INLAND FOUNDATION ENGINEERING, INC.
Consulting Geotechnical Engineers and Geologists
www.inlandfoundation.com

February 22, 2022
Project No. S168-183

STK ARCHITECTURE, INC.
Attention: Mr. Tony Finaldi
42095 Zevo Drive, Suite A15
Temecula, California 92590

Subject: Geologic Hazards Evaluation/Geotechnical Investigation
San Bernardino County Fire Station No. 226 Replacement
1920 Del Rosa Avenue North
San Bernardino, California.

Dear Mr. Finaldi:

We are pleased to submit this geotechnical report prepared for the subject project. The report includes geotechnical conclusions and recommendations for project design and construction. The primary conditions that will require mitigation are variable soil types and density of the near-surface alluvial soil on the site.

We appreciate being of service to you on this project. If you have any questions, please contact our office.

Respectfully,
INLAND FOUNDATION ENGINEERING, INC.

PROFESSIONAL GEOLOGIST DANIEL R. LIND NO. 2451 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

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Allen D. Evans, P.E., G.E.
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DRL:ADE:es
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INTRODUCTION

This report presents the results of the geotechnical investigation conducted for the proposed County of San Bernardino replacement Fire Station 226. The project site is located at 1920 Del Rosa Avenue North in the City of San Bernardino, California. The following reference was provided for our use during this study.

- Preliminary Site Plan 1, San Bernardino County Fire Department Station 226, 1920 Del Rosa Avenue N., San Bernardino, CA 92404, dated January 2021, prepared by STK Architects, Inc.

SCOPE OF SERVICE

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

- *Review of the general geologic and subsurface conditions at the project site.*
- *Evaluation of the engineering and geologic data collected for the project site.*
- *Evaluation of existing geologic conditions at the site and review of potential geologic and/or seismic hazards from a geologic standpoint.*
- *Evaluation of the local and regional tectonic setting and historical seismic activity, including a site-specific ground motion analysis.*

The tasks performed to achieve these objectives included:

- *Review of available geologic data pertinent to the site*
- *Field reconnaissance of the site and surrounding area to ascertain the presence of unstable or adverse geologic conditions.*
- *Seismic shear wave geophysical survey.*
- *Site specific geoseismic analysis and computation of 2019 California Building Code (CBC) seismic design parameters.*
- *Subsurface sampling and laboratory testing.*

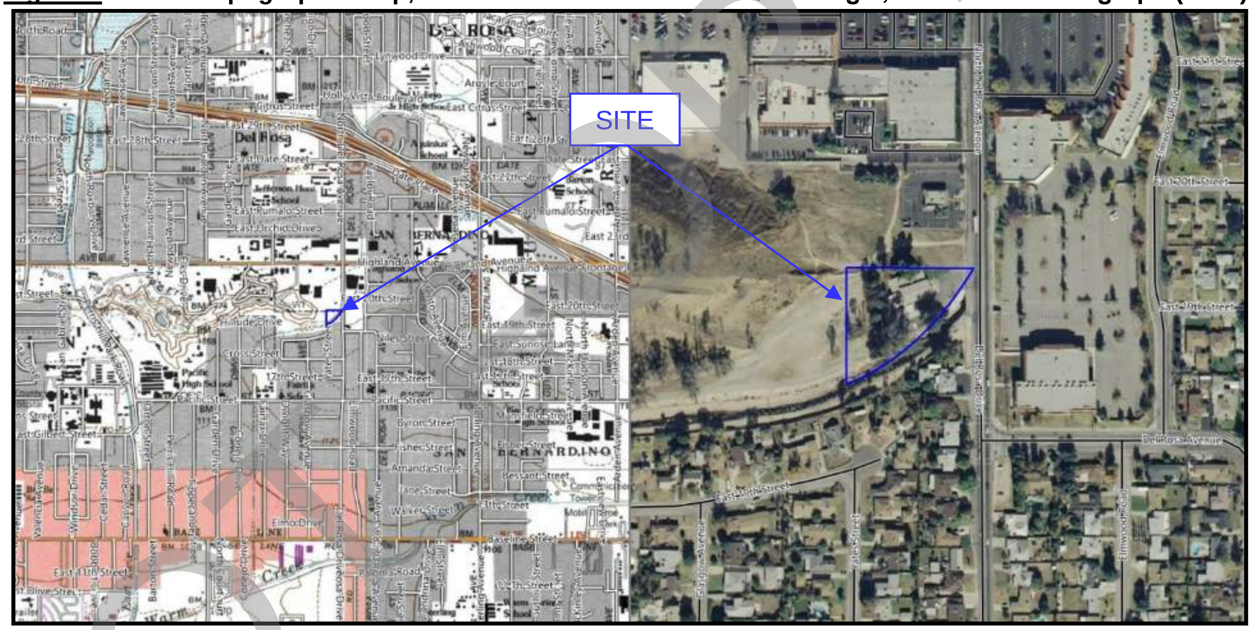
- *Analysis of the data collected and the preparation of this report with our geotechnical conclusions and recommendations.*

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

SITE AND PROJECT DESCRIPTION

The ±1.2-acre fire station site lies within the northwesterly portion of Section 36, Township 1 North, Range 4 West, S.B.B.&M. The irregular-shaped parcel is located at 1920 Del Rosa Avenue North in the City of San Bernardino, California. The site lies just east of Perris Hill. The Assessor Parcel Number for the property is 0273-011-22. The location of the fire station site is shown on Figure 1 below.

<u>Figure 1</u>: USGS Topographic Map, San Bernardino North 7.5' Quadrangle, and Aerial Photograph (2020)



An existing fire station is currently present on the easterly portion of the property that is to be demolished. The westerly portion of the property is undeveloped. Concrete drive aprons are present on the west and east sides of the existing fire station. The remaining driveway and parking areas are paved with asphalt concrete.

Topographically, the westerly portion of the site dips gently to the west-southwest. Large erosional rills traverse the westerly portion of the site. The erosional channels are up to approximately 1.5 feet deep and vary from about 8 inches to 3 feet wide. Vegetation on the site consists of several large trees south and west of the existing fire station. An existing CMU wall is present on the northerly portion of the site. The height of the existing wall ranges from about 2 to 4 feet tall. A drainage channel with rock lined slopes is present along the southerly border of the property.

The proposed project will consist of the construction of a new single-story fire station comprising approximately 7,000 square feet. The 2-bay fire station will include sleeping quarters for 6 crew members and will provide storage for two Type 1 Engines and a future ladder truck. Access to the station will be from N. Del Rosa Avenue.

We anticipate that the foundations for the new structure will consist of shallow spread and continuous footings with concrete slab-on-grade floors. Grading is expected to consist of preparation of the structure building pad area as well as pavement and landscape areas. We assume that rough cuts and fills on the order of 3 feet or less will be required to achieve final site grades (not including any remedial over-excavation).

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 No. 48, 2019).

The geologic hazards study indicates that development of the intended fire station project appears 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 appear to be prevalent within the proposed construction area, with the possible exception of seismic settlement potential and strong ground shaking originating from nearby large seismogenic fault sources.

The geologic hazards study included a site-specific ground motion analysis per the California Geologic Survey Note 48 (CGS, 2019). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters were evaluated using the OSHPD Seismic Design Maps web application (OSHPD, 2022) and the California Building Code criteria (CBC, 2019), 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: Summary of Seismic Design Parameters

Factor or Coefficient	Value
S_S	2.500g
S_1	0.936g
F_a	1.2
F_V	1.4
S_{DS}	1.600g
S_{D1}	1.020g
S_{MS}	2.400g
S_{M1}	1.530g
T_L	8 Seconds
$MCE_G PGA$	0.99g
Shear-Wave Velocity (V100)	2,018.9 ft./sec.
Site Classification	C
Risk Category	IV

Groundwater: Groundwater was not encountered within the exploratory borings, which extended to a maximum depth of 25 feet below the existing ground surface. The subject site is located within the Bunker Hill Basin, which is a sub-unit of the Upper Santa Ana Valley Groundwater Basin in Southern California. The water-bearing material in this basin consists of alluvial deposits of sand, gravel, and boulders interspersed with lenticular deposits of silt and clay.

Groundwater data prepared by Matti and Carson (1991) indicates that high groundwater was estimated to be around 85± feet in depth based on contour data. Since the entire site is underlain by crystalline metamorphic bedrock, free groundwater is not anticipated to be present, but could locally occur along fractures and/or local impermeable layers.

SUBSURFACE CONDITIONS

The field and laboratory exploration and testing indicate that the proposed construction site is underlain by a mantle of alluvial soil consisting of interbedded silty gravel (GM), silty sand (SM), sand with silt (SW-SM), clayey sand (SC), gravel with sand and silt (GP-GM), and silty clayey sand with gravel (SC-SM), The alluvial soils were encountered to depths ranging from about 6 to 10 feet below the existing ground surface. These soils generally range from very loose to very dense. Underlying these deposits at depth is metamorphic bedrock comprised of the Pelona Schist Formation. The bedrock was observed to be slightly weathered, fractured, and very hard. When broken down, the bedrock classifies as silty sand with gravel (SM).

Analytical testing indicates the concentration of sulfates is very low (22 ppm). In accordance with ACI 318, Table 4.2.1, the soil is classified as Class S0 with respect to sulfate exposure. The chloride concentration in the tested sample was 19 ppm and indicates that the soil is generally not corrosive with respect to ferrous metal. The soil is slightly alkaline with a pH value of 8.0. The saturated minimum resistivity value of 16,027 ohm-cm indicates the soil is mildly corrosive to buried ferrous metal. Inland Foundation Engineering, Inc. does not practice corrosion engineering. We recommend a qualified corrosion engineer be consulted for additional guidance.

CONCLUSIONS AND RECOMMENDATIONS

The primary geotechnical issues that will require mitigation are variable near-surface soil type and density conditions within the building pad and pavement areas. This soil is not suitable for support of foundations or pavement in its existing condition and should be over-excavated and recompacted. These and other geotechnical engineering recommendations for project design and construction are presented below.

Foundation Design: Shallow continuous and isolated spread footings for the proposed fire station should be designed with an allowable bearing pressure of 1,700 pounds per square foot (psf). Footings should have a minimum width of 12 inches and be founded a minimum depth of 12 inches below the lowest adjacent grade. The allowable bearing pressure may be increased by 1,200 psf for each additional foot of depth and by 450 psf for each additional foot of width, to a maximum allowable bearing pressure of 3,400 psf. The allowable bearing capacity may also be increased by ⅓ for short-term transient wind and seismic loads. All footings should be supported by a minimum thickness of compacted fill of at least 18 inches.

Settlement of foundations properly designed and constructed as recommended herein is expected to be less than one inch total. The total differential settlement between foundations of similar size and load is expected to be less than one inch vertical in 40 feet horizontal.

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 0.45 between soil and concrete may be used with dead load forces only. A passive earth pressure of 280 psf/ft may be used for the sides of footings poured against recompacted or dense native material. These values may be increased by ⅓ 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 40 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.

Excavation and Trench Wall Stability: All excavations should be configured in accordance with the requirements of CalOSHA. The soil should be classified as Type C. The classification of the soil and the shoring and/or slope configuration should be the responsibility of the contractor on the basis of the excavation depth and the soil encountered. The contractor should have a "competent person" onsite for the purpose of assuring safety within and about all construction excavations.

Concrete Slabs-on-Grade: 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 not exceeding 200 pounds per square inch per inch.

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. ACI typically recommend control joint spacings in unreinforced concrete at maximum intervals equal to the slab thickness times 24.

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 El745 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: We recommend that all surfaces that will support fire apparatus be paved with Portland cement concrete (PCC). PCC pavement should consist of 9 inches of PCC over 6 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. 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 above recommendations are based on the assumption that the concrete pavement will be constructed with doweled joints. We have also assumed that the concrete pavement will be restrained laterally by concrete curb/gutter or building foundations and that the edges of the concrete will 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 ¼ 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 2. The recommended sections are based on current Caltrans design procedures and the traffic index (T.I.) values shown.

Table 2: 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	0.45

Inland Foundation Engineering, Inc. does not practice traffic engineering. The T.I. values used to develop the recommended pavement sections are typical for projects of this type. We recommend that the project civil engineer or traffic engineer review the T.I. values used to verify that they are appropriate for this project.

Infiltration: Infiltration testing was performed at representative locations on the site. The testing procedures and test results are described in Appendix C. Table 3 below provides a summary of the test data with values for I_c. Note that the values shown do not include safety factors.

Table 3: Percolation Test Data and Infiltration Rates

Percolation Test No.	Percolation Rate (Min./Inch)	Depth Below Existing Ground Surface (In.)	Infiltration Rate (I_c) (In./Hr.)
P-01	4.2	48	0.8
P-02	1.4	60	2.8
P-03	2.6	60	1.6
P-04	4.0	48	1.0

General Site Grading: All grading should be performed per the applicable provisions of the 2019 California Building Code. The following specifications have been developed on the basis of the field and laboratory testing:

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.

   Any undocumented fill and loose alluvial soil encountered during site grading should be completed removed. Such material is suitable for replacement as compacted fill as recommended herein.

   Any abandoned underground utility lines should be traced out and completely removed from the site. Any abandoned septic systems, including septic tanks, seepage pits and leach lines, should be removed and backfilled at the direction of the geotechnical engineer.

2. **Preparation of Surfaces to Receive Compacted Fill:** All surfaces to receive compacted fill should be observed by the project geotechnical engineer to verify the exposed soil conditions are as expected. If roots or other deleterious materials are encountered, or if the relative compaction fails to meet the acceptance criterion, additional overexcavation may be required until satisfactory conditions are encountered. Upon approval, surfaces to receive fill should be scarified, 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. **Preparation of Building Areas:** The proposed building area for the new fire station and other appurtenant structures should be over-excavated to a depth of at least 5 feet below finish grade or 1.5 feet below the bottom of the deepest

footing, whichever is greater. Over-excavation should extend laterally for at least 5 feet outside of exterior building foundation lines.

5. **Preparation of Slab and Paving Areas:** During final grading and immediately prior to the placement of concrete or a base course, all surfaces to receive asphalt concrete paving or concrete slabs-on-grade should be processed and tested to assure compaction for a depth of at least of 12 inches. This may be accomplished by a combination of overexcavation, scarification and recompaction of the surface, and replacement of the excavated material as controlled compacted fill. Compaction of slab areas should be to a minimum of 90 percent relative compaction. Compaction within proposed pavement areas should be to a minimum of 95 percent relative compaction for both the subgrade and base course.

6. **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, except for the upper 12 inches under pavement areas where the minimum relative compaction should be 95 percent. Jetting of the native soils is not recommended.

7. **Testing and Observation:** During grading, tests and observations should be performed by the project geotechnical engineer or his/her representative to verify that the grading is performed per the project specifications. Soil 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.

GENERAL

The findings and recommendations presented in this report are based upon the soil conditions encountered at our 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 prior to the preparation of a grading plan for the project. We recommend that a pre-job conference be held on the site prior to the initiation of site grading. The purpose of this meeting will be to assure a complete understanding of the recommendations presented in this report as they apply to the actual grading performed.

This report was prepared for STK Architecture, Inc. for their use in the design of the proposed Fire Station 226. 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.

LIMITATIONS

The findings and recommendations of this report are based upon an interpolation of soil conditions between test locations. It is likely that conditions occur between borings that are different than those indicated in this report. Should such conditions be encountered during construction, our office should be notified in order to determine if revisions or retesting are warranted.

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 other warranty, either expressed or implied, is made as to the professional advice included in this report.

REFERENCES

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, 2019, California Building Code (CBC), California Code of Regulations, Title 24, Part 2, Volume 2.

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Matti, J.C., and Carson, S.E., 1991, Liquefaction Susceptibility in the San Bernardino Valley and Vicinity, Southern California, A Regional Evaluation, U.S.G.S. Bulleting 1898.

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APPENDIX A – Field Exploration

APPENDIX A

SITE EXPLORATION

Five exploratory borings were drilled with a truck-mounted hollow-stem auger drill rig at the approximate locations shown on Figure A-8. The materials encountered during drilling were logged by a staff geologist. Boring logs are included with this report as Figures A-3 through A-7.

Representative soil samples were obtained within the borings by driving a thin-walled steel penetration sampler with successive 30-inch drops of a 140-pound hammer. The numbers of blows required to achieve each six inches of penetration were recorded on the boring logs. Two different samplers were used; a Standard Penetration Test (SPT) sampler and a modified California sampler with brass sample rings. Representative bulk soil samples were also obtained from the auger 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

RELATIVE DENSITY	SPT * (# BLOWS/FT)	RELATIVE DENSITY (%)
VERY LOOSE	<4	0-15
LOOSE	4-10	15-35
MEDIUM DENSE	10-30	35-65
DENSE	30-50	65-85
VERY DENSE	>50	85-100

CONSISTENCY – FINE-GRAIN SOIL

CONSISTENCY	SPT* (# BLOWS/FT)	TORVANE UNDRAINED SHEAR STRENGTH (tsf)	POCKET ** PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)
Very Soft	<2	<0.13	<0.25
Soft	2-4	0.13-0.25	0.25-0.5
Medium Stiff	4-8	0.25-0.5	0.5-1.0
Stiff	8-15	0.5-1.0	1.0-2.0
Very Stiff	15-30	1.0-2.0	2.0-4.0
Hard	>30	>2.0	>4.0

* 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)

** UNCONFINED COMPRESSIVE STRENGTH IN TONS/SQ.FT. READ FROM POCKET PENETROMETER

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	Absence of moisture, dusty, dry to the touch
MOIST	Damp but no visible water
WET	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbled or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

EXPLANATION OF LOGS

LOG OF BORING B-01

DRILLING RIG	**Mobile B-61**	HAMMER TYPE	**Auto-Trip**
DRILLING METHOD	**Rotary Auger**	HAMMER WEIGHT	**140-lb.**
LOGGED BY	**FWC**	HAMMER DROP	**30-inches**
GROUND ELEVATION	**+/-**	BORING DIAMETER	**8-inches**
DATE DRILLED	**1/6/22**		

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.

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)
	GM		**SILTY GRAVEL with SAND,** fine- to medium, olive-brown (2.5Y 4/3), slightly moist, loose.			AU			
	SM		**SILTY SAND,** very fine- to fine, olive-brown (2.5Y 4/3), slightly moist, loose, interbedded with gravelly sand.			AU			
						SS	7 5	5	119
5						SS	4 5	7	113
	SM		**METAMORPHIC BEDROCK - PELONA SCHIST,** (SILTY SAND with GRAVEL), highly to slightly weathered, light olive-brown (2.5Y 5/6).			SS	50	4	126
10						SS	50		
			End of boring at 13 feet. Auger refusal. No groundwater encountered. Backfilled with native soils.			SPT	50/1"		

Inland Foundation Engineering, Inc.

CLIENT	**STK**
PROJECT NAME	**S.B. County Fire Station #226**
PROJECT LOCATION	**1920 N. Del Rosa Ave.** **San Bernardino, CA**
PROJECT NUMBER	**S168-183**
FIGURE NO.	A-3

LOG OF BORING B-02

DRILLING RIG	Mobile B-61	DATE DRILLED 1/6/22	HAMMER TYPE	Auto-Trip
DRILLING METHOD	Rotary Auger		HAMMER WEIGHT	140-lb.
LOGGED BY	FWC		HAMMER DROP	30-inches
GROUND ELEVATION	+/-		BORING DIAMETER	8-inches

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.

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)
0–5	SW-SM		**SAND with SILT,** fine- to coarse, brown (10YR 5/3), moist, loose.			AU			
						SS	11 14	5	114
5–10	GM		**SILTY GRAVEL with SAND,** olive-brown (2.5Y 4/3), moist, dense.			AU SS	33 50/5"	4	126
						SS	32 50	2	132
10.5–25.1	SM		**METAMORPHIC BEDROCK - PELONA SCHIST,** (SILTY SAND with GRAVEL), highly to slightly weathered, olive (5Y 4/4).			SS	50	1	122
15			- very rocky -			SS	50/4"	2	120
20						SPT	50/5"	4	
25						SPT	50/1"		
			End of boring at 25.1 feet. Auger refusal. No groundwater encountered. Backfilled with native soils.						

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CLIENT	STK
PROJECT NAME	S.B. County Fire Station #226
PROJECT LOCATION	1920 N. Del Rosa Ave. San Bernardino, CA
PROJECT NUMBER	S168-183

FIGURE NO. A-4

LOG OF BORING B-03

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

DATE DRILLED 1/6/22

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.

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)
	SM		**SILTY SAND,** with trace gravel, fine- to medium, olive-brown, moist, loose to medium dense.			AU			
5	SC		**CLAYEY SAND,** very fine- to fine, olive-brown (2.5Y 4/3), moist, dense.			SS	27 32	7	137
	GP-GM		**GRAVEL with SAND and SILT,** very fine- to fine, olive-brown (2.5Y 6/4), moist, dense.			SS	31 50/5"	5	122
10	SM		**METAMORPHIC BEDROCK - PELONA SCHIST,** (SILTY SAND with GRAVEL), slightly weathered, very dark gray (5Y 3/1), slightly moist, fractured, hard.			SS	50/4"	1	
			End of boring at 11.5 feet. Auger refusal. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT	STK
PROJECT NAME	S.B. County Fire Station #226
PROJECT LOCATION	1920 N. Del Rosa Ave. San Bernardino, CA
PROJECT NUMBER	S168-183

FIGURE NO. A-5

IFE BORING - GINT STD US LAB.GDT - 2/21/22 13:12 - P:\S168\S168-183-FIRE STATION 226\GINT.GPJ

LOG OF BORING B-04

DRILLING RIG	**Mobile B-61**	DATE DRILLED	**1/6/22**	HAMMER TYPE	**Auto-Trip**
DRILLING METHOD	**Rotary Auger**			HAMMER WEIGHT	**140-lb.**
LOGGED BY	**FWC**			HAMMER DROP	**30-inches**
GROUND ELEVATION	**+/-**			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)
	SM		**SILTY SAND,** fine- to medium, olive-brown, moist, loose.						
	SC-SM		**SILTY, CLAYEY SAND with GRAVEL,** very fine- to fine, dark olive-brown, moist, medium dense to dense.		X	SS	15 50/4"		
5	SM		**METAMORHPIC BEDROCK - PELONA SCHIST,** (SILTY SAND with GRAVEL), slightly weathered, dark gray-brown, slightly moist, fractured, hard.						
			End of boring at 5 feet. Auger refusal. No groundwater encountered. Backfilled with native soils.						

Inland Foundation Engineering, Inc.

CLIENT	**STK**
PROJECT NAME	**S.B. County Fire Station #226**
PROJECT LOCATION	**1920 N. Del Rosa Ave.**
	San Bernardino, CA
PROJECT NUMBER	**S168-183**

FIGURE NO. A-6

LOG OF BORING B-05

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

DATE DRILLED 1/6/22

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.

DEPTH (ft)	U.S.C.S.	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
	SM	**SILTY SAND,** with trace clay, fine- to medium, olive-brown (2.5Y 4/3), moist, dense.	AU			
	SC-SM	**SILTY, CLAYEY SAND with GRAVEL,** very fine- to fine, olive-brown (2.5Y 6/4), moist, dense.	AU			
			SS	17 50	4	132
5						
			SS	35 50	8	126
	SM	**METAMORPHIC BEDROCK - PELONA SCHIST,** (SILTY SAND with GRAVEL), slightly weathered, light olive-brown (2.5Y 5/4), slightly moist, fractured, hard.	AU			
10			SS	40 50/4"	4	123
			SS	50	4	119
15			SPT	50/5"	4	
20			SPT	50/5"	2	

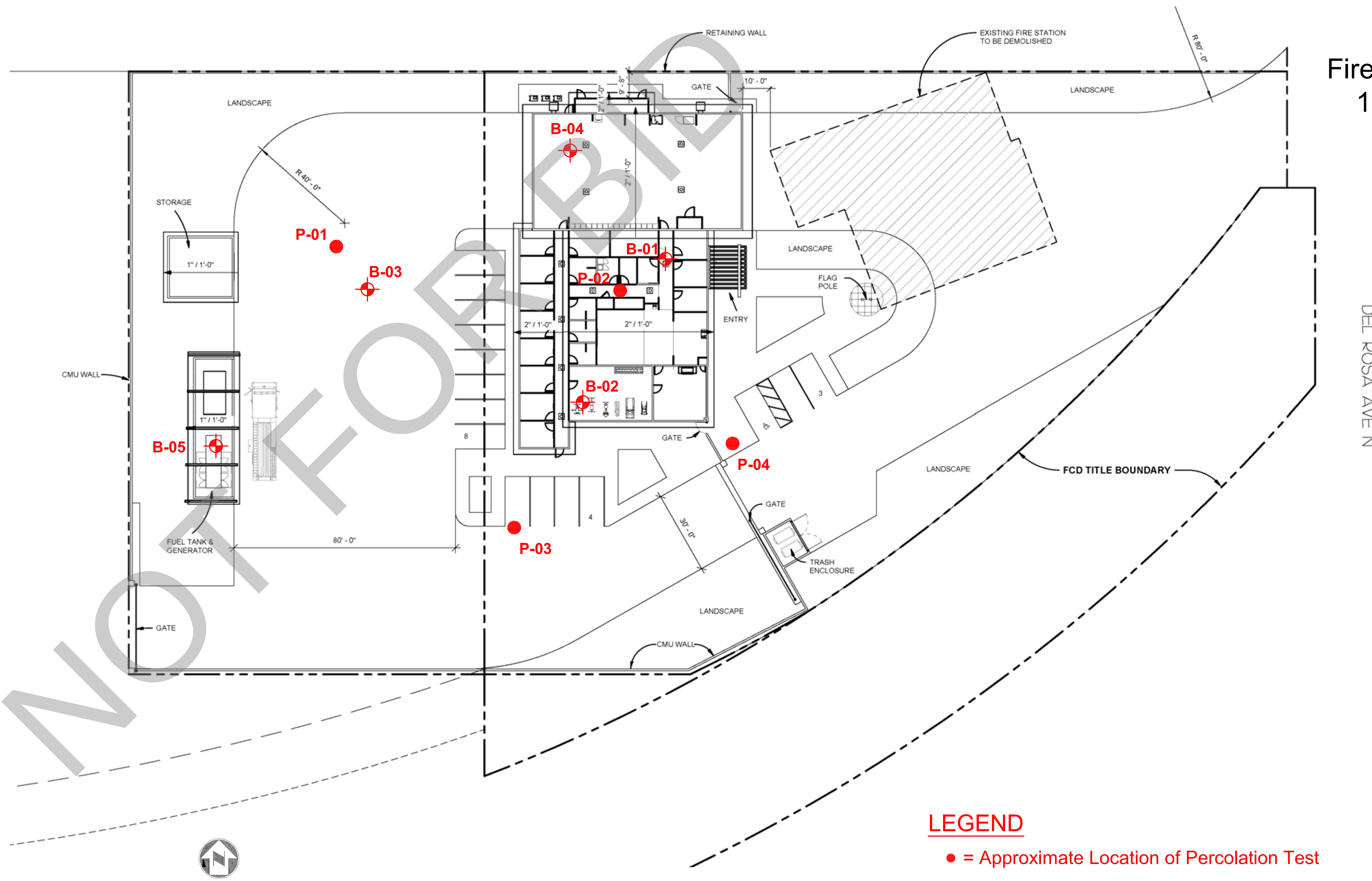
End of boring at 20.5 feet. Auger refusal. No groundwater encountered. Backfilled with native soils.

Inland Foundation Engineering, Inc.

CLIENT	STK
PROJECT NAME	S.B. County Fire Station #226
PROJECT LOCATION	1920 N. Del Rosa Ave. San Bernardino, CA
PROJECT NUMBER	S168-183

FIGURE NO. A-7

SITE PLAN
San Bernardino County
Fire Station No. 226 Replacement
1920 Del Rosa Avenue North
San Bernardino, California
APN 0273-011-22



LEGEND

● = Approximate Location of Percolation Test

⌖ = Approximate Location of Exploration Borings

INLAND FOUNDATION ENGINEERING, INC.
1310 South Santa Fe Avenue
San Jacinto, California
(951) 654-1555 FAX (951) 654-0551

DRAWN BY: ES	JOB NO.: S168-183	
SCALE: 1" = 40'	DATE: February 2022	A-8

APPENDIX B –
Laboratory Testing

APPENDIX B

LABORATORY TESTING

Representative soil samples obtained from our borings were returned to our laboratory for additional observation and testing. Descriptions of the tests performed are provided below.

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 the testing are shown on the boring logs (Figures A-3 through A-7).

Sieve Analysis: Six 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 this testing are shown on Figures B-2 and B-3.

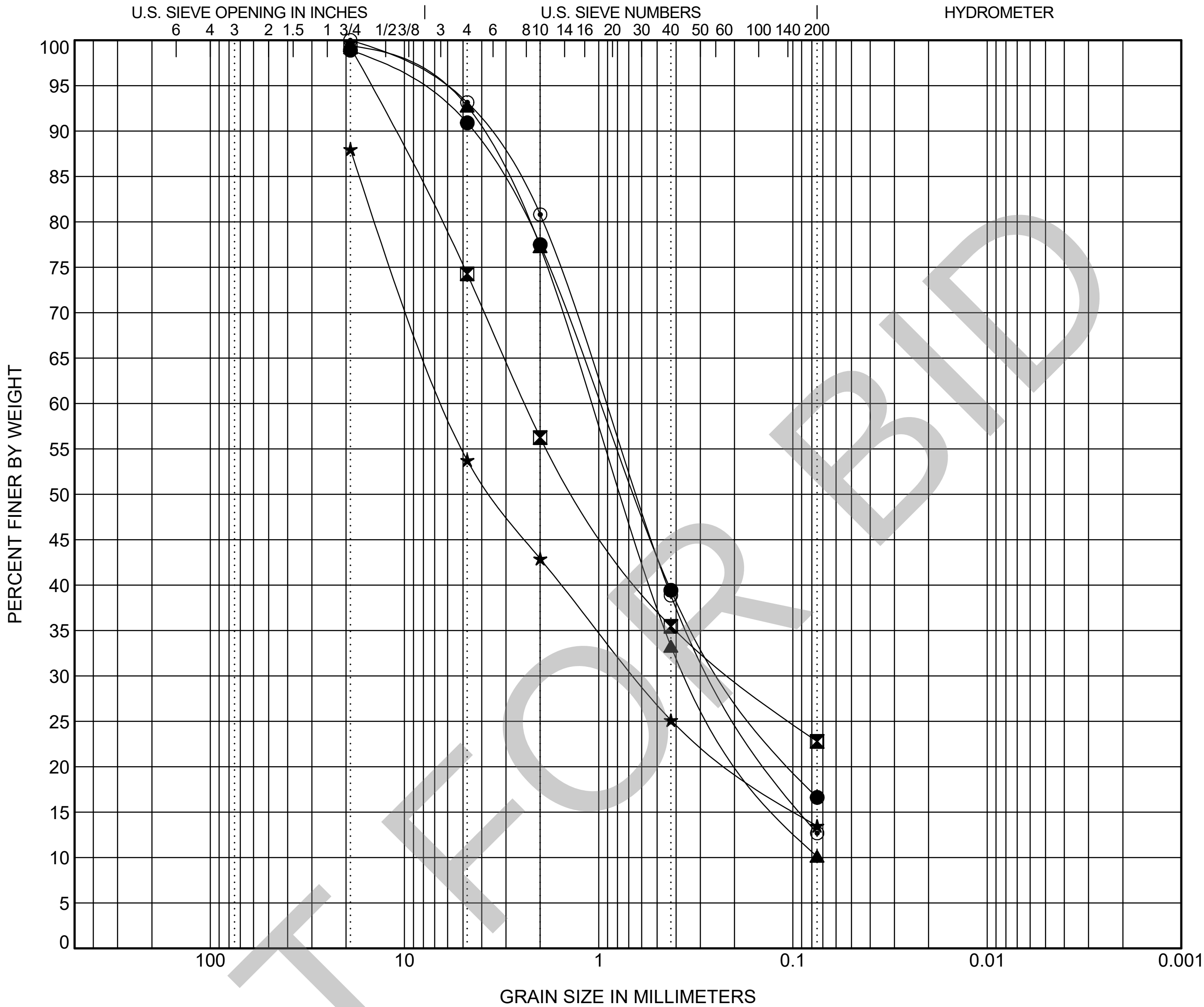
Plastic Index: Two samples were delivered to AP Engineering and Testing in Pomona, California for plastic index testing in accordance with ASTM D4318. These tests provide information regarding soil plasticity and are also used for developing classifications for the soil in accordance with the Unified Classification System. The results are shown on Figures B-4.

Direct Shear Strength: One sample was delivered to AP Engineering and Testing in Pomona, California for direct shear strength testing in accordance with ASTM D3080. This test measures the shear strength of the soil under various normal pressures and is used in developing parameters for foundation design and lateral earth pressure. The results are shown on Figure B-5.

Consolidation: One sample was 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 are shown on Figure B-6.

Analytical Testing: One sample was delivered to AP Engineering and Testing in Pomona, California to evaluate the concentration of soluble sulfates, chlorides, pH level, and resistivity of and within the on-site soils. The results are shown on Figure B-7.

R-value: One sample was selected for R-value and delivered to AP Engineering and Testing in Pomona, California 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-8.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	SAMPLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**1.5**	**SILTY SAND (SM)**					
⧗	**B-01**	**7.0**	**SILTY SAND with GRAVEL (SM)**					
▲	**B-02**	**0.0**	**WELL-GRADED SAND with SILT (SW-SM)**				**1.37**	**14.67**
★	**B-02**	**5.0**	**SILTY GRAVEL with SAND (GM)**					
⊙	**B-03**	**0.0**	**SILTY SAND (SM)**	**NP**	**NP**	**NP**		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**1.5**	**19**	**0.981**	**0.207**		**8.0**	**74.3**	**16.6**	
⧗	**B-01**	**7.0**	**19**	**2.396**	**0.201**		**25.0**	**51.5**	**22.8**	
▲	**B-02**	**0.0**	**19**	**1.089**	**0.332**		**6.7**	**82.6**	**10.1**	
★	**B-02**	**5.0**	**19**	**6.119**	**0.65**		**34.2**	**40.3**	**13.5**	
⊙	**B-03**	**0.0**	**19**	**0.927**	**0.236**		**6.8**	**80.5**	**12.7**	

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

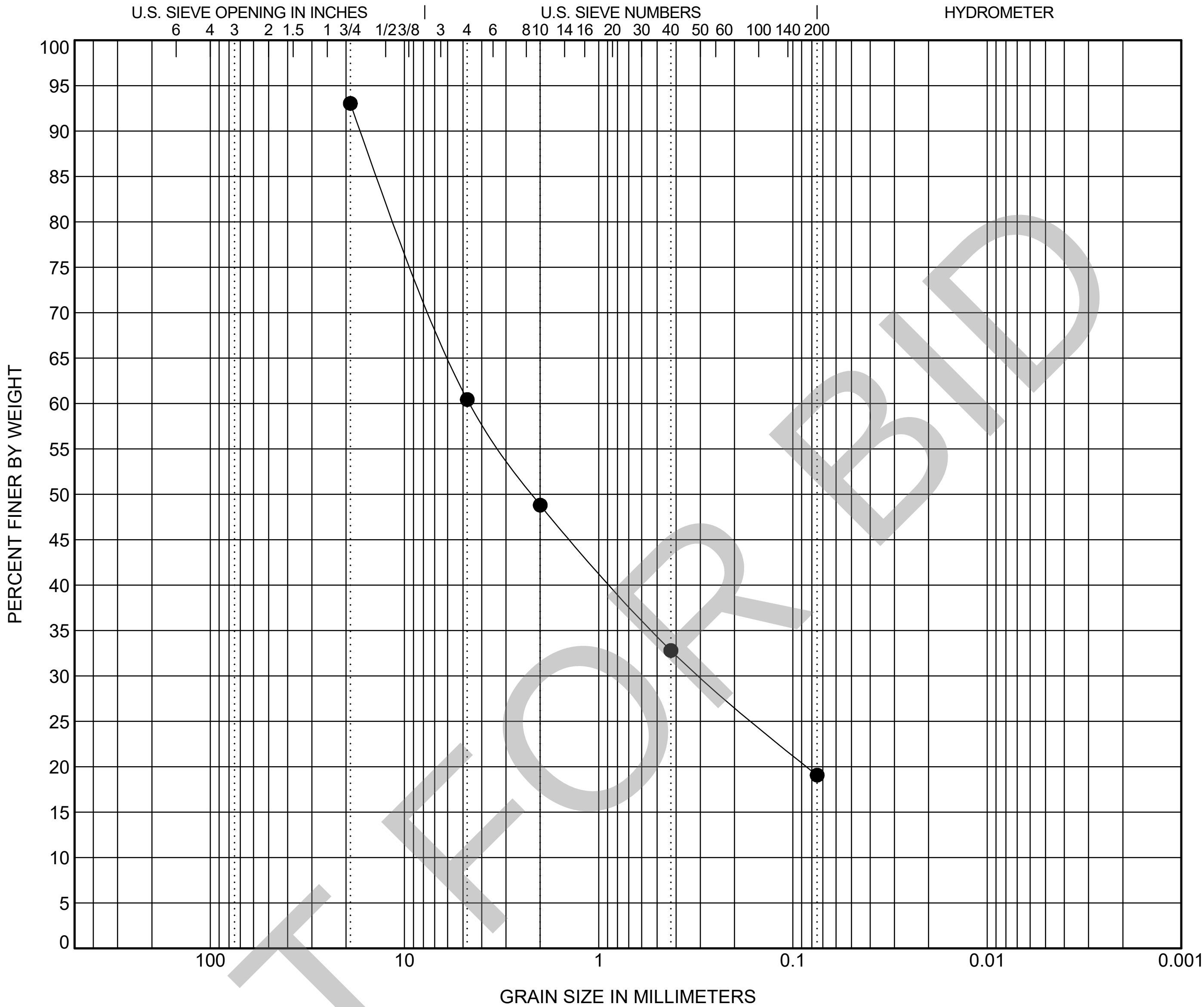
FIGURE NO. B-2

CLIENT STK

PROJECT NUMBER S168-183

PROJECT NAME S.B. County Fire Station #226

PROJECT LOCATION 1920 N. Del Rosa Ave.
San Bernardino, CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	SAMPLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-05**	**3.0**	**SILTY, CLAYEY SAND with GRAVEL (SM)**	**22**	**17**	**5**		

	BOREHOLE	DEPTH	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-05**	**3.0**	**19**	**4.6**	**0.298**		**32.6**	**41.4**	**19.1**	

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-3

CLIENT STK

PROJECT NUMBER S168-183

PROJECT NAME S.B. County Fire Station #226

PROJECT LOCATION 1920 N. Del Rosa Ave.
San Bernardino, CA

IFE SIEVE ANALYSIS - GINT STD US LAB.GDT - 2/21/22 14:53 - P:\S168\S168-183-FIRE STATION 226\GINT.GPJ

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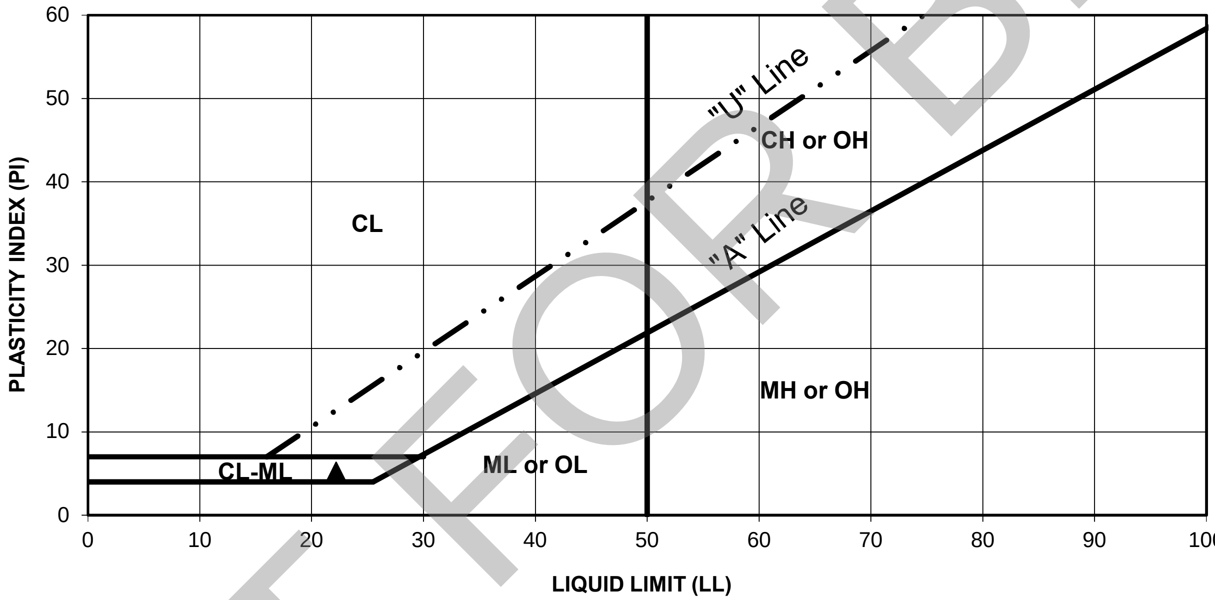
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

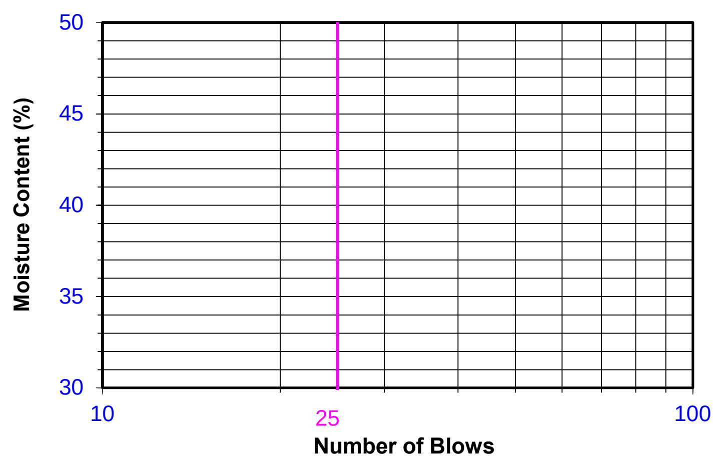
ATTERBERG LIMITS
ASTM D 4318

Client Name:	Inland Foundation Engineering	Tested By:	DK	Date:	02/07/22
Project Name:	STK: S.B. County Fire Station #226	Computed By:	NR	Date:	02/08/22
Project No.:	S168-183	Checked By:	AP	Date:	02/08/22



PROCEDURE USED

- [ ] Wet Preparation
- [X] Dry Preparation
- [X] Procedure A Multipoint Test
- [ ] Procedure B One-point Test



Symbol	Boring Number	Sample Number	Depth (feet)	LL	PL	PI	Plasticity Chart Symbol
	B-03	-	0-3	NP	NP	NP	
▲	B-05	-	3-4	22	17	5	CL-ML

* NP denotes "non-plastic"

Figure No. B-4

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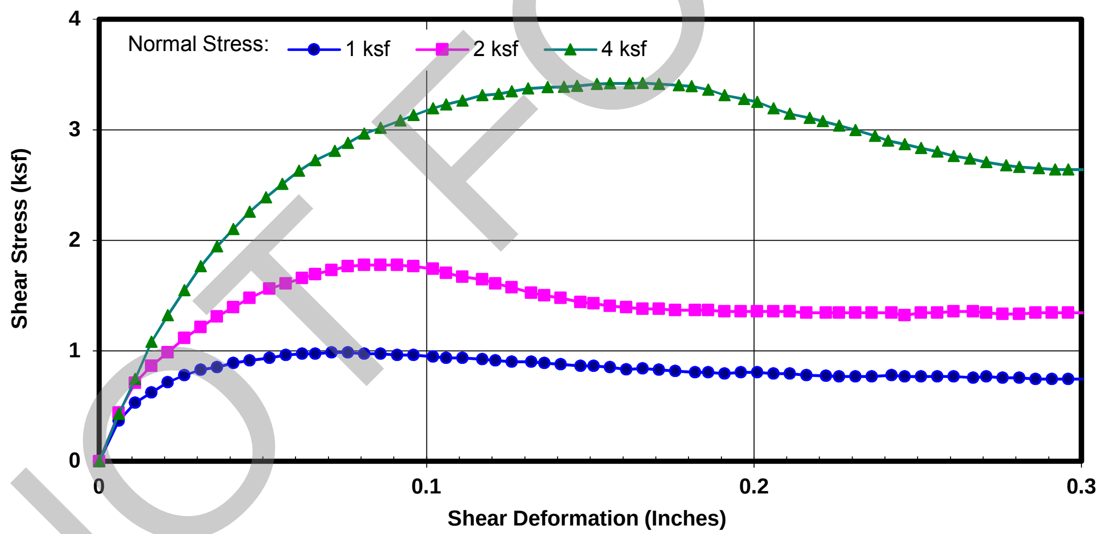
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Client: Inland Foundation Engineering
Project Name: STK: S.B. County Fire Station #226
Project No.: S168-183
Boring No.: B-02
Sample No.: - **Depth (ft):** 2.5-3.5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 01/28/22
Computed By: NR **Date:** 01/31/22
Checked by: AP **Date:** 01/31/22

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)
111.4	105.9	5.3	20.0	24	91	1	0.984	0.744
						2	1.776	1.344
						4	3.423	2.640



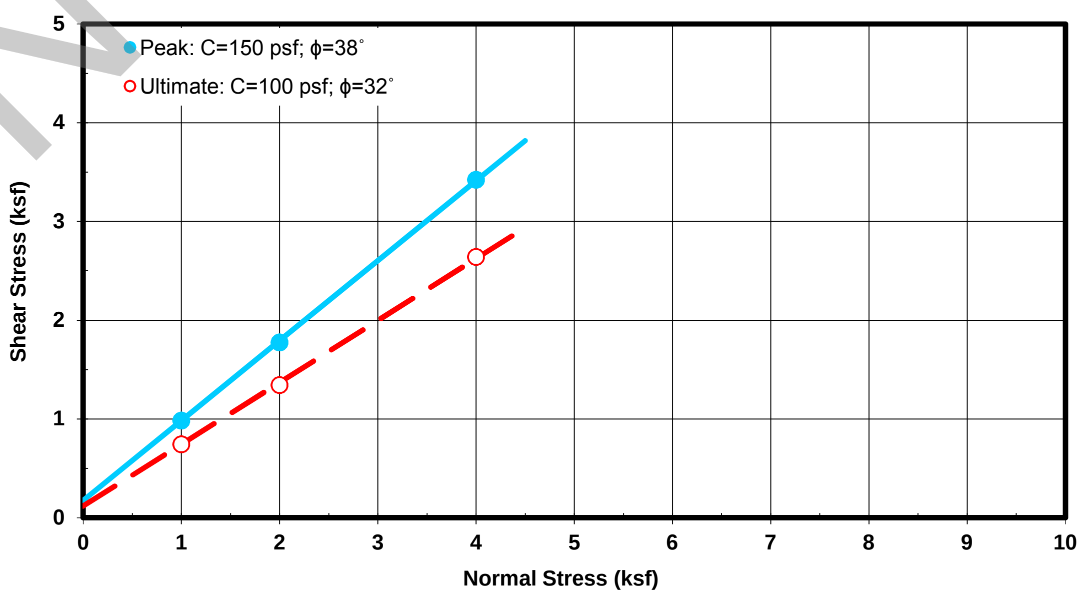
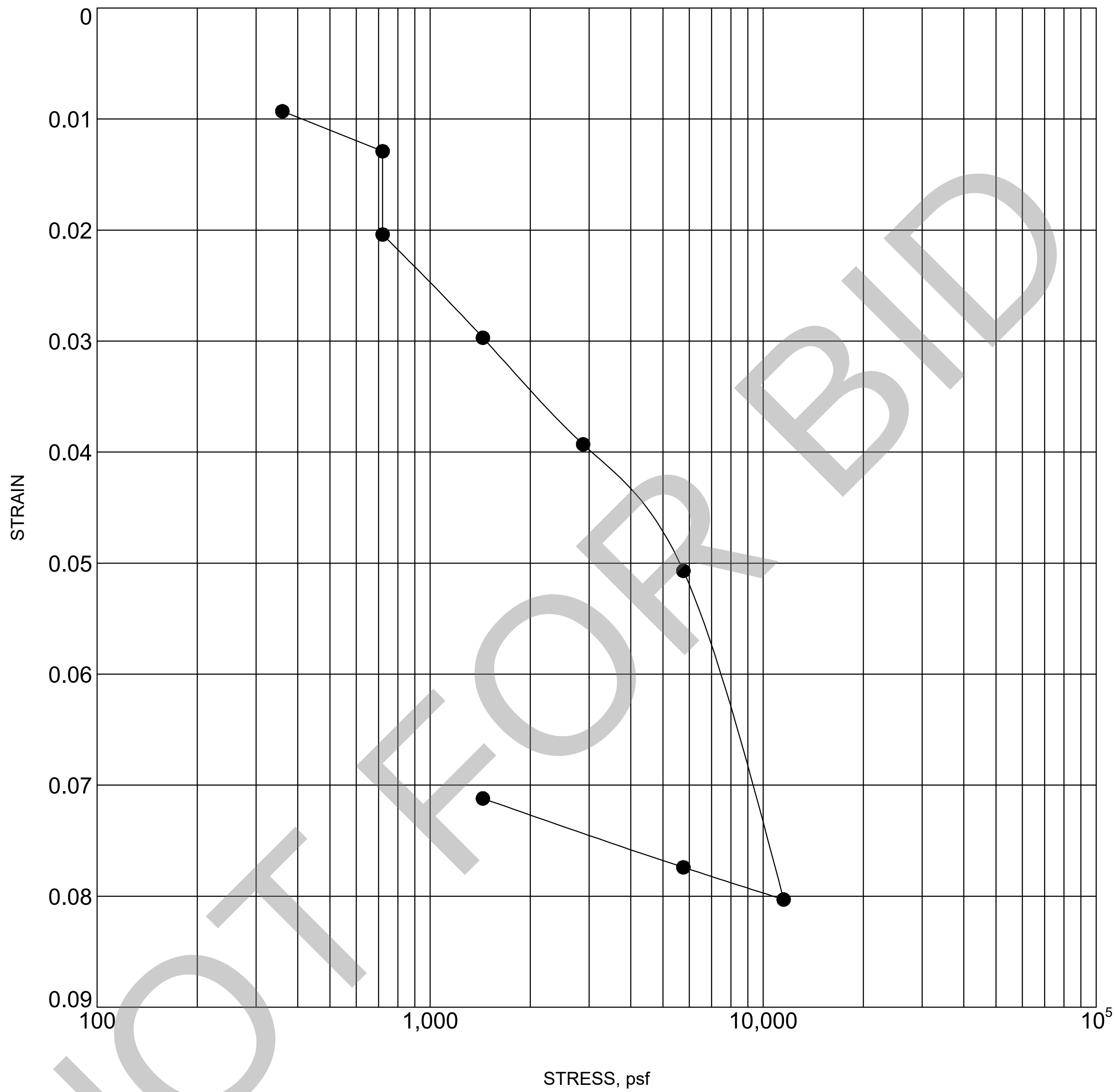


Figure No. B-5



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-01**	**2.5**	**SILTY SAND (SM)**	**110**	**5**

CONSOLIDATION TEST (ASTM D2435)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-6

CLIENT STK

PROJECT NUMBER S168-183

PROJECT NAME S.B. County Fire Station #226

PROJECT LOCATION 1920 N. Del Rosa Ave.
San Bernardino, CA

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CORROSION TEST RESULTS

Client Name: Inland Foundation Engineering
Project Name: STK: S.B. County Fire Station #226
Project No.: S168-183

AP Job No.: 22-0149
Date: 01/27/22

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-02	-	0-5	Silty Sand w/gravel	16,027	8.0	22	19

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

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R-VALUE TEST DATA

ASTM D2844

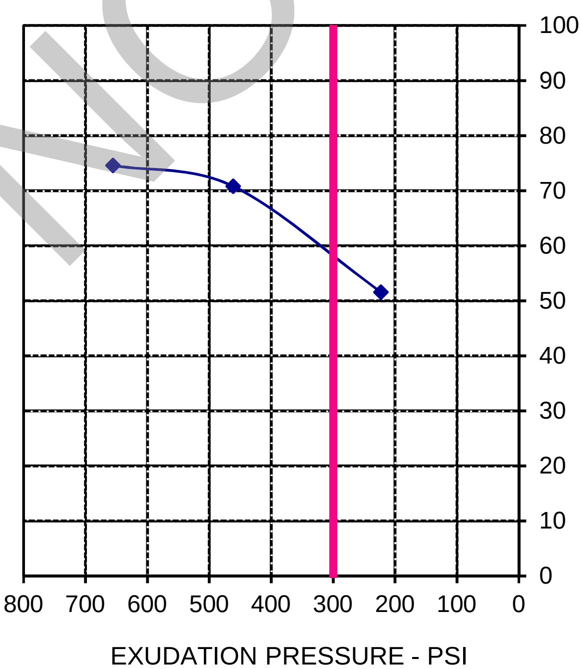
Project Name:	STK: S.B. County Fire Station #226	Tested By:	ST	Date:	01/25/22
Project Number:	S168-183	Computed By:	KM	Date:	01/26/22
Boring No.:	B-03	Checked By:	AP	Date:	01/31/22
Sample No.:	-	Depth (ft.):	0-3		
Location:	N/A				
Soil Description:	Silty Sand				

Mold Number	G	H	I	
Water Added, g	16	37	54	
Compact Moisture(%)	6.6	8.7	10.4	
Compaction Gage Pressure, psi	250	250	250	
Exudation Pressure, psi	656	461	223	
Sample Height, Inches	2.4	2.5	2.5	
Gross Weight Mold, g	2889	2916	2912	
Tare Weight Mold, g	1827	1837	1819	
Net Sample Weight, g	1062	1079	1094	
Expansion, inchesx10^{-4}	3	16	25	
Stability 2,000 (160 psi)	12/24	15/30	28/55	
Turns Displacement	4.54	4.47	4.48	
R-Value Uncorrected	76	71	52	
R-Value Corrected	75	71	52	
Dry Density, pcf	125.7	120.3	120.1	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	0.49	0.56	0.93	
G.E. by Expansion	0.01	0.05	0.08	

R-VALUE		
	By Exudation:	58
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	58

Remarks

Gf = 1.34, and 0.0 % Retained on the ¾"
*Not Applicable



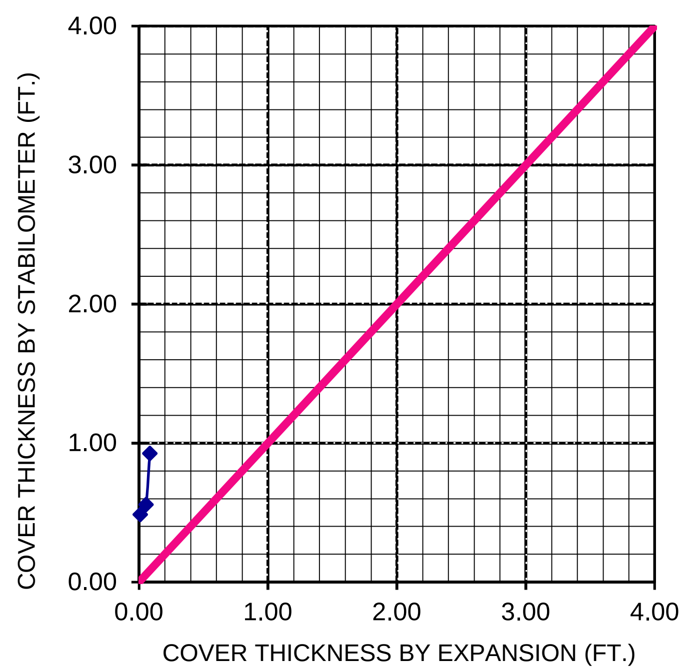


Figure No. B-8

APPENDIX C –

Infiltration Testing

APPENDIX C

INFILTRATION TESTING

Infiltration testing was conducted in general accordance with Appendix D of the Technical Guidance Document for Water Quality Management Plans, prepared by CDM Smith for the County of San Bernardino Areawide Stormwater Program (2013). The Riverside County Department of Environmental Health percolation test procedure was used for this study. The percolation rates were converted to infiltration rates using the Porchet method.

Four (4) percolation tests were performed at the locations shown on Figure No. A-8. The tests were performed at depths of approximately 48 and 60 inches below the existing ground surface. The test holes were approximately eight (8) inches in diameter. Per the specified percolation test procedure, the test holes were filled with water to a depth of at least five (5) times their radius. A two-inch thick layer of gravel was placed in the bottom of each test hole. In this case, the test holes were excavated and filled to a depth of at least 20 inches above the top of the gravel.

The test holes were presoaked prior to actual testing. The measured percolation rates ranged from 1.3 to 4.6 minutes per inch. Percolation test rates were converted to infiltration rates (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 0.2 to 2.1 inches per hour. These values <u>exclude</u> factors of safety. The table below provides a summary of the test data with values for I_c:

Percolation Test No.	Percolation Rate (min/in)	Depth Below Existing Ground Surface (in)	Infiltration Rate (I_c) (in/hr)
P-01	4.6	48	0.8
P-02	1.4	60	2.8
P-03	2.6	60	1.6
P-04	4.0	48	1.0

APPENDIX D –
Geologic Hazards Report

GEOLOGIC HAZARDS REPORT

SAN BERNARDINO COUNTY FIRE STATION 226

1920 N. DEL ROSA AVENUE

SAN BERNARDINO, CALIFORNIA

Project No. 223764-1

January 17, 2022

Prepared for:

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

Consulting Engineering Geology & Geophysics

P.O. Box 1090, Loma Linda, CA 92354 • 909 796-4667

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
San Bernardino County Fire Station 226
1920 Del Rosa Avenue
San Bernardino, California
IFE Project No. S168-183

At your request, this firm has prepared a geologic hazards report for the proposed new San Bernardino County Fire Station 226, 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" (CGS Note 48, 2019).

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

We understand that this report will be appended to your current geotechnical investigation, therefore, some descriptive sections such as site description, proposed development, etc., have been purposely omitted as they have been described in detail in your referenced report. No grading plans were available for this evaluation, and no field or subsurface exploration was performed by this firm. Only a review of available geologic and geotechnical data in our files was undertaken, including observation of the exploratory borings that were drilled by Inland Foundation engineering, Inc. (IFE) on January 6, 2022, including performing a seismic shear-wave survey.

GEOLOGIC SETTING

The subject site lies within a natural geomorphic province in California known as the Peninsular Ranges. This province is characterized by northwest-trending valleys and mountains that are, in part, due to the tectonic framework of this area, which is also dominated by a northwest-trending structure. Locally, the study area is included within a sub-structural unit of the Peninsular Ranges known as the San Bernardino Valley Block. This block is essentially a depressed region bounded by faults to the northeast (San Andreas), the southwest (San Jacinto), and the south (Banning). The San Bernardino Valley is formed by a series of coalescing alluvial fans, of which the combined fan of the Santa Ana River and Mill Creek, originating from to the northeast, is the largest and most distinct. This and other alluvial fans (i.e., Lytle and Cajon Creeks, Devil Canyon, East Twin and City Creeks) emanate the mountains, then coalesce to form part of a broad alluvial plain, which then forms the San Bernardino Valley.

The subject area investigated for this report is included within the flood/alluvial plain limits of the San Bernardino Valley, situated along the eastern flank of Perris Hill, which is a low-lying bedrock hill that locally protrudes from the San Bernardino Valley. Geologic mapping of the area by Miller et al. (2001), as illustrated on Plate 1, indicates that the project development area is locally underlain by unconsolidated to slightly consolidated Holocene and late Pleistocene young alluvial fan deposits (map symbol Qyf_3), generally described as sand and pebble-boulder gravel. Additionally, late Holocene age very young wash deposits also traverse the site deposits (map symbol Qw), consisting of unconsolidated to locally cemented sand, gravel, and boulder deposits. Underlying these deposits at depth is bedrock of the adjacent Perris Hill, which is generally comprised of Mesozoic age metamorphic bedrock of the Pelona Schist Formation that also contains fine-grained quartzite and greenstone.

The exploratory logs prepared by IFE (2022) indicate that the site is mantled by interbedded clayey sand, silty sand, gravel with sand, and silty clayey sand with gravel, that are in a generally loose to dense condition, to a depth of at least 10½ feet locally. Encountered below these surficial alluvial materials is metamorphic bedrock (Pelona Schist) that is slightly weathered, fractured, and very hard.

FAULTING

There are at least forty-three major late Quaternary active/potentially active faults that are located within a 100-kilometer (62-mile) radius of the subject site (Blake, 1989-2000a). Of these, there are no known active faults that traverse the site based on available published literature, nor was there any surficial geomorphic evidence that was suggestive of faulting. Additionally, the subject site is not located within a State of California "Alquist-Priolo Earthquake Fault Zone" for surface-fault rupture hazard (California Geological Survey, 2018).

The nearest known “active” fault that is zoned by the California Geological Survey, 2018 is the San Andreas Fault (San Bernardino North Segment), located approximately 1.8 miles to the northeast (C.D.M.G., 1974). This fault segment is a right-lateral, strike-slip fault, being approximately 103-kilometers in length, with an associated maximum moment magnitude (M_w) of 7.4 and a slip-rate of 24 ±6 mm/year (C.D.M.G., 1996, Cao, et al., 2003, and Petersen et al., 2008).

However, for seismic design purposes, we are considering that a cascading effect of rupture will occur along the entire length of the southern San Andreas Fault Zone (which includes ten segments, collectively) rather than just the San Bernardino North segment. Based on the recently published rupture-model data (Petersen et al., 2008), the total rupture area of these combined faults is 6,849.7 square kilometers and has an associated Maximum Moment Magnitude (M_W) of 8.1.

GROUND MOTION ANALYSIS

According to California Geological Survey Note 48 (CGS, 2019), a site-specific ground motion analysis is required for the subject site (CBC, 2019, Section 1613A and also as required by ASCE 7-16, Chapter 21). The results of this analysis are presented within Appendix B for documentation purposes. 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 soil Site Classification and V_{S30} input values for the ground motion analysis. This survey was performed within the limits of the proposed construction.

Geographically, the subject construction area is centrally located at Latitude 34.13315 and Longitude -117.25332 and (World Geodetic System of 1984 coordinates). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the California’s Office of Statewide Health Planning and Development Seismic Design Maps (OSHPD, 2022) and the California Building Code criteria (CBC, 2019), 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	**2.500g**
S_1	**0.936g**
F_a	**1.2**
F_v	**1.4**
S_{DS}	**1.600g**
S_{D1}	**1.020g**
S_{MS}	**2.400g**
S_{M1}	**1.530g**
T_L	**8 Seconds**
MCEG PGA	**0.99g**
Shear-Wave Velocity (V100)	**2,018.9 ft/sec**
Site Classification	**C**
Risk Category	**IV**

HISTORIC SEISMICITY

A computerized search, based on Southern California historical earthquake catalogs, has been performed using the computer program EQSEARCH (Blake, 1989-2000b) and the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2022a). 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 January 2022, within a 100-kilometer radius of the site.

TABLE 2 - HISTORIC SEISMIC EVENTS; 1800-2022 (100-kilometer radius)

Richter Magnitude (M)	No. of Events
4.0 - 4.9	620
5.0 - 5.9	76
6.0 - 6.9	15
7.0 - 7.9	1
8.0+	0

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 closest recorded notable earthquake epicenter (magnitude 4.0 or greater) is a M4.5 event (January 9, 2009), which occurred approximately three miles to the southwest.

- The nearest estimated significant historic earthquake epicenter (pre-1932) was approximately 3½ miles southwest of the site (July 15, 1905, M5.3).

- The nearest recorded significant historic earthquake epicenter was a M5.6 event of October 16, 1999, located approximately 14 miles northeast of the site.

- The largest estimated historical earthquake epicenter (pre-1932) within a 62-mile radius of the site is a M6.9 event of December 8, 1812 (28± miles northwest).

- The largest recorded historical earthquake was the M7.6 Landers's event, located approximately 47 miles to the east-northeast (June 28, 1992).

- The largest estimated ground acceleration estimated to have been experienced at the site was at least 0.1820g which resulted from the M5.3 event of July 15, 1905, located approximately 3½ miles to the southwest (Blake, 1989-2000b) based on the attenuation relationship of Boore et al. (1997).

An Earthquake Epicenter Map which includes magnitudes 4.0 and greater for a 100-kilometer (62-mile) radius (blue circle) from the site (central blue dot), has been included below as Figure 1. This map was prepared using the ANSS Comprehensive Earthquake Catalog (U.S.G.S, 2022a) of instrumentally recorded events from the period of 1932 to January 2022.

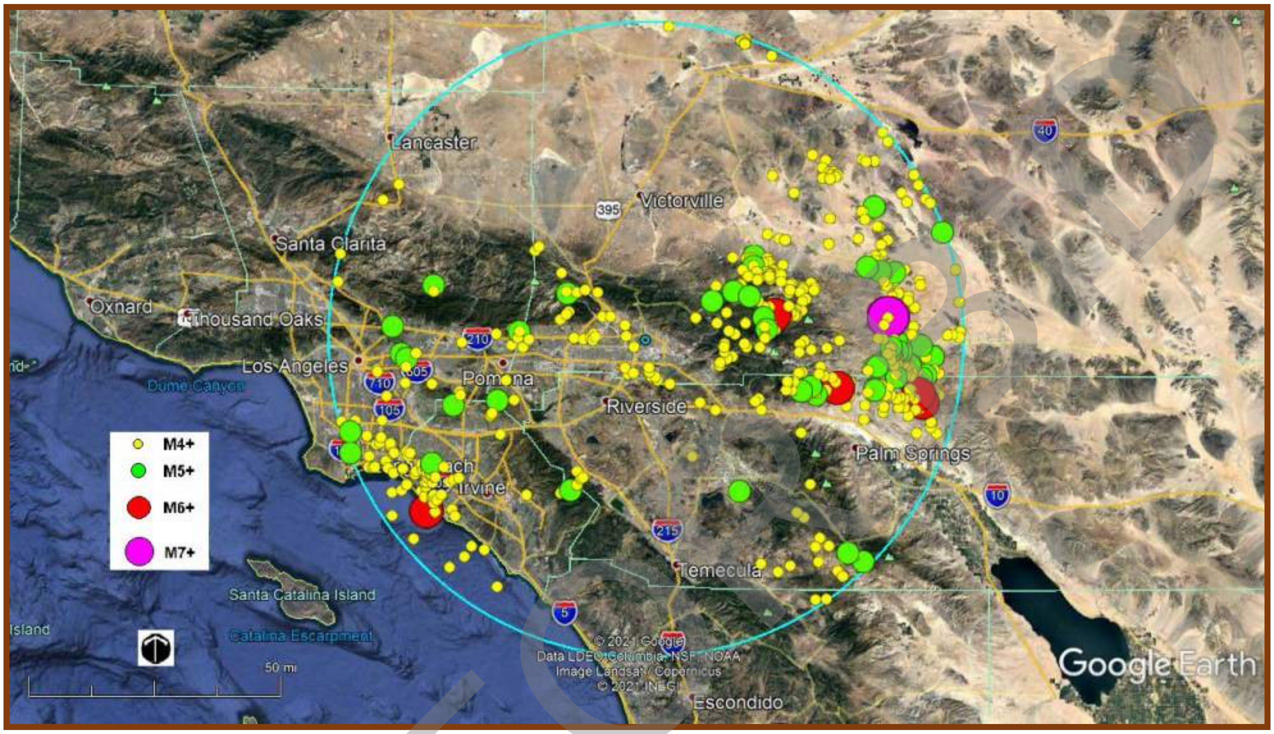


FIGURE 1- Earthquake Epicenter Map showing events of M4.0+ within a 100-kilometer radius.

GROUNDWATER

The subject site is located within the Bunker Hill Basin, which is a subunit of the greater Upper Santa Ana Valley Groundwater Basin in Southern California. This basin is bordered on the west by the San Jacinto Fault, the northeast by the San Bernardino Mountains, the south by the Badlands, and east by Crafton Hills. The area of the basin is approximately 110 square miles. The water-bearing material in the basin consists of alluvial deposits of sand, gravel, and boulders interspersed with lenticular deposits of silt and clay. In the Bunker Hill Basin, most of the recharge to groundwater is supplied by runoff from the San Bernardino Mountains, and smaller amounts by deep penetration of rainfall and artificial recharge. Within the Bunker Hill Basin, groundwater generally flows similar to that of surface draining. Locally, groundwater flows toward the west (Duell and Schroeder, 1989).

Groundwater data prepared by Matti and Carson (1991) indicates that high groundwater was estimated to be around 85± feet in depth based on contour data. During the recent subsurface investigation performed by IFE (2022), groundwater was not encountered within any of the exploratory borings excavated at the site to a depth of at least 25 feet. Since the site is entirely underlain by crystalline metamorphic bedrock, free groundwater is not anticipated to be present, but could locally occur along fractures and local perched layers.

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 no known active faults are believed to traverse the subject site, the probability of ground rupture is considered very low to nil.

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 flat-lying nature of the site, distance from embankments, the potential for ground lurching and/or lateral spreading is nil. The bedrock forming the hillside to the west is not considered to create a potential hazard from ground lurching and/or lateral spreading.

Flooding (Water Storage Facility Failure)- Since no water storage facility (i.e., water tank, dam, etc.) is located above the site, the potential for flooding, caused by water storage facility failure, is considered nil. The nearest inundation flood zone approaches within 2,600± feet to the southeast (California Department of Water Resources, 2022), which is associated with catastrophic dam failure of the Seven Oaks Dam (located 9± miles to the east). Additionally, the City of San Bernardino Seven Oaks Dam Inundation Map (2005, Figure S-2), indicates the site be located outside of the "Limits of Flooded Area with Dam Failure".

Seismically-Induced Settlement- Seismically-induced settlement generally occurs within areas of loose granular soils. The proposed construction area is locally mantled by interbedded clayey sand, silty sand, gravel with sand, and silty clayey sand with gravel, that are in a generally loose to dense condition, to a depth of at least 10½ feet locally. Below this depth, the subject site is underlain by crystalline metamorphic bedrock. Therefore, there appears to be only a very low potential for seismically-induced settlement to occur.

Landsliding- Due to the relatively low-lying relief of the site, landsliding of the site due to seismic shaking is considered nil. The exposed bedrock along the southern flank of Perris Hill (located approximately 100± feet to the east) rises 100± feet above the surrounding ground level, wherein at this time, no known landsides have been mapped there locally. Due to the distance from the hillside to the proposed building, landsliding due to seismically-induced landsliding that would affect the site, appears to be a low potential. According to the City of San Bernardino Slope Stability and Major Landslides

Map (2005, Figure S-7), Perris Hill to the east is not shown to have any existing landslides or landslide susceptibility.

Liquefaction- In general, liquefaction is a phenomenon that occurs where there is a loss of strength or stiffness in the soils from repeated disturbances of saturated cohesionless soil that can result in the settlement of buildings, ground failures, or other such related hazards. The main factors generally contributing to this phenomenon are: 1) cohesionless, granular soils having relatively low densities (usually of Holocene age); 2) shallow groundwater (generally less than 40 feet); and 3) moderate-high seismic ground shaking. According to the City of San Bernardino Liquefaction Susceptibility Map (2005, Figure S-5), the subject site is not shown to be located within the limits of a liquefaction zone. Additionally, due to the greater than 50-foot depth to groundwater, dense nature and presence of bedrock at depth, there does not appear to be a potential for liquefaction to occur.

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.

Rockfalls- The site lies upon a relatively flat-lying alluvial plain, but is however, adjacent to a hillside along the east. Since no large rock outcrops were observed to be present at or adjacent to the proposed construction area, the possibility of rockfalls during seismic shaking is nil.

FLOODING

According to the Federal Emergency Management Agency (FEMA, 2008), the southern portion of the subject fire station site is shown to be located within the boundaries of a 100-year flood (Community Panel Number 06071CC7944H, August 28, 2008. This zoned area is located along the existing wash boundaries that borders the southern portion of the site, labeled as “Zone A”. This zone is defined as “The annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceed in any given year. Zone A does not have the base flood elevations determined.

The remainder of the subject site in the north is shown to be located within "Zone X" which is defined as “Areas determined to be outside the 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than one-foot, and areas protected by levees from 1% annual chance flood”. These zones, along with the approximated site boundaries, is shown below in Figure 2 for reference. According to the City of San Bernardino 100-Year Flood Plain Map (2005, Figure S-1), the 100-year flood zone is shown to be located along the wash limits in the south, with the remainder of the site not being included within any flood zone.

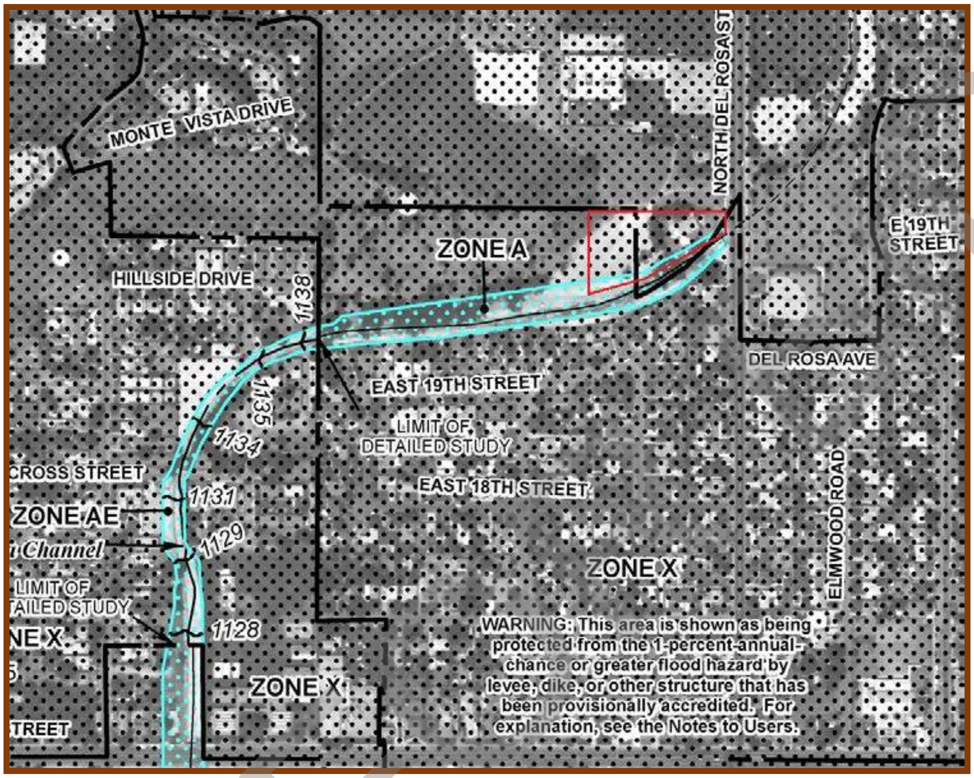


FIGURE 2- FEMA Flood Zone Map; Site boundary approximated by red outline.

GROUND SUBSIDENCE

Ground subsidence can be caused by natural geologic processes or by human activity such as groundwater and/or oil withdrawal and subsurface mining. Historic ground subsidence within the City of San Bernardino was generally located within the thick, poorly consolidated alluvial and marsh deposits of an old artesian area north of Loma Linda. Beginning in 1972, the San Bernardino Municipal Water District has maintained groundwater levels from recharge to percolation basins that, in turn, filter back into the alluvial deposits. Since the groundwater recharge program began, problems with ground subsidence in the valley have not been identified. According to the City of San Bernardino Potential Subsidence Areas Map (2005, Figure S-6), the subject site is not shown to be located within the limits of "Areas of Potential Ground Subsidence".

OTHER GEOLOGIC HAZARDS

There are other potential geologic hazards not necessarily associated with seismic activity that occur statewide. These hazards include; natural hazardous materials (such as methane gas, hydrogen-sulfide gas, and tar seeps); Radon-222 gas (EPA, 1993); 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 new fire station facility 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 underlain by Quaternary age (Holocene and late Pleistocene) alluvial fan deposits, generally consisting of unconsolidated to slightly consolidated sand and pebble-boulder gravel. Additionally, late Holocene age very young wash deposits also underlie portions of the site, consisting of unconsolidated to locally cemented sand, gravel, and boulder deposits. Underlying these deposits at depth is metamorphic rock comprised of the Pelona Schist Formation (fine-grained quartzite and greenstone). Site-specific exploration performed by IFE indicates the proposed construction area to be mantled by interbedded clayey sand, silty sand, gravel with sand, and silty clayey sand with gravel, that are in a generally loose to dense condition, to a depth of at least 10½ feet, directly underlain by metamorphic bedrock that is slightly weathered, fractured, and very hard.

2. Groundwater was not encountered within the exploratory excavations performed by IFE to a depth of at least 25 feet. Historic and current groundwater data indicate that groundwater may have been as high as 85± feet in depth, locally. Due to the underlying crystalline metamorphic bedrock, free groundwater is not anticipated but could be found locally along fractures. No shallow groundwater conditions are anticipated to be encountered during construction.

3. Based on our literature research, there are no active faults that are known to traverse the subject construction area. The nearest zoned active fault is associated with the active San Andreas Fault (North Branch) located approximately 1.8± miles to the northeast, that has an estimated maximum moment magnitude of M_W 7.4.

4. The <u>primary</u> 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 development.

5. Based on our review of available geologic/geotechnical data and our field reconnaissance, there do not appear any permanent and/or transient secondary seismic hazards are expected to occur within the proposed construction area. Both seismically-induced settlement and landsliding appear to have only a low to very low potential to occur and do not appear likely.

Recommendations:

1. It is recommended that all structures be designed to at least meet the current California Building Code provisions in the latest 2019 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.

   When considering that a cascading rupture event could occur along the entire length of the San Andreas Fault Zone (which includes all segments), the resulting maximum moment magnitude earthquake is estimated to be M_W8.1, which should be used for seismic design purposes.

CLOSURE

Our conclusions and recommendations are based on a review of available existing geologic/seismic data and the provided site-specific provided 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

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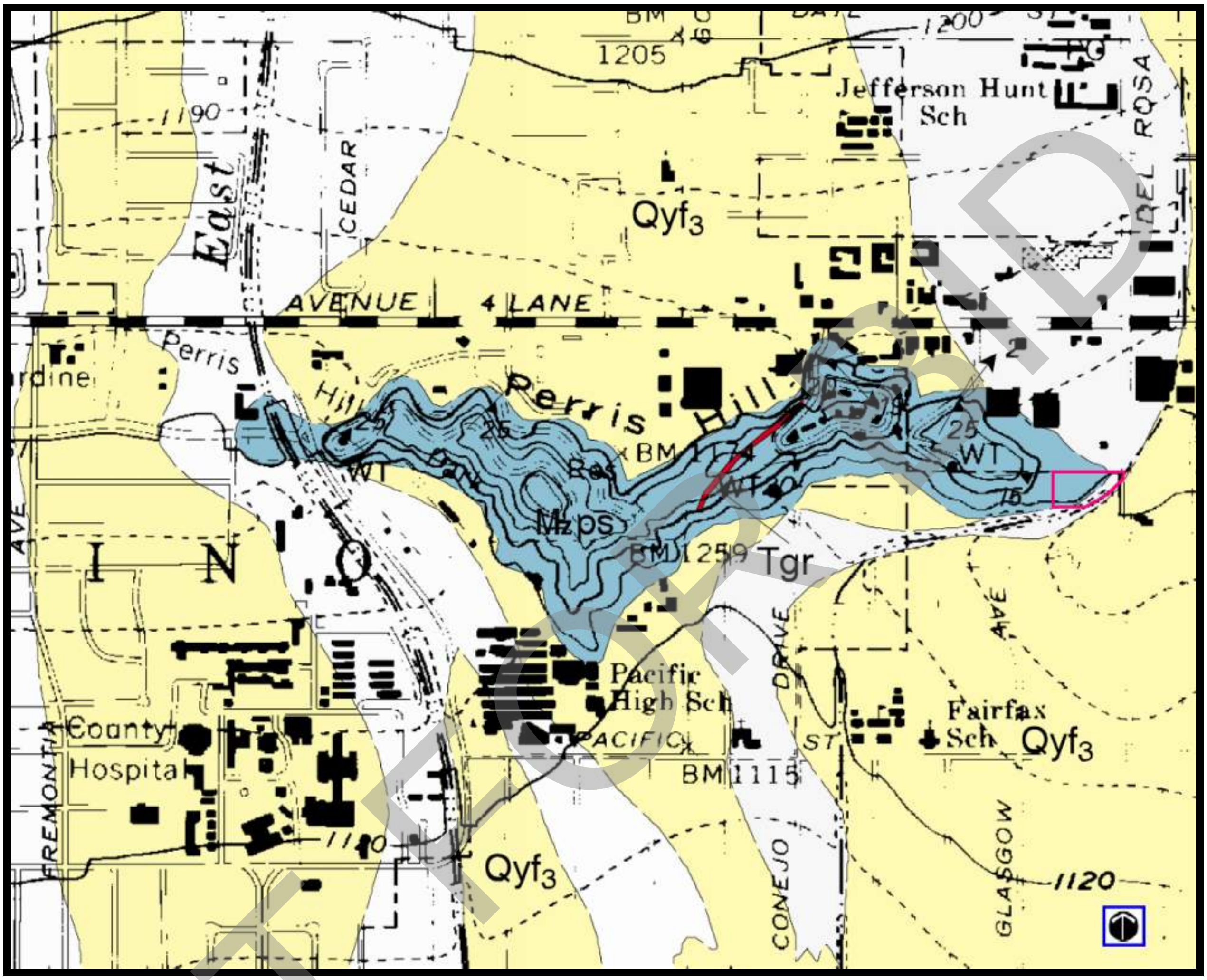
REGIONAL GEOLOGIC MAP

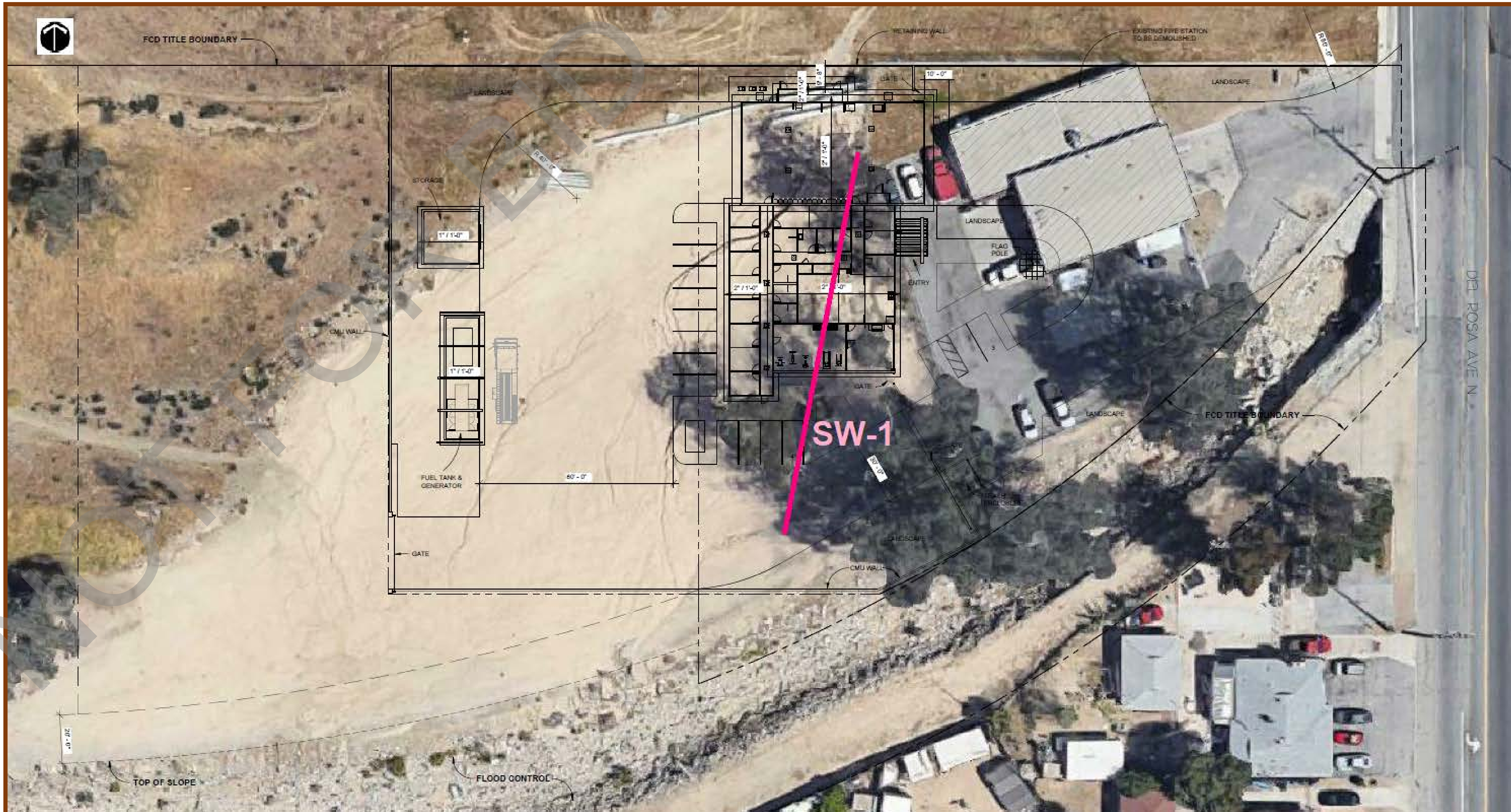


BASE MAP: Miller et al. (2001), U.S.G.S., Open File Report 01-131, Scale 1: 24,000, Site outlined in red.

PARTIAL LEGEND

Qw	**YOUNG WASH DEPOSITS**	Unconsolidated to locally cemented sand, gravel and boulders (late Holocene).
Qyf_3	**YOUNG FAN DEPOSITS**	Unconsolidated to slightly-consolidated sand and pebble-boulder gravel (Holocene and late Pleistocene).
Mzps	**PELONA SCHIST**	Muscovite-chlorite-albite-quartz schist, fine-grained (Mesozoic).
	GEOLOGIC CONTACT	Solid where well-located to approximately-located, dashed where inferred.

SITE PLAN



Base Map: Provided Preliminary Site Plan; Seismic 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 within proposed construction area, as approximated on the Site Plan (see Plate 2). For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP 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 161-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular seven-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 25-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. (2009). 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 **2,018.9** feet per second (615.4 meters/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 Site Class "**C**" (Very Dense Soil and Soft Rock profile), which has a velocity range from 1,200 to 2,500 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).

$$\mathbf{Vs = 100/[(d1/v1) + (d2/v2) + ...+ (dn/vn)]}$$

Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, 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 215-foot profile where locally measured. The constrained data is 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.

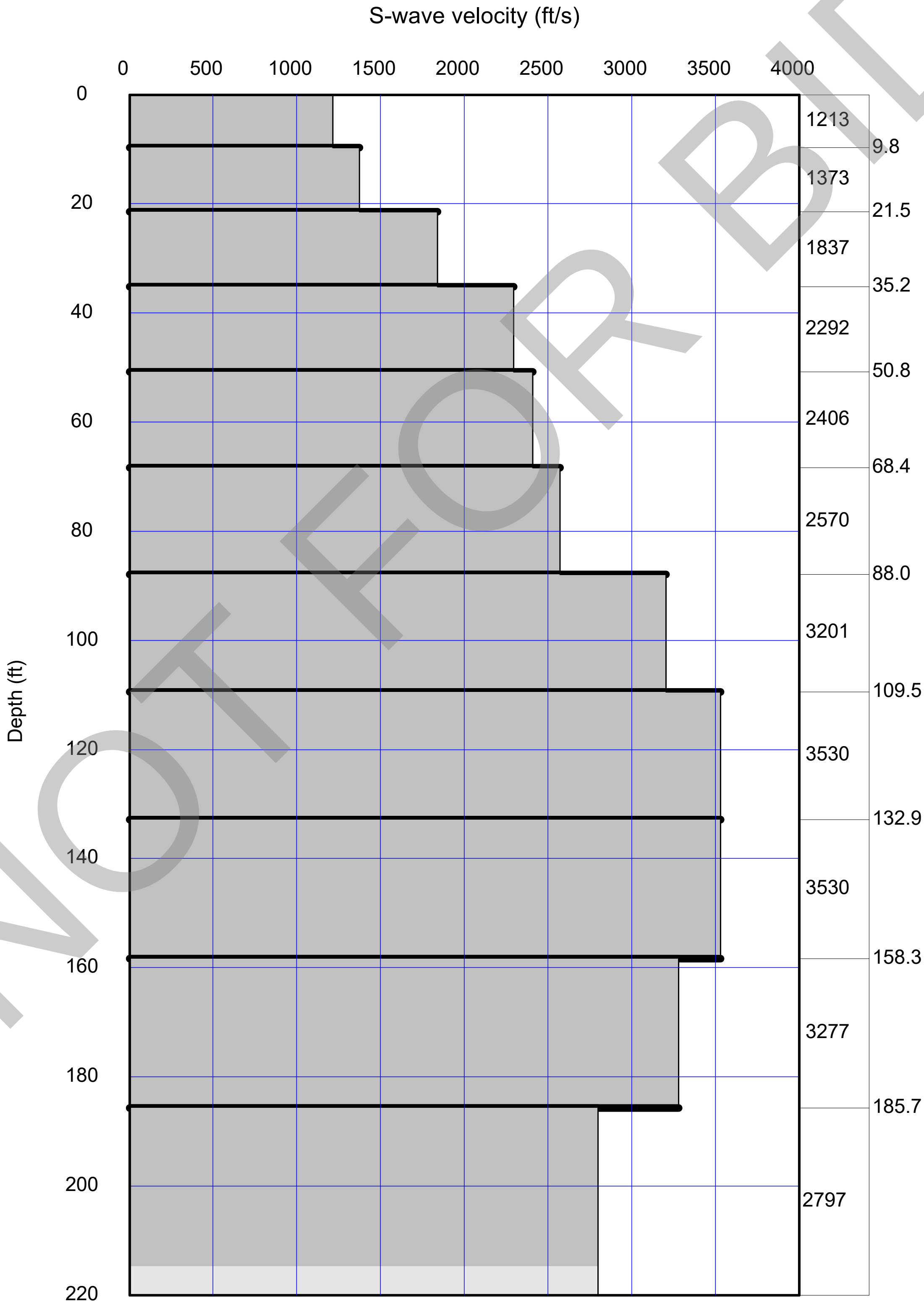
SURVEY LINE PHOTOGRAPHS

View looking southwest along Seismic Line SW-1.

View looking northeast along Seismic Line SW-1.

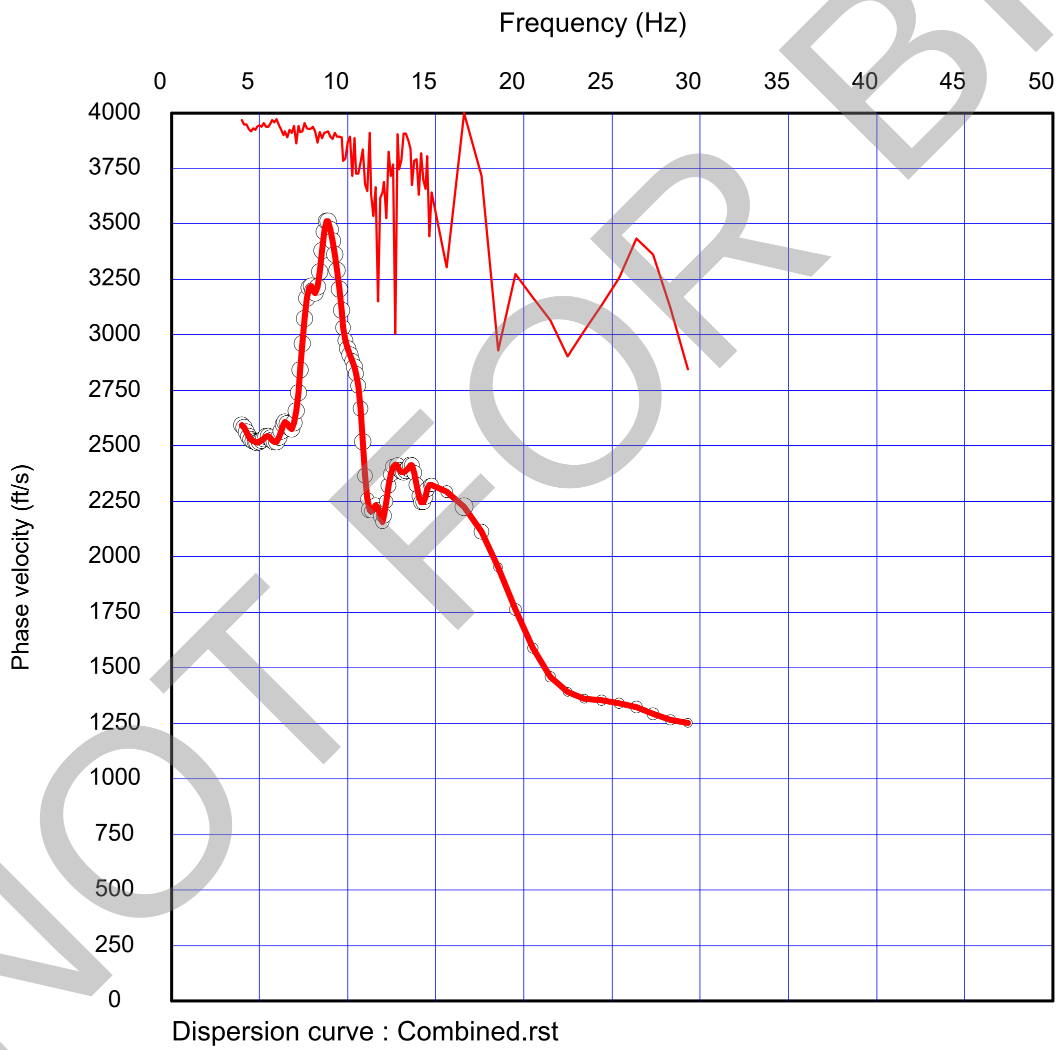
SEISMIC LINE SW-1

SHEAR-WAVE MODEL



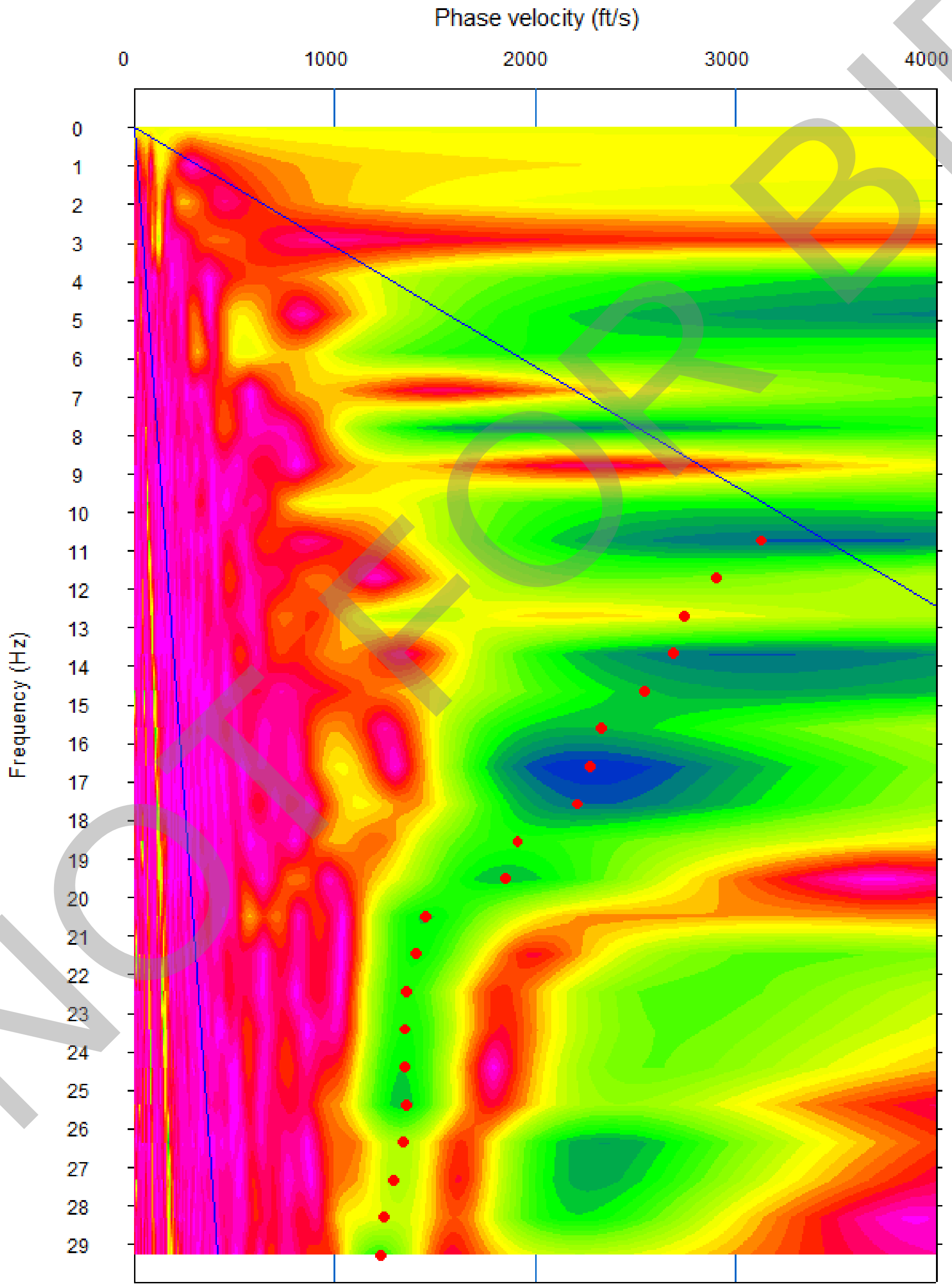
S-wave velocity model (inverted) : Final.rst **Average Vs 100ft = 2018.9 ft/sec**

SHEAR-WAVE MODEL SW-1



COMBINED DISPERSION CURVE

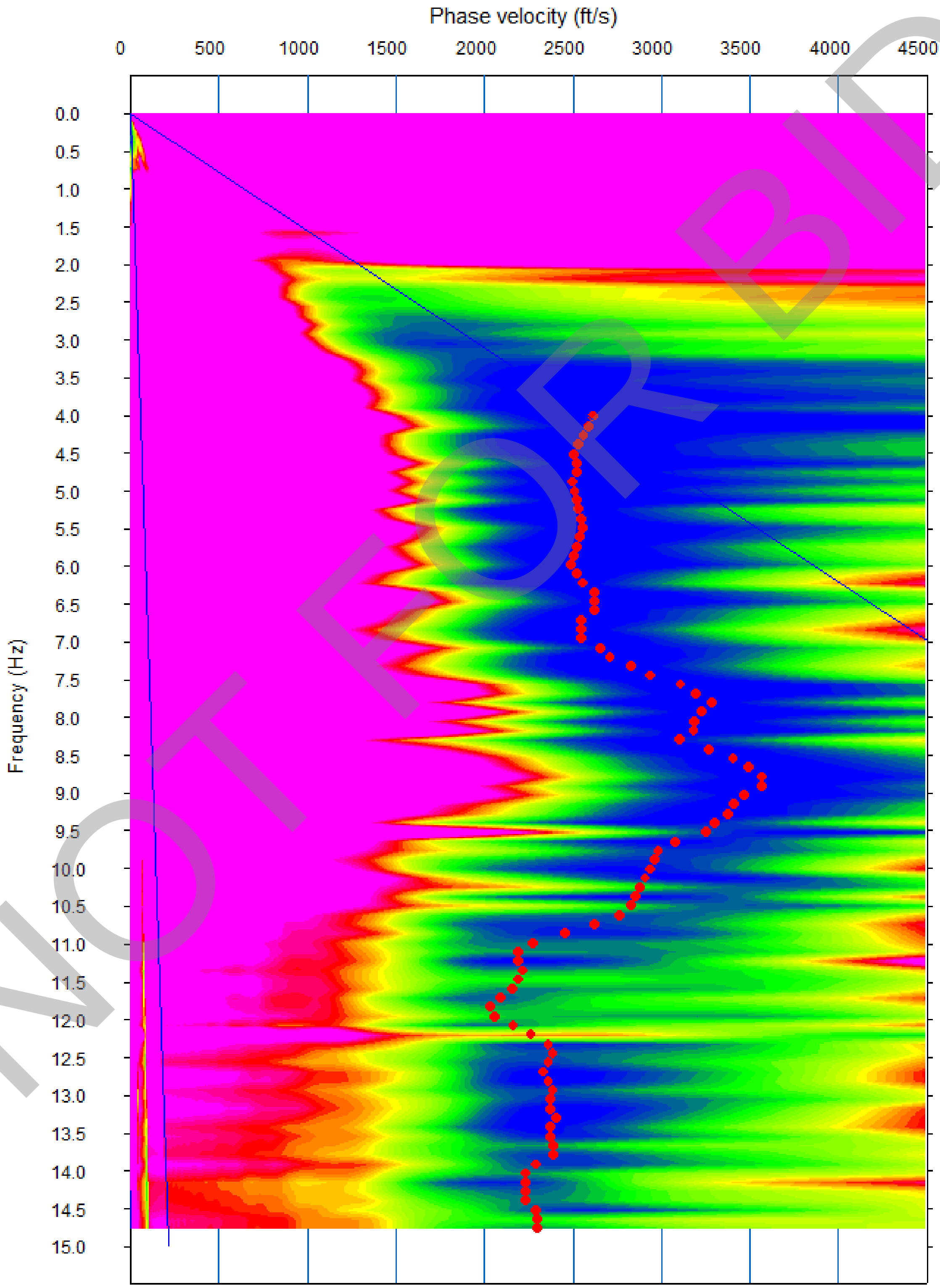
SEISMIC LINE SW-1



Dispersion Cure: 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 2019 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 **2.422g** for the 0.2 second period (S_s) and **0.894g** 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 2,018.9 feet/second (615.4 meters/second), the soil profile type used should be Site Class **"C."** 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 1,200 to 2,500 feet/second (360 to 760 meters/second), as detailed within Appendix A.

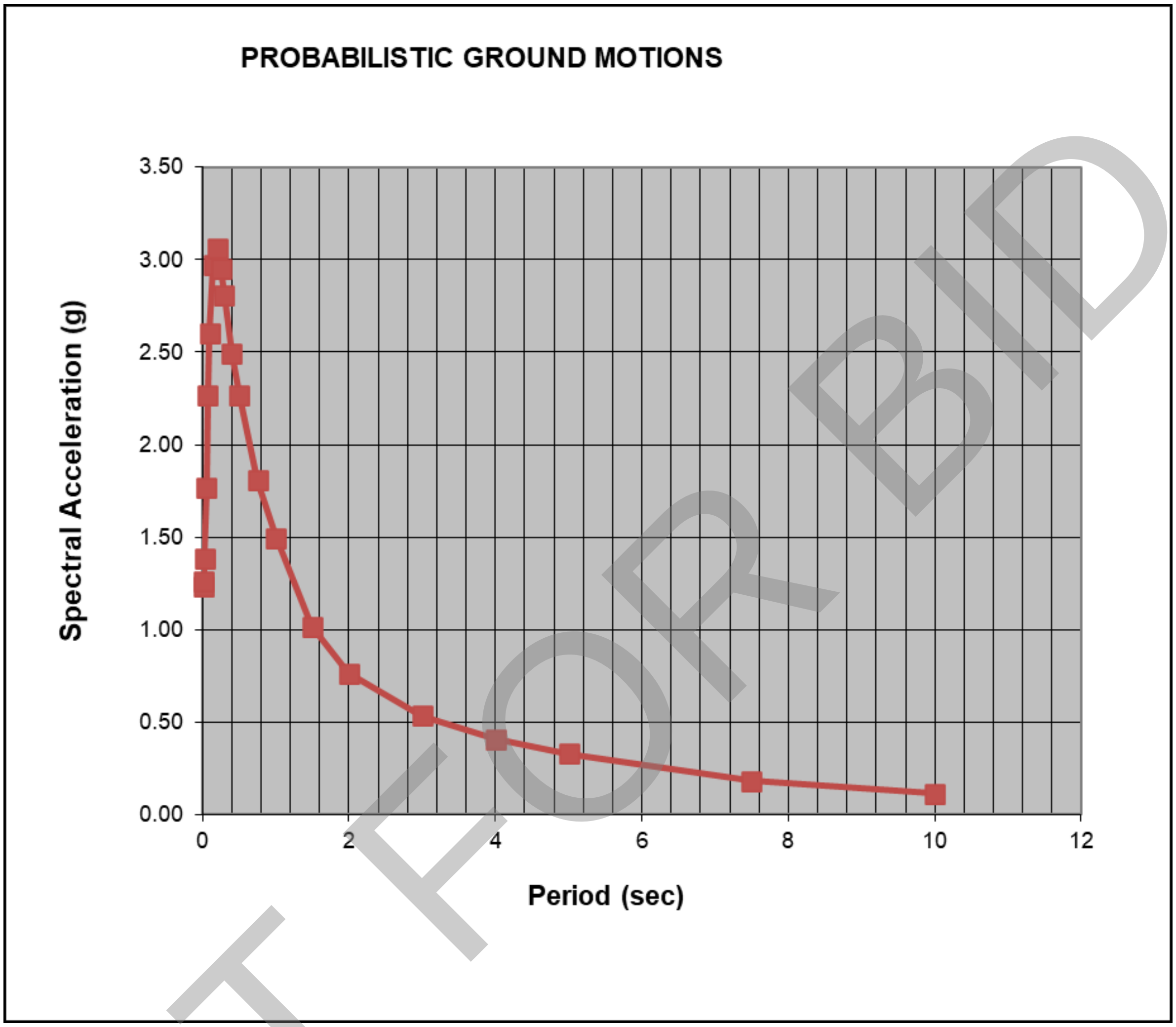
- **Site Coefficients (CBC 1613A.2.3)**-

  Based on CBC Tables 1613A.2.3(1) and 1613A.2.3(2), the site coefficient $\mathbf{F_a = 1.2}$ and $\mathbf{F_v = 1.4}$, respectively.

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

  Per Section 21.2.1, 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:



- **<u>Deterministic Spectral Response Analyses (ASCE 7 Section 21.2.2)</u>**-

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 (combined segments) and maximum magnitude of the San Andreas Fault Zone (southern section) located 2.8 kilometers to the northeast, a moment magnitude (M_W) used for this fault was 8.1.

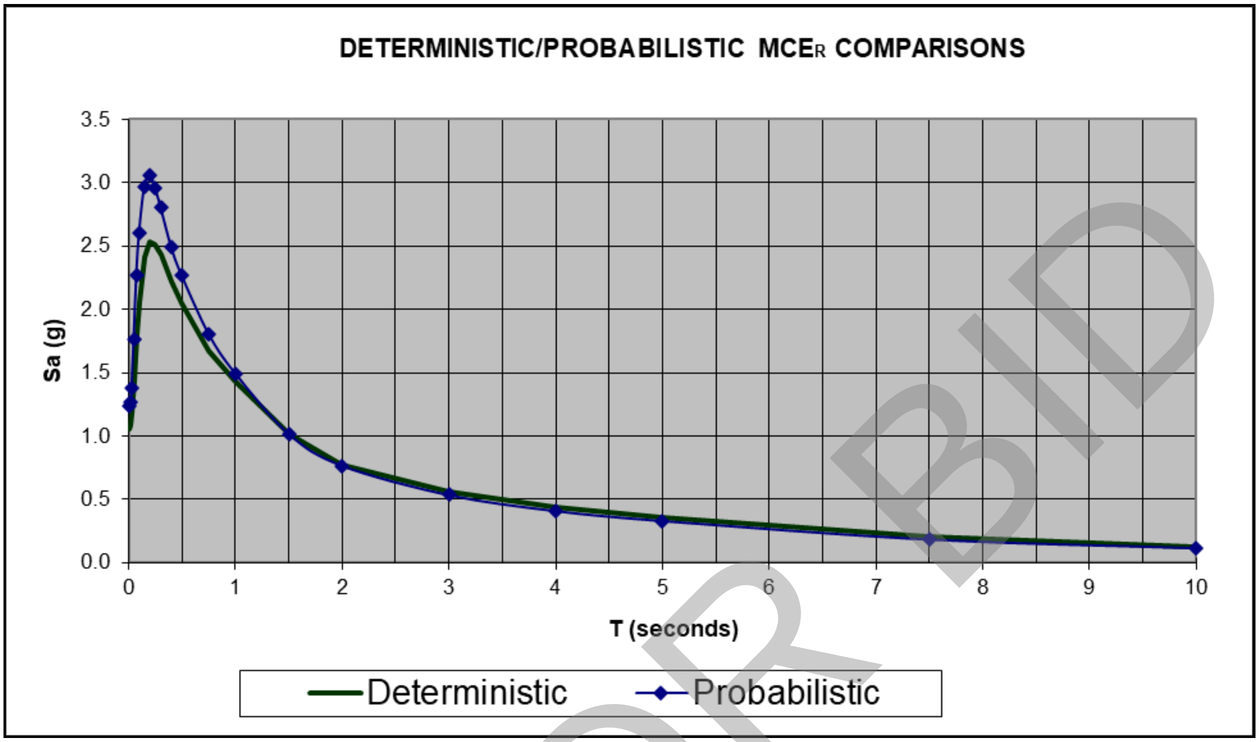
- **<u>Site Specific MCE_R (ASCE 7 Section 21.2.3)</u>-**

  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.

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

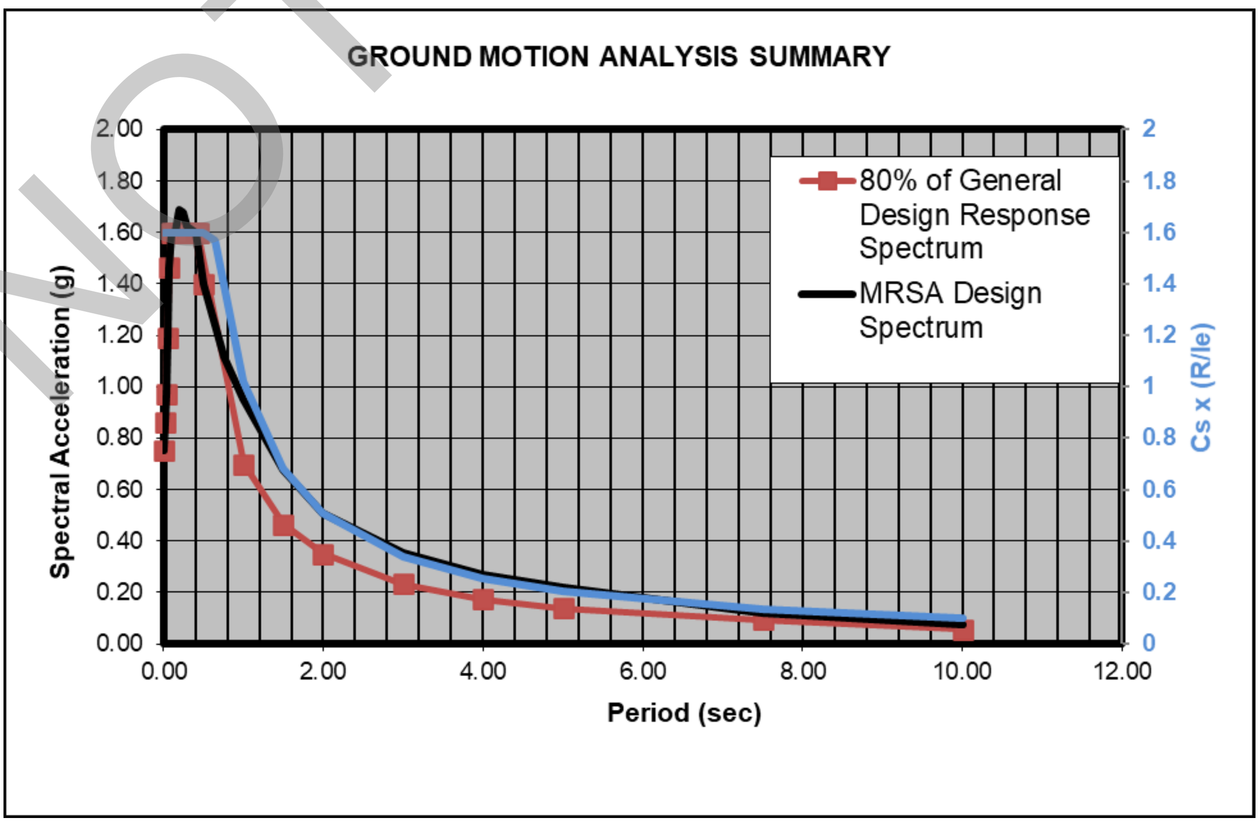
Period	Deterministic	Probabilistic	Lower Value	Governing Method
T	MCE_R	MCE_R	(Site Specific MCE_R)	
0.010	1.06	1.24	1.06	Deterministic Governs
0.020	1.08	1.26	1.08	Deterministic Governs
0.030	1.17	1.38	1.17	Deterministic Governs
0.050	1.44	1.77	1.44	Deterministic Governs
0.075	1.80	2.27	1.80	Deterministic Governs
0.100	2.06	2.60	2.06	Deterministic Governs
0.150	2.41	2.97	2.41	Deterministic Governs
0.200	2.53	3.06	2.53	Deterministic Governs
0.250	2.52	2.95	2.52	Deterministic Governs
0.300	2.43	2.81	2.43	Deterministic Governs
0.400	2.22	2.49	2.22	Deterministic Governs
0.500	2.05	2.27	2.05	Deterministic Governs
0.750	1.67	1.81	1.67	Deterministic Governs
1.000	1.43	1.49	1.43	Deterministic Governs
1.500	1.02	1.02	1.02	Deterministic Governs
2.000	0.77	0.77	0.77	Probabilistic Governs
3.000	0.56	0.53	0.53	Probabilistic Governs
4.000	0.44	0.41	0.41	Probabilistic Governs
5.000	0.36	0.33	0.33	Probabilistic Governs
7.500	0.20	0.18	0.18	Probabilistic Governs
10.000	0.13	0.11	0.11	Probabilistic Governs

These are plotted in the following diagram:



- <u>***Design Response Spectrum (ASCE 7 Section 21.3)***</u>-

In accordance with Section 21.3, the Design Response Spectrum was developed by the following equation: $S_a = 2/3S_{aM}$, 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 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 Sa * 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}), the maximum average acceleration for any period exceeding 0.2 seconds was 1.69g occurring at T=0.20 seconds. This was multiplied by 0.9 to produce a value of 1.52g. Since this value was less than 80% of the mapped S_{DS} value (which is 1.60g), 1.60g becomes the design S_{DS} Value. A value of 1.02g 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 2.400g (S_{MS}) was computed, along with a value of 1.530g (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 1.24g. The deterministic geometric mean peak ground acceleration (largest 84^{th} percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.96g. The site-specific MCE_G peak ground acceleration was calculated to be **0.99g**, which was determined by using the lesser of the probabilistic (1.24g) or the deterministic (0.96g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 1.24g x 0.80 = **0.99g**).

SEISMIC DESIGN PARAMETERS SUMMARY

Project:	SBC Fire Station 226	Lattitude:	34.13315
Project #:	223764-1	Longitude:	-117.25332
Date:	1/15/22		

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	2.5	Figure 22-1
S_1=	0.936	Figure 22-2

Site Class per Table 20.3-1

Site Class=	C - Very Dense Soil and Soft Rock

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a=	1.2	Table 11.4-1	=	1.20	For Site Specific Analysis per ASCE7-16 21.3
F_v=	1.40	Table 11.4-2	=	1.40	For Site Specific Analysis per ASCE7-16 21.3

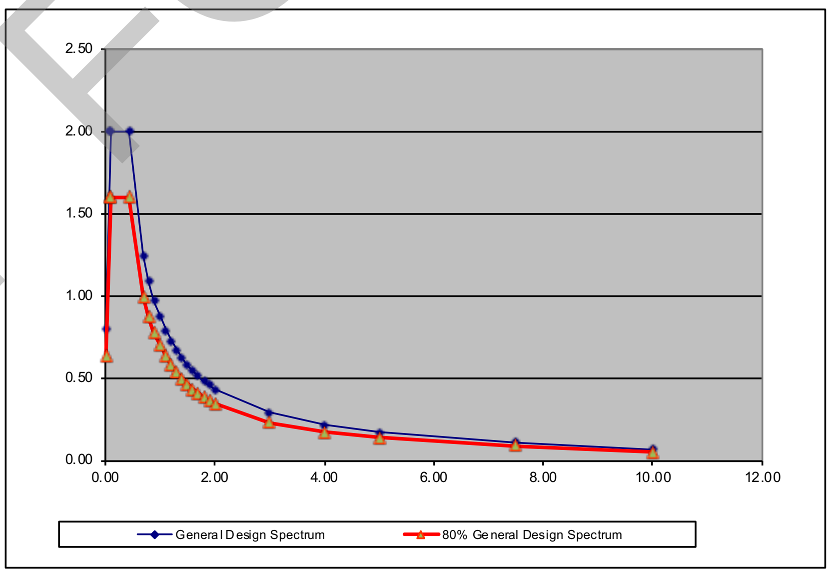
Mapped Design Spectral Response Acceleration Parameters

S_{MS}=	3	Equation 11.4-1	3	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	1.310	Equation 11.4-2	1.310	For Site Specific Analysis per ASCE7-16 21.3

T_0=	0.087	sec	
T_S=	0.437	sec	
T_L=	8	sec	From Fig 22-12
PGA	1.032	g	
F_{PGA}=	1.2		From Table 11.8-1
C_{RS}=	0.905		Figure 22-17
C_{R1}=	0.886		Figure 22-18

S_{DS}=	2.000	Equation 11.4-3
S_{D1}=	0.874	Equation 11.4-4

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.80	0.641
0.09	2.00	1.600
0.09	2.00	1.600
0.44	2.00	1.600
0.70	1.25	0.998
0.80	1.09	0.874
0.90	0.97	0.777
1.00	0.87	0.699
1.10	0.79	0.635
1.20	0.73	0.582
1.30	0.67	0.538
1.40	0.62	0.499
1.50	0.58	0.466
1.60	0.55	0.437
1.70	0.51	0.411
1.80	0.49	0.388
1.90	0.46	0.368
2.00	0.44	0.349
3.00	0.29	0.233
4.00	0.22	0.175
5.00	0.17	0.140
7.50	0.12	0.093
10.00	0.07	0.056



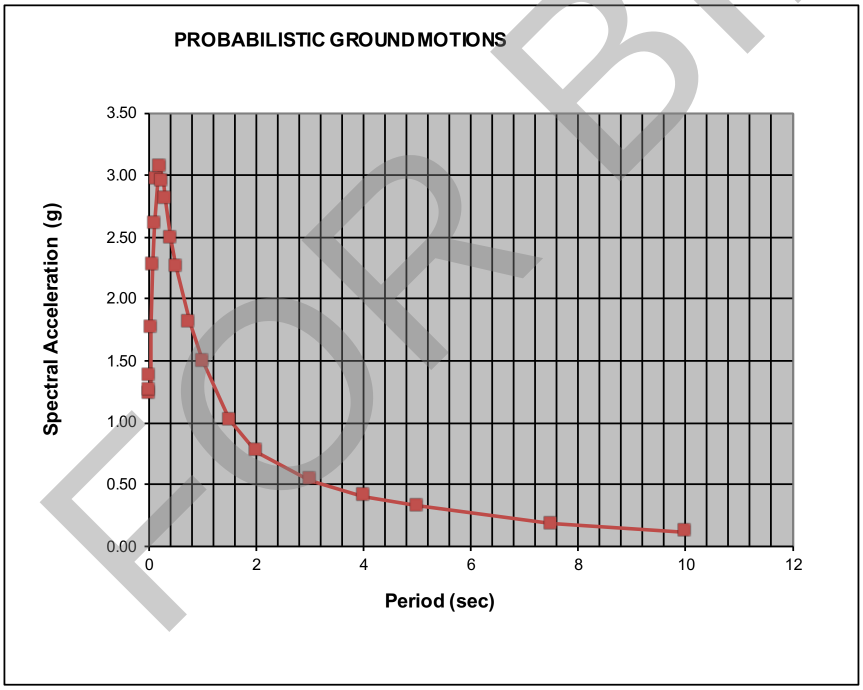
ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

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

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
Field, E.H., T.H. Jordan, and C.A. Cornell (2003), OpenSHA: A Developing Community-Modeling Environment for Seismic Hazard Analysis, Seismological Research Letters, 74, no. 4, p. 406-419.
Earthquake Rupture Forecast - UCERF3 Single Branch ERF, Fault Model 3.1

PROBABILISTIC MCER per 21.2.1.1 **Method 1**
Risk Coefficients taken from Figures 22-18 and 22-19 of ASCE 7-16
OpenSHA data
2% Probability Of Exceedance in 50 years
Maximum Rotated Horizontal Component determined per ASCE7-16 Ssection 21.2

T	Sa 2% in 50	MCER
0.01	1.37	1.24
0.02	1.40	1.26
0.03	1.53	1.38
0.05	1.95	1.77
0.08	2.51	2.27
0.10	2.88	2.60
0.15	3.28	2.97
0.20	3.38	3.06
0.25	3.27	2.95
0.30	3.11	2.81
0.40	2.77	2.49
0.50	2.52	2.27
0.75	2.03	1.81
1.00	1.69	1.49
1.50	1.15	1.02
2.00	0.86	0.77
3.00	0.60	0.53
4.00	0.46	0.41
5.00	0.37	0.33
7.50	0.21	0.18
10.00	0.13	0.11



S_s=	3.38	3.06
S_1=	1.69	1.49
PGA	1.24 g	

Risk Coefficients:			
C_{RS}	0.905	Figure 22-18	Get from Mapped Values
C_{R1}	0.886	Figure 22-19	
Fa=	1.2	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

Input Parameters		
Fault		San Andreas
$\mathbf{M}$	= Moment magnitude	8.1
R_{RUP}	= Closest distance to coseismic rupture (km)	2.8
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	2.8
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	2.8
U	= Unspecified Faulting Flag (Boore et.al.)	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	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
Z_{TOR}	= Depth to top of coseismic rupture (km)	0
δ	= Average dip of rupture plane (degrees)	90
V_{S30}	= Average shear-wave velocity in top 30m of site profile	615.4
$F_{Measured}$		1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.2
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	1.1
Site Class		C
W (km)	= Fault rupture width (km)	12.5
F_{AS}	= 0 for mainshock; 1 for aftershock	0
σ	=Standard Deviation	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

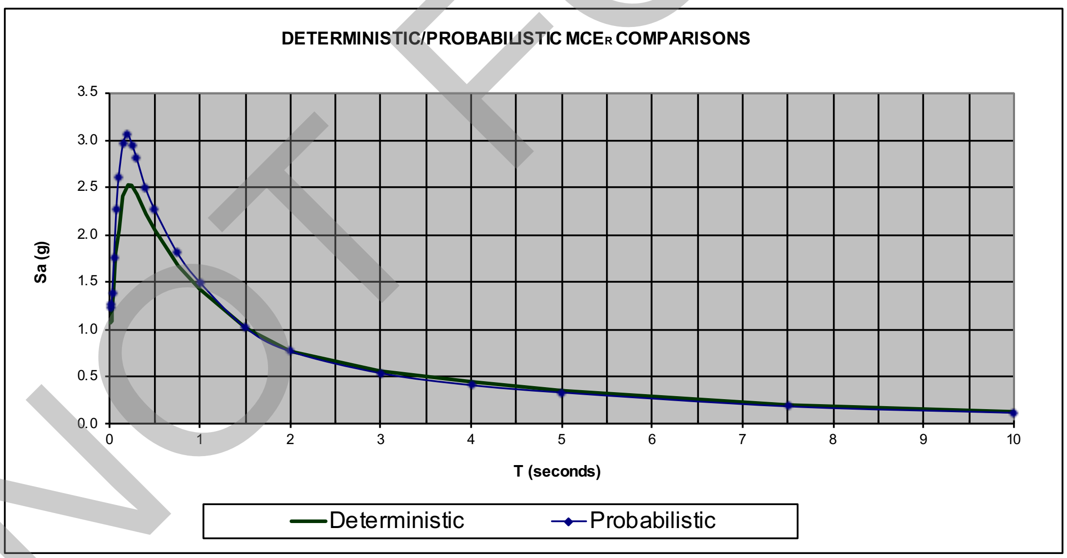
T	Median S_a (Average)	*Corrected** S_a (per ASCE7-16)	*Scaled* $S_{a(Average)}$
0.010	0.96	1.06	1.06
0.020	0.98	1.08	1.08
0.030	1.07	1.17	1.17
0.050	1.31	1.44	1.44
0.075	1.64	1.80	1.80
0.100	1.87	2.06	2.06
0.150	2.19	2.41	2.41
0.200	2.30	2.53	2.53
0.250	2.26	2.52	2.52
0.300	2.16	2.43	2.43
0.400	1.93	2.22	2.22
0.500	1.74	2.05	2.05
0.750	1.35	1.67	1.67
1.000	1.10	1.43	1.43
1.500	0.77	1.02	1.02
2.000	0.57	0.77	0.77
3.000	0.40	0.56	0.56
4.000	0.30	0.44	0.44
5.000	0.24	0.36	0.36
7.500	0.13	0.20	0.20
10.000	0.08	0.13	0.13
PGA	0.96		0.96 g
Max Sa=	2.53		
Fa =	1.20	Per ASCE7-16 21.2.2	
1.5XFa=	1.8		
Scaling Factor=	1.00		

* 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 T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific MCE_R)	Governing Method
0.010	1.06	1.24	1.06	Deterministic Governs
0.020	1.08	1.26	1.08	Deterministic Governs
0.030	1.17	1.38	1.17	Deterministic Governs
0.050	1.44	1.77	1.44	Deterministic Governs
0.075	1.80	2.27	1.80	Deterministic Governs
0.100	2.06	2.60	2.06	Deterministic Governs
0.150	2.41	2.97	2.41	Deterministic Governs
0.200	2.53	3.06	2.53	Deterministic Governs
0.250	2.52	2.95	2.52	Deterministic Governs
0.300	2.43	2.81	2.43	Deterministic Governs
0.400	2.22	2.49	2.22	Deterministic Governs
0.500	2.05	2.27	2.05	Deterministic Governs
0.750	1.67	1.81	1.67	Deterministic Governs
1.000	1.43	1.49	1.43	Deterministic Governs
1.500	1.02	1.02	1.02	Deterministic Governs
2.000	0.77	0.77	0.77	Probabilistic Governs
3.000	0.56	0.53	0.53	Probabilistic Governs
4.000	0.44	0.41	0.41	Probabilistic Governs
5.000	0.36	0.33	0.33	Probabilistic Governs
7.500	0.20	0.18	0.18	Probabilistic Governs
10.000	0.13	0.11	0.11	Probabilistic Governs



DESIGN RESPONSE SPECTRUM per Section 21.3

DESIGN ACCELERATION PARAMETERS per Section 21.4 (MRSA)

Period	2/3*MCE_R	80% General Design Response Spectrum (per ASCE 7-16 Figure 11.4-1)	Design Response Spectrum	TXSa
0.01	0.71	0.75	0.75	
0.02	0.72	0.86	0.86	
0.03	0.78	0.97	0.97	
0.05	0.96	1.19	1.19	
0.08	1.20	1.46	1.46	
0.10	1.37	1.60	1.60	
0.15	1.61	1.60	1.61	
0.20	1.69	1.60	1.69	
0.25	1.68	1.60	1.68	
0.30	1.62	1.60	1.62	
0.40	1.48	1.60	1.60	
0.50	1.36	1.40	1.40	
0.75	1.12	0.93	1.12	
1.00	0.95	0.70	0.95	0.95
1.50	0.68	0.47	0.68	1.02
2.00	0.51	0.35	0.51	1.02
3.00	0.36	0.23	0.36	1.07
4.00	0.27	0.17	0.27	1.09
5.00	0.22	0.14	0.22	1.10
7.50	0.12	0.09	0.12	
10.00	0.08	0.06	0.08	

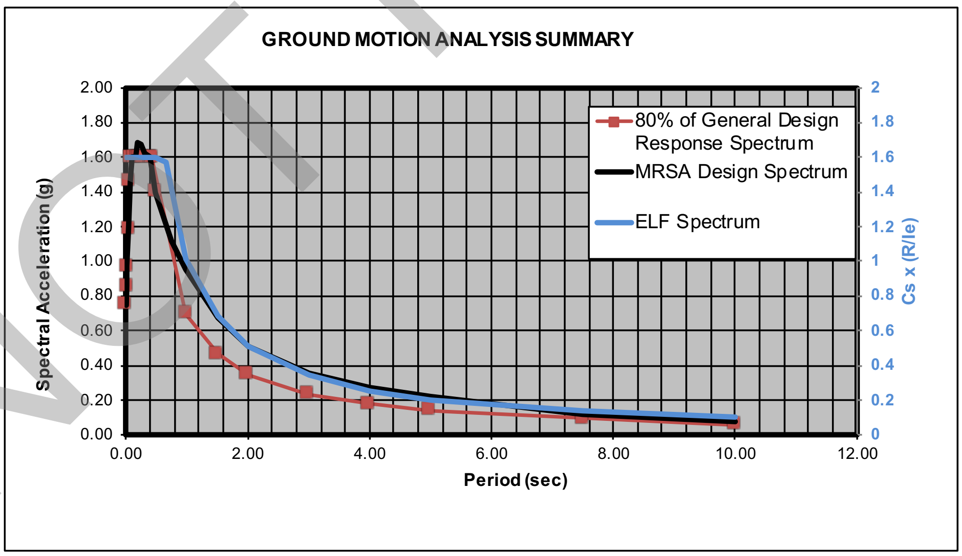
Highest value of S_a for any period exceeding 0.2 sec.=	1.69
90%of Highest Value =	1.52
80% of Mapped S_{DS}=	1.60
Max Tsa from T=1s-2s =	1.02
80% of Mapped S_{D1}=	0.70

S_{DS}=	1.60
S_{D1}=	1.02
T_s =	0.64

S_{MS}=	2.400
S_{M1}=	1.530

PGA Determination:

Site Coefficient F_{PGA}=	1.2	
Mapped PGA=	1.03	Figure 22-7
PGA_M =	1.24	g
Deterministic PGA =	0.96	g
Probabilistic PGA =	1.24	g
Lesser of Deterministic/Probabilistic =	0.96	g
80% of PGA_M=	0.99	g
MCE_G PGA=	0.99	g



APPENDIX C

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REFERENCES

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