5						48	31	13	37					
1260			SILTY SAND (SM) , brown, dense, moist, fine- to coarse-grained sand, approximately 15% fines, <5% fine gravel, trace oxidation [FILL]			74				12.2	119.5	6389		UC
10			SILTSTONE , yellowish brown, weak to moderately weak, intensely to moderately weathered, fine gravel [SAUGUS FORMATION (QTs)]											
1255			becomes reddish brown			50/5"								
15			SANDSTONE , reddish brown, moderately weak, intensely weathered, fine gravel, oxidized											
1250			cuttings become reddish brown and increase in gravel content			55								

ADDENDUM #2 - APRIL 24, 2026

SOIL BORING 1-B02

LATITUDE: 34.4145

LONGITUDE: -118.60182

Mission Village Fire Station
Valencia, California
6538.100.312

DATE DRILLED: 04/01/2025
HOLE DEPTH: 31 ft
HOLE DIAMETER: 8 in
SURFACE ELEV.: 1266 ft (NAVD88)

LOGGED BY / REVIEWED BY: MO / RTH
DRILLING CONTRACTOR: ABC Liovin
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
1245			increase in gravel content			55								
25			contains oxidation nodules			79								
1240														
30			becomes gray			50/6"								
1235														

Bottom of boring at approximately 31 feet below the existing ground surface. Groundwater was not encountered during drilling.

SOIL BORING 1-B03

LATITUDE: 34.41384

LONGITUDE: -118.60191

Mission Village Fire Station
Valencia, California
6538.100.312

DATE DRILLED: 04/01/2025
HOLE DEPTH: 31 ft
HOLE DIAMETER: 8 in
SURFACE ELEV.: 1268 ft (NAVD88)

LOGGED BY / REVIEWED BY: MO / RTH
DRILLING CONTRACTOR: ABC Liovin
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
1265			CLAYEY SAND (SC) , reddish brown, medium dense, moist, medium- to coarse-grained sand, approximately 15% fines, approximately 10% fine and coarse gravel, oxidized [FILL] switch from hand auger to hollow-stem drilling			50				7.7 6.1	119.2 127.6	2784		UC
5			fines content increases to approximately 20 to 25%			53				9.7	114.4	4597		UC
1260			CLAYSTONE , yellowish brown, weak, intensely weathered, <5% fine gravel, trace oxidation [SAUGUS FORMATION (QTs)]			94/11"								
10			SANDSTONE , gray, weak to moderately strong, intensely to moderately weathered, ample fine gravel gravel content decreases in cuttings, becomes more reddish brown			50/6"								
1255			becomes gray											
15														
1250			SILTSTONE , gray, weak, intensely weathered, <5% fine-grained sand											

ADDENDUM #2 - APRIL 24, 2026

SOIL BORING 1-B03

LATITUDE: 34.41384

LONGITUDE: -118.60191

Mission Village Fire Station
Valencia, California
6538.100.312

DATE DRILLED: 04/01/2025
HOLE DEPTH: 31 ft
HOLE DIAMETER: 8 in
SURFACE ELEV.: 1268 ft (NAVD88)

LOGGED BY / REVIEWED BY: MO / RTH
DRILLING CONTRACTOR: ABC Liovin
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
1245						92/11"								
25			SANDSTONE, gray, weak to moderately strong, intensely weathered, <5% fine gravel, trace oxidation nodules			50/3"								
1240			cuttings become brown											
30						50/5"								

Bottom of boring at approximately 31 feet below the existing ground surface. Groundwater was not encountered during drilling.

SOIL BORING 1-B01

LATITUDE: 34.41421

LONGITUDE: -118.60193

Mission Village Fire Station
Valencia, California
6538.100.312

DATE DRILLED: 04/02/2025
HOLE DEPTH: 50.5 ft
HOLE DIAMETER: 8 in
SURFACE ELEV.: 1266 ft (NAVD88)

LOGGED BY / REVIEWED BY: MO / RTH
DRILLING CONTRACTOR: ABC Liovin
DRILLING METHOD: Hollow Stem Auger
HAMMER TYPE: 140 lb. Auto Trip

Depth (ft)	Elevation (ft)	Sampler Type	MATERIAL DESCRIPTION	Graphic Log	Water Levels	Blow Count (blows/ft) or Penetration Resistance	Liquid Limit	Plasticity Index	Fines (%)	Moisture Content (%)	Dry Density (pcf)	Compressive Strength (psf) *Field Approximation (tsf)	Shear Strength (psf) *Field Approximation (tsf)	Strength Test Type
1265			LEAN CLAY WITH SAND (CL) , very stiff, moist, fine- to medium-grained sand, <5% fine gravel [FILL]											
			switch from hand auger to hollow-stem drilling											
5						50	39	16	78	9.2	113.5			
1260			SILTY SAND WITH GRAVEL (SM) , brown, medium dense, moist, fine- to coarse-grained sand, fine gravel [FILL]			13			15					
10			SILTSTONE , light brown, weak to moderately strong, intensely to moderately weathered, fine- to medium-grained sand [SAUGUS FORMATION (QTs)]			50/5"				7.5	106.9			
1255			cuttings grade lighter in color											
15			fine and coarse gravel in cuttings grades to light gray			48								
1250														

ADDENDUM #2 - APRIL 24, 2026

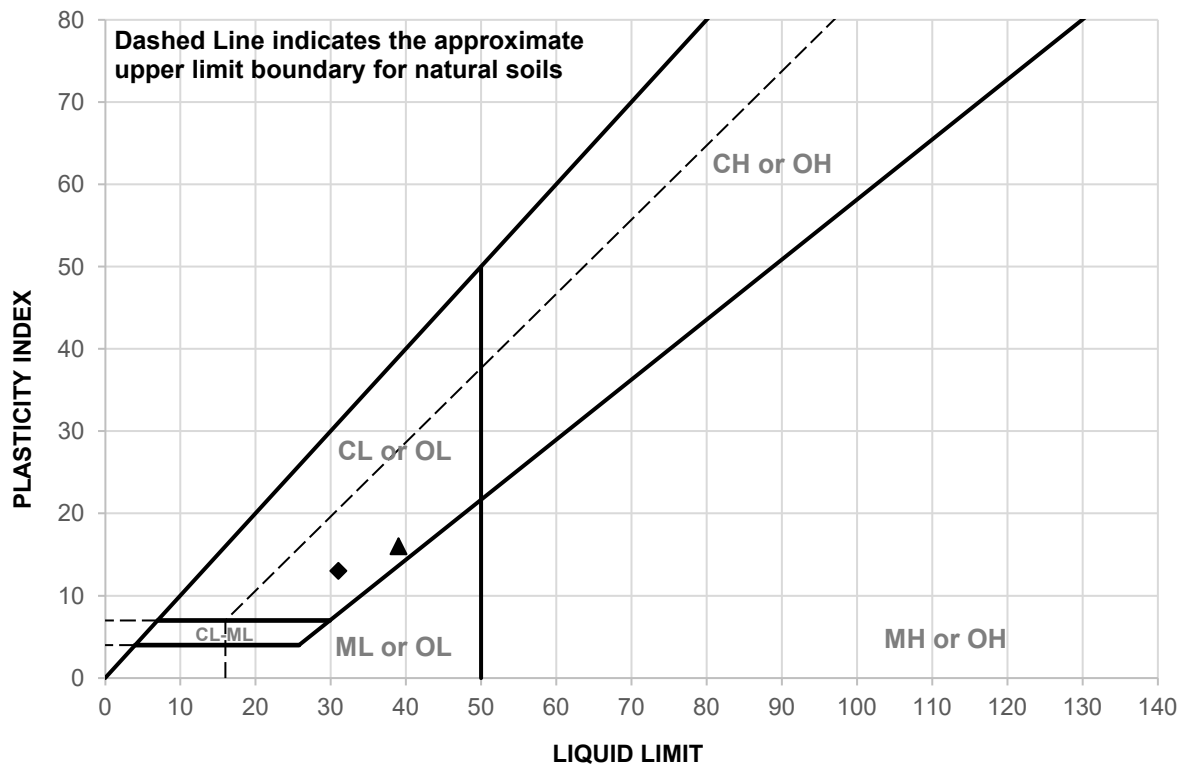


APPENDIX B

LABORATORY TEST DATA

LIQUID AND PLASTIC LIMITS TEST REPORT

ASTM D4318



	SAMPLE ID	DEPTH (ft)	MATERIAL DESCRIPTION	LL	PL	PI
▲	1-B01	4-4.5	See Exploration Logs	39	23	16
◆	1-B02	4-4.5	See Exploration Logs	31	18	13

	SAMPLE ID	TEST METHOD	REMARKS
▲	1-B01	PI: ASTM D4318, Wet Method	
◆	1-B02	PI: ASTM D4318, Wet Method	



CLIENT: Newhall Land and Farming Company
PROJECT NAME: Mission Village Fire Station
PROJECT NO: 6538.100.312 PH001
PROJECT LOCATION: Los Angeles County, CA
REPORT DATE: 4/4/2025
TESTED BY: K. Paul
REVIEWED BY: K. Lecce

EXPANSION INDEX TEST REPORT

ASTM D4829

SAMPLE ID	SOIL DESCRIPTION	SAMPLE LOCATION	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE CONTENT (%)	FINAL MOISTURE CONTENT (%)	EXPANSION INDEX
1-B01 at 3.5-4.5'	See Exploration Logs	1-B01 at 3.5-4.5'	113.5	9.2	20.2	39

TABLE 1: CLASSIFICATION OF EXPANSIVE SOIL
ASTM D4829

EXPANSION INDEX	POTENTIAL EXPANSION
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High



CLIENT: Newhall Land and Farming Company

PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

REPORT DATE: 4/3/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

EXPANSION INDEX TEST REPORT

ASTM D4829

SAMPLE ID	SOIL DESCRIPTION	SAMPLE LOCATION	INITIAL DRY DENSITY (pcf)	INITIAL MOISTURE CONTENT (%)	FINAL MOISTURE CONTENT (%)	EXPANSION INDEX
1-B03 at 3.5-4'	See Exploration Logs	1-B03 at 3.5-4'	119.2	7.7	15.5	12

TABLE 1: CLASSIFICATION OF EXPANSIVE SOIL
ASTM D4829

EXPANSION INDEX	POTENTIAL EXPANSION
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High



CLIENT: Newhall Land and Farming Company

PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

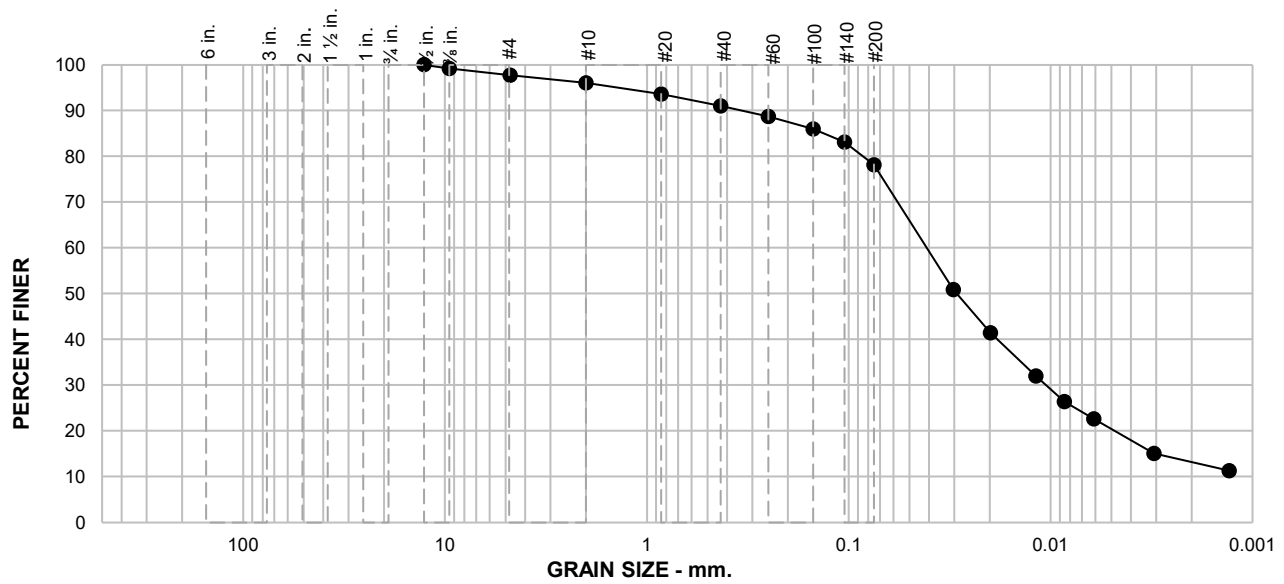
REPORT DATE: 4/4/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D422



SAMPLE ID: 1-B01

DEPTH (ft): 4-4.5

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
			2	2	5	13	65.0	13.0
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION See Exploration Logs				
1/2 in.	100			ATTERBERG LIMITS PL = 23 LL = 39 PI = 16				
3/8 in.	99							
#4	98			COEFFICIENTS D ₉₀ = 0.3279 mm D ₈₅ = 0.1332 mm D ₆₀ = 0.0412 mm D ₅₀ = 0.0292 mm D ₃₀ = 0.0105 mm D ₁₅ = 0.0030 mm D ₁₀ = C _u = C _c =				
#10	96							
#20	94			CLASSIFICATION USCS = CL				
#40	91							
#60	89			REMARKS Silt/clay division of 0.002mm used PI: ASTM D4318, Wet Method USCS: ASTM D2487				
#100	86							
#140	83							
#200	78							
0.0303 mm.	50.8							
0.0199 mm.	41.4							
0.0118 mm.	32.0							
0.0085 mm.	26.4							
0.0061 mm.	22.6							
0.0031 mm.	15.1							
0.0013 mm.	11.3							

* (no specification provided)

CLIENT: Newhall Land and Farming Company



PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

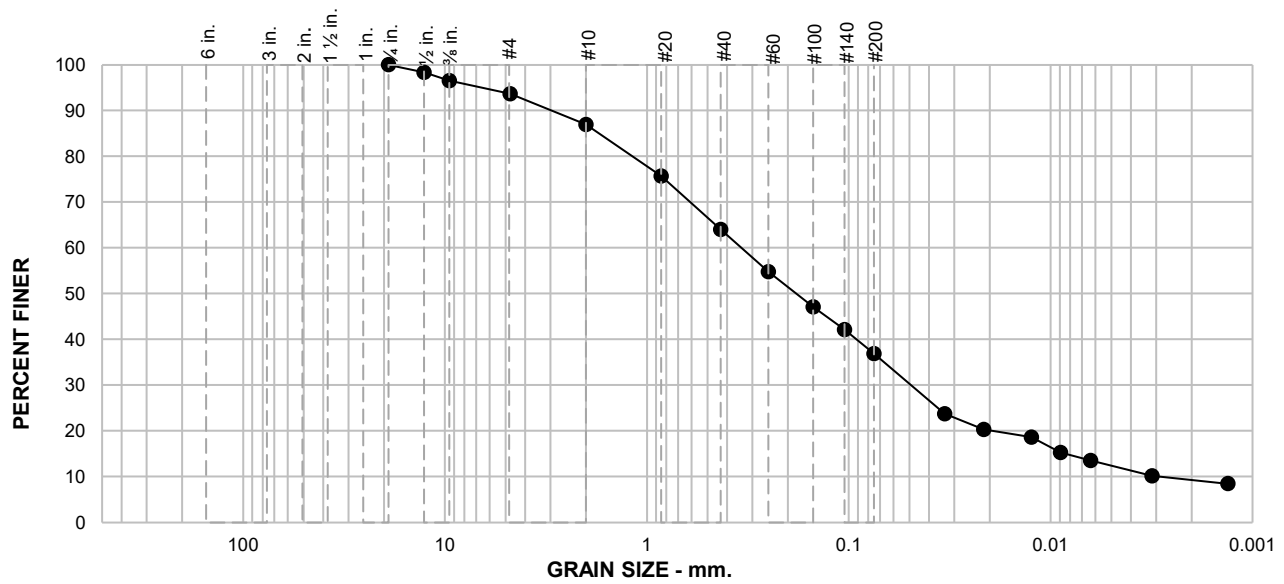
REPORT DATE: 4/4/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D422



SAMPLE ID: 1-B02

DEPTH (ft): 4-4.5

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
			6	7	23	27	28.0	9.0
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION See Exploration Logs				
3/4 in.	100			ATTERBERG LIMITS PL = 18 LL = 31 PI = 13				
1/2 in.	98							
3/8 in.	97			COEFFICIENTS D ₉₀ = 2.8975 mm D ₈₅ = 1.7118 mm D ₆₀ = 0.3379 mm D ₅₀ = 0.1817 mm D ₃₀ = 0.0491 mm D ₁₅ = 0.0085 mm D ₁₀ = 0.0028 mm C _u = 118.76 C _c = 2.51				
#4	94							
#10	87			CLASSIFICATION USCS = SC				
#20	76							
#40	64			REMARKS Silt/clay division of 0.002mm used PI: ASTM D4318, Wet Method USCS: ASTM D2487				
#60	55							
#100	47							
#140	42							
#200	37							
0.0335 mm.	23.7							
0.0215 mm.	20.3							
0.0125 mm.	18.6							
0.0089 mm.	15.2							
0.0063 mm.	13.6							
0.0032 mm.	10.2							
0.0013 mm.	8.5							

* (no specification provided)

CLIENT: Newhall Land and Farming Company



PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

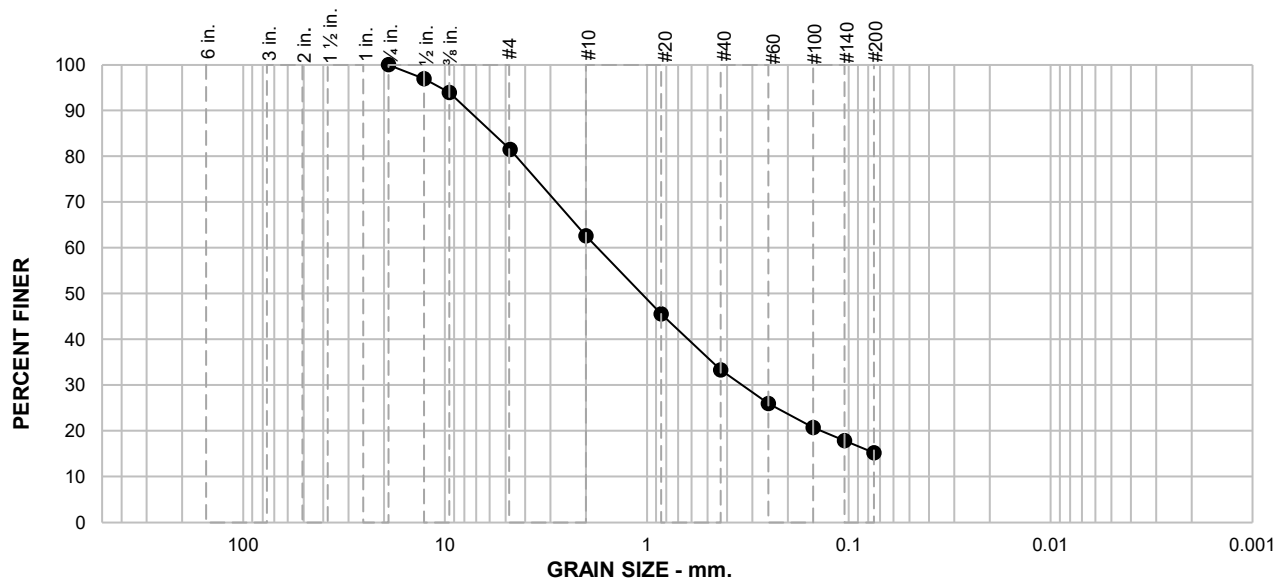
REPORT DATE: 4/4/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

PARTICLE SIZE DISTRIBUTION REPORT

ASTM D6913, Method B



SAMPLE ID: 1-B01

DEPTH (ft): 7.5-8.5

% +75mm		% GRAVEL		% SAND			% FINES	
		COARSE	FINE	COARSE	MEDIUM	FINE	SILT	CLAY
			19	18	30	18	15	
SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)	SOIL DESCRIPTION				
				See Exploration Logs				

* (no specification provided)

CLIENT: Newhall Land and Farming Company



PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

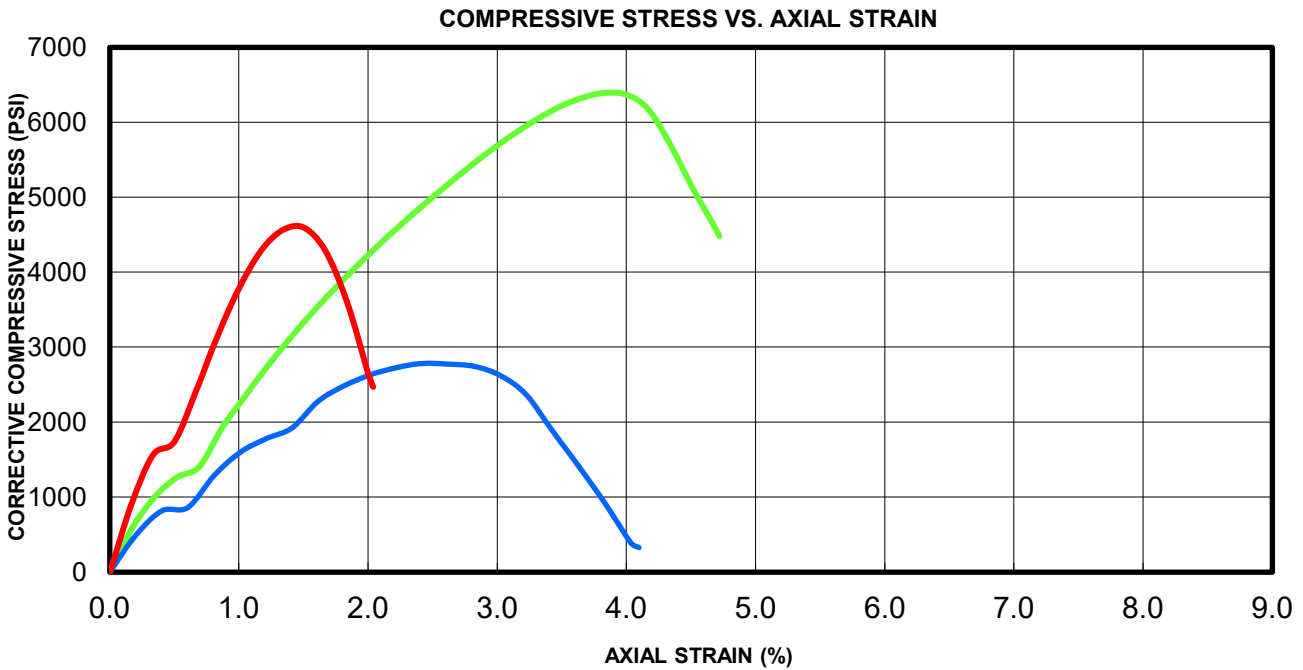
REPORT DATE: 4/3/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

UNCONFINED COMPRESSION TEST REPORT

ASTM D2166



SPECIMEN				
INITIAL PARAMETERS	1-B02 at 7.5-8'	1-B03 at 4-4.5'	1-B03 at 7.5-8'	
MOISTURE (%)	12.18	6.08	9.67	
DRY DENSITY (PCF)	119.5	127.6	114.4	
SATURATION (%)	78.8	50.0	54.3	
VOID RATIO	0.42	0.33	0.48	
DIAMETER (IN.)	2.380	2.380	2.400	
HEIGHT (IN.)	5.770	4.960	6.000	
HEIGHT-DIAMETER RATIO	2.42	2.08	2.50	
TEST DATA	1-B02 at 7.5-8'	1-B03 at 4-4.5'	1-B03 at 7.5-8'	
UNCONFINED COMPRESSIVE STRENGTH	6389	2784	4597	
UNDRAINED SHEAR STRENGTH	3194	1392	2298	
STRAIN RATE (in/min)	0.050	0.050	0.050	
SPECIFIC GRAVITY	2.72	2.72	2.72	
STRAIN AT FAILURE (%)	3.81	2.42	1.50	
SPECIMEN ID		DESCRIPTION		
1	See Exploration Logs			
2	See Exploration Logs			
3	See Exploration Logs			
REMARKS				



CLIENT: Newhall Land and Farming Company

PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

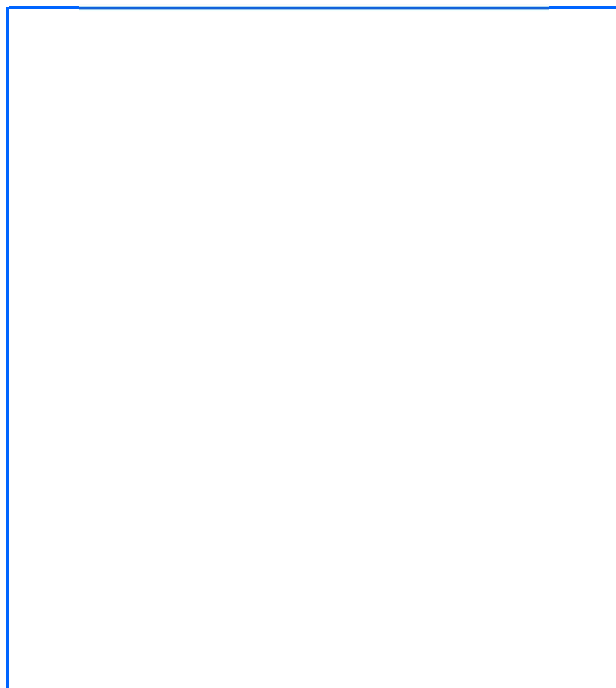
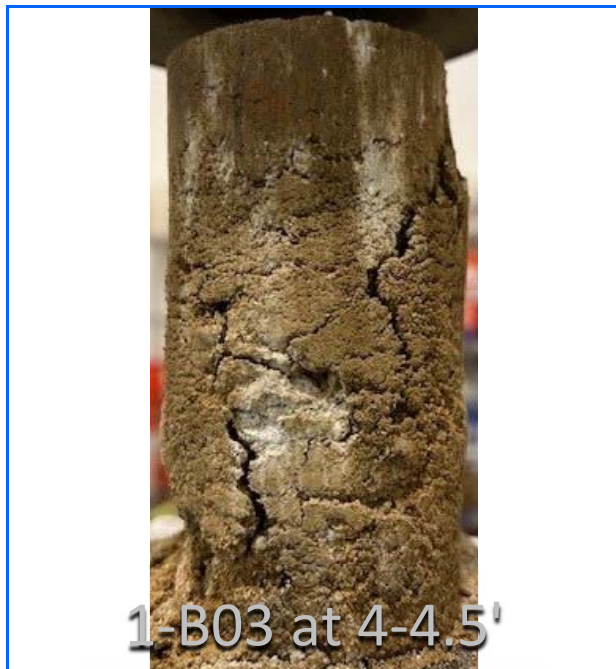
PROJECT LOCATION: Los Angeles County, CA

REPORT DATE: 4/8/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

UNCONFINED COMPRESSION TEST REPORT
ASTM D2166



CLIENT: Newhall Land and Farming Company
PROJECT NAME: Mission Village Fire Station
PROJECT NO: 6538.100.312 PH001
PROJECT LOCATION: Los Angeles County, CA
REPORT DATE: 4/8/2025
TESTED BY: K. Paul
REVIEWED BY: K. Lecce

MOISTURE-DENSITY DETERMINATION REPORT
ASTM D7263

SAMPLE ID	1-B01	1-B02	1-B03	1-B03				
DEPTH (ft.)	10.5-11	7.5-8	4-4.5	7.5-8				
METHOD A OR B	B	B	B	B				
MOISTURE CONTENT (%)	7.5	12.2	6.1	9.7				
DRY DENSITY (pcf)	106.9	119.6	127.6	114.4				



CLIENT: Newhall Land and Farming Company

PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

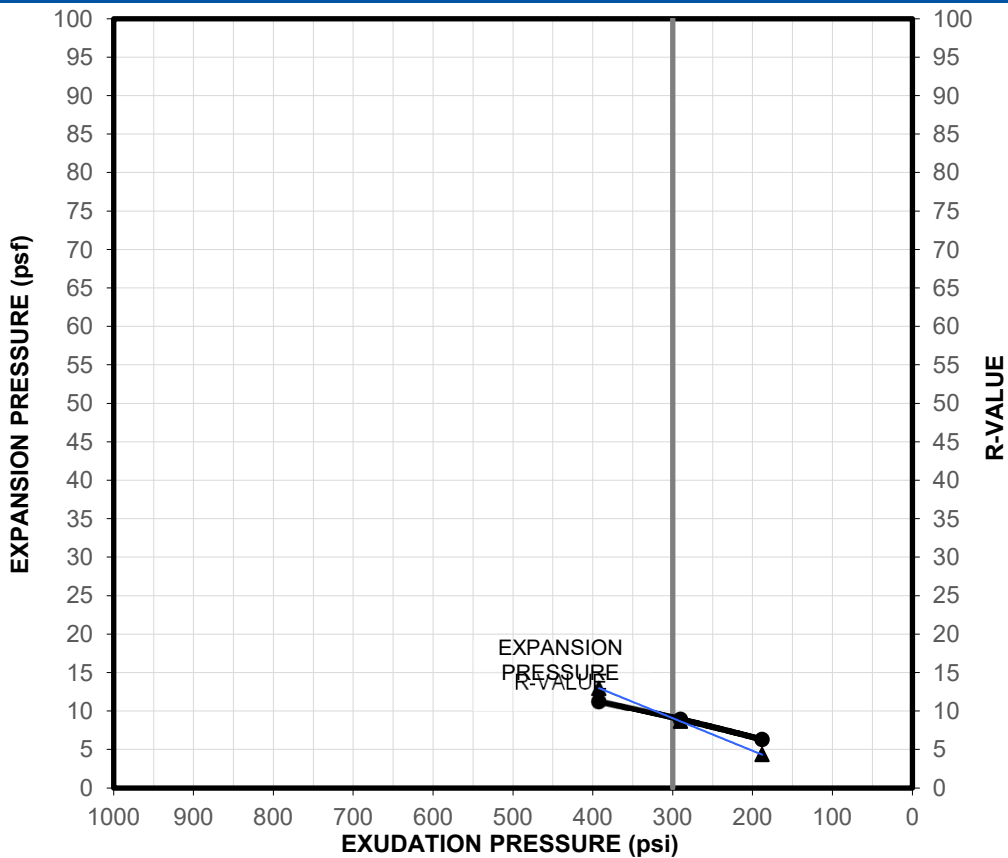
REPORT DATE: 4/10/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION		
1-B02 at 0-5'	See Exploration Logs	1-B02 at 0-5'		
SPECIMENS		1	2	3
EXUDATION PRESSURE (psi)		393	290	188
EXPANSION PRESSURE (psf)		13	9	4
R-VALUE		11	9	6
MOISTURE CONTENT (%)		13.9	14.7	15.6
DRY DENSITY (pcf)		124.7	122.5	120.8
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		9		
R-VALUE AT EXUDATION PRESSURE OF 300 psi		TEST RESULT		
		9		



CLIENT: Newhall Land and Farming Company

PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

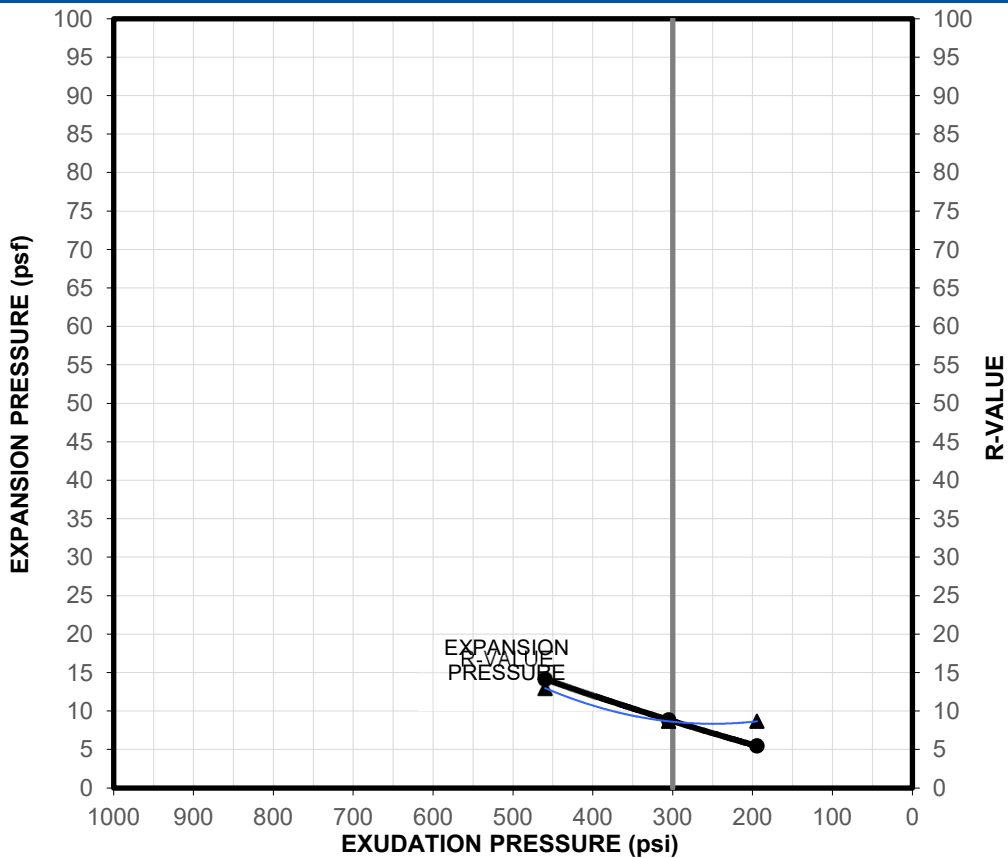
REPORT DATE: 4/7/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

R-VALUE TEST REPORT

CTM 301



SAMPLE ID	MATERIAL DESCRIPTION	SAMPLE LOCATION		
1-B03 at 0-5'	See Exploration Logs	1-B03 at 0-5'		
SPECIMENS		1	2	3
EXUDATION PRESSURE (psi)		460	305	195
EXPANSION PRESSURE (psf)		13	9	9
R-VALUE		14	9	5
MOISTURE CONTENT (%)		12.0	12.9	13.7
DRY DENSITY (pcf)		125.6	123.9	123.5
EXPANSION PRESSURE (psf) AT EXUDATION PRESSURE OF 300 psi		9		
R-VALUE AT EXUDATION PRESSURE OF 300 psi		TEST RESULT		
		9		



CLIENT: Newhall Land and Farming Company

PROJECT NAME: Mission Village Fire Station

PROJECT NO: 6538.100.312 PH001

PROJECT LOCATION: Los Angeles County, CA

REPORT DATE: 4/7/2025

TESTED BY: K. Paul

REVIEWED BY: K. Lecce

9 April, 2025

Job No. 2504020
Cust. No. 13174

Mr. Marlon Oseguera
ENGEO Inc.
27742 Hancock Parkway
Valencia, CA 91355

Subject: Project No.: 6538.100.312
Project Name: MV – FS 46
Corrosivity Analysis – ASTM Test Methods

Dear Mr. Oseguera:

Pursuant to your request, CERCO Analytical has analyzed the soil sample submitted on April 03, 2025. Based on the analytical results, this brief corrosivity evaluation is enclosed for your consideration.

Based upon the resistivity measurement, the sample is classified as “moderately corrosive”. All buried iron, steel, cast iron, ductile iron, galvanized steel and dielectric coated steel or iron should be properly protected against corrosion depending upon the critical nature of the structure. All buried metallic pressure piping such as ductile iron firewater pipelines should be protected against corrosion.

The chloride ion concentration is 42 mg/kg and is determined to be insufficient to attack steel embedded in a concrete mortar coating.

The sulfate ion concentration is 22 mg/kg and is determined to be insufficient to damage reinforced concrete structures and cement mortar-coated steel at this location.


The pH of the soil is 8.41, which does not present corrosion problems for buried iron, steel, mortar-coated steel and reinforced concrete structures.

The redox potential is 240-mV and is indicative of potentially “slightly corrosive” soils resulting from anaerobic soil conditions.

This corrosivity evaluation is based on general corrosion engineering standards and is non-specific in nature. For specific long-term corrosion control design recommendations or consultation, please call *JDH Corrosion Consultants, Inc.* at (925) 927-6630.

We appreciate the opportunity of working with you on this project. If you have any questions, or if you require further information, please do not hesitate to contact us.

Very truly yours,
CERCO ANALYTICAL, INC.


J. Darby Howard, Jr., P.E.
President

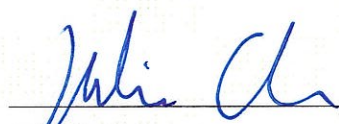
JDH/jdl
Enclosure

Client: ENGEO Incorporated
Client's Project No.: 6538.100.312
Client's Project Name: MV - FS 46
Date Sampled: 31-Mar-25
Date Received: 3-Apr-25
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 9-Apr-2025

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Conductivity (umhos/cm)*	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
2504020-001	1-B02 3.5-4'	240	8.41	-	2,800	-	42	22

Method:	ASTM D1498	ASTM D4972	ASTM D1125M	ASTM G187	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	10	-	50	15	15
Date Analyzed:	3-Apr-2025	4-Apr-2025	-	7-Apr-2025	-	4-Apr-2025	4-Apr-2025


Julia Clauson
Chemist

* Results Reported on "As Received" Basis
N.D. - None Detected

Chain of Custody

1100 Willow Pass Court
Concord, CA 94520-1005
925 462-2771
Fax: 925 462-2775
www.cercoanalytical.com



Page of

Job No. 2504020	CU# 13174	Client Project I.D. 6538.100.312
---------------------------	---------------------	--

Schedule Analyte	Standard TAT	Date Sampled 3/31/25	Date Due
---------------------	--------------	-------------------------	----------

Full Name Marlon Oseguera	Phone 9512028833 X Fax
Company and/or Mailing Address ENGE0 27742 Hancock Parkway, Valencia, CA 91355	Cell 9512028833 <input checked="" type="checkbox"/>
Sample Source 6538.100.312 - MV - FS 46	

Lab No.	Sample I.D.	Date	Time	Matrix	Contain.	Size	Preserv.	Qty.
1	1-B02 3.5-4'	3/31/25		S	Liner			1

ANALYSIS						ASTM w/Brief Evaluation							
Redox Potential	pH	Sulfate	Chloride	Resistivity-100% Saturated	Brief Evaluation								
X	X	X	X	X	X								

MATRIX	DW - Drinking Water	ABBREVIATIONS	HB - Hosebib	SAMPLE RECEIPT	Total No. of Containers	
	GW - Ground Water		PV - Petcock Valve		Rec'd Good Cond/Cold	
	SW - Surface Water		PT - Pressure Tank		Conforms to Record	
	WW - Waste Water		PH - Pump House		Temp. at Lab -°C	
Water	RR - Restroom					
SL - Sludge	GL - Glass					
S - Soil	PL - Plastic					
Product	ST - Sterile					

Comments:
HERE IS AN ADDITIONAL CHARGE FOR EXTRUDING SOIL FROM TUBES

Relinquished By: Marlon Oseguera <i>GLS</i>	Date 4/1/25	Time
Received By: <i>Julia Ch</i>	Date 4/3/25	Time 1015
Relinquished By:	Date	Time
Received By:	Date	Time
Relinquished By:	Date	Time
Received By:	Date	Time

Email Address: moseguera@engeo.com

ADDENDUM #2 - APRIL 24, 2026



APPENDIX C

GEOVISION REPORT



REPORT
PS SUSPENSION VELOCITIES

LENNAR BUILDER AREA
VALENCIA, CALIFORNIA

Prepared for

ENGEO
2010 Crow Canyon Place Suite 250
San Ramon, CA 94583
(925) 866-9000

Prepared by

GEOVision Geophysical Services
1124 Olympic Drive
Corona, California 92881
(951) 549-1234

February 19, 2020

Report 20046-01 rev 0

ADDENDUM #2 - APRIL 24, 2026

TABLE OF CONTENTS

TABLE OF CONTENTS	2
TABLE OF FIGURES	3
TABLE OF TABLES.....	3
APPENDICES	3
INTRODUCTION.....	4
SCOPE OF WORK	4
INSTRUMENTATION	5
SUSPENSION VELOCITY INSTRUMENTATION	5
MEASUREMENT PROCEDURES	8
SUSPENSION VELOCITY MEASUREMENT PROCEDURES.....	8
DATA ANALYSIS	9
SUSPENSION VELOCITY ANALYSIS	9
Vs30 ANALYSIS.....	11
RESULTS.....	12
SUSPENSION VELOCITY RESULTS.....	12
Vs30 RESULTS	12
SUMMARY.....	13
DISCUSSION OF SUSPENSION VELOCITY RESULTS	13
QUALITY ASSURANCE	14
SUSPENSION VELOCITY DATA RELIABILITY.....	14
CERTIFICATION	15

Table of Figures

Figure 1: Concept illustration of PS logging system	17
Figure 2: Example of filtered (1400 Hz lowpass) suspension record.....	18
Figure 3: Example of unfiltered suspension record	19
Figure 4: Borehole 1-B01, Suspension R1-R2 P- and S _H -wave velocities.....	20
Figure 5: Borehole 1-B02, Suspension R1-R2 P- and S _H -wave velocities.....	23
Figure 6: Borehole 1-B03, Suspension R1-R2 P- and S _H -wave velocities.....	25

Table of Tables

Table 1. Borehole locations and logging dates	16
Table 2. Logging dates and depth range.....	16
Table 3. Borehole 1-B01, Suspension R1-R2 depths and P- and S _H -wave velocities	21
Table 4. Borehole 1-B02, Suspension R1-R2 depths and P- and S _H -wave velocities	24
Table 5. Borehole 1-B03, Suspension R1-R2 depths and P- and S _H -wave velocities	26

APPENDICES

APPENDIX A	SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS
APPENDIX B	GEOPHYSICAL LOGGING SYSTEMS - NIST TRACEABLE CALIBRATION RECORDS

INTRODUCTION

GEOVision acquired PS Suspension velocity data within three open boreholes in the Lennar Builders Area near Valencia, California. A GEOVision Professional Geophysicist or Engineer reviewed fieldwork, data analysis, and report preparation. A summary of the instrumentation, methods, data analysis, and results follow in this report.

SCOPE OF WORK

This report presents the results of PS Suspension velocity data acquired in three boreholes on February 3 through February 5, 2019, as detailed in Table 1. The purpose of these measurements was to supplement stratigraphic information by acquiring shear wave and compressional wave velocities as a function of depth.

The OYO PS Suspension Logging System was used to obtain in-situ horizontal shear (S_H), and compressional (P) wave velocity measurements in two cased boreholes at 1.6-foot intervals. Measurements followed **GEOVision** Procedure for PS Suspension Seismic Velocity Logging, revision 1.5. Acquired data were analyzed, and a profile of velocity versus depth was produced for both S_H and P waves.

A detailed reference for the PS Suspension velocity measurement techniques used in this study is:

Guidelines for Determining Design Basis Ground Motions, Report TR-102293,
Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7
and 8.

INSTRUMENTATION

Suspension Velocity Instrumentation

Suspension velocity measurements were performed using the PS Suspension logging system, manufactured by OYO Corporation, and their subsidiary, Robertson Geologging. This system directly determines the average velocity of a 3.3-foot high segment of the soil column surrounding the borehole of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the borehole, producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source and compressional-wave source, joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe in these surveys is approximately 25 feet, with the center point of the receiver pair 12.5 feet above the bottom end of the probe.

The probe receives control signals from and sends the digitized receiver signals to the instrumentation on the surface via an armored multi-conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data using a sheave of known circumference fitted with a digital rotary encoder.

The entire probe is suspended in the borehole by the cable; therefore, source motion is not coupled directly to the borehole walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the borehole and surrounding the source. This pressure wave is converted to P and S_H -waves in the surrounding soil and rock as it impinges upon the wall of the borehole. These waves propagate through the soil and rock surrounding the borehole, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H -waves at the receivers is performed using the following steps:

1. The orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H -wave signals are recorded with the source actuated in opposite directions, producing S_H -wave signals of opposite polarity, providing a characteristic S_H -wave signature distinct from the P-wave signal.
3. The 6.3-foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H -wave signal arrives at the receiver. In faster soils or rock, the isolation cylinder is extended to allow greater separation of the P- and S_H -wave signals.
4. In saturated soils, the received P-wave signal is typical of much higher frequency than the received S_H -wave signal, permitting additional separation of the two signals by low pass filtering.
5. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in the fluid is significantly greater than the dimension of the fluid annulus surrounding the probe (feet versus inches scale), preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction, and the horizontal receiver signals are recorded.
3. The source is fired again, and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H -wave arrivals; reversal of the source changes the polarity of the S_H -wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The PS Suspension system has six channels (two simultaneous recording

channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale.

A review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), and sample rate to optimize the quality of the data before recording. Verification of the calibration of the PS Suspension digital recorder is performed at least every twelve months using a NIST traceable frequency source and counter, as presented in Appendix B.

MEASUREMENT PROCEDURES

Suspension Velocity Measurement Procedures

The logging occurred in open, fluid-filled boreholes. Measurements followed the **GEO***Vision* Procedure for PS Suspension Seismic Velocity Logging, revision 1.5. Prior to the logging run, the probe was positioned with the top of the probe even with a stationary reference point. The electronic depth counter was set to the distance between the mid-point of the receiver and the top of the probe, minus the height of the stationary reference point if any. Measurements were verified with a tape measure, and calculations recorded on a field log.

The probe was lowered to the bottom of the borehole, stopping at 1.6-foot intervals to collect data, as summarized in Table 2. At each measurement depth, the measurement sequence of two opposite horizontal records and one vertical record was performed. Gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and saved to disk before moving to the next depth.

Upon completion of the measurements, the probe was returned to the surface, and the zero-depth indication at the depth reference point was verified prior to removal from the borehole.

DATA ANALYSIS

Suspension Velocity Analysis

The recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or the first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 1.0-meter segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into a template to complete the velocity calculations based on the arrival time picks made in PSLOG. The Microsoft Excel® analysis file accompanies this report.

The P-wave velocity over the 6.3-foot interval from source to receiver 1 (S-R1) was also picked, calculated, and plotted for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting the calculated and experimentally verified delay, in milliseconds, from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of the acceleration of the solenoid before the impact.

As with the P-wave records, the recorded digital waveforms were analyzed to locate clear S_H-wave pulses, as indicated by the presence of opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital Fast Fourier Transform – Inverse Fast Fourier Transform (FFT – IFFT) lowpass filtering was used to remove the higher frequency P-wave signal from the S_H-wave signal. Different filter cutoffs were used to separate P- and S_H-waves at different depths, ranging from 600 Hz in the slowest zones to 4000 Hz in the regions of highest velocity. At each depth, the filter frequency was selected to be at least twice the fundamental frequency of the S_H-wave signal being filtered.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted. The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source, or by borehole inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuation.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 6.33-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 4.8 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting the calculated and experimentally verified delay, in milliseconds, from the beginning of the record at the source trigger pulse to source impact.

Poisson's Ratio, ν , was calculated using the following formula:

$$\nu = \frac{\left(\frac{v_s}{v_p}\right)^2 - 0.5}{\left(\frac{v_s}{v_p}\right)^2 - 1.0}$$

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 2, the time difference over the 3.3-foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher

frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H -wave by the residual P-wave signal.

Data and analyses were reviewed by a **GEOVision** Professional Geophysicist or Engineer as a component of the in-house data validation program.

Vs30 Analysis

The average shear wave velocity in the upper 30 meters (V_{s30}) was calculated using the NEHRP method. The PS Suspension logger measures directly the travel time over a 1 meter interval. However, data are logged at $\frac{1}{2}$ meter intervals. The overlapped measurements (at nominal 0.5 m intervals) are overlapping travel times. It is not explicitly correct to use these as representing individual 0.5 m interval velocities. As a result, it is necessary to interpolate to obtain a distance-weighted average V_s value at each 1 m interval. These are then used to calculate the interval times, which are then accumulated to obtain the total travel time over 30 m. V_{s30} is 30 m divided by this total travel time.

RESULTS

Suspension Velocity Results

Suspension R1-R2 P- and S_H-wave velocities for the boreholes 1-B01, 1-B02, and 1-B03 are plotted in Figure 4, 5, and 6, and data are compiled in Tables 3, 4, and 5 respectively. The associated Microsoft Excel[®] analysis files accompany this report. Included in the Microsoft Excel[®] analysis files are Poisson's Ratio calculations, tabulated data, and plots.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figures A-1 through A-3 in Appendix A to aid in visual comparison. Note that R1-R2 data are an average velocity over a 3.3-foot segment of the soil column; S-R1 data are an average over 6.3 feet, creating a significant smoothing relative to the R1-R2 plots. The S-R1 velocity data displayed in this figure are also compiled in Tables A-1 through A-3.

Vs30 Results

The Vs30 value for borehole 1-B03 is 502 meters/second (1,647 feet/second), characterizing it as NEHRP site class C.*

* Site Classifications taken from Table 1615 1.1 Site Class Definitions published in 2000 International Building code, International Code Council, Inc. on page 350

SUMMARY

Discussion of Suspension Velocity Results

PS Suspension velocity data for this project were collected in PVC cased, fluid-filled boreholes.

Suspension PS velocity data quality is judged based upon 5 criteria:

	Criteria	1-B01	1-B02	1-B03
1	Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.	Yes	Yes.	Yes
2	Consistency between data from adjacent depth intervals.	Yes	Yes	Yes
3	Consistent relationship between P-wave and S_H -wave (excluding transition to saturated soils)	Yes No indication of saturation	Yes Saturation occurs at about 15ft BGS	Yes. Appears to be a saturated zone from 48 to 56 feet BGS
4	Clarity of P-wave and S_H -wave onset, as well as damping of later oscillations.	S-wave data is of good quality. P-wave waveforms were not as clear, and more difficult to pick	S-wave data is of good quality. P-wave waveforms were not as clear, and more difficult to pick.	S-wave data is of good quality. P-wave waveforms were not as clear, and more difficult to pick
5	Consistency of profile between adjacent boreholes, if available.	Stiff soils, but lower velocities than 1-B02.	Very stiff soils, approaching soft rock.	Similar stiff soils. $V_{s30}=1647$ fps, a Site Class C

Quality Assurance

These borehole geophysical measurements were performed using industry-standard or better methods for measurements and analysis. All work was performed under **GEOVision** quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of velocity data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Suspension Velocity Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.3-foot interval of depth. This high resolution results in the scatter of values shown in the graphs. Individual measurements are very reliable, with an estimated precision of +/- 5%. Depth indications are very reliable with an estimated precision of +/- 0.2 feet. Standardized field procedures and quality assurance checks contribute to the reliability of these data.

CERTIFICATION

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

Prepared by:

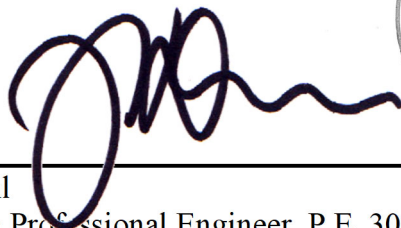


2/19/2020

Jonathan J Jordan
GEOVision Geophysical Services

Date

Reviewed and approved by




2/19/2020

John Diehl
California Professional Engineer, P.E. 30362
GEOVision Geophysical Services

Date

- * 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 and 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.

Table 1. Borehole locations and logging dates

BOREHOLE	DATES	COORDINATES ⁽¹⁾ (US Survey Feet)	
DESIGNATION	LOGGED	Northing	Easting
1-B01	2/4/2020	34.418008	-118.612125
1-B02	2/3/2020	34.418606	-118.612125
1-B03	2/5/2020	34.422538	-118.605729

⁽¹⁾ Coordinates provided by Engeo

Table 2. Logging dates and depth ranges

BOREHOLE NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (FEET)	OPEN HOLE (FEET)	SAMPLE INTERVAL (FEET)	DATE LOGGED
1-B01	SUSPENSION DOWN01	3.28 – 67.26	115	1.6	2/4/2020
1-B02	SUSPENSION DOWN01	3.28 – 49.21	115	1.6	2/3/2020
1-B03	SUSPENSION DOWN01	3.28– 96.78	115	1.6	2/5/2020

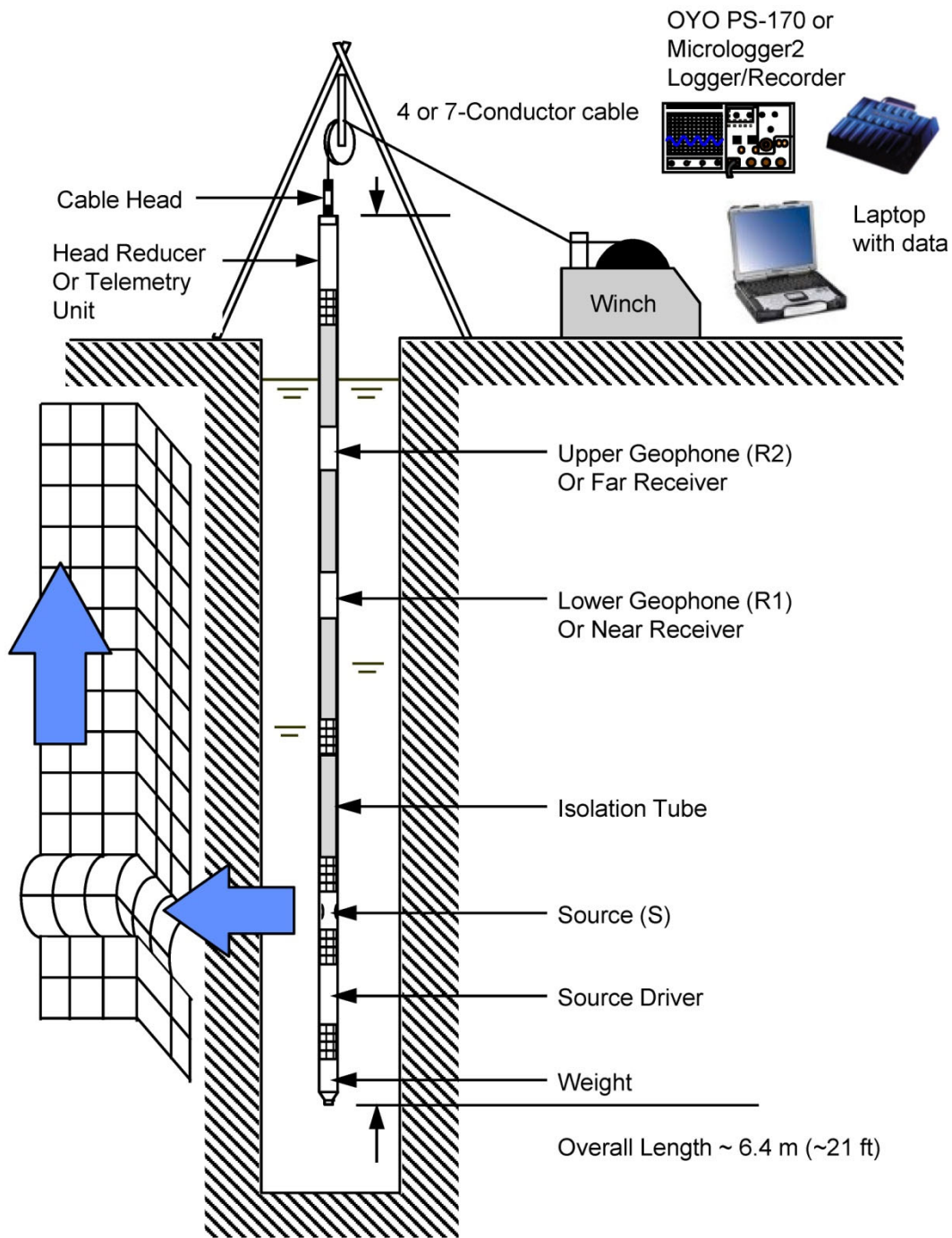


Figure 1: Concept illustration of PS logging system

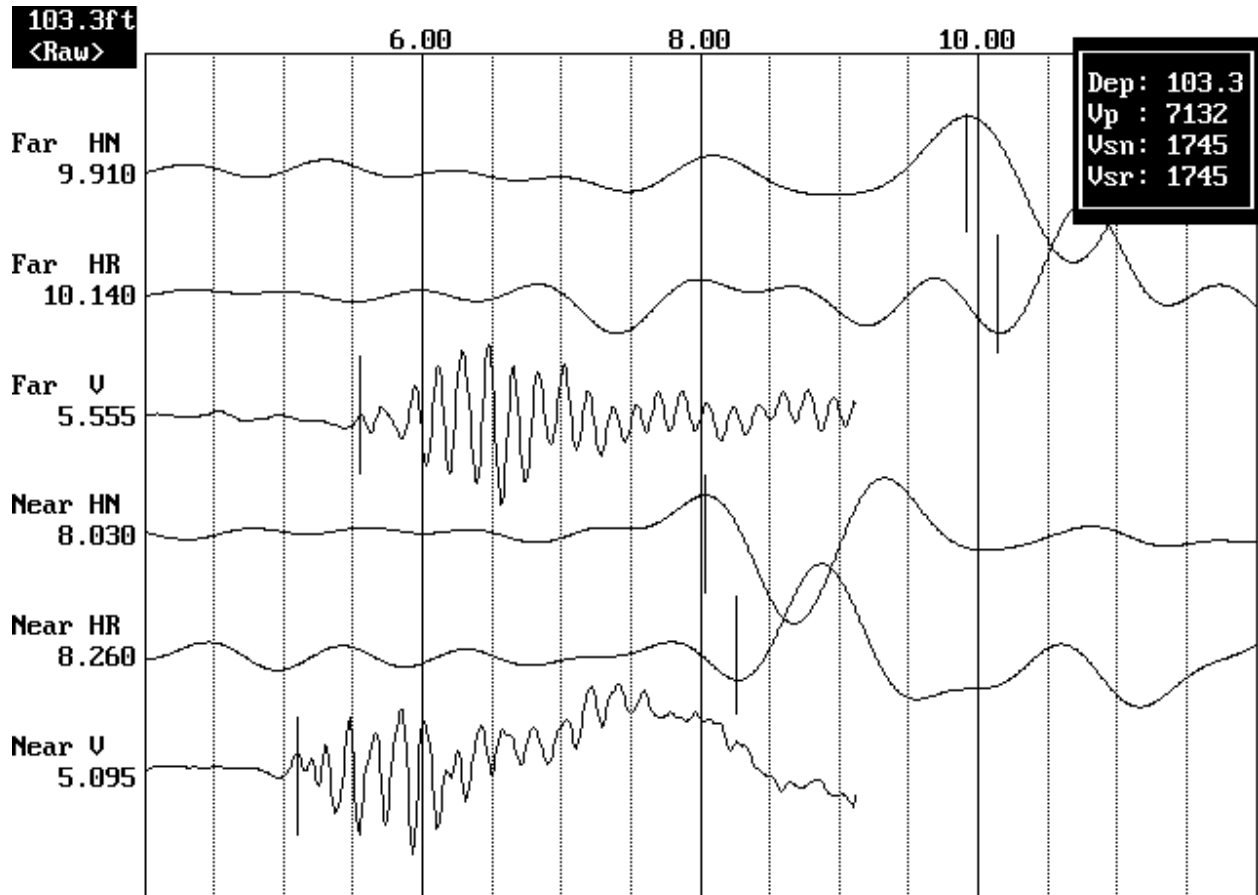


Figure 2: Example of filtered (1400 Hz lowpass) suspension record

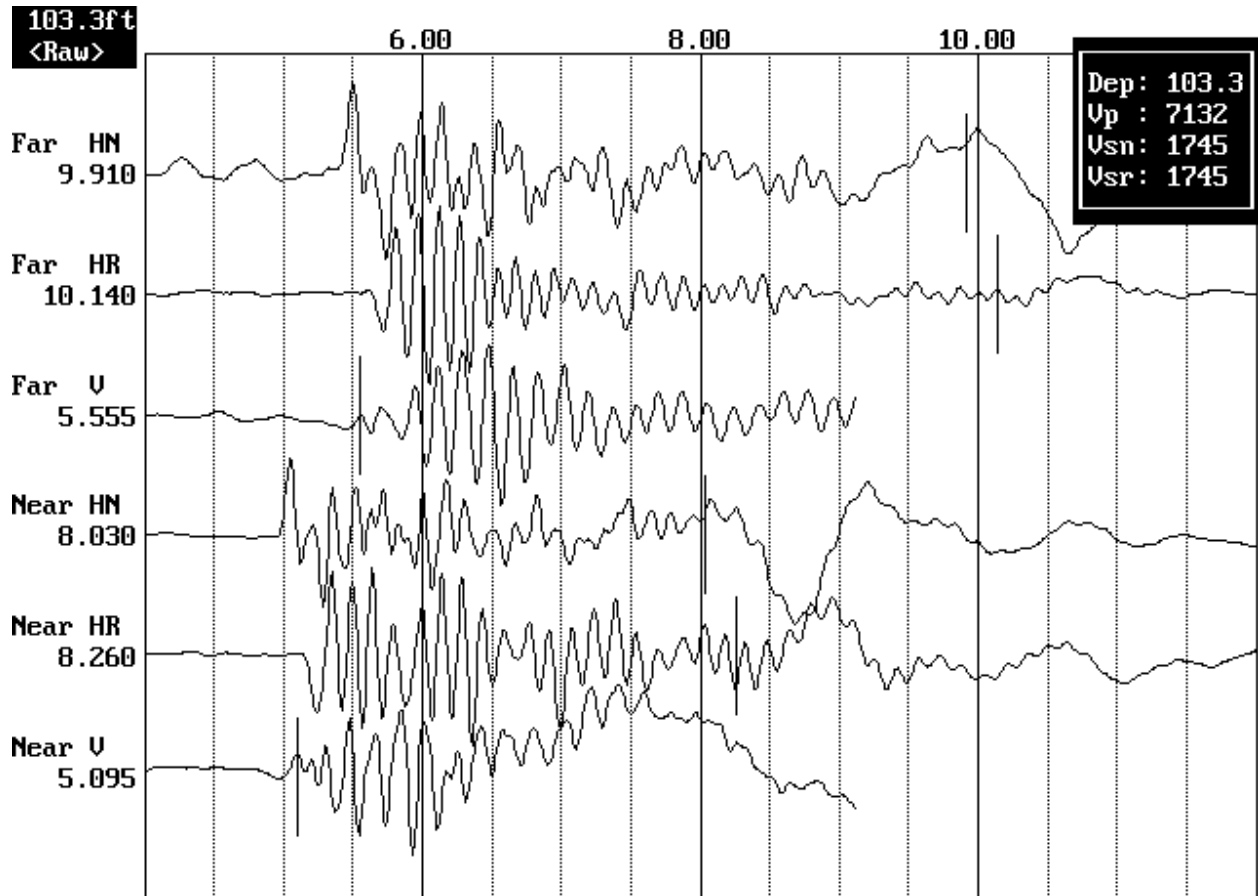


Figure 3. Example of unfiltered suspension record

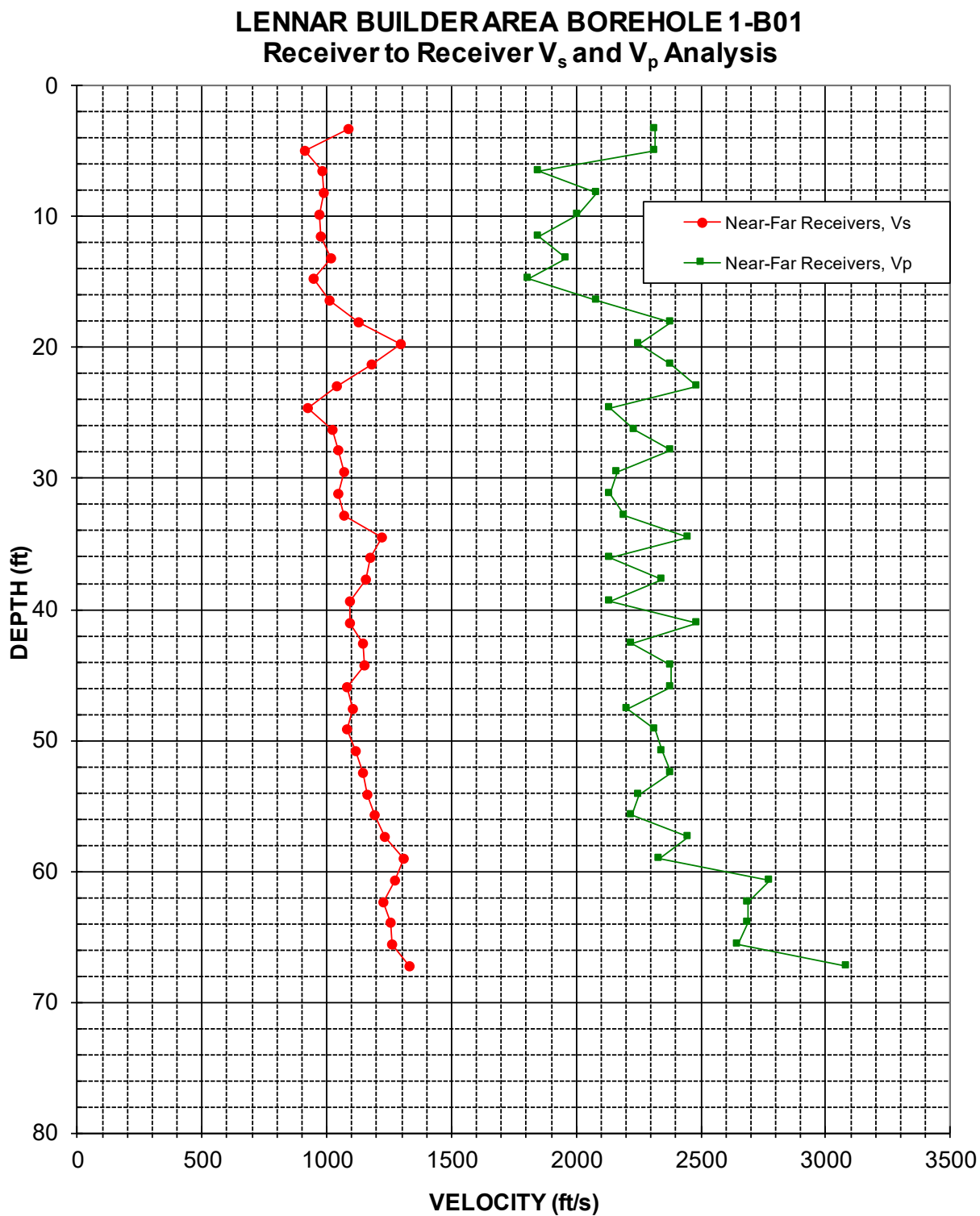


Figure 4: Borehole 1-B01, Suspension R1-R2 P- and S_H -wave velocities

Table 3. Borehole 1-B01, Suspension R1-R2 depths and P- and SH-wave velocities

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole 1-B01**

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V _s	V _p			V _s	V _p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
3.3	1090	2310	0.36	1.0	330	710	0.36
4.9	910	2310	0.41	1.5	280	710	0.41
6.6	980	1850	0.31	2.0	300	560	0.31
8.2	990	2080	0.35	2.5	300	640	0.35
9.8	970	2010	0.35	3.0	300	610	0.35
11.5	980	1850	0.31	3.5	300	560	0.31
13.1	1020	1960	0.32	4.0	310	600	0.32
14.8	940	1810	0.31	4.5	290	550	0.31
16.4	1010	2080	0.35	5.0	310	640	0.35
18.0	1130	2380	0.36	5.5	340	730	0.36
19.7	1300	2250	0.25	6.0	400	690	0.25
21.3	1180	2380	0.34	6.5	360	730	0.34
23.0	1040	2490	0.39	7.0	320	760	0.39
24.6	930	2140	0.38	7.5	280	650	0.38
26.3	1020	2240	0.37	8.0	310	680	0.37
27.9	1040	2380	0.38	8.5	320	730	0.38
29.5	1070	2160	0.34	9.0	330	660	0.34
31.2	1040	2140	0.34	9.5	320	650	0.34
32.8	1070	2190	0.34	10.0	330	670	0.34
34.5	1220	2450	0.33	10.5	370	750	0.33
36.1	1170	2140	0.28	11.0	360	650	0.28
37.7	1160	2350	0.34	11.5	350	720	0.34
39.4	1090	2140	0.32	12.0	330	650	0.32
41.0	1090	2490	0.38	12.5	330	760	0.38
42.7	1150	2220	0.32	13.0	350	680	0.32
44.3	1150	2380	0.35	13.5	350	730	0.35
45.9	1080	2380	0.37	14.0	330	730	0.37
47.6	1100	2210	0.33	14.5	340	670	0.33
49.2	1080	2310	0.36	15.0	330	710	0.36
50.9	1110	2350	0.35	15.5	340	720	0.35
52.5	1150	2380	0.35	16.0	350	730	0.35
54.1	1160	2250	0.32	16.5	350	690	0.32
55.8	1190	2220	0.30	17.0	360	680	0.30
57.4	1230	2450	0.33	17.5	380	750	0.33
59.1	1310	2330	0.27	18.0	400	710	0.27
60.7	1270	2780	0.37	18.5	390	850	0.37
62.3	1230	2690	0.37	19.0	370	820	0.37
64.0	1260	2690	0.36	19.5	380	820	0.36

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole 1-B01**

American Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V _s	V _p	
(ft)	(ft/s)	(ft/s)	
65.6	1260	2650	0.35
67.3	1330	3090	0.39

Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V _s	V _p	
(m)	(m/s)	(m/s)	
20.0	380	810	0.35
20.5	410	940	0.39

ADDENDUM #2 - APRIL 24, 2026

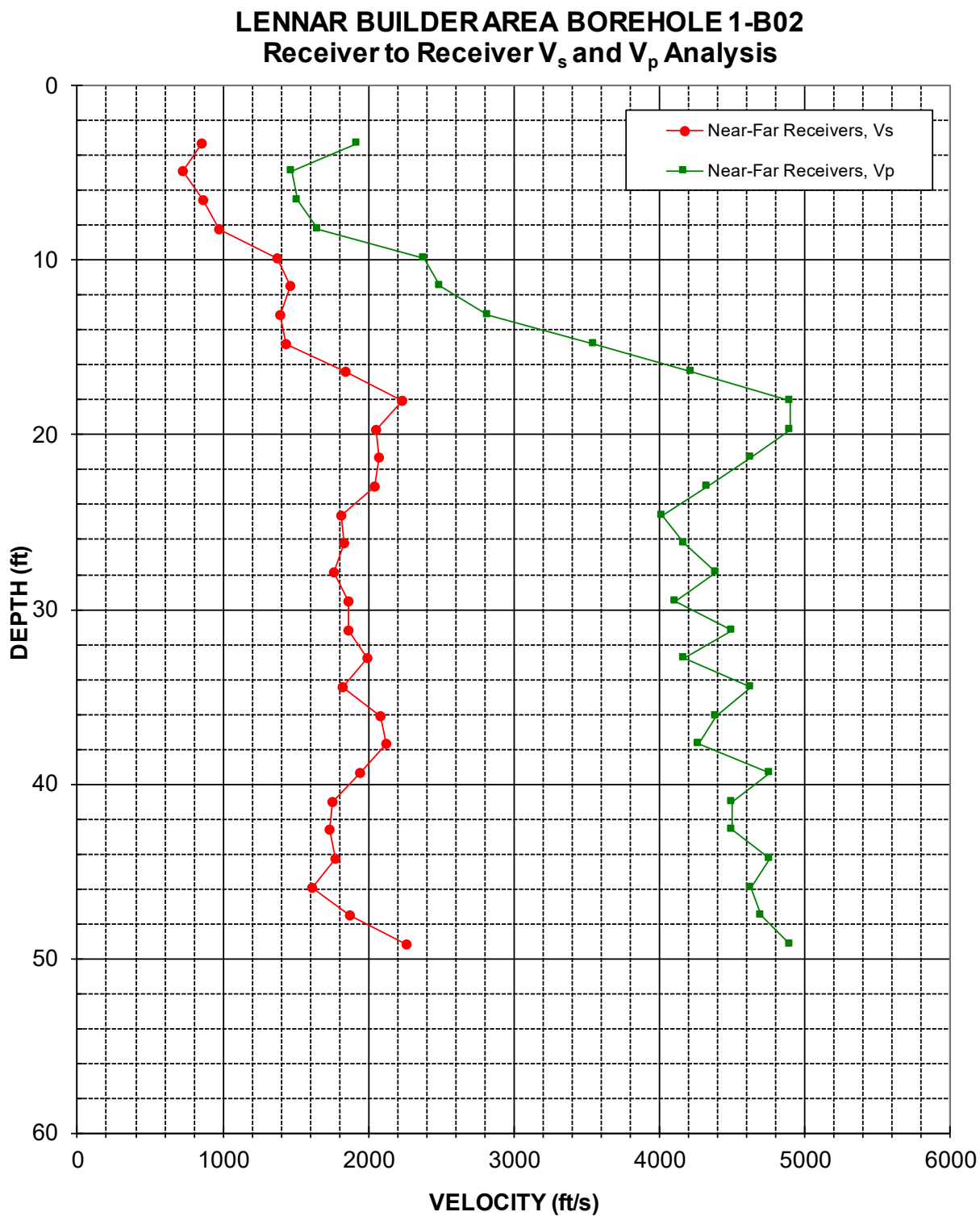


Figure 5: Borehole 1-B02, Suspension R1-R2 P- and S_H -wave velocities

Table 4. Borehole 1-B02, Suspension R1-R2 depths and P- and SH-wave velocities

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole 1-B02**

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V _s	V _p			V _s	V _p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
3.3	850	1930	0.38	1.0	260	590	0.38
4.9	730	1470	0.34	1.5	220	450	0.34
6.6	870	1520	0.26	2.0	260	460	0.26
8.2	970	1650	0.23	2.5	300	500	0.23
9.8	1380	2380	0.25	3.0	420	730	0.25
11.5	1460	2490	0.24	3.5	450	760	0.24
13.1	1390	2820	0.34	4.0	420	860	0.34
14.8	1430	3550	0.40	4.5	440	1080	0.40
16.4	1840	4220	0.38	5.0	560	1290	0.38
18.0	2240	4900	0.37	5.5	680	1490	0.37
19.7	2060	4900	0.39	6.0	630	1490	0.39
21.3	2070	4630	0.38	6.5	630	1410	0.38
23.0	2040	4330	0.36	7.0	620	1320	0.36
24.6	1810	4020	0.37	7.5	550	1220	0.37
26.3	1830	4170	0.38	8.0	560	1270	0.38
27.9	1760	4390	0.40	8.5	540	1340	0.40
29.5	1870	4120	0.37	9.0	570	1250	0.37
31.2	1860	4500	0.40	9.5	570	1370	0.40
32.8	2000	4170	0.35	10.0	610	1270	0.35
34.5	1820	4630	0.41	10.5	560	1410	0.41
36.1	2080	4390	0.35	11.0	640	1340	0.35
37.7	2120	4270	0.34	11.5	650	1300	0.34
39.4	1940	4760	0.40	12.0	590	1450	0.40
41.0	1750	4500	0.41	12.5	530	1370	0.41
42.7	1740	4500	0.41	13.0	530	1370	0.41
44.3	1770	4760	0.42	13.5	540	1450	0.42
45.9	1610	4630	0.43	14.0	490	1410	0.43
47.6	1870	4690	0.41	14.5	570	1430	0.41
49.2	2270	4900	0.36	15.0	690	1490	0.36

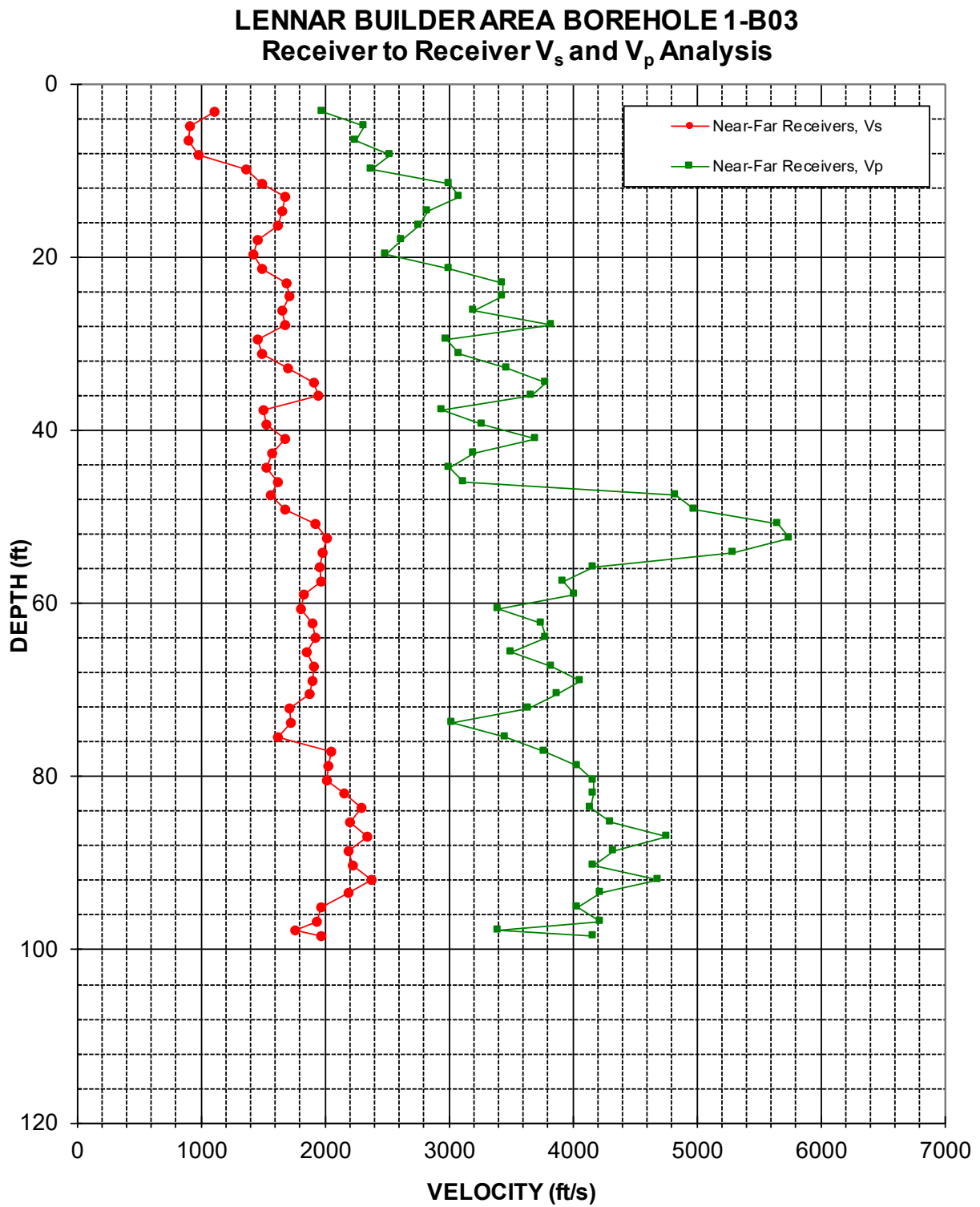


Figure 6: Borehole 1-B03, Suspension R1-R2 P- and S_H -wave velocities

Table 5. Borehole 1-B03, Suspension R1-R2 depths and P- and SH-wave velocities

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole 1-B03**

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V _s	V _p			V _s	V _p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
3.3	1110	1980	0.27	1.0	340	600	0.27
4.9	920	2310	0.41	1.5	280	710	0.41
6.6	900	2250	0.40	2.0	270	690	0.40
8.2	980	2530	0.41	2.5	300	770	0.41
9.8	1360	2380	0.26	3.0	410	730	0.26
11.5	1490	3000	0.34	3.5	450	920	0.34
13.1	1680	3090	0.29	4.0	510	940	0.29
14.8	1660	2820	0.24	4.5	510	860	0.24
16.4	1630	2750	0.23	5.0	500	840	0.23
18.0	1460	2620	0.28	5.5	440	800	0.28
19.7	1420	2490	0.26	6.0	430	760	0.26
21.3	1490	3000	0.34	6.5	460	920	0.34
23.0	1690	3440	0.34	7.0	520	1050	0.34
24.6	1720	3440	0.33	7.5	520	1050	0.33
26.3	1650	3210	0.32	8.0	500	980	0.32
27.9	1680	3830	0.38	8.5	510	1170	0.38
29.5	1460	2980	0.34	9.0	450	910	0.34
31.2	1490	3090	0.35	9.5	450	940	0.35
32.8	1700	3470	0.34	10.0	520	1060	0.34
34.5	1920	3790	0.33	10.5	580	1150	0.33
36.1	1950	3660	0.30	11.0	590	1120	0.30
37.7	1500	2950	0.33	11.5	460	900	0.33
39.4	1520	3270	0.36	12.0	460	1000	0.36
41.0	1680	3700	0.37	12.5	510	1130	0.37
42.7	1580	3210	0.34	13.0	480	980	0.34
44.3	1530	3000	0.33	13.5	470	920	0.33
45.9	1620	3120	0.32	14.0	490	950	0.32
47.6	1560	4830	0.44	14.5	470	1470	0.44
49.2	1680	4980	0.44	15.0	510	1520	0.44
50.9	1930	5650	0.43	15.5	590	1720	0.43
52.5	2020	5750	0.43	16.0	620	1750	0.43
54.1	1980	5290	0.42	16.5	600	1610	0.42
55.8	1960	4170	0.36	17.0	600	1270	0.36
57.4	1970	3920	0.33	17.5	600	1200	0.33
59.1	1830	4020	0.37	18.0	560	1220	0.37
60.7	1800	3400	0.30	18.5	550	1040	0.30
62.3	1900	3750	0.33	19.0	580	1140	0.33
64.0	1930	3790	0.33	19.5	590	1150	0.33

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Receiver-to-Receiver Travel Time Data - Borehole 1-B03**

American Units				Metric Units			
Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio	Depth at Midpoint Between Receivers	Velocity		Poisson's Ratio
	V _s	V _p			V _s	V _p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
65.6	1850	3510	0.31	20.0	560	1070	0.31
67.3	1920	3830	0.33	20.5	580	1170	0.33
68.9	1890	4070	0.36	21.0	580	1240	0.36
70.5	1870	3880	0.35	21.5	570	1180	0.35
72.2	1720	3640	0.36	22.0	520	1110	0.36
73.8	1720	3030	0.26	22.5	530	920	0.26
75.5	1630	3450	0.36	23.0	500	1050	0.36
77.1	2050	3770	0.29	23.5	630	1150	0.29
78.7	2030	4040	0.33	24.0	620	1230	0.33
80.4	2010	4170	0.35	24.5	610	1270	0.35
82.0	2150	4170	0.32	25.0	660	1270	0.32
83.7	2290	4140	0.28	25.5	700	1260	0.28
85.3	2200	4300	0.32	26.0	670	1310	0.32
86.9	2340	4760	0.34	26.5	710	1450	0.34
88.6	2190	4330	0.33	27.0	670	1320	0.33
90.2	2230	4170	0.30	27.5	680	1270	0.30
91.9	2380	4690	0.33	28.0	730	1430	0.33
93.5	2190	4220	0.32	28.5	670	1290	0.32
95.1	1970	4040	0.34	29.0	600	1230	0.34
96.8	1930	4220	0.37	29.5	590	1290	0.37
97.8	1760	3400	0.32	29.8	540	1040	0.32
98.4	1970	4170		30.0	600	1270	0.36

ADDENDUM #2 - APRIL 24, 2026

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT

QUALITY ASSURANCE SUSPENSION SOURCE

TO RECEIVER ANALYSIS RESULTS

LENNAR BUILDER AREA BOREHOLE 1-B01 Source to Receiver and Receiver to Receiver Analysis

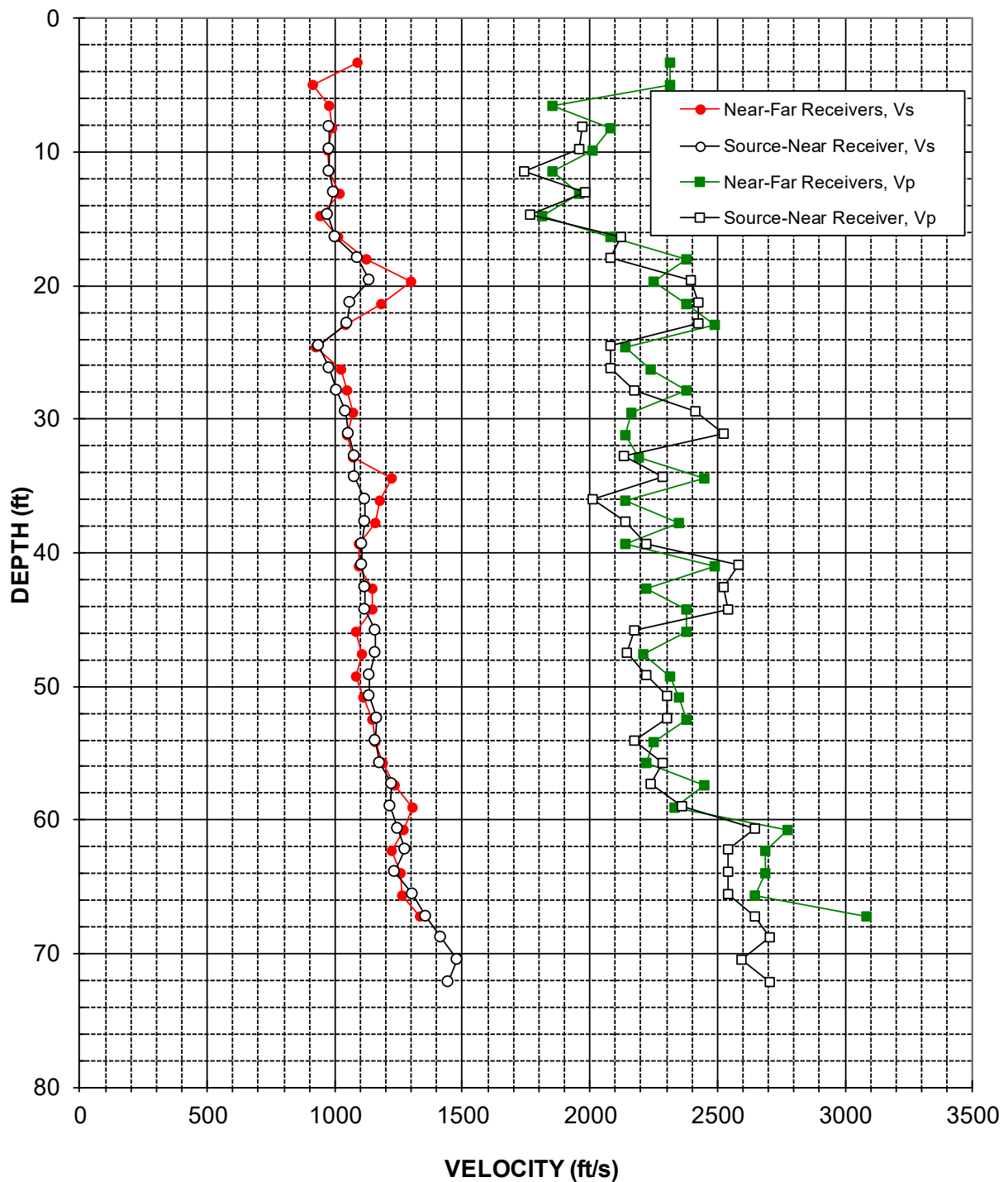


Figure A-1: Borehole 1-B01, Suspension S-R1 P- and S_H-wave velocities

Table A-1. Borehole 1-B01, S - R1 quality assurance analysis P- and S_H-wave data

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole 1-B01**

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p			V _s	V _p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
8.1	980	1970	0.34	2.5	300	600	0.34
9.8	980	1960	0.33	3.0	300	600	0.33
11.4	980	1740	0.27	3.5	300	530	0.27
13.0	990	1980	0.33	4.0	300	600	0.33
14.7	970	1760	0.28	4.5	300	540	0.28
16.3	1000	2120	0.36	5.0	300	650	0.36
18.0	1090	2080	0.31	5.5	330	630	0.31
19.6	1140	2400	0.36	6.0	350	730	0.36
21.2	1060	2430	0.38	6.5	320	740	0.38
22.9	1050	2430	0.38	7.0	320	740	0.38
24.5	940	2080	0.37	7.5	280	630	0.37
26.2	980	2080	0.36	8.0	300	630	0.36
27.8	1010	2180	0.36	8.5	310	660	0.36
29.4	1040	2420	0.39	9.0	320	740	0.39
31.1	1050	2520	0.39	9.5	320	770	0.39
32.7	1080	2130	0.33	10.0	330	650	0.33
34.4	1080	2290	0.36	10.5	330	700	0.36
36.0	1120	2010	0.28	11.0	340	610	0.28
37.6	1120	2140	0.31	11.5	340	650	0.31
39.3	1100	2220	0.34	12.0	340	680	0.34
40.9	1100	2580	0.39	12.5	340	790	0.39
42.6	1120	2520	0.38	13.0	340	770	0.38
44.2	1120	2540	0.38	13.5	340	770	0.38
45.8	1160	2180	0.30	14.0	350	660	0.30
47.5	1160	2150	0.29	14.5	350	650	0.29
49.1	1140	2220	0.32	15.0	350	680	0.32
50.8	1140	2300	0.34	15.5	350	700	0.34
52.4	1170	2300	0.33	16.0	360	700	0.33
54.0	1160	2180	0.30	16.5	350	660	0.30
55.7	1180	2290	0.32	17.0	360	700	0.32
57.3	1220	2240	0.29	17.5	370	680	0.29
59.0	1220	2360	0.32	18.0	370	720	0.32
60.6	1240	2650	0.36	18.5	380	810	0.36
62.2	1270	2540	0.33	19.0	390	770	0.33
63.9	1230	2540	0.35	19.5	380	770	0.35
65.5	1310	2540	0.32	20.0	400	770	0.32
67.2	1360	2650	0.32	20.5	410	810	0.32
68.8	1420	2710	0.31	21.0	430	820	0.31

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole 1-B01**

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p	
(ft)	(ft/s)	(ft/s)	
70.5	1480	2590	0.26
72.1	1440	2710	0.30

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p	
(m)	(m/s)	(m/s)	
21.5	450	790	0.26
22.0	440	820	0.30

ADDENDUM #2 - APRIL 24, 2026

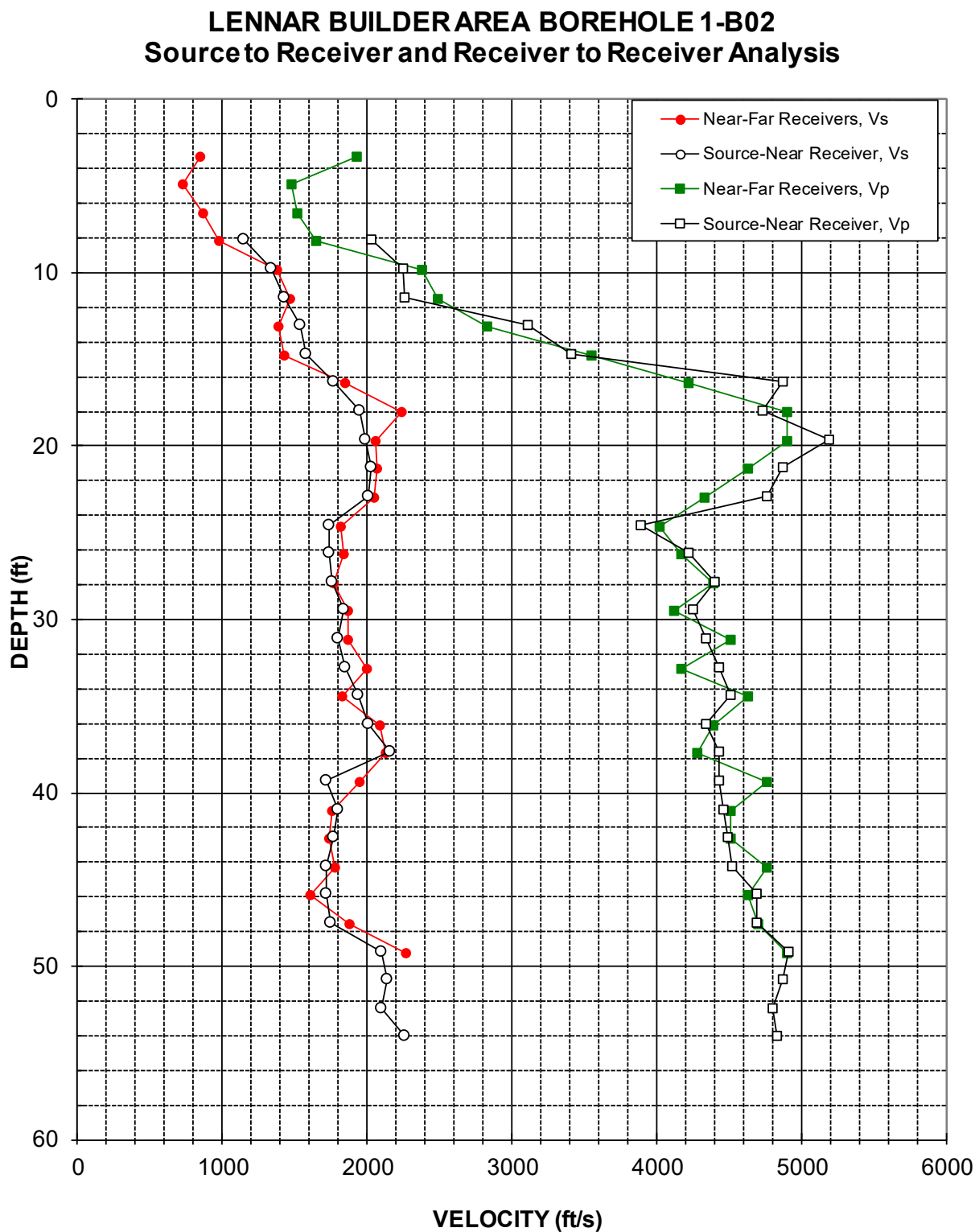


Figure A-2: Borehole 1-B02, Suspension S-R1 P- and S_H-wave velocities

Table A-2. Borehole 1-B02, S - R1 quality assurance analysis P- and S_H-wave data
**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole 1-B02**

American Units				Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio	Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p			V _s	V _p	
(ft)	(ft/s)	(ft/s)		(m)	(m/s)	(m/s)	
8.1	1150	2030	0.27	2.5	350	620	0.27
9.8	1340	2240	0.23	3.0	410	680	0.23
11.4	1420	2260	0.17	3.5	430	690	0.17
13.0	1540	3100	0.34	4.0	470	950	0.34
14.7	1570	3400	0.36	4.5	480	1040	0.36
16.3	1760	4870	0.42	5.0	540	1480	0.42
18.0	1940	4720	0.40	5.5	590	1440	0.40
19.6	1990	5190	0.41	6.0	610	1580	0.41
21.2	2030	4870	0.39	6.5	620	1480	0.39
22.9	2000	4760	0.39	7.0	610	1450	0.39
24.5	1740	3880	0.37	7.5	530	1180	0.37
26.2	1740	4220	0.40	8.0	530	1290	0.40
27.8	1760	4400	0.40	8.5	540	1340	0.40
29.4	1840	4250	0.38	9.0	560	1290	0.38
31.1	1800	4340	0.40	9.5	550	1320	0.40
32.7	1850	4430	0.39	10.0	560	1350	0.39
34.4	1940	4510	0.39	10.5	590	1370	0.39
36.0	2000	4340	0.36	11.0	610	1320	0.36
37.6	2150	4430	0.35	11.5	660	1350	0.35
39.3	1720	4430	0.41	12.0	520	1350	0.41
40.9	1800	4460	0.40	12.5	550	1360	0.40
42.6	1770	4490	0.41	13.0	540	1370	0.41
44.2	1720	4520	0.42	13.5	520	1380	0.42
45.8	1720	4690	0.42	14.0	520	1430	0.42
47.5	1750	4690	0.42	14.5	530	1430	0.42
49.1	2100	4910	0.39	15.0	640	1500	0.39
50.8	2140	4870	0.38	15.5	650	1480	0.38
52.4	2100	4800	0.38	16.0	640	1460	0.38
54.0	2260	4830	0.36	16.5	690	1470	0.36

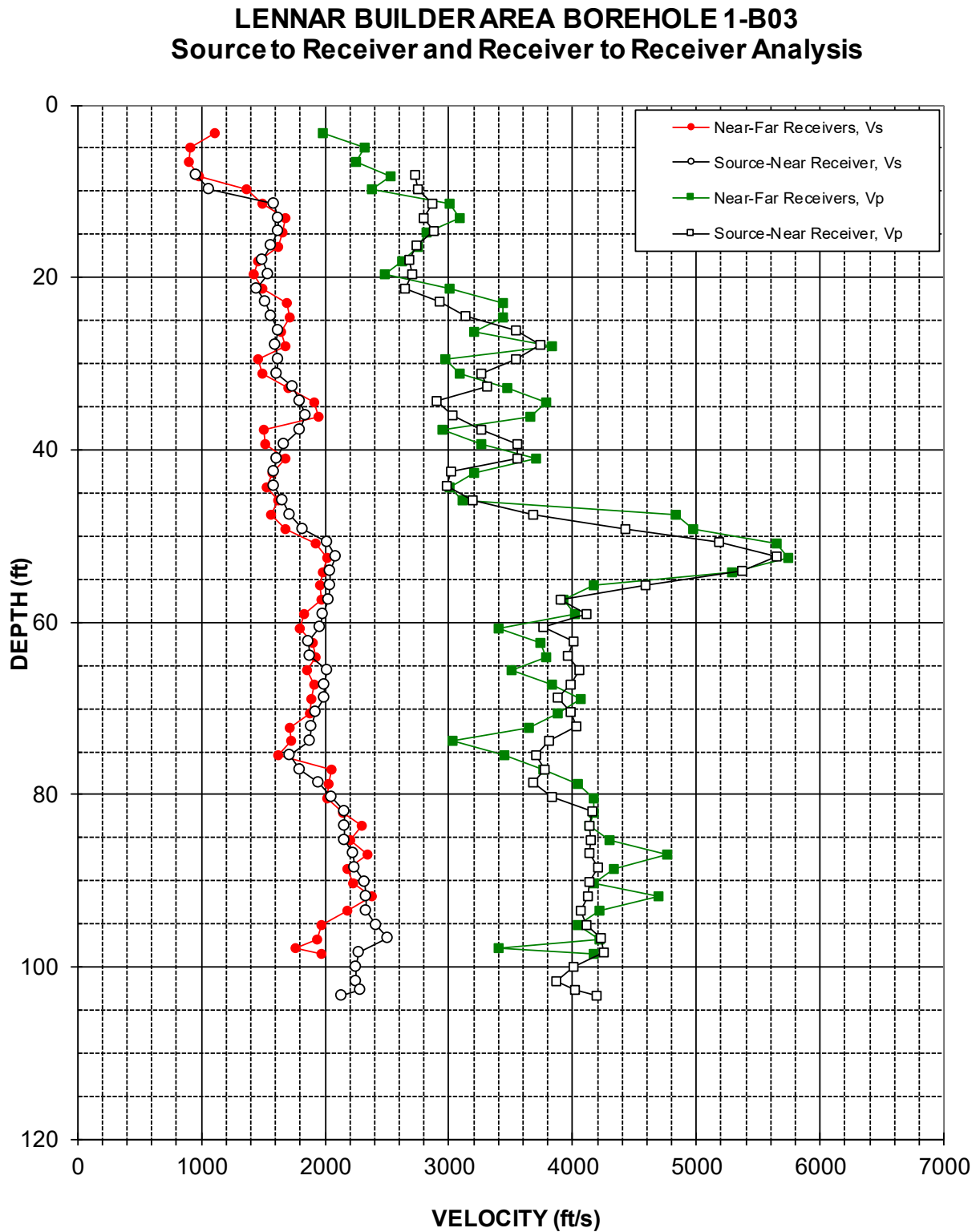


Figure A-3: Borehole 1-B03, Suspension S-R1 P- and S_H -wave velocities

Table A-3. Borehole 1-B03, S - R1 quality assurance analysis P- and S_H-wave data

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole 1-B03**

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p	
(ft)	(ft/s)	(ft/s)	
8.1	960	2730	0.43
9.8	1070	2750	0.41
11.4	1590	2860	0.28
13.0	1620	2800	0.25
14.7	1620	2880	0.27
16.3	1560	2740	0.26
18.0	1490	2680	0.28
19.6	1540	2710	0.26
21.2	1450	2650	0.29
22.9	1520	2930	0.32
24.5	1560	3130	0.34
26.2	1620	3540	0.37
27.8	1600	3750	0.39
29.4	1620	3540	0.37
31.1	1610	3260	0.34
32.7	1740	3310	0.31
34.4	1800	2900	0.19
36.0	1840	3030	0.21
37.6	1800	3260	0.28
39.3	1670	3560	0.36
40.9	1610	3560	0.37
42.6	1580	3010	0.31
44.2	1590	2990	0.30
45.8	1660	3200	0.32
47.5	1720	3680	0.36
49.1	1820	4430	0.40
50.8	2020	5190	0.41
52.4	2080	5650	0.42
54.0	2040	5360	0.42
55.7	2040	4590	0.38
57.3	2030	3910	0.32
59.0	1980	4110	0.35
60.6	1950	3770	0.32
62.2	1860	4010	0.36
63.9	1870	3960	0.36
65.5	2020	4060	0.34
67.2	1990	3980	0.33
68.8	1990	3880	0.32

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p	
(m)	(m/s)	(m/s)	
2.5	290	830	0.43
3.0	320	840	0.41
3.5	480	870	0.28
4.0	490	850	0.25
4.5	490	880	0.27
5.0	480	840	0.26
5.5	460	820	0.28
6.0	470	820	0.26
6.5	440	810	0.29
7.0	460	890	0.32
7.5	480	960	0.34
8.0	490	1080	0.37
8.5	490	1140	0.39
9.0	490	1080	0.37
9.5	490	990	0.34
10.0	530	1010	0.31
10.5	550	890	0.19
11.0	560	920	0.21
11.5	550	990	0.28
12.0	510	1080	0.36
12.5	490	1080	0.37
13.0	480	920	0.31
13.5	480	910	0.30
14.0	510	970	0.32
14.5	520	1120	0.36
15.0	550	1350	0.40
15.5	610	1580	0.41
16.0	630	1720	0.42
16.5	620	1640	0.42
17.0	620	1400	0.38
17.5	620	1190	0.32
18.0	600	1250	0.35
18.5	600	1150	0.32
19.0	570	1220	0.36
19.5	570	1210	0.36
20.0	610	1240	0.34
20.5	610	1210	0.33
21.0	610	1180	0.32

**Summary of Compressional Wave Velocity, Shear Wave Velocity, and Poisson's Ratio
Based on Source-to-Receiver Travel Time Data - Borehole 1-B03**

American Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p	
(ft)	(ft/s)	(ft/s)	
70.5	1930	3980	0.35
72.1	1880	4030	0.36
73.7	1870	3810	0.34
75.4	1710	3700	0.36
77.0	1790	3780	0.35
78.7	1940	3680	0.31
80.3	2050	3840	0.30
81.9	2160	4160	0.32
83.6	2150	4140	0.31
85.2	2150	4150	0.32
86.9	2230	4140	0.30
88.5	2240	4210	0.30
90.1	2320	4140	0.27
91.8	2330	4120	0.27
93.4	2330	4070	0.26
95.1	2420	4110	0.24
96.7	2510	4230	0.23
98.3	2270	4250	0.30
100.0	2250	4010	0.27
101.6	2250	3870	0.24
102.6	2290	4020	0.26
103.3	2130	4190	0.33

Metric Units			
Depth at Midpoint Between Source and Near Receiver	Velocity		Poisson's Ratio
	V _s	V _p	
(m)	(m/s)	(m/s)	
21.5	590	1210	0.35
22.0	570	1230	0.36
22.5	570	1160	0.34
23.0	520	1130	0.36
23.5	550	1150	0.35
24.0	590	1120	0.31
24.5	620	1170	0.30
25.0	660	1270	0.32
25.5	660	1260	0.31
26.0	660	1270	0.32
26.5	680	1260	0.30
27.0	680	1280	0.30
27.5	710	1260	0.27
28.0	710	1260	0.27
28.5	710	1240	0.26
29.0	740	1250	0.24
29.5	770	1290	0.23
30.0	690	1290	0.30
30.5	690	1220	0.27
31.0	690	1180	0.24
31.3	700	1230	0.26
31.5	650	1280	0.33

ADDENDUM #2 - APRIL 24, 2026

APPENDIX B

BOREHOLE GEOPHYSICAL LOGGING

SYSTEMS - NIST TRACEABLE

CALIBRATION RECORDS



MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659



Certificate of Calibration

Date: May 28, 2019

Cert No. 551220083036990

Customer:

GEOVISION

1124 OLYMPIC DRIVE
CORONA CA 92881

MPC Control #: AM6767
Asset ID: 160023
Gage Type: LOGGER
Manufacturer: OYO
Model Number: 3403
Size: N/A
Temp/RH: 22.5°C / 42.9%
Location: Calibration performed at MPC facility

Work Order #: LA-90043197
Purchase Order #: 19160-190520-01
Serial Number: 160023
Department: N/A
Performed By: TYLER MCKEEN
Received Condition: IN TOLERANCE
Returned Condition: IN TOLERANCE
Cal. Date: May 24, 2019
Cal. Interval: 12 MONTHS
Cal. Due Date: May 24, 2020

Calibration Notes:

See attached data sheet for calculations. (1 Page)

Calibrated IAW customer supplied data form Rev 2.1

Frequency measurement uncertainty = 0.0005 Hz

Unit calibrated with Laptop Panasonic Model CF-29,s/n: 6AKSB99869 and RG Micrologger II Serial No. 5772

Calibrated To 4:1 Accuracy Ratio

Calibration performed in accordance with approved GEOVision calibration procedures included in work Instruction No. 06
Software: ML PS 4.00 Suspension Logger, GVLog.jar (2004) and pslog.exe ver 1.00 software.

Standards Used to Calibrate Equipment

I.D.	Description.	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
DB8748	GPS TIME AND FREQUENCY RECEIVER	58503A	3625A01225	HEWLETT PACKARD	Apr 30, 2021	551220083021224
LAS0018	ARB / FUNC GENERATOR	33250A	US40001522	AGILENT	Apr 30, 2020	551220083009506
BD7715	UNIVERSAL COUNTER	53131A	3416A05377	HEWLETT PACKARD	Apr 30, 2020	551220082934517

Calibrating Technician:

TYLER MCKEEN

QC Approval:

ILYA VAKS

Statements of Pass or Fail Conformance: The uncertainty of measurement has been taken into account when determining compliance with specification, as per ILAC-G8:03/2009. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCSL Z540.3-2006.

The status of compliance with the acceptance criteria is reported as:

PASS - Compliant with specification.

FAIL - Not compliant with specification.

FAIL² - The measured value is not within the acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% is within the specified tolerance.

PASS² - The measured value is within acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% exceeds the specified tolerance.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor k=2, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCSL Z540.3 Method 6-Guaranteed Bands based on Test Uncertainty Ratio. Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. The information on this report pertains only to the instrument identified, this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.



MICRO PRECISION CALIBRATION, INC
2165 N. Glassell St.,
Orange, CA 92865
714-901-5659



Certificate of Calibration

Date: May 28, 2019

Cert No. 551220083036990

Procedures Used in this Event

Procedure Name
GEOVISION SEISMIC Rev. 2.1

Description
Seismic Logger/Recorder Calibration Procedure, Rev. 2.1

Calibrating Technician:

TYLER MCKEEN

QC Approval:

ILYA VAKS

Statements of Pass or Fail Conformance: The uncertainty of measurement has been taken into account when determining compliance with specification, as per ILAC-G8:03/2009. All measurements and test results guard banded to ensure the probability of false-accept does not exceed 2% in compliance with ANSI/NCCL Z540.3-2006.

The status of compliance with the acceptance criteria is reported as:

PASS - Compliant with specification.

FAIL - Not compliant with specification.

FAIL² - The measured value is not within the acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% is within the specified tolerance.

PASS² - The measured value is within acceptance limits. However, a portion of the expanded uncertainty of measurement at 95% exceeds the specified tolerance.

The expanded uncertainty of measurement is stated as the standard uncertainty of measurement multiplied by the coverage factor $k=2$, which for a normal distribution corresponds to a coverage probability of approximately 95%, unless otherwise stated. This calibration report complies with ISO/IEC 17025:2017 and ANSI/NCCL Z540.3 Method B-Guard Bands based on Test Uncertainty Ratio. Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. All standards are traceable to SI through the National Institute of Standards and Technology (NIST) and/or recognized national or international standards laboratories. Services rendered include proper manufacturer's service instruction and are warranted for no less than thirty (30) days. The information on this report pertains only to the instrument identified; this may not be reproduced in part or in a whole without the prior written approval of the issuing MP Calibration Laboratory.



SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.: <u>OYO</u>	Model no.: <u>3403</u>
Serial no.: <u>160023</u>	Calibration date: <u>5/24/2019</u>
By: <u>Micro Precision</u>	Due date: <u>5/24/2020</u>
Counter mfg.: <u>Hewlett Packard</u>	Model no.: <u>53131A</u>
Serial no.: <u>3416A05377</u>	Calibration date: <u>4/02/2019</u>
By: <u>Micro Precision</u>	Due date: <u>4/30/2020</u>
Signal generator mfg.: <u>Agilent</u>	Model no.: <u>33250A</u>
Serial no.: <u>11540001522</u>	Calibration date: <u>4/03/2019</u>
By: <u>Micro Precision</u>	Due date: <u>4/30/2020</u>
Laptop controller mfg.: <u>Panasonic</u>	Model no.: <u>CF-29</u>
Serial no.: <u>6AKSB 99869</u>	Calibration date: <u>N/A</u>

SYSTEM SETTINGS:

Gain: 0

Filter: Low Pass 1k

Range: 5-200

Delay: 0

Stack (1 std): 1

System date = correct date and time: ✓ 11:17 AM 5/24/2019

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak

Note actual frequency on data form.

Set sample period and record data file to disk. Note file name on data form. Acquired using ML PS 4.00

Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found EF 5/30/19 0.11% 0.12% As left EF 5/30/19 0.11% 0.12%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	001	179.8	50.06	179.8	50.06	179.8	50.06
100.0	100.0	100	002	89.90	100.1	90.00	100.0	89.90	100.1
200.0	200.0	50	003	45.00	200.0	44.95	200.2	45.00	200.0
500.0	500.0	20	004	18.00	500.0	17.98	500.6	17.98	500.6
1000	1000	10	005	9.000	1000	9.000	1000	9.000	1000.0
2000	2000	5	006	4.500	2000	4.505	1998	4.500	2000

Calibrated by: Tyler McKee 5/24/19 [Signature]

Name Date Signature

Witnessed by: Emily Feldman 5/24/19 [Signature]

Name Date Signature

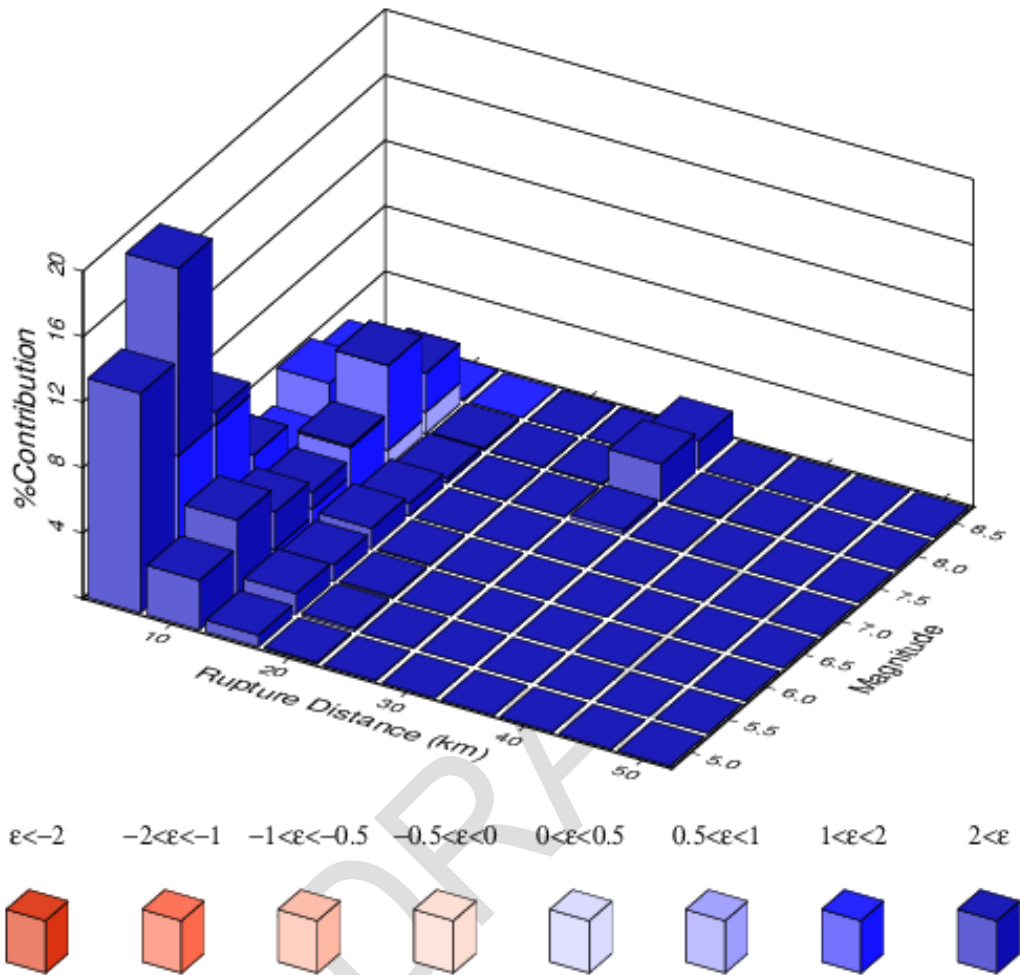
Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.1 February 7, 2012



APPENDIX D

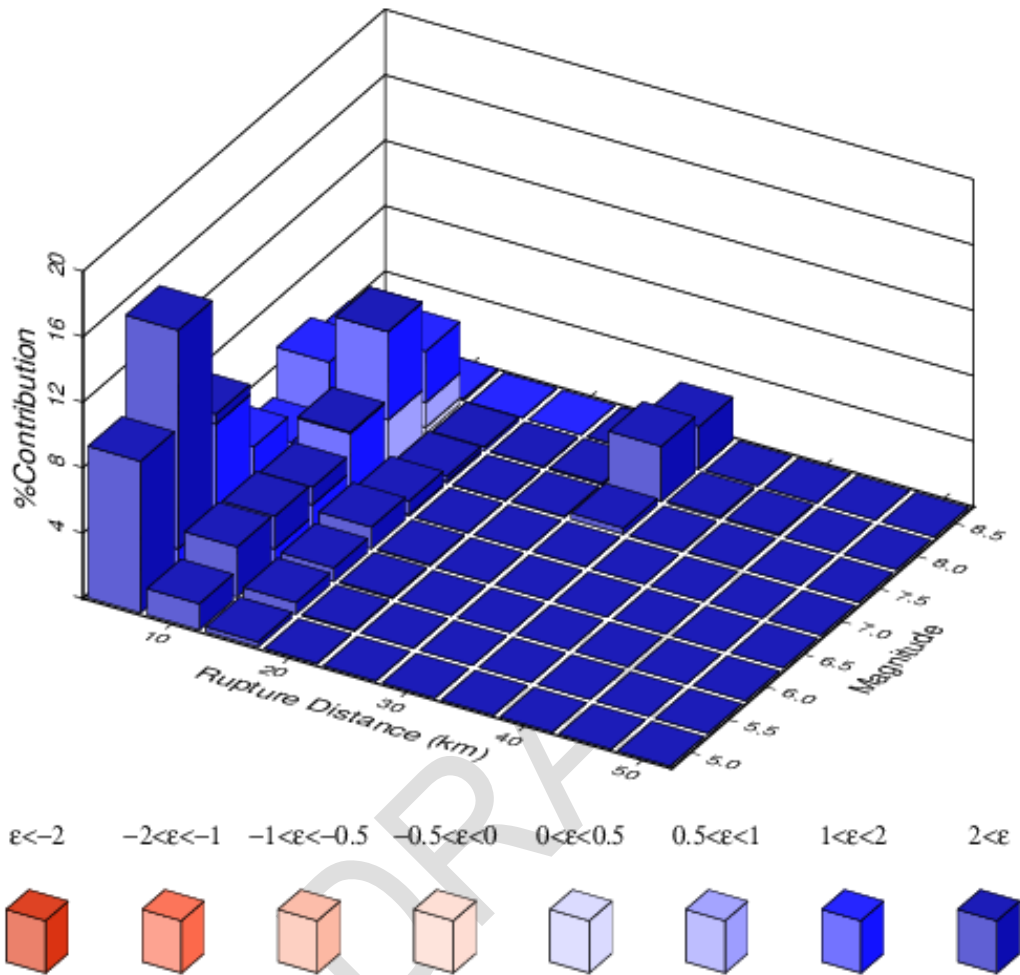
DISAGGREGATION RESULTS

EXHIBIT 1: Disaggregation Results for a 2,475-Year Return Period at a 0.1 Second Period



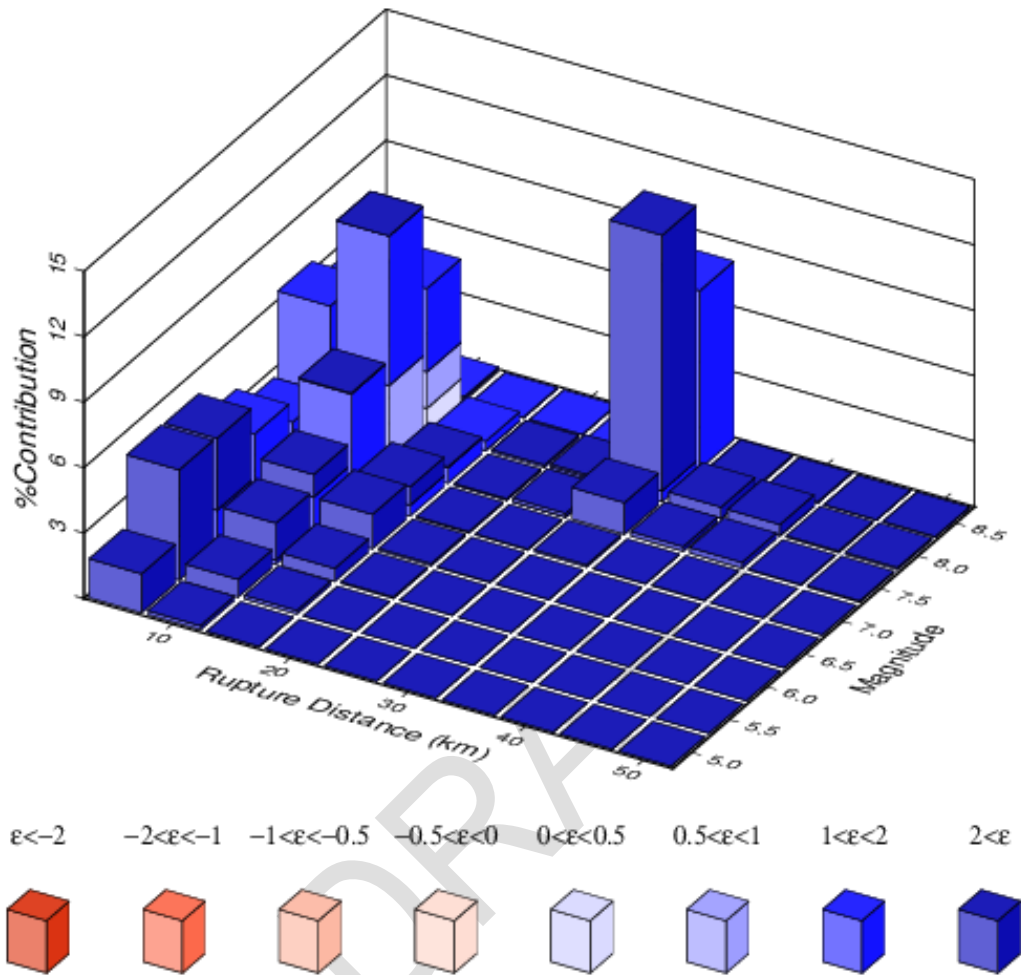
ADDENDUM #2 - APRIL 24, 2026

EXHIBIT 2: Disaggregation Results for a 2,475-Year Return Period at a 0.2 Second Period



ADDENDUM #2 - APRIL 24, 2026

EXHIBIT 3: Disaggregation Results for a 2,475-Year Return Period at a 0.5 Second Period



ADDENDUM #2 - APRIL 24, 2026

