

REVIEWED FOR CODE COMPLIANCE

THESE PLANS AND DOCUMENTS HAVE BEEN REVIEWED AND FOUND TO BE IN COMPLIANCE WITH THE APPLICABLE CODE REQUIREMENTS OF THE JURISDICTION. ISSUANCE OF A PERMIT IS RECOMMENDED, SUBJECT TO APPROVAL BY OTHER DEPARTMENTS AND ANY NOTED CONDITIONS. THE STAMPING OF THESE PLANS SHALL NOT BE HELD TO PERMIT OR BE AN APPROVAL OF ANY VIOLATION OF APPLICABLE CODES AND STANDARDS, NOR RELIEVE THE OWNER, DESIGN PROFESSIONAL OF RECORD, OR CONTRACTOR OF COMPLIANCE WITH THE APPLICABLE CODES AND STANDARDS. PLAN REVIEW OF DOCUMENTS DOES NOT AUTHORIZE CONSTRUCTION TO PROCEED IN VIOLATION OF ANY FEDERAL, STATE, OR LOCAL REGULATION.

BUILDING BUREAU SOLUTIONS, INC.
REVIEWED BY: STEVE KIM, PE, CAsp

THIS SET OF THE PLANS AND SPECIFICATIONS MUST BE KEPT ON THE JOB SITE AT ALL TIMES AND IT IS UNLAWFUL TO MAKE ANY CHANGES OR ALTERATIONS WITHOUT PERMISSION FROM THE CITY. OCCUPANCY OF STRUCTURE(S) IS NOT PERMITTED UNTIL FINAL APPROVAL IS GRANTED BY ALL APPLICABLE DEPARTMENTS.

BBS
BUILDING BUREAU
SOLUTIONS INC.

Appendix “A”

GEOTECHNICAL INVESTIGATION

September 23, 2025
Project No. S168-196

STK Architecture, Inc.
42095 Zeno Drive, Suite A15
Temecula, California 92590

Attention: Tony Finaldi

Subject: Geotechnical Investigation
South Perris Fire Station
East Side of Murrieta Road, North of Watson Road
Perris, California

Dear Mr. Finaldi:

This report presents the results of the geotechnical investigation for the proposed South Perris fire station. The investigation was conducted in general conformance with our proposal dated February 21, 2025.

This report includes project design and construction recommendations along with the field and laboratory data. The primary geotechnical issue is the presence of expansive soil. Non-expansive import soil is recommended to mitigate the potential for soil expansion.

We appreciate the opportunity to work with you on this project. Please call if you have any questions or need any other information.

Sincerely,
INLAND FOUNDATION ENGINEERING, INC.


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

ADE:sd

TABLE OF CONTENTS

INTRODUCTION1

SCOPE OF SERVICE.....1

PROJECT DESCRIPTION.....1

SITE DESCRIPTION.....2

GEOLOGIC HAZARD EVALUATION.....3

SUBSURFACE CONDITIONS4

INFILTRATION TESTING5

CONCLUSIONS AND RECOMMENDATIONS.....6

 Foundation Design6

 Lateral Resistance6

 Lateral Earth Pressure6

 Concrete Slabs-on-Grade7

 Portland Cement Concrete (PCC) Pavement7

 Asphalt Concrete Pavement.....8

 General Site Grading9

LIMITATIONS11

REFERENCES12

APPENDICES

APPENDIX A - Site Exploration A-1 - A-9

 Explanation of Logs..... A-2

 Exploratory Borings and Trenches A-3 - A-8

 Site Plan A-9

APPENDIX B - Laboratory Testing..... B-1 - B-9

 Sieve Analysis..... B-3 - B-4

 Plastic Index..... B-3 - B-4

 Maximum Density - Optimum Moisture..... B-5

 Consolidation B-6

 Direct Shear Strength..... B-7 - B-8

 Corrosion B-9

 R-value B-10

APPENDIX C - Infiltration Testing C-1 - C-5

APPENDIX D - Geologic Hazard Report D

LIST OF FIGURES

- Figure 1 – USGS Topographical Map.....2
- Figure 2 – Site Photographs.....3

LIST OF TABLES

- Table 1 – Seismic Design Parameters.....4
- Table 2 – Infiltration Rates.....6
- Table 3 – Portland Cement Concrete Pavement8
- Table 4 – Asphalt Concrete Pavement9
- Table 5 – Recommended Import Soil Criteria10

Introduction

This report presents the results of the geotechnical investigation conducted for the proposed South Perris fire station. The proposed fire station will be located on a 3.4-acre site north of Watson Road on the east side of Murrieta Road in Perris, California. The following documents were used as references for this investigation.

- Request for Proposal, Architectural Design Services for the South Perris Fire Station Project CIP F077, prepared by the City of Perris, dated January 31, 2025
- Undated Site Plan, prepared by STK Architecture, Inc.

Scope of Service

The purpose of this geotechnical investigation was to provide geotechnical parameters for design and construction of the proposed fire station. The scope of the geotechnical services included:

- *Evaluation of existing geologic conditions at the site and review of potential geologic and seismic hazards.*
- *Evaluation of the local and regional tectonic setting and historical seismic activity, including a site-specific ground motion analysis.*
- *Reconnaissance of the site and surrounding area to ascertain the presence of unstable or adverse geologic conditions.*
- *Subsurface sampling and laboratory testing.*
- *Analysis of the data collected and the preparation of this report with geotechnical engineering conclusions and recommendations for design and construction.*

Evaluation of hazardous waste was not within the scope of services provided.

Project Description

The South Perris fire station will be a single-story wood-frame structure occupying approximately 10,900 square feet. It will be constructed in the south portion of the project site and will include three drive-through vehicle bays for storage of fire apparatus vehicles, living quarters for up to 10 personnel, locker rooms, kitchen area, a fitness room, laundry room, conference room, and offices.

An administration building will be constructed in the north portion of the site that will occupy approximately 5,200 square feet. It will also be a single-story wood-frame structure that will include offices, conference room, training room, break room and restrooms.

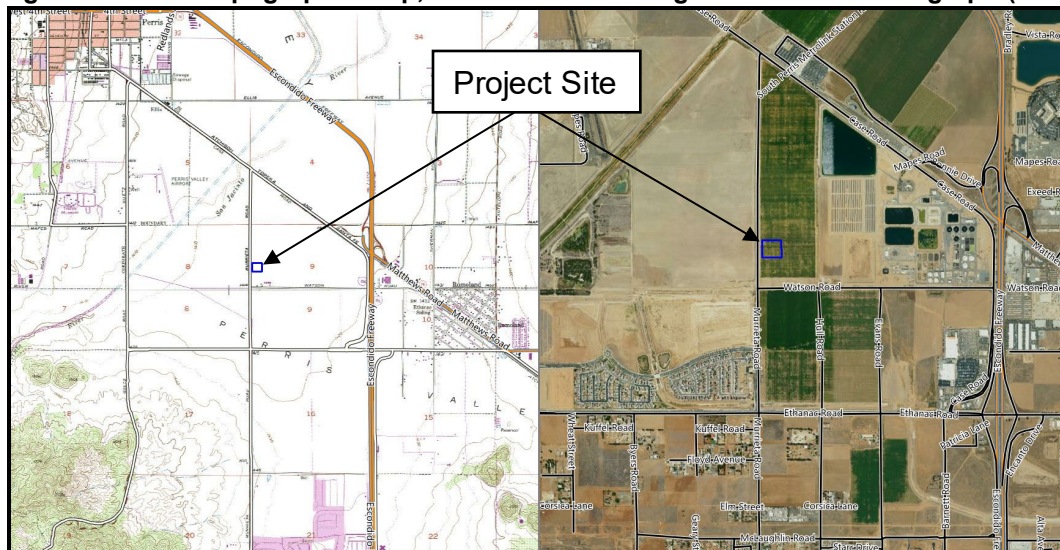
Foundations for the proposed structures are expected to consist of shallow continuous and isolated concrete spread footings with slab-on-grade floors. Site grading is expected to consist of preparation of a building pad for the proposed structures as high as four feet above existing site grades.

Two stormwater bioretention basins are planned on the west side of the site. The basin depths are expected to be no deeper than five feet below existing surface grades. Additional onsite improvements will also include driving lanes and parking areas, storage buildings, trash enclosure, transformer pad, and battery storage area. Off-site improvements to Murrieta Road will also be required.

Site Description

The subject site is located on the east side of Murrieta Road, approximately 640 feet north of Watson Road in Perris, California (33.75263°, -117.20584°), and occupies 3.4 acres. Figure 1 below shows the site location.

Figure 1: USGS Topographic Map, Perris 7.5' Quadrangle and Aerial Photograph (2022)



The site elevation is approximately 1,415 feet above mean sea level (msl). The site is relatively flat. The immediate site vicinity slopes to the west at an overall rate of less than 1 percent. Current site vegetation consists of a sparse growth of alfalfa. Figure 2 below shows the current site conditions.

Figure 2: View of Project Site Toward South



Geologic Hazards Evaluation

A geologic hazards report for this project was prepared by our subconsultant, Terra Geosciences, and is appended. The engineering geology and seismicity review was performed using the suggested “Checklist for the Review of Geologic/Seismic Reports for California Public Schools, Hospitals and Essential Services Buildings” (California Geologic Survey, Note 48, 2022).

The geologic hazards study indicates that the proposed fire station and associated structures are considered feasible from a geologic standpoint, providing that the conclusions and recommendations presented in the report are considered during planning and construction. No adverse geologic conditions were found within the proposed construction area, with the exception of the potential for strong ground shaking from nearby seismogenic fault sources.

The geologic hazards study included a site-specific ground motion analysis. The mapped spectral acceleration parameters, coefficients, and other related seismic parameters were evaluated using the OSHPD Seismic Design Maps web application (OSHPD, 2020) and the California Building Code criteria (CBC, 2022), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (2017). The results of the site-specific analysis are summarized and tabulated in Table 1 below:

Table 1: Seismic Design Parameters

Factor or Coefficient	Value
S_s	1.422g
S_1	0.526g
F_a	1.2g
F_v	1.774g
S_{DS}	0.990g
S_{D1}	0.700g
S_{MS}	1.480g
S_{M1}	1.052g
T_L	8 Seconds
MCE_G PGA	0.64g
Site Class	D

Subsurface Conditions

Subsurface exploration at the site consisted of six (6) exploratory borings to depths ranging from approximately 17.0 to 51.5 feet below existing site grades. The site exploration is described in Appendix A. Boring locations are shown on Figure A-9.

The soil encountered in the borings consisted of interlayered alluvial deposits comprised of clayey sand (SC), sandy clay (CL), silty clayey sand (SC-SM), silty sand (SM), and sand with silt (SP-SM). The soil encountered was generally very loose to loose within the upper 2 feet. Below 2 feet, the coarse-grained soil encountered was generally medium dense to dense. The fine-grained soil encountered was stiff to hard. The soil was moist to very moist at the time of investigation.

Corrosion Potential: Analytical testing indicates the concentration of sulfates is 32 ppm. In accordance with ACI 318, Table 4.2.1, the soil is classified as having a negligible sulfate exposure. The chloride concentration in the tested sample was 28 ppm and indicates that the soil is generally not corrosive with respect to ferrous metal. The soil is alkaline with a pH value of 8.4. The saturated minimum resistivity value of 1,552 ohm-cm indicates the soil may be corrosive to buried ferrous metal. Alternative material such as PVC piping should be considered. Inland Foundation Engineering, Inc. does not practice corrosion engineering. A qualified corrosion engineer should be consulted for additional guidance.

Hydrocollapse Potential: Consolidation testing indicates that the soil is compressible and normally consolidated. The results show a slight potential for expansion when

saturated under anticipated foundation and soil overburden loads. Provided that the building pad area is prepared as recommended herein, and appropriate surface drainage is provided in accordance with contemporary design practice, the potential for adverse building settlement due to hydrocollapse is not significant.

Expansive Soil: Laboratory testing indicates an expansion index of 69, representing a medium expansion class. The California Building Code (CBC) requires that slabs-on-grade be designed for soil expansion if constructed on soil with an expansion index higher than 20. Conventional slabs-on-grade may be utilized but should be supported by at least 4.0 feet of imported non-expansive soil. Final recommendations to mitigate expansive soil should be made during precise grading when actual soil types are known.

Groundwater: Groundwater was encountered in borings B-02 and B-05 at depths of approximately 40.5 and 42.0 feet, respectively. Based on a review of pertinent groundwater data (referenced in appended geologic hazards report), the documented depth to groundwater in local wells is at least 49 feet. The mottled soil encountered at a depth of 35 feet in boring B-02 likely represents the historic high groundwater depth at the site.

Liquefaction and Seismically-Induced Settlement: In general, liquefaction is a phenomenon that occurs where there is a loss of strength or stiffness in the soil that can result in the settlement of buildings, ground failure, or other hazards. The main factors contributing to this phenomenon are: 1) cohesionless, granular soil with relatively low density (usually of Holocene age); 2) shallow ground water (generally less than 50 feet); and 3) moderate to high seismic ground shaking. Based on the medium dense to dense conditions encountered in the borings, the potential for soil liquefaction is not significant.

“Dry sand” settlement occurs in loose granular soil as a result of seismic ground shaking. The potential for “dry sand” settlement also is not significant, based on the medium dense to dense conditions encountered in the borings.

Infiltration Testing

Infiltration testing was conducted in general accordance with Appendix A of the Riverside County Low Impact Development BMP Design Handbook (2011). Four percolation tests were performed at the locations shown on Figure A-9. The testing procedures and the test data are described and included in Appendix C of this report.

The test results are shown in Table 2. The corresponding calculated infiltration rate (I_c) ranges from 0.3 inches per hour to 1.2 inches per hour. These values exclude a factor of safety. The appropriate factor of safety should be determined by the design engineer.

Table 2: Infiltration Rate

Percolation Test No.	Percolation Rate (min/in)	Depth Below Ground Surface (in)	Infiltration Rate (I_c) (in/hr)
I-01	4.0	60	1.2
I-02	17.1	48	0.3
I-03	17.1	60	0.3
I-04	17.1	48	0.3

Conclusions and Recommendations

The primary geotechnical issue that will require mitigation is the presence of expansive soil within the building pad area. At least 4 feet of imported non-expansive fill soil will be required below conventional slabs-on-grade to mitigate soil expansion. This and other geotechnical engineering recommendations for project design and construction are presented below.

Foundation Design: The proposed fire station and associated structures can be supported by shallow continuous and isolated spread footings designed with an allowable bearing pressure of 2,000 pounds per square foot (psf). Footings should have a minimum width of 12 inches and bottoms a minimum depth of 12 inches below the lowest adjacent grade. The allowable bearing pressure can be increased by 200 psf for each additional foot of width and by 500 psf for each additional foot of depth, to a maximum allowable bearing pressure of 3,000 psf. The allowable bearing pressure can be further increased by $\frac{1}{3}$ for short-term transient wind and seismic loads.

Static settlement of footings designed and constructed as recommended herein is expected to be less than one inch. Differential settlement between footings of similar size and load is expected to be less than one-half inch.

Lateral Resistance: Resistance to lateral loads will be provided by a combination of friction acting at the base of the slab or foundation and passive earth pressure. A coefficient of friction of 0.35 between soil and concrete may be used with dead load forces only. A passive earth pressure of 200 psf/ft may be used for the sides of footings poured against recompacted or dense native material. These values may be increased by $\frac{1}{3}$ for short-term transient wind and seismic loads. Passive earth pressure should be ignored within the upper one foot, except where confined as beneath a floor slab, for example.

Lateral Earth Pressure: Retaining walls should be designed for an active earth pressure equivalent to that exerted by a fluid weighing not less than 43 pcf. Any applicable construction or seismic surcharges should be added to this pressure. Retaining wall backfill should have an expansion index of less than 20.

Concrete Slabs-on-Grade: Potentially expansive soil is present throughout the project site. The California Building Code (CBC) requires that slab-on-grade foundations on expansive soils be designed in accordance with *WRI/CRSI Design of Slab-on-Ground Foundations (1981)* or *PTI Standard Requirements for Analysis of Shallow Concrete Foundations on Expansive Soils (2012)*. Conventional slabs-on-grade may be utilized but should be supported by at least four feet of imported non-expansive soil. Development of WRI/CRSI or PTI design parameters was beyond the scope of this investigation. This firm should be contacted if WRI/CRSI or PTI recommendations are required.

Concrete slabs-on-grade should have a minimum thickness of four inches. During final grading and prior to the placement of concrete, all surfaces to receive concrete slabs-on-grade should be compacted to maintain a minimum compacted fill thickness of 12 inches.

Load bearing slabs should be designed using a modulus of subgrade reaction (k) not exceeding 150 pounds per square inch per inch. This value is based on an applied foundation load area of 1.0 square foot. The k value should be reduced for larger foundation areas according to the following formula:

$$k_R = k * ((B+1) / 2B))^2$$

where k_R = reduced modulus of subgrade reaction
B = foundation width (feet)

Slabs should be designed and constructed in accordance with the provisions of the American Concrete Institute (ACI). Shrinkage of concrete should be anticipated and will result in cracks in all concrete slabs-on-grade. Shrinkage cracks may be directed to saw-cut "control joints" spaced on the basis of slab thickness and reinforcement. Control joint spacing in unreinforced concrete at maximum intervals equal to the slab thickness times 24 is recommended.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder/barrier designed and constructed according to the American Concrete Institute 302.1 R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder/barrier construction. At a minimum, the vapor retarder/barrier should comply with ASTM E1745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage.

Portland Cement Concrete (PCC) Pavement: All surfaces that will support fire apparatus should be paved with Portland cement concrete (PCC). PCC pavement should consist of 9 inches of PCC over 12 inches of Class 2 aggregate base. The

concrete should have a minimum 28-day modulus of rupture of 600 psi. This corresponds to a compressive strength of approximately 4,500 psi.

For all other areas that will utilize PCC pavement the below table can be utilized for design sections. The following Portland cement concrete pavement sections are based on the American Concrete Institute (ACI) Guide for Design and Construction of Concrete Parking Lots and Site Paving (ACI 330-21). The concrete to be utilized for Category A and B areas as well as pedestrian areas should have a minimum 28-day modulus of rupture of 550 psi. This corresponds to a compressive strength of approximately 3,000 psi. The actual pavement subgrade soil should be evaluated during construction to verify that the recommended pavement sections are appropriate.

Table 3: Portland Cement Concrete Pavement

Service	Concrete Thickness (in.)	Aggregate Base (in.)
Car parking and access lanes (Category A)	4.25	6.0
Entrance and truck service lanes (Category B)	5.25	6.0
Pedestrian, non-vehicular hardscape	4.0	0.0

The Class 2 aggregate base should comply with current Caltrans requirements. The aggregate base should be compacted to at least 95 percent relative compaction based on ASTM D1557. The upper 12 inches of pavement subgrade soil, below the aggregate base, should also be compacted to a minimum relative compaction of 95 percent. The concrete pavement should be constructed with doweled joints and be restrained laterally by concrete curb/gutter or building foundations. The edges of the concrete should be protected from traffic loads by curbs or paved shoulders. If unrestrained pavement edges or non-doweled joints are desired, this firm should be contacted so that revised recommendations can be developed.

Construction joints should be sawcut in the pavement at a maximum spacing of 30 times the thickness of the slab, up to a maximum of 15 feet. Pavement sawcutting should be performed within 12 hours of concrete placement, preferably sooner. Sawcut depths should be equal to approximately $\frac{1}{4}$ of the slab thickness for conventional saws or one inch when early-entry saws are utilized on slabs nine inches thick or less. Construction joints should not be placed near flow lines. The use of plastic strips for formation of jointing is not recommended. The use of expansion joints is not recommended, except where the pavement will adjoin structures.

Asphalt Concrete Pavement: Recommended asphalt concrete structural pavement sections are shown below in Table 4.

Table 4: Asphalt Concrete Pavement

Service	Asphalt Concrete Thickness (ft.)	Base Course Thickness (ft.)
Light traffic (autos, parking areas, T.I. = 5.0)	0.25	0.35
Heavy traffic (trucks, driveways, T.I. = 7.0)	0.30	0.55
Murrieta Road (T.I. = 9.0)	0.70	1.00

Inland Foundation Engineering, Inc. does not practice traffic engineering. The pavement section for Murrieta Road is based on the City of Perris minimum for a traffic index (T.I.) of 9.0. The other T.I. values used to develop the recommended pavement sections are typical for projects of this type. The project civil engineer or traffic engineer should review the T.I. values used to verify that they are appropriate for this project.

General Site Grading: All grading should be performed per the applicable provisions of the 2022 California Building Code and the following recommendations.

- 1. Clearing and Grubbing:** All building and pavement areas and all surfaces to receive compacted fill should be cleared of vegetation, debris, and other unsuitable materials. All such material should be disposed of off-site.

All undocumented artificial fill and loose native soil within the grading limits should be completely removed. Such material is suitable for use as compacted fill as recommended herein.

- 2. Preparation of Surfaces to Receive Compacted Fill:** All surfaces to receive compacted fill should be reviewed by a geologist or engineer from this firm prior to processing. If roots or other deleterious materials are encountered or if the exposed excavation bottom is loose or unstable, additional over-excavation may be required until satisfactory conditions are encountered. Upon approval, surfaces to receive fill should be scarified to a minimum depth of eight inches, brought to near optimum moisture content, and compacted to a minimum of 90 percent relative compaction.
- 3. Placement of Compacted Fill:** Fill materials consisting of on-site soil or approved imported granular soil should be spread in shallow lifts and compacted at near optimum moisture content to a minimum of 90 percent relative compaction, based on ASTM D1557.
- 4. Import Soil:** All proposed import soil should be tested prior to placement on the site to verify that it is not corrosive or expansive. Recommended import soil criteria are shown in the following Table 5.

Table 5: Recommended Import Soil Criteria

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

5. **Preparation of Building Areas:** The existing ground surface in the building pad area should be over-excavated to a depth of at least 3 feet. The over-excavation area should extend at least 5 feet outside the limits of the building pad. The building pad area should be further over-excavated to the depth necessary to provide at least 2 feet of compacted fill below the deepest footing. Following excavation, the exposed soil should be evaluated by this firm to verify it is suitable to receive compacted fill. The removed soil should be placed and compacted as recommended above.
6. **Preparation of Paving Areas:** During final grading and immediately prior to the placement of aggregate base, all surfaces to receive asphalt concrete or Portland cement concrete paving should be processed and recompact to a depth of at least 12 inches. Compaction within proposed pavement areas should be to a minimum of 95 percent relative compaction for both the subgrade and base course.
7. **Utility Trench Backfill:** Utility trench backfill consisting of the on-site soil types should be placed by mechanical compaction to a minimum of 90 percent relative compaction. This is with the exception of the upper 12 inches under pavement areas where the minimum relative compaction should be 95 percent. Jetting of the native soil is not recommended.
8. **Testing and Observation:** During grading, tests and observations should be performed by a representative of this firm to verify that the grading is performed per the project specifications. Density testing should be performed per the current ASTM D1556 or ASTM D6938 test methods. The minimum acceptable degree of compaction should be 90 percent of the maximum dry density, based on ASTM D1557, except where superseded by more stringent requirements, such as beneath

pavement. Where testing indicates insufficient density, additional compactive effort should be applied until retesting indicates satisfactory compaction.

Limitations

The findings and recommendations presented in this report are based on the soil conditions encountered at the boring locations. Should conditions be encountered during grading that appear to be different than those indicated by this report, this office should be notified.

This report was prepared for STK Architecture, Inc. for their use in the design of the proposed South Perris fire station. This report may only be used by STK Architecture, Inc for this purpose. The use of this report by parties or for other purposes is not authorized without written permission by Inland Foundation Engineering, Inc. Inland Foundation Engineering, Inc. will not be liable for any projects connected with the unauthorized use of this report.

The recommendations of this report are considered to be preliminary. The final design parameters may only be determined or confirmed at the completion of site grading on the basis of observations made during the site grading operation. To this extent, this report is not considered to be complete until the completion of both the design process and the site preparation.

The information in this report represents professional opinions that have been developed using that degree of care and skill ordinarily exercised, under similar circumstances, by reputable geotechnical consultants practicing in this or similar localities. No warranty, express or implied, is made.

References

American Concrete Institute 318 (2019), Building Code Requirements for Structural Concrete.

American Concrete Institute PRC -330 (2021), Commercial Concrete Parking Lots and Site Paving Design and Construction - Guide

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads and Associated Criteria for Buildings and other Structures, ASCE Standard 7-16, 889pp.

California Building Standards Commission, 2022, California Building Code (CBC), California Code of Regulations, Title 24, Part 2, Volume 2.

Riverside County Low Impact Development BMP Design Handbook (2011)

Terra Geosciences, Geologic Hazards Report, South Perris Fire Station CIP F077, Project No. 254143-1, dated August 6, 2025

United States Geologic Survey, Perris 7.5' Quadrangle (2021)

***APPENDIX A –
Site Exploration***

Appendix A

Site Exploration

Six exploratory borings were drilled at the approximate locations shown on Figure A-9. The materials encountered during drilling were logged by a staff geologist. Boring logs are included with this report as Figures A-3 through A-8.

Representative undisturbed soil samples were obtained within the borings by driving a modified California split spoon sampler and Standard Penetration Test (SPT) sampler. Representative bulk soil samples were also obtained from the excavation cuttings. Samples were placed in moisture sealed containers and transported to our laboratory for further testing and evaluation. Laboratory tests results are discussed and included in Appendix B.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

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

CONSISTENCY CRITERIA BASES ON FIELD TESTS

RELATIVE DENSITY – COARSE – GRAIN SOIL			CONSISTENCY – FINE-GRAIN SOIL		TORVANE	POCKET ** PENETROMETER	* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM -1586 STANDARD PENETRATION TEST)
RELATIVE DENSITY	SPT * (# BLOWS/FT)	RELATIVE DENSITY (%)	CONSISTENCY	SPT* (# BLOWS/FT)	UNDRAINED SHEAR STRENGTH (tsf)	UNCONFINED COMPRESSIVE STRENGTH (tsf)	
VERY LOOSE	<4	0-15	Very Soft	<2	<0.13	<0.25	
LOOSE	4-10	15-35	Soft	2-4	0.13-0.25	0.25-0.5	
MEDIUM DENSE	10-30	35-65	Medium Stiff	4-8	0.25-0.5	0.5-1.0	
DENSE	30-50	65-85	Stiff	8-15	0.5-1.0	1.0-2.0	
VERY DENSE	>50	85-100	Very Stiff	15-30	1.0-2.0	2.0-4.0	** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER
			Hard	>30	>2.0	>4.0	
MOISTURE CONTENT			CEMENTATION				
DESCRIPTION	FIELD TEST		DESCRIPTION	FIELD TEST			
DRY	Absence of moisture, dusty, dry to the touch		Weakly	Crumbled or breaks with handling or slight finger pressure			
MOIST	Damp but no visible water		Moderately	Crumbles or breaks with considerable finger pressure			
WET	Visible free water, usually soil is below water table		Strongly	Will not crumble or break with finger pressure			

EXPLANATION OF LOGS

LOG OF BORING B-01

CITY OF PERRIS PERMIT
NUMBER:
BCOM-25-00416

DRILLING RIG	<u>Mobile B-61</u>	DATE DRILLED	<u>7/31/25</u>	HAMMER TYPE	<u>Auto-Trip</u>
DRILLING METHOD	<u>Rotary Auger</u>	HAMMER WEIGHT	<u>140-lb.</u>	HAMMER DROP	<u>30-inches</u>
LOGGED BY	<u>FWC</u>	BORING DIAMETER	<u>8-inches</u>		
GROUND ELEVATION	<u>+/-</u>				

SUMMARY OF SUBSURFACE CONDITIONS										
DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)	
	SC		CLAYEY SAND , very fine to fine, dark grayish-brown (10YR 4/2), moist, very loose to dense.			AU				
5	CL		SANDY CLAY , grayish-brown (10YR 5/2) to dark yellowish-brown (10YR 4/4), moist, stiff, caliche.			AU SS	22 36	13	112	
						SS	12 10	14	112	
10	SC		CLAYEY SAND , very fine to fine, light olive-brown (2.5Y 5/4), moist, medium dense.			SS AU	10 13	27	96	
							SS	7 11	7	121
15	SC-SM		SILTY, CLAYEY SAND , fine to very coarse, olive (5Y 5/3), moist, medium dense to dense, moderately cemented.			SS	16 29	14	116	
20	SM		SILTY SAND , with trace clay, fine to coarse, dark yellowish-brown (10YR 4/4), moist, dense.			SPT	19 18	9		
							SPT	16 23	13	
25					End of boring at 26.5 feet. No groundwater encountered. Backfilled with native soil.					



CLIENT	<u>STK Architecture, Inc.</u>
PROJECT NAME	<u>South Perris Fire Station CIP F077</u>
PROJECT LOCATION	<u>Murrieta Rd</u>
	<u>Perris, CA</u>
PROJECT NUMBER	<u>S168-196</u>

FIGURE NO.

A-3

LOG OF BORING B-02

CITY OF PERRIS PERMIT
NUMBER:
BCOM-25-00416

DRILLING RIG	Mobile B-61	DATE DRILLED	7/31/25	HAMMER TYPE	Auto-Trip
DRILLING METHOD	Rotary Auger	HAMMER WEIGHT	140-lb.	HAMMER DROP	30-inches
LOGGED BY	FWC	BORING DIAMETER	8-inches		
GROUND ELEVATION	+/-				

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS				BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
			This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.									
5	SM		SILTY SAND , with trace clay, very fine to fine, dark grayish-brown (10YR 4/2), slightly moist, very loose to medium dense.						AU	4	12	106
	SC								AU			
	SS								9			
10	CL		CLAYEY SAND , very fine to fine, brown (10YR 5/3), moist, medium dense.						AU	16	14	110
	SS								29			
	SS								12			
15	SC		SANDY CLAY , olive-brown (2.5Y 4/3), moist, very stiff, caliche, moderately cemented.						AU	19	19	112
	SS								15			
	SS								9			
20	SM		CLAYEY SAND , very fine to fine, olive-brown (2.5Y 4/3), moist, medium dense, moderately cemented.						SS	34	9	127
	AU								15			
	SS								50/4"			
25	SM		SILTY SAND , fine to coarse, dark yellowish-brown (10YR 4/4), moist, dense, indurated, moderately cemented.						SS	26	6	119
	AU								31			
	SS								19			
30	SM		SILTY SAND , with trace clay, fine to coarse, brown (10YR 5/3), moist, very dense, moderately cemented.						SPT	22	6	
	SS								26			
	SS								31			
35	SW-SM		SAND with SILT , fine to coarse, dark yellowish-brown (10YR 4/4), slightly moist, dense.						SPT	19	6	
	SS								26			
	SS								31			
40	SM		SILTY SAND , fine to medium, dark yellowish-brown (10YR 4/4), moist, dense.						SPT	22	6	
	SS								26			
	SS								31			
45	SC		SILTY SAND , fine to medium, dark yellowish-brown (10YR 4/4), moist, dense.						SPT	24	17	
	SS								24			
	SS								14			
50	SC		CLAYEY SAND , very fine to fine, brown (7.5YR 4/3), moist, dense.						SPT	30	10	
	SS								36			
	SS								26			
55	SM		SILTY SAND , with trace clay, fine to medium, brown (7.5YR 5/3), moist, dense.						SPT	30	15	
	SS								30			
	SS								34			
60	SM		SILTY, CLAYEY SAND , very fine to fine, dark yellowish-brown (10YR 4/4), moist, dense.						SPT	16	16	
	SS								30			
	SS								34			
65	SM		SILTY SAND , with trace clay, very fine to fine, dark yellowish-brown (10YR 4/4), moist, very dense.						SPT	27	13	
	SS								41			
	SS								27			
70	SM		SAND with SILT , fine to coarse, dark yellowish-brown (10YR 4/4), slightly moist, very dense.						SPT	41	13	
	SS								27			
	SS								41			
75	SM		CLAYEY SAND , very fine to fine, olive, wet, very dense.						SPT	27	13	
	SS								41			
	SS								27			
80	SM		SILTY SAND , with trace clay, fine to medium, dark yellowish-brown (10YR 4/4), moist, very dense.						SPT	41	13	
	SS								41			
	SS								27			
85	SM		End of boring at 51.5 feet. Groundwater initially encountered at 45.5 feet. Final groundwater at 40.5 feet. Mottling encountered at 35 feet. Backfilled with native soil.						SPT	41	13	
	SS								41			
	SS								27			



CLIENT	STK Architecture, Inc.
PROJECT NAME	South Perris Fire Station CIP F077
PROJECT LOCATION	Murrieta Rd
	Perris, CA
PROJECT NUMBER	S168-196

FIGURE NO.

A-4

LOG OF BORING B-03

CITY OF PERRIS PERMIT
NUMBER:
BCOM-25-00416

DRILLING RIG	<u>Mobile B-61</u>	DATE DRILLED	<u>7/31/25</u>	HAMMER TYPE	<u>Auto-Trip</u>
DRILLING METHOD	<u>Rotary Auger</u>	HAMMER WEIGHT	<u>140-lb.</u>	HAMMER DROP	<u>30-inches</u>
LOGGED BY	<u>FWC</u>	BORING DIAMETER	<u>8-inches</u>		
GROUND ELEVATION	<u>+/-</u>				

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
	SM		SILTY SAND , with trace clay, very fine to fine, dark grayish-brown, slightly moist, very loose to medium dense.						
	SC		CLAYEY SAND , very fine to fine, dark grayish-brown (10YR 4/2), moist, dense, moderately cemented.						
5				X	SS		20 38	10	123
	SC		CLAYEY SAND , very fine to fine, brown (10YR 5/3), moist, medium dense, caliche, moderately cemented.						
				X	SS		16 18	4	123
10				X	SS		15 16	13	119
	SC		CLAYEY SAND , very fine to fine, brown (10YR 5/3), moist, very dense.						
				X	SS		15 31	18	115
15				X	SS		26 52	10	129
	SC- SM		SILTY, CLAYEY SAND , fine to very coarse, yellowish-brown (10YR 5/4), moist, very dense, moderately cemented.						
20				X	SS		28 41	9	127
	SM		SILTY SAND , with trace clay, fine to very coarse, yellowish-brown (10YR 5/4), moist, dense.						
				X	SS		31 43	7	120
25									
	SM		SILTY SAND , with trace gravel, fine to coarse, yellowish-brown (10YR 5/4), moist, dense to very dense.						
				X	SS				
			End of boring at 26.5 feet. No groundwater encountered. Backfilled with native soil.						



CLIENT	<u>STK Architecture, Inc.</u>
PROJECT NAME	<u>South Perris Fire Station CIP F077</u>
PROJECT LOCATION	<u>Murrieta Rd</u>
	<u>Perris, CA</u>
PROJECT NUMBER	<u>S168-196</u>

FIGURE NO.

A-5

LOG OF BORING B-04

CITY OF PERRIS PERMIT
NUMBER:
BCOM-25-00416

DRILLING RIG	<u>Mobile B-61</u>	DATE DRILLED	<u>7/31/25</u>	HAMMER TYPE	<u>Auto-Trip</u>
DRILLING METHOD	<u>Rotary Auger</u>	HAMMER WEIGHT	<u>140-lb.</u>	HAMMER DROP	<u>30-inches</u>
LOGGED BY	<u>FWC</u>	BORING DIAMETER	<u>8-inches</u>		
GROUND ELEVATION	<u>+/-</u>				

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)	
5	SC		ALFALFA CLAYEY SAND , very fine to fine, dark gray-brown, slightly moist to moist, loose to medium dense.							
	CL		SANDY CLAY , grayish-brown, moist, stiff.							
	ML		SANDY SILT , dark grayish-brown, moist, stiff, porous, rootlets.			AU SS	9 16	11	94	
	CL		SANDY CLAY , dark grayish-brown (10YR 4/2), moist, hard, caliche, indurated.			SS	28 40	21	108	
10	SC		CLAYEY SAND , very fine to fine, yellowish-brown (10YR 5/6), moist, medium dense, caliche.			SS	8 12	18	110	
15	SC-SM			SILTY, CLAYEY SAND , fine to coarse, yellowish-brown (10YR 5/6), moist, medium dense.			SS	14 20	14	119
							SS	14 18	10	
					SM	SILTY SAND , with trace clay, fine to coarse, yellowish-brown (10YR 5/6), moist, medium dense to dense.			SPT	13 18
25	SM		SILTY SAND , with trace clay, fine to medium, olive, moist, medium dense.							
	SW		SAND , fine to coarse, yellowish-brown (10YR 5/6), slightly moist, dense.							
	SC		CLAYEY SAND , very fine to fine, strong brown (7.5YR 4/6), moist, dense.		SPT	15 22	19			
			End of boring at 26.5 feet. No groundwater encountered. Backfilled with native soils.							



CLIENT	<u>STK Architecture, Inc.</u>
PROJECT NAME	<u>South Perris Fire Station CIP F077</u>
PROJECT LOCATION	<u>Murrieta Rd</u>
	<u>Perris, CA</u>
PROJECT NUMBER	<u>S168-196</u>

FIGURE NO.

A-6

LOG OF BORING B-05

CITY OF PERRIS PERMIT
NUMBER:
BCOM-25-00416

DRILLING RIG	<u>Mobile B-61</u>	DATE DRILLED	<u>7/31/25</u>	HAMMER TYPE	<u>Auto-Trip</u>
DRILLING METHOD	<u>Rotary Auger</u>	HAMMER WEIGHT	<u>140-lb.</u>	HAMMER DROP	<u>30-inches</u>
LOGGED BY	<u>FWC</u>	BORING DIAMETER	<u>8-inches</u>		
GROUND ELEVATION	<u>+/-</u>				

DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
5	SC		ALFALFA CLAYEY SAND , very fine to fine, dark grayish-brown (10YR 4/2), moist, very loose.		AU	SS	2 3	17	104
	CL		SANDY CLAY , yellowish-brown (10YR 5/4), moist, hard, caliche.		SS	17 26	26	103	
10	SC		CLAYEY SAND , very fine to fine, strong brown (7.5YR 4/6), moist to very moist, medium dense, caliche, with thin interbeds of silty sand.		AU	SS	26 23	16	115
	SC		CLAYEY SAND , very fine to fine, strong brown (7.5YR 4/6), moist, medium dense.		SS	8 12	25	104	
15	SM		CLAYEY SAND , very fine to fine, strong brown (7.5YR 4/6), moist, medium dense.		AU	SS	20 18	8	127
	SC-SM		SILTY SAND , fine to coarse, dark yellowish-brown (10YR 4/4), moist, medium dense.		SS	32 52	7	125	
20	SM		SILTY, CLAYEY SAND , very fine to medium, dark yellowish-brown (10YR 4/4), moist, medium dense to dense.		AU	SS	32 52	7	125
	SC		SILTY SAND , fine to coarse, dark yellowish-brown (10YR 4/6), moist, dense to very dense.		SS	30 46	10	126	
25	SC		CLAYEY SAND , very fine to fine, dark yellowish-brown (10YR 4/6), moist, dense.						
	SC		CLAYEY SAND , very fine to fine, brown ((7.5YR 4/6), mmoist, medium dense to dense.		SPT	13 18	18		
30			SILTY SAND , with trace clay, fine to medium, strong brown (7.5YR 4/6), moist, very dense.						
					SPT	20 32	11		
35	SM								
					SPT	24 36	10		
40									
					SPT	28 35	14		
45	SW		SAND , fine to coarse, mottled grayish-brown, wet, very dense.						
					SPT	24 31	14		
50	SC		CLAYEY SAND , very fine to medium, brown (10YR 4/3), wet, very dense.						
					SPT	19 44	19		
			End of boring at 51.5 feet. Groundwater initially encountered at 46 feet. Final groundwater at 42 feet. Mottling encountered at 41 feet. Backfilled with native soil.						



CLIENT	<u>STK Architecture, Inc.</u>
PROJECT NAME	<u>South Perris Fire Station CIP F077</u>
PROJECT LOCATION	<u>Murrieta Rd</u>
	<u>Perris, CA</u>
PROJECT NUMBER	<u>S168-196</u>

FIGURE NO.

A-7

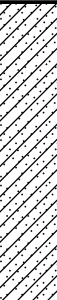





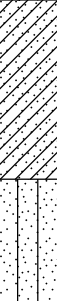


LOG OF BORING B-06

CITY OF PERRIS PERMIT
NUMBER:
BCOM-25-00416

DRILLING RIG Mobile B-61
DRILLING METHOD Rotary Auger
LOGGED BY FWC
GROUND ELEVATION +/-

DATE DRILLED 7/31/25

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

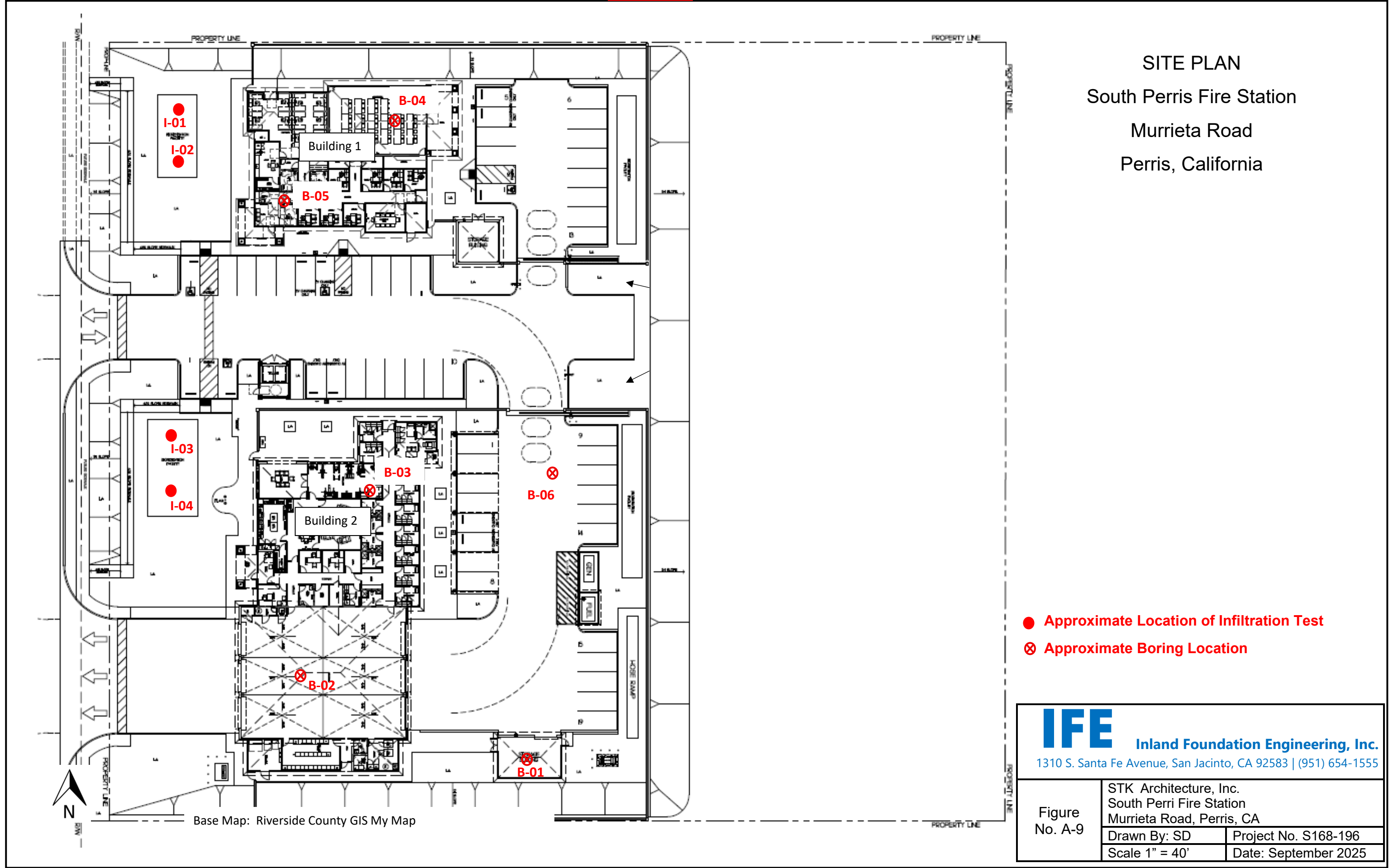
DEPTH (ft)	U.S.C.S.	GRAPHIC LOG	SUMMARY OF SUBSURFACE CONDITIONS This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered and is representative of interpretations made during drilling. Contrasting data derived from laboratory analysis may not be reflected in these representations.	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
5	SC		<u>CLAYEY SAND</u> , very fine to fine, dark grayish-brown (10YR 4/2), slightly moist, loose to medium dense, rootlets.			AU			
	SC		<u>CLAYEY SAND</u> , very fine to fine, dark grayish-brown (10YR 4/2), moist, medium dense.			AU SS	15 17	9	121
	10	SM		<u>SILTY SAND</u> , with trace clay, very fine to fine, dark yellowish-brown (10YR 4/4), slightly moist, medium dense, caliche.			SS	12 14	9
					SS	17 20	16	112	
15		SC		<u>CLAYEY SAND</u> , very fine to fine, yellowish-brown (10YR 5/4), moist, loose to medium dense, caliche.			SS	6 9	18
	SM	<u>SILTY SAND</u> , with trace clay, fine to medium, yellowish-brown (10YR 5/4), moist, medium dense, moderately cemented.				SS	12 19	10	127
				End of boring at 17 feet. No groundwater encountered. Backfilled with native soil.					



CLIENT STK Architecture, Inc.
PROJECT NAME South Perris Fire Station CIP F077
PROJECT LOCATION Murrieta Rd
Perris, CA
PROJECT NUMBER S168-196

FIGURE NO.

A-8



***APPENDIX B –
Laboratory Testing***

Appendix B

Laboratory Testing

Representative soil samples obtained from our borings were delivered to our laboratory. Descriptions of the tests performed are provided below. Results of the testing are appended.

Unit Weight and Moisture Content: Ring samples were weighed and measured to evaluate their unit weight. A small portion of each sample was then tested for moisture content. The testing was performed per ASTM D2937 and D2216. The results of this testing are shown on the boring logs (Figures A-3 through A-8).

Sieve Analysis: Seven soil samples were selected for sieve analysis testing in accordance with ASTM D6913. These tests provide information for classifying the soil in accordance with the Unified Classification System. This classification system categorizes the soil into groups having similar engineering characteristics. The results of the testing are shown on Figures B-3 and B-4.

Plastic Index: Seven samples were selected for plastic index testing in accordance with ASTM D4318. These tests provide information regarding soil plasticity and are also used for developing classifications of the soil in accordance with the Unified Classification System. The results of the testing are shown on Figures B-3 and B-4.

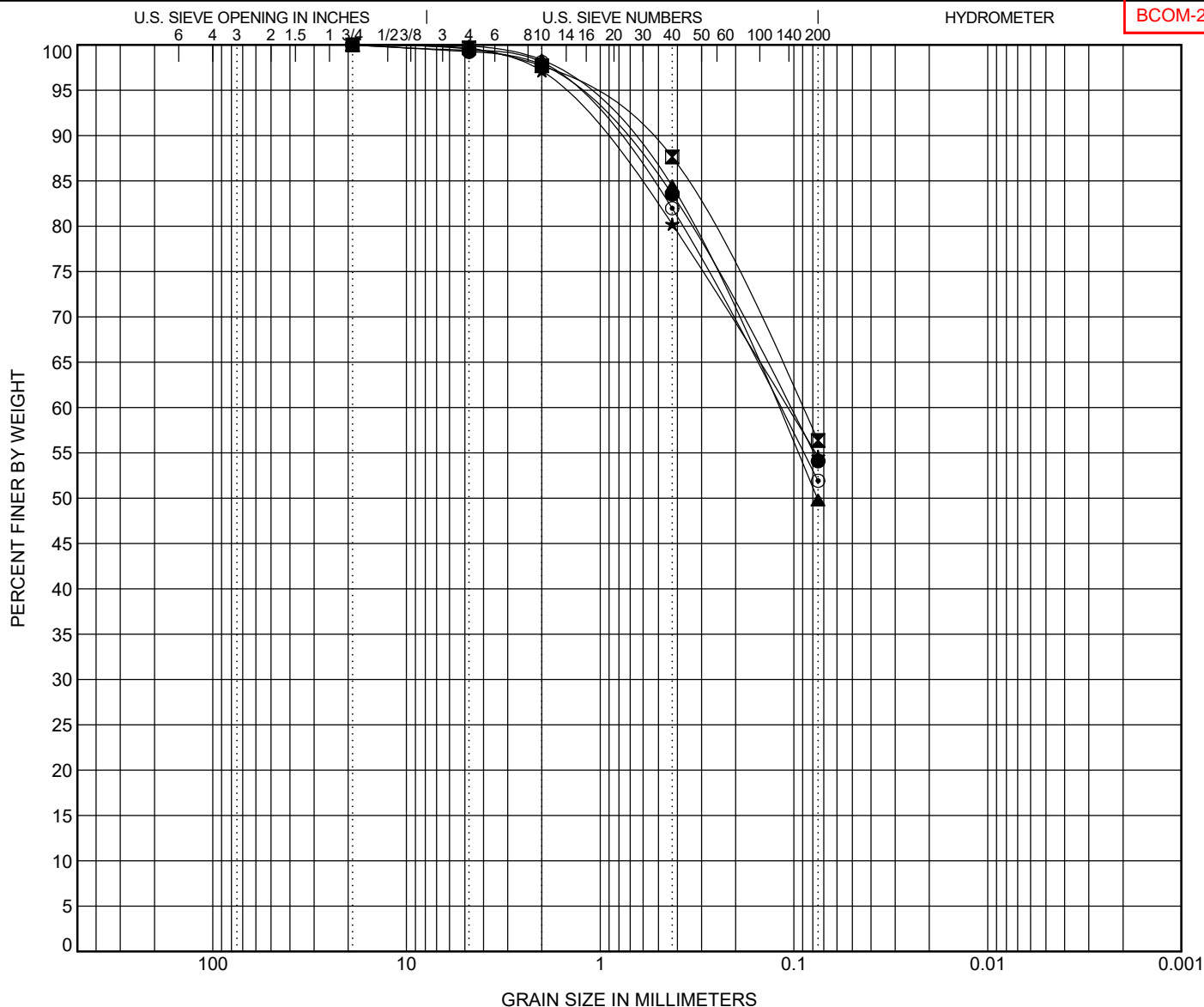
Maximum Density-Optimum Moisture: Two samples were selected for maximum density testing in accordance with ASTM D1557. The maximum density is compared to the in-situ density of the soil to evaluate the relative compaction of the soil. The results of the testing are shown on Figure B-5.

Consolidation Testing: Two samples were selected for consolidation testing in accordance with ASTM D2435. This test is used to evaluate the magnitude and rate of settlement of a structure or earth fill. The results of this testing are presented graphically on Figure B-6.

Direct Shear Strength: Two samples were selected and transported to AP Engineering and Testing in Pomona, California for direct shear strength testing in accordance with ASTM D3080. This testing measures the shear strength of the soil under various normal pressures and is used to develop parameters for foundation bearing capacity and lateral earth pressure. Test results are shown on Figures B-7 and B-8.

Corrosion Testing: One sample was selected and transported to AP Engineering and Testing in Pomona, California to evaluate the concentration of soluble sulfates and chlorides, pH level, and resistivity of and within the on-site soils. The test results are shown on Figure B-9.

R-value: One sample was selected for R-value and transported to AP Engineering and Testing in Pomona, California for testing in accordance with ASTM D2844. This test measures the potential strength of subgrade, subbase, and base course materials for use in pavements. Test results are shown on Figure No. B-10.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

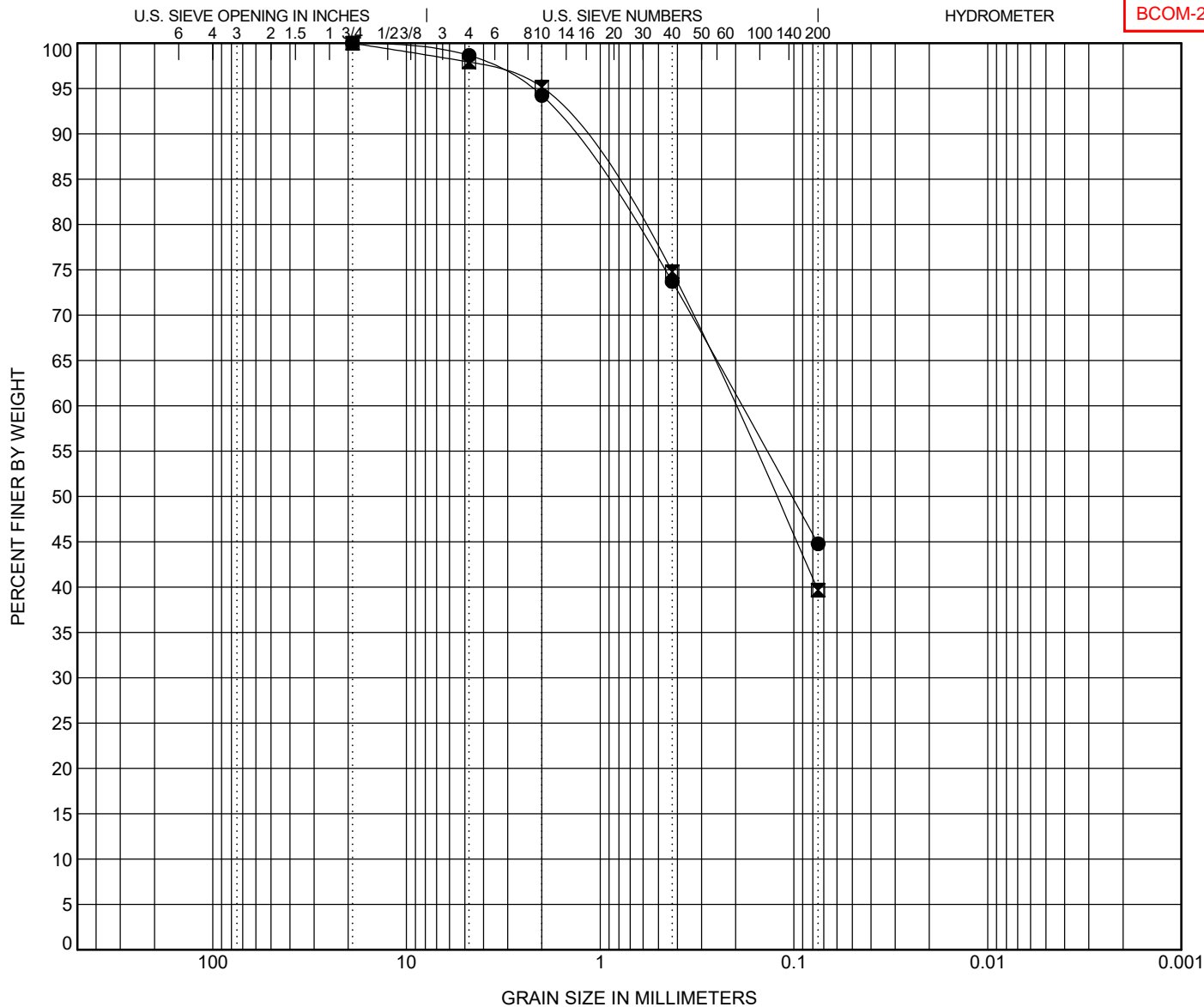
SAMPLE	DEPTH	Classification					LL	PL	PI	Cc	Cu
● B-01	2.8	SANDY LEAN CLAY(CL)					31	20	11		
⊠ B-02	4.0	SANDY LEAN CLAY(CL)					34	17	17		
▲ B-02	8.3	CLAYEY SAND(SC)					30	15	15		
★ B-04	3.5	SANDY SILT(ML)					38	27	11		
⊙ B-05	3.5	SANDY LEAN CLAY(CL)					29	20	9		
BOREHOLE	DEPTH	D100	D90	D50	D10	%Gravel	%Sand	%Silt		%Clay	
● B-01	2.8	19	0.857			0.7	45.2	54.1			
⊠ B-02	4.0	19	0.611			0.3	43.3	56.4			
▲ B-02	8.3	19	0.79	0.076		0.1	50.1	49.8			
★ B-04	3.5	19	1.041			0.5	44.9	54.6			
⊙ B-05	3.5	19	0.919			0.6	47.5	51.9			

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-3

CLIENT	<u>STK Architecture, Inc.</u>	PROJECT NAME	<u>South Perris Fire Station CIP F077</u>
PROJECT NUMBER	<u>S168-196</u>	PROJECT LOCATION	<u>Murrieta Rd</u>
			<u>Perris, CA</u>



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

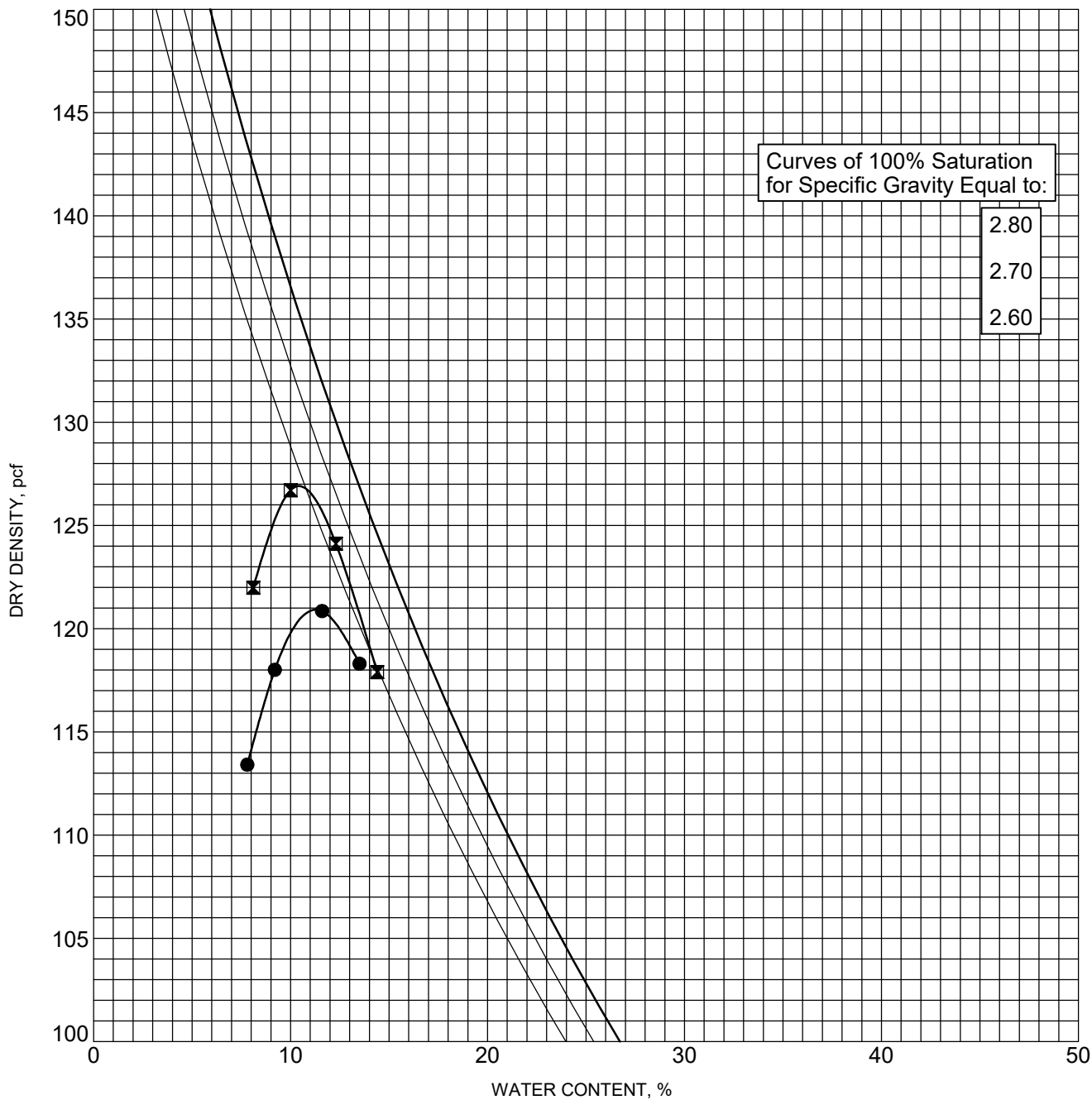
SAMPLE	DEPTH	Classification					LL	PL	PI	Cc	Cu
● B-05	7.3	CLAYEY SAND(SC)					30	16	14		
☒ B-06	0.0	CLAYEY SAND(SC)					29	21	8		
BOREHOLE	DEPTH	D100	D90	D50	D10	%Gravel	%Sand	%Silt		%Clay	
● B-05	7.3	19	1.453	0.103		1.4	53.9	44.8			
☒ B-06	0.0	19	1.351	0.125		2.1	58.3	39.7			

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-4

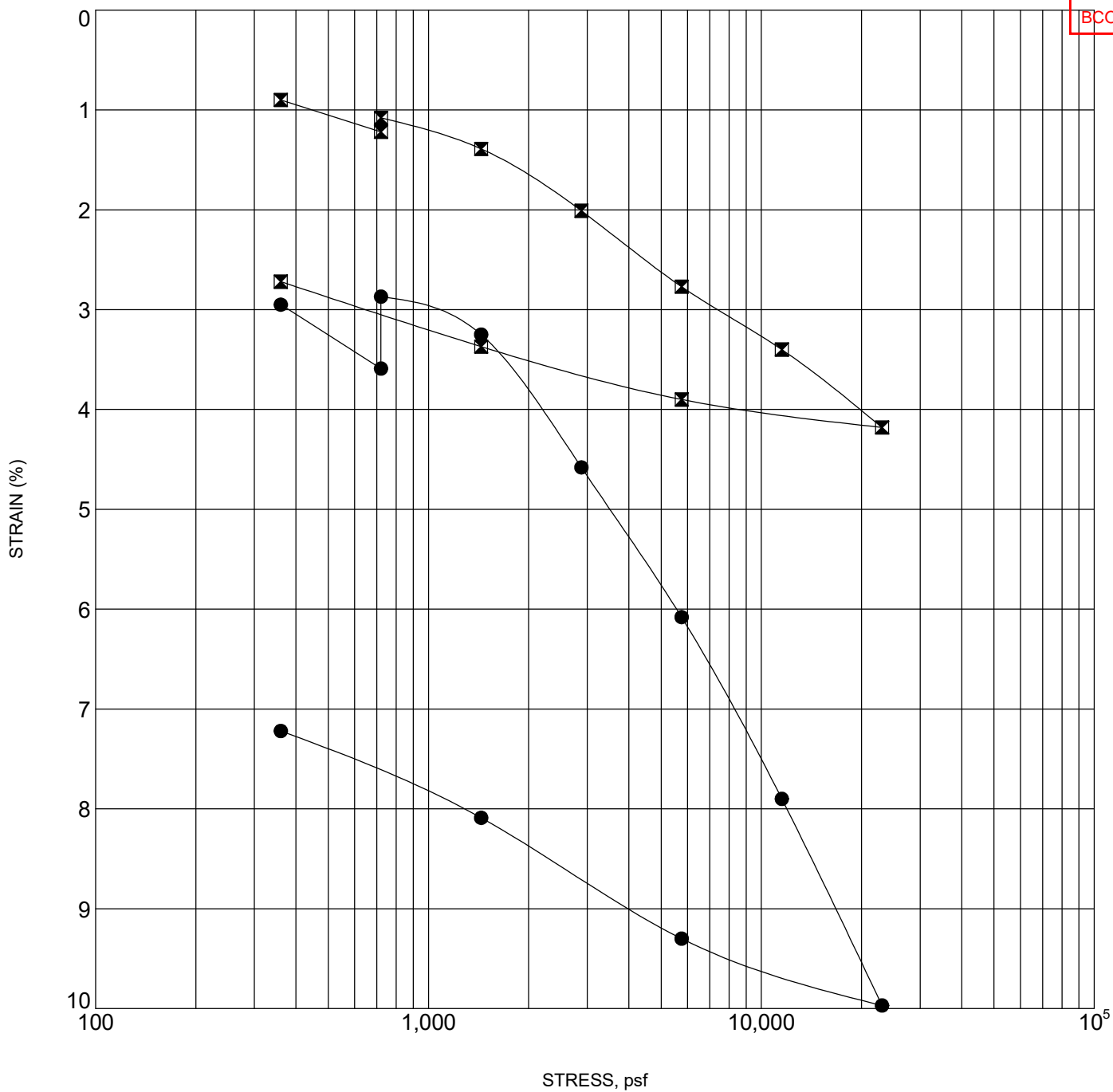
CLIENT STK Architecture, Inc. PROJECT NAME South Perris Fire Station CIP F077
PROJECT NUMBER S168-196 PROJECT LOCATION Murrieta Rd
Perris, CA



BOREHOLE	DEPTH	Description of Materials	Max DD	Optimum WC
● B-02	1.5	CLAYEY SAND(SC)	120.9 PCF	11.3 %
☒ B-05	0.3	CLAYEY SAND(SC)	126.9 PCF	10.5 %

INLAND FOUNDATION ENGINEERING, INC. **MOISTURE-DENSITY CURVES (ASTM D1557)**

CLIENT STK Architecture, Inc. PROJECT NAME South Perris Fire Station CIP F077
PROJECT NUMBER S168-196 PROJECT LOCATION Murrieta Rd
Perris, CA



BOREHOLE	DEPTH	Classification	γ_d	MC%
● B-03	6.5	CLAYEY SAND(SC)	115	13
⊠ B-05	7.5	CLAYEY SAND(SC)		

CONSOLIDATION TEST (ASTM D2435)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-6



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS **ASTM D 3080**

Project Name: STK CIP FO77
Project No.: S168-196
Boring No.: B-03
Sample No.: - **Depth (ft):** 3.5-4.5
Sample Type: Mod. Cal.
Soil Description: Sandy Silt
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/02/25
Computed By: JP **Date:** 09/08/25
Checked by: AP **Date:** 09/08/25

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
133.2	119.3	11.6	15.2	76	100	1	1.328	0.875
						2	2.160	1.424
						3	2.868	2.184

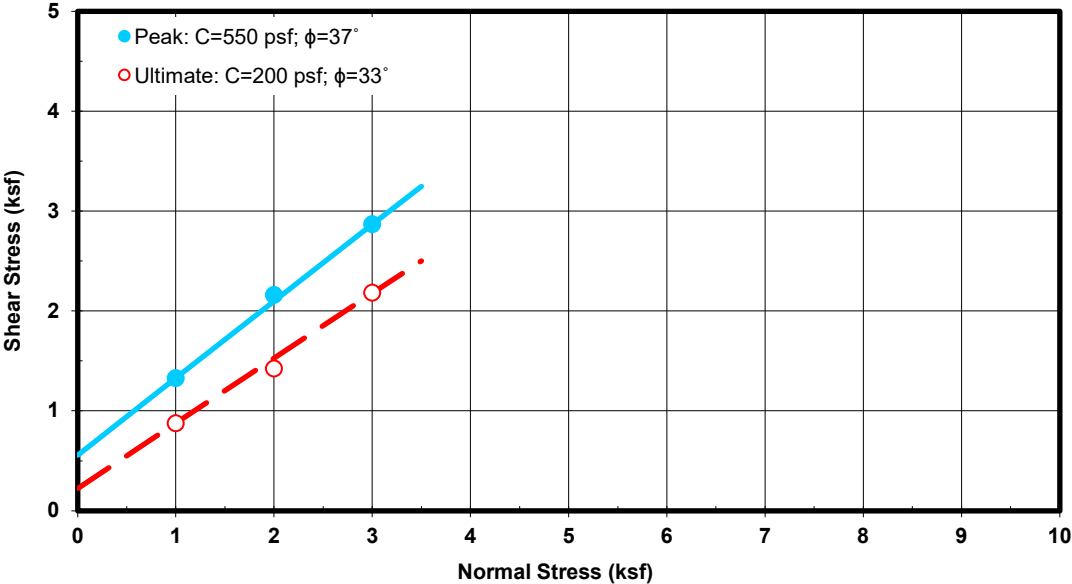
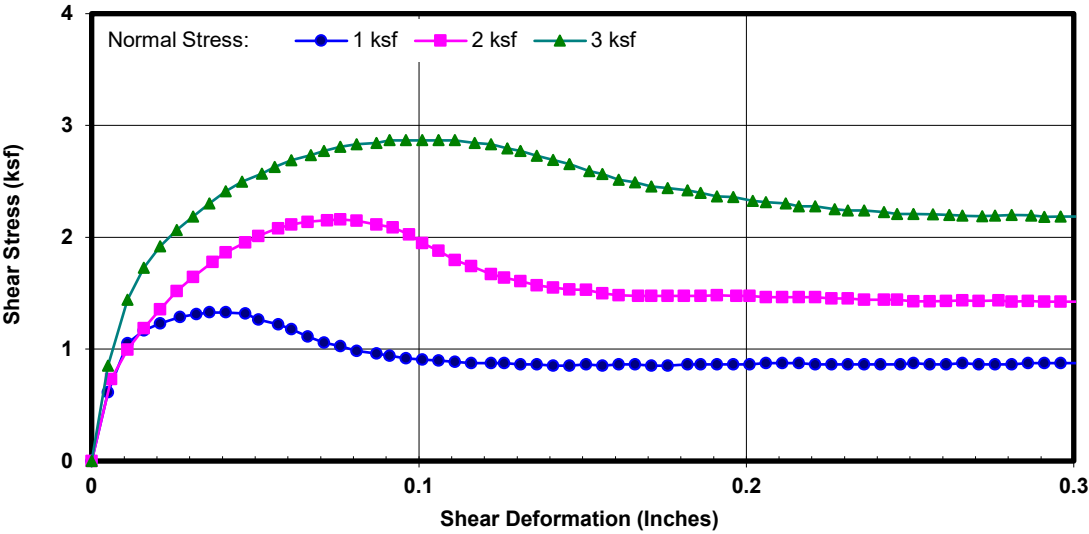


Figure No. B-7



AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS ASTM D 3080

Project Name: STK CIP FO77
Project No.: S168-196
Boring No.: B-04
Sample No.: - **Depth (ft):** 6.5-7.5
Sample Type: Mod. Cal.
Soil Description: Clay w/sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 09/02/25
Computed By: JP **Date:** 09/08/25
Checked by: AP **Date:** 09/08/25

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
117.4	100.1	17.3	25.2	68	100	1	1.644	0.744
						2	2.472	1.368
						3	3.036	1.782

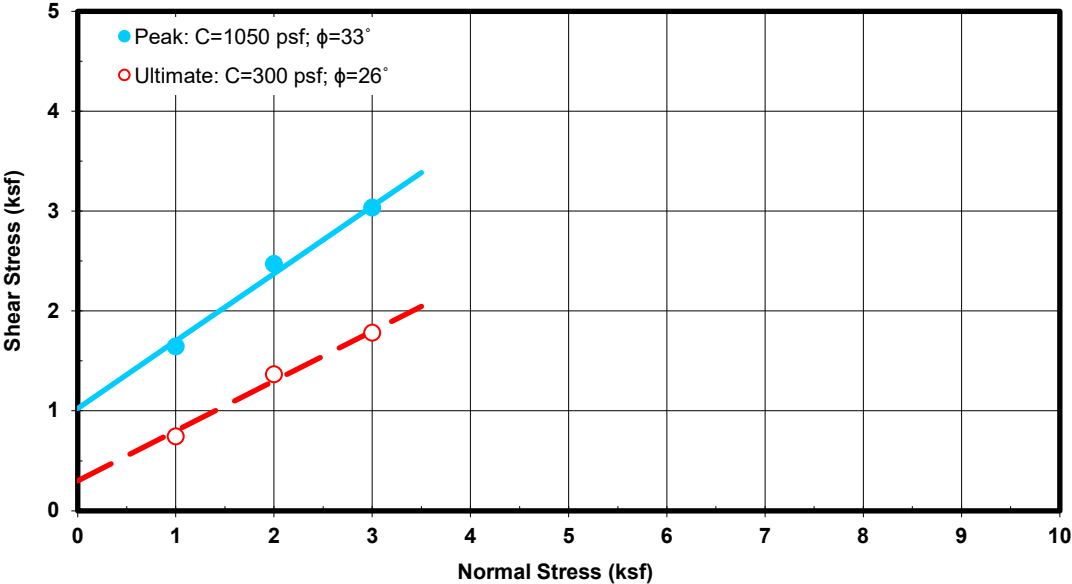
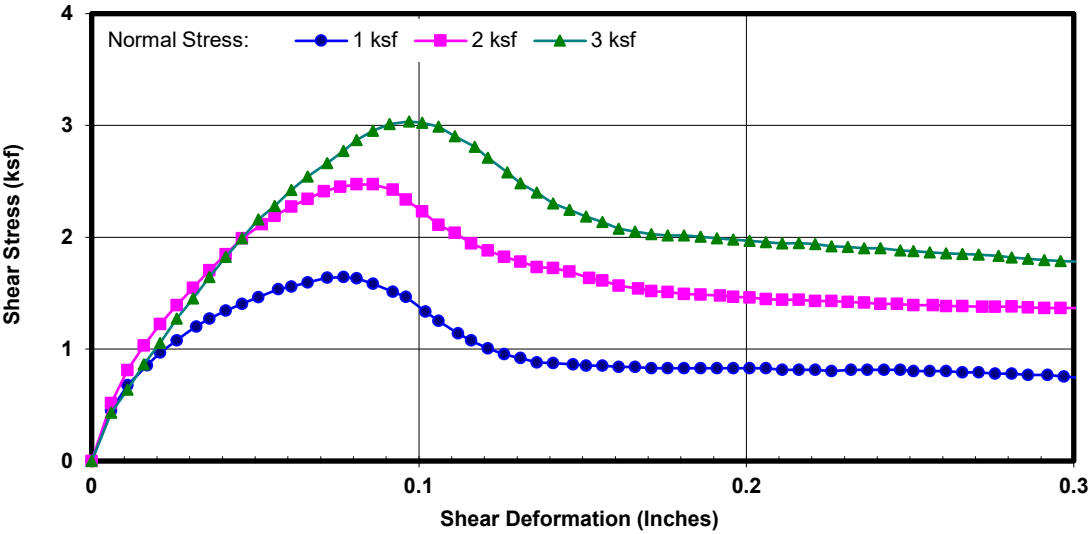


Figure No. B-8

**AP Engineering and Testing, Inc.**

DBE | MBE | SBE

2607 Pomona Boulevard | Pomona, CA 91768

t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CITY OF PERRIS PERMIT
NUMBER:
BCOM-25-00416**R-VALUE TEST DATA**

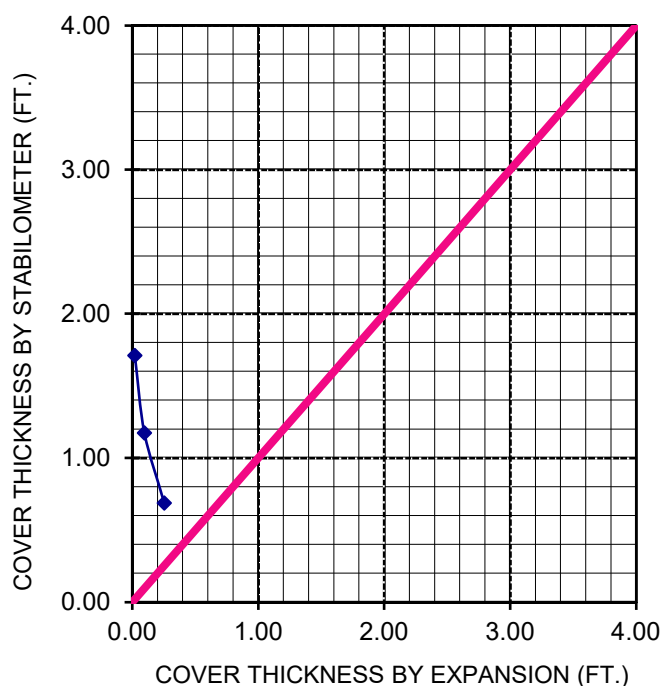
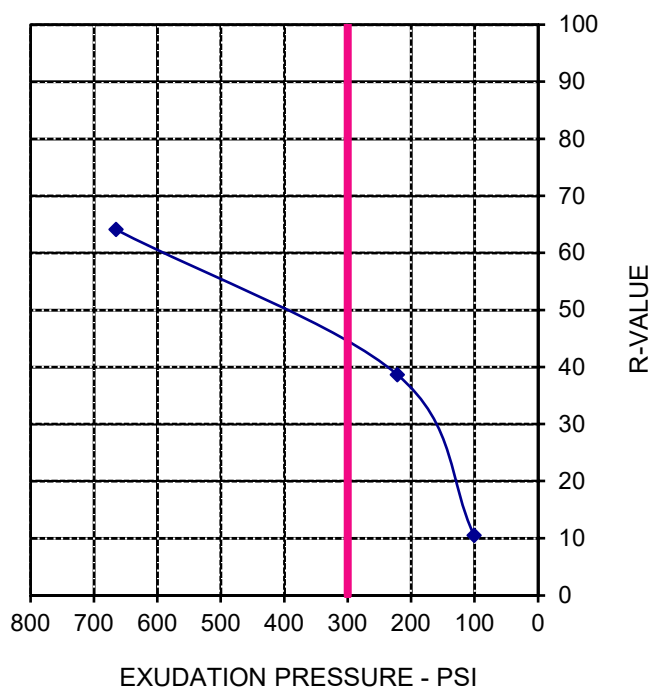
ASTM D2844

Project Name: STK CIP FO77
Project Number: S168-196
Boring No.: B-6
Sample No.: - Depth (ft.): 0-2.5
Location: N/A
Soil Description: Sandy Silt w/trace clay

Tested By: ST Date: 08/27/25
Computed By: KM Date: 08/28/25
Checked By: AP Date: 09/08/25

Mold Number	G	I	H	
Water Added, g	139	113	96	
Compact Moisture(%)	16.7	14.1	12.4	
Compaction Gage Pressure, psi	100	250	250	
Exudation Pressure, psi	101	222	665	
Sample Height, Inches	2.6	2.5	2.5	
Gross Weight Mold, g	2911	2876	2906	
Tare Weight Mold, g	1825	1817	1835	
Net Sample Weight, g	1085	1059	1071	
Expansion, inches $\times 10^{-4}$	6	29	76	
Stability 2,000 (160 psi)	52/125	30/64	15/31	
Turns Displacement	6.17	5.95	5.83	
R-Value Uncorrected	10	39	64	
R-Value Corrected	11	39	64	
Dry Density, pcf	108.4	112.5	115.4	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	1.71	1.17	0.69	
G.E. by Expansion	0.02	0.10	0.25	

R-VALUE	By Exudation:	44
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	44
Remarks	Gf = 1.34, and 0.0 % Retained on the 3/4" *Not Applicable	





AP Engineering and Testing, Inc.
DBE | MBE | SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

CORROSION TEST RESULTS

Client Name: Inland Foundation Engineering
Project Name: STK CIP FO77
Project No.: S168-196

AP Job No.: 25-0864
Date: 09/04/25

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-2	-	1.5-4	Silty Clay w/sand	1,552	8.4	32	28

NOTES: Resistivity Test and pH: California Test Method 643
Sulfate Content : California Test Method 417
Chloride Content : California Test Method 422
ND = Not Detectable
NA = Not Sufficient Sample
NR = Not Requested

APPENDIX C

Infiltration Testing

Appendix C

Infiltration Testing

Infiltration testing was conducted in general accordance with Appendix A of the Riverside County Low Impact Development BMP Design Handbook (2011). The shallow percolation test method was used per the Riverside County Department of Environmental Health guidelines. The percolation rates were converted to infiltration rates using the Porchet method.

Four percolation tests were performed at the locations shown on Figure A-9. The test holes were drilled on August 1, 2025 to depths of approximately 4 and 5 feet below existing ground surface. The test holes were approximately eight (8) inches in diameter. Gravel was placed in the bottom of each test hole. The test holes were then pre-soaked by inverting 5-gallons of water above the test hole.

Testing was conducted the same day following the presoak. For all tests, more than 6 inches of water seeped away twice consecutively in less than 25 minutes, which meets the sandy soil criteria. The tests were then run for an additional hour with measurements taken every 10 minutes.

The percolation rates were calculated to range from 4.0 to 17.1 minutes per inch (mpi). The percolation test rate was converted to an infiltration rate (I_c) using the Porchet method and the following equation:

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

Where:

r = Test Hole Radius (in.)

H_{avg} = Average Height of Water during Test Interval (in.)

ΔH = Change in Water Height during Test Interval (in.), and

Δt = Time Interval (in.)

The corresponding calculated infiltration rates (I_c) ranged from 0.3 to 1.2 inches per hour. These values exclude a factor of safety. Copies of the field test sheets are included with this report as Figures C-2 through C-5.

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: South Perris FS				Project No.: S168-196		Date: 8/1/2025		
Test Hole No.: I-01				Tested By: Floyd Collins				
Depth of Test Hole (D_T): 60"				USCS Soil Classification: SC-SM				
Test Hole Dimensions (inches)				Length		Width		
Diameter (if round)= 8"		Sides (if rectangular) =						
Sandy Soil Criteria Test*								
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" (Y/N)	
1	7:48	8:13	25	36	44	8	Y	
2	8:19	8:41	24	36	46 ¾	7 ¾	Y	
3								
<p>*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".</p>								
Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D _o Initial Depth to Water (in.)	D _f Final Depth to Water (in.)	ΔD=ΔH Change in Water Level (in.)	Perc. Rate min./in.	I _T $\frac{\Delta H 60r}{\Delta t(r+2H)}$ Avg
1	8:41	8:51	10	36	38 ¾	2 ¾	3.6	
2	8:52	9:02	10	36	39	3	3.3	
3	9:03	9:13	10	36	38 ¾	2 ¾	3.6	
4	9:14	9:24	10	36	38 ½	2 ½	4.0	1.21
5	9:24	9:35	10	36	38 ½	2 ½	4.0	1.21
6	9:35	9:45	10	36	38 ½	2 ½	4.0	1.21
7								
8								
9								
10								
11								
12								
13								
14								
15								
<p>COMMENTS: Sunny clear (70° to 95°) Ground dry. First two measurements met sandy soil criteria. Pre-soaked on 7/31/25.</p>								

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: South Perris FS				Project No.: S168-196		Date: 8/1/2025		
Test Hole No.: I-02				Tested By: Floyd Collins				
Depth of Test Hole (D_T): 48"				USCS Soil Classification: CL				
Test Hole Dimensions (inches)				Length		Width		
Diameter (if round)= 8"				Sides (if rectangular) =				
Sandy Soil Criteria Test*								
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" (Y/N)	
1	7:53	8:23	30	24	26 ½	2 ½	N	
2								
3								
<p>*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".</p>								
Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D _o Initial Depth to Water (in.)	D _f Final Depth to Water (in.)	ΔD=ΔH Change in Water Level (in.)	Perc. Rate min./in.	I _T $\frac{\Delta H}{\Delta t(r+2H)}$ 60r Avg
1	7:53	8:23	30	24	26 ½	2 ½	12	
2	8:25	8:55	30	24	26 ½	2 ½	12	
3	8:56	9:26	30	24	26	2	15	
4	9:26	9:56	30	24	26	2	15	
5	9:57	10:27	30	24	25 ¾	1 ¾	17.1	.28
6	10:28	10:58	30	24	25 ¾	1 ¾	17.1	.28
7	10:59	11:29	30	24	25 ¾	1 ¾	17.1	.28
8	11:29	11:59	30	24	25 ¾	1 ¾	17.1	.28
9	12:00	12:30	30	24	25 ¾	1 ¾	17.1	.28
10	12:30	1:00	30	24	25 ¾	1 ¾	17.1	.28
11	1:00	1:30	30	24	25 ¾	1 ¾	17.1	.28
12	1:31	2:01	30	24	25 ¾	1 ¾	17.1	.28
13								
14								
15								
<p>COMMENTS: Sunny clear (70° to 95°) Ground dry. First two measurements didn't meet sandy soil criteria. Pre-soaked on 7/31/25.</p>								

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: South Perris FS				Project No.: S168-196		Date: 8/1/2025		
Test Hole No.: I-03				Tested By: Floyd Collins				
Depth of Test Hole (D_T): 60"				USCS Soil Classification: CL				
Test Hole Dimensions (inches)				Length		Width		
Diameter (if round)= 8"		Sides (if rectangular) =						
Sandy Soil Criteria Test*								
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" (Y/N)	
1	7:50	8:20	30	33 ½	37 ½	4	N	
2								
3								
<p>*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".</p>								
Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D _o Initial Depth to Water (in.)	D _f Final Depth to Water (in.)	ΔD=ΔH Change in Water Level (in.)	Perc. Rate min./in.	I _T $\frac{\Delta H 60r}{\Delta t(r+2H)}$ Avg
1	7:50	8:20	30	33 ½	37 ½	4	7.5	
2	8:22	8:52	30	34	36	2	15	
3	8:52	9:22	30	36	38	2	15	
4	9:22	9:52	30	36	38	2	15	
5	9:53	10:23	30	36	38	2	15	
6	10:23	10:53	30	36	37 ¾	1 ¾	17.1	.28
7	10:53	11:23	30	36	37 ¾	1 ¾	17.1	.28
8	11:24	11:54	30	36	37 ¾	1 ¾	17.1	.28
9	11:54	12:24	30	36	37 ¾	1 ¾	17.1	.28
10	12:25	12:55	30	36	37 ¾	1 ¾	17.1	.28
11	12:55	1:25	30	36	37 ¾	1 ¾	17.1	.28
12	1:26	1:56	30	36	37 ¾	1 ¾	17.1	.28
13								
14								
15								
<p>COMMENTS: Sunny clear (70° to 95°) Ground dry. First two measurements didn't meet sandy soil criteria. Pre-soaked on 7/31/25.</p>								

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: South Perris FS				Project No.: S168-196		Date: 8/1/2025		
Test Hole No.: I-04				Tested By: Floyd Collins				
Depth of Test Hole (D_T): 48"				USCS Soil Classification: CL				
Test Hole Dimensions (inches)				Length		Width		
Diameter (if round)= 8"				Sides (if rectangular) =				
Sandy Soil Criteria Test*								
Trial No.	Start Time	Stop Time	Time Interval (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6" (Y/N)	
1	7:47	8:17	30	24	28	4	N	
2								
3								
<p>*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".</p>								
Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D _o Initial Depth to Water (in.)	D _f Final Depth to Water (in.)	ΔD=ΔH Change in Water Level (in.)	Perc. Rate min./in.	I _T $\frac{\Delta H 60r}{\Delta t(r+2H)}$ Avg
1	7:47	8:17	30	24	28	4	7.5	
2	8:18	8:48	30	24	26	2	15	
3	8:49	9:19	30	24	25 ¾	1 ¾	17.1	
4	9:19	9:49	30	24	25 ¾	1 ¾	17.1	.28
5	9:50	10:20	30	24	25 ¾	1 ¾	17.1	.28
6	10:20	10:50	30	24	25 ¾	1 ¾	17.1	.28
7	10:51	11:21	30	24	25 ¾	1 ¾	17.1	.28
8	11:21	11:51	30	24	25 ¾	1 ¾	17.1	.28
9	11:52	12:22	30	24	25 ¾	1 ¾	17.1	.28
10	12:22	12:52	30	24	25 ¾	1 ¾	17.1	.28
11	12:53	1:23	30	24	25 ¾	1 ¾	17.1	.28
12	1:263	1:53	30	24	25 ¾	1 ¾	17.1	.28
13								
14								
15								
<p>COMMENTS: Sunny clear (70° to 95°) Ground dry. First two measurements didn't meet sandy soil criteria. Pre-soaked on 7/31/25.</p>								

APPENDIX D – Geologic Hazard Report



GEOLOGIC HAZARDS REPORT
SOUTH PERRIS FIRE STATION CIP F077
NORTHEAST OF MURRIETA AND WATSON ROADS
CITY OF PERRIS, CALIFORNIA

Project No. 254143-1

August 6, 2025

Prepared for:

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

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

Attention: Mr. Allen Evans, P.E., G.E., Principal

Regarding: Geologic Hazards Report
South Perris Fire Station CIP F077
Northeast of Murrieta and Watson Roads
City of Perris, California
IFE Project No. S168-196

INTRODUCTION

At your request, this firm has prepared a geologic hazards report for the proposed new fire station facilities and associated appurtenances, as referenced above. The purpose of this study was to evaluate the existing geologic conditions of the property and any corresponding potential geologic and/or seismic hazards, with respect to the proposed development from a geologic standpoint. This report has been prepared utilizing the suggested "Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings" (California Geological Survey Note 48, 2022), along with the accompanying CGS Note 48 Checklist (Sydnor, 2004).

The scope of services provided for this evaluation included the following:

- **Review of available published and unpublished geologic/seismic data in our files pertinent to the site, including the provided site-specific boring logs.**
- **Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.**
- **Evaluation of the local and regional tectonic setting and historical seismic activity, including performing a site-specific CBC ground motion analysis.**
- **Preparation of this report presenting our findings, conclusions, and recommendations from a geologic standpoint.**

Accompanying Maps and Appendices

- Plate 1 - Regional Geologic Map
- Plate 2 - Site Plan
- Appendix A - Shear-Wave Survey
- Appendix B - Site-Specific Ground Motion Analysis
- Appendix C - References

PROJECT SUMMARY

Based on the information that has been provided, we understand that a new fire station and associated appurtenances, will be constructed, that will be located along the east side of Murrieta Road, north of Watson Road, in the City of Perris, California. Since this report will be appended into the geotechnical report prepared for the site by Inland Foundation Engineering, Inc. (IFE), some descriptive sections such as site description, proposed development, etc., have been purposely omitted as they are described in detail in the main geotechnical report.

No grading plans were available for this evaluation, and no subsurface exploration was performed by this firm. Our scope of services included a field reconnaissance, performing a seismic shear-wave survey, and a review of available geologic and geotechnical data in our files. This review also included the provided exploratory boring logs (drilled July 31, 2025) that were prepared by IFE, which were excavated within the proposed construction area. The approximate location of the seismic shear-wave traverse (Seismic Line SW-1) is shown on a partial modified copy of the provided Site Plan (STK, Sheet A1.0, dated July 2025), which is presented on the Site Plan, Plate 2. Photographic views of the seismic shear-wave survey traverse have been included within Appendix A for visual and reference purposes.

GEOLOGIC SETTING

The subject site is situated within a natural geomorphic province in southwestern California known as the Peninsular Ranges, which is characterized by steep, elongated ranges and valleys that trend northwesterly. This province is believed to have begun as a thick accumulation of predominantly marine sedimentary and volcanic rocks during the late Paleozoic and early Mesozoic. Following this accumulation, in mid-Cretaceous time, the province underwent a pronounced episode of mountain building. The accumulated rocks were then complexly metamorphosed and intruded by igneous rocks, known locally as the Southern California Batholith. A period of erosion followed the mountain building, and during the late Cretaceous and Cenozoic time, sedimentary and subordinate volcanic rocks were deposited upon the eroded surfaces of the batholithic and pre-batholithic rocks.

More specifically, the site is situated along the northeastern portion of the Perris Block (sub-structural block of the Peninsular Ranges), that is an eroded mass of Cretaceous and older crystalline rock. The Perris Block, approximately 20 miles by 50 miles in extent, is bounded by the San Jacinto Fault Zone to the northeast, the Elsinore Fault Zone to the southwest, the Cucamonga Fault to the northwest, and to the southeast by the fringes of the Temecula basin where the boundary is ill-defined.

The Perris Block has had a complex history, apparently undergoing relative vertical land movements of several thousand feet in response to movement on the Elsinore and San

Jacinto Fault Zones. These movements of the geologic past, in conjunction with the semi-arid climate and the weathering resistance of the rock, are responsible for the formation and preservation of ancient, generally flat-lying erosion surfaces now present at various elevations that give this region its unique geologic character.

EARTH MATERIALS

Geologic mapping of the area by the Morton (2003), as illustrated on Plate 1, indicates that the subject site is mantled by late Holocene age alluvial fan deposits (symbol Qv_{sc}, see Plate 1). These materials have been generally described as being comprised of unconsolidated gravel, sand, and silt fluvial deposits. Site-specific subsurface exploratory borings excavated by IFE (2025) within the subject construction area, indicate the earth materials to consist of highly-interbedded and thinly layered silty clay, sandy clay, sandy silt, clayey sand, silty sand, and sand, that have a wide range in grain size from fine to very coarse-grained, which are in an overall medium- to very-dense/stiff to very stiff condition, to a depth of at least 51½ feet.

FAULTING

At least forty-one major "potentially active/active" (late Quaternary) faults are within a 100-kilometer (62-mile) radius of the subject site (Blake, 1989-2000a). Of these, there are no active faults known to traverse the site based on published literature. In addition, the site is not located within a State of California "Alquist-Priolo Earthquake Fault Zone" for fault rupture hazard (California Geological Survey, 2018).

The nearest known active fault is the Glen Ivy North Fault, which is a segment of the Elsinore Fault Zone and is located approximately 9.3± miles to the southwest. The Elsinore Fault zone is a major dextral strike-slip fault zone that is part of the San Andreas Fault system and is locally comprised of at least five sections, of which is locally referred to as the Glen Ivy section. This fault forms the northeast boundary of the Santa Ana Mountains, and, together with the Temecula section, forms the Elsinore trough.

The Glen Ivy North Fault is a right-lateral, strike-slip fault, being approximately 36-kilometers in length, with an estimated maximum moment magnitude (M_w) earthquake of $M_w 6.8$ and an associated slip-rate of 5.0 ± 2.0 mm/year (C.D.M.G., 1996, Cao, et al., 2003, and Petersen, 2008). Collectively, the Elsinore Fault Zone (which includes the Whittier, Glen Ivy, Temecula, Julian, and Coyote Mountain Faults segments) is a 232-kilometer long right-lateral, strike-slip fault. When considering that a cascading effect of rupture will occur along the entire length of the Elsinore Fault Zone, the total rupture area of these combined fault segments is 3,841.7 square kilometers and has an associated Maximum Moment Magnitude (M_w) of 7.8.

GROUND MOTION ANALYSIS

According to California Geological Survey Note 48 (CGS, 2022), a site-specific ground motion analysis is required for the subject site (CBC, 2022, Section 1613A and also as required by ASCE 7-16, Chapter 21), the detailed results of which are presented within Appendix B. Additionally, a seismic shear-wave survey was conducted for this study by our firm as presented within Appendix A of this report, for purposes of determining the Site Classification and V_{S30} input values for the ground motion analysis.

Geographically, the proposed construction area is centrally located at Latitude 33.75263, Longitude -117.20584 and (World Geodetic System of 1984 coordinates). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps (OSHPD, 2025) and the California Building Code criteria (CBC, 2022), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (2017). The results of this site-specific analysis have been summarized and are tabulated below:

TABLE 1 – SUMMARY OF SEISMIC DESIGN PARAMETERS

Factor or Coefficient	Value
S_s	1.422g
S_1	0.526g
F_a	1.0
F_v	1.774
S_{DS}	0.990g
S_{D1}	0.700g
S_{MS}	1.480g
S_{M1}	1.052g
T_L	8 Seconds
MCE_G PGA	0.64g
Shear-Wave Velocity (V_{100})	1,126.9 ft/sec
Site Classification	D
Risk Category	III

GROUNDWATER

The subject site is located within the southwestern-most portion of the San Jacinto groundwater basin and more specifically, within the Menifee subbasin. The basin deposits consist of cemented sands at depth and partly cemented sands nearer to the land surface and extends to a depth of at least 445 feet (Rees, 1995). The boundary of this subbasin is generally formed by the surrounding outcrops of the underlying crystalline rocks.

Groundwater data provided by the California Department of Water Resources (2025b), indicates that the closest measures water well is located approximately 1,500± feet to the southeast (State Well No. 05S03W09E001S). The water-level data indicates that during the time period of 1995 to 2025, groundwater varied between 50 to 85± feet in depth. Another nearby well, located 2,500± feet to the southeast (Local Well Name EMWD14355) indicates that during 2011 to 2025, groundwater varied between 49 to 56± feet in depth. Based on subsurface exploratory data provided by IFE (2025), groundwater was encountered at a depth of 41± feet. The exploratory borings logs indicate that mottled soils were encountered as shallow as 35± feet in depth, which most likely represents the historic high-groundwater level.

HISTORIC SEISMICITY

A computerized search, based on Southern California historical earthquake catalogs, has been performed using the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2025a). The following table and discussion summarizes the historic seismic events (greater than or equal to M4.0) that have been estimated and/or recorded during the time period of 1800 to July 2025, within a 100-kilometer radius of the site.

TABLE 2 - HISTORIC SEISMIC EVENTS; 1800-2025 (100-kilometer radius)

<u>Richter Magnitude (M)</u>	<u>No. of Events</u>
4.0 - 4.9	489
5.0 - 5.9	60
6.0 - 6.9	15
7.0 - 7.9	2
8.0+	0

An Earthquake Epicenter Map which includes magnitudes 4.0 and greater for a 100-kilometer (62-mile) radius from the subject site, has been included below as Figure 1, for reference. This map was prepared using the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2025a) of instrumentally recorded events from the period of 1932 to July 2025, in turn overlain on Google™ Earth imagery (2025).

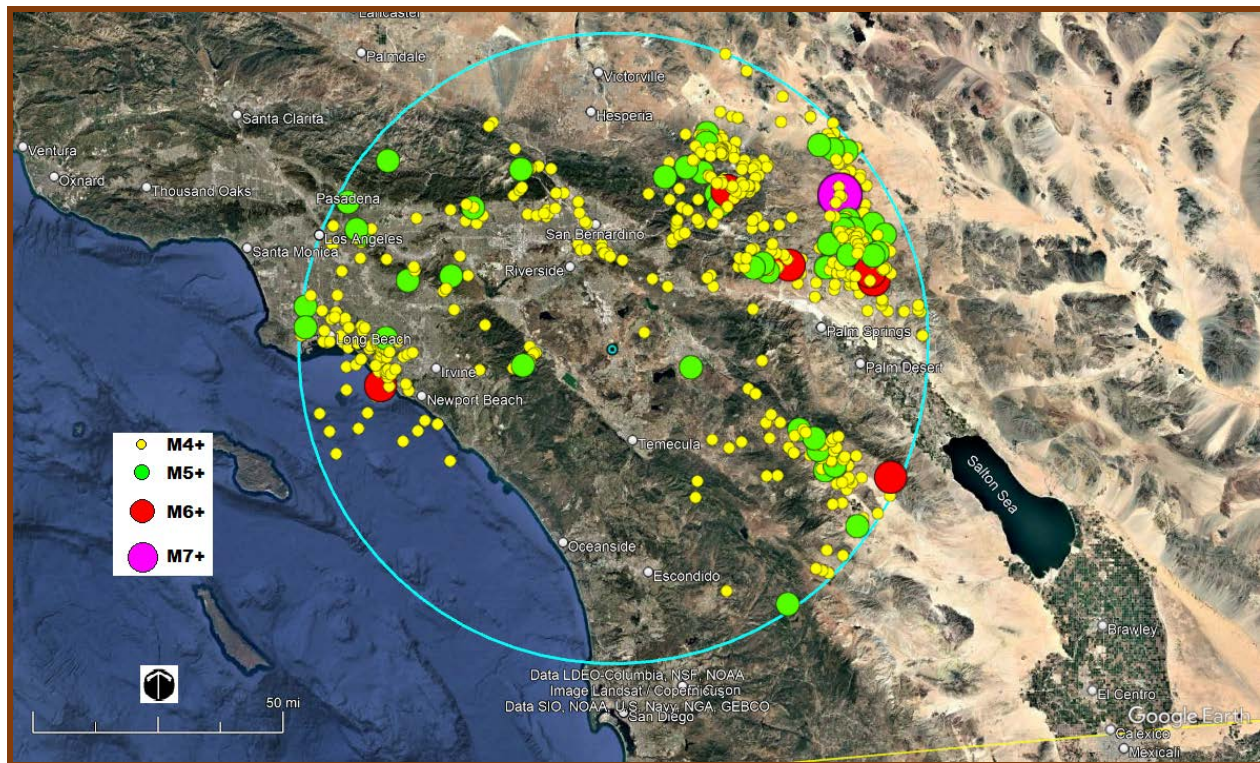


FIGURE 1- Earthquake Epicenter Map showing events of M4.0+ within a 100-kilometer radius.

It should be noted that pre-instrumental seismic events (generally before 1932) have been estimated from isoseismal maps (Toppozada, et al., 1981 and 1982). These data have been compiled generally based on the reported intensities throughout the region, thus focusing in on the most likely epicentral location. Instrumentation beyond 1932 has greatly increased the accuracy of locating earthquake epicenters. A summary of the historic earthquake data is as follows:

- ❑ The largest estimated historical earthquake magnitude (pre-1932) within a 62-mile radius is the M7.5 event of December 8, 1812 (approximately 49 miles northwest).
- ❑ The largest recorded historical earthquake was the M7.6 (M_w 7.3) Landers event, located approximately 54 miles to the northeast (June 28, 1992).
- ❑ The nearest estimated significant earthquake epicenter (pre-1932) was located approximately 13 miles east, being a M6.8 event (April 21, 1918).
- ❑ The nearest recorded significant historic earthquake epicenter was approximately 15½ miles southeast of the site (September 23, 1963, M5.3).
- ❑ The largest estimated ground acceleration estimated to have been experienced at the site was 0.255g, which resulted from the M6.8 earthquake of April 21, 1918, as noted above, based on the attenuation relationship of Boore et al. (1997).

FLOODING

According to the Federal Emergency Management Agency (FEMA), the subject site is shown to be located within the boundaries of a 100-year flood (Community Panel No. 06065C 1440H, August 18, 2014). More specifically, the site is shown to be located within "Zone AE", which is defined as "The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights". Additionally, based on the City of Perris General Plan (2021), the site is shown to be located within the limits of a "Floodway", within the mapped 100-Year Flood (Figure S-3 - FEMA Flood Hazards Zone).



FIGURE 2- Flood Insurance Rate Map (FEMA, 2014), Site outlined in red.

SECONDARY SEISMIC HAZARDS

Secondary permanent or transient seismic hazards that are generally associated with severe ground shaking during an earthquake include ground rupture, liquefaction, seiches or tsunamis, flooding (water storage facility failure), ground lurching/lateral spreading, landsliding, rockfalls, and seismically-induced settlement. These hazards are discussed below.

Ground Rupture:

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

Landsliding:

Due to the low-lying relief of the site and adjacent areas, landsliding due to seismic shaking is considered nil. Additionally, based on the City of Perris General Plan (2021), the site is not located within a zone of potential landsliding (Figure S-7, Landslide Susceptibility).

Seiches/Tsunamis:

Based on the far distance of large, open bodies of water and the elevation of the site with respect to sea level, the possibility of seiches/tsunamis is considered nil. Additionally, mapping by the California Geological Survey (2014) does not indicate the site to be located within a tsunami inundation zone.

Liquefaction:

In general, liquefaction is a phenomenon that occurs where there is a loss of strength or stiffness in the soils that can result in the settlement of buildings, ground failures, or other hazards. The main factors contributing to this phenomenon are: 1) cohesionless, granular soils having relatively low densities (usually of Holocene age); 2) shallow ground water (generally less than 50 feet); and 3) moderate-high seismic ground shaking. The City of Perris General Plan (2021) indicates the site to be located within a zone of moderate liquefaction susceptibility (Figure S-6; Earthquake Faults and Liquefaction Susceptibility). Additionally, groundwater was encountered by IFE during the on-site subsurface exploration at a depth of 41± feet. It was also noted that mottled soils were present as shallow as 35± feet, most likely indicating historic high-groundwater levels. Based on the shallow groundwater conditions and the presence of unconsolidated fine-grained alluvial sediments, the potential for liquefaction to occur appears to be possible.

Flooding (Water Storage Facility Failure)-

According to the California Department of Water Resources (2025a) "California Dam Inundation Maps", the subject site is shown to be located within a flood inundation zone, originating from a full dam breach failure (storm-induced) of the Lake Hemet Dam, which is located approximately 29½± miles to the east-southeast. The location of the site with respect to the flood limits is shown on the Dam Inundation Map, Figure 3 below. This area is indicated to have a Floodwave Maximum Elevation of 1429± feet, which is around 13± feet above the current site elevation (approximately 1,416± feet). This figure is a partial modified copy of Sheet 7 of 7 of the Lake Hemet Dam Inundation Map.



FIGURE 3- Dam Inundation Map (C.D.W.R., 2025a); Site outlined in green.

In addition, The City of Perris General Plan (2021) indicates the site to be located within a Dam Inundation Zone associated with the Perris Dam, located approximately $6\frac{1}{2}\pm$ miles to the north (Figure S-4; Dam Inundation Zones). According to the United States Army Corp of Engineers (2025), in the event of a catastrophic failure (Sunny Day Breach) of the main Perris Dam, the site can be expected to be covered by 6-15 feet of water, as illustrated below in Figure 4.



FIGURE 4- Dam Inundation Map (U.S.A.C.E., 2025); Site outlined in red.

Rockfalls:

Since no large rock outcrops are present at or adjacent to the site, the possibility of rockfalls during seismic shaking is nil

Seismically-Induced Settlement:

Seismically-induced settlement generally occurs within areas of loose granular soils. Since the subject site is underlain by generally medium-dense to very-dense and/or stiff to very stiff alluvial deposits, the potential for settlement is considered to be low.

Ground Lurching/Lateral Spreading-

Ground lurching is the horizontal movement of soil, sediments, or fill located on relatively steep embankments or scarps as a result of seismic activity, forming irregular ground surface cracks. The potential for lateral spreading or lurching is highest in areas underlain by soft, saturated materials, especially where bordered by steep banks or adjacent hard ground. Due to the relatively flat-lying nature of the site, distance from embankments, and dense underlying sediments, the potential for ground lurching and/or lateral spreading is nil.

OTHER GEOLOGIC HAZARDS

There are other potential geologic hazards not necessarily associated with seismic activity that occur statewide. These hazards include, but are not limited too; natural hazardous materials (such as methane gas, hydrogen-sulfide gas, and tar seeps); Radon-222 gas (EPA, 1993 and CGS, 2022b); naturally occurring asbestos; volcanic hazards (Martin, 1982); and regional subsidence. Of these hazards, there are none that appear to impact the site.

CONCLUSIONS AND RECOMMENDATIONS**General:**

Based on our review of available pertinent published and unpublished geologic/seismic literature, construction of the proposed fire station appears to be feasible from a geologic standpoint, providing our recommendations are considered during planning and construction.

Conclusions:

1. Based on available published geologic data, the subject site is shown to be underlain by late Holocene age fluvial deposits, generally described as being comprised of unconsolidated gravel, sand, and silt. Subsurface exploratory borings performed by IFE indicate the earth materials to consist of highly-

interbedded and thinly layered silty clay, sandy clay, sandy silt, clayey sand, silty sand, and sand, that have a wide range in grain size from fine to very coarse-grained, which are in an overall medium- to very-dense/stiff to very stiff condition, to a depth of at least 51½ feet.

2. Groundwater was encountered within the exploratory excavations at a depth of at 41± feet. Additionally, mottled soils were noted to be present as shallow as 35± feet, indicating possible historic high-ground water levels. Based on available published water-level data, measured groundwater levels indicate that groundwater has been as shallow locally as 49-50± feet in depth. No shallow groundwater conditions are anticipated to be encountered during construction.
3. No active faults are known to traverse the subject site, based on published literature. In addition, the existing site is not located within a designated Alquist-Priolo Earthquake Fault Zone for fault rupture hazards. The nearest "known" active fault is the Glen Ivy North Fault (one of the five segments of the greater Elsinore Fault Zone) which is located approximately 9.3± miles to the southwest. This fault has an associated maximum moment magnitude (M_w) earthquake of M_w 6.8, but when considering a cascading effect along all segments of the Elsinore Fault Zone, a maximum moment magnitude (M_w) earthquake of M_w 7.8 is considered possible.
4. The primary geologic hazard that exists at the site is that of ground shaking, which accounts for nearly all earthquake losses. Moderate to severe ground shaking could be anticipated during the life of the proposed facilities.
5. Based on our study and review of available published literature, there appears to a potential for flooding to occur (dam inundation) in the event of catastrophic failure of the Perris Dam, located approximately 6½± miles to the northeast. Additionally, the site is also located within flood inundation limits associated with Lake Hemet Dam, located approximately 29½± miles to the east-southeast. No other permanent and/or transient secondary seismic hazards are expected to occur within the proposed construction area.
6. The subject site is also located within a floodway path associated with a 100-year flood.

Recommendations:

1. The potential for flooding occurring as a result of dam inundation resulting from the catastrophic failure of both Perris and Lake Hemet Dams, in addition to a 100-year flood, should be properly evaluated by the project Civil Engineer. Appropriate site-specific mitigation measures, with respect to flooding potentials, should be implemented as recommended, if warranted.

2. It is recommended that all structures be designed to at least meet the current California Building Code provisions in the latest 2022 CBC edition and the 2016 ASCE Standard 7-16, where applicable. However, it should be noted that the building code is intended as a minimum construction design and is often the maximum level to which structures are designed. Structures that are built to minimum code are designed to at least remain operational after an earthquake. It is the responsibility of both the property owner and project structural engineer to determine the risk factors with respect to using CBC minimum design values for the proposed facilities.
3. For seismic design purposes, it is recommended that the design earthquake magnitudes, with respect to multi-segment rupture potentials, be taken into account when considering that a cascading effect of rupture will occur along the entire length of the Elsinore Fault Zone. Such an event should use a design earthquake magnitude of M_w 7.8.

CLOSURE

Our conclusions and recommendations are based on a review of available existing geologic/seismic data and the provided site-specific subsurface exploratory boring logs. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted. It is assumed that all the conclusions and recommendations outlined in this report are understood and followed. If any portion of this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

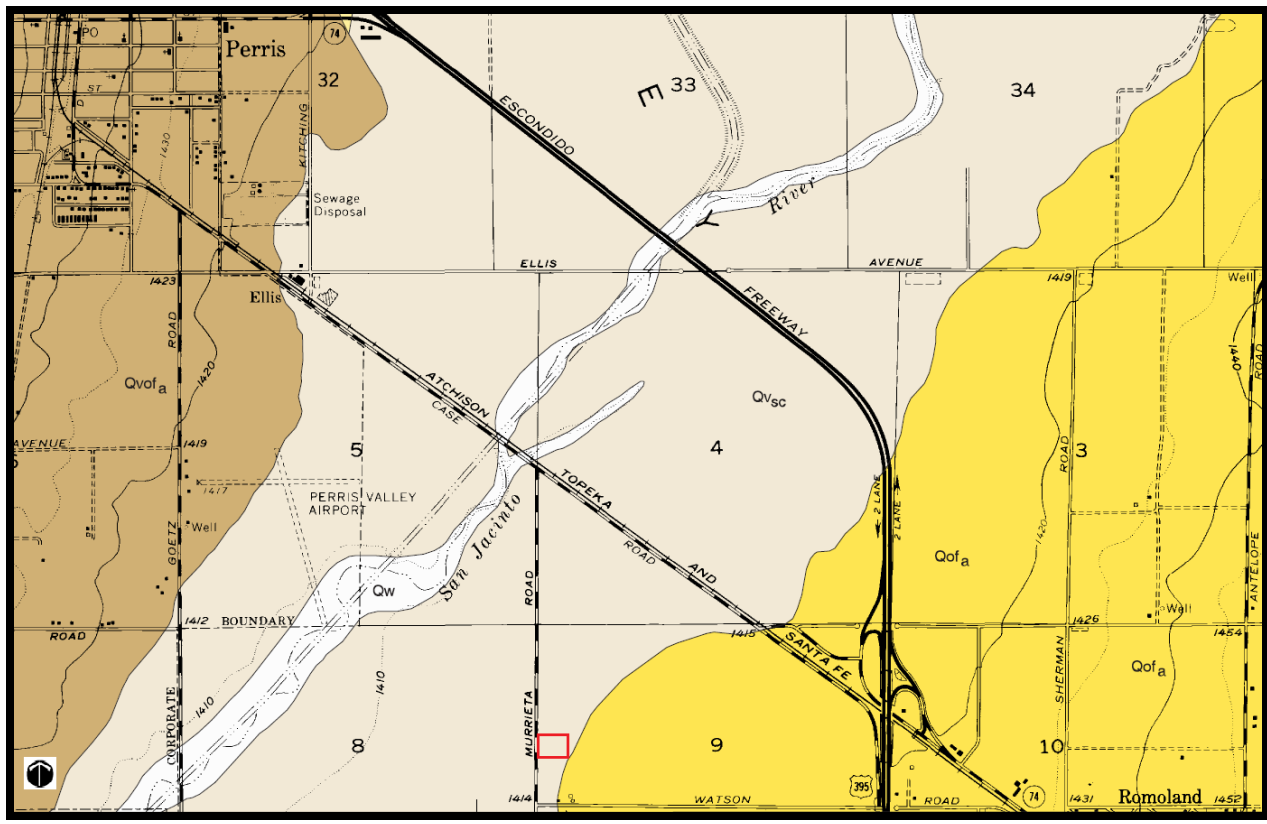
Respectfully submitted,
TERRA GEOSCIENCES



Donn C. Schwartzkopf
Principal Geologist / Geophysicist
CEG 1459 / PGP 1002

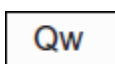
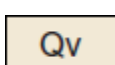
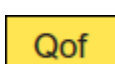
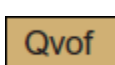



REGIONAL GEOLOGIC MAP

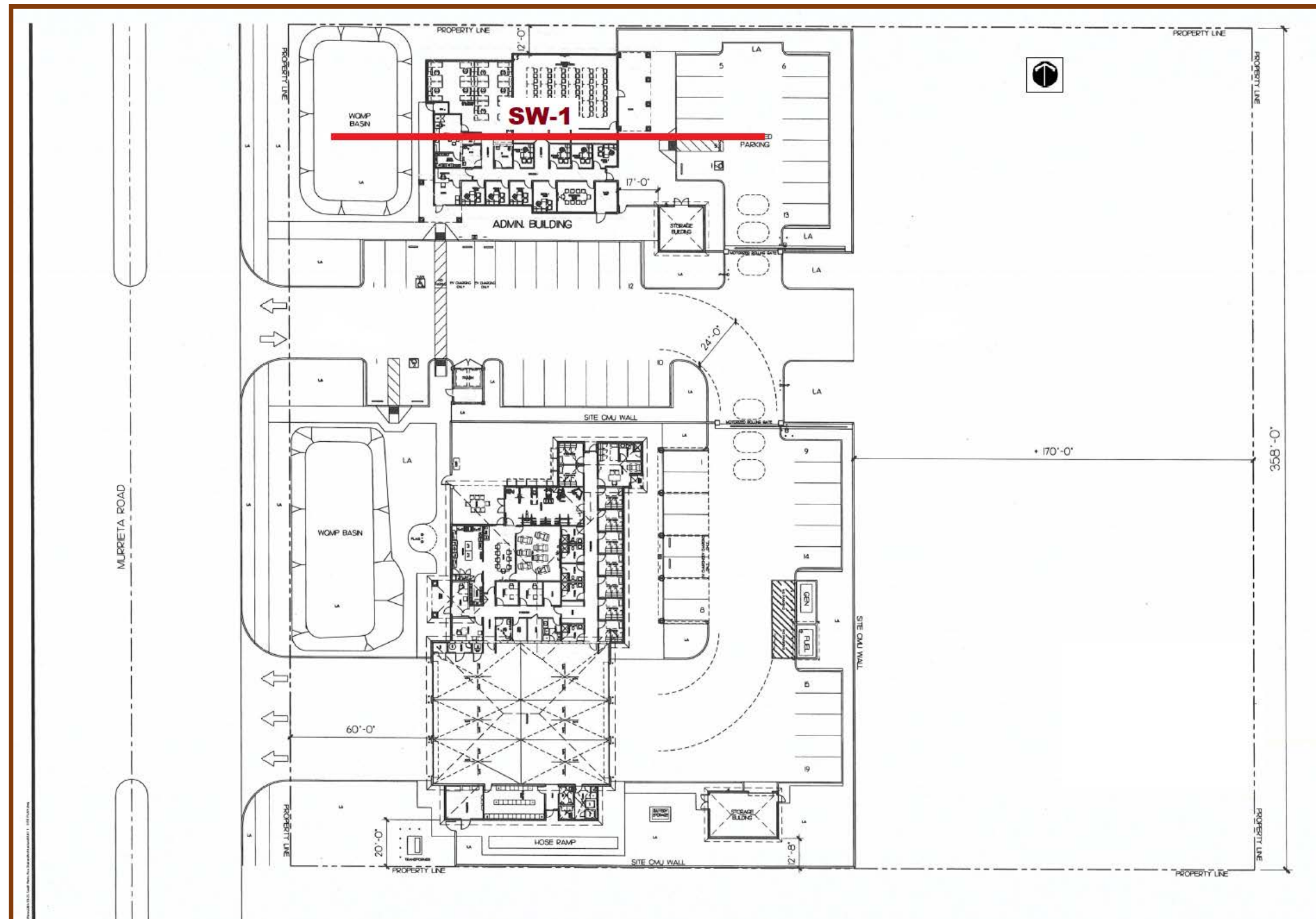


BASE MAP: Morton (2003), U.S.G.S. Open-File Report 03-270, Scale 1: 24,000; Site outlined in red.

PARTIAL LEGEND

	WASH DEPOSITS	Unconsolidated sand and gravel in ephemeral river channel (Late Holocene).
	VALLEY DEPOSITS	Unconsolidated gravel, sand, and silt fluvial deposits (Late Holocene).
	OLDER ALLUVIUM	Indurated reddish-brown sandy alluvial fan deposits (late to middle Pleistocene).
	OLDER FAN DEPOSITS	Well dissected and indurated reddish-brown sand deposits (early Pleistocene).
	GEOLOGIC CONTACT	Solid where located within 15±-meters, dashed where located within 30±-meters.

SITE PLAN



BASE MAP: Partial modified copy of the provided "Site Plan", prepared by STK (Sheet A1.0; dated July 2025); Shear-wave traverse **SW-1** shown as red line.

APPENDIX A

SHEAR-WAVE SURVEY



SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources.

For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_s) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One shear-wave survey traverse (SW-1) was performed in the northern portion of the subject site, as approximated on Plate 2. For data collection, the field survey employed a twenty-four channel Geometrics Geode model signal-enhancement refraction seismograph. This survey employed both active source (MASW) and passive (MAM) methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005).

Both the MASW and MAM survey lines used the same linear geometry array that consisted of a 184-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals. For the active source MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two separate seismic records were obtained using a 30-foot shot offset at both ends of the line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Numerous seismic impacts were used at each shot location to improve the signal-to-noise ratio.

The MAM survey did not require the introduction of any artificial seismic sources with only background ambient noise (i.e., air and vehicle traffic, etc.) being necessary. These ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 20 separate seismic records being obtained for quality control purposes. The frequency spectrum data that was displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Reduction

For analysis and presentation of the shear-wave profile and supportive illustration, this study used the **SeisImager/SW™** computer software program that was developed by Geometrics, Inc. (2021). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys.

However, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies and this model should be considered as an approximation. Processing of the data then proceeded by calculating the dispersion curve from the input data from both the active and passive data records, which were subsequently combined creating an initial shear-wave (V_s) model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final V_s curve as presented within this appendix.

Summary of Data Analysis

Data acquisition went very smoothly and the quality was considered to be good. Analysis revealed that the average shear-wave velocity (“weighted average”) in the upper 100 feet of the subject survey area is **1,126.9** feet per second as shown on the shear-wave model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of a Site Class “**D**” (“Stiff Soil” profile), which has a seismic velocity range from 600 to 1,200 ft/sec (ASCE, 2017; Table 20.3-1).

The “weighted average” velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100).

$$V_s = 100 / [(d_1/v_1) + (d_2/v_2) + \dots + (d_n/v_n)]$$

Where $d_1, d_2, d_3, \dots, d_n$, are the thicknesses for layers 1, 2, 3, ..., n , up to 100 feet, and $v_1, v_2, v_3, \dots, v_n$, are the seismic velocities (feet/second) for layers 1, 2, 3, ..., n . The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 228-foot profile where locally measured. The constrained data limits are represented by the dark-gray shading on the shear-wave model. The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within this appendix, for reference purposes.

SURVEY LINE PHOTOGRAPHS



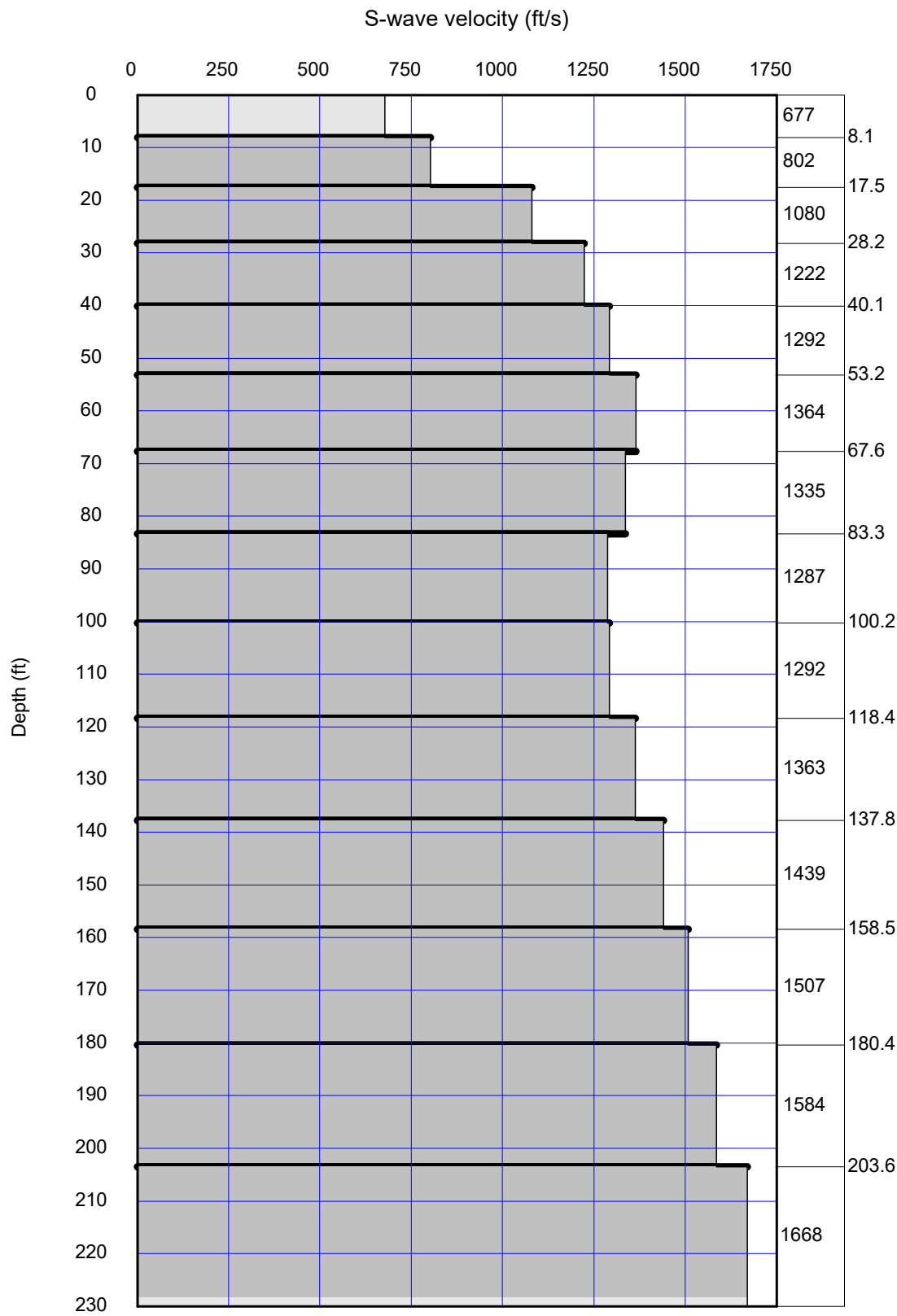
View looking west along Seismic Line SW-1.



View looking east along Seismic Line SW-1.

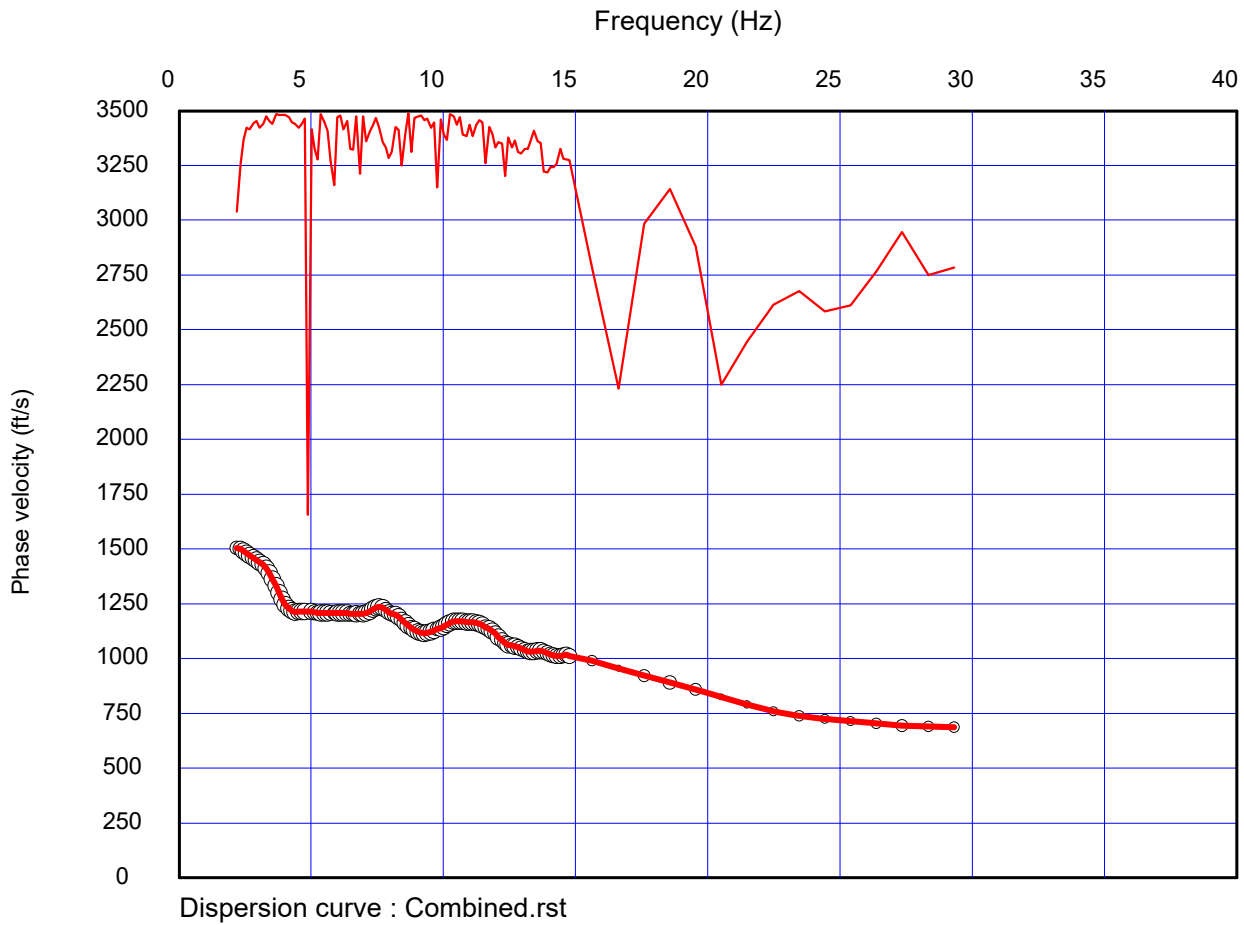
SEISMIC LINE SW-1

SHEAR-WAVE MODEL



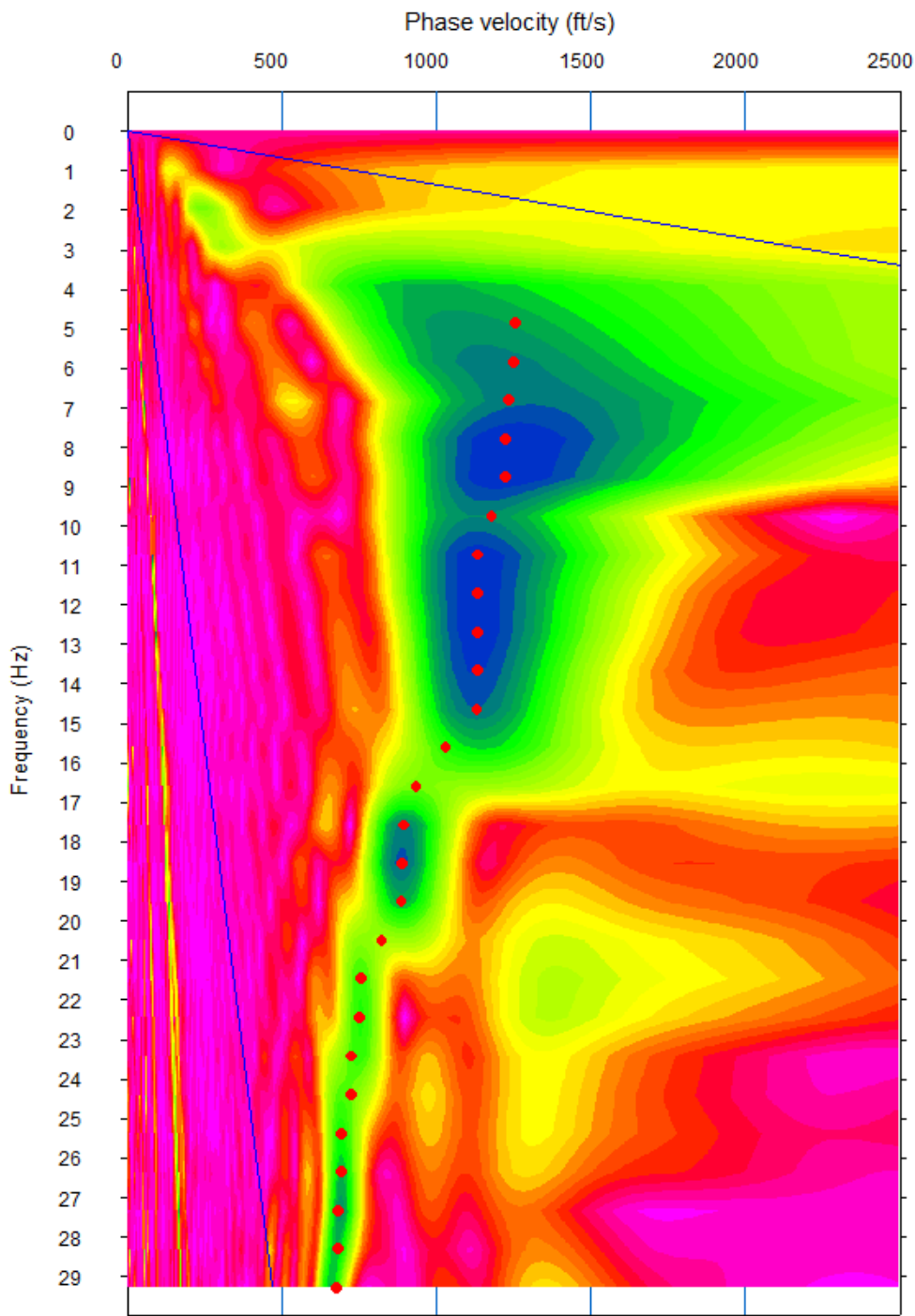
S-wave velocity model (initial): Final.rst **Average Vs 100ft = 1126.9 ft/sec**

SEISMIC LINE SW-1



COMBINED DISPERSION CURVE

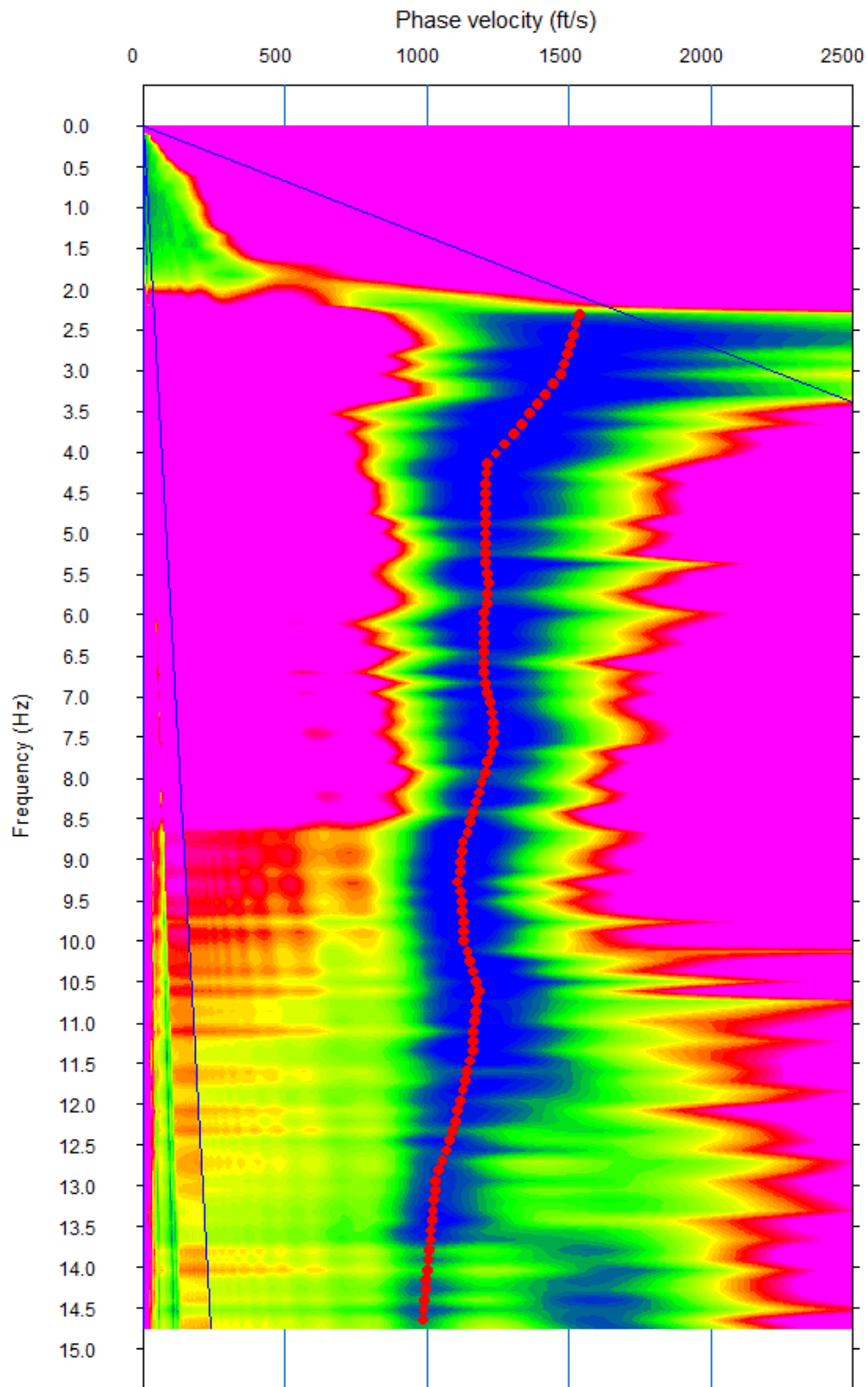
SEISMIC LINE SW-1



Dispersion Curve: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



Dispersion Curve: Passive.dat

PASSIVE DISPERSION CURVE

APPENDIX B

SITE-SPECIFIC GROUND MOTION ANALYSIS



SITE-SPECIFIC GROUND MOTION ANALYSIS

A detailed summary of the site-specific ground motion analysis, which follows Section 21 of the ASCE Standard 7-16 (2017) and the 2022 California Building Code is presented below, with the Seismic Design Parameters Summary included within this appendix following the summary text.

◆ **Mapped Spectral Acceleration Parameters (CBC 1613A.2.1)-**

Based on maps prepared by the U.S.G.S (Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping; Site Class B/C), a value of **1.422g** for the 0.2 second period (S_s) and **0.526g** for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1613A.2.1).

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

Based on the site-specific measured shear-wave value of 1,126.9 feet/second (343.5 m/sec), the soil profile type used should be Site Class “D.” This Class is defined as having the upper 100 feet (30 meters) of the subsurface being underlain by “Stiff Soil” with average shear-wave velocities of 600 to 1,200 feet/second (180 to 360 meters/second), as detailed within Appendix A.

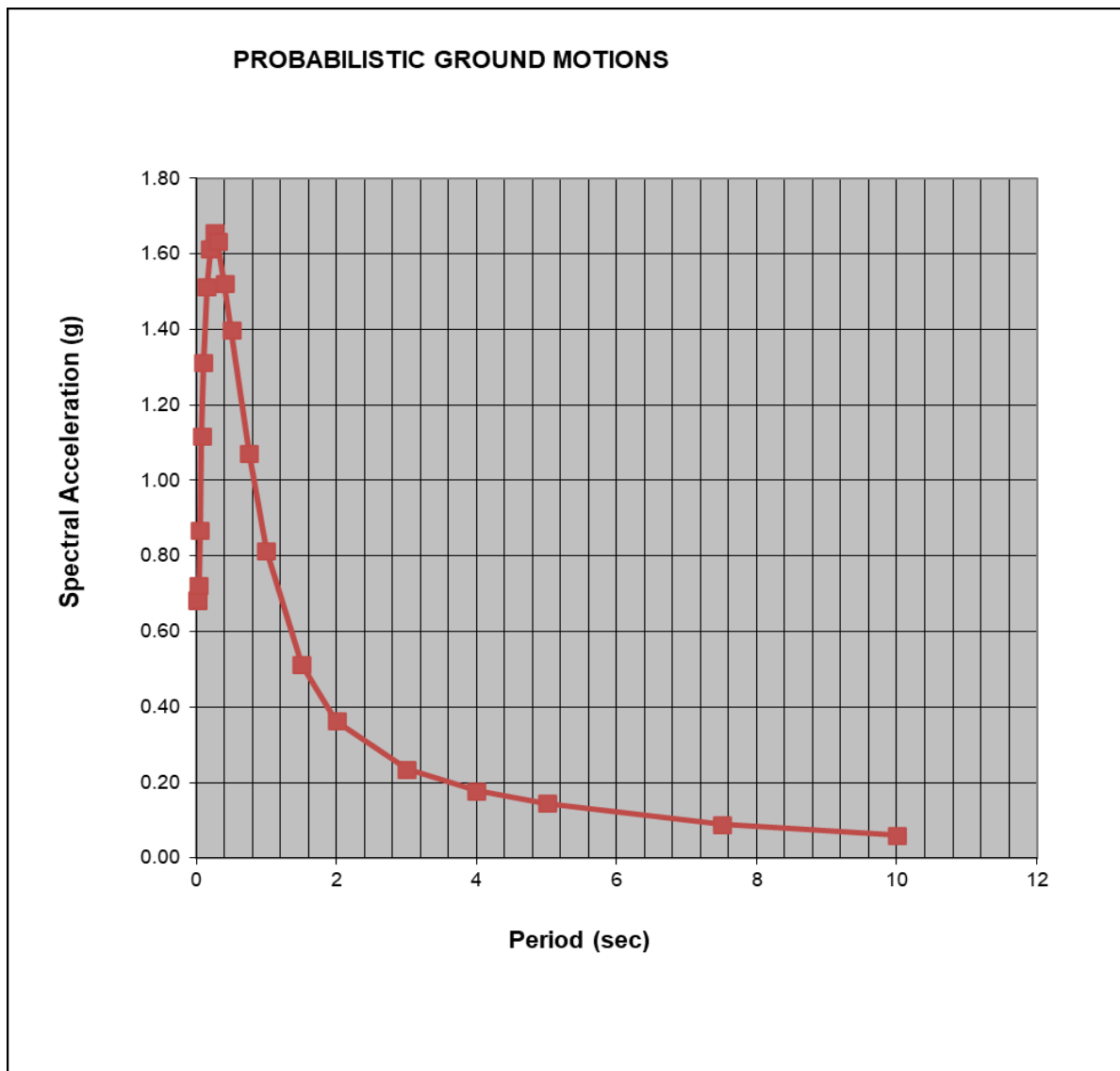
◆ **Site Coefficients (CBC 1613A.2.3)-**

Based on CBC Tables 1613A.2.3(1) and 1613A.2.3(2), the site coefficient $F_a = 1.0$ and $F_v = 1.774$, respectively.

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

Per Section 21.2.1.1 (**Method 1**), the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

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



◆ **Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)-**

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

Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013), published geologic data, and based on the length of the combined individual segments of the Elsinore Fault Zone (controlling Fault), which is located approximately 9.3 miles to the southwest. The moment magnitude (M_W) used for this fault was 7.8 (using combined segments).

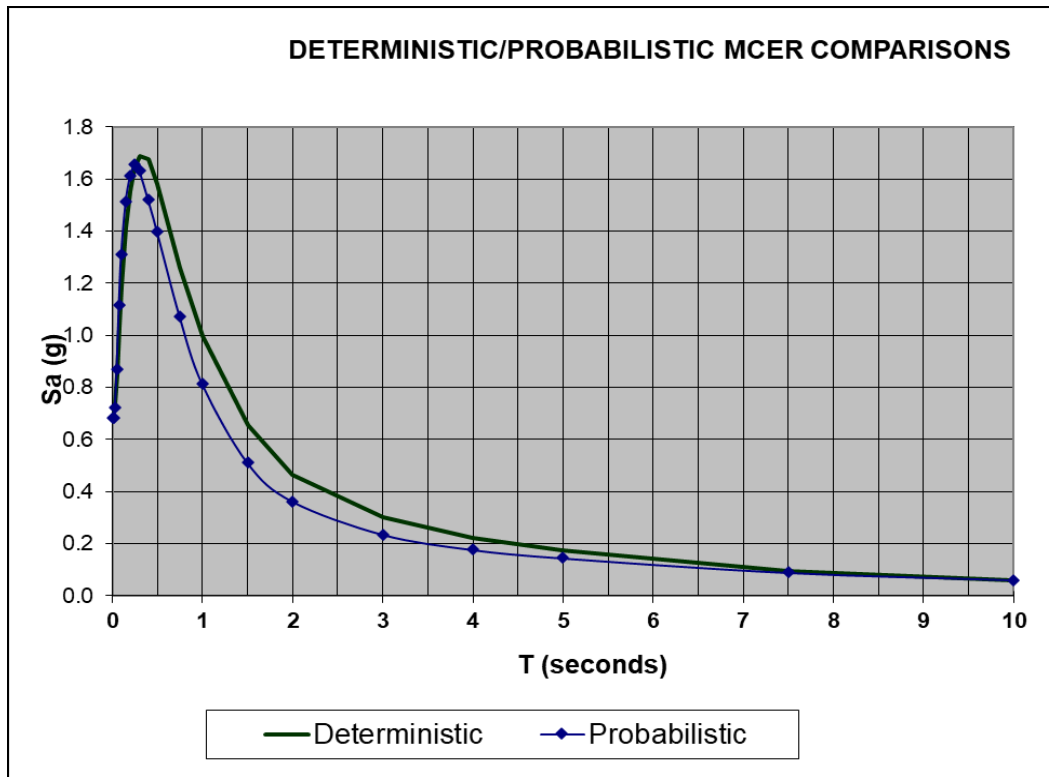
◆ **Site Specific MCE_R (ASCE 7 Section 21.2.3)-**

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

Comparison of Deterministic MCE_R Values with Probabilistic MCE_R Values - Section 21.2.3

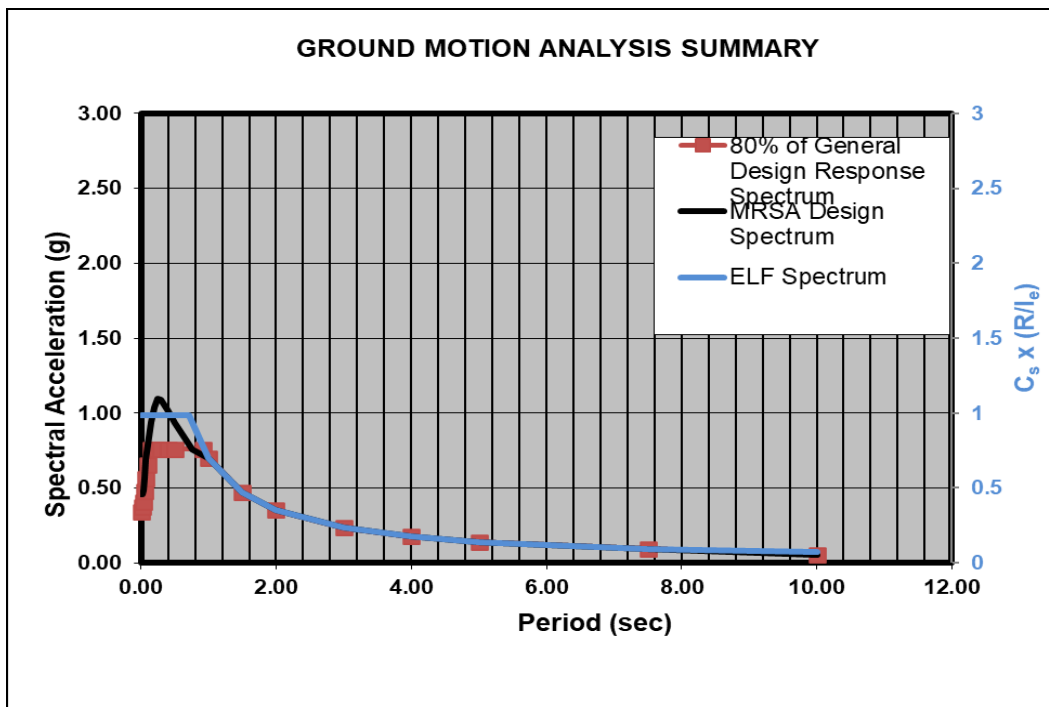
Period	Deterministic	Probabilistic	Lower Value (Site Specific MCE_R)	Governing Method
T	MCE_R	MCE_R		
0.010	0.71	0.68	0.68	Probabilistic Governs
0.020	0.71	0.68	0.68	Probabilistic Governs
0.030	0.74	0.72	0.72	Probabilistic Governs
0.050	0.85	0.87	0.85	Deterministic Governs
0.075	1.03	1.12	1.03	Deterministic Governs
0.100	1.19	1.31	1.19	Deterministic Governs
0.150	1.41	1.51	1.41	Deterministic Governs
0.200	1.55	1.61	1.55	Deterministic Governs
0.250	1.64	1.66	1.64	Deterministic Governs
0.300	1.69	1.63	1.63	Probabilistic Governs
0.400	1.68	1.52	1.52	Probabilistic Governs
0.500	1.58	1.40	1.40	Probabilistic Governs
0.750	1.26	1.07	1.07	Probabilistic Governs
1.000	1.00	0.81	0.81	Probabilistic Governs
1.500	0.66	0.51	0.51	Probabilistic Governs
2.000	0.47	0.36	0.36	Probabilistic Governs
3.000	0.30	0.24	0.24	Probabilistic Governs
4.000	0.22	0.18	0.18	Probabilistic Governs
5.000	0.17	0.15	0.15	Probabilistic Governs
7.500	0.10	0.09	0.09	Probabilistic Governs
10.000	0.06	0.06	0.06	Deterministic Governs

These comparisons are plotted in the following diagram:



◆ **Design Response Spectrum (ASCE 7 Section 21.3)-**

In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3 S_{aM}$, where S_{aM} is the MCE_R spectral response acceleration obtained from Section 21.1 or 21.2. The design spectral response acceleration shall not be taken less than 80 percent of S_a . These are plotted and compared with 80% of the CBC Spectrum values in the following diagram:



◆ **Design Acceleration Parameters (ASCE 7 Section 21.4)-**

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

◆ **Site Specific Design Parameters -**

For the 0.2 second period (S_{DS}), a value of 0.99g was computed, based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.10g occurring at $T=0.25$ seconds. This was multiplied by 0.9 to produce a value of 0.99g making this the applicable value. A value of 0.70g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4). For the MCE_R 0.2 second period, a value of 1.480g (S_{MS}) was computed, along with a value of 1.052g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

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

The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.66g. The deterministic geometric mean peak ground acceleration (largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.64g. The site-specific MCE_G peak ground acceleration was calculated to be **0.64g**, which was determined by using the lesser of the probabilistic (0.66g) or the deterministic (0.64g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., $0.55g \times 0.80 = 0.44g$).

SEISMIC DESIGN PARAMETERS SUMMARY

Project: South Perris Fire Station
Project #: 254143-1
Date: 8/2/2025

Latitude: 33.75263
Longitude: -117.20584

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s	=	1.422	Figure 22-1
S_1	=	0.526	Figure 22-2

Site Class per Table 20.3-1

Site Class	=	D
------------	---	---

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a	=	1	Table 11.4-1	=	1	For Site Specific Analysis per ASCE7-16 21.3
F_v	=	1.774	Table 11.4-2	=	2.50	For Site Specific Analysis per ASCE7-16 21.3

Mapped Design Spectral Response Acceleration Parameters

S_{MS}	=	1.422	Equation 11.4-1	=	1.422	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}	=	0.933	Equation 11.4-2	=	1.315	For Site Specific Analysis per ASCE7-16 21.3

S_{DS}	=	0.948	Equation 11.4-3
S_{D1}	=	0.622	Equation 11.4-4

Period (T)	S_a (ASCE7-16 - 11.4.6)	80% General Design Spectrum	80% Modified Design Spectrum
0.01	0.38	0.30	0.33
0.13	0.95	0.76	0.60
0.20	0.95	0.76	0.76
0.66	0.95	0.76	0.76
0.70	0.89	0.71	0.76
0.80	0.78	0.62	0.76
0.90	0.69	0.55	0.76
1.00	0.62	0.50	0.70
1.10	0.57	0.45	0.64
1.20	0.52	0.41	0.58
1.30	0.48	0.38	0.54
1.40	0.44	0.36	0.50
1.50	0.41	0.33	0.47
1.60	0.39	0.31	0.44
1.70	0.37	0.29	0.41
1.80	0.35	0.28	0.39
1.90	0.33	0.26	0.37
2.00	0.31	0.25	0.35
3.00	0.21	0.17	0.23
4.00	0.16	0.12	0.18
5.00	0.12	0.10	0.14
7.50	0.08	0.07	0.09
10.00	0.05	0.04	0.06

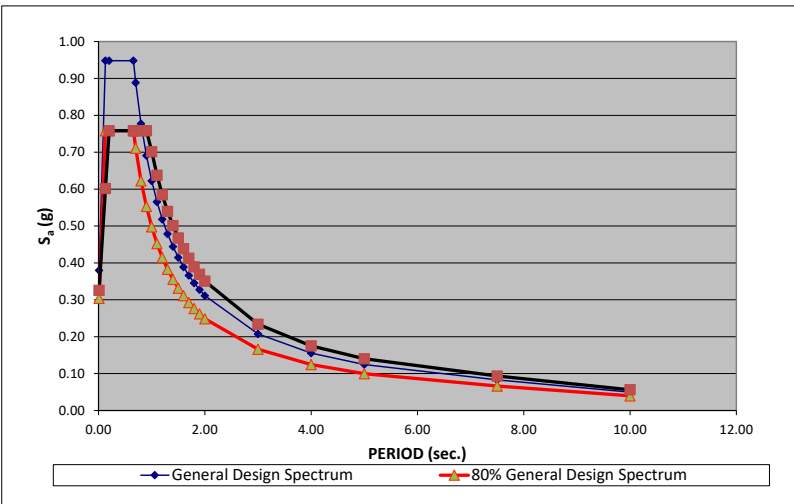
T_0	=	0.131	sec
T_S	=	0.656	sec
T_L	=	8	sec
PGA	=	0.5	g
F_{PGA}	=	1.1	
C_{RS}	=	0.938	
C_{R1}	=	0.92	

From Fig 22-12

From Table 11.8-1

Figure 22-17

Figure 22-18



ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

Use Maximum Rotated Horizontal Component?* (Y/N)

Y

Maximum Rotated Horizontal Component determined per ASCE7-16 (Huang et al. 2008)

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014), Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

PROBABILISTIC MCER per 21.2.1.1

Method 1

Earthquake Rupture Forecast - UCERF3 FM 3.2

Risk Coefficients taken from Figures 22-18 and 22-19 of ASCE 7-16

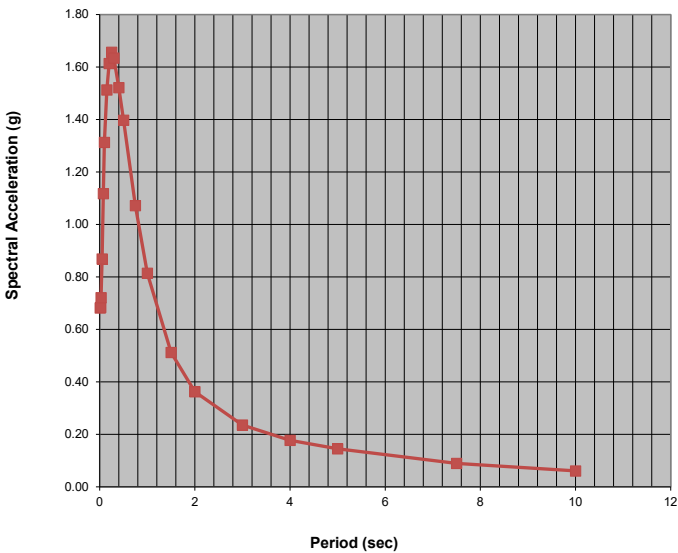
OpenSHA data

2% Probability Of Exceedance in 50 years

T	Sa in 50	C _R *	MCER
0.01	0.73	0.9380	0.68
0.02	0.73	0.9380	0.68
0.03	0.77	0.9380	0.72
0.05	0.93	0.9380	0.87
0.08	1.19	0.9380	1.12
0.10	1.40	0.9380	1.31
0.15	1.61	0.9380	1.51
0.20	1.72	0.9380	1.61
0.25	1.77	0.9369	1.66
0.30	1.75	0.9358	1.63
0.40	1.63	0.9335	1.52
0.50	1.50	0.9313	1.40
0.75	1.16	0.9256	1.07
1.00	0.88	0.9200	0.81
1.50	0.56	0.9200	0.51
2.00	0.39	0.9200	0.36
3.00	0.26	0.9200	0.24
4.00	0.19	0.9200	0.18
5.00	0.16	0.9200	0.15
7.50	0.10	0.9200	0.09
10.00	0.07	0.9200	0.06

* The risk coefficient C_r is interpolated between T=0.2 and 1.0 seconds. Where T<0.2 C_r=C_{RS}. Where T>1.0, C_r=C_{R1}

PROBABILISTIC GROUND MOTIONS



S _s =	1.72	1.61
S ₁ =	0.88	0.81
PGA	0.66 g	

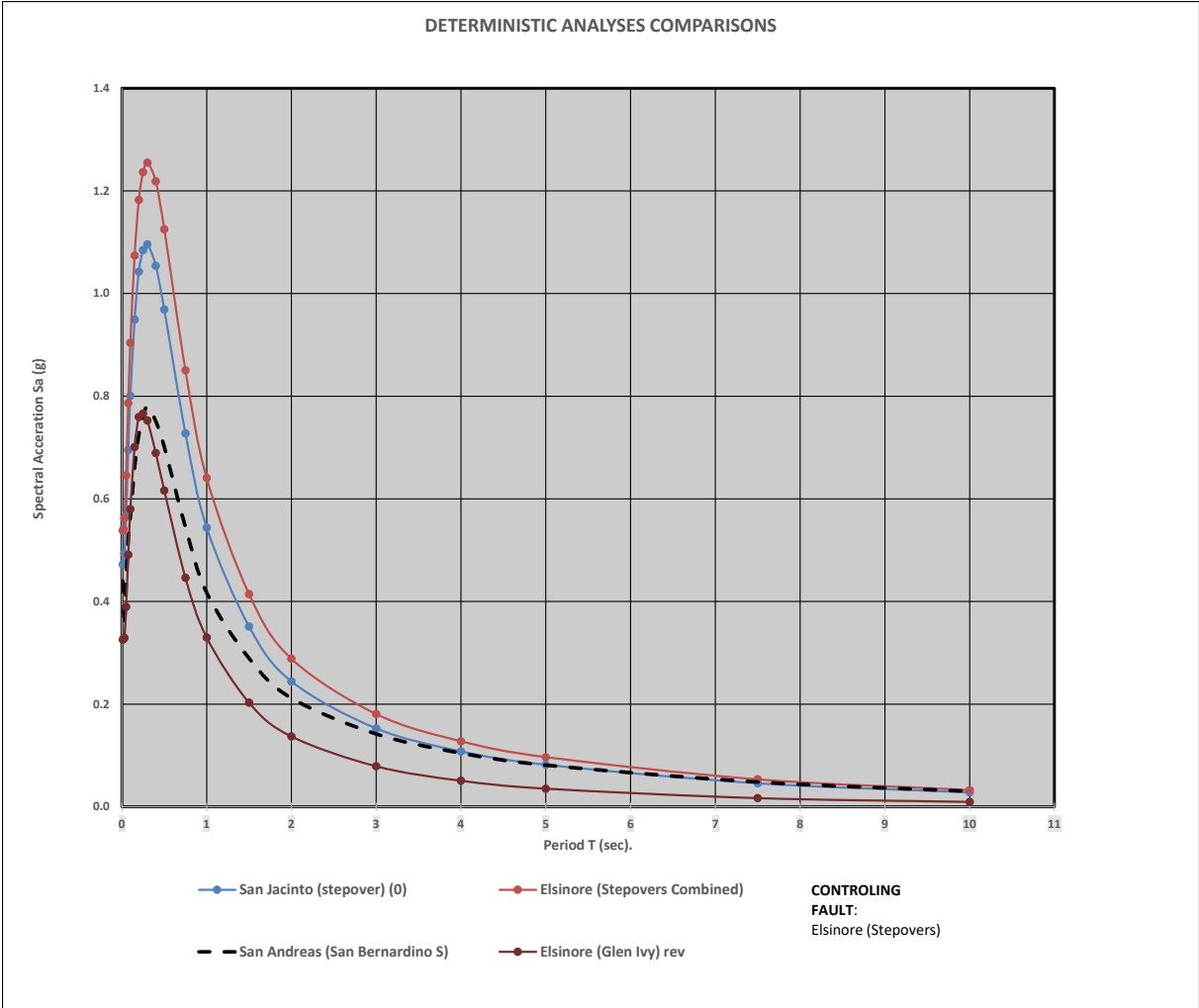
Risk Coefficients:			
C _{RS}	0.938	Figure 22-18	Get from Mapped Values
C _{R1}	0.92	Figure 22-19	
Fa=	1	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is Sa _(max) <1.2XFa?	NO		If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Preliminary Assessment:

Fault	Distance (km)
San Jacinto (stepover) (0)	19.10
Elsinore (Stepovers Combined)	15.00
San Andreas (San Bernardino S)	40.20
Elsinore (Glen Ivy) rev	20.90

The Probalistic Analyses revealed four faults that contribute more than 10% to the seismic hazard. These faults were the subject of deterministic analyses.



Input Parameters		San Jacinto (stepover) (0)	Elsinore (Stepovers Combined)	San Andreas (San Bernardino S)	Elsinore (Glen Ivy) rev
Fault					
M	= Moment magnitude	7.8	7.8	8.1	6.8
R_{RUP}	= Closest distance to coseismic rupture (km)	19.1	15	40.2	20.9
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	19.1	15	40.2	20.9
R_X	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	19.1	15	40.2	20.9
U	= Unspecified Faulting Flag (Boore et.al.)	0	0	0	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0	0	0	0
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	0	0	0	0
F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08	0	0	0	0
Z_{TOR}	= Depth to top of coseismic rupture (km)	0	0	0	0
d	= Average dip of rupture plane (degrees)	90	90	90	90
V_{S30}	= Average shear-wave velocity in top 30m of site profile	343.5	343.5	343.5	343.5
F_{Measured}		1	1	1	1
Z_{1.0}	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.05	0.05	0.2	0.2
Z_{2.5}	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	0.21	0.21	0.4	0.4
Site Class		D	D	D	D
W (km)	= Fault rupture width (km)	16.5	13.7	12.8	13.2
F_{AS}	= 0 for mainshock; 1 for aftershock	0	0	0	0
σ	=Standard Deviation	1	1	1	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

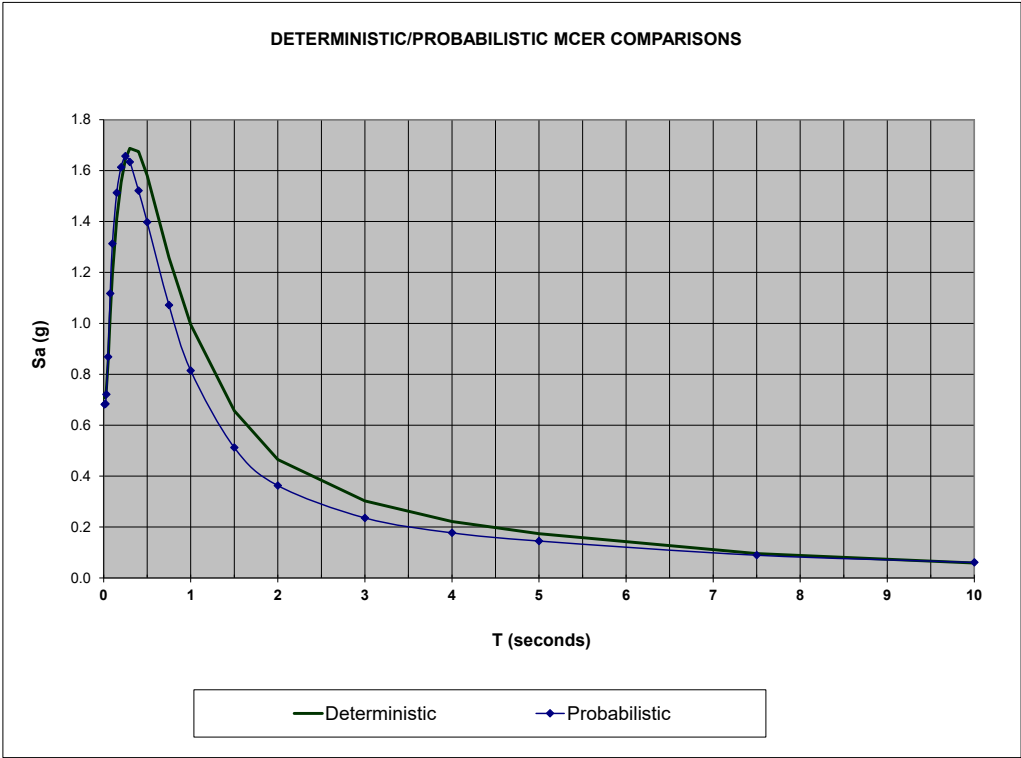
T	San Jacinto (stepover) (0)	Elsinore (Stepovers Combined)	San Andreas (San Bernardino S)	Elsinore (Glen Ivy) rev	Maximum S _a (Average)	Corrected* S _a (per ASCE7- 16)	Scaled S _a (Average)	Controlling Fault
0.010	0.47	0.54	0.44	0.33	0.54	0.59	0.71	Elsinore (Stepovers Combined)
0.020	0.47	0.54	0.39	0.33	0.54	0.59	0.71	Elsinore (Stepovers Combined)
0.030	0.49	0.56	0.35	0.33	0.56	0.62	0.74	Elsinore (Stepovers Combined)
0.050	0.57	0.65	0.44	0.39	0.65	0.71	0.85	Elsinore (Stepovers Combined)
0.075	0.70	0.79	0.52	0.49	0.79	0.87	1.03	Elsinore (Stepovers Combined)
0.100	0.80	0.90	0.56	0.58	0.90	0.99	1.19	Elsinore (Stepovers Combined)
0.150	0.95	1.07	0.66	0.70	1.07	1.18	1.41	Elsinore (Stepovers Combined)
0.200	1.04	1.18	0.73	0.76	1.18	1.30	1.55	Elsinore (Stepovers Combined)
0.250	1.08	1.24	0.76	0.77	1.24	1.38	1.64	Elsinore (Stepovers Combined)
0.300	1.10	1.25	0.78	0.75	1.25	1.41	1.69	Elsinore (Stepovers Combined)
0.400	1.05	1.22	0.75	0.69	1.22	1.40	1.68	Elsinore (Stepovers Combined)
0.500	0.97	1.13	0.70	0.62	1.13	1.32	1.58	Elsinore (Stepovers Combined)
0.750	0.73	0.85	0.54	0.45	0.85	1.05	1.26	Elsinore (Stepovers Combined)
1.000	0.54	0.64	0.42	0.33	0.64	0.83	1.00	Elsinore (Stepovers Combined)
1.500	0.35	0.41	0.29	0.20	0.41	0.55	0.66	Elsinore (Stepovers Combined)
2.000	0.24	0.29	0.21	0.14	0.29	0.39	0.47	Elsinore (Stepovers Combined)
3.000	0.15	0.18	0.14	0.08	0.18	0.25	0.30	Elsinore (Stepovers Combined)
4.000	0.11	0.13	0.10	0.05	0.13	0.19	0.22	Elsinore (Stepovers Combined)
5.000	0.08	0.10	0.08	0.04	0.10	0.15	0.17	Elsinore (Stepovers Combined)
7.500	0.05	0.05	0.05	0.02	0.05	0.08	0.10	Elsinore (Stepovers Combined)
10.000	0.03	0.03	0.03	0.01	0.03	0.05	0.06	Elsinore (Stepovers Combined)
PGA	0.47	0.54	0.34	0.31	0.54		0.64	g
Max Sa=	1.25							
Fa =	1.00	Per ASCE7-16 21.2.2						
1.5XFa=	1.5							
Scaling Factor=	1.20							

* Correction is the adjustment for Maximum Rotated Value if Applicable

SITE SPECIFIC MCE_R - Compare Deterministic MCE_R Values (S_a) with Probabilistic MCE_R Values (S_a) per 21.2.3

Presented data are the average of Chiou & Youngs (2014), Abrahamson et. al. (2014) , Boore et. al (2014) and Campbell & Bozorgnia (2014) NGA West-2 Relationships

Period	Deterministic	Probabilistic	Lower Value (Site Specific MCE _R)	Governing Method
T	MCE _R	MCE _R		
0.010	0.71	0.68	0.68	Probabilistic Governs
0.020	0.71	0.68	0.68	Probabilistic Governs
0.030	0.74	0.72	0.72	Probabilistic Governs
0.050	0.85	0.87	0.85	Deterministic Governs
0.075	1.03	1.12	1.03	Deterministic Governs
0.100	1.19	1.31	1.19	Deterministic Governs
0.150	1.41	1.51	1.41	Deterministic Governs
0.200	1.55	1.61	1.55	Deterministic Governs
0.250	1.64	1.66	1.64	Deterministic Governs
0.300	1.69	1.63	1.63	Probabilistic Governs
0.400	1.68	1.52	1.52	Probabilistic Governs
0.500	1.58	1.40	1.40	Probabilistic Governs
0.750	1.26	1.07	1.07	Probabilistic Governs
1.000	1.00	0.81	0.81	Probabilistic Governs
1.500	0.66	0.51	0.51	Probabilistic Governs
2.000	0.47	0.36	0.36	Probabilistic Governs
3.000	0.30	0.24	0.24	Probabilistic Governs
4.000	0.22	0.18	0.18	Probabilistic Governs
5.000	0.17	0.15	0.15	Probabilistic Governs
7.500	0.10	0.09	0.09	Probabilistic Governs
10.000	0.06	0.06	0.06	Deterministic Governs



DESIGN RESPONSE SPECTRUM per Section 21.3

DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

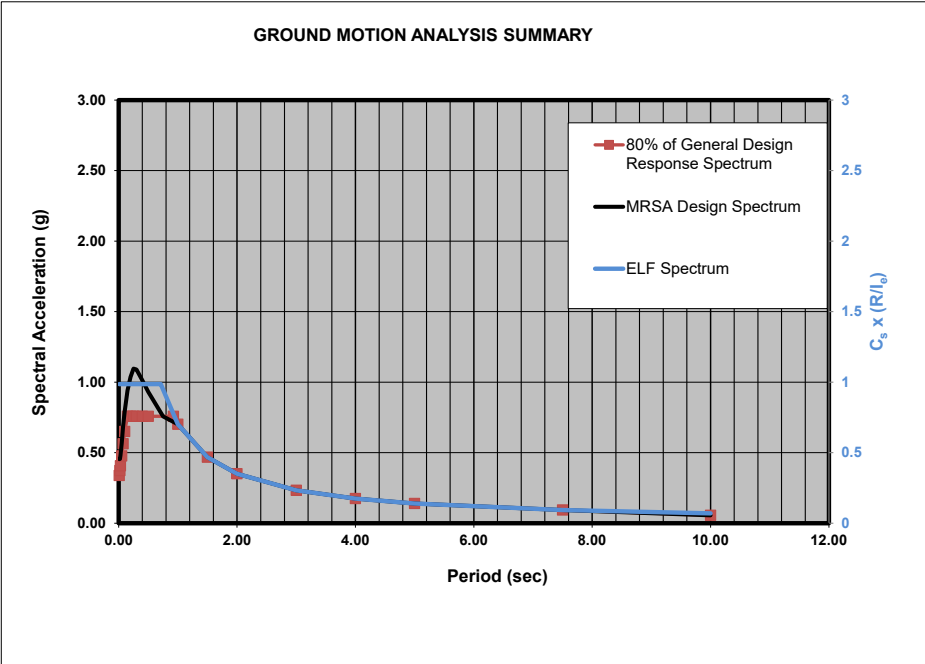
Period	2/3*MCE _R	80% Modified Design Response Spectrum (per ASCE 7-16 23.3-1)	Design Response Spectrum	TXSa
0.01	0.45	0.33	0.45	
0.02	0.46	0.35	0.46	
0.03	0.48	0.37	0.48	
0.05	0.57	0.42	0.57	
0.08	0.69	0.47	0.69	
0.10	0.79	0.53	0.79	
0.15	0.94	0.64	0.94	
0.20	1.04	0.76	1.04	
0.25	1.10	0.76	1.10	
0.30	1.09	0.76	1.09	
0.40	1.01	0.76	1.01	
0.50	0.93	0.76	0.93	
0.75	0.71	0.76	0.76	
1.00	0.54	0.70	0.70	0.70
1.50	0.34	0.47	0.47	0.70
2.00	0.24	0.35	0.35	0.70
3.00	0.16	0.23	0.23	0.70
4.00	0.12	0.18	0.18	0.70
5.00	0.10	0.14	0.14	0.70
7.50	0.06	0.09	0.09	
10.00	0.04	0.06	0.06	

Highest value of S _a for any period exceeding 0.2 sec.=	1.10
90%of Highest Value =	0.99
80% Of Mapped S _{DS} =	0.76
Maximum TXSa from T=1s-5s =	0.70
80% of Mapped S _{D1} =	0.50

S _{DS} =	0.99	S _{MS} =	1.480
S _{D1} =	0.70	S _{M1} =	1.052
Ts =	0.71		

PGA Determination:

Site Coefficient F _{PGA} =	1.1	
Mapped PGA=	0.50	Figure 22-7
PGA _M =	0.55	g
Deterministic PGA =	0.64	g
Probabilistic PGA =	0.66	g
Lesser of Deterministic/Probabilistic=	0.64	g
80% of PGA _M =	0.44	g
MCE _G PGA=	0.64	g



SUMMARY OF SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS DATA

1	2	3		4	5	6	7	8	9	10	11	12
Period (sec)	Mapped MCE _R Spectrum	Mapped Design Spectrum	Period (sec)	Risk Coefficient C _R	Scaled MCE _R Deterministic Spectrum	Probabilistic MCE _R Spectrum	Probabilistic w/Risk Coefficient C _R	84th Percentile Deterministic Spectrum	2/3 Site Specific MCE _R Spectrum	80% of Modified Design Spectrum	Site Specific MCE _R Spectrum	Design Response Spectrum
0.01	0.57	0.38	0.01	0.938	0.59	0.68	0.68	0.59	0.45	0.33	0.68	0.45
0.13	1.42	0.95	0.02	0.938	0.59	0.68	0.68	0.59	0.46	0.35	0.68	0.46
0.20	1.42	0.95	0.03	0.938	0.62	0.72	0.72	0.62	0.48	0.37	0.72	0.48
0.66	1.42	0.95	0.05	0.938	0.71	0.87	0.87	0.71	0.57	0.42	0.85	0.57
0.70	1.33	0.89	0.08	0.938	0.87	1.12	1.12	0.87	0.69	0.47	1.03	0.69
0.80	1.17	0.78	0.10	0.938	0.99	1.31	1.31	0.99	0.79	0.53	1.19	0.79
0.90	1.04	0.69	0.15	0.938	1.18	1.51	1.51	1.18	0.94	0.64	1.41	0.94
1.00	0.93	0.62	0.20	0.938	1.30	1.61	1.61	1.30	1.04	0.76	1.55	1.04
1.10	0.85	0.57	0.25	0.937	1.38	1.66	1.66	1.38	1.10	0.76	1.64	1.10
1.20	0.78	0.52	0.30	0.936	1.41	1.63	1.63	1.41	1.09	0.76	1.63	1.09
1.30	0.72	0.48	0.40	0.934	1.40	1.52	1.52	1.40	1.01	0.76	1.52	1.01
1.40	0.67	0.44	0.50	0.931	1.32	1.40	1.40	1.32	0.93	0.76	1.40	0.93
1.50	0.62	0.41	0.75	0.926	1.05	1.07	1.07	1.05	0.71	0.76	1.14	0.76
1.60	0.58	0.39	1.00	0.920	0.83	0.81	0.81	0.83	0.54	0.70	1.05	0.70
1.70	0.55	0.37	1.50	0.920	0.55	0.51	0.51	0.55	0.34	0.47	0.70	0.47
1.80	0.52	0.35	2.00	0.920	0.39	0.36	0.36	0.39	0.24	0.35	0.53	0.35
1.90	0.49	0.33	3.00	0.920	0.25	0.24	0.24	0.25	0.16	0.23	0.35	0.23
2.00	0.47	0.31	4.00	0.920	0.19	0.18	0.18	0.19	0.12	0.18	0.26	0.18
3.00	0.31	0.21	5.00	0.920	0.15	0.15	0.15	0.15	0.10	0.14	0.21	0.14
4.00	0.23	0.16	7.50	0.920	0.08	0.09	0.09	0.08	0.06	0.09	0.14	0.09
5.00	0.19	0.12	10.00	0.920	0.05	0.06	0.06	0.05	0.04	0.06	0.08	0.06
7.50	0.12	0.08										
10.00	0.07	0.05										

APPENDIX C

REFERENCES



REFERENCES

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads and Associated Criteria for Buildings and other Structures, ASCE Standard 7-16, 889pp.

Blake, T.F. 1989-2000, EQFAULT, A computer program for the deterministic prediction of peak horizontal acceleration from digitized California faults, Version 2.03, revised 1993.

Blake, T.F. 1989-2021, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from Southern California Historical Earthquake Catalog, Version 3.00b.

Boore, D.M., Joyner, W.B., and Fumal, T.E., 1997, Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work, *in*, Seismological Research Letters, Volume 68, pp. 128-153.

California Building Standards Commission (CBSC), 2022, 2022 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2.

California Department of Water Resources, 2003, California's Groundwater, DWR Bulletin 118, 2003 Update, 266 pp.

California Department of Water Resources (C.D.W.R.), 2025a, California Dam Breach Inundation Maps, <https://fmds.water.ca.gov/maps/damim/>.

California Department of Water Resources (C.D.W.R.), 2025b, California Statewide Groundwater Elevation Monitoring System (CASGEM) Online System-Public Portal, [https://www.casgem.water.ca.gov/OSS/\(S\(55ycplfqqlsga0raytn2oIn\)\)/Public/ApplicationHome.aspx](https://www.casgem.water.ca.gov/OSS/(S(55ycplfqqlsga0raytn2oIn))/Public/ApplicationHome.aspx).

California Division of Mines & Geology (C.D.M.G.), 1986, "Guidelines to Geologic/Seismic Reports," Note No. 42.

California Division of Mines & Geology (C.D.M.G.), 1996, Probabilistic Seismic Hazard Assessment for the State of California, C.D.M.G. Open File Report 96-08.

California Division of the State Architect (DSA), 2023, Geohazard Reports Requirements: 2022 CBC, Interpretation of Regulations Document IR A-4, 7 pp.

California Geological Survey (C.G.S.), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards, in California C.D.M.G. Special Publication 117.

California Geological Survey (C.G.S.), 2018, Earthquake Fault Zones: A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California, CGS Special Publication 42, 93 pp.

California Geological Survey (C.G.S), 2022a, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings, Note 48, November 2022, 2 pp.

California Geological Survey (C.G.S), 2022b, California Indoor Radon Potential, Special Report 247, 287 pp.

California's Office of Statewide Health Planning and Development (OSHPD), 2025, OSHPD Seismic Design Maps Tool web application, <https://seismicmaps.org/>.

California State Board for Geologists and Geophysicists, Department of Consumer Affairs, 1998, Guidelines for Geophysical Reports for Environmental and Engineering Geology, 5 pp.

Cao, T., Bryant, W.A., Rowshandel, B., Branum, D., and Wills, C.J., 2003, The Revised 2002 California Probabilistic Seismic Hazard Maps, June 2003, California Geological Survey.

City of Perris, 2021, City of Perris General Plan, Safety Element, 44 pp.

Dudley, Paul H., 1936, Physiographic History of a Portion of the Perris Block, Southern California, from "Journal of Geology," 1936, Volume 44, pp. 358-378.

Geometrics, Inc., 2012, Geode Operation and Reference Manual, Revision M, Software Version 9.3, San Jose, California, 226 pp.

Geometrics, Inc., 2009, SeisImager/SW™ Manual, Windows Software for Analysis of Surface Waves, Version 3.0, 314 pp.

Geometrics, Inc., 2021, SeisImager/SW Analysis of Surface Waves, Pickwin Version 6.0.2.1. and WaveEq Version 5.2.6.0.

Google™ Earth, 2025, <http://earth.google.com/>, Version 7.3.6.10201 (64-bit).

Inland Foundation Engineering, Inc., 2025, Boring Log B-1 through B-6, Project No. S168-196, dated July 31. 2025.

Louie, J.N., 2001, Faster, Better: Shear-Wave Velocity to 100 Meters Depth from Refraction Microtremor Arrays, *in*, Bulletin of the Seismological Society of America, Volume 91, pp. 347-364.

Martin, R.C., and Davis, J.F., 1982, Status of Volcanic Prediction and Emergency Response Capabilities in Volcanic Hazard Zones of California, C.D.M.G. Special Publication 63.

Okada, H., 2003, The Microtremor Survey Method, Society of Exploration Geophysicists, Geophysical Monograph Series Number 12, 135 pp.

Park, C.B, Milner, R.D., Rynden, N., Xia, J., and Ivanov, J., 2005, Combined use of Active and Passive Surface Waves, *in*, Journal of Environmental and Engineering Geophysics, Volume 10, Issue 3, pp. 323-334.

Petersen, M.D. et al., 2008, Documentation for the 2008 Update of the United States National Seismic Hazard Maps, United States Geological Survey Open-File Report 2008-1128.

Riverside County, 2025, Riverside County "Map My County", Website Version 11.9, https://gis1.countyofriverside.us/Html5Viewer/index.html?viewer=MMC_Public.

Sydnor, R.H., 2004, Engineering Geology and Seismology for Public School and Hospitals; to Accompany California Geological Survey Note 48 Checklist, 264 pp.

Toppozada, T.R., et al., 1981, Preparation of Isoseismal Maps and Summaries of Reported Effects for pre-1900 California Earthquakes, C.D.M.G. Open File Report 81-11.

Toppozada, T.R., and Parke, D.L., 1982, Areas Damaged by California Earthquakes, 1900 - 1949, C.D.M.G. Open File Report 82-17.

United States Army Corps of Engineers, 2025, National Inventory of Dams, Website Version 3.21.0; API Version 3.21.0, <https://nid.sec.usace.army.mil/#/>.

United States Environmental Protection Agency (EPA), 1993, EPA's Map of Radon Zones, California, Manual 402-R-93-025, 78 pp.

United States Geological Survey (U.S.G.S.), 2025a, OpenSHA Desktop Applications, Version 25.4.1, <http://opensha.org>.

United States Geological Survey (U.S.G.S.), 2025b, ANSS Comprehensive Earthquake Catalog (ComCat), <https://earthquake.usgs.gov/data/comcat/>.

United States Geological Survey (U.S.G.S.), 2025c, Quaternary Faults and Folds in the U.S. database, <https://earthquake.usgs.gov/static/lfs/nshm/qfaults/qfaults.kmz>.

United States Geological Survey (U.S.G.S.), 2025d, National Water Information Service: Web Interface, Groundwater Levels for the Nation, updated May 2025 <http://nwis.waterdata.usgs.gov/nwis/gwlevels?>.

Woodford, A., Shelton, J., Doehring, D., and Morton, R., 1971, Pliocene-Pleistocene History of the Perris Block, Southern California, Geological Society of America Bulletin, V. 82, pp. 3421-3448, 18 Figures, December, 1971.

Xia, J., Miller, R.D., and Park, C.B., 1999, Estimation of Near-Surface Shear-Wave Velocity by Inversion of Rayleigh Wave: Geophysics, v. 64, p.691-700.

MAPS UTILIZED

California Division of Mines and Geology, 1980, State of California Special Studies Zone Map, Elsinore Quadrangle, Scale 1: 24,000.

California Geological Survey (CGS), 2012, Geologic Compilation of Quaternary Surficial Deposits in Southern California, Santa Ana 30' X 60' Quadrangle, CGS Special Report 217, Plate 16, Scale 1: 100,000.

California Geological Survey, 2014, Tsunami Inundation Maps, California Department of Conservation, http://www.quake.ca.gov/gmaps/tsunami/tsunami_maps.htm.

Federal Emergency Management Agency (FEMA), 2014, Community Panel Number 06065C 1440H, August 18, 2014, Scale 1: 24,000.

Greenwood, R.B., and Morton, D.M., 1991, Geologic Map of the Santa Ana 1:100,000 Quadrangle, California, C.D.M.G. Open File Report 91-17.

Gutierrez, C., Bryant, W.A., Saucedo, G., and Wills, C., 2010, Fault Activity Map of California, Scale 1: 750,000, California Geological Survey, Geologic Data Map No. 6. Scale 1:750,000.

Jennings, C.W., 1994, Fault Activity Map of California and Adjacent Areas, Scale 1: 750,000, C.D.M.G. Geologic Data Map No. 8.

Morton, D.M., 2003, Geologic Map of the Perris 7.5-Minute Quadrangle, Riverside County, California, U.S.G.S. Open-File Report OFR 03-270, Scale 1:24,000.

Morton, D.M., 1999, Preliminary Digital Geologic Map of the Santa Ana 30' x 60' Quadrangle, Southern California, Scale 1: 100,000, U.S.G.S. Open-File Report 99-172, Version 1.0.

Morton, D.M. and Miller, F.K., 2006, Preliminary Geologic Map of the San Bernardino and Santa Ana 30' x 60' Quadrangles, California, Scale 1: 100,000, U.S.G.S. Open-File Report 2006-1217, 6 sheets, Version 1.0.

Rodgers, T.H., 1966, Geologic Map of California, Santa Ana Sheet, California Division of Mines & Geology, Scale 1:250,000 (Second Printing 1973).

Ziony, J.I., and Jones, L.M., 1989, Map Showing Late Quaternary Faults and 1978-1984 Seismicity of the Los Angeles Region, California, U.S.G.S. Miscellaneous Field Studies Map MF-1964.