IFE

Inland Foundation Engineering, Inc.

Revised October 10, 2023
Project No. S168-185

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

Subject: Revised Geologic Hazards Evaluation/Geotechnical Investigation
San Bernardino County Fire Station No. 305
8331 Caliente Road
Hesperia, 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 the presence of expansive native soil and undocumented fill soil in the building area.

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

Respectfully,
INLAND FOUNDATION ENGINEERING, INC.

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

ADE:es
Distribution: Addressee

inlandfoundation.com
1310 South Santa Fe Ave. PO Box 937 San Jacinto, CA 92581 I (951) 654-1555
77622 Country Club Dr. Suite Q, Palm Desert, CA 92211 I (760) 200-2400

TABLE OF CONTENTS

INTRODUCTION

This report presents the results of the geotechnical investigation conducted for a new metal building to be constructed on the site of County of San Bernardino Fire Station 305. The project site is located at 8331 Caliente Road in Hesperia, California.

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 2022 California Building Code (CBC) seismic design parameters.*
- *Subsurface sampling and laboratory testing.*
- *Analysis of the data collected and the preparation of this report with geotechnical conclusions and recommendations.*

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

SITE AND PROJECT DESCRIPTION

The ±3.5-acre fire station site is located within the southeastern portion of Section 28, Township 4 North, Range 5 West, S.B.B.&M. The rectangular-shaped parcel is located at 8331 Caliente Road in Hesperia, California. The Assessor Parcel Number for the property is 3039-351-09. The location of the fire station site is shown on Figure 1 below.

Figure 1: USGS Topographic Map, Baldy Mesa 7.5' Quadrangle and Aerial Photograph (2020)

The existing Fire Station 305 is located on the westerly portion of the property, with concrete paving, landscaping and a stormwater retention basin. The easterly portion of the property is undeveloped. Topographically, the site is relatively flat and slopes slightly to the north. Vegetation on the east portion of the site consists of a sparse growth of weeds and grass.

The proposed project will consist of the construction of a 50 ft. by 70 ft. metal building near the southeast corner of the site to be used for fire apparatus storage. We anticipate that foundations for the new building will consist of shallow spread and continuous footings with a concrete slab-on-grade floor. Grading is expected to consist of preparation of the structure building pad and new pavement area to the north. Cuts and fills on the order of 2 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 construction of the project appears feasible from a geologic standpoint, providing that the conclusions and recommendations presented in the report are considered during planning and construction. No unusual geologic hazards or conditions were observed during the field reconnaissance or literature research.

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, 2022), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (2017). The results of the site-specific analysis are summarized and tabulated in Table 1 below:

Table 1: Summary of Seismic Design Parameters

Factor or Coefficient	Value
S_S	1.500g
S_1	0.600g
F_a	1.0
F_V	1.7
S_{DS}	0.950g
S_{D1}	0.800g
S_{MS}	1.419g
S_{M1}	1.200g
T_L	12 Seconds
$MCE_G PGA$	0.59g
Shear-Wave Velocity (V100)	1,147.7 ft./sec.
Site Classification	D
Risk Category	III

SUBSURFACE CONDITIONS

Subsurface exploration at the site consisted of three (3) exploratory borings to depths ranging from approximately 16.5 to 51.5 feet below existing site grades. The site exploration is described in Appendix A. Boring locations are shown on Figure A-6.

Soil Classification, Density and Moisture Content: The soil encountered in the borings generally consisted of alluvial deposits predominately comprised of interbedded sandy clay (CL), silty sand and silty sand with gravel (SM), sand with gravel (SP), sand with silt (SP-SM) and silty clayey sand (SC-SM). Undocumented fill consisting of sand with gravel (SP), clayey sand and clayey sand with gravel (SC), and sandy gravel (GP) was encountered within the upper 3.5 to 4.5 feet of borings B-02 and B-03. The soil encountered was generally medium dense and slightly moist to moist at the time of drilling.

Corrosion Potential: Analytical testing indicates the concentration of sulfates is very low (64 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 20 ppm and indicates that the soil is generally not corrosive with respect to ferrous metal. The soil is alkaline with a pH value of 8.0. The saturated minimum resistivity value of 3,308 ohm-cm indicates the soil is moderately 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.

Expansive Soil: Potentially expansive soil is present with the proposed building area. A conventional slab-on grade can be used if supported by at least two feet of imported non-expansive soil. Refer to the Concrete Slabs-on-Grade section of this report.

Groundwater: Groundwater was not encountered within the exploratory borings, which extended to a maximum depth of 51.5 feet below the existing ground surface. Based on a review of pertinent groundwater data (referenced in appended geologic hazards report), groundwater is deeper than 800± feet in the general site vicinity.

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

CONCLUSIONS AND RECOMMENDATIONS

The primary geotechnical issues that will require mitigation are the presence of undocumented fill soil and expansive native soil within the proposed building pad and pavement areas. The soil is not suitable for support of foundations or pavement in its existing condition and should be over-excavated and recompacted. Additionally, the building floor slab should be designed to mitigate the effects of expansive soil, unless supported on imported non-expansive soil. These and other geotechnical engineering recommendations for project design and construction are presented below.

Foundation Design: The proposed storage building can be supported by shallow continuous and isolated spread footings designed with an allowable bearing pressure of 2,500 pounds per square foot (psf). Footings should have a minimum width of 12 inches with bottoms a minimum depth of 12 inches below the lowest adjacent grade. The allowable bearing pressure can be increased by ⅓ for short-term transient wind and seismic loads

Settlement of foundations properly designed and constructed as recommended herein is expected to be less than 1.0 inch total. Differential settlement between foundations of similar size and load is expected to be less than one-half inch.

Lateral Resistance: Resistance to lateral loads will be provided by a combination of friction acting at the base of the slab or foundation and passive earth pressure. A coefficient of friction of 0.50 between soil and concrete may be used with dead load forces only. A passive earth pressure of 300 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 35 pcf. Any applicable construction or seismic surcharges should be added to this pressure.

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: Potentially expansive soils are present within the proposed building area. Conventional slabs-on grade may be utilized if supported by at least 24 inches of imported non-expansive soil. Recommended import soil criteria are shown in Table 3 in the General Site Grading section of this report.

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 100 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. Control joint spacing in unreinforced concrete at maximum intervals equal to the slab thickness times 24 is recommended.

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder/barrier designed and constructed according to the American Concrete Institute 302.1 R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder/barrier construction. At a minimum, the vapor retarder/barrier should comply with ASTM EI745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage.

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

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	0.45
Heavy traffic (trucks, driveways, T.I. =7.0)	0.30	0.85

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 the project civil engineer or traffic engineer review the T.I. values used to verify that they are appropriate for this project.

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

2. **Preparation of Surfaces to Receive Compacted Fill:** All surfaces to receive compacted fill should be observed by a representative of this firm to verify the exposed soil conditions are as expected. If roots, other deleterious materials, or loose soil conditions are encountered, additional overexcavation may be necessary. 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 building should be over-excavated to a depth of at least 5 feet below existing grade or 2.0 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 aggregate base, 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. **Import Soil:** All proposed import soil should be tested prior to placement on the site to verify that it is not corrosive or expansive. Recommended import soil criteria are shown in the following Table 3.

Table 3: Recommended Import Soil Criteria

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

8. **Testing and Observation:** During grading, tests and observations should be performed by a representative of this firm to verify that the grading is performed per the project specifications. 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 on 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 firm 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 a new metal building at County of San Bernardino Fire Station 305. 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 Concrete Institute 318 (2019), Building Code Requirements for Structural Concrete

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

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

California Department of Transportation, 2022, Highway Design Manual

APPENDIX A –
Site Exploration

APPENDIX A

SITE EXPLORATION

Three exploratory borings were drilled with a truck-mounted hollow-stem auger drill rig at the approximate locations shown on Figure A-6. The materials encountered during drilling were logged by a staff geologist. Boring logs are included with this report as Figures A-3 through A-5.

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	DATE DRILLED	11/10/22	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.	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0	CL	**SANDY CLAY,** dark yellowish-brown (10YR 4/4), moist, hard, moderately cemented.	AU			
			SS	13 22	13	124
5	SM	**SILTY SAND,** fine to medium, strong brown, moist, medium dense.	AU			
	SM	**SILTY SAND with GRAVEL,** fine to coarse, dark yellowish-brown (10YR 4/4), slightly moist, medium dense, with thinly interbedded sand.	SS	14 19	4	120
10			SS	18 23	2	119
			SS	16 21	5	112
15	SP-SM	**SAND with SILT,** with trace gravel, fine to coarse, light olive-brown (2.5Y 5/3), moist, dense.	AU			
			SS	16 24	4	111
20	SM	**SILTY SAND,** very fine to fine, dark yellowish-brown (10YR 5/5), moist, medium dense.	AU			
			SS	10 17	13	116
25	SM	**SILTY SAND with GRAVEL,** fine to coarse, brown (7.5yr 5/2), slightly moist, medium dense.	SPT	22 21	5	
	SM	**SILTY SAND,** with trace gravel, fine to coarse, brown (7.5YR 5/2), sligfhtly moist, medium dense.	AU			
30			SPT	14 18	4	
35	SM	**SILTY SAND with GRAVEL,** fine to coarse, brown (7.5YR 5/3), slightly moist, dense, with thinly interbedded clayey sand.	SPT	14 22	4	
40	CL	**SANDY CLAY,** light gray-brown, moist, very stiff.				
	SM	**SILTY SAND,** with trace clay, fine to medium, brown (7.5YR 5/4), moist, medium dense.	SPT	8 13	12	
45	SC-SM	**SILTY, CLAYEY SAND,** fine to medium, yellowish-brown (10YR 5/6), moist, medium dense.	SPT	14 13	7	
50	SC-SM	**CLAYEY SAND,** very fine to fine, light olive-brown (2.5Y 5/4), moist, dense.	SPT	14 20	13	

End of boring at 51.5 feet. No groundwater encountered. Backfilled with native soil.

IFE Inland Foundation Engineering, Inc.

CLIENT	STK
PROJECT NAME	S.B. County Fire Station 305
PROJECT LOCATION	8331 Caliente Rd Hesperia, CA
PROJECT NUMBER	S168-185
FIGURE NO.	A-3

NOT FOR BID

LOG OF BORING B-02

DRILLING RIG	Mobile B-61	DATE DRILLED	11/10/22
DRILLING METHOD	Rotary Auger	HAMMER TYPE	Auto-Trip
LOGGED BY	FWC	HAMMER WEIGHT	140-lb.
GROUND ELEVATION	+/-	HAMMER DROP	30-inches
		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)
	SP		**ARTIFICIAL FILL,** SAND with GRAVEL, fine to coarse, yellow-brown, slightly moist, dense.			AU			
						SS	12 19	8	128
	SC		**ARTIFICIAL FILL,** CLAYEY SAND with GRAVEL, fine to medium, brown (10YR4/3), moist, dense.			AU			
5	CL		**SANDY CLAY,** brown (10YR 4/3), slightly moist, hard.			SS	20 27	4	
	SM		**SILTY SAND with GRAVEL,** with trace clay, fine to very coarse, dark yellowish-brown (10YR 4/4), slightly moist, medium dense.			AU SS	14 20	4	118
10						SS	17 24	4	117
15						SS	10 16	5	120
	SM		**SILTY SAND,** fine to coarse, grayish-brown (10YR 5/2), slightly moist, medium dense.			SS	17 21	4	113
20	SC-SM		**SILTY, CLAYEY SAND,** fine to medium, strong brown, moist, medium dense.						
	SM		**SILTY SAND,** fine to coarse, grayish-brown (10YR 4/4), slightly moist, medium dense to dense.			SPT	9 10	4	
25						SPT	11 15	4	
30	SP		**SAND with GRAVEL,** fine to coarse, yellowish-brown (10YR 5/4), slightly moist, medium dense.			SPT	10 14	5	

End of boring at 31.5 feet. No groundwater encountered. Backfilled with native soil.

IFE Inland Foundation Engineering, Inc.

CLIENT	STK	FIGURE NO.
PROJECT NAME	S.B. County Fire Station 305	
PROJECT LOCATION	8331 Caliente Rd	
	Hesperia, CA	
PROJECT NUMBER	S168-185	A-4

LOG OF BORING B-03

DRILLING RIG	Mobile B-61	DATE DRILLED	11/10/22
DRILLING METHOD	Rotary Auger	HAMMER TYPE	Auto-Trip
LOGGED BY	FWC	HAMMER WEIGHT	140-lb.
GROUND ELEVATION	+/-	HAMMER DROP	30-inches
		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.	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
	GP	**ARTIFICIAL FILL,** SANDY GRAVEL, fine to coarse, grayish-brown (10YR 3/2), slightly moist, medium dense.	AU			
			SS	11 15	1	120
	SC	**ARTIFICIAL FILL,** CLAYEY SAND, fine to medium, dark grayish-brown (10YR 4/2), moist, medium dense.				
	CL	**SANDY CLAY,** , dark grayish-brown (10YR 4/2), moist, stiff.	AU			
5			SS	9 11	9	125
	SC	**CLAYEY SAND,** with trace gravel, fine to medium, dark yellowish-brown (10YR 4/4), moist, medium dense.	AU			
			SS	8 13	9	129
10			SS	8 14	11	116
	SM	**SILTY SAND,** with trace gravel, fine to coarse, dark yellowish-brown (10YR 4/4), moist, medium dense.				
15			SS	14 21	3	112
	SP-SM	**SAND with SILT,** with trace gravel, fine to very coarse, dark gray (10YR 4/1), slightly moist, medium dense.				

End of boring at 16.5 feet. No groundwater encountered. Backfilled with native soil.

Inland Foundation Engineering, Inc.

CLIENT	STK	FIGURE NO.
PROJECT NAME	S.B. County Fire Station 305	
PROJECT LOCATION	8331 Caliente Rd Hesperia, CA	
PROJECT NUMBER	S168-185	A-5

SITE PLAN

SAN BERNARDINO COUNTY

FIRE STATION NO. 305

NEW METAL BUILDING

8331 CALIENTE ROAD, HESPERIA, CALIFORNIA

Base Map: Google Earth Imagery

N

⊕ **Approximate Location of Exploratory Boring**

IFE	**INLAND FOUNDATION ENGINEERING, INC.** **1310 S. Santa Fe Ave., San Jacinto, CA 92581** **(951) 654-1555**

Figure No. A-6	STK Architecture, Inc. San Bernardino County Fire Station No. 305 Hesperia, California	
	Drawn By: ES	Project No. S168-185
	Not to Scale	Date: December 2022

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-5).

Maximum Density-Optimum Moisture: One soil sample was selected for maximum density testing in accordance with ASTM D1557. The maximum density is compared to the field density of the soil to evaluate the existing relative compaction of the soil. The results of the testing are shown on Figure B-3.

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-4 and B-5.

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

Consolidation: Two samples were selected for consolidation testing in accordance with ASTM D2435. This test is used to evaluate the magnitude and rate of settlement of a structure or earth fill. The results are shown on Figure B-6 and B-7.

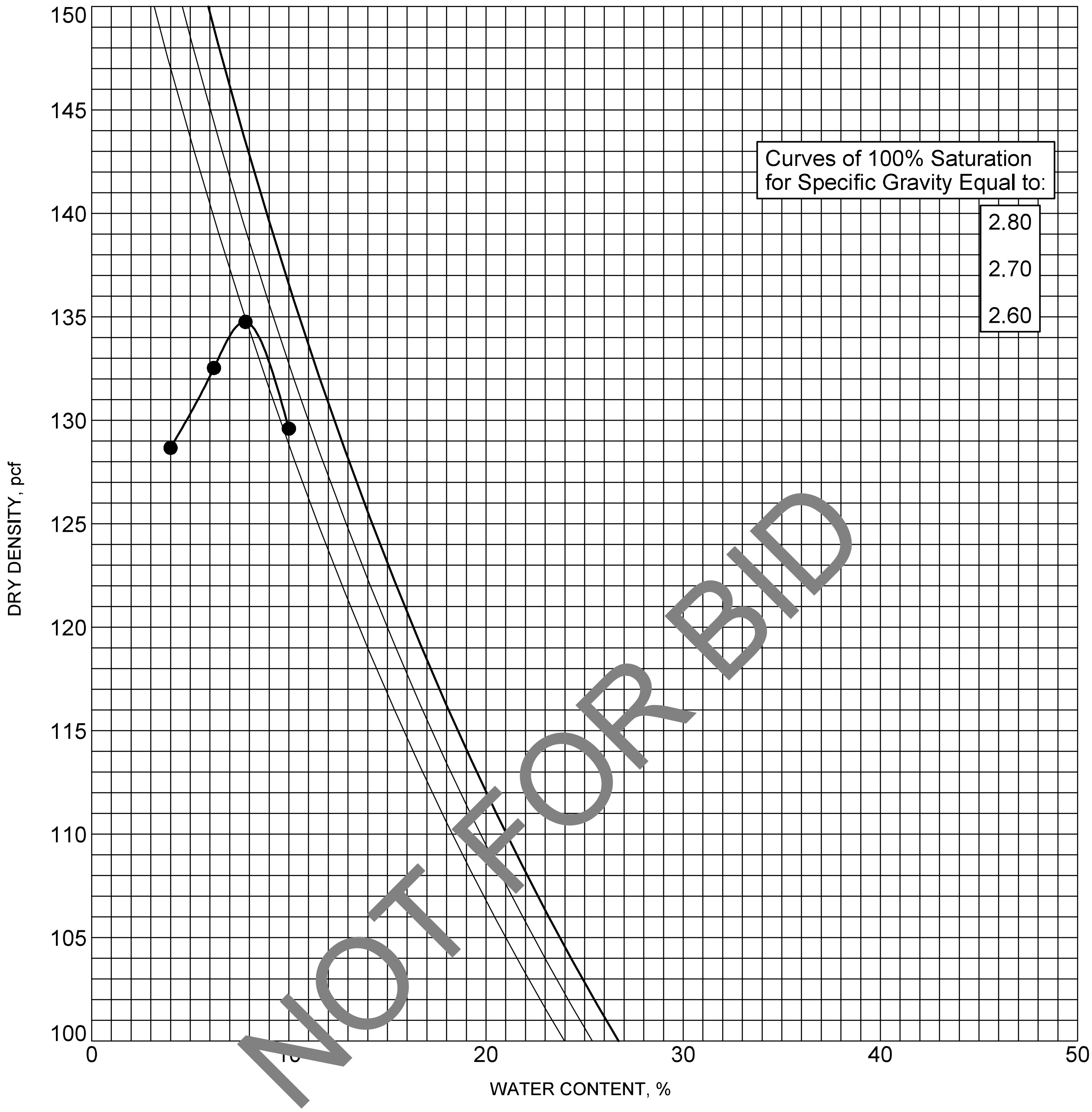
Direct Shear Strength: One sample was delivered to AP Engineering and Testing in Pomona, California for direct shear strength testing in accordance with ASTM D3080. This test measures the shear strength of the soil under various normal pressures and is used in developing parameters for foundation design and lateral earth pressure. The results are shown on Figure B-8.

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-9.

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-10.

Expansion Index: One sample was selected for expansion index testing in accordance with ASTM D4829. This test provides information regarding the expansive characteristics of soil under standardized test conditions. The following table presents the results of this testing.

Sample Location	Sample Depth (ft)	Initial Dry Density (pcf)	Initial Moisture Content (%)	Expansion Index	Expansion Class
B-01	0.0-4.0	116.6	8.4	37	Low



	BOREHOLE	DEPTH	Description of Materials	Max DD	Optimum WC
●	**B-02**	**2.5**	**CLAYEY SAND with GRAVEL (SC)**	**134.8 PCF**	**7.7 %**

MOISTURE-DENSITY CURVES (ASTM D1557)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-3

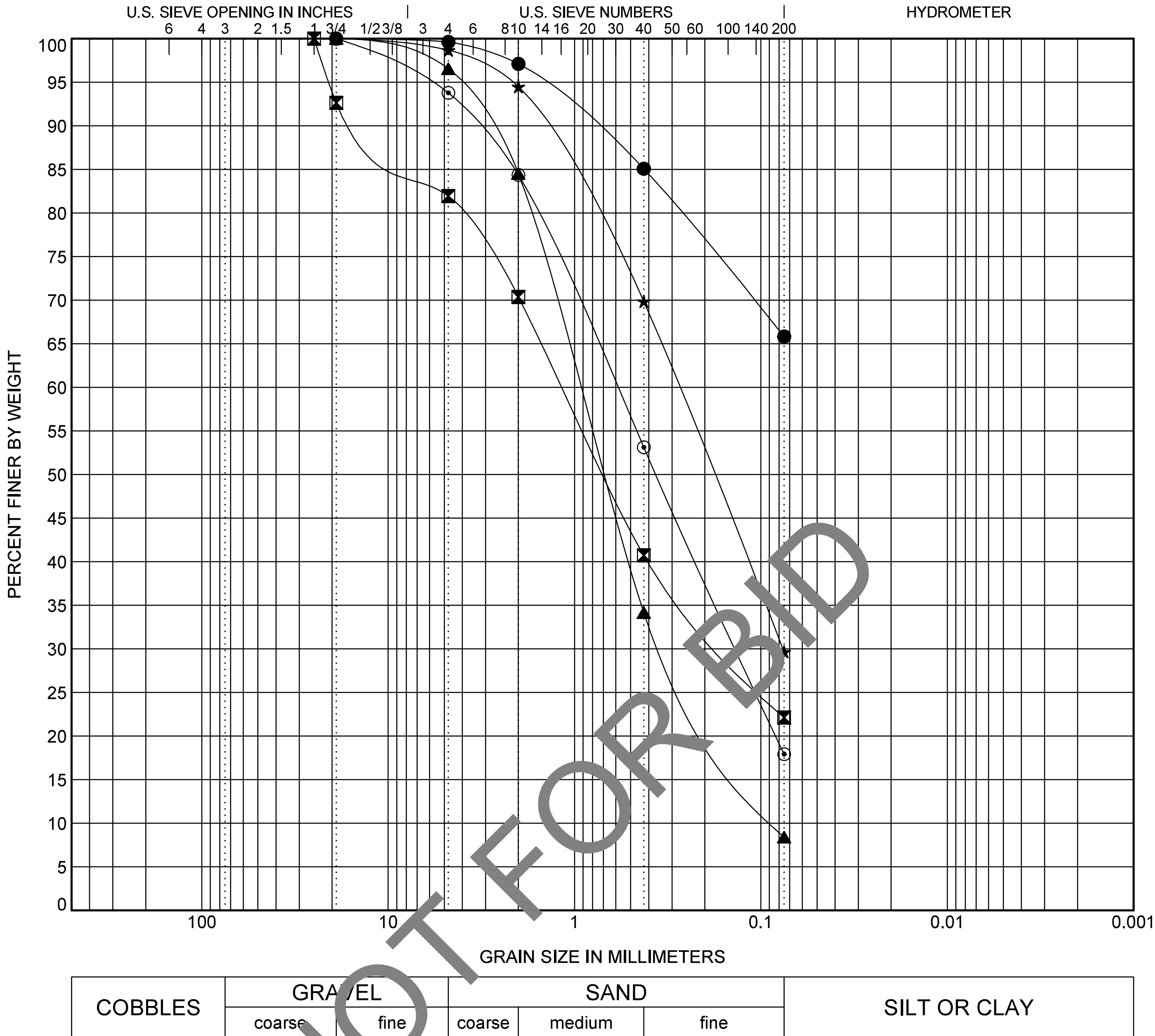
CLIENT STK

PROJECT NAME S.B. County Fire Station 305

PROJECT NUMBER S168-185

PROJECT LOCATION 8331 Caliente Rd
Hesperia, CA

IFE COMPACTION - GINT STD US LAB.GDT - 12/19/22 14:55 - P:\S168\S168-185 SB FIRE STATION 305\GINT.GPJ



	SAMPLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**0.0**	**SANDY LEAN CLAY (CL)**	**30**	**19**	**11**		
⧗	**B-01**	**5.3**	**SILTY SAND with GRAVEL (SM)**					
▲	**B-01**	**14.0**	**SAND with SILT (SP-SM)**				**1.31**	**11.26**
★	**B-01**	**19.5**	**SILTY SAND (SM)**	**NP**	**NP**	**NP**		
⊙	**B-01**	**26.0**	**SILTY SAND (SM)**					

	BOREHOLE	DEPTH	D100	D90	D50	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**0.0**	**19**	**0.802**			**0.4**	**33.8**	**65.8**	
⧗	**B-01**	**5.3**	**25**	**13.498**	**0.69**		**18.1**	**59.8**	**22.1**	
▲	**B-01**	**14.0**	**19**	**2.967**	**0.692**	**0.084**	**3.5**	**88.1**	**8.4**	
★	**B-01**	**19.5**	**19**	**1.509**	**0.18**		**1.3**	**69.0**	**29.6**	
⊙	**B-01**	**26.0**	**19**	**3.356**	**0.364**		**6.2**	**75.9**	**17.9**	

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

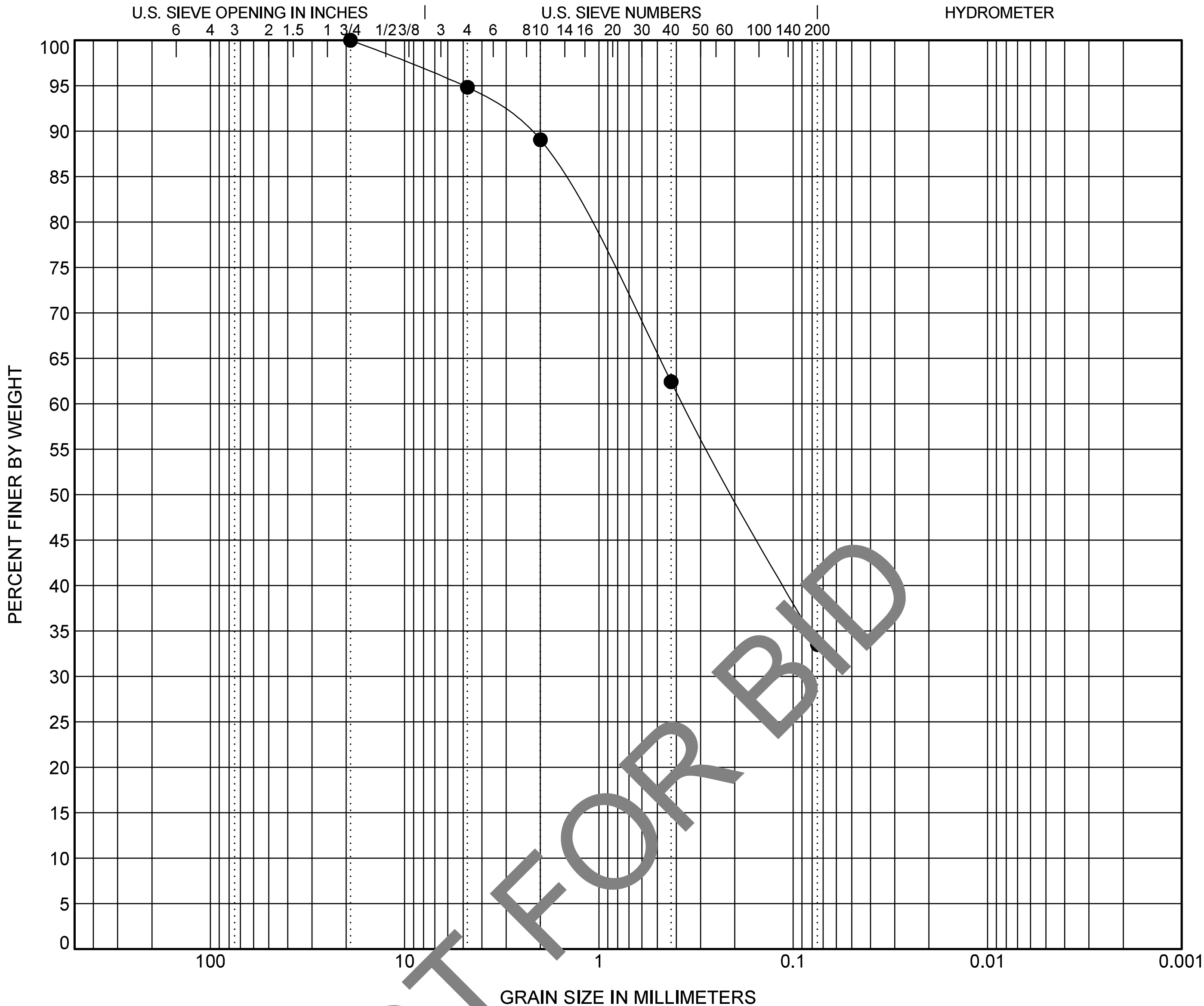
FIGURE NO. B-4

CLIENT STK

PROJECT NAME S.B. County Fire Station 305

PROJECT NUMBER S168-185

PROJECT LOCATION 8331 Caliente Rd
Hesperia, CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

NOT FOR BID

	SAMPLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	B-01	45.5	**SILTY, CLAYEY SAND (SC-SM)**					

	BOREHOLE	DEPTH	D100	D90	D50	D10	%Gravel	%Sand	%Silt	%Clay
●	B-01	45.5	19	2.305	0.202		5.2	61.3	33.5	

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-5

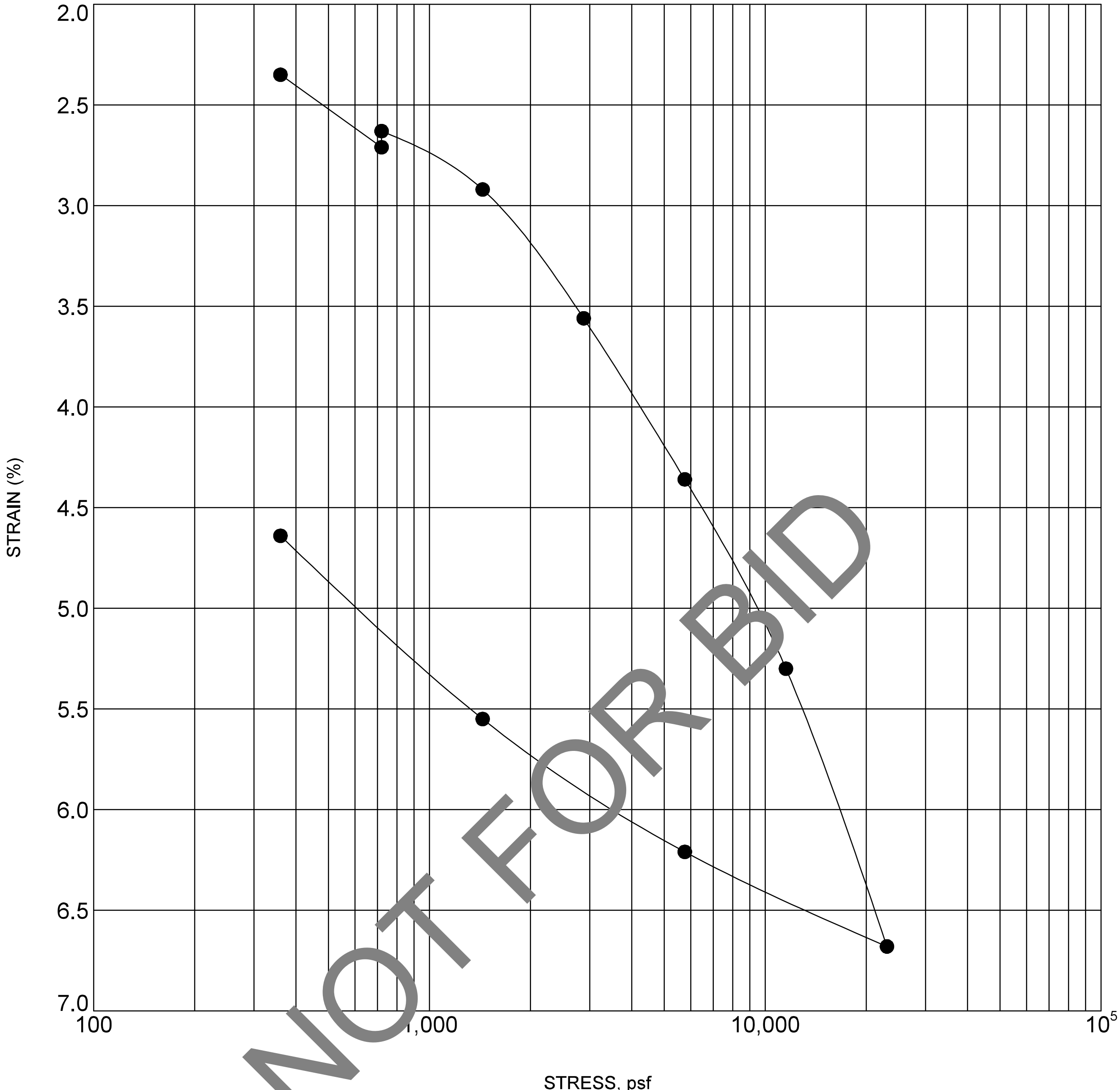
CLIENT STK

PROJECT NUMBER S168-185

PROJECT NAME S.B. County Fire Station 305

PROJECT LOCATION 8331 Caliente Rd
Hesperia, CA

IFE SIEVE ANALYSIS - GINT STD US LAB.GDT - 12/19/22 14:55 - P:\S168\S168-185 SB FIRE STATION 305\GINT.GPJ



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-01**	**2.5**	**SANDY CLAY (CL)**	**126**	**10**

CONSOLIDATION TEST (ASTM D2435)

FIGURE NO. B-6

INLAND FOUNDATION ENGINEERING, INC.

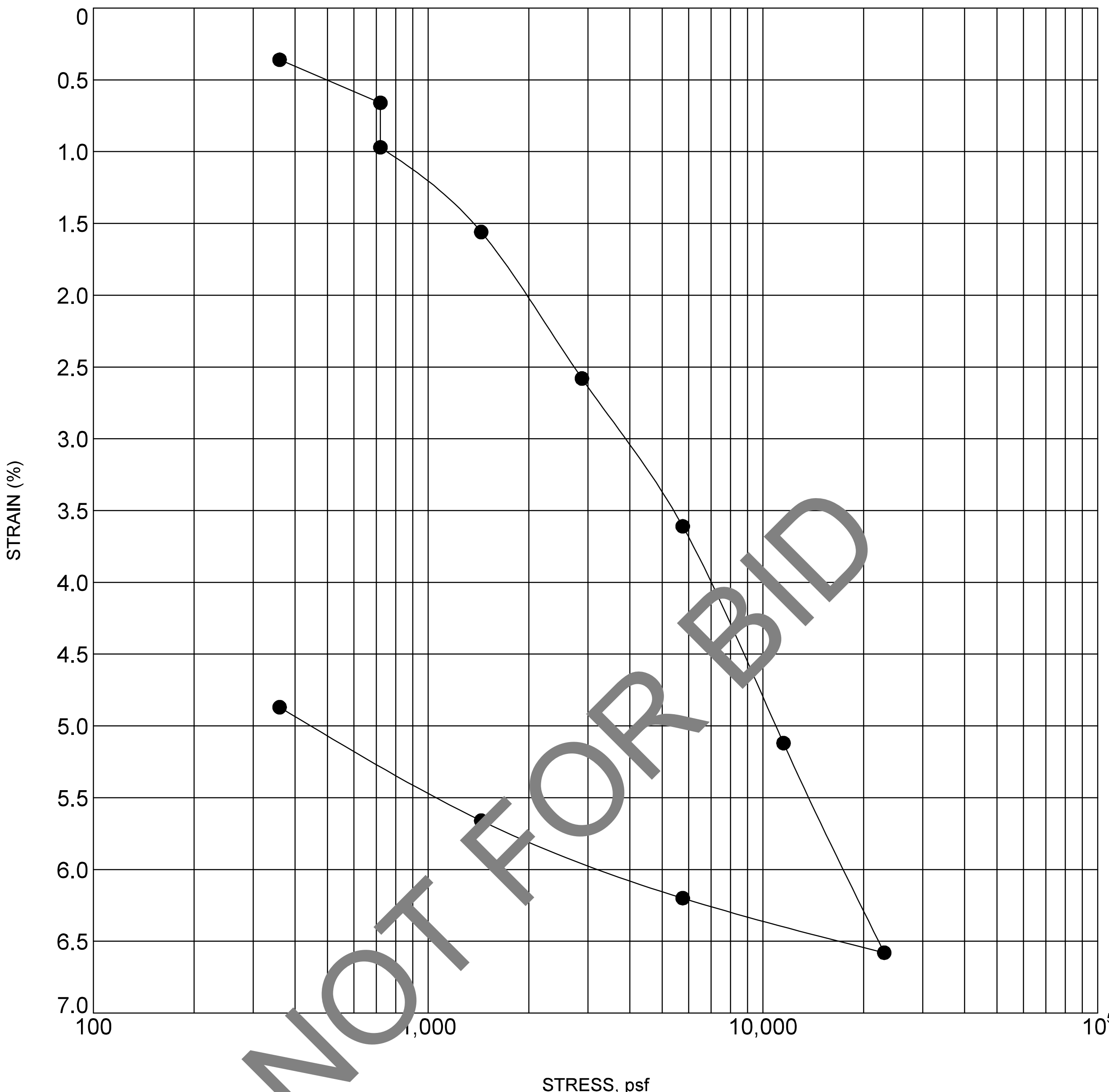
CLIENT STK

PROJECT NAME S.B. County Fire Station 305

PROJECT NUMBER S168-185

PROJECT LOCATION 8331 Caliente Rd
Hesperia, CA

IFE CONSOLIDATION - GINT STD US LAB.GDT - 12/19/22 16:50 - P:\S168\S168-185 SB FIRE STATION 305\GINT.GPJ



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-01**	**20.5**	**SILTY SAND (SM)**	**111**	**9**

CONSOLIDATION TEST (ASTM D2435)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-7

CLIENT STK

PROJECT NAME S.B. County Fire Station 305

PROJECT NUMBER S168-185

PROJECT LOCATION 8331 Caliente Rd
Hesperia, CA

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

CORROSION TEST RESULTS

Client Name: Inland Foundation Engineering
Project Name: STK - Fire Station 305
Project No.: S168-185

AP Job No.: 22-1215
Date: 12/13/22

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-1	-	5.25-14	Silty Sand w/gravel	3,[illegible]8	8.9	64	20

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

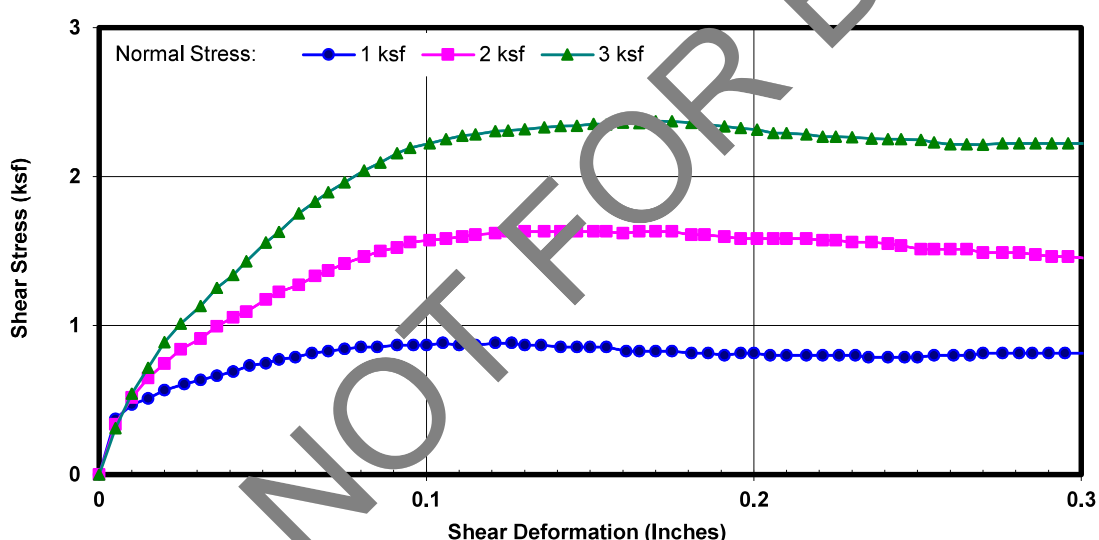
Figure No. B-8

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

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	STK - Fire Station 305	**Tested By:**	ST	**Date:**	12/13/22
Project No.:	S168-185	**Computed By:**	JP	**Date:**	01/15/22
Boring No.:	B-2	**Checked by:**	AP	**Date:**	12/15/22
Sample No.:	- **Depth (ft):** 4.5-5.5				
Sample Type:	Mod. Cal.				
Soil Description:	Silty Sand				
Test Condition:	Inundated **Shear Type:** Regular				

Wet Unit Weight (pcf)	Dry Unit Weight (pcf)	Initial Moisture Content (%)	Final Moisture Content (%)	Initial Degree Saturation (%)	Final Degree Saturation (%)	Normal Stress (ksf)	Peak Shear Stress (ksf)	Ultimate Shear Stress (ksf)
117.1	114.2	2.5	16.2	14	92	[illegible]	0.883	0.814
						2	1.632	1.452
						3	2.371	2.223



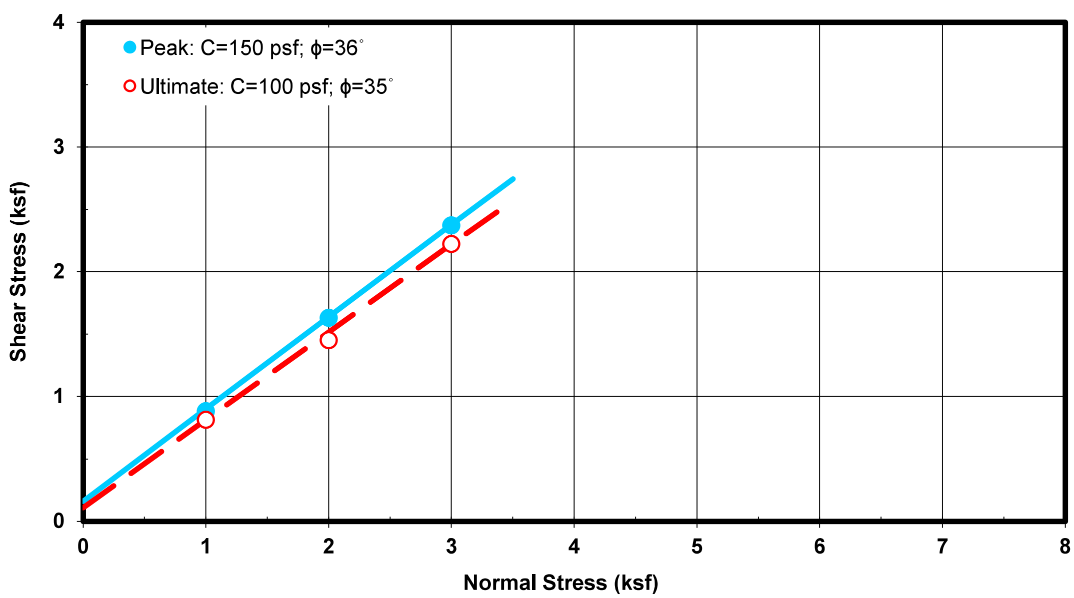


Figure No. B-9

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

R-VALUE TEST DATA

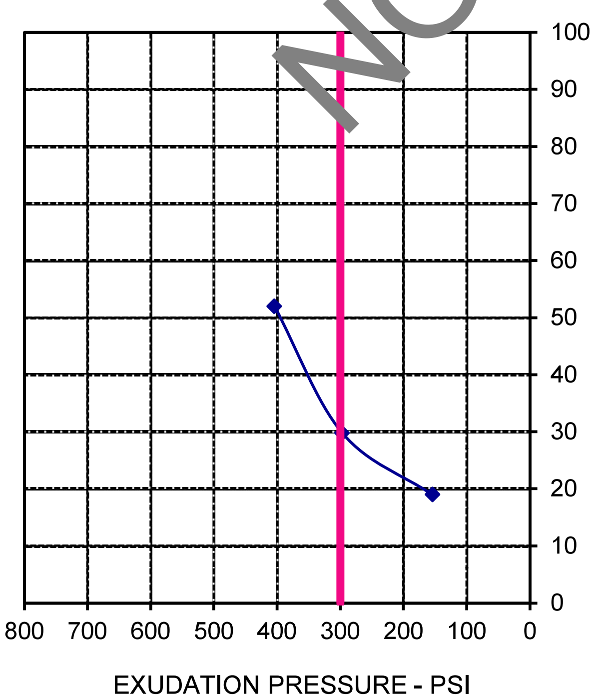
ASTM D2844

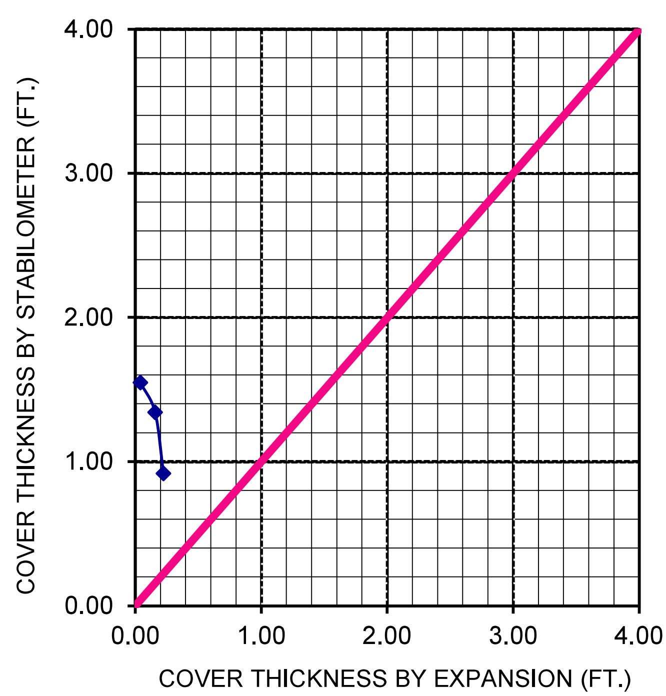
Project Name:	STK - Fire Station 305	Tested By:	ST	Date:	12/13/22
Project Number:	S168-185	Computed By:	KM	Date:	12/14/22
Boring No.:	B-1	Checked By:	AP	Date:	12/15/22
Sample No.:	-	Depth (ft.):	0-4		
Location:	N/A				
Soil Description:	Clayey Sand				

Mold Number	D	E	F	
Water Added, g	31	15	0	
Compact Moisture(%)	16.0	14.3	12.8	
Compaction Gage Pressure, psi	100	100	250	
Exudation Pressure, psi	154	297	404	
Sample Height, Inches	2.6	2.6	2.5	
Gross Weight Mold, g	3072	3068	2970	
Tare Weight Mold, g	1964	1954	1868	
Net Sample Weight, g	1108	1114	1102	
Expansion, inchesx10^{-4}	12	47	67	
Stability 2,000 (160 psi)	42/110	36/90	22/52	
Turns Displacement	5.27	4.92	4.79	
R-Value Uncorrected	18	28	52	
R-Value Corrected	19	30	52	
Dry Density, pcf	111.4	113.6	118.4	
Traffic Index	8.0	8.0	8.0	
G.E. by Stability	1.55	1.34	0.92	
G.E. by Expansion	0.04	0.16	0.22	

R-VALUE		
	By Exudation:	30
	By Expansion:	*N/A
	At Equilibrium: (by Exudation)	30

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





APPENDIX C –
Geologic Hazards Report

TERRA GEOSCIENCES

GEOLOGIC HAZARDS REPORT

PROPOSED NEW METAL BUILDING

SAN BERNARDINO COUNTY FIRE STATION NO. 305

8331 CALIENTE ROAD, HESPERIA, CALIFORNIA

Project No. 223896-1

December 2, 2022

Prepared for:

Inland Foundation Engineering, Inc.
1310 S. Santa Fe Avenue
San Jacinto, CA 92581

Consulting Engineering Geology & Geophysics

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

Inland Foundation Engineering, Inc.
1310 S. Santa Fe Avenue
San Jacinto, CA 92581

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

Regarding: Geologic Hazards Report
Proposed New Metal Building
San Bernardino County Fire Station No. 305
8331 Caliente Road, Hesperia, California
IFE Project No. S168-185

INTRODUCTION

At your request, this firm has prepared a geologic hazards report for the proposed new metal building, 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), along with the Geohazard Reports requirements outlined by the California Division of the State Architect (DSA, 2021). 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, Illustrations, and Appendices

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

GEOLOGIC SETTING

The subject site is located within a natural geomorphic province in southern California known as the Mojave Desert. This province consists of a broad interior region of isolated mountain ranges separated by expanses of desert plains, and is characterized by the numerous interior enclosed drainages and playas. The Mojave Desert is in large, bounded structurally on the southwest by the San Andreas Fault and on the northwest by the Garlock Faults, and is ill-defined along the east where the structural patterns resemble the Basin and Range Province to the north and east. This province exhibits interior drainage, including the Mojave River, which has its source in the San Bernardino Mountains and would extend into Death Valley if there was enough water. The geologic units of this region generally consist of three main divisions being: 1) Crystalline rocks of pre-Tertiary age; 2) sediments and volcanic rocks of Tertiary age; and 3) sediments and basalt flows of Quaternary age. Regionally, the site is located along a large alluvial plain, locally underlain by Quaternary age alluvium and older that has been derived predominantly as outwash from the San Bernardino and San Gabriel Mountains to the south and southwest, respectively. These sediments are believed to be as thick as 3,300± feet locally (Subsurface Surveys, 1990).

The Mojave Desert locally, extends north from the San Bernardino Mountains, which is an area of low relief consisting of largely alluvial fan deposits punctuated by the relatively low but rugged Granite Mountains a few miles east of the site. Figure 1 below depicts the major physiographic features of the region showing the subject site to be located within the Victorville Fan province.

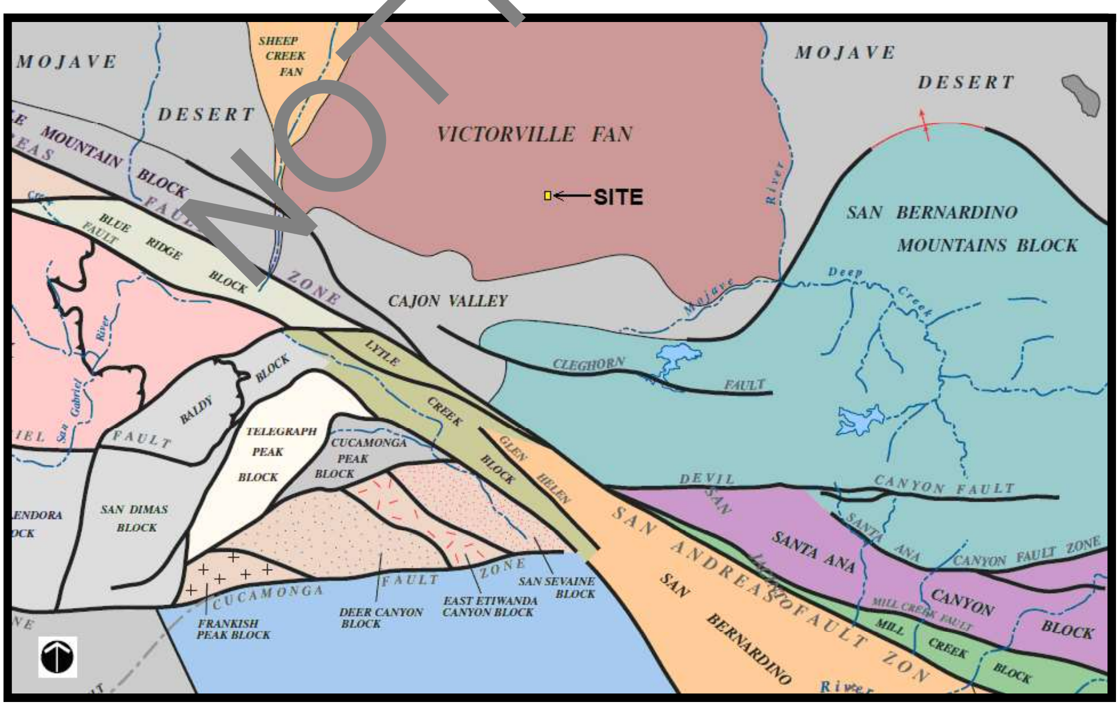


FIGURE 1- Major Physiographic Features (from Morton and Miller, 2006, Figure 3).

Locally as mapped by Morton and Miller (2006) and as shown on the Regional Geologic Map, Plate 1, the subject site is shown to be underlain by middle to early-middle Pleistocene age old fan deposits (map symbol Qvof). These deposits are generally described as being comprised of moderately- to well-consolidated silt, sand, and gravel. Subsurface exploratory boring excavations performed Inland Foundation Engineering, Inc. (IFE, 2022) indicate the subject site to be underlain by predominantly interbedded clayey sand, sandy clay, silty clayey sand, fine- to coarse-grained silty sand, fine- to medium-grained silty sand, silty sand with gravel, and fine- to coarse-grained sandy gravel, to a depth of at least 51½ feet. These sediments were noted to be in a medium/stiff to dense/hard condition.

FAULTING

There are at least forty-one major "potentially active/active" (late Quaternary) faults that are within a 100-kilometer (62 mile) radius of the site as shown on Figure 2 below (site shown as small black square in middle).

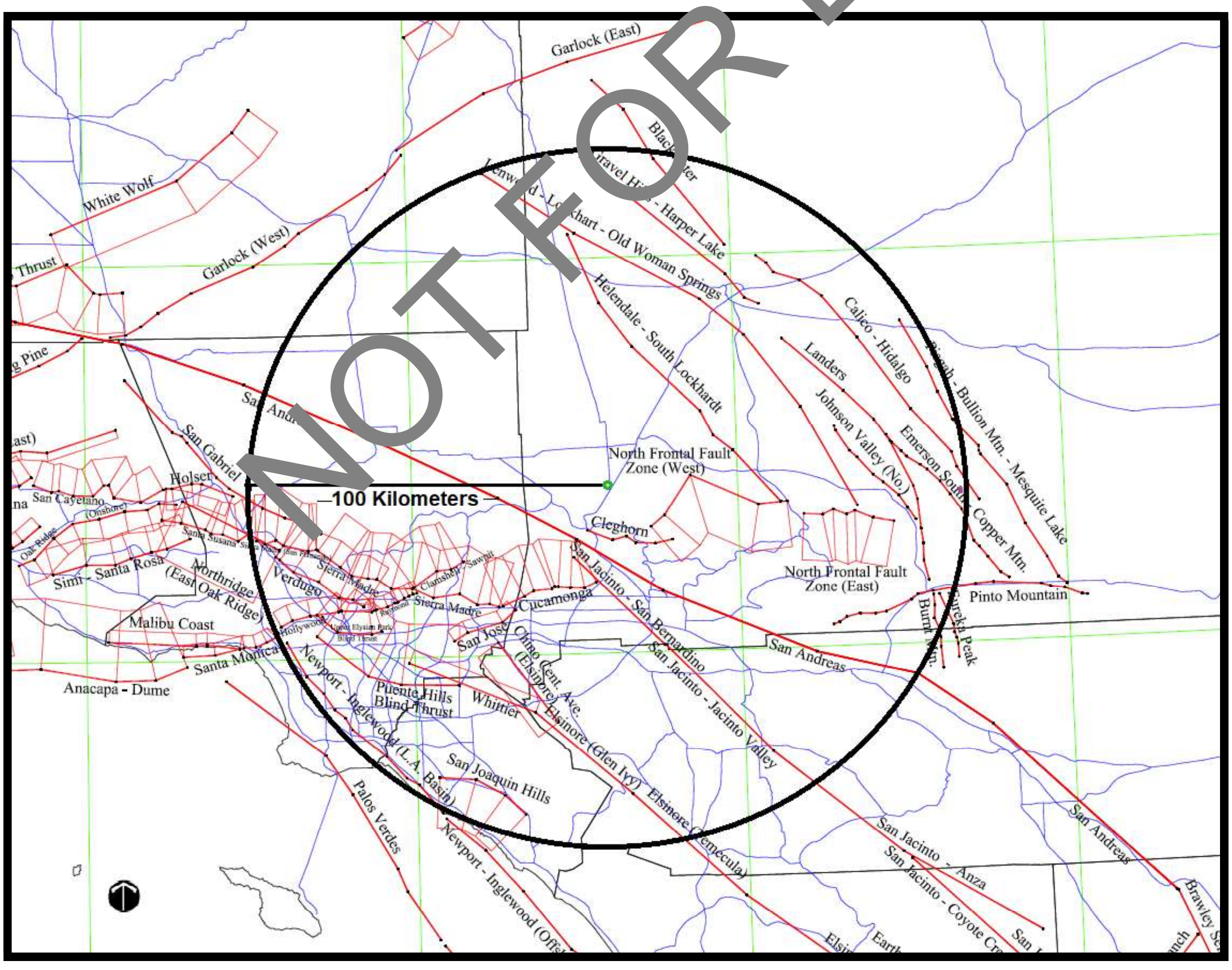


FIGURE 2- Regional Fault Map showing 100 km radius (from CGS 2002 California Fault Model).

Of these, there are no active faults known to traverse the site based on published literature our field reconnaissance. In addition, the subject site is not located within a State of California "Alquist-Priolo Earthquake Fault Zone" for surface fault rupture hazards (California Geological Survey, 2018).

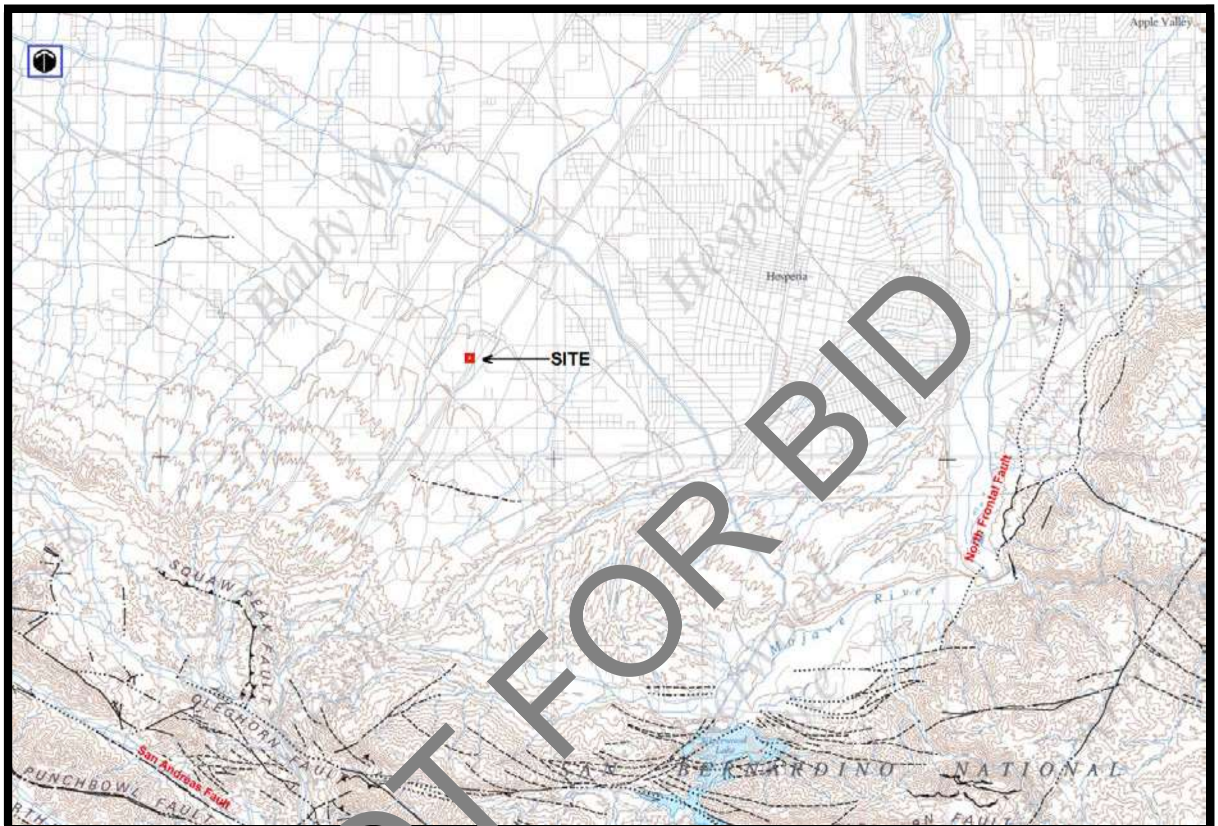


FIGURE 3- Major Fault Map (from Morton and Miller, 2006, Sheet 2 of 4).

The nearest mapped zoned active fault is associated with the San Andreas Fault Zone (see Figure 3 above for reference), located approximately 9.2± miles to the southwest (C.D.M.G., 1974), which is locally referred to as the San Bernardino North Fault segment. 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). There are at least ten segments that comprise the entire length of the San Andreas Fault Zone. When considering a cascading rupture, 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 (Petersen et al., 2008).

Another nearby active fault is located approximately 9.8± miles to the southeast (C.D.M.G., 1988) which is locally referred to as the Ord Mountain Fault (western segment of the North Frontal Fault system), which is a southern dipping reverse fault, being approximately 50.1 kilometers in length, with an estimated maximum moment magnitude of M_W 7.2.

GROUNDWATER

The study area lies within the Upper Mojave River Groundwater Basin of southern California. The Mojave River Basin is part of the Mojave Desert region and is bordered by the San Bernardino and San Gabriel Mountains to the south and extends to Afton Canyon to the northeast, with Lucerne Valley and Antelope Valleys bordering the east and west, respectively. The Mojave River, which is located to the east, is the principal source of water recharge to the basin, which originates from the junctions of Deep Creek and West Fork Mojave River at the northern foot of the San Bernardino Mountains. Other sources of recharge include other lesser river tributaries from the San Bernardino and San Gabriel Mountains, as well as deep percolation from rainwater and other artificial means.

The water-bearing deposits are principally unconsolidated and partially consolidated continental sedimentary deposits that form two aquifers (Stamos and Predmore, 1995), the upper one being shallow alluvium (200± feet thick, within 1± mile of the Mojave River), with the regional aquifer underlying most of the basin at depth. The regional aquifer is comprised of unconsolidated older alluvium and fan deposits of Pleistocene to Tertiary age, and partly consolidated to consolidated sediments of Tertiary age. These deposits are as much as 1,000 feet thick in some places and their permeability generally decreases with depth.

Based on groundwater data provided by the California Department of Water Resources (2022b), the closest measured well is located approximately 1¼ miles to the north (State Well No. 04N05W21H001S), which has been measured from 1995 to the present. The groundwater level was fairly uniform throughout this period, varying between 647 to 658 feet in depth. Several regional groundwater reports were reviewed to help evaluate the historic and recent local groundwater levels and characteristics, which included the following; Lines (1996), Mendez and Christensen (1997), Smith (2000 and 2004), and Stamos and Predmore (1995). These reports are listed in Appendix C for reference purposes. The U.S.G.S. well database was also searched which provided groundwater level data for numerous nearby wells (U.S.G.S., 2022c). Based on a review of this data, groundwater is shown to be at a depth of greater than 800± feet in the general site vicinity. Subsurface exploration performed by IFE (2022), did not encounter groundwater to a depth of at least 51½ feet.

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 detailed results of which are presented within Appendix B. Additionally, a seismic shear-wave survey was conducted for this study by our firm as presented within Appendix A of this report, for purposes of determining the Site Classification and V_{S30} input values for the ground motion analysis.

Geographically, the proposed construction area is located at Longitude -117.4022 and Latitude 34.4016 (World Geodetic System of 1984 coordinates). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the OSHPD Seismic Design Maps (OSHPD, 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	**1.500g**
S_1	**0.600**
F_a	**1.0**
F_v	**1.7**
S_{DS}	**0.950g**
S_{D1}	**0.800g**
S_{MS}	**1.419g**
S_{M1}	**1.200g**
T_L	**12 Seconds**
MCE_G PGA	**0.59g**
Shear-Wave Velocity (V100)	**1,147.7 ft/sec**
Site Classification	**D**
Risk Category	**III**

HISTORIC SEISMIC ACTIVITY

A computerized search, based on Southern California historical earthquake catalogs, has been performed using the programs EQSEARCH (Blake, 1989-2021) and the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2022a). The following table and discussion summarize the known historic seismic events (≥M4.0) that have been estimated and/or recorded during this time period of 1800 to November 2022 within a 100-kilometer (62-mile) radius of the site.

TABLE 2 - HISTORIC SEISMIC EVENTS; 1800-2022 (100 Kilometer Radius)

Richter Magnitude	No. of Events
4.0 - 4.9	597
5.0 - 5.9	66
6.0 - 6.9	13
7.0 - 7.9	1
8.0+	0

A summary of the historic earthquake data is as follows:

- At least 80 significant historical earthquakes of magnitude 5.0 and greater, and at least 597 notable earthquakes of magnitude 4.0 to 4.9, have been estimated and/or recorded during the period of 1800 to November 2022, within a 100-kilometer (62 mile) radius of the subject site.

- The closest recorded notable earthquake epicenter (magnitude 4.0 or greater) is a M4.6 event (March 1, 1948), located 10± miles to the south-southwest.

- The largest estimated historical earthquake magnitude within a 62-mile radius of the site is a M6.9 event of December 8, 1812 (approximately 14 miles west-southwest).

- The largest recorded historical earthquake was the M7.6 (M_W7.3) Landers's event, located approximately 57 miles to the east-southeast (June 28, 1992).

- The nearest estimated significant historic earthquake epicenter was approximately 9 miles southwest of the site (July 22, 1899, M6.5).

- The nearest recorded significant historic earthquake epicenter was approximately 12½ miles southwest of the site (September 12, 1970, M5.4).

- The largest ground acceleration estimated to have been experienced at the site was 0.260g which resulted from the M6.5 event of July 22, 1899, which was located approximately 19miles to the southwest (Blake, 1989-2000), based on the attenuation relationship of Boore et al. (1997).

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.

An Earthquake Epicenter Map which includes magnitudes 4.0 and greater for a 100-kilometer (62-mile) radius (blue circle) has been included below as Figure 4. 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 November 2022, in turn overlain on Google™ Earth imagery (2022). The subject site is the blue dot located at center of circle.

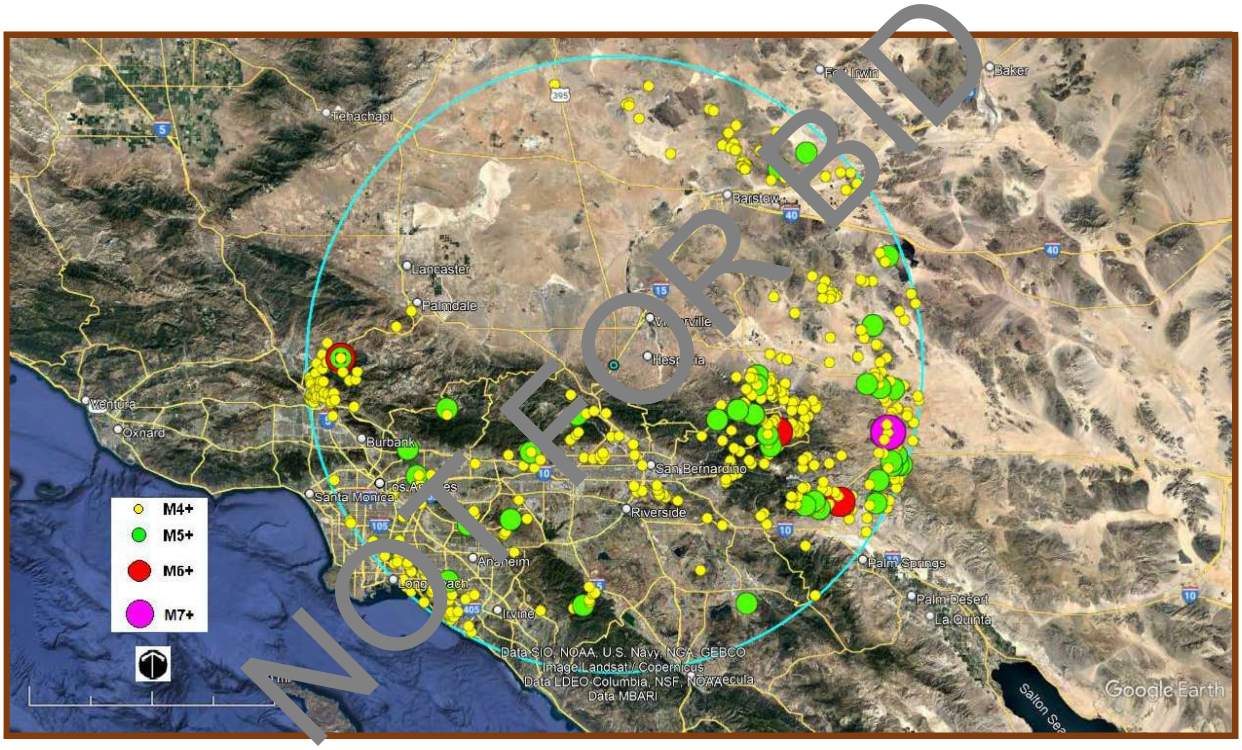


FIGURE 4- Earthquake Epicenter Map showing events of M4.0+ within a 100-kilometer radius.

FLOODING

According to the Federal Emergency Management Agency (2008), the site is not shown to be located within the boundaries of a designated flood hazard area (FEMA, 2008). This map indicates that the site is located within "Zone X," which is defined as "Areas of Minimal Flood Hazard." Additionally, the "Very High Fire Hazards Area, Flood Zones, and Significant Hazardous Material Sites" map (City of Hesperia, 2010, Exhibit SF-2) does not indicate the site to be located within a designated flood hazard area. During peak periods of rainfall heavy runoff could be anticipated and should be properly evaluated by the project Civil Engineer.

SECONDARY SEISMIC HAZARDS

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

Ground Rupture:

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

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 related hazards. The main factors 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. The City of Hesperia (2010) does not indicate the site to be located within a zone of potential liquefaction (“Map Showing the Seismic Hazards”, Exhibit SF-1). Additionally, due to the absence of shallow groundwater at the site and the estimated depth of greater than 800 feet, the potential for liquefaction to occur at the site appears to be nil.

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.

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. Additionally, the San Bernardino County Hazards Overlay Map (San Bernardino County, 2010) does not indicate the site to be located within a dam inundation hazard area.

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 subject property and distance from embankments, the potential for ground lurching and/or lateral spreading to occur is considered nil.

Rockfalls:

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

Seismically-Induced Settlement:

Seismically-induced settlement generally occurs within areas of loose granular soils. Based on the data provided within the boring logs (IFE, 2022), the proposed construction area appears to be underlain by generally medium/stiff to dense/hard sediments, therefore the potential for settlement appears to be low.

Landsliding:

Due to the low-lying relief of the site and vicinity, landsliding due to seismic shaking is considered nil. Additionally, the City of Hesperia (2010) does not indicate the site to be located within a zone of earthquake-induced landsliding ("Map Showing the Seismic Hazards", Exhibit SF-1).

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 (including the site-specific boring log data), construction of the proposed new metal building appears to be feasible from a geologic standpoint, providing that our recommendations are considered during planning and construction. No unusual geologic hazards or conditions were observed during our field reconnaissance or literature research.

CONCLUSIONS:

1. **Earth Materials**

   Based on our review of available published data, the subject site is mapped as being mantled by middle to early-middle Pleistocene age old fan deposits, generally described as being comprised of moderately- to well-consolidated silt, sand, and gravel. More specifically, the provided borings logs indicate the underlying earth materials to consist of predominantly interbedded clayey sand, sandy clay, silty clayey sand, fine- to coarse-grained silty sand, fine- to medium-grained silty sand, silty sand with gravel, and fine- to coarse-grained sandy gravel, to a depth of at least 51½ feet. These sediments were noted to be in a medium/stiff to dense/hard condition. These relatively surficial deposits have been derived as wash deposits from the San Bernardino and San Gabriel Mountains to the south and appear to be consistent with regional geologic mapping.

2. ***Faulting***

No active faults are known to traverse the site, based on published literature, and no surficial indications or geomorphic features were observed that are suggestive of faulting. In addition, the site is not located within a designated Alquist-Priolo Earthquake Fault Zone for fault rupture hazards. The nearest mapped (zoned) "active" fault is the San Andreas Fault Zone (San Bernardino North segment), located approximately 9.2± miles to the southwest.

3. ***Seismicity***

The <u>primary</u> geologic hazard that exists at the site is that of ground shaking. Ground shaking from earthquakes accounts for nearly all earthquake losses. Many factors determine the severity of ground shaking at a given location, such as size of earthquake, length of fault rupture (if any), depth of hypocenter, type of faulting (dip slip/strike slip), directional attenuation, amplification, earth materials, and others. Due to the location of the site with respect to regional faulting and the recorded historical seismic activity in the region, moderate to severe ground shaking could be anticipated during the life of the proposed facilities.

4 ***Flooding***

According to the Federal Emergency Management Agency and the City of Hesperia, the proposed development is not located within the boundaries of a designated flood zone.

5. ***Groundwater***

Available published data in the local vicinity indicates that the depth to groundwater historically is greater than 800± feet in depth within the vicinity of the proposed development. Shallow groundwater is therefore not anticipated to be encountered during grading.

6. ***Secondary Seismic Hazards***

Based on the data obtained during this study as previously discussed, there do not appear to be any permanent or transient secondary seismic hazards that are expected to occur at the subject site.

RECOMMENDATIONS:

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.

CLOSURE

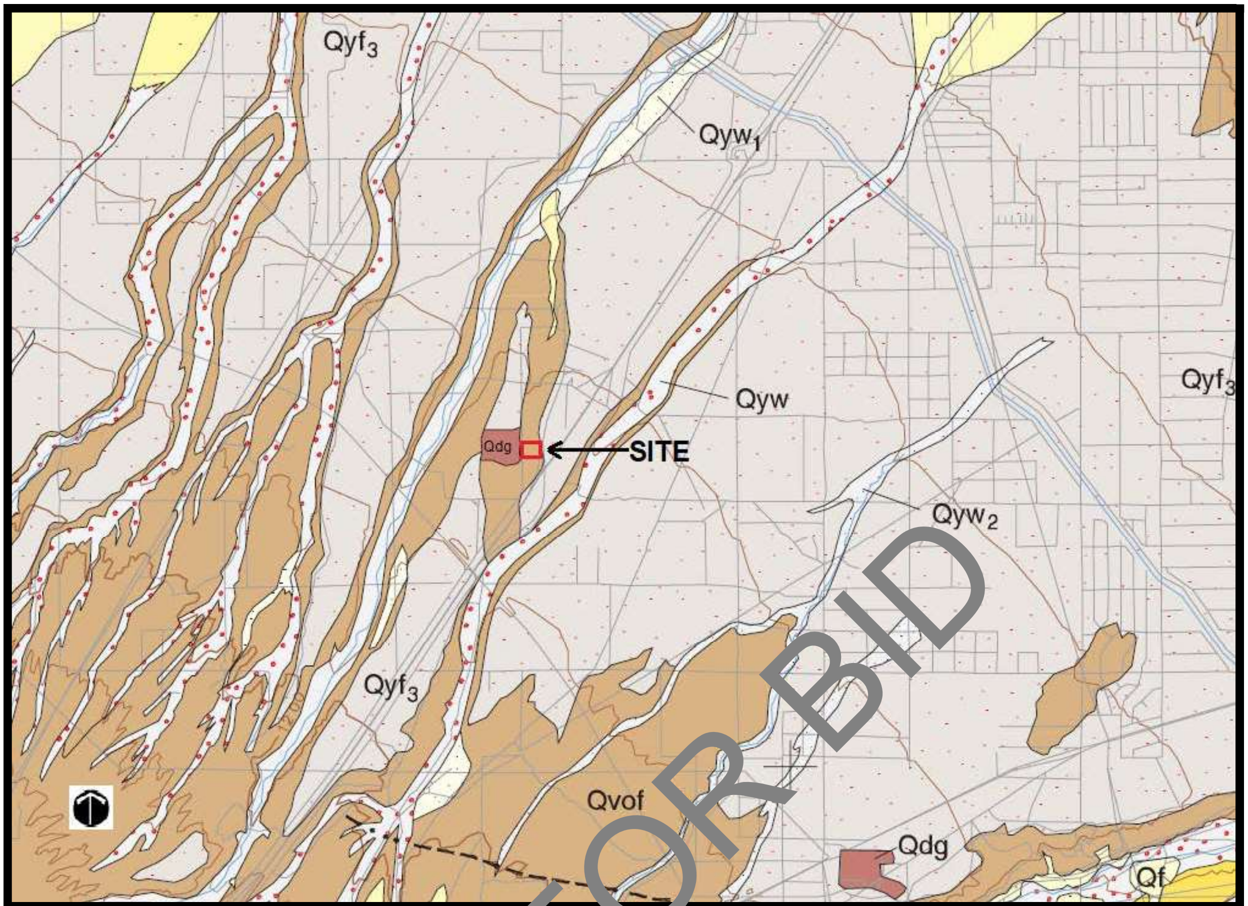
Our conclusions and recommendations are based on a field reconnaissance, review of subsurface exploratory boring excavations, and an interpretation of available existing geotechnical and geologic/seismic data. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted.

It is assumed that all the conclusions and recommendations outlined in this report are understood and followed. If any portion of this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

Respectfully submitted,
TERRA GEOSCIENCES

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

REGIONAL GEOLOGIC MAP

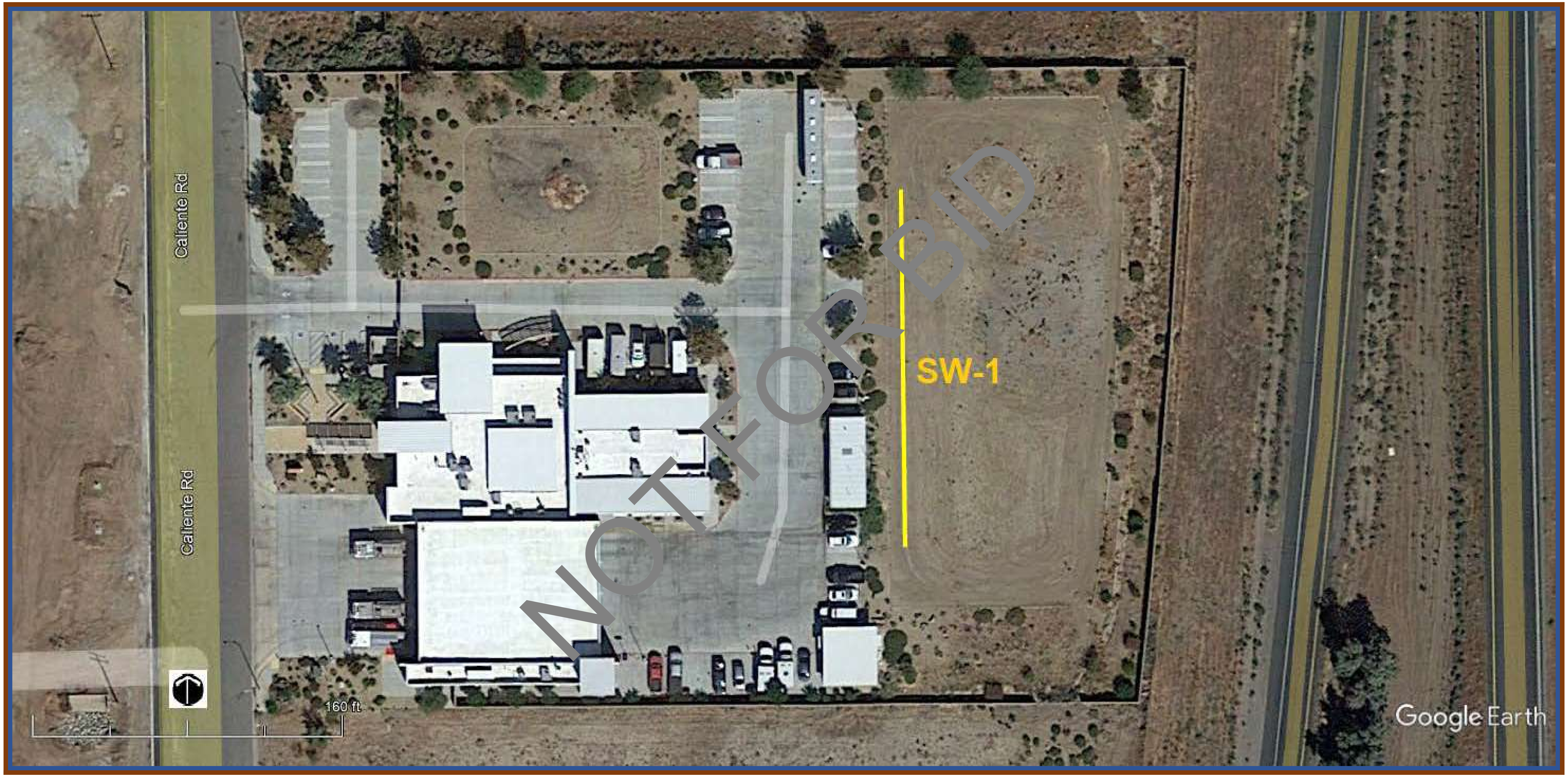


BASE MAP: Morton and Miller (2006), Scale 1: 100,000; Site outlined in red.

PARTIAL LEGEND

Qdg	**DISTURBED GROUND**	Areas where human activity obscures accurate identification or classification of natural geologic units (late Holocene).
Qyf_3	**YOUNG FAN DEPOSITS**	Slightly- to moderately-consolidated silt, sand, and coarse-grained sand to bouldery alluvium (middle Holocene).
Qvof	**VERY OLD FAN DEPOSITS**	Moderately to well consolidated silt, sand, and gravel (middle to early middle Pleistocene).
	GEOLOGIC CONTACT	Solid where well-located to approximately-located, dashed where inferred.
	FAULT	Solid where accurately located, dashed where approximate, dotted where concealed.

GOOGLE™ EARTH IMAGERY MAP



Base Map: Google™ Earth (2022); Seismic shear-wave traverse SW-1 shown as yellow 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 along the western portion of the subject development area, as approximated on 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 184-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals. For the active source MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two separate seismic records were obtained using a 30-foot shot offset at both ends of the line utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Numerous seismic impacts were used at each shot location to improve the signal-to-noise ratio.

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

Data Reduction

For analysis and presentation of the shear-wave profile and supportive illustration, this study used the **SeisImager/SW™** computer software program that was developed by Geometrics, Inc. (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 **1,147.7** feet per second as shown on the shear-wave model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of Site Class "**D**" (Stiff Soil), which has a velocity range from 600 to 1,200 ft/sec (ASCE, 2017; Table 20.3-1).

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

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

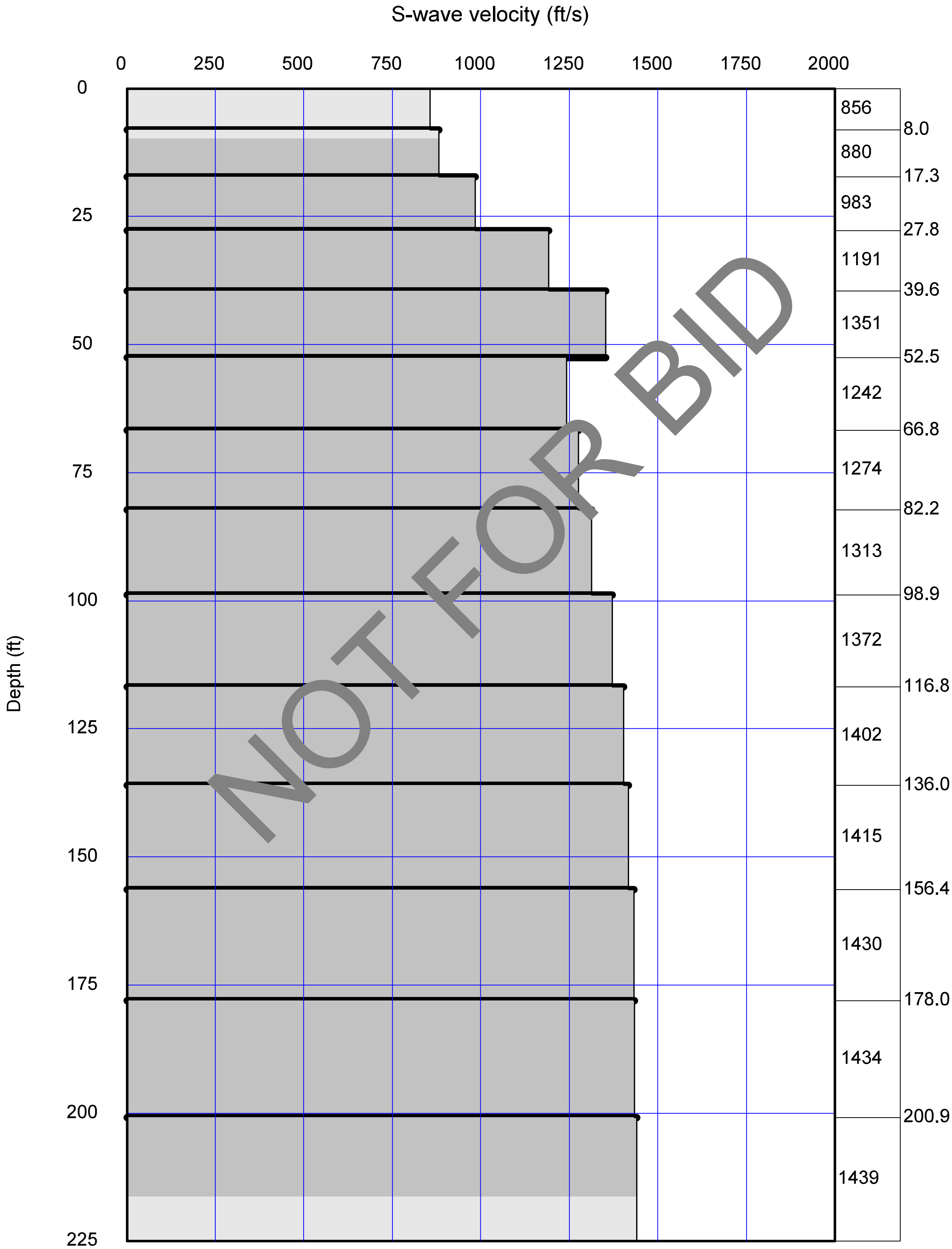
Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 218-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 north along Seismic Line SW-1.

View looking south along Seismic Line SW-1.

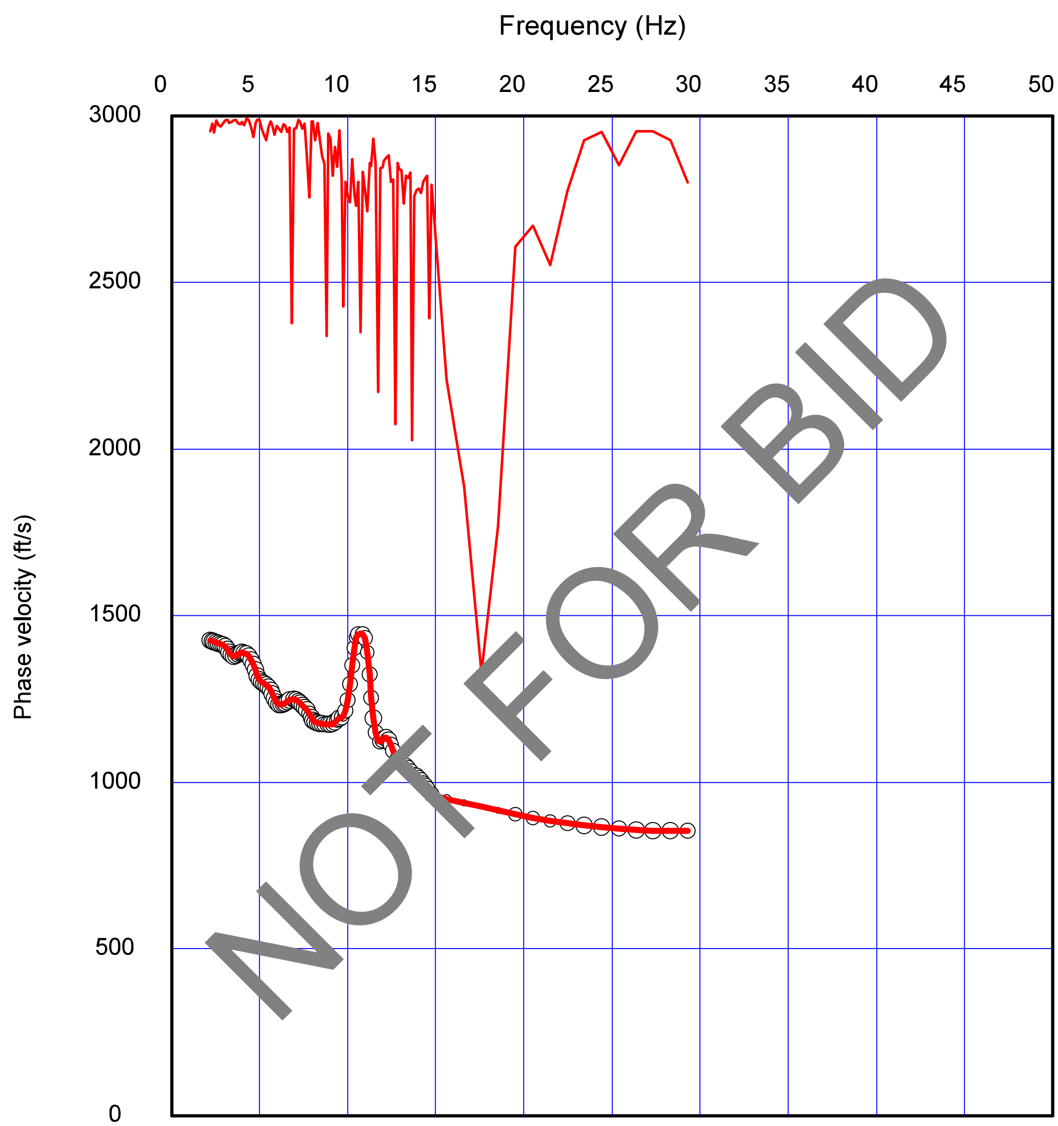
SEISMIC LINE SW-1
SHEAR-WAVE MODEL



S-wave velocity model (inverted) : Final.rst

Average Vs 100ft = 1147.7 ft/sec

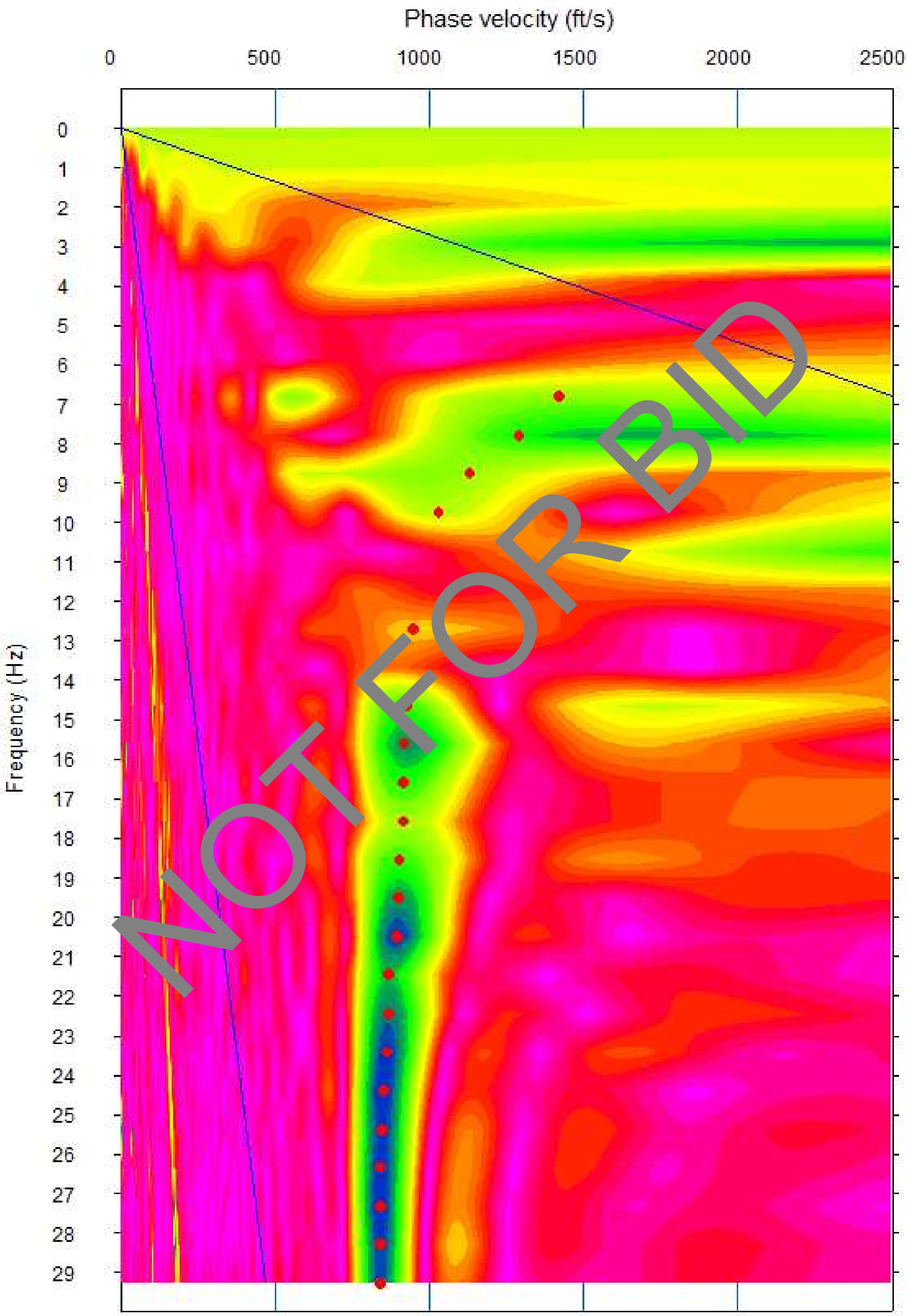
SEISMIC LINE SW-1



Dispersion curve : Combined.rst

COMBINED DISPERSION CURVE

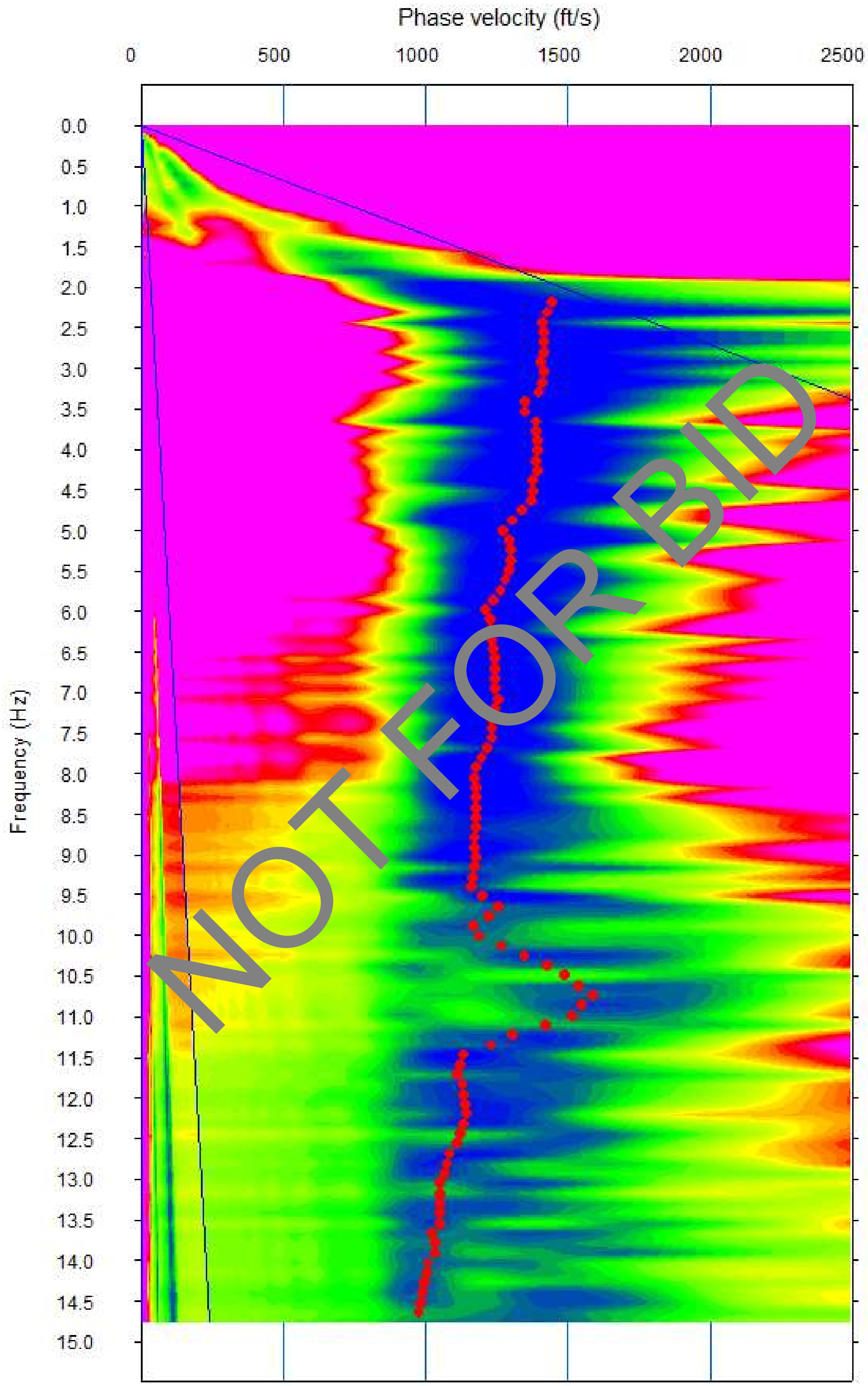
SEISMIC LINE SW-1



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 **1.500g** for the 0.2 second period (S_s) and **0.600** for the 1.0 second period (S_1) was calculated (ASCE 7-16 Figures 22-1, 22-2 and CBC 1613A.2.1).

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

  Based on the site-specific measured shear-wave value of 1,147.7 feet/second (349.8 m/sec), the soil profile type used should be Site Class **"D."** This Class is defined as having the upper 100 feet (30 meters) of the subsurface being underlain by "Stiff Soil" with average shear-wave velocities of 600 to 1,200 feet/second (180 to 360 meters/second), as detailed within Appendix A.

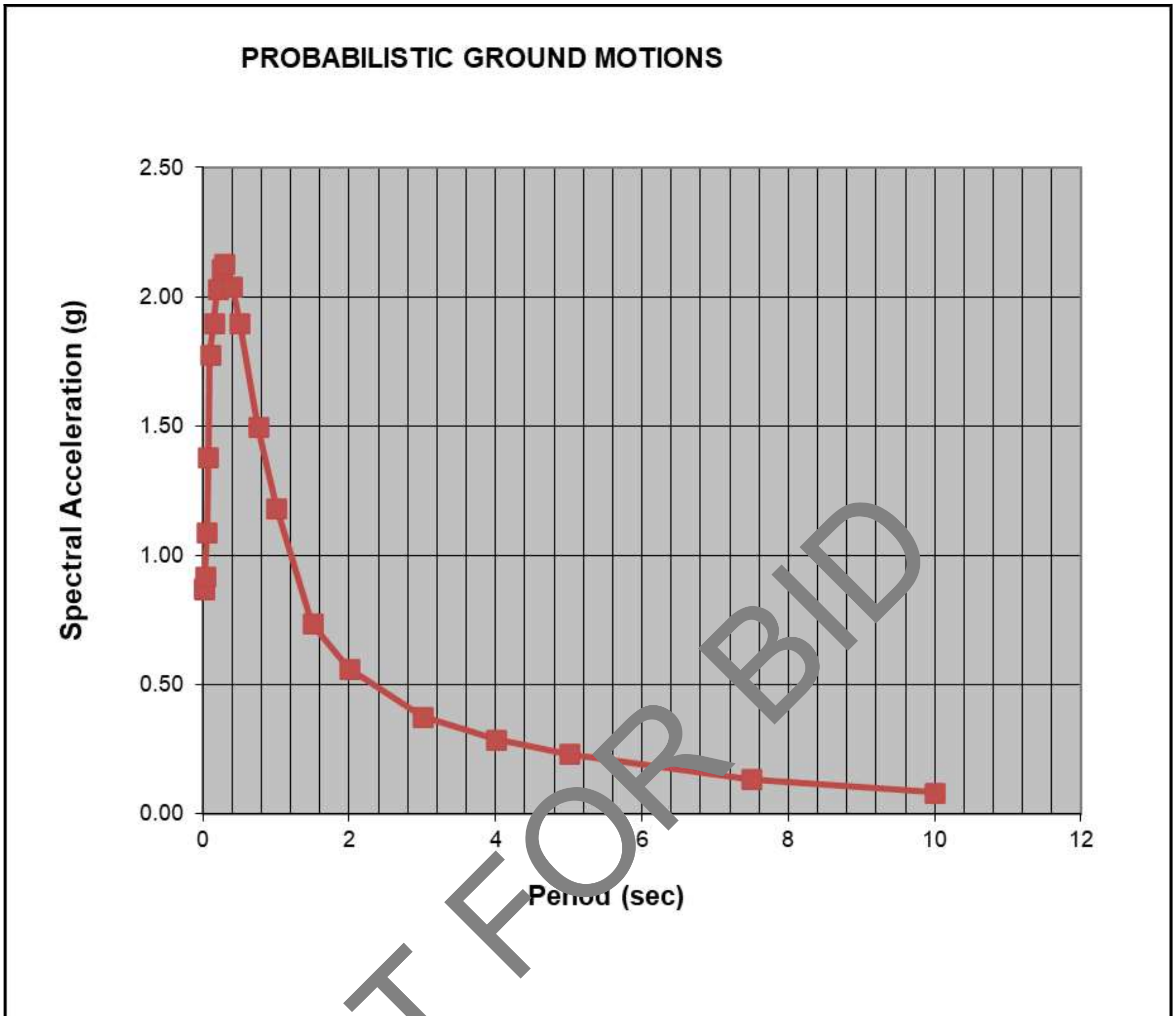
- **Site Coefficients (CBC 1613A.2.3)**-

  Based on CBC Tables 1613A.2.3(1) and 1613A.2.3(2), the site coefficient $\mathbf{F_a = 1.0}$ and $\mathbf{F_v = 1.7}$, respectively.

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

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

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



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

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

  Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013) and published geologic data, the nearest and most significant faults were used for this analysis. These faults included the San Andreas Fault (M_W 8.1), the North Frontal Fault (M_W 7.2), the San Jacinto (M_W 6.8), and the Cleghorn Fault (M_W 6.7), as listed on Page 4 of 6 in the following "Seismic Design Parameter Summary" table.

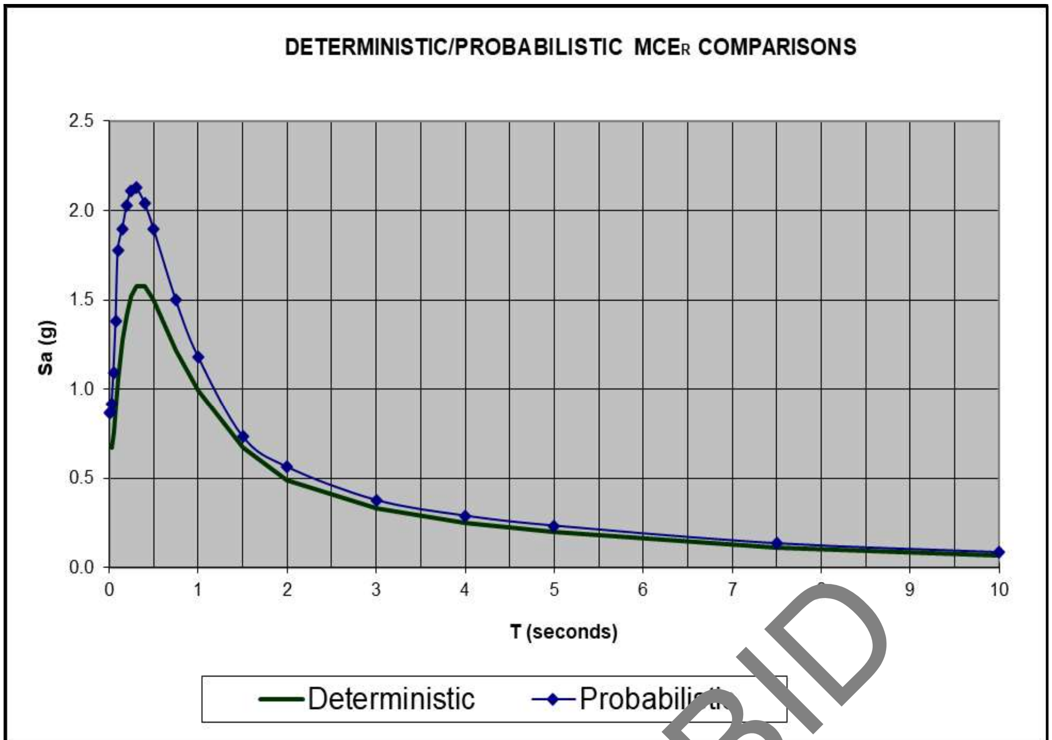
♦ **Site Specific MCE_R (ASCE 7 Section 21.2.3)**-

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

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

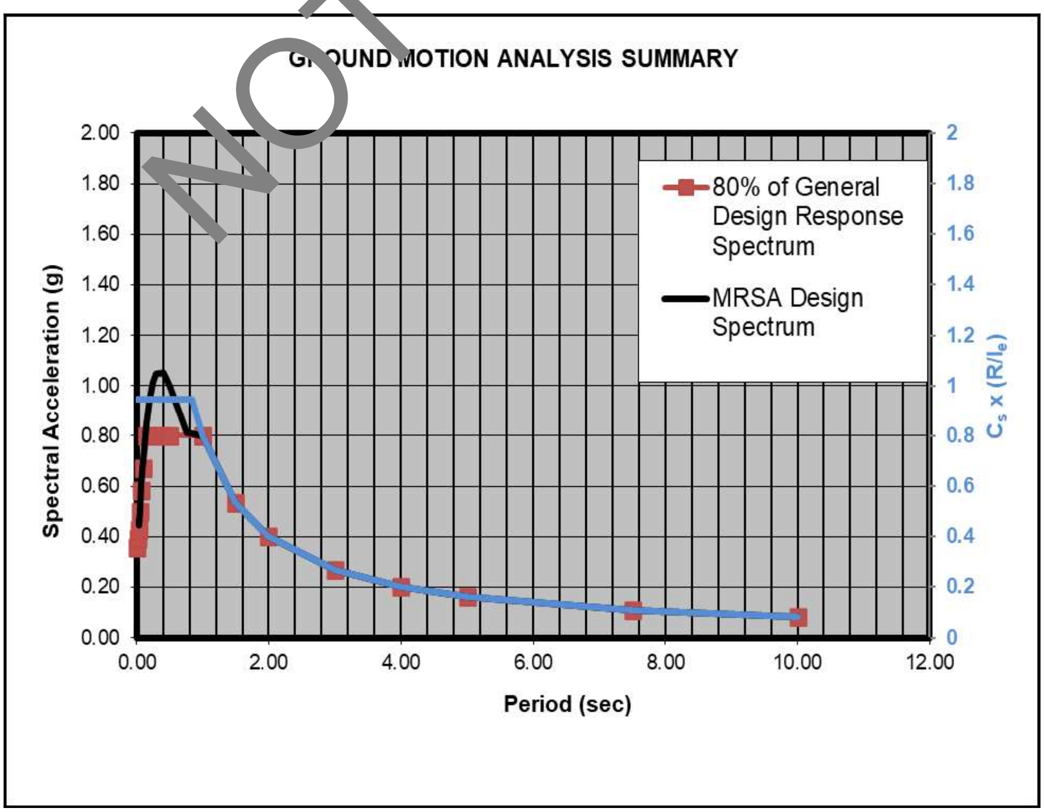
Period	Deterministic	Probabilistic	Lower Value	Governing Method
T	MCE_R	MCE_R	(Site Specific MCE_R)	
0.010	0.69	0.87	0.69	Deterministic Governs
0.020	0.68	0.87	0.68	Deterministic Governs
0.030	0.67	0.92	0.67	Deterministic Governs
0.050	0.76	1.09	0.76	Deterministic Governs
0.075	0.92	1.38	0.92	Deterministic Governs
0.100	1.06	1.78	1.06	Deterministic Governs
0.150	1.27	1.90	1.27	Deterministic Governs
0.200	1.42	2.03	1.42	Deterministic Governs
0.250	1.52	2.11	1.52	Deterministic Governs
0.300	1.57	2.13	1.57	Deterministic Governs
0.400	1.58	2.04	1.58	Deterministic Governs
0.500	1.50	1.90	1.50	Deterministic Governs
0.750	1.22	1.50	1.22	Deterministic Governs
1.000	0.99	1.18	0.99	Deterministic Governs
1.500	0.67	0.73	0.67	Deterministic Governs
2.000	0.49	0.56	0.49	Deterministic Governs
3.000	0.33	0.38	0.33	Deterministic Governs
4.000	0.25	0.29	0.25	Deterministic Governs
5.000	0.20	0.23	0.20	Deterministic Governs
7.500	0.11	0.13	0.11	Deterministic Governs
10.000	0.07	0.08	0.07	Deterministic Governs

These comparisons 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}), a value of 0.95g was computed, based upon the average spectral accelerations. The maximum average acceleration for any period exceeding 0.2 seconds was 1.05g occurring at T=0.30 and 0.40 seconds. This was multiplied by 0.9 to produce a value of 0.95g making this the applicable value. A value of 0.80g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4). For the MCE_R 0.2 second period, a value of 1.4[illegible]9g (S_{MS}) was computed, along with a value of 1.200g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

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

  The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 0.92g. The deterministic geometric mean peak ground acceleration (largest 84th percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.58g. The site-specific MCE_G peak ground acceleration was calculated to be **0.58g**, which was determined by using the lesser of the probabilistic (0.92) or the deterministic (0.58g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 0.60g x 0.80 = 0.48g).

SEISMIC DESIGN PARAMETERS SUMMARY

Project:	Hesperia Fire Station 305	Lattitude:	34.4016
Project #:	223896-1	Longitude:	-117.4022
Date:	11/30/22		

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	1.5	Figure 22-1
S_1=	0.6	Figure 22-2

Site Class per Table 20.3-1

Site Class=	D - Stiff Soil

Site Coefficients per ASCE 7-16 CHAPTER 11

F_a=	1	Table 11.4-1	=	1	For Site Specific Analysis per ASCE7-16 21.3
F_v=	1.7	Table 11.4-2	=	2.50	For Site Specific Analysis per ASCE7-16 21.3

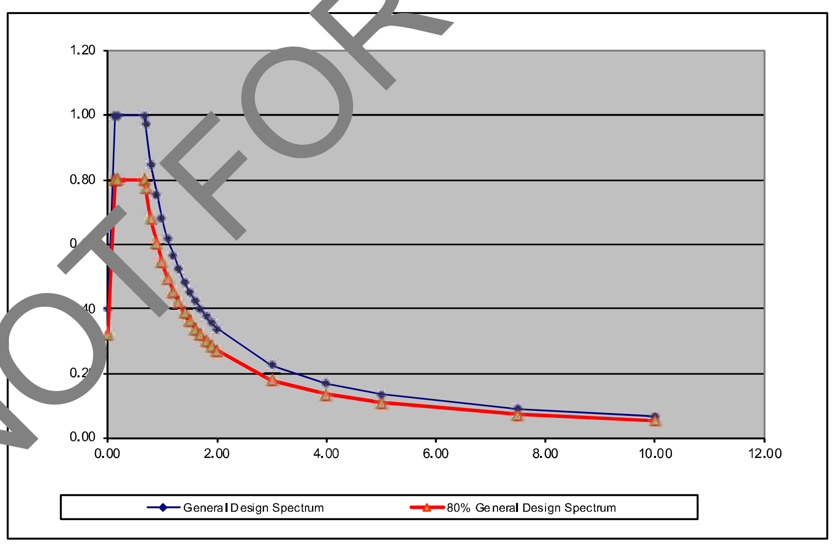
Mapped Design Spectral Response Acceleration Parameters

S_{MS}=	1.5	Equation 11.4-1	1.5	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	1.020	Equation 11.4-2	1.500	For Site Specific Analysis per ASCE7-16 21.3

S_{DS}=	1.000	Equation 11.4-3
S_{D1}=	0.680	Equation 11.4-4

T_0=	0.136	sec	
T_S=	0.680	sec	
T_L=	12	sec	From Fig 22-12
PGA	0.541	g	
F_{PGA}=	1		From Table 11.8-1
C_{RS}=	0.9[illegible]		Figure 22-17
C_{R1}=	0.906		Figure 22-18

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.40	0.32
0.14	1.00	0.80
0.20	1.00	0.80
0.68	1.00	0.80
0.70	0.97	0.78
0.80	0.85	0.68
0.90	0.76	0.60
1.00	0.68	0.54
1.10	0.62	0.49
1.20	0.57	0.45
1.30	0.52	0.42
1.40	0.49	0.39
1.50	0.45	0.36
1.60	0.43	0.34
1.70	0.40	0.32
1.80	0.38	0.30
1.90	0.36	0.29
2.00	0.34	0.27
3.00	0.23	0.18
4.00	0.17	0.14
5.00	0.14	0.11
7.50	0.09	0.07
10.00	0.07	0.05



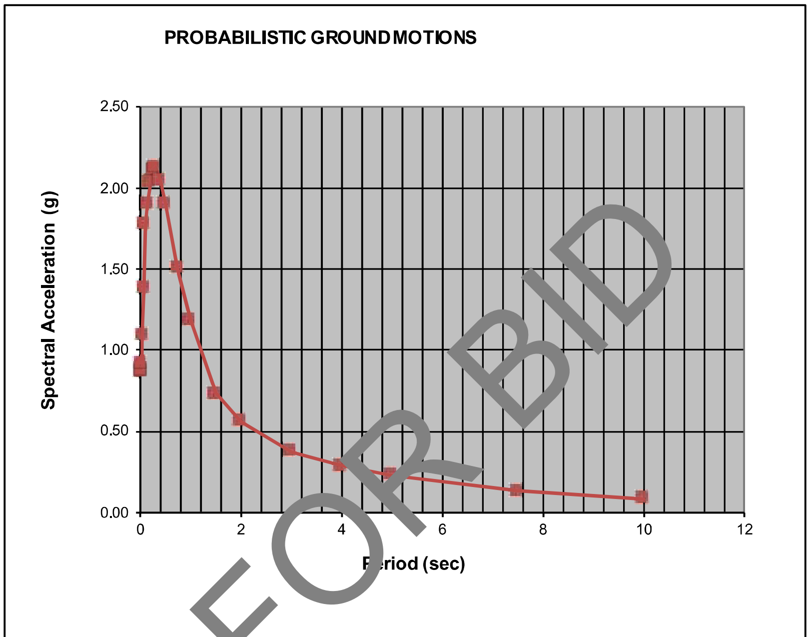
ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

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
Earthquake Rupture Forecast - UCERF3 FM 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
Al Atik, L., and Youngs, R. R., 2013. Epistemic Uncertainty for NGA-West2 Models,PEER Report No. 2013/11, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA, 59 pp.

T	Sa 2% in 50	MCER
0.01	0.93	0.87
0.02	0.94	0.87
0.03	0.99	0.92
0.05	1.17	1.09
0.08	1.49	1.38
0.10	1.74	1.78
0.15	2.04	1.90
0.20	2.19	2.03
0.25	2.28	2.11
0.30	2.30	2.13
0.40	2.21	2.04
0.50	2.06	1.90
0.75	1.64	1.50
1.00	1.30	1.18
1.50	0.81	0.73
2.00	0.62	0.56
3.00	0.42	0.38
4.00	0.32	0.29
5.00	0.26	0.23
7.50	0.15	0.13
10.00	0.09	0.08



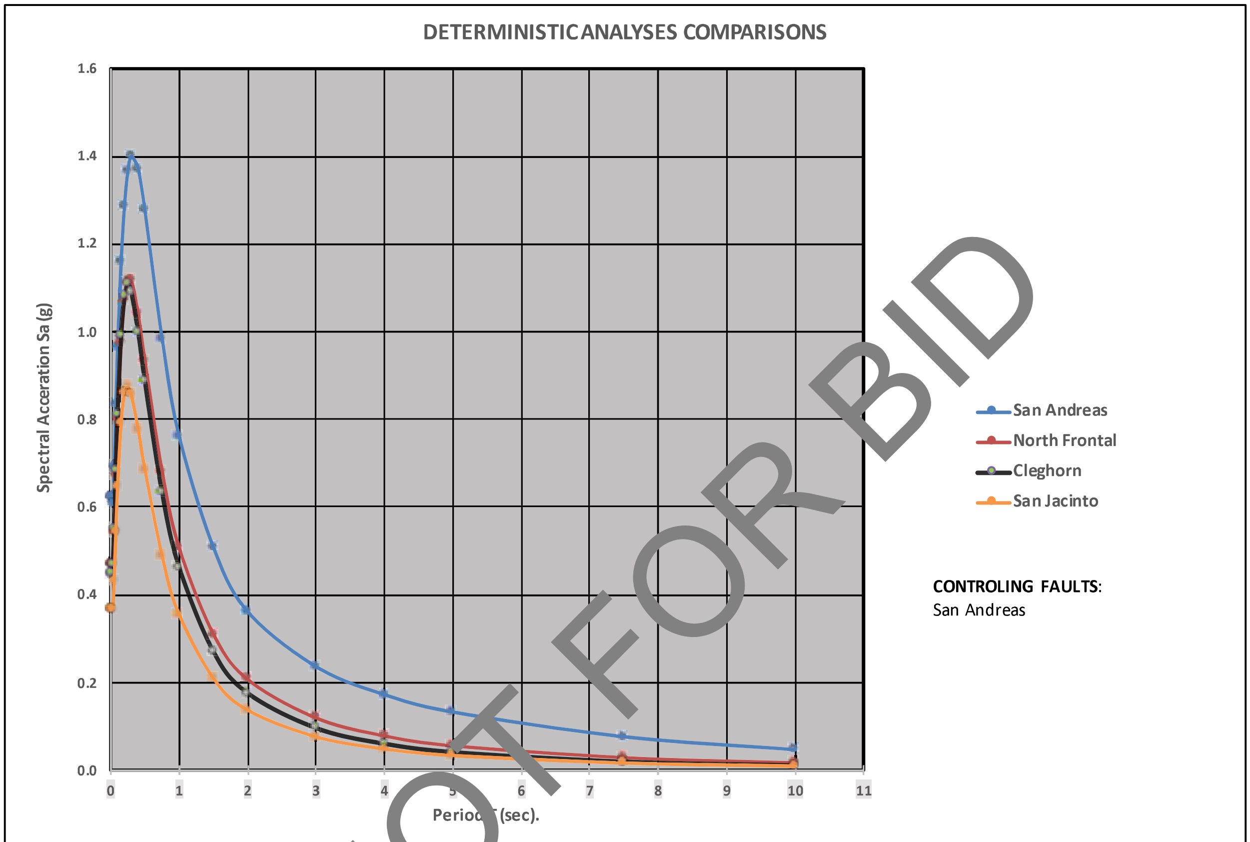
S_s=	2.19	2.03
S_1=	1.30	1.18
PGA	0.92 g	

Risk Coefficients:			
C_{RS}	0.928	Figure 22-18	Get from Mapped Values
C_{R1}	0.906	Figure 22-19	
Fa=	1	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is $Sa_{(max)}$<1.2XFa?		NO	If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Preliminary Assessment: Four faults were considered on the basis of their relative proximities to the site. The San Andreas Fault clearly is dominant.

Fault	Distance (km)
San Andreas	14.90
North Frontal	15.70
San Jacinto	17.90
Cleghorn	12.10



Input Parameters					
Fault		San Andreas	North Frontal	San Jacinto	Cleghorn
M	= Moment magnitude	8.1	7.2	6.8	6.7
R_{RUP}	= Closest distance to coseismic rupture (km)	14.9	15.7	17.9	12.1
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	14.9	15.7	17.9	12.1
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	14.9	15.7	17.9	12.1
U	= Unspecified Faulting Flag (Boore et.al.)	0	0	0	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0	1	0	0
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	0	0	0	0
F_{HW}	= Hanging-wall factor: 1 for site on down-dip side of top of rupture; 0 otherwise, used in AS08 and CY08	0	0	0	0
Z_{TOR}	= Depth to top of coseismic rupture (km)	0	0	0	0
δ	= Average dip of rupture plane (degrees)	90	49	80	90
V_{S30}	= Average shear-wave velocity in top 30m of site profile	349.8	349.8	349.8	349.8
$F_{Measured}$		1	1	1	1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.05	0.05	0.05	0.05
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	1.45	1.45	1.45	1.45
Site Class		D	D	D	D
W (km)	= Fault rupture width (km)	12.5	20.8	16.3	13
F_{AS}	= 0 for mainshock; 1 for aftershock	0	0	0	0
σ	=Standard Deviation	1	[illegible]	1	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

T	San Andreas	North Frontal	San Jacinto	Cleghorn	Maximum S_a (Average)	*Corrected* S_a (per ASCE7-16)	*Scaled* $S_{a(Average)}$	Controlling Fault
0.010	0.62	0.47	0.37	0.45	[illegible]	0.69	0.69	San Andreas
0.020	0.62	0.47	0.37	0.45	[illegible]	0.68	0.68	San Andreas
0.030	0.61	0.47	0.37	0.47	0.61	0.67	0.67	San Andreas
0.050	0.69	0.54	0.43	0.55	[illegible]	0.76	0.76	San Andreas
0.075	0.83	0.67	0.54	0.68	0.83	0.92	0.92	San Andreas
0.100	0.96	0.80	0.65	[illegible]	[illegible]	1.06	1.06	San Andreas
0.150	1.16	0.97	0.79	0.99	1.16	1.27	1.27	San Andreas
0.200	1.29	1.07	0.86	[illegible]	1.29	1.42	1.42	San Andreas
0.250	1.37	1.11	0.87	[illegible]	1.37	1.52	1.52	San Andreas
0.300	1.40	1.12	[illegible]	1.09	1.40	1.57	1.57	San Andreas
0.400	1.37	1.04	0.78	1.00	1.37	1.58	1.58	San Andreas
0.500	1.28	0.93	0.69	0.89	1.28	1.50	1.50	San Andreas
0.750	0.98	0.68	0.49	0.63	0.98	1.22	1.22	San Andreas
1.000	0.76	0.51	[illegible]	0.46	0.76	0.99	0.99	San Andreas
1.500	0.51	0.31	0.21	0.27	0.51	0.67	0.67	San Andreas
2.000	0.36	[illegible]	[illegible]	0.18	0.36	0.49	0.49	San Andreas
3.000	0.24	0.12	0.08	0.10	0.24	0.33	0.33	San Andreas
4.000	0.17	[illegible]	0.05	0.06	0.17	0.25	0.25	San Andreas
5.000	0.13	0.06	0.03	0.04	0.13	0.20	0.20	San Andreas
7.500	0.08	[illegible]	0.02	0.02	0.08	0.11	0.11	San Andreas
10.000	0.05	0.02	0.01	0.01	0.05	0.07	0.07	San Andreas
PGA	0.58	0.45	0.35	0.44	0.58		0.58	g

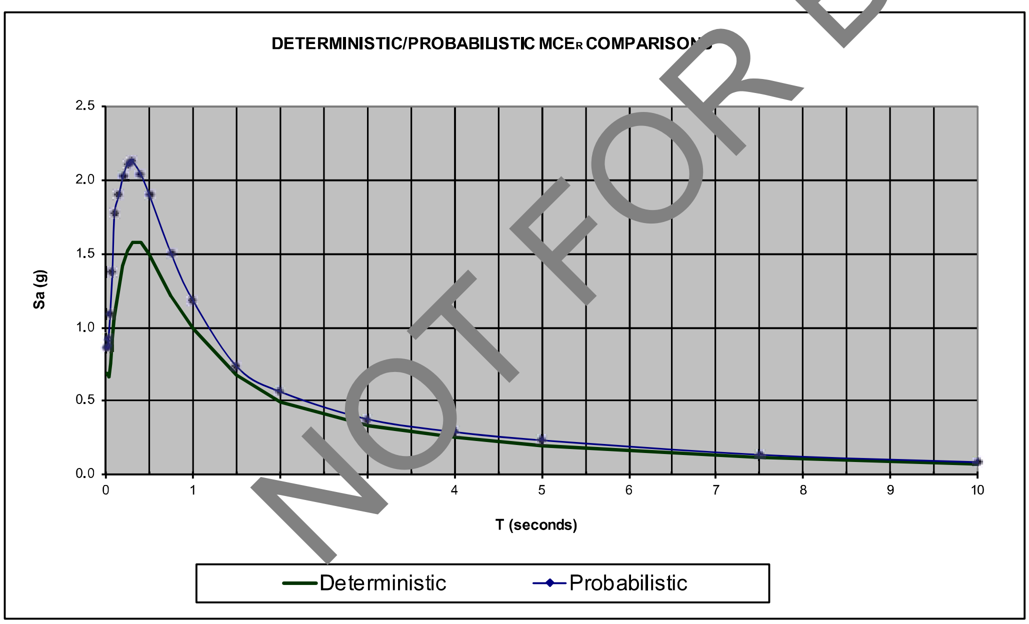
Max Sa=	1.58	
Fa =	1.00	Per ASCE7-16 21.2.2
1.5XFa=	1.5	
Scaling Factor=	1.00	

* 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	0.69	0.87	0.69	Deterministic Governs
0.020	0.68	0.87	0.68	Deterministic Governs
0.030	0.67	0.92	0.67	Deterministic Governs
0.050	0.76	1.09	0.76	Deterministic Governs
0.075	0.92	1.38	0.92	Deterministic Governs
0.100	1.06	1.78	1.06	Deterministic Governs
0.150	1.27	1.90	1.27	Deterministic Governs
0.200	1.42	2.03	1.42	Deterministic Governs
0.250	1.52	2.11	1.52	Deterministic Governs
0.300	1.57	2.13	1.57	Deterministic Governs
0.400	1.58	2.04	1.58	Deterministic Governs
0.500	1.50	1.90	1.50	Deterministic Governs
0.750	1.22	1.50	1.22	Deterministic Governs
1.000	0.99	1.18	0.99	Deterministic Governs
1.500	0.67	0.73	0.67	Deterministic Governs
2.000	0.49	0.56	0.49	Deterministic Governs
3.000	0.33	0.38	0.33	Deterministic Governs
4.000	0.25	0.29	0.25	Deterministic Governs
5.000	0.20	0.23	0.20	Deterministic Governs
7.500	0.11	0.13	0.11	Deterministic Governs
10.000	0.07	0.08	0.07	Deterministic 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 23.3-1)	Design Response Spectrum	TXSa
0.01	0.46	0.36	0.46	
0.02	0.46	0.39	0.46	
0.03	0.45	0.43	0.45	
0.05	0.51	0.50	0.51	
0.08	0.61	0.58	0.61	
0.10	0.71	0.67	0.71	
0.15	0.85	0.80	0.85	
0.20	0.94	0.80	0.94	
0.25	1.01	0.80	1.01	
0.30	1.05	0.80	1.05	
0.40	1.05	0.80	1.05	
0.50	1.00	0.80	1.00	
0.75	0.81	0.80	0.81	
1.00	0.66	0.80	0.80	0.80
1.50	0.45	0.53	0.53	0.80
2.00	0.33	0.40	0.40	0.80
3.00	0.22	0.27	0.27	0.80
4.00	0.17	0.20	0.20	0.80
5.00	0.13	0.16	0.16	0.80
7.50	0.08	0.11	0.11	
10.00	0.05	0.08	0.08	

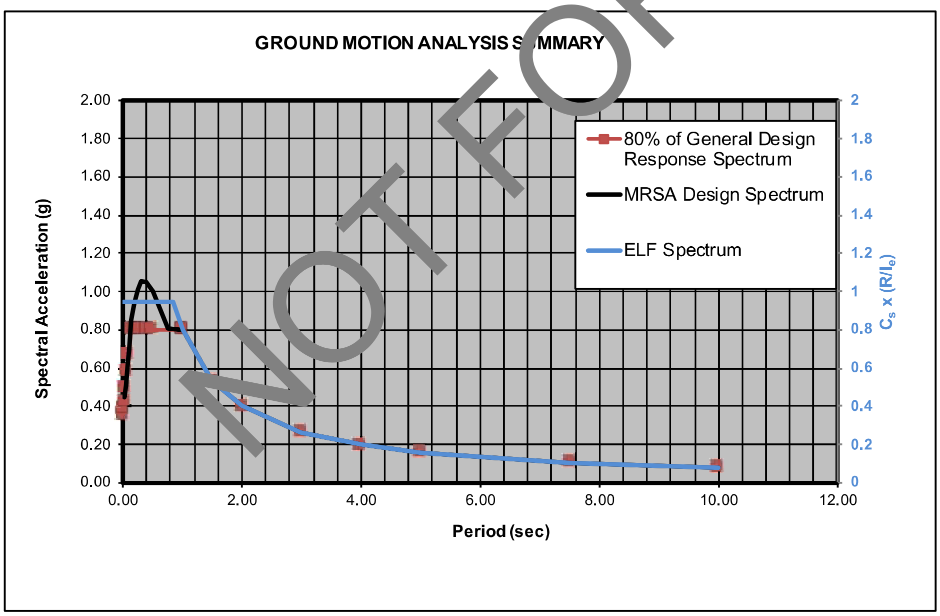
Highest value of S_a for any period exceeding 0.2 sec.=	1.05
90%of Highest Value =	0.95
80% 0f Mapped S_{DS}=	0.80
Maximum TXSa from T=1s-5s =	0.80
80% of Mapped S_{D1}=	0.54

S_{DS}=	0.95
S_{D1}=	0.80
T_s =	0.85

S_{MS}=	1.419
S_{M1}=	1.200

PGA Determination:

Site Coefficie... F_{PGA}	1.1	
Map...ed PGA=	0.54	Figure 22-7
...GA_M =	0.60	g
De...minist... PGA =	0.58	g
Pr...istic ...GA =	0.92	g
Lesser of Dete...stic/Pr...abilistic =	0.58	g
8...% of PGA_{M}=	0.48	g
MCE_G PGA=	0.58	g



APPENDIX C

REFERENCES

REFERENCES

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

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

Blake, T.F. 1989-2000b, EQSEARCH, A computer program for the estimation of peak horizontal acceleration from Southern California Historical Earthquake Catalog, Version 3.00b, database updated June 2021.

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

California Building Standards Commission (CBSC), 2019, 2019 California Building Code, California Code of Regulations, Title 24, Part 2, Volume 2 of 2.

California Department of Water Resources (CDWR), 2022a, DSOD Dam Breach Inundation Maps, https://fmds.water.ca.gov/maps/damim/.

California Department of Water Resources (C.D.W.R.), 2022b, California Statewide Groundwater Elevation Monitoring System (CASGEM) Online System-Public Portal, https://www.casgem.water.ca.gov/OSS/(S(55ycplfqqslsga0raytn2oln))/Public/ApplicationHome.aspx

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

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

California Division of the State Architect (DSA), 2021, Geohazard Reports Requirements: 2019 CBC, Interpretation of Regulations Document IR A-4, 7 pp.

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

California Geological Survey (C.G.S), 2019, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings, Note 48, November 2019, 2 pp.

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

California's Office of Statewide Health Planning and Development (OSHPD), 2022, OSHPD Seismic Design Maps Tool web application, https://seismicmaps.org/.

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

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

City of Hesperia, 2010, City of Hesperia General Plan 2010, Safety Element, 67 pp.

Cox, B.F., and Hillhouse, J.W., 2000, Pliocene and Pleistocene Evolution of the Mojave River, Associated Tectonic Development of the Transverse Ranges and Mojave Desert, Based on Borehole Stratigraphy Studies near Victorville, California, U.S.G.S. Open-File Report 00-147.

Dibblee, Thomas W., Jr., 1967, Areal Geology of the Western Mojave Desert, California, U.S. Geological Survey Professional Paper 522.

Dibblee, Thomas W., Jr., 1980, Geologic Structure of the Mojave Desert, *in* Geology and Mineral Wealth of the California Desert, South Coast Geological Society, Dibblee Volume, pp., 69-100.

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

Google™ Earth, 2022, http://earth.google.com/, Version 7.3.6.9285 (64-bit).

Inland Foundation Engineering, Inc., 2022, Log of Borings B-01 through B-03, IFE Project No. S168-185, dated November 10, 2022.

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

Lines, Gregory, 1996, Ground-Water and Surface-Water Relations along the Mojave River, Southern California, U.S. Geological Survey Water-Resources Investigations Report 95-4189.

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

Mendez, G.O., and Christensen, A.H., 1997, Regional Water Table (1996) and Water-Level Changes in the Mojave River, the Morongo, and the Fort Irwin Ground-Water Basins, San Bernardino County, California, U.S. Geological Survey Water-Resources Investigations Report 97-4160.

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

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

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

Smith, G.A., and Pimentel, M.I., 2000, Regional Table (1998) and Ground-Walter-Level Changes in the Mojave River and the Morongo Ground-Water Basins, San Bernardino County, California, U.S.G.S. Water Resources Investigations Report 00-4090.

Smith, G.A., Stamos, C.L. and Predmore, S.K., 2004, Regional Water Table (2002) and Water-Level Changes in the Mojave River and the Morongo Ground-Water Basins, Southwestern Mojave Desert, California, U.S.G.S. Scientific Investigations Report 2004-5081.

Stamos, C.L., and Predmore, S.K., 1995, Data and Water-Table Map of the Mojave River Ground-Water Basin, San Bernardino County, California, November 1992, U.S.G.S. Water-Resources Investigations Report 95-4148.

Subsurface Surveys, Inc., 1990, Inventory of Ground Water Stored in the Mojave River Basins, 46 pp., 5 Maps.

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

Toppozada, TR. et al., 1981, Preparation of Isoseismal Maps and Summaries of Reported Effects for pre-1900 California Earthquakes, C.D.M.G. OFR 81-11.

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

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

United States Geological Survey (U.S.G.S.), 2022a, ANSS Comprehensive Earthquake Catalog (ComCat), https://earthquake.usgs.gov/data/comcat/.

United States Geological Survey (U.S.G.S.), 2022b, Quaternary Faults and Folds in the U.S. database, https://earthquake.usgs.gov/static/lfs/nshm/qfaults/qfaults.kmz.

United States Geological Survey (U.S.G.S.), 2022c, National Water Information Service: Web Interface, Groundwater Levels for the Nation, updated January 2022 http://nwis.waterdata.usgs.gov/nwis/gwlevels?.

MAPS UTILIZED

Bortugno, E.J., and Spittler, T.E., 1986, Geologic Map of the San Bernardino Quadrangle, Scale 1: 250,000, Regional Geologic Map Series, Map No. 3A, 5 Sheets.

California Division of Mines and Geology (C.D.M.G.), 1988, Apple Valley South 7½' Quadrangle, Earthquake Fault Zone Map, Scale 1" = 2,000'.

California Division of Mines and Geology (C.D.M.G.), 1974, Cajon 7½' Quadrangle, Earthquake Fault Zone Map, Scale 1" = 2,000'.

California Geological Survey (CGS), 2010, Geologic Compilation of Quaternary Surficial Deposits in Southern California, San Bernardino 30' X 60' Quadrangle, CGS Special Report 217, Plate 13, Scale 1: 100,000.

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

Dibblee, T.W., Jr., 1965, Geologic Map of the Hesperia 15-Minute Quadrangle, San Bernardino County, California, U.S.G.S. Open-File Report 65-43, Scale 1: 24,000.

Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map, Community Panel No. 06071C-6475H, August 28, 2008, Scale 1: 24,000.

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

Dibblee, T.W., Jr., 2008, Geologic Map of the Hesperia 15-Minute Quadrangle, San Bernardino County, California, Dibblee Geology Center Map #DF-382, Scale 1: 62,500.

Jennings, C.W., 1992, Preliminary Fault Activity Map of California, Scale 1: 750,000, C.D.M.G. Open File Report 92-03.

Morton, D.M. and Miller, F.K., 2006, Geologic Map of the 30' x 60' San Bernardino Quadrangle, California, Scale 1: 100,000, U.S.G.S. Open-File Report 2006-1217, 4 sheets, 5 figures, Version 1.0.

San Bernardino County, 2010, San Bernardino County Land Use Plan, General Plan, Hazard Overlays, Hesperia Map FH06-B, dated March 9, 2010, Scale 1: 14,400.

San Bernardino County, 2010, San Bernardino County Land Use Plan, General Plan, Geologic Hazard Overlays, Victorville/San Bernardino Map EHFH-C, dated March 9, 2010, Scale 1: 115,200.

Toppozada, T.R. et al., 2000, Epicenters of and Areas Damaged by M≥4 California Earthquakes, Map Sheet 49, Scale 1" = 25 Miles.