September 6, 2024
Project No. S168-193

STK ARCHITECTURE, INC.
42095 Zeno Drive, Suite A15
Temecula, California 92590

Attention: Tony Finaldi

Subject: Geotechnical Investigation
Proposed Fire Station 227
NWC Genevieve Street N. and W. 38^{th} Street
San Bernardino, California

Dear Mr. Finaldi:

This report presents the results of the geotechnical investigation for the proposed Fire Station 227. The investigation was conducted in general conformance with our proposal dated March 6, 2024.

This report includes project design and construction recommendations along with the field and laboratory data. The primary geotechnical issues are the potential for strong ground shaking and the presence of variable soil conditions within the building areas that will require removal and recompaction.

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

Sincerely,
INLAND FOUNDATION ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
ALLEN D. EVANS
No. 2060
GEOTECHNICAL
STATE OF CALIFORNIA

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

PROFESSIONAL GEOLOGIST
CHRISTOPHER HOGAN RANGEL
No. 10071
Exp. 11/2024
STATE OF CALIFORNIA

Christopher Hogan Rangel, PG
Project Geologist
ADE:CHR:sd

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INTRODUCTION

This report presents the results of the geotechnical investigation conducted for the proposed San Bernardino County Fire Station 227. Fire Station 227 will be located on the northwest corner of Genevieve Street N. and W. 38th Street in San Bernardino, California. Our project understanding was based on the discussions with STK Architecture and review of the following plan.

- FS 227 – Conceptual Site Plan, W. 38th Street, San Bernardino, CA, prepared by STK Architecture Inc, dated June 12, 2024

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:

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

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

PROJECT DESCRIPTION

The Fire Station No. 227 project will consist of the construction of a new single-story structure comprising approximately 9,870 square feet. The station will be constructed on the southerly portion of the existing Arrowhead Elementary School property.

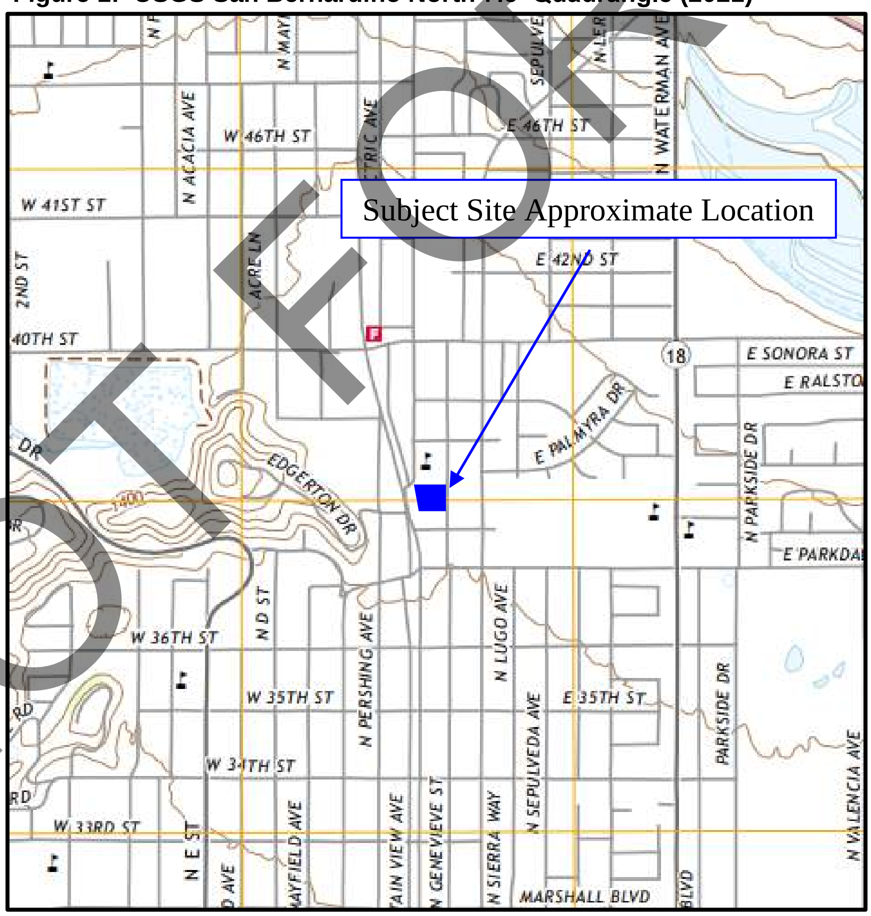
The fire station will include 3 truck bays, sleeping quarters for 8 crew members and will provide storage for 2 Type 1 Engines and a future ladder truck. A storage building and generator / fuel tank pad will be constructed in the northwest site area. Foundations for the proposed structures are expected to consist of shallow continuous and isolated concrete spread footings with slab-on-grade floors. Off-site improvements on Genevieve Street and 38th Street will be necessary. Site grading is expected to consist of minor cuts and fills of 2 to 3 feet, exclusive of remedial removals as recommended in this report.

Stormwater infiltration basins are planned in the southern and northwestern portions of the site. The basin depths are expected to be no deeper than five feet below existing surface grades.

SITE DESCRIPTION

The subject site is located on the northwest corner of Genevieve Street N. and W. 38th Street in San Bernardino, California (34.160118˚, -117.286677˚). The site occupies 1.21 acres and is currently covered with a grass field and large trees on the east, west and south sides. Figure 1 below shows the site location.

Figure 1: USGS San Bernardino North 7.5' Quadrangle (2021)



According to Google Earth, elevations at the site range from 1,279 to 1,283 feet above mean sea level (msl). The site slopes generally to the south at an overall rate of approximately 2 percent.

GEOLOGIC HAZARDS EVALUATION

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

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

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

Table 1: Seismic Design Parameters

Factor or Coefficient	Value
S_S	2.506g
S_1	1.002g
F_a	1.2g
F_V	1.7g
S_{DS}	1.670g
S_{D1}	1.620g
S_{MS}	2.506g
S_{M1}	2.429g
T_L	8 Seconds
MCE_G PGA	0.95g
Site Class	D

SUBSURFACE CONDITIONS

Subsurface exploration at the site consisted of four (4) exploratory borings to depths ranging from approximately 16.5 to 50.5 feet below existing site grades. The site exploration is described in Appendix A. Boring locations are shown on Figure A-7.

The soil encountered in the borings consisted of quaternary alluvial materials comprised of interlayered silty sand (SM), sand with silt and gravel (SP/SW-SM), and silty clayey sand (SC-SM). Cobbles and boulders were encountered in boring B-01 at a depth of 30.5 feet. The soil encountered was generally fine to very coarse grained, loose to very dense and in a slightly moist to moist state.

Corrosion Potential: Analytical testing indicates the concentration of sulfates is 52 ppm. In accordance with ACI 318, Table 4.2.1, the soil is classified as having a negligible sulfate exposure. The chloride concentration in the tested sample was 30 ppm and indicates that the soil is generally not corrosive with respect to ferrous metal. The soil is alkaline with a pH value of 8.5. The saturated minimum resistivity value of 7,790 ohm-cm indicates the soil may be moderately corrosive to buried ferrous metal. Alternative material such as PVC piping should be considered in the project design. Inland Foundation Engineering, Inc. does not practice corrosion engineering. A qualified corrosion engineer should be consulted for additional guidance.

Hydrocollapse Potential: Consolidation testing indicates that the soil is compressible and normally consolidated. The results show a slight to moderate potential for hydrocollapse when saturated under anticipated foundation and soil overburden loads. Provided that the building pad area is prepared as recommended herein, and appropriate surface drainage is provided in accordance with contemporary design practice, the potential for adverse building settlement due to hydrocollapse is not significant.

Expansive Soil: The site soil is granular, non-plastic, and non-expansive. Design measures to mitigate soil expansion are not necessary.

Groundwater: Groundwater was not encountered within the exploratory borings, which extended to a maximum depth of 50.5 feet below existing ground surface. Based on a review of pertinent groundwater data (referenced in appended geologic hazards report), the depth to the high groundwater mark in the general region is greater than 120 feet.

Liquefaction and Seismically-Induced Settlement: In general, liquefaction is a phenomenon that occurs where there is a loss of strength or stiffness in the soil that can result in the settlement of buildings, ground failure, or other hazards. The main factors contributing to this phenomenon are: 1) cohesionless, granular soil with relatively low density (usually of Holocene age); 2) shallow ground water (generally less than 50 feet);

and 3) moderate to high seismic ground shaking. Based on current and historical groundwater levels of more than 120 feet below ground surface, the potential for soil liquefaction is not significant.

"Dry sand" settlement occurs in loose granular soil as a result of seismic ground shaking. The potential for "dry sand" settlement was evaluated using GeoSuite® software and Pradel's method (1998). The results indicate a potential for seismically-induced "dry sand" settlement of less than 1 inch. The estimated differential seismic settlement is less than ½ inch over 30 feet. A discussion of the seismic settlement analysis, with graphic results, is included in Appendix D.

INFILTRATION TESTING

Infiltration testing was conducted in general accordance with Appendix D of the Technical Guidance Document for Water Quality Management Plans, prepared by CDM Smith for the County of San Bernardino Areawide Stormwater Program (2013). The shallow percolation test method was used per the Riverside County Department of Environmental Health guidelines. Four percolation tests were performed at the locations shown on Figure A-7. The testing procedures are described, and the test data is included in Appendix C of this report.

The test results are shown in Table 2. The corresponding calculated infiltration rate (I_c) ranges from 2.0 inches per hour to 5.7 inches per hour. These values exclude a factor of safety. The appropriate factor of safety should be determined by the design engineer.

Table 2: Infiltration Rate

Percolation Test No.	Percolation Rate (min/in)	Depth Below Ground Surface (in)	Infiltration Rate (I_c) (in/hr)
P-01	1.0	60	5.7
P-02	1.0	48	5.7
P-03	2.5	60	2.0
P-04	1.0	48	5.7

CONCLUSIONS AND RECOMMENDATIONS

The primary geotechnical issue that will require mitigation is the presence of loose compressible near surface soil conditions within the proposed structure areas. The near surface soil is not suitable for supporting foundations in its existing condition and should be over-excavated and recompacted. This and other geotechnical engineering recommendations for project design and construction are presented below.

Foundation Design: The proposed fire station and associated structures can be supported by shallow continuous and isolated spread footings designed with an allowable bearing pressure of 2,000 pounds per square foot (psf). Footings should have a minimum width of 12 inches and bottoms a minimum depth of 12 inches below the lowest adjacent grade. The allowable bearing pressure can be increased by 400 psf for each additional foot of width and by 800 psf for each additional foot of depth, to a maximum allowable bearing pressure of 4,000 psf. The allowable bearing pressure can be further increased by ⅓ for short-term transient wind and seismic loads.

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

Lateral Resistance: Resistance to lateral loads will be provided by a combination of friction acting at the base of the slab or foundation and passive earth pressure. A coefficient of friction of 0.45 between soil and concrete may be used with dead load forces only. A passive earth pressure of 250 psf/ft may be used for the sides of footings poured against recompacted or dense native material. These values may be increased by ⅓ for short-term transient wind and seismic loads. Passive earth pressure should be ignored within the upper one foot, except where confined as beneath a floor slab, for example.

Lateral Earth Pressure: Retaining walls should be designed for an active earth pressure equivalent to that exerted by a fluid weighing not less than 40 pcf. Any applicable construction or seismic surcharges should be added to this pressure. Retaining wall backfill should have an expansion index of less than 20.

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

Load bearing slabs should be designed using a modulus of subgrade reaction (k) not exceeding 200 pounds per square inch per inch. This value is based on an applied foundation load area of 1.0 square foot. The k value should be reduced for larger foundation areas according to the following formula:

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

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

Slabs should be designed and constructed in accordance with the provisions of the American Concrete Institute (ACI). Shrinkage of concrete should be anticipated and will result in cracks in all concrete slabs-on-grade. Shrinkage cracks may be directed to saw-cut "control joints" spaced on the basis of slab thickness and reinforcement.

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

Slabs to receive moisture-sensitive coverings should be provided with a moisture vapor retarder/barrier designed and constructed according to the American Concrete Institute 302.1 R, Concrete Floor and Slab Construction, which addresses moisture vapor retarder/barrier construction. At a minimum, the vapor retarder/barrier should comply with ASTM El745 and have a nominal thickness of at least 10 mils. The vapor retarder/barrier should be properly sealed, per the manufacturer's recommendations, and protected from punctures and other damage.

Portland Cement Concrete (PCC) Pavement: 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.

For all other areas that will utilize PCC pavement the below table can be utilized for design sections. The following Portland cement concrete pavement sections are based on the American Concrete Institute (ACI) Guide for Design and Construction of Concrete Parking Lots and Site Paving (ACI 330-21). The concrete to be utilized for Category A and B areas as well as pedestrian areas should have a minimum 28-day modulus of rupture of 500 psi. This corresponds to a compressive strength of approximately 2,500 psi. The actual pavement subgrade soil should be evaluated during construction to verify that the recommended pavement sections are appropriate.

Table 3: Portland Cement Concrete Pavement

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

The Class 2 aggregate base should comply with current Caltrans requirements. The aggregate base should be compacted to at least 95 percent relative compaction based on ASTM D1557. The upper 12 inches of pavement subgrade soil, below the aggregate base, should also be compacted to a minimum relative compaction of 95 percent.

The concrete pavement should be constructed with doweled joints and be restrained laterally by concrete curb/gutter or building foundations. The edges of the concrete should be protected from traffic loads by curbs or paved shoulders. If unrestrained pavement edges or non-doweled joints are desired, this firm should be contacted so that revised recommendations can be developed.

Construction joints should be sawcut in the pavement at a maximum spacing of 30 times the thickness of the slab, up to a maximum of 15 feet. Pavement sawcutting should be performed within 12 hours of concrete placement, preferably sooner. Sawcut depths should be equal to approximately 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 4.

Table 4: Asphalt Concrete Pavement

Service	Asphalt Concrete Thickness (ft.)	Base Course Thickness (ft.)
Light traffic (autos, parking areas, T.I. = 5.0)	0.25	0.35
Heavy traffic (trucks, driveways, T.I. =7.0)	0.30	0.45

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

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

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

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

2. **Preparation of Surfaces to Receive Compacted Fill:** All surfaces to receive compacted fill should be reviewed by a geologist or engineer from this firm prior

to processing. If roots or other deleterious materials are encountered or if the exposed excavation bottom is loose or unstable, additional over-excavation may be required until satisfactory conditions are encountered. Upon approval, surfaces to receive fill should be scarified to a minimum depth of eight inches, brought to near optimum moisture content, and compacted to a minimum of 90 percent relative compaction.

3. **Placement of Compacted Fill:** Fill materials consisting of on-site soil or approved imported granular soil should be spread in shallow lifts and compacted at near optimum moisture content to a minimum of 90 percent relative compaction, based on ASTM D1557. Fill placed within 10 feet of finish grade should not contain any particles larger than 12 inches (boulders). Boulder-size particles should be disposed of off-site or in designated rock disposal fill areas.

4. **Import Soil:** All proposed import soil should be tested prior to placement on the site to verify that it is not corrosive or expansive. Recommended import soil criteria are shown in the following Table 5.

Table 5: Recommended Import Soil Criteria

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

5. **Preparation of Building Areas:** All proposed building areas should be over-excavated to a depth of at least 8 feet below existing grade or 24 inches below the bottom of the deepest footing, whichever is greater. Building area excavation should extend laterally for at least 5 feet outside of exterior building foundation lines. Following excavation, the exposed soil should be evaluated by this firm to verify it is suitable to receive compacted fill. The removed soil should be placed and compacted as recommended above. Soil within 5 feet of finish grade and within 2 feet of footing and slab bottoms should not contain any particles larger than 3 inches (cobbles).

6. **Preparation of Paving Areas:** During final grading and immediately prior to the placement of aggregate base, all surfaces to receive asphalt concrete or Portland cement concrete paving should be processed to remove all particles larger than 3 inches within 12 inches of subgrade. The upper 12 inches of pavement subgrade should be tested to assure compaction for a depth of at least 12 inches. Compaction within proposed pavement areas should be to a minimum of 95 percent relative compaction for both the subgrade and base course.

7. **Utility Trench Backfill:** Utility trench backfill consisting of the on-site soil types should be placed by mechanical compaction to a minimum of 90 percent relative compaction. This is with the exception of the upper 12 inches under pavement areas where the minimum relative compaction should be 95 percent. Jetting of the native soil is not recommended.

8. **Testing and Observation:** During grading, tests and observations should be performed by a representative of this firm to verify that the grading is performed per the project specifications. Density testing should be performed per the current ASTM D1556 or ASTM D6938 test methods. The minimum acceptable degree of compaction should be 90 percent of the maximum dry density, based on ASTM D1557, except where superseded by more stringent requirements, such as beneath pavement. Where testing indicates insufficient density, additional compactive effort should be applied until retesting indicates satisfactory compaction.

LIMITATIONS

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

This report was prepared for STK Architecture, Inc. for their use in the design of the proposed Fire Station No. 227. This report may only be used by STK Architecture, Inc for this purpose. The use of this report by parties or for other purposes is not authorized without written permission by Inland Foundation Engineering, Inc. Inland Foundation Engineering, Inc. will not be liable for any projects connected with the unauthorized use of this report.

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

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

REFERENCES

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

American 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.

County of San Bernardino (2013), Areawide Stormwater Program, Technical Guidance Document for Water Quality Management Plans

Terra Geosciences, Geologic Hazards Report, San Bernardino County Fire Station 227, NWC of 38th Street and Genevieve Avenue, City of San Bernardino, California, Project No. 244073-1, dated July 20, 2024

United States Geologic Survey, San Bernardino North 7.5' Quadrangle (2021)

APPENDIX A –
Site Exploration

APPENDIX A

SITE EXPLORATION

Four exploratory borings were drilled at the approximate locations shown on Figure A-7. The materials encountered during drilling were logged by a staff geologist. Boring logs are included with this report as Figures A-3 through A-6.

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

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487)

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

CONSISTENCY CRITERIA BASES ON FIELD TESTS

RELATIVE DENSITY – COARSE – GRAIN SOIL

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

CONSISTENCY – FINE-GRAIN SOIL

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

* NUMBER OF BLOWS OF 140 POUND HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1 3/8 INCH I.D.) SPLIT BARREL SAMPLER (ASTM -1586 STANDARD PENETRATION TEST)

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

MOISTURE CONTENT

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

CEMENTATION

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

EXPLANATION OF LOGS

LOG OF BORING B-01

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

DATE DRILLED 7/11/24

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.

U.S.C.S.	Description
	GRASS,
SM	**SILTY SAND,** with trace gravel, fine to very coarse, very dark grayish-brown (10YR 3/2), moist, medium dense.
SP-SM	**SAND with SILT and GRAVEL,** fine to very coarse, gray-brown (2.5Y 5/2), slightly moist, medium dense.
SM	**SILTY SAND,** with trace gravel, fine to very coarse, very dark grayish-brown (10YR 3/2), moist, medium dense.
SW-SM	**SAND with SILT and GRAVEL,** fine to coarse, grayish-brown (10YR 5/2), slightly moist, medium dense.
SC-SM	**SILTY, CLAYEY SAND,** very fine to fine, olive-brown (2.5Y 4/4), moist, medium dense.
SW-SM	**SAND with SILT and GRAVEL,** fine to very coarse, light olive-brown (2.5Y 5/4), slightly moist, medium dense.
SM	**SILTY SAND,** with trace gravel, very fine to fine, grayish-brown (2.5Y 5/2), moist, dense.
SW-SM	**SAND with SILT and GRAVEL,** fine to very coarse, gray-brown (2.5Y 5/2), slightly moist, medium dense.
	COBBLES and BOULDERS,
	End of boring at 30.8 feet. No groundwater encountered. Backfilled with native soil.

SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
AU			
SS	9 9	11	112
AU			
SS	6 11	6	93
SS	8 13	5	113
AU			
SS	23 18	2	130
SS	9 10	12	117
SS	18 25	5	114
SPT	33 28	10	
SPT	30 50/3"	2	

Depth (ft): 5, 10, 15, 20, 25, 30

IFE Inland Foundation Engineering, Inc.

CLIENT	STK Architecture, Inc.
PROJECT NAME	Fire Station #227
PROJECT LOCATION	NWC Genevieve St. & W. 38th St. San Bernardino, CA
PROJECT NUMBER	S168-193

FIGURE NO. A-3

IFE BORING - GINT STD US LAB.GDT - 8/19/24 14:46 - P:\S168\S168-193 SB FS 227\GINT.GPJ

LOG OF BORING B-02

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

DATE DRILLED 7/11/24

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.	DESCRIPTION	BULK SAMPLE	DRIVE SAMPLE	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
0		**GRASS,**						
	SM	**SILTY SAND,** with trace gravel, fine to medium, dark grayish-brown (10YR 5/2), moist, loose.						
5				X	SS	6 6	5	109
				X	SS	4 5	9	106
10	SM	**SILTY SAND,** with trace gravel, fine to medium, light olive-brown (2.5Y 5/2), slightly moist, medium dense.		X	SS	8 12	3	109
	SP-SM	**SAND with SILT,** fine to coarse, dark grayish-brown (10YR 4/2), moist, loose to medium dense.		X	SS	6 8	8	110
15				X	SS	12 14	6	107
20	SW-SM	**SAND with SILT and GRAVEL,** fine to medium, grayish-brown (10YR 5/2), slightly moist, medium dense.		X	SS	17 24	3	117
25				X	SPT	13 19	5	
30	GW	**GRAVEL with SAND,** fine to coarse, olive-gray (5Y 4/2), moist, dense.		X	SPT	35 50	3	
35		**SAND with SILT and GRAVEL,** fine to medium, olive-gray (5Y 5/2), slightly moist, dense.		X	SPT	50	2	
40				X	SPT	50/5"	3	
45				X	SPT	31 50	3	
50				X	SPT	50/5"	7	

End of boring at 50.5 feet. No groundwater encountered. Backfilled with native soil.

IFE Inland Foundation Engineering, Inc.

CLIENT	STK Architecture, Inc.
PROJECT NAME	Fire Station #227
PROJECT LOCATION	NWC Genevieve St. & W. 38th St. San Bernardino, CA
PROJECT NUMBER	S168-193

FIGURE NO. A-4

LOG OF BORING B-03

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

DATE DRILLED 7/11/24

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)
		GRASS,	AU			
	SM	**SILTY SAND,** fine to medium, dark grayish-brown (2.5Y 4/2), slightly moist, loose.				
			SS	3 6	4	107
5						
			SS	3 8	7	111
		- rootlets throughout -				
	SP-SM	**SAND with SILT and GRAVEL,** fine to coarse, light olive-brown (2.5Y 5/3), slightly moist, medium dense.	AU			
			SS	8 11	3	108
10						
			SS	12 25	4	116
	SM	**SILTY SAND,** fine to coarse, brown (10YR 4/2), moist, medium dense.				
	SW-SM	**SAND with SILT and GRAVEL,** fine to coarse, light olive-brown (2.5Y 5/3), slightly moist, medium dense.				
15	SM	**SILTY SAND with GRAVEL,** fine to medium, light olive-brown (2.5Y 5/3), slightly moist, medium dense.	SS	10 14	5	113
	SW-SM	**SAND with SILT and GRAVEL,** fine to coarse, OLIVE-GRAY (5y 4/2), slightly moist, medium dense.				
20						
			SS	21 35	2	131

End of boring at 21.5 feet. No groundwater encountered. Backfilled with native soil.

IFE Inland Foundation Engineering, Inc.

CLIENT	STK Architecture, Inc.
PROJECT NAME	Fire Station #227
PROJECT LOCATION	NWC Genevieve St. & W. 38th St. San Bernardino, CA
PROJECT NUMBER	S168-193

FIGURE NO. A-5

LOG OF BORING B-04

DRILLING RIG	**Mobile B-61**	DATE DRILLED	**7/11/24**	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.

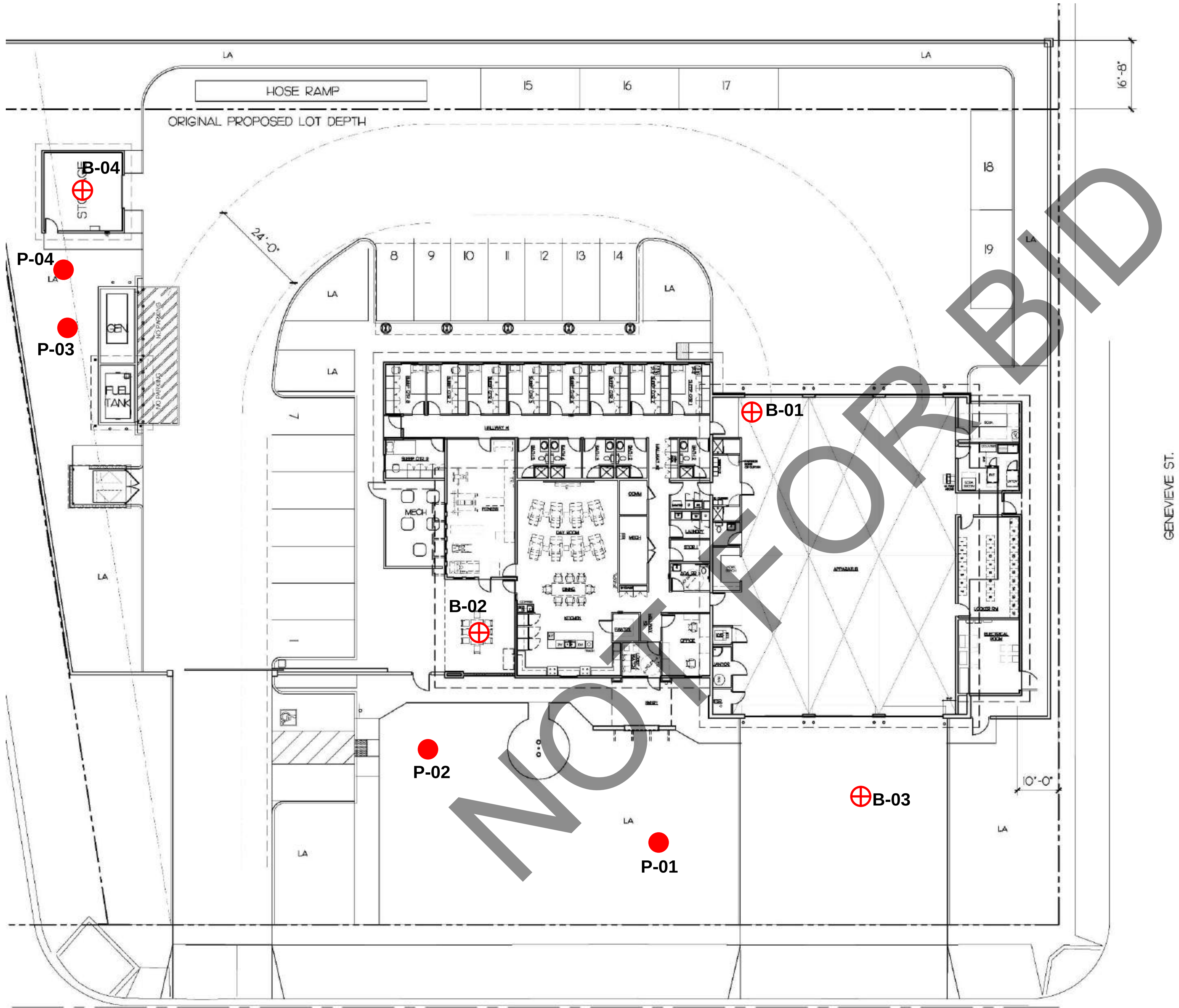
DEPTH (ft)	U.S.C.S.	SUMMARY OF SUBSURFACE CONDITIONS	SAMPLE TYPE	BLOW COUNTS /6"	MOISTURE (%)	DRY UNIT WT. (pcf)
		GRASS,	AU			
	SM	**ARTIFICIAL FILL,** SILTY SAND, with trace gravel, fine to medium, grayish-brown (10YR 5/2), slightly moist, medium dense.				
			SS	16 18	3	110
	SM	**SILTY SAND,** fine to coarse, grayish-brown (10YR 5/2), slightly moist, medium dense.	AU			
5	SW-SM	**SAND with SILT and GRAVEL,** with trace gravel, fine to coarse, light olive-brown 2.5Y 5/4), slightly moist, medium dense.	SS	10 14	3	115
			SS	8 8	6	102
10	GW	**GRAVEL with SAND,** fine to coarse, light olive-brown (2.5Y 5/4), slightly moist, medium dense.	SS	11 14	3	115
15	SW-SM	**SAND with SILT and GRAVEL,** with trace gravel, fine to coarse, light olive-brown 2.5Y 5/4), slightly moist, medium dense.	SS	22 31	4	

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

IFE Inland Foundation Engineering, Inc.

CLIENT	**STK Architecture, Inc.**
PROJECT NAME	**Fire Station #227**
PROJECT LOCATION	**NWC Genevieve St. & W. 38th St.** **San Bernardino, CA**
PROJECT NUMBER	**S168-193**

FIGURE NO.
A-6



SITE PLAN

STK Architecture, Inc.

Proposed Fire Station 227

NWC Genevieve Street N. and W. 38th Street

San Bernardino County, California

LEGEND

Approximate Location of Exploratory Boring -

Approximate Location of Percolation Test -

Base Map: Prepared by STK Architecture, Inc.

N

IFE Inland Foundation Engineering, Inc.
1310 S. Santa Fe Avenue, San Jacinto, CA 92583 | (951) 654-1555

Figure A-7	STK Architecture, Inc. Proposed Fire Station 227 San Bernardino County, California	
	Drawn By: HR	Project No. S168-193
	Not to Scale	Date: August 2024

APPENDIX B –
Laboratory Testing

APPENDIX B

LABORATORY TESTING

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

Unit Weight and Moisture Content: Ring samples were weighed and measured to evaluate their unit weight. A small portion of each sample was then tested for moisture content. The testing was performed per ASTM D2937 and D2216. The results of this testing are shown on the boring logs (Figures A-3 through A-6).

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

Sieve Analysis: Ten soil samples were selected for sieve analysis testing in accordance with ASTM D6913. These tests provide information for classifying the soil in accordance with the Unified Classification System. This classification system categorizes the soil into groups having similar engineering characteristics. The results of the testing are shown on Figures B-4 and B-5.

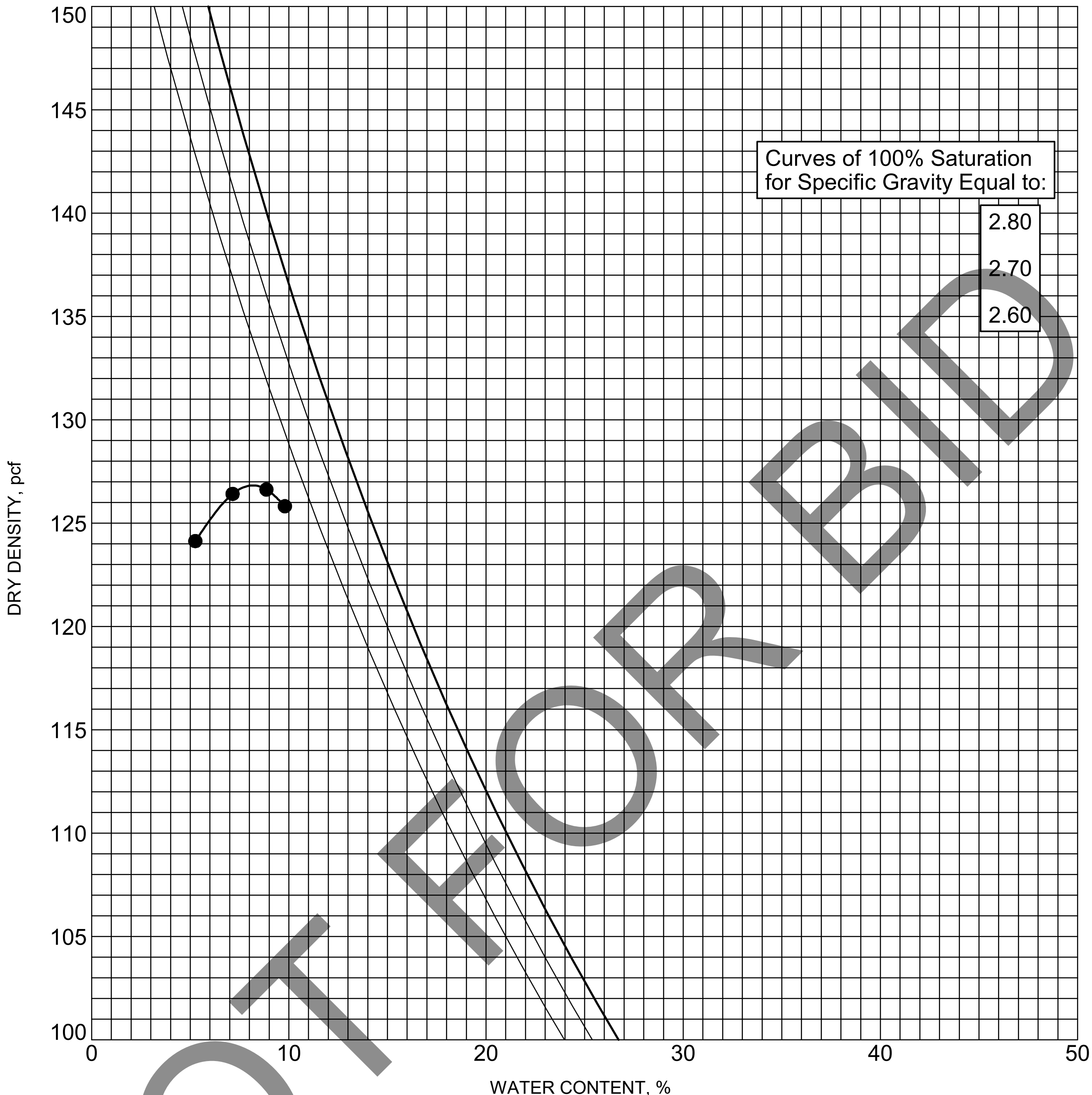
Plastic Index: Two samples were selected for plastic index testing in accordance with ASTM D4318. These tests provide information regarding soil plasticity and are also used for developing classifications for the soil in accordance with the Unified Classification System. The results of the testing are shown on Figures B-4 and B-5.

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

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

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

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



	BOREHOLE	DEPTH	Description of Materials	Max DD	Optimum WC
●	**B-01**	**0.0**	**SILTY SAND with GRAVEL(SM)**	**126.8 PCF**	**8.1 %**

MOISTURE-DENSITY CURVES (ASTM D1557)

INLAND FOUNDATION ENGINEERING, INC.

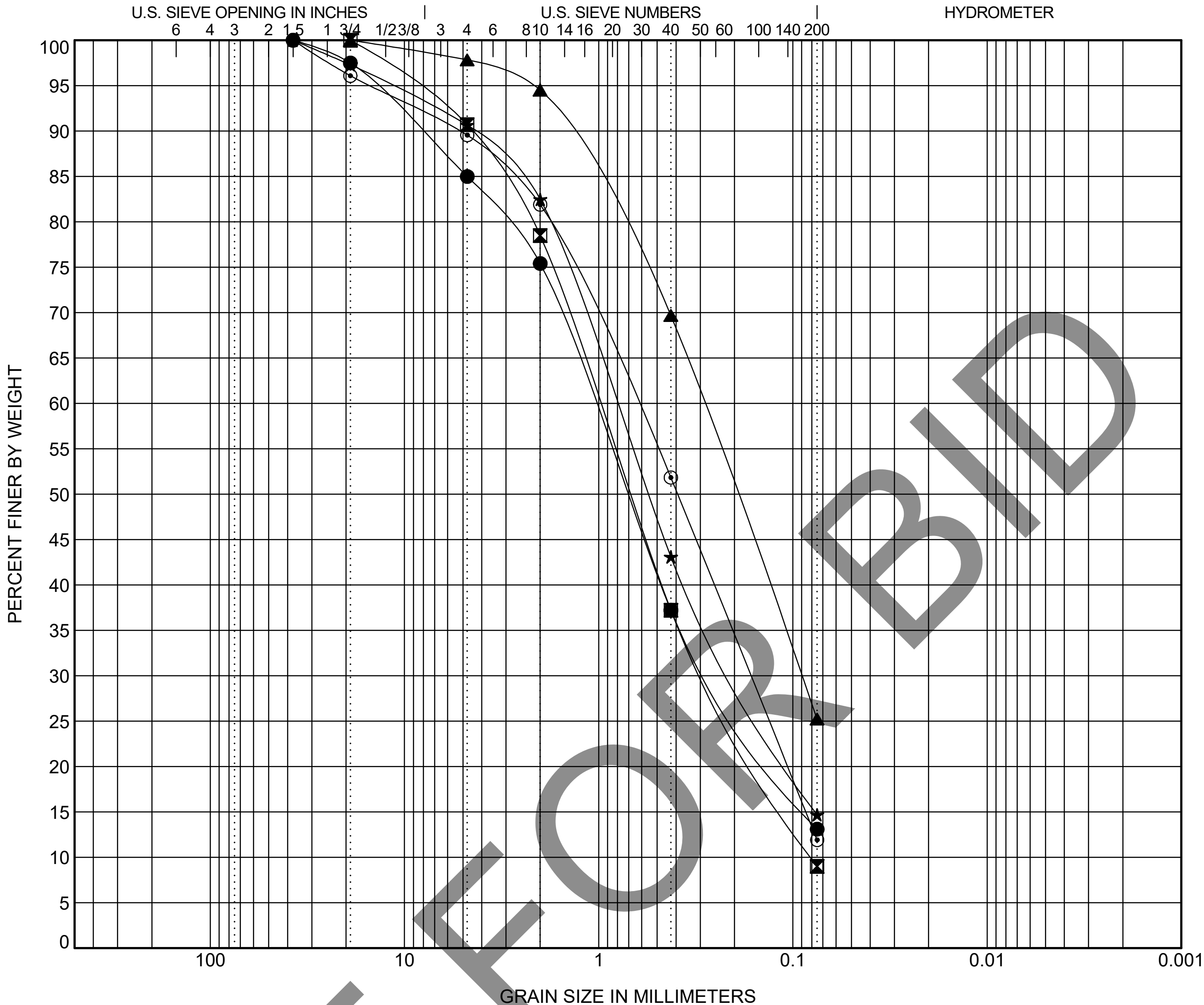
FIGURE NO. B-3

CLIENT STK Architecture, Inc.

PROJECT NAME Fire Station #227

PROJECT NUMBER S168-193

PROJECT LOCATION NWC Genevieve St. & W. 38th St.
San Bernardino, CA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	SAMPLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-01**	**0.0**	**SILTY SAND with GRAVEL(SM)**	**NP**	**NP**	**NP**		
⧗	**B-01**	**4.5**	**POORLY GRADED SAND with SILT(SP-SM)**	**NP**	**NP**	**NP**	**0.93**	**12.53**
▲	**B-02**	**0.3**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
★	**B-02**	**8.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
⊙	**B-02**	**12.5**	**POORLY GRADED SAND with SILT(SP-SM)**	**NP**	**NP**	**NP**	**0.61**	**9.38**

	BOREHOLE	DEPTH	D100	D90	D50	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-01**	**0.0**	**37.5**	**8.278**	**0.714**		**15.0**	**71.9**	**13.1**	
⧗	**B-01**	**4.5**	**19**	**4.517**	**0.687**	**0.08**	**9.3**	**81.7**	**9.0**	
▲	**B-02**	**0.3**	**19**	**1.51**	**0.197**		**2.2**	**72.6**	**25.2**	
★	**B-02**	**8.0**	**37.5**	**4.43**	**0.558**		**9.3**	**76.0**	**14.7**	
⊙	**B-02**	**12.5**	**37.5**	**5.223**	**0.393**		**10.4**	**77.6**	**11.9**	

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-4

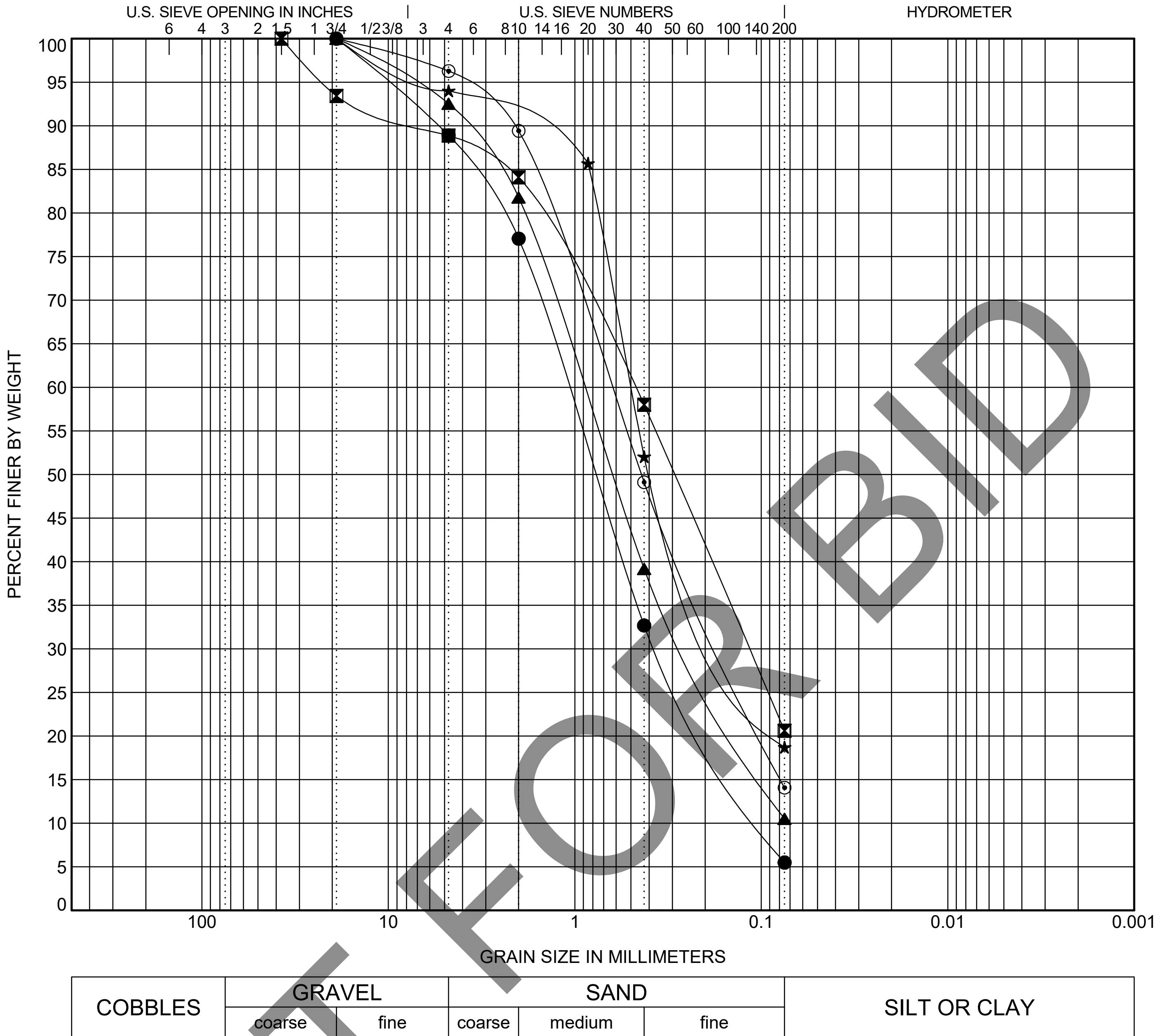
CLIENT STK Architecture, Inc.

PROJECT NAME Fire Station #227

PROJECT NUMBER S168-193

PROJECT LOCATION NWC Genevieve St. & W. 38th St.
San Bernardino, CA

IFE SIEVE ANALYSIS - GINT STD US LAB.GDT - 8/20/24 15:29 - P:\S168\S168-193 SB FS 227\GINT.GPJ



	SAMPLE	DEPTH	Classification	LL	PL	PI	Cc	Cu
●	**B-02**	**20.5**	**WELL-GRADED SAND with SILT(SW-SM)**	**NP**	**NP**	**NP**	**1.16**	**11.03**
⊠	**B-03**	**0.3**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
▲	**B-03**	**7.8**	**POORLY GRADED SAND with SILT(SP-SM)**	**NP**	**NP**	**NP**	**0.90**	**12.46**
★	**B-04**	**0.3**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		
⊙	**B-04**	**2.0**	**SILTY SAND(SM)**	**NP**	**NP**	**NP**		

	BOREHOLE	DEPTH	D100	D90	D50	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-02**	**20.5**	**19**	**5.451**	**0.778**	**0.1**	**11.1**	**83.4**	**5.5**	
⊠	**B-03**	**0.3**	**37.5**	**6.699**	**0.293**		**11.1**	**68.2**	**20.6**	
▲	**B-03**	**7.8**	**19**	**3.874**	**0.63**		**7.5**	**82.0**	**10.5**	
★	**B-04**	**0.3**	**19**	**2.069**	**0.382**		**6.0**	**75.3**	**18.7**	
⊙	**B-04**	**2.0**	**19**	**2.151**	**0.44**		**3.7**	**82.2**	**14.1**	

GRADATION CURVES (ASTM D6913, ASTM D4318)

INLAND FOUNDATION ENGINEERING, INC.

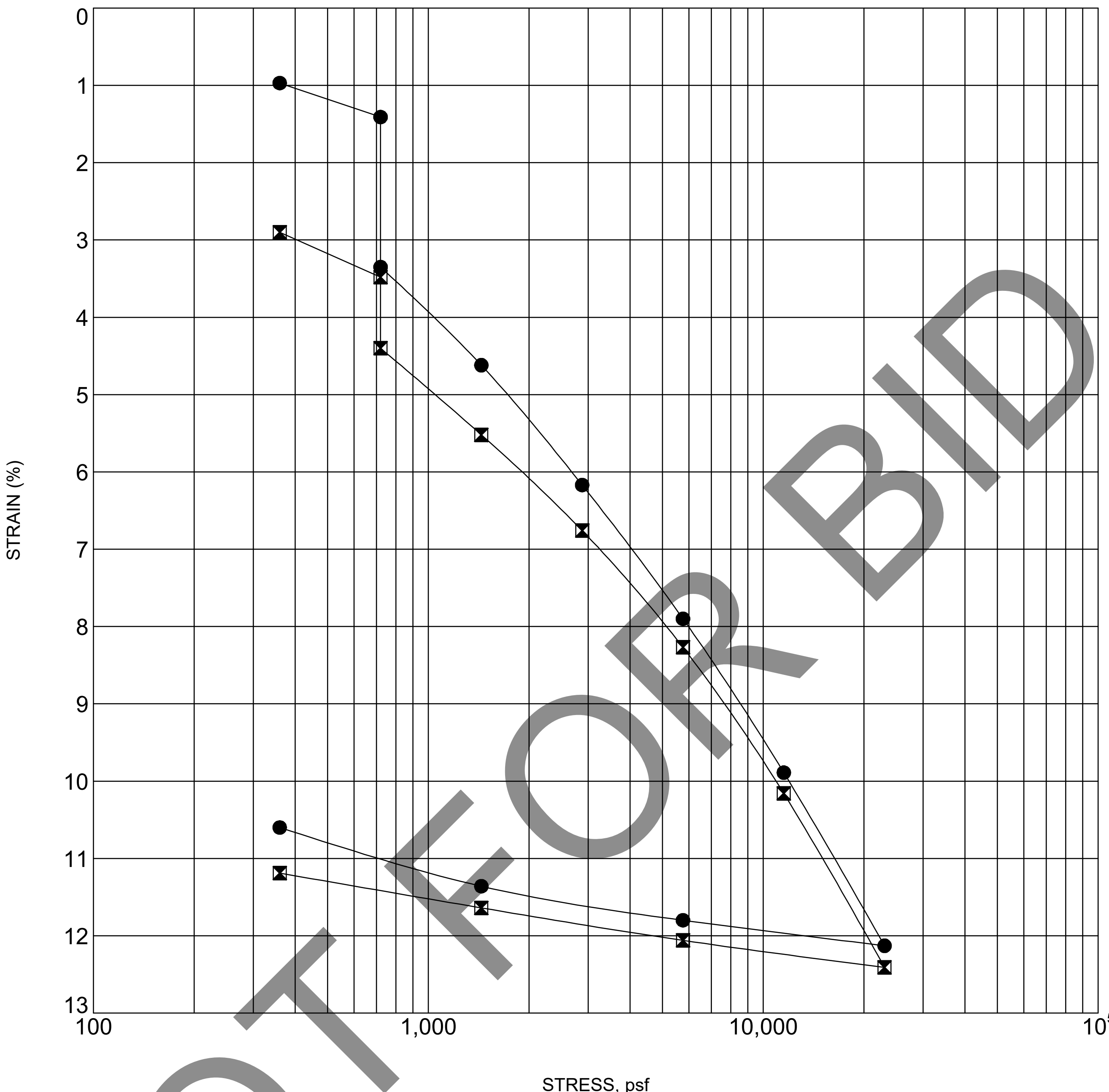
FIGURE NO. B-5

CLIENT STK Architecture, Inc.

PROJECT NAME Fire Station #227

PROJECT NUMBER S168-193

PROJECT LOCATION NWC Genevieve St. & W. 38th St.
San Bernardino, CA



	BOREHOLE	DEPTH	Classification	γ_d	MC%
●	**B-02**	**6.5**			
⧗	**B-03**	**5.5**			

CONSOLIDATION TEST (ASTM D2435)

INLAND FOUNDATION ENGINEERING, INC.

FIGURE NO. B-6

CLIENT STK Architecture, Inc.

PROJECT NAME Fire Station #227

PROJECT NUMBER S168-193

PROJECT LOCATION NWC Genevieve St. & W. 38th St.
San Bernardino, CA

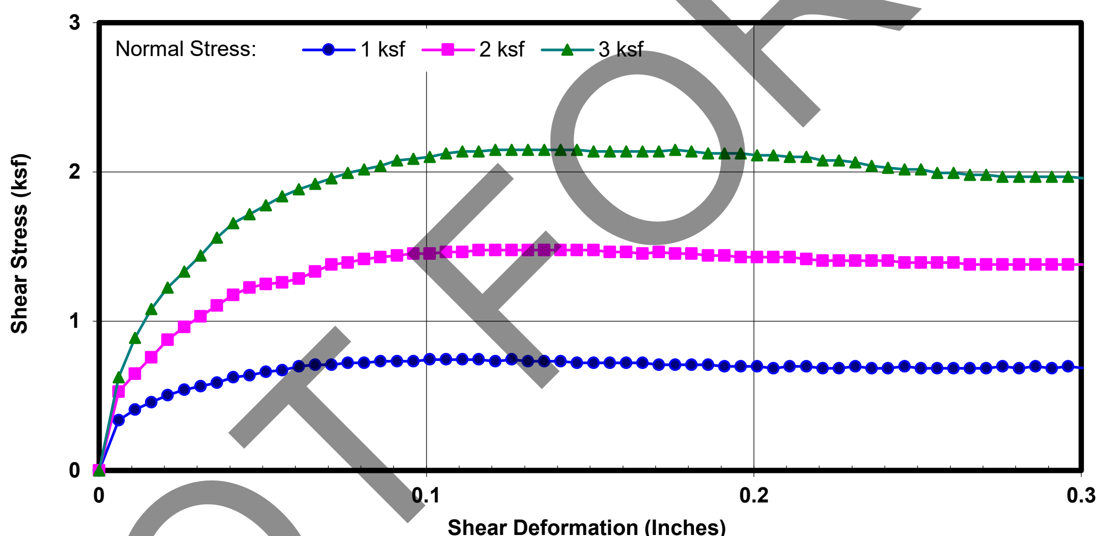
AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

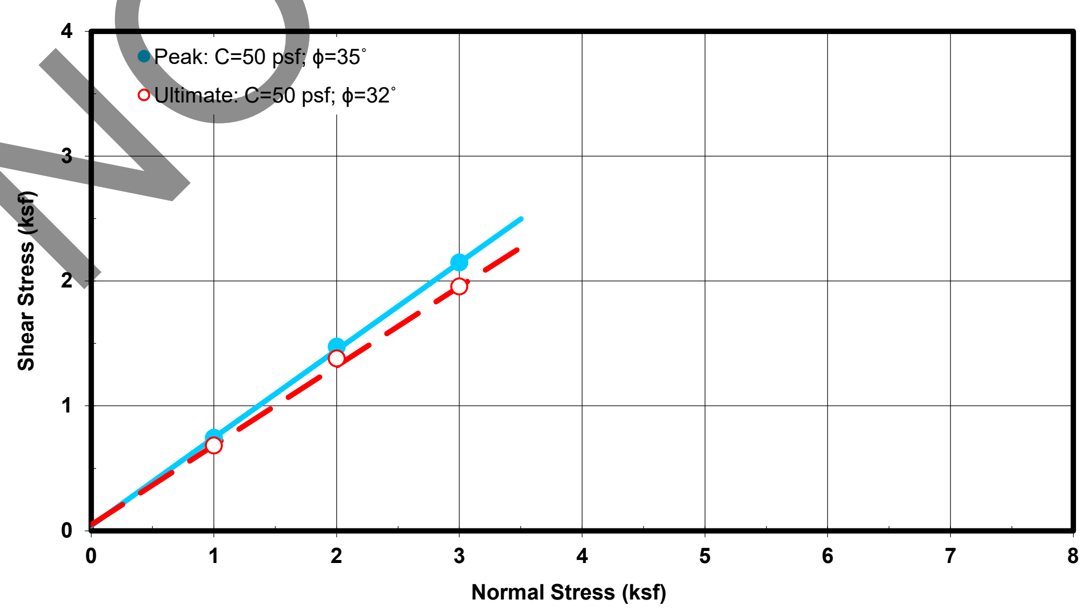
DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name: STK - Fire Station 227
Project No.: 5163-193
Boring No.: B-02
Sample No.: - **Depth (ft):** 3.5-4.5
Sample Type: Mod. Cal.
Soil Description: Silty Sand
Test Condition: Inundated **Shear Type:** Regular

Tested By: ST **Date:** 08/02/24
Computed By: JP **Date:** 08/07/24
Checked by: AP **Date:** 08/07/24

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)
110.0	106.4	3.4	19.3	16	89	1	0.744	0.684
						2	1.476	1.380
						3	2.148	1.956



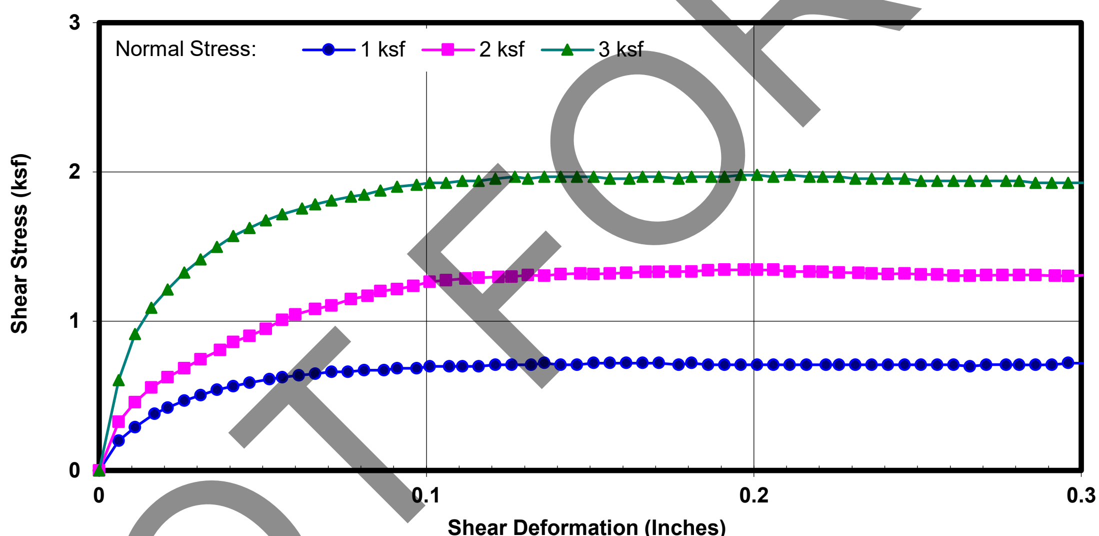


AP Engineering and Testing, Inc.
DBE|MBE|SBE
2607 Pomona Boulevard | Pomona, CA 91768
t. 909.869.6316 | f. 909.869.6318 | www.aplaboratory.com

DIRECT SHEAR TEST RESULTS
ASTM D 3080

Project Name:	STK - Fire Station 227	**Tested By:** ST	**Date:** 08/02/24
Project No.:	5163-193	**Computed By:** JP	**Date:** 08/07/24
Boring No.:	B-03	**Checked by:** AP	**Date:** 08/07/24
Sample No.:	- **Depth (ft):** 2.5-3.5		
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)
106.1	102.4	3.6	21.0	15	88	1	0.720	0.720
						2	1.344	1.308
						3	1.980	1.927



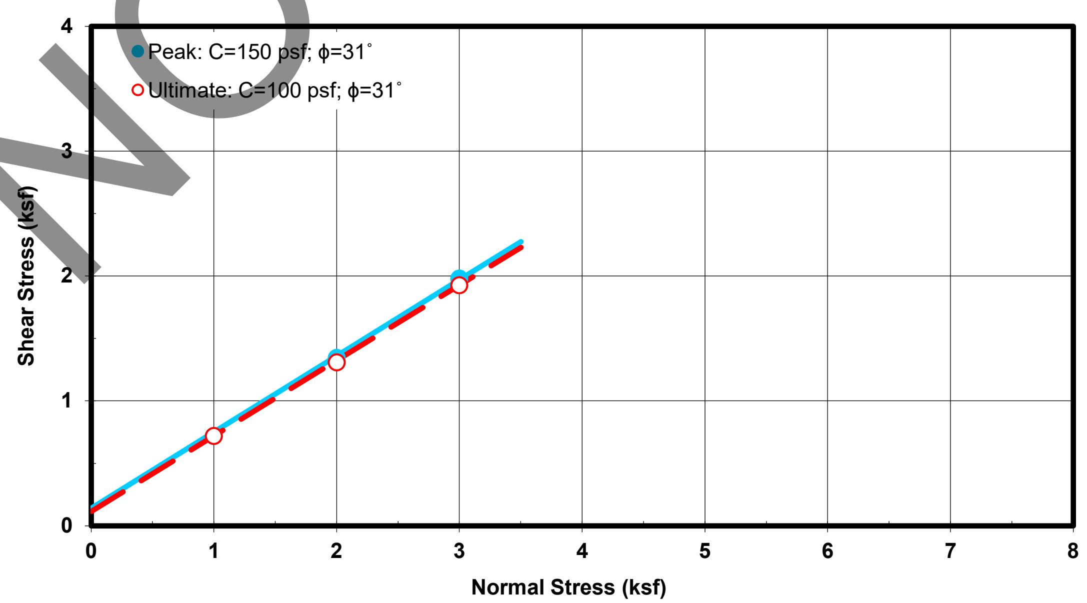


Figure No. B-8

CORROSION TEST RESULTS

Client Name: Inland Foundation Engineering
Project Name: STK - Fire Station 227
Project No.: 5163-193

AP Job No.: 24-0764
Date: 08/01/24

Boring No.	Sample No.	Depth (feet)	Soil Description	Minimum Resistivity (ohm-cm)	pH	Sulfate Content (ppm)	Chloride Content (ppm)
B-01	-	0-4.5	Silty Sand w/gravel	7,790	8.5	52	30

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

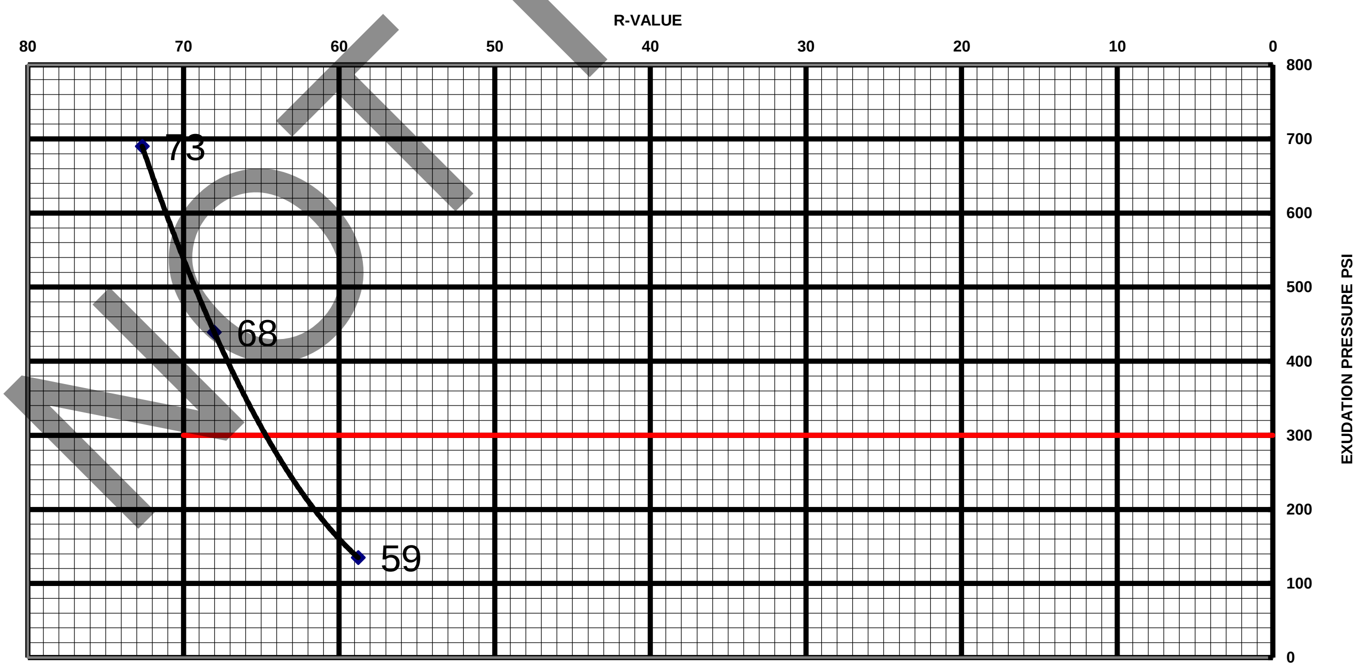
Job No. CB191148
Date. 8/8/2024

LAB0RATORY RECORD OF TESTS MADE ON BASE, SUBBASE, AND BASEMENT SOILS

CLIENT: **Inland Foundation Engineering**
PROJECT **STK-Fire Station 227 S168-193**
LOCATION:
R-VALUE # : **B-03**
T.I. :

	A	B	C	D
COMPACTOR AIR PRESSURE P.S.I.	350	350	350	
INITIAL MOISTURE %	3.7	3.7	3.7	
WATER ADDED, ML	70	60	50	
WATER ADDED %	6.6	5.6	4.7	
MOISTURE AT COMPACTION %	10.3	9.3	8.4	
HEIGHT OF BRIQUETTE	2.48	2.50	2.50	
WET WEIGHT OF BRIQUETTE	1106	1107	1106	
DENSITY LB. PER CU.FT.	122.6	122.7	123.7	
STABILOMETER PH AT 1000 LBS.	23	18	16	
2000 LBS.	38	29	26	
DISPLACEMENT	5.63	5.31	4.85	
R-VALUE	59	68	73	
EXUDATION PRESSURE	135	439	690	
THICK. INDICATED BY STAB.	0.00	0.00	0.00	
EXPANSION PRESSURE	0	0	0	
THICK. INDICATED BY E.P.	0.00	0.00	0.00	

EXUDATION CHART



R-Value: **64**

Figure No. B-10

APPENDIX C
Infiltration Testing

APPENDIX C

INFILTRATION TESTING

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

Four percolation tests were performed at the locations shown on Figure A-2. The test holes were drilled on July 11, 2024 to depths of approximately 4 and 5 feet below existing ground surface. The test holes were approximately eight (8) inches in diameter. Gravel was placed in the bottom of each test hole. The test holes were then pre-soaked by inverting 5-gallons of water above the test hole.

Testing was conducted 24 hours after the pre-soak on July 12, 2024. All pre-soak water had percolated through the test holes. For all tests, more than 6 inches of water seeped away twice consecutively in less than 25 minutes, which meets the sandy soil criteria. The tests were then run for an additional hour with measurements taken every 10 minutes.

The water percolated through all test holes within all 10-minute test intervals. The percolation rates were calculated to range from 1.0 to 2.5 minutes per inch (mpi). The percolation test rate was converted to an infiltration rate (I_c) using the Porchet method and the following equation:

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

Where:

r = Test Hole Radius (in.)
H_{avg} = Average Height of Water during Test Interval (in.)
ΔH = Change in Water Height during Test Interval (in.), and
Δt = Time Interval (in.)

The corresponding calculated infiltration rates (I_c) ranged from 2.0 to 5.7 inches per hour. These values exclude a factor of safety. Copies of the field test sheets are included with this report as Figures C-2 through C-5.

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: Fire Station 227	**Project No.: S168-193**	**Date: 7/12/2024**
Test Hole No.: P-01	**Tested By: Floyd Collins**	
Depth of Test Hole (D_T): 60”	**USCS Soil Classification: SM**	

Test Hole Dimensions (inches)		**Length**	**Width**
Diameter (if round)= 8”	**Sides (if rectangular) =**		

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6″ (Y/N)
1	6:59	7:24	25	31	58	27	Y
2	7:25	7:50	25	36	58	22	Y
3							

***If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25″.**

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	$\Delta D = \Delta H$ Change in Water Level (in.)	Perc. Rate min./in.	H_{Avg} $(D_T - D_o) + (D_T - D_f) \div 2$	I_T $\frac{\Delta H\ 60r}{\Delta t(r+2H_{Avg})}$
1	7:51	8:01	10	36	51	15	.67	16.5	9.7
2	8:02	8:12	10	36	48.5	12.5	.80	17.8	7.6
3	8:13	8:23	10	36	47	11	.91	18.5	6.4
4	8:24	8:34	10	36	46	10	1.0	19	5.7
5	8:35	8:45	10	36	46	10	1.0	19	5.7
6	8:47	8:57	10	36	46	10	1.0	19	5.7
7									
8									
9									
10									
11									
12									
13									
14									
15									

COMMENTS: Presoaked hole on 7/11/2024. Dry hole next day. First two measurements met sandy soil criteria. Overcast (75°)

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: Fire Station 227	**Project No.: S168-193**	**Date: 7/12/2024**
Test Hole No.: P-02	**Tested By: Floyd Collins**	
Depth of Test Hole (D_T): 48”	**USCS Soil Classification: SM**	
Test Hole Dimensions (inches)	**Length**	**Width**
Diameter (if round)= 8”	**Sides (if rectangular) =**	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6″ (Y/N)
1	7:03	7:28	25	24	46	22	Y
2	7:29	7:54	25	24	45.5	21.5	Y
3							

***If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25″.**

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	$\Delta D=\Delta H$ Change in Water Level (in.)	Perc. Rate min./in.	H_{Avg} $(D_T - D_o) + (D_T - D_f) \div 2$	I_T $\frac{\Delta H\,60r}{\Delta t(r+2H_{Avg})}$
1	8:59	9:09	10	24	36	12	.83	18	7.2
2	9:10	9:20	10	24	34.5	11.5	.87	18.8	6.0
3	9:20	9:30	10	24	34.5	11.5	.87	18.8	6.0
4	9:31	9:41	10	24	34	10	1.0	19	5.7
5	9:42	9:52	10	24	34	10	1.0	19	5.7
6	9:53	10:03	10	24	34	10	1.0	19	5.7
7									
8									
9									
10									
11									
12									
13									
14									
15									

COMMENTS: Presoaked hole on 7/11/2024. Dry hole next day. First two measurements met sandy soil criteria. Overcast (77°F)

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: Fire Station 227	**Project No.: S168-193**	**Date: 7/12/2024**
Test Hole No.: P-03	**Tested By: Floyd Collins**	
Depth of Test Hole (D_T): 60”	**USCS Soil Classification: SM**	
Test Hole Dimensions (inches)	**Length**	**Width**
Diameter (if round)= 8”	**Sides (if rectangular) =**	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6″ (Y/N)
1	7:06	7:31	25	35	46	11	Y
2	7:32	7:57	25	36	45	9	Y
3							

***If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25″.**

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	$\Delta D=\Delta H$ Change in Water Level (in.)	Perc. Rate min./in.	H_{Avg} $(D_T - D_o) + (D_T - D_f) \div 2$	I_T $\frac{\Delta H\, 60r}{\Delta t(r+2H_{Avg})}$
1	10:04	10:14	10	36	40	4	2.5	22	2.0
2	10:15	10:25	10	36	40	4	2.5	22	2.0
3	10:26	10:36	10	36	40	4	2.5	22	2.0
4	10:37	10:47	10	36	40	4	2.5	22	2.0
5	10:48	10:58	10	36	40	4	2.5	22	2.0
6	10:59	11:09	10	36	40	4	2.5	22	2.0
7									
8									
9									
10									
11									
12									
13									
14									
15									

COMMENTS: Presoaked hole on 7/11/2024. Dry hole next day. First two measurements met sandy soil criteria. Partly Cloudy (83°F)

PERCOLATION TEST DATA SHEET – INFILTRATION TESTING

Project: Fire Station 227	**Project No.: S168-193**	**Date: 7/12/2024**
Test Hole No.: P-04	**Tested By: Floyd Collins**	
Depth of Test Hole (D_T): 48”	**USCS Soil Classification: SM**	
Test Hole Dimensions (inches)	**Length**	**Width**
Diameter (if round)= 8”	**Sides (if rectangular) =**	

Sandy Soil Criteria Test*

Trial No.	Start Time	Stop Time	Time Interval, (min.)	Initial Depth to Water (in.)	Final Depth to Water (in.)	Change in Water Level (in.)	Greater than or Equal to 6″ (Y/N)
1	7:07	7:32	25	24	46	22	Y
2	7:33	7:58	25	24	45	21	Y
3							

***If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Otherwise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25″.**

Trial No.	Start Time	Stop Time	Δt Time Interval (min.)	D_o Initial Depth to Water (in.)	D_f Final Depth to Water (in.)	$\Delta D=\Delta H$ Change in Water Level (in.)	Perc. Rate min./in.	H_{Avg} $(D_T - D_o) + (D_T - D_f) \div 2$	I_T $\frac{\Delta H\, 60r}{\Delta t(r+2H_{Avg})}$
1	11:09	11:19	10	24	36	12	.83	18	7.2
2	11:20	11:30	10	24	35	11	.91	18.5	6.4
3	11:31	11:41	10	24	34.5	10.5	.95	18.8	6.0
4	11:42	11:52	10	24	34	10	1.0	19	5.7
5	11:53	12:03	10	24	34	10	1.0	19	5.7
6	12:04	12:14	10	24	34	10	1.0	19	5.7
7									
8									
9									
10									
11									
12									
13									
14									
15									

COMMENTS: Presoaked hole on 7/11/2024. Dry hole next day. First two measurements met sandy soil criteria. Partly cloudy (89°F)

APPENDIX D – Liquefaction and Seismic Settlement Analysis

APPENDIX D

LIQUEFACTION AND SEISMIC SETTLEMENT ANALYSIS

Liquefaction and seismic settlement potential were evaluated using the GeoSuite® computer program (version 3.2.1.6). The seismic parameters included a horizontal acceleration of 0.95g and a Moment Magnitude of 8.1. We analyzed the soil profile logged for exploratory boring B-02. The GeoSuite® program calculates corrected normalized SPT N-values $(N_1)_{60}$ using the following formula (SCEC, 1999).

$$(N_1)_{60} = N_M C_N C_E C_B C_R C_S$$

Where; N_M = measured standard penetration resistance. Modified California sample blowcounts were converted to SPT blowcounts using Burmister's formula (1948) prior to input in the program. The modified California sample blowcounts were also corrected to account for lined samplers, as described in the C_S factor discussion below.

C_N = depth correction factor. GeoSuite® calculates C_N for each layer in the soil profile using the relationship suggested by Idriss and Boulanger (2008)

C_E = hammer energy ratio (ER) correction factor. A C_E factor of 1.3 was applied for the automatic trip hammer used during drilling and was calculated using the relationship suggested by Idriss and Boulanger (2008).

C_B = borehole diameter correction factor. A C_B factor of 1.0 was applied for the 8-inch diameter hollow-stem augers with inside diameters of four (4) inches (SCEC 1999).

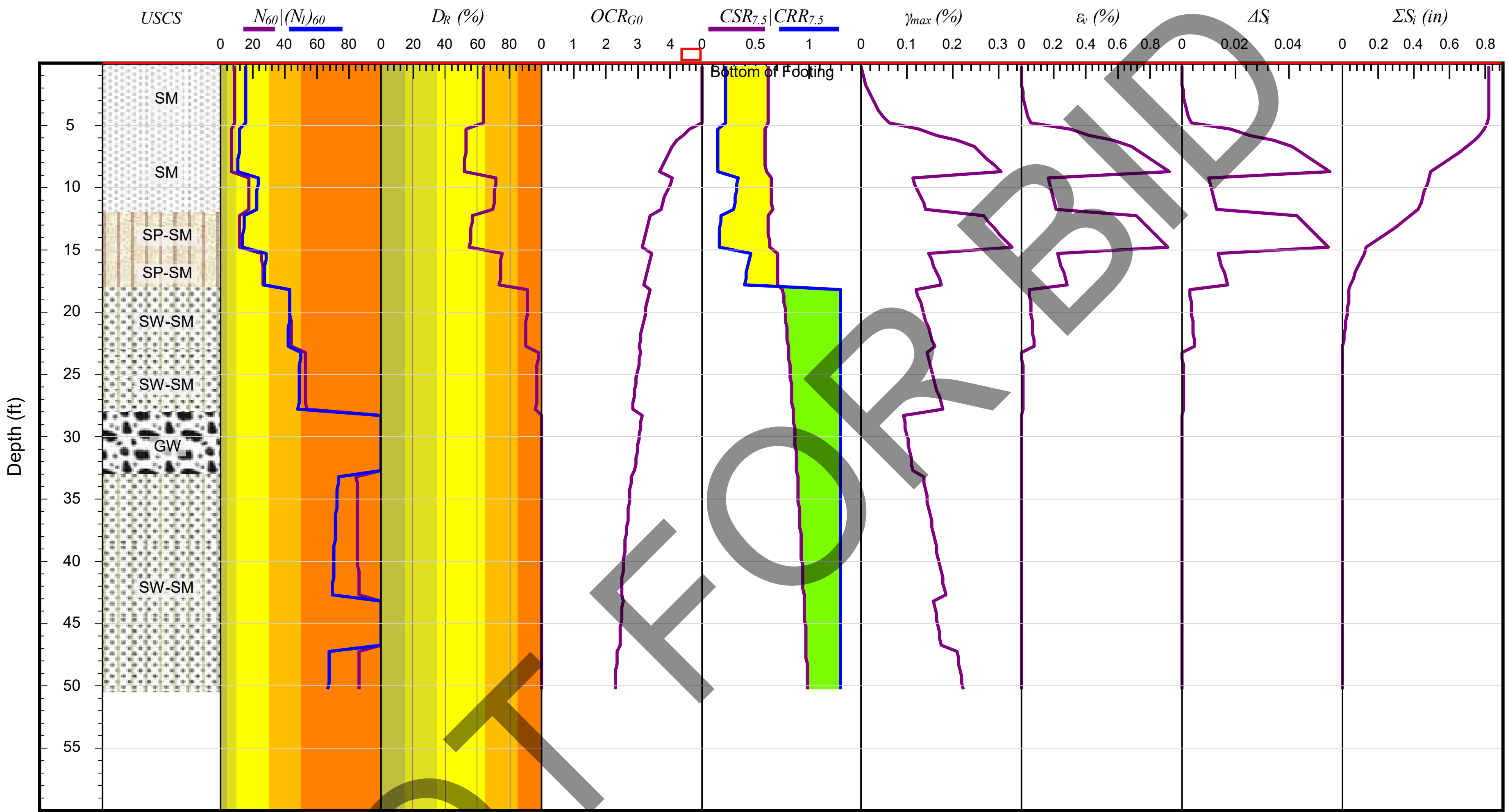
C_R = rod length correction factor. GeoSuite® applies a C_R factor for each layer in the soil profile using the values in Table 5.2 of the 1999 SCEC guidelines, and assuming a rod stick up length (above the ground surface) of 3 feet.

C_S = correction factor for samplers with or without liners. SPT samplers without liners were used for this project. For SPT samplers without liners, GeoSuite® applies a C_S factor for each layer in the soil profile using the relationships from Seed et al. (1984) and suggested by Idriss and Boulanger (2008). Since GeoSuite® applies a C_S factor to all layers in the soil profile, it is necessary to adjust blowcounts for modified California samplers with liners.

This was done through an iterative process by initially dividing the modified California sampler blowcounts by an assumed C_S value of 1.2 prior to input in the program.

Calculated C_S values were then checked against the assumed values and adjusted where necessary, so that the actual applied C_S value for modified California samples is 1.0.

The results of the analysis are shown on Figure D-2.



SM
SW-SM
SP-SM
GW

Silt Correction: UCLA method

Earthquake & Groundwater Information:
Magnitude = 8.1
Max. Acceleration = 0.95 g
Project GW = 100 ft
Maximum Settlement = 0.82 in
Settl. at Bottom of Footing = 0.82 in

Liquefaction: Boulanger & Idriss (2010-16)
Settl.: [dry] Yi (2022)
Lateral spreading: Idriss & Boulanger (2008)
M correction: [Sand] Boulanger & Idriss(2004)
σv correction: Idriss & Boulanger (2008)
Stress reduction: Idriss & Boulanger (2008)

IFE

Seismic Settlement Potential - SPT Data

Project:	Fire Station 227		
Location:	Genevieve and 38th		
Project No.:	S168-193	Boring No.:	B-02
Figure:	D-2		0

P:\S168\S168-193 SB FS 227\Geosuite\GeoSuite_S168-193_B-02 Insitu.csv

Prepared at 8/20/2024 4:11:09 PM

APPENDIX E –
Geologic Hazard Report

TERRA GEOSCIENCES

GEOLOGIC HAZARDS REPORT

SAN BERNARDINO COUNTY FIRE STATION 227

NWC OF 38TH STREET AND GENEVIEVE AVENUE

CITY OF SAN BERNARDINO, CALIFORNIA

Project No. 244073-1

July 20, 2024

Prepared for:

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

Consulting Engineering Geology & Geophysics

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

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

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

Regarding: Geologic Hazards Report
San Bernardino County Fire Station 227
NWC of 38th Street and Genevieve Avenue
City of San Bernardino, California
IFE Project No. S168-193

At your request, this firm has prepared a geologic hazards report for the proposed new San Bernardino County Fire Station 227, 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, 2022).

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.***

<u>Accompanying Maps and Appendices</u>

Plate 1	-	Regional Geologic Map
Plate 2	-	Google™ Earth Imagery Map
Plate 3	-	Site Plan
Appendix A	-	Shear-Wave Survey
Appendix B	-	Site-Specific Ground Motion Analysis
Appendix C	-	References

PROJECT SUMMARY

We understand that this report will be appended to your current geotechnical investigation, therefore, some descriptive sections such as site description, proposed development, etc., have been purposely omitted as they have been described in detail in your referenced report. No grading plans were available for this evaluation, and no field or subsurface exploration was performed by this firm. Only a review of available geologic and geotechnical data in our files was undertaken, including observation of the exploratory borings that were drilled by Inland Foundation engineering, Inc. (IFE) on July 11, 2024, including performing a seismic shear-wave survey.

GEOLOGIC SETTING

The subject site lies within a natural geomorphic province in California known as the Peninsular Ranges. This province is characterized by northwest-trending valleys and mountains that are, in part, due to the tectonic framework of this area, which is also dominated by a northwest-trending structure. Locally, the study area is included within a sub-structural unit of the Peninsular Ranges known as the San Bernardino Valley Block. This block is essentially a depressed region bounded by faults to the northeast (San Andreas), the southwest (San Jacinto), and the south (Banning).

The San Bernardino Valley is formed by a series of coalescing alluvial fans, of which the combined fan of the Santa Ana River and Mill Creek, originating from to the northeast, is the largest and most distinct. This and other alluvial fans (i.e., Lytle and Cajon Creeks, Devil Canyon, East Twin and City Creeks) emanate the mountains, then coalesce to form part of a broad alluvial plain, which then forms the San Bernardino Valley.

The subject area investigated for this report is included within the flood/alluvial plain limits of the San Bernardino Valley, situated near the eastern flank of Little Mountain, which is a low-lying bedrock hill that locally protrudes from the San Bernardino Valley. Geologic mapping of the area by Miller et al. (2001), as illustrated on Plate 1, indicates that the project development area is locally underlain by both slightly- to moderately consolidated early Holocene and late Pleistocene alluvial fan deposits (map symbol Qyf_1), generally described as sand and pebble-boulder gravel, along with late Holocene age very young wash deposits (map symbol Qw), consisting of unconsolidated to locally cemented sand, gravel, and boulder deposits. Relatively older and more consolidated alluvial deposits are presumed to underlie the subject site at depth.

The exploratory boring logs prepared by IFE (2024) indicate that the subject site is underlain predominantly by interbedded fine- to medium-grained silty sand, fine- to coarse-grained sand with silt, fine- to coarse-grained sand, and gravel with fine- to coarse-grained sand, along with gravel and cobbles throughout. These alluvial deposits were noted to be in a generally loose to very dense condition, to a depth of at least 50½ feet locally.

FAULTING

There are at least forty-three major late Quaternary active/potentially active faults that are located within a 100-kilometer (62-mile) radius of the subject site (Blake, 1989-2000). Of these, there are no known active faults that traverse the site based on available published literature, nor was there any surficial geomorphic evidence that was suggestive of faulting. Additionally, the subject site is not located within a State of California "Alquist-Priolo Earthquake Fault Zone" for surface-fault rupture hazard (California Division of Mines and Geology, 1974).

The nearest known "active" fault that is zoned by the California Geological Survey is the San Andreas Fault (San Bernardino North Segment), located approximately 1.1± miles to the northeast (C.D.M.G., 1974), as shown on the Regional Geologic Map, Plate 1, for reference. This fault segment is a right-lateral, strike-slip fault, being approximately 103-kilometers in length, with an associated maximum moment magnitude (M_W) of 7.4 and a slip-rate of 24 ±6 mm/year (C.D.M.G., 1996, Cao, et al., 2003, and Petersen et al., 2008).

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

GROUND MOTION ANALYSIS

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

Geographically, the subject construction area is centrally located at Latitude 34.1601 and Longitude -117.2866 and (World Geodetic System of 1984 coordinates). The mapped spectral acceleration parameters, coefficients, and other related seismic parameters, were evaluated using the California's Office of Statewide Health Planning and Development Seismic Design Maps (OSHPD, 2024) and the California Building Code criteria (CBC, 2022), with the site-specific ground motion analysis being performed following Section 21 of the ASCE 7-16 Standard (2017). The results of this site-specific analysis have been summarized and are tabulated below:

TABLE 1 – SUMMARY OF SEISMIC DESIGN PARAMETERS

Factor or Coefficient	Value
S_S	**2.506g**
S_1	**1.002g**
F_a	**1.2**
F_v	**1.7**
S_{DS}	**1.670g**
S_{D1}	**1.620g**
S_{MS}	**2.506g**
S_{M1}	**2.429g**
T_L	**8 Seconds**
MCEG PGA	**0.95g**
Shear-Wave Velocity (V100)	**1,075.1 ft/sec**
Site Classification	**D**
Risk Category	**IV**

HISTORIC SEISMICITY

A computerized search, based on Southern California historical earthquake catalogs, has been performed using the computer program EQSEARCH (Blake, 1989-2021) and the ANSS Comprehensive Earthquake Catalog (U.S.G.S., 2024a). The following table and discussion summarizes the historic seismic events (greater than or equal to M4.0) that have been estimated and/or recorded during the time period of 1800 to July 2024, within a 100-kilometer radius of the site.

TABLE 2 - HISTORIC SEISMIC EVENTS; 1800-2024 (100-kilometer radius)

Richter Magnitude (M)	No. of Events
4.0 - 4.9	628
5.0 - 5.9	73
6.0 - 6.9	15
7.0 - 7.9	1
8.0+	0

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

- The closest recorded notable earthquake epicenter (magnitude 4.0 or greater) is a M4.2 event (June 28, 1997), which occurred approximately three miles to the west-northwest.

- The nearest estimated significant historic earthquake epicenter (pre-1932) was approximately 4± miles southwest of the site (July 15, 1905, M5.3).

- The nearest recorded significant historic earthquake epicenter was a M5.6 event of October 16, 1999, located approximately 15 miles northeast of the site.

- The largest estimated historical earthquake epicenter (pre-1932) within a 62-mile radius of the site is a M6.9 event of December 8, 1812 (25± miles northwest).

- The largest recorded historical earthquake was the M7.6 Landers's event, located approximately 49 miles to the east (June 28, 1992).

- The largest estimated ground acceleration estimated to have been experienced at the site was at least 0.215g which resulted from the M5.3 event of July 15, 1905, located approximately 4± miles to the southwest (Blake, 1989-2000b) based on the attenuation relationship of Boore et al. (1997).

An Earthquake Epicenter Map which includes magnitudes 4.0 and greater for a 100-kilometer (62-mile) radius (blue circle) from the site (central blue dot), has been included below as Figure 1. This map was prepared using the ANSS Comprehensive Earthquake Catalog (U.S.G.S, 2024a) of instrumentally recorded events from the period of 1932 to July 2024.

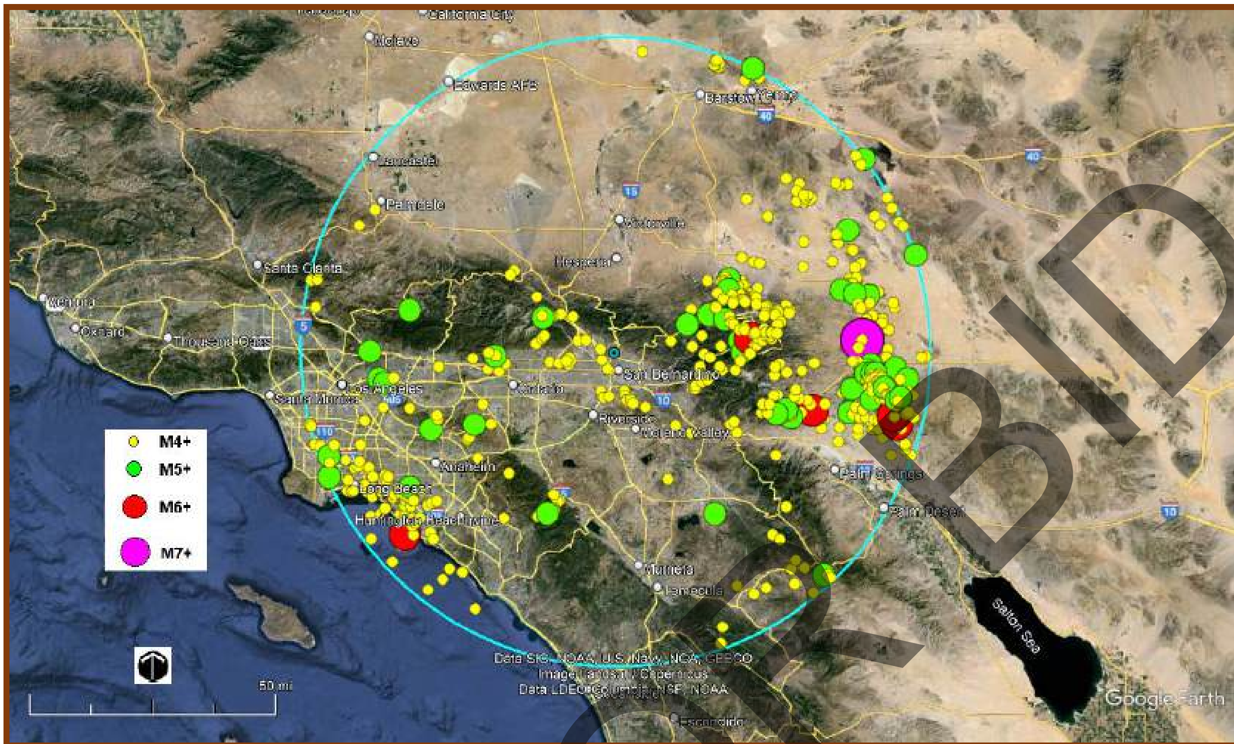


FIGURE 1- Earthquake Epicenter Map showing events of M4.0+ within a 100-kilometer radius.

GROUNDWATER

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

Based on groundwater data provided by the California Department of Water Resources (2024b), the closest measured well was located 1,900± feet southeast of the site (State Well No. 01N04W22J001S), which indicates that groundwater had ranged from a depth of 124 to 154± feet between the time period of 1940 to 1944. Groundwater data prepared by Matti and Carson (1991) indicates that high groundwater was estimated to be around 150± feet in depth based on contour data. During the recent subsurface investigation performed by IFE (2024), groundwater was not encountered within any of the exploratory borings excavated at the site to a depth of at least 50½ feet.

SECONDARY SEISMIC HAZARDS

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

Ground Rupture- Ground rupture is generally considered most likely to occur along pre-existing faults. Since no known active faults are believed to traverse the subject site, the probability of ground rupture is considered very low to nil.

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

Seismically-Induced Settlement- Seismically-induced settlement generally occurs within areas of loose granular soils. The proposed construction area is locally underlain by interbedded fine- to medium-grained silty sand, fine- to coarse-grained sand with silt, fine- to coarse-grained sand, and gravel with fine- to coarse-grained sand, with gravel and cobbles throughout. Locally, portions of the upper 8± feet of the surface were noted to be in a loose condition, directly underlain by medium dense to very dense sediments, to a depth of at least 50½ feet. Therefore, there appears to be at least a low potential for seismically-induced settlement to occur.

Landsliding- Due to the relatively low-lying relief of the site, landsliding of the site due to seismic shaking is considered nil. According to the City of San Bernardino Slope Stability and Major Landslides Map (2005, Figure S-7), the site is not shown to be within the limits of generalized landslide susceptibility.

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

Flooding (Water Storage Facility Failure)- Based on the data prepared by the California Department of Water Resources (2024a), the subject site is shown to be located within the limits of flood inundation in the event of catastrophic failure of the Little Mountain Dam, which is located approximately 2,700± feet to the northwest, as generally indicated on Figure 2 below (site outlined in red). Therefore, the potential for flooding due to water storage facility failure is considered possible. There are no other water-storage facilities that are topographically higher than that of the subject site, which could cause flooding due to catastrophic failure.

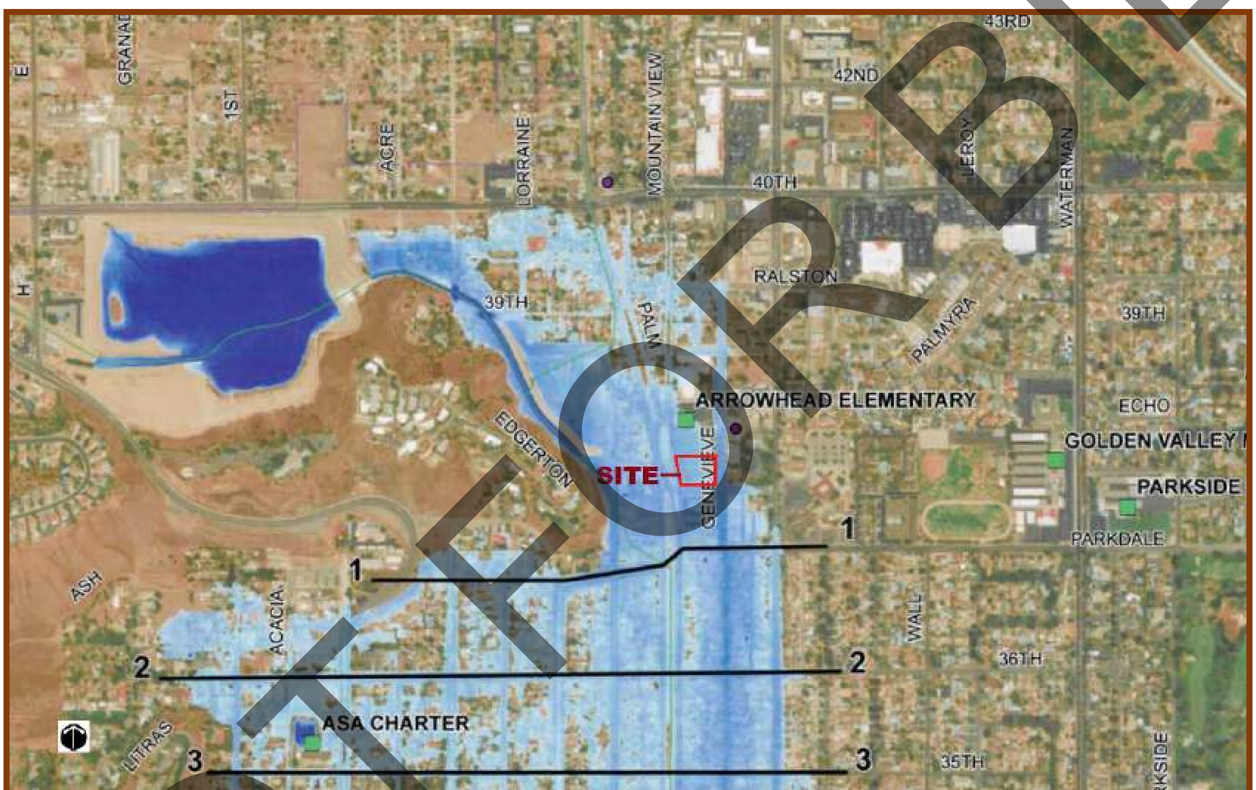


FIGURE 2- Dam Inundation Map (San Bernardino County, 2018); flooding shown as blue shading.

Seiches/Tsunamis-

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

Rockfalls-

The subject site lies upon a relatively flat-lying alluvial plain. Since no large rock outcrops are present at or adjacent to the site, the possibility of rockfalls during seismic shaking is nil.

FLOODING

According to the Federal Emergency Management Agency (FEMA), the subject site is not located within the boundaries of a 100-year flood (Community Panel No. 06071C 7945H, September 26, 2008). The site is shown to be located within "Other Flood Areas - Zone X," which is defined as "Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood." A portion of the FEMA Flood Zone Map is shown below in Figure 2 for reference.

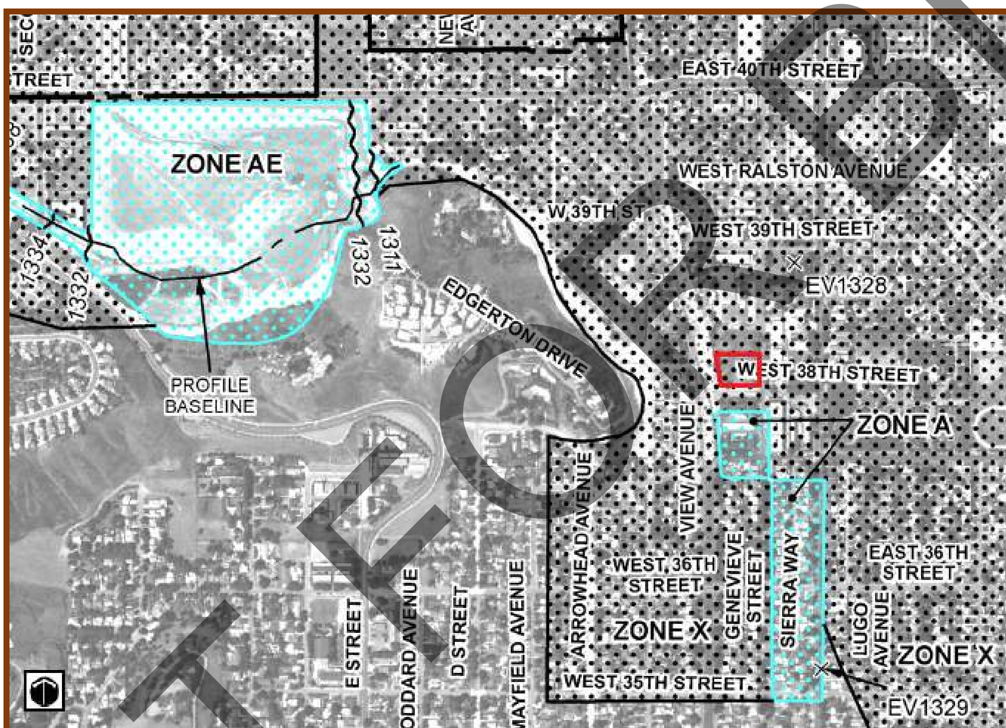


FIGURE 2- FEMA Flood Zone Map; Site boundary approximated by red outline.

GROUND SUBSIDENCE

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

OTHER GEOLOGIC HAZARDS

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

CONCLUSIONS AND RECOMMENDATIONS

General:

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

Conclusions:

1. Based on available published geologic data, the subject site is underlain by both slightly- to moderately consolidated early Holocene and late Pleistocene alluvial fan deposits, generally described as sand and pebble-boulder gravel, along with late Holocene age very young wash deposits, consisting of unconsolidated to locally cemented sand, gravel, and boulder deposits. Site-specific exploration performed by IFE indicates the site to be underlain by interbedded fine- to medium-grained silty sand, fine- to coarse-grained sand with silt, fine- to coarse-grained sand, and gravel with fine- to coarse-grained sand, with gravel and cobbles throughout. Locally, portions of the upper 8± feet of the surface were noted to be in a loose condition, directly underlain by medium dense to very dense sediments, to a depth of at least 50½ feet.

2. Groundwater was not encountered within the exploratory excavations performed by IFE to a depth of at least 50½ feet. Nearby historic and current groundwater data indicate that groundwater may have been as high as 125± feet in depth, locally. No shallow groundwater conditions are anticipated to be encountered during construction.

3. Based on our literature research, there are no active faults that are known to traverse the subject site. The nearest zoned active fault is associated with the active San Andreas Fault (North Branch) located approximately 1.1± miles to the northeast.

4. The primary geologic hazard that exists at the site is that of ground shaking, which accounts for nearly all earthquake losses. Moderate to severe ground shaking could be anticipated during the life of the proposed development.

5. Due to the nature of the surficial underlying unconsolidated sediments, there may be a potential for secondary seismic settlement to occur. Additionally, the site lies within the inundation limits in the event of catastrophic failure of the Little Mountain Dam, located approximately 2,700± feet to the northwest. No other permanent and/or transient secondary seismic hazards are expected to occur within the proposed construction area.

Recommendations:

1. The potential for seismically-induced settlement should be properly evaluated by the project Geotechnical Engineer. Appropriate site-specific mitigation measures, should be implemented as recommended, if warranted.

2. The potential for flooding due to catastrophic failure of Little Mountain Dam should be properly evaluated by the project Civil Engineer or other appropriate design professional. Appropriate site-specific mitigation measures, should be implemented as recommended, if warranted.

3. It is recommended that all structures be designed to at least meet the current California Building Code provisions in the latest 2022 CBC edition and the 2016 ASCE Standard 7-16, where applicable. However, it should be noted that the building code is intended as a minimum construction design and is often the maximum level to which structures are designed. Structures that are built to minimum code are designed to at least remain operational after an earthquake. It is the responsibility of both the property owner and project structural engineer to determine the risk factors with respect to using CBC minimum design values for the proposed facilities. When considering that a cascading rupture event could occur along the entire length of the San Andreas Fault Zone (which includes all segments), the resulting maximum moment magnitude earthquake is estimated to be M_W8.1, which should be used for seismic design purposes.

CLOSURE

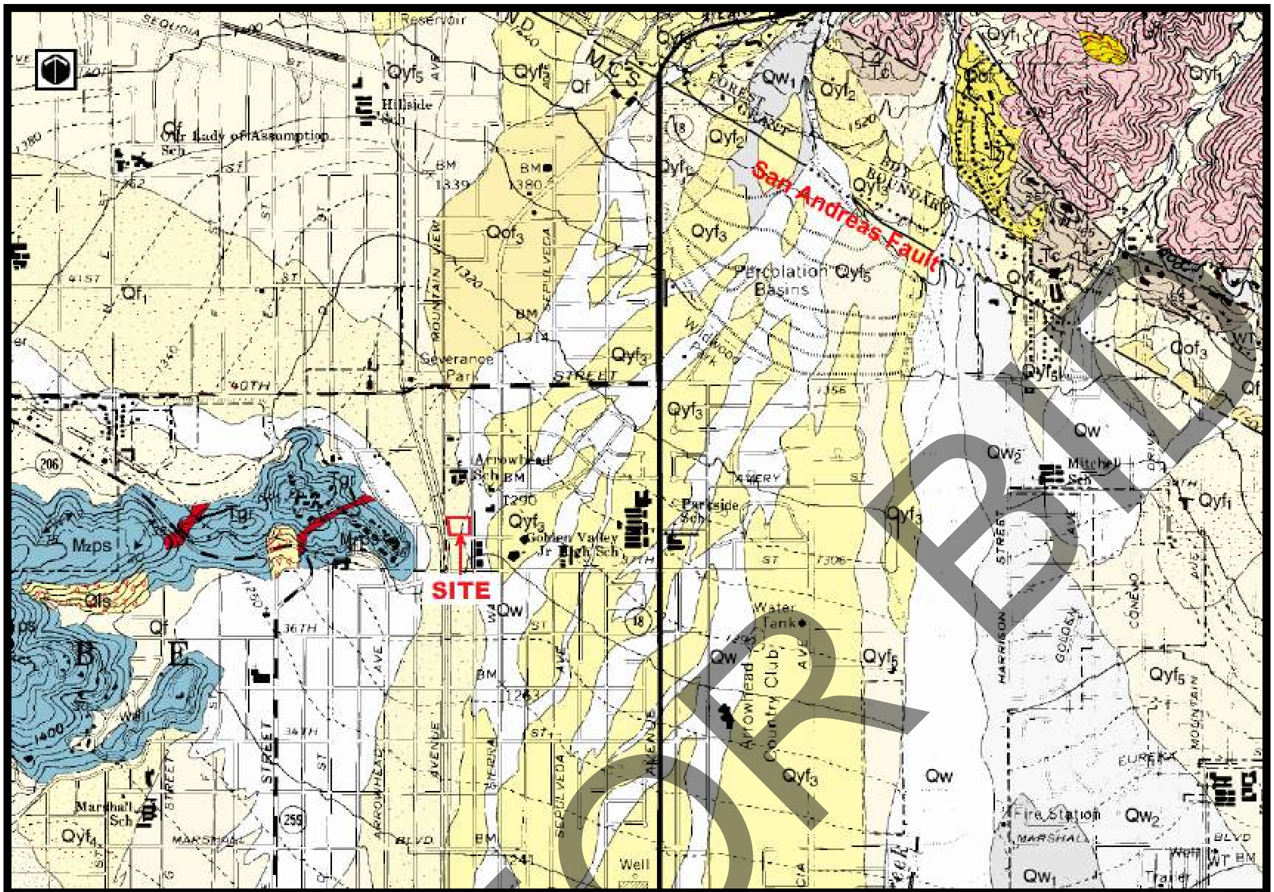
Our conclusions and recommendations are based on a review of available existing published geologic/seismic data and the provided site-specific subsurface exploratory boring logs. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. Should conditions be encountered at a later date or more information becomes available that appear to be different than those indicated in this report, we reserve the right to reevaluate our conclusions and recommendations and provide appropriate mitigation measures, if warranted. It is assumed that all the conclusions and recommendations outlined in this report are understood and followed.

If any portion of this report is not understood, it is the responsibility of the owner, contractor, engineer, and/or governmental agency, etc., to contact this office for further clarification.

Respectfully submitted,
TERRA GEOSCIENCES

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

REGIONAL GEOLOGIC MAP

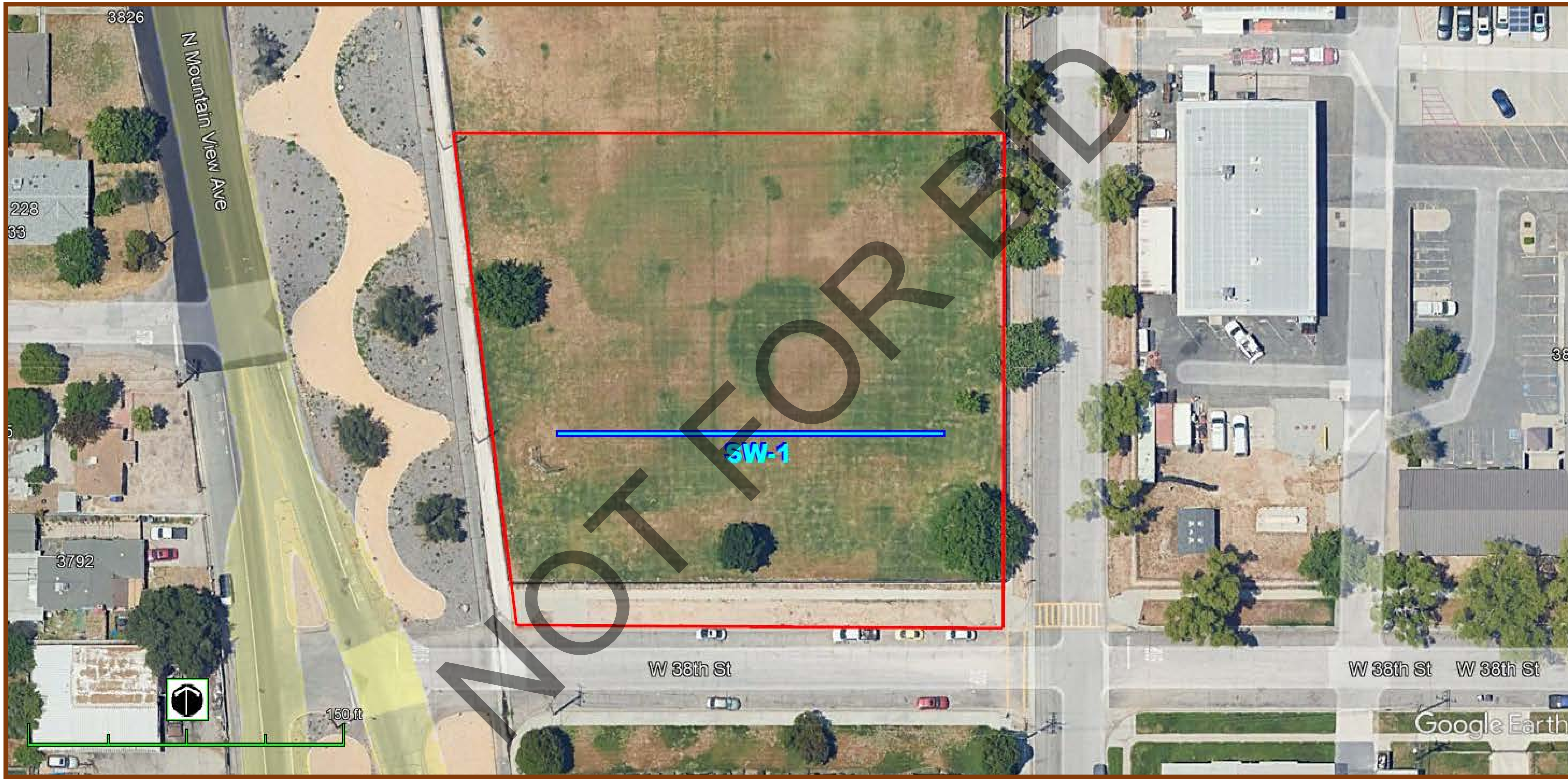


BASE MAP: Miller et al. (2001), U.S.G.S., Open File Report 01-131, Scale 1: 24,000, Site outlined in red.

PARTIAL LEGEND

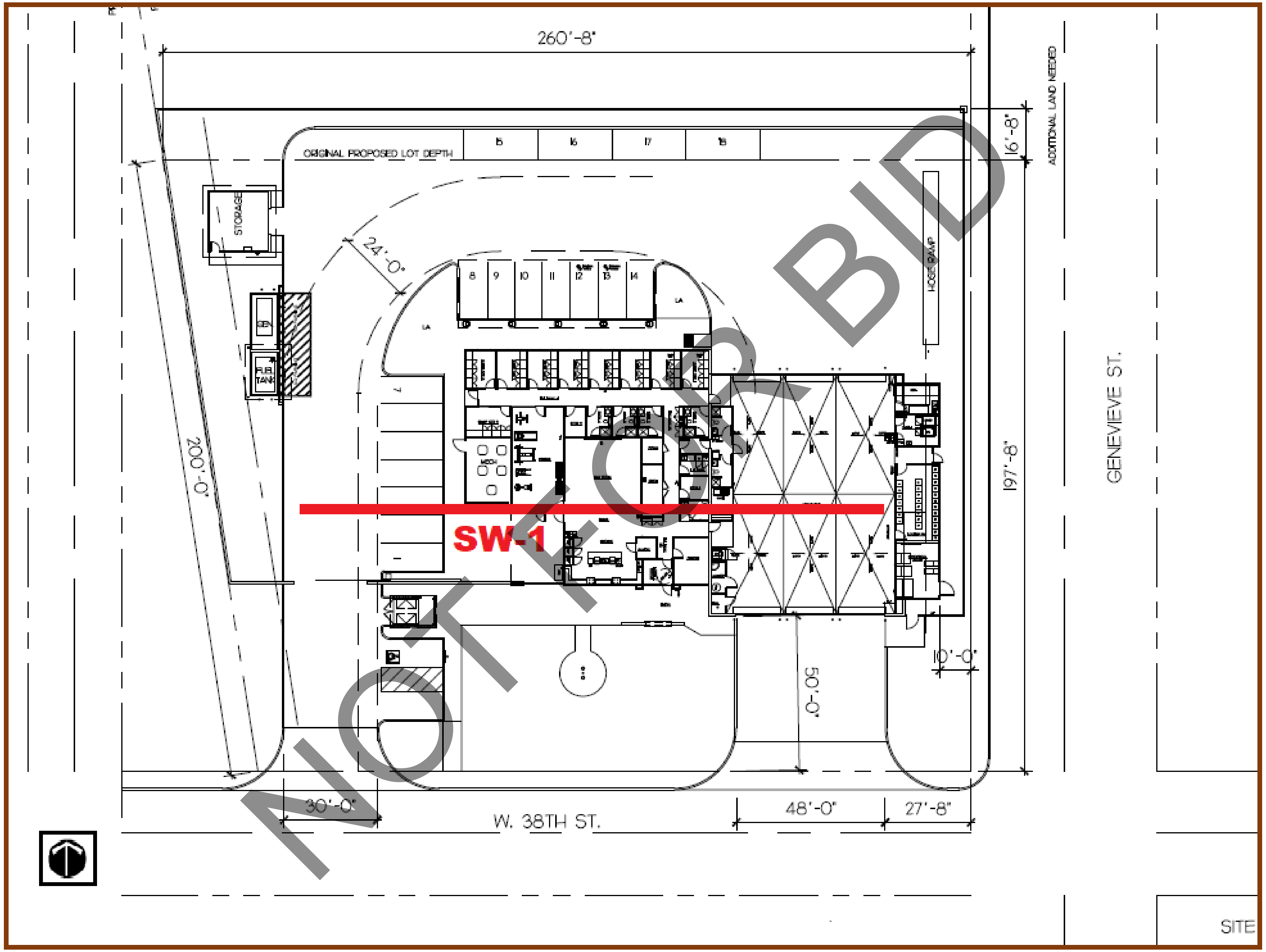
Qw	**YOUNG WASH DEPOSITS**	Unconsolidated to locally cemented sand, gravel and boulders (late Holocene).
Qyf_1	**YOUNG FAN DEPOSITS**	Slightly- to moderately-consolidated sand and pebble-boulder gravel (early Holocene and late Pleistocene).
Mzps	**PELONA SCHIST**	Muscovite-chlorite-albite-quartz schist, fine-grained (Mesozoic).
	GEOLOGIC CONTACT	Solid where located within 15± meters; dashed where located within 30± meters.
70	**FAULT**	Solid where located within 15± meters; dashed where located within 30± meters; dotted where concealed.

GOOGLE™ EARTH IMAGERY MAP



Base Map: Captured Google™ Earth (2024); Seismic shear-wave traverse SW-1 shown as blue line, approximate site boundary outlined in red.

SITE PLAN



BASE MAP: Provided "FS 227 Conceptual Site Plan" (Sheet A0.1, dated 6/12/24); prepared by STK Architecture, Inc., Temecula, California.

APPENDIX A

SHEAR-WAVE SURVEY

SHEAR-WAVE SURVEY

Methodology

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

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

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

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

Field Procedures

One shear-wave survey traverse (SW-1) was performed within proposed construction area, as approximated on Plates 1 and 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 21 separate seismic records being obtained for quality control purposes. The frequency spectrum data that was displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Reduction

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

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

Summary of Data Analysis

Data acquisition went very smoothly and the quality was considered to be good. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area is **1,075.1** feet per second (327.7 meters/second) as shown on the shear-wave model for Seismic Line SW-1, as presented within this appendix. This average velocity classifies the underlying soils to that of Site Class "**D**" ("Stiff Soil" profile), 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).

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

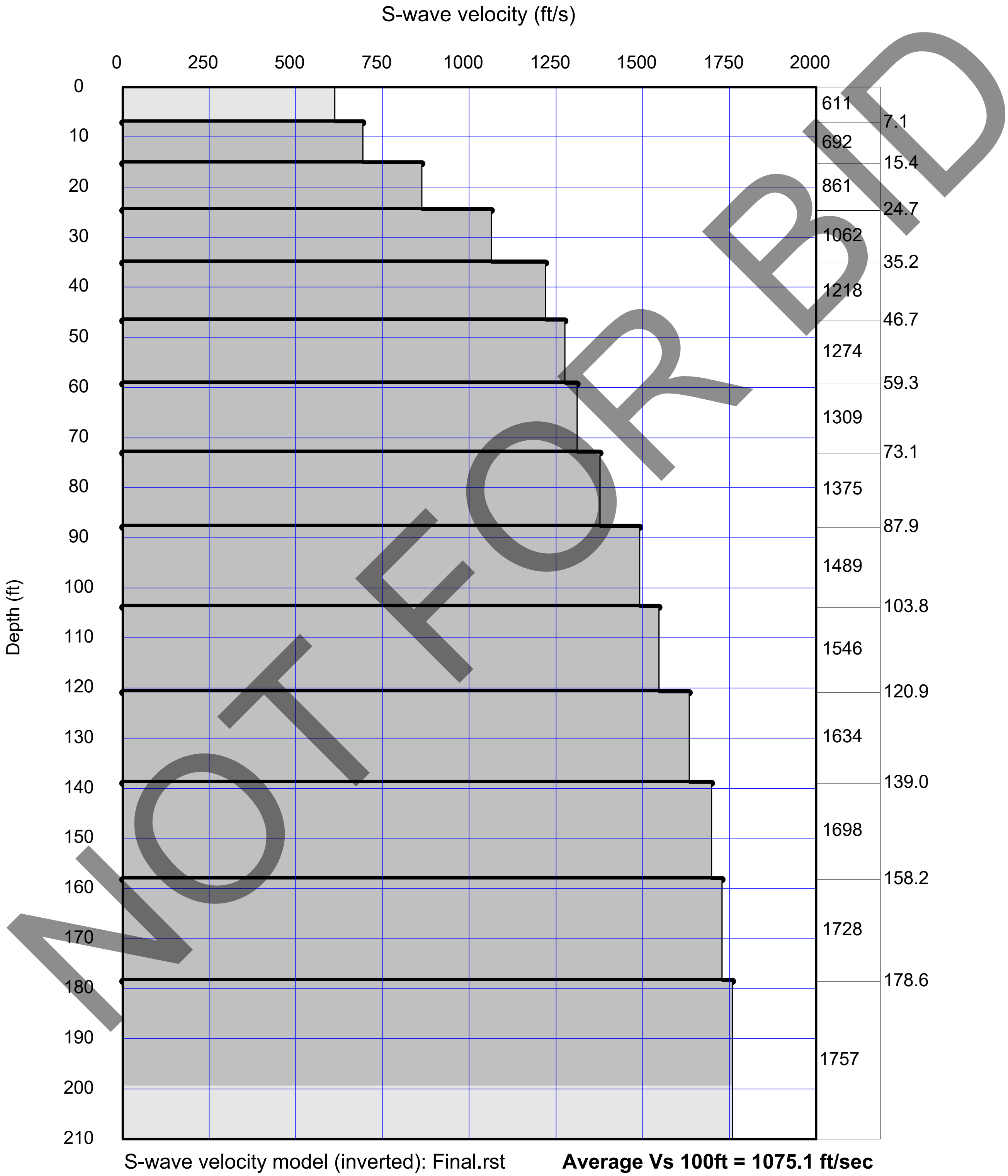
Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the 200-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

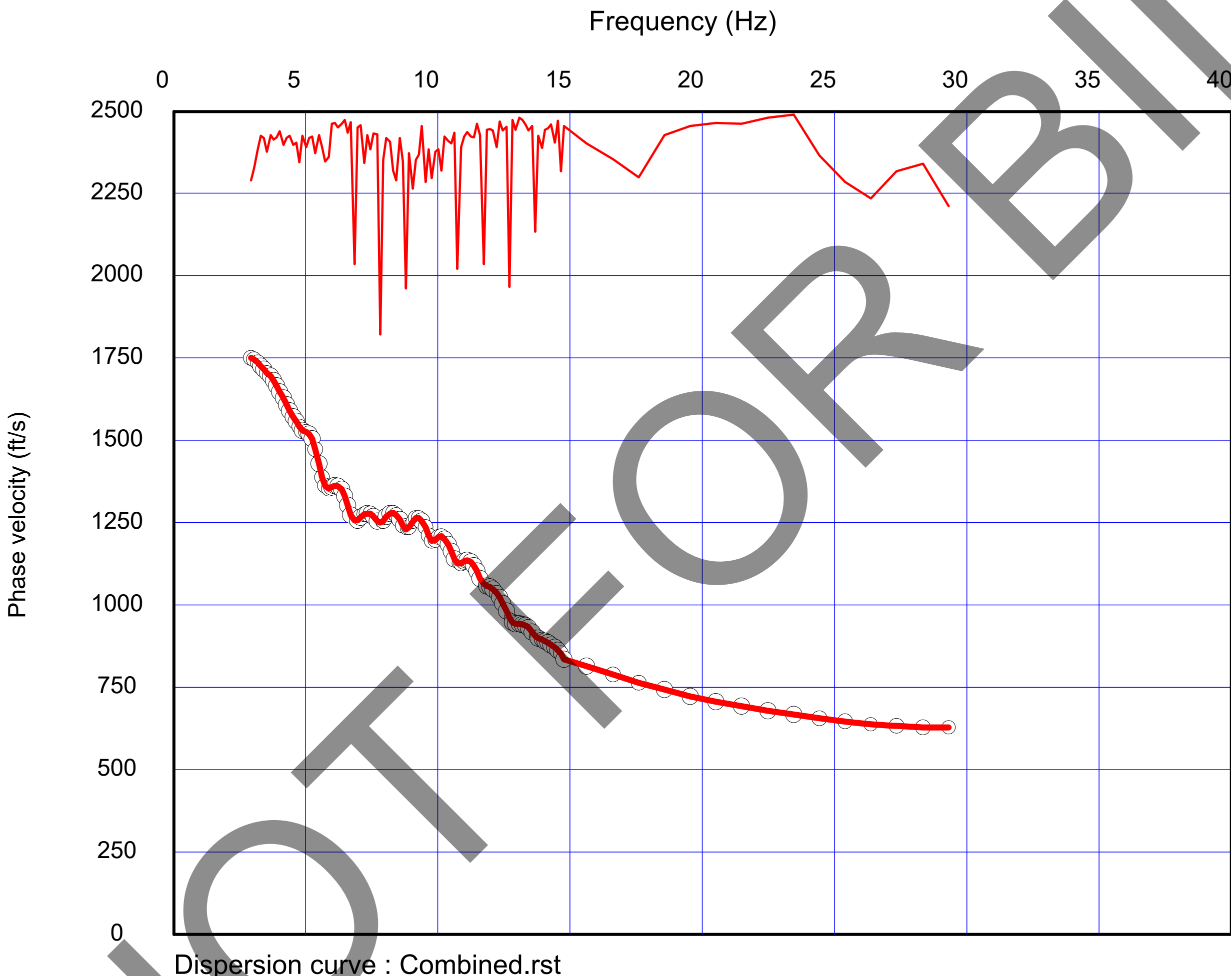
View looking west along Seismic Line SW-1.

View looking east along Seismic Line SW-1.

SEISMIC LINE SW-1
SHEAR-WAVE MODEL

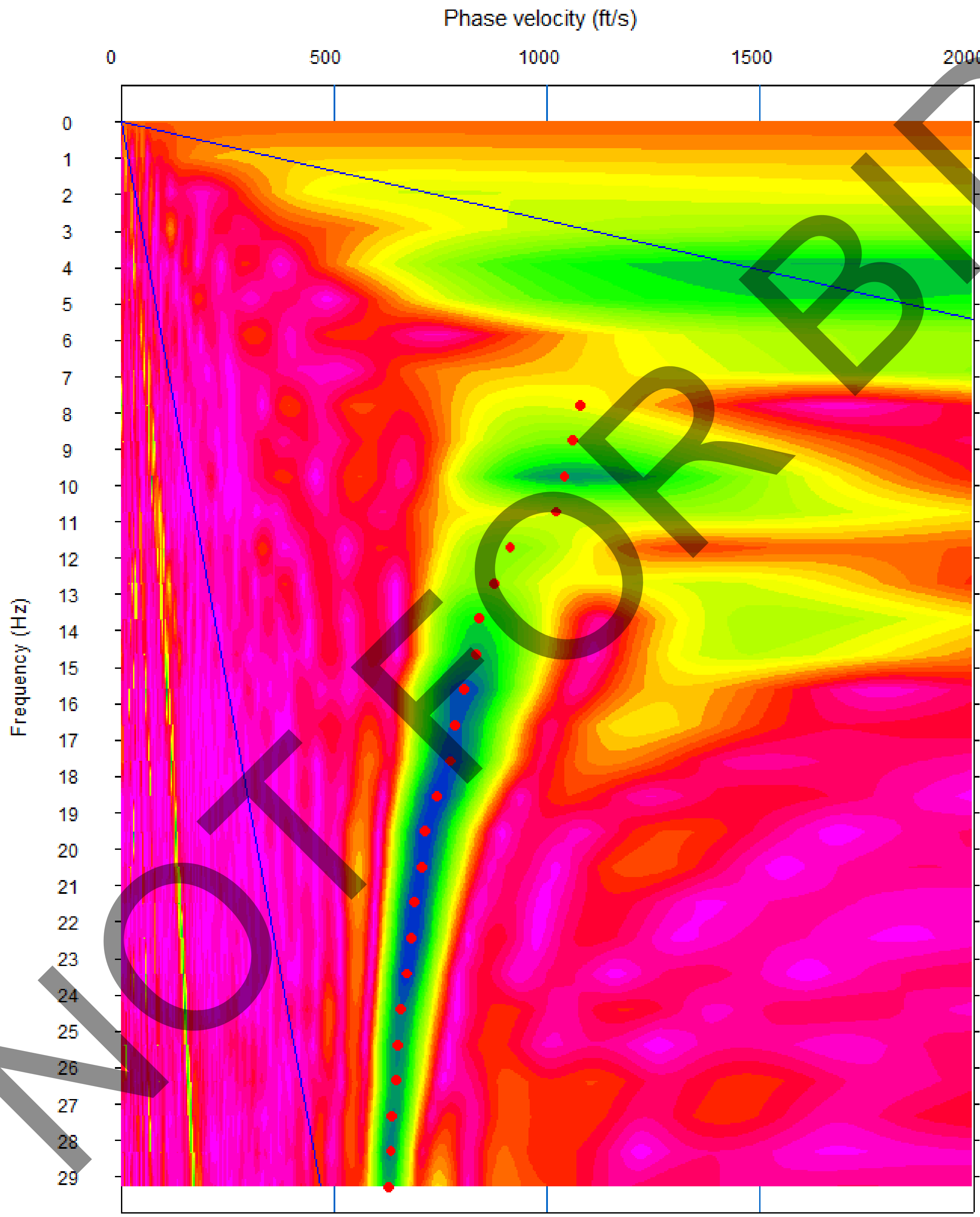


SEISMIC LINE SW-1



COMBINED DISPERSION CURVE

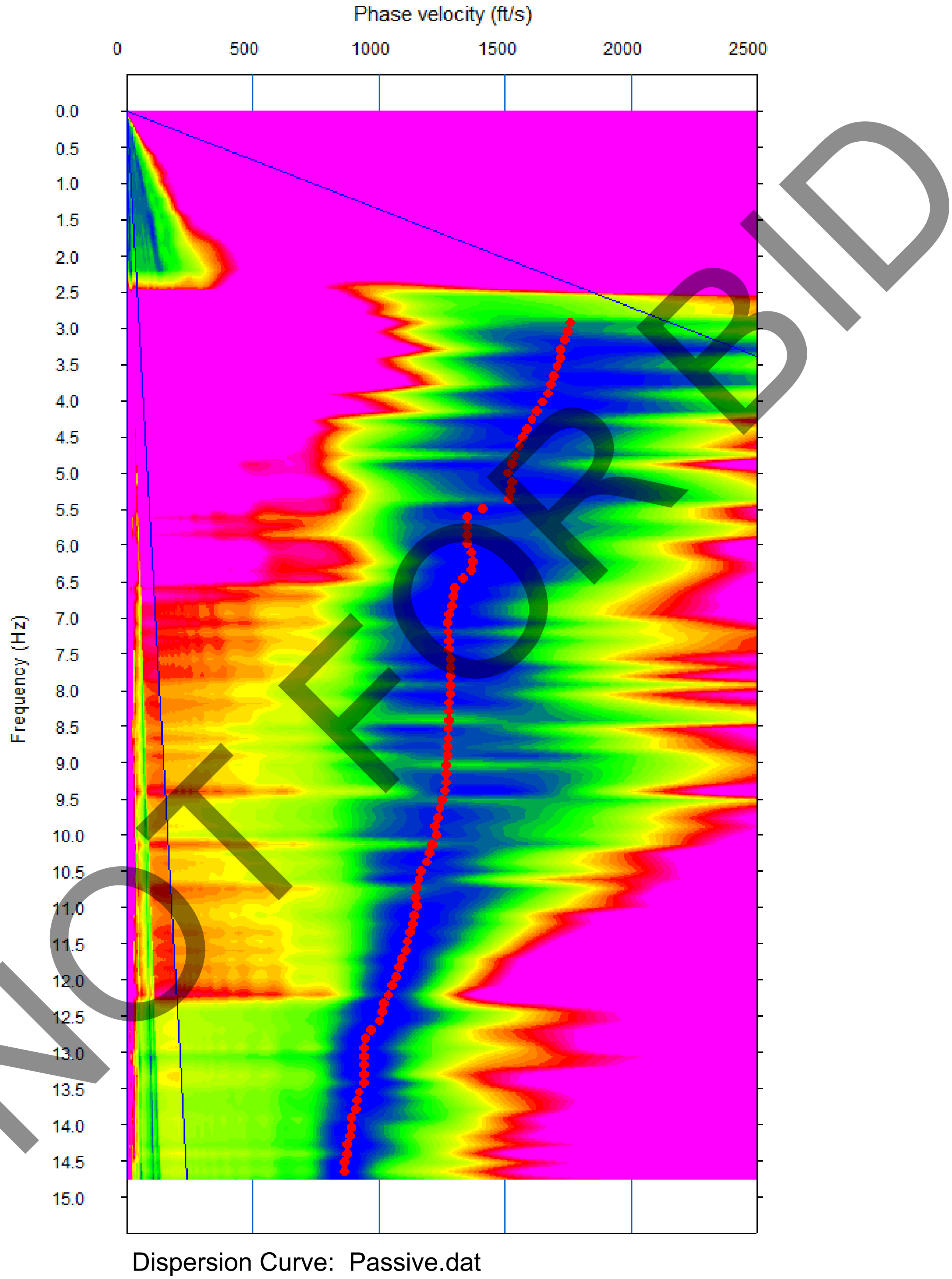
SEISMIC LINE SW-1



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



PASSIVE DISPERSION CURVE

APPENDIX B

SITE-SPECIFIC GROUND MOTION ANALYSIS

SITE-SPECIFIC GROUND MOTION ANALYSIS

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

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

  Based on maps prepared by the U.S.G.S (Risk-Adjusted Maximum Considered Earthquake (MCE_R) Ground Motion Parameter for the Conterminous United States for the 0.2 and 1-second Spectral Response Acceleration (5% of Critical Damping; Site Class B/C), a value of **2.506g** for the 0.2 second period (S_S) and **1.002g** 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 1,075.1 feet/second (327.7 meters/second), 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.

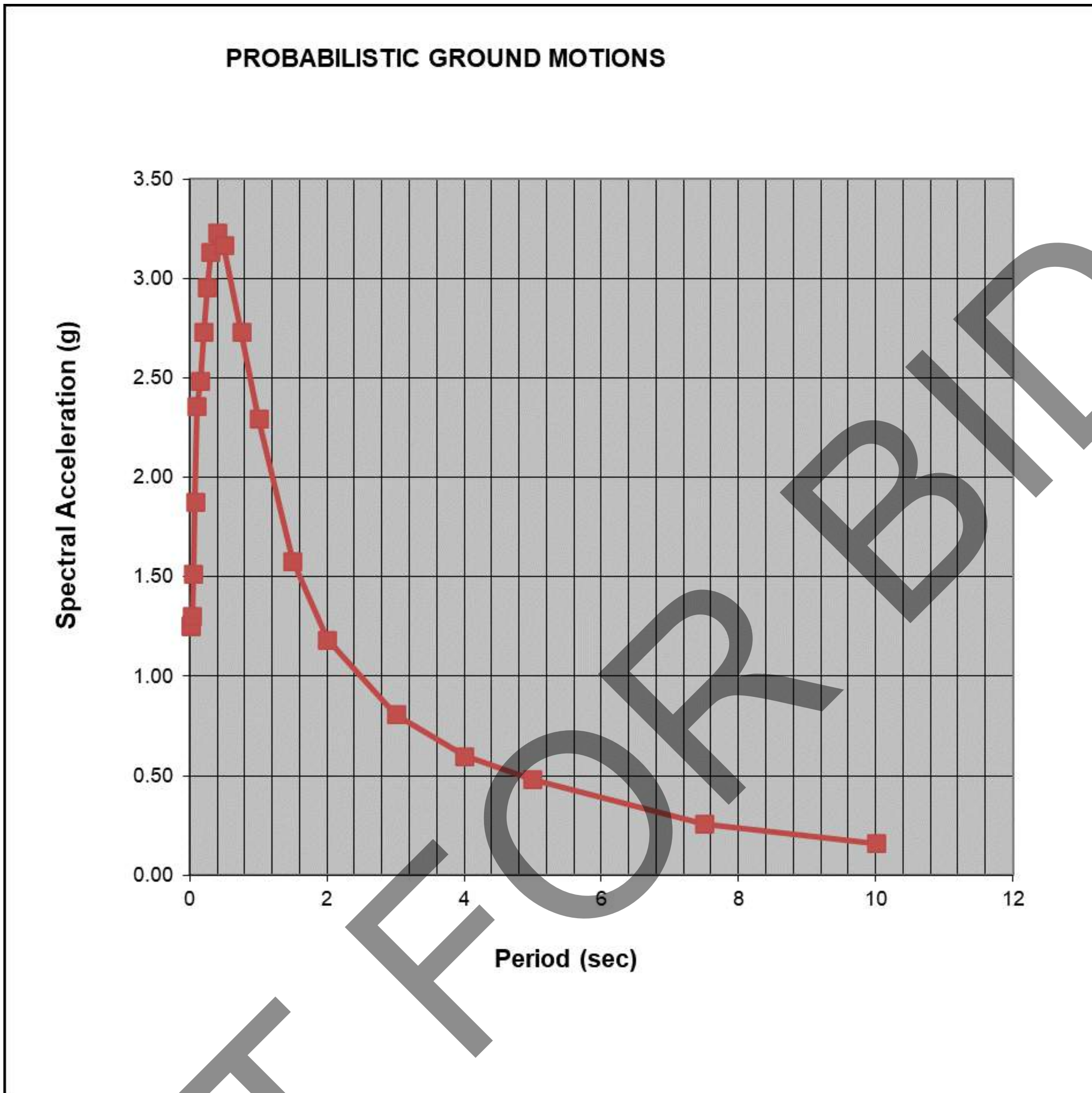
- **Site Coefficients (CBC 1613A.2.3)**-

  Based on CBC Tables 1613A.2.3(1) and 1613A.2.3(2), the site coefficient $\mathbf{F_a = 1.2}$ and $\mathbf{F_v = 1.7}$, respectively.

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

  Per Section 21.2.1, the probabilistic MCE spectral accelerations shall be taken as the spectral response accelerations in the direction of maximum response represented by a five percent damped acceleration response spectrum that is expected to achieve a one percent probability of collapse within a 50-year period.

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



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

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

  Based on our review of the Fault Section Database within the Uniform California Earthquake Rupture Forecast (UCERF 3; Field et al., 2013), published geologic data, and based on the length (combined segments) and maximum magnitude of the San Andreas Fault Zone (southern section) located 1.8 kilometers to the northeast, a moment magnitude (M_W) used for this fault was 8.1.

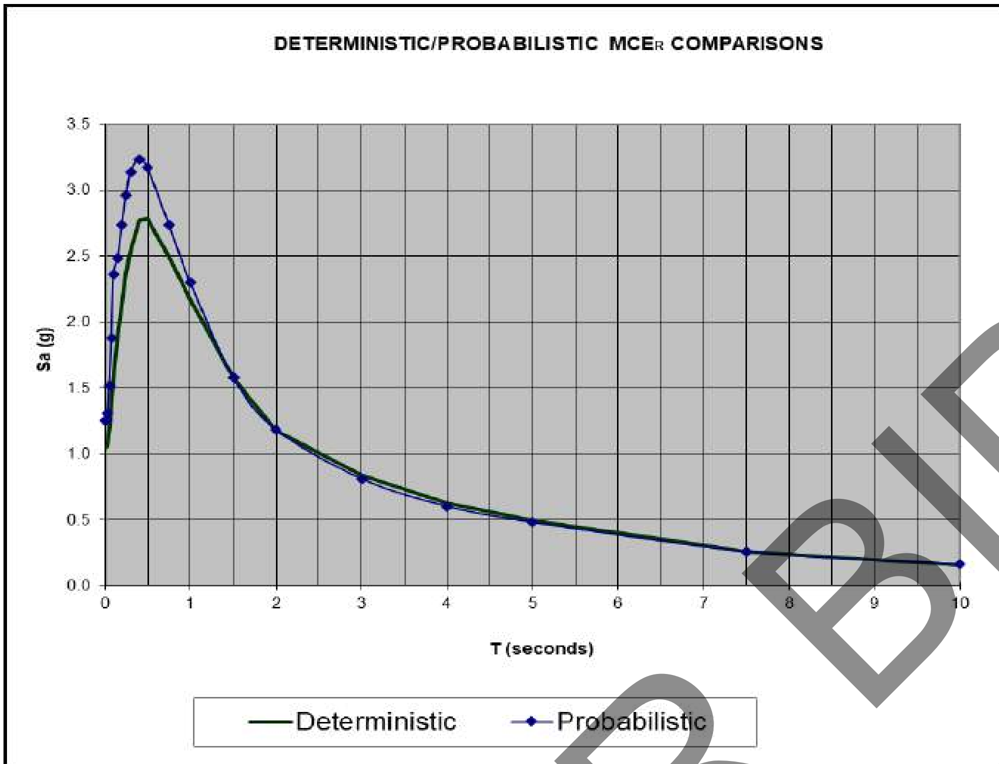
- **Site Specific MCE_R (ASCE 7 Section 21.2.3)**-

  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.

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

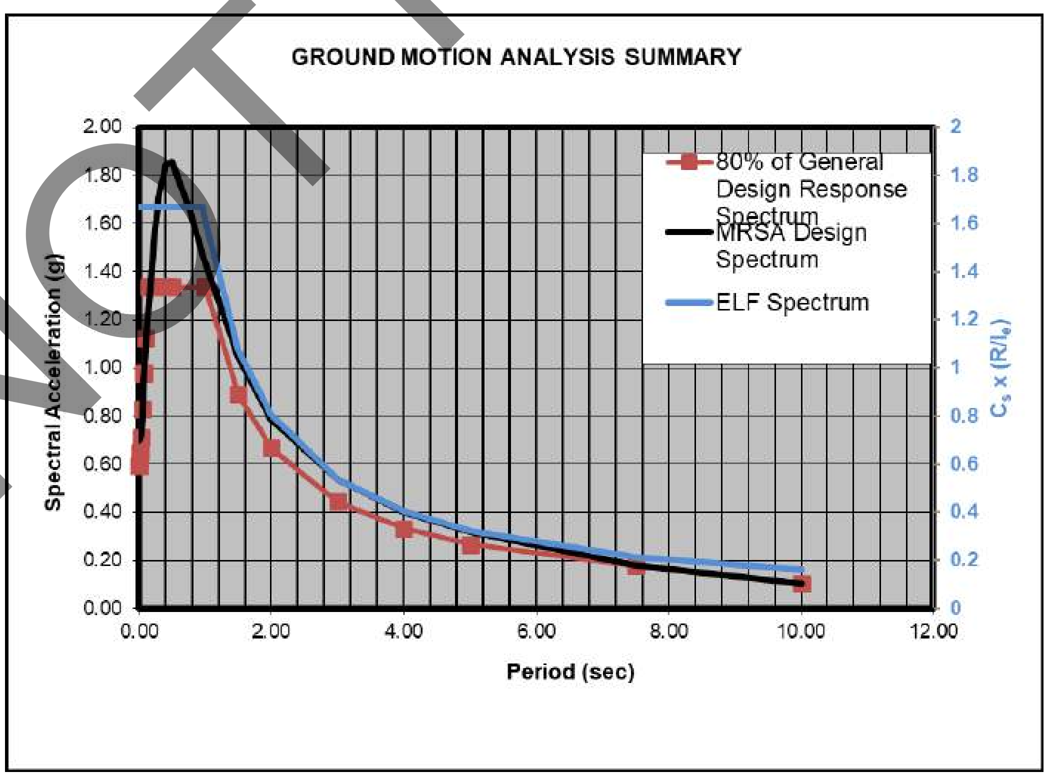
Period	Deterministic	Probabilistic	Lower Value (Site Specific MCE_R)	Governing Method
T	MCE_R	MCE_R		
0.010	1.05	1.25	1.05	Deterministic Governs
0.020	1.06	1.26	1.06	Deterministic Governs
0.030	1.09	1.31	1.09	Deterministic Governs
0.050	1.21	1.52	1.21	Deterministic Governs
0.075	1.42	1.88	1.42	Deterministic Governs
0.100	1.61	2.36	1.61	Deterministic Governs
0.150	1.90	2.49	1.90	Deterministic Governs
0.200	2.13	2.73	2.13	Deterministic Governs
0.250	2.37	2.96	2.37	Deterministic Governs
0.300	2.56	3.13	2.56	Deterministic Governs
0.400	2.77	3.23	2.77	Deterministic Governs
0.500	2.78	3.17	2.78	Deterministic Governs
0.750	2.49	2.73	2.49	Deterministic Governs
1.000	2.16	2.30	2.16	Deterministic Governs
1.500	1.57	1.58	1.57	Deterministic Governs
2.000	1.18	1.18	1.18	Deterministic Governs
3.000	0.84	0.81	0.81	Probabilistic Governs
4.000	0.63	0.60	0.60	Probabilistic Governs
5.000	0.49	0.48	0.48	Probabilistic Governs
7.500	0.26	0.26	0.26	Deterministic Governs
10.000	0.15	0.16	0.15	Deterministic Governs

These are plotted in the following diagram:



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

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



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

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

- **Site Specific Design Parameters** -

For the 0.2 second period (S_{DS}), the maximum average acceleration for any period exceeding 0.2 seconds was 1.86g occurring at T=0.50 seconds. This was multiplied by 0.9 to produce a value of 1.67g making this the applicable value. A value of 1.62g was calculated for S_{D1} at a period of 1 second (ASCE 7-16, 21.4). For the MCE_R 0.2 second period, a value of 2.506g (S_{MS}) was computed, along with a value of 2.429g (S_{M1}) for the MCE_R 1.0 second period was also calculated (ASCE 7-16, 21.2.3).

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

The probabilistic geometric mean peak ground acceleration (2 percent probability of exceedance within a 50-year period) was calculated as 1.24g. The deterministic geometric mean peak ground acceleration (largest 84^{th} percentile geometric mean peak ground acceleration for characteristic earthquakes on all known active faults within the site region) was calculated as 0.95g. The site-specific MCE_G peak ground acceleration was calculated to be **0.95g**, which was determined by using the lesser of the probabilistic (1.24g) or the deterministic (0.95g) geometric mean peak ground accelerations, but not taken as less than 80 percent of PGA_M (i.e., 1.14g x 0.80 = 0.92g).

SEISMIC DESIGN PARAMETERS SUMMARY

Project: San Bernardino County Fire Station #227 Lattitude: 34.1601
Project #: 244073-1 Longitude: -117.2866
Date: 7/14/2024

CALIFORNIA BUILDING CODE CHAPTER 16/ASCE7-16

Mapped Acceleration Parameters per ASCE 7-16, Chapter 22

S_s=	2.506	Figure 22-1
S_1=	1.002	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

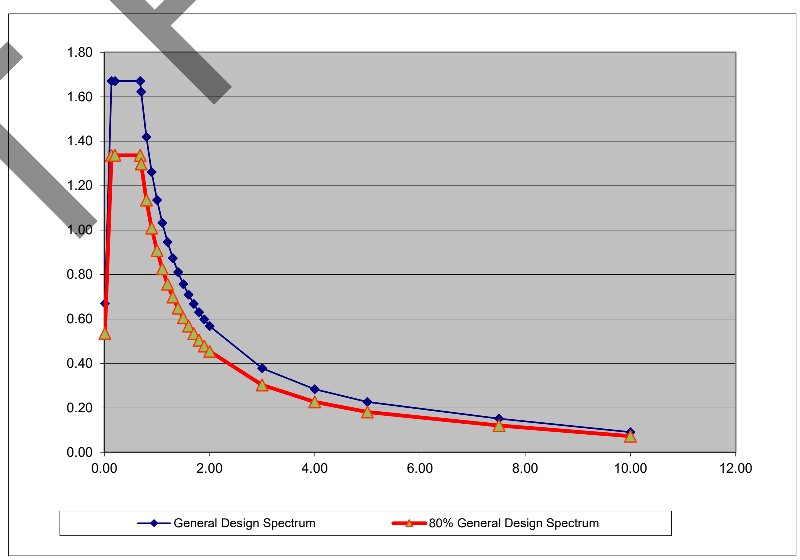
Mapped Design Spectral Response Acceleration Parameters

S_{MS}=	2.506	Equation 11.4-1	2.506	For Site Specific Analysis per ASCE7-16 21.3
S_{M1}=	1.703	Equation 11.4-2	2.505	For Site Specific Analysis per ASCE7-16 21.3

S_{DS}=	1.671	Equation 11.4-3
S_{D1}=	1.136	Equation 11.4-4

Period (T)	Sa (ASCE7-16 - 11.4.6)	80% General Design Spectrum
0.01	0.67	0.54
0.14	1.67	1.34
0.20	1.67	1.34
0.68	1.67	1.34
0.70	1.62	1.30
0.80	1.42	1.14
0.90	1.26	1.01
1.00	1.14	0.91
1.10	1.03	0.83
1.20	0.95	0.76
1.30	0.87	0.70
1.40	0.81	0.65
1.50	0.76	0.61
1.60	0.71	0.57
1.70	0.67	0.53
1.80	0.63	0.50
1.90	0.60	0.48
2.00	0.57	0.45
3.00	0.38	0.30
4.00	0.28	0.23
5.00	0.23	0.18
7.50	0.15	0.12
10.00	0.09	0.07

T_0=	0.136	sec	
T_S=	0.680	sec	
T_L=	8	sec	From Fig 22-12
PGA	1.04	g	
F_{PGA}=	1.1		From Table 11.8-1
C_{RS}=	0.905		Figure 22-17
C_{R1}=	0.884		Figure 22-18



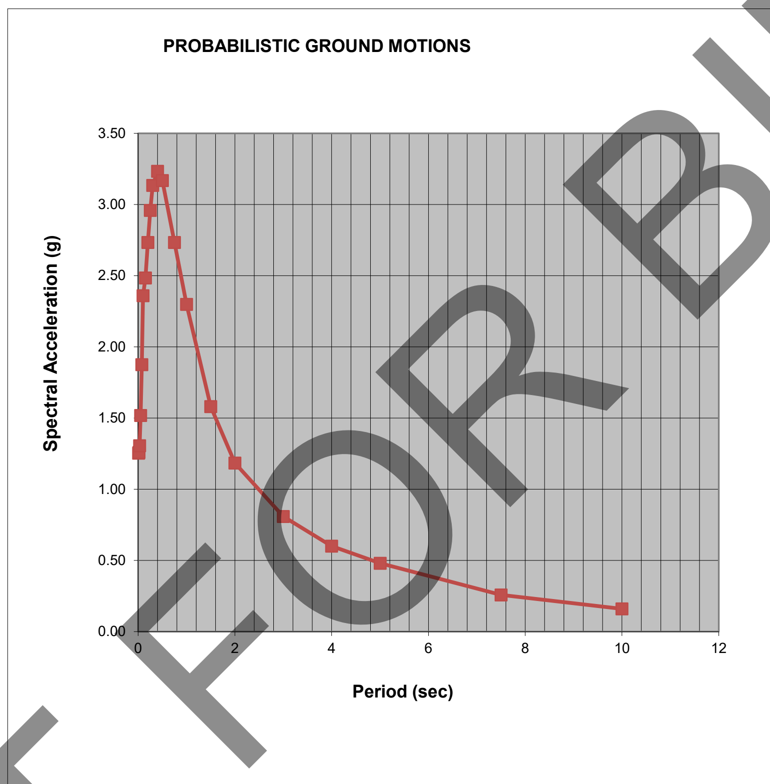
ASCE 7-16 - RISK-TARGETED MAXIMUM CONSIDERED EARTHQUAKE GROUND MOTION ANALYSIS

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 Relat
Earthquake Rupture Forecast - UCERF3 Mean, FM 3.1 & 3.2

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

T	Sa 2% in 50	MCER
0.01	1.39	1.25
0.02	1.39	1.26
0.03	1.44	1.31
0.05	1.68	1.52
0.08	2.07	1.88
0.10	2.37	2.36
0.15	2.75	2.49
0.20	3.02	2.73
0.25	3.27	2.96
0.30	3.47	3.13
0.40	3.59	3.23
0.50	3.53	3.17
0.75	3.07	2.73
1.00	2.60	2.30
1.50	1.79	1.58
2.00	1.34	1.18
3.00	0.92	0.81
4.00	0.68	0.60
5.00	0.54	0.48
7.50	0.29	0.26
10.00	0.18	0.16



S_s=	3.02	2.73
S_1=	2.60	2.30
PGA	1.24 g	

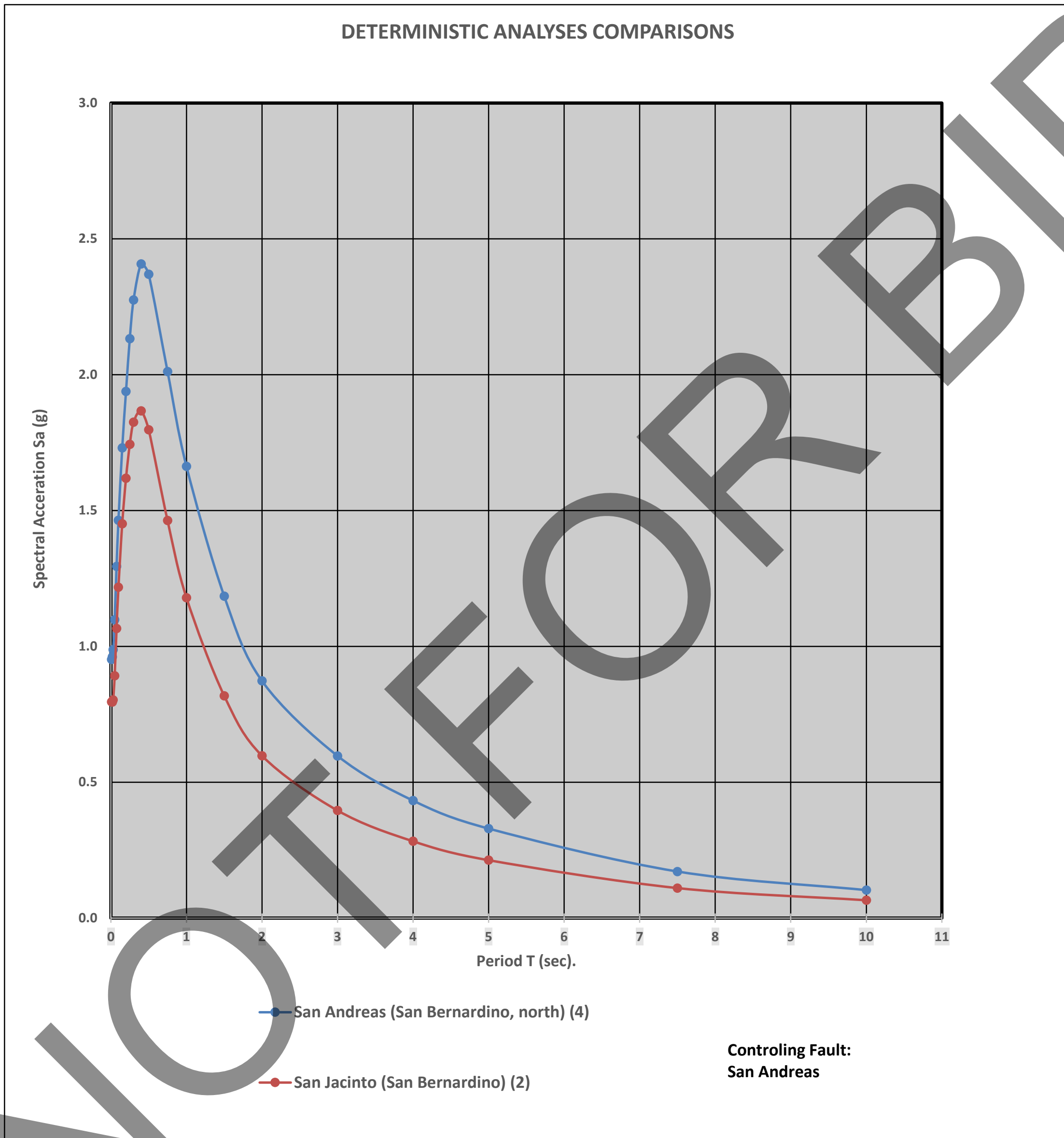
Risk Coefficients:			
C_{RS}	0.905	Figure 22-18	Get from Mapped Values
C_{R1}	0.884	Figure 22-19	
Fa=	1	Table 11.4-1	Per ASCE7-16 - 21.2.3
Is $Sa_{(max)}$<1.2XFa?		NO	If "YES", Probabilistic Spectrum prevails

DETERMINISTIC MCE per 21.2.2

Preliminary Assessment:

Fault	Distance (km)
San Andreas (San Bernardino, north) (4)	1.80
San Jacinto (San Bernardino) (2)	6.30

The Probalistic Analyses revealed 5 faults contributing more than 10% to the seismic hazard. These were considered in the Deterministic Analyses along with the Newport-Inglewood Fault.



Input Parameters		San Andreas (San Bernardino, north) (4)	San Jacinto (San Bernardino) (2)
Fault			
M	= Moment magnitude	8.1	7.8
R_{RUP}	= Closest distance to coseismic rupture (km)	1.8	6.3
R_{JB}	= Closest distance to surface projection of coseismic rupture (km)	1.8	6.3
Rx	= Horizontal distance to top edge of rupture measured perpendicular to strike (km)	1.8	6.3
U	= Unspecified Faulting Flag (Boore et.al.)	0	0
F_{RV}	= Reverse-faulting factor: 0 for strike slip, normal, normal-oblique; 1 for reverse, reverse-oblique and thrust	0	0
F_{NM}	= Normal-faulting factor: 0 for strike slip, reverse, reverse-oblique and thrust; 1 for normal and normal-oblique	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
Z_{TOR}	= Depth to top of coseismic rupture (km)	0	0
δ	= Average dip of rupture plane (degrees)	90	90
V_{S30}	= Average shear-wave velocity in top 30m of site profile	327.7	327.7
$F_{Measured}$		1	1
$Z_{1.0}$	= Depth to Shear Wave Velocity of 1.0 km/sec (km)	0.25	0.25
$Z_{2.5}$	= Depth to Shear Wave Velocity of 2.5 km/sec (km)	0.35	0.35
Site Class		D	D
W (km)	= Fault rupture width (km)	12.5	16.5
F_{AS}	= 0 for mainshock; 1 for aftershock	0	0
σ	=Standard Deviation	1	1

Deterministic Summary - Section 21.2.2 (Supplement 1)

T	San Andreas (San Bernardino, north) (4)	San Jacinto (San Bernardino) (2)	Maximum S_a (Average)	*Corrected* S_a (per ASCE7-16)*	*Scaled $S_{a(Average)}$*	Controlling Fault
0.010	0.95	0.80	0.95	1.05	1.05	San Andreas (San
0.020	0.96	0.79	0.96	1.06	1.06	San Andreas (San
0.030	0.99	0.80	0.99	1.09	1.09	San Andreas (San
0.050	1.10	0.89	1.10	1.21	1.21	San Andreas (San
0.075	1.29	1.07	1.29	1.42	1.42	San Andreas (San
0.100	1.46	1.22	1.46	1.61	1.61	San Andreas (San
0.150	1.73	1.45	1.73	1.90	1.90	San Andreas (San
0.200	1.94	1.62	1.94	2.13	2.13	San Andreas (San
0.250	2.13	1.74	2.13	2.37	2.37	San Andreas (San
0.300	2.28	1.83	2.28	2.56	2.56	San Andreas (San
0.400	2.41	1.87	2.41	2.77	2.77	San Andreas (San
0.500	2.37	1.80	2.37	2.78	2.78	San Andreas (San
0.750	2.01	1.46	2.01	2.49	2.49	San Andreas (San
1.000	1.66	1.18	1.66	2.16	2.16	San Andreas (San
1.500	1.19	0.82	1.19	1.57	1.57	San Andreas (San
2.000	0.87	0.60	0.87	1.18	1.18	San Andreas (San
3.000	0.60	0.40	0.60	0.84	0.84	San Andreas (San
4.000	0.43	0.28	0.43	0.63	0.63	San Andreas (San
5.000	0.33	0.21	0.33	0.49	0.49	San Andreas (San
7.500	0.17	0.11	0.17	0.26	0.26	San Andreas (San
10.000	0.10	0.07	0.10	0.15	0.15	San Andreas (San
PGA	0.95	0.76	0.95		0.95	g

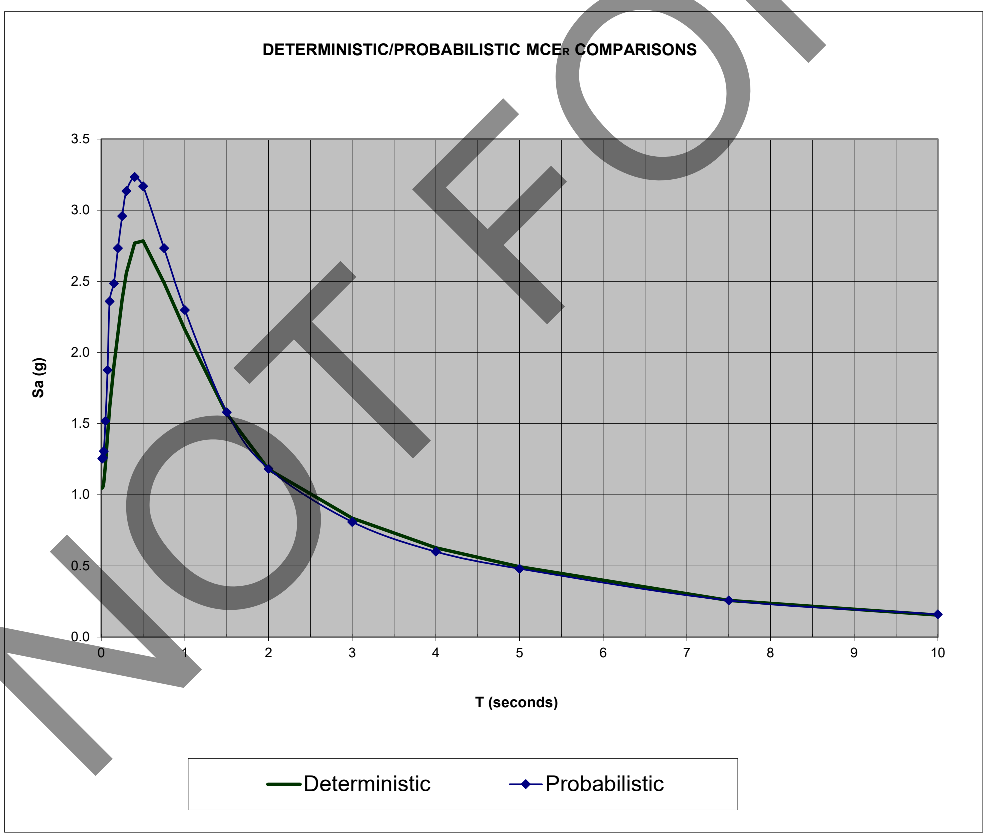
Max Sa=	2.78	
Fa =	1.00	Per ASCE7-16 21.2.2
1.5XFa=	1.5	
Scaling Factor=	1.00	

* 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 Relat

Period T	Deterministic MCE_R	Probabilistic MCE_R	Lower Value (Site Specific $MCE_{R)}$	Governing Method
0.010	1.05	1.25	1.05	Deterministic Governs
0.020	1.06	1.26	1.06	Deterministic Governs
0.030	1.09	1.31	1.09	Deterministic Governs
0.050	1.21	1.52	1.21	Deterministic Governs
0.075	1.42	1.88	1.42	Deterministic Governs
0.100	1.61	2.36	1.61	Deterministic Governs
0.150	1.90	2.49	1.90	Deterministic Governs
0.200	2.13	2.73	2.13	Deterministic Governs
0.250	2.37	2.96	2.37	Deterministic Governs
0.300	2.56	3.13	2.56	Deterministic Governs
0.400	2.77	3.23	2.77	Deterministic Governs
0.500	2.78	3.17	2.78	Deterministic Governs
0.750	2.49	2.73	2.49	Deterministic Governs
1.000	2.16	2.30	2.16	Deterministic Governs
1.500	1.57	1.58	1.57	Deterministic Governs
2.000	1.18	1.18	1.18	Deterministic Governs
3.000	0.84	0.81	0.81	ProbabilisticGoverns
4.000	0.63	0.60	0.60	ProbabilisticGoverns
5.000	0.49	0.48	0.48	ProbabilisticGoverns
7.500	0.26	0.26	0.26	Deterministic Governs
10.000	0.15	0.16	0.15	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	0.70	0.57	0.70	
0.02	0.71	0.61	0.71	
0.03	0.72	0.65	0.72	
0.05	0.81	0.74	0.81	
0.08	0.95	0.84	0.95	
0.10	1.07	0.94	1.07	
0.15	1.27	1.14	1.27	
0.20	1.42	1.34	1.42	
0.25	1.58	1.34	1.58	
0.30	1.71	1.34	1.71	
0.40	1.85	1.34	1.85	
0.50	1.86	1.34	1.86	
0.75	1.66	1.34	1.66	
1.00	1.44	1.34	1.44	1.44
1.50	1.05	0.89	1.05	1.57
2.00	0.79	0.67	0.79	1.57
3.00	0.54	0.45	0.54	1.62
4.00	0.40	0.33	0.40	1.60
5.00	0.32	0.27	0.32	1.60
7.50	0.17	0.18	0.18	
10.00	0.10	0.11	0.11	

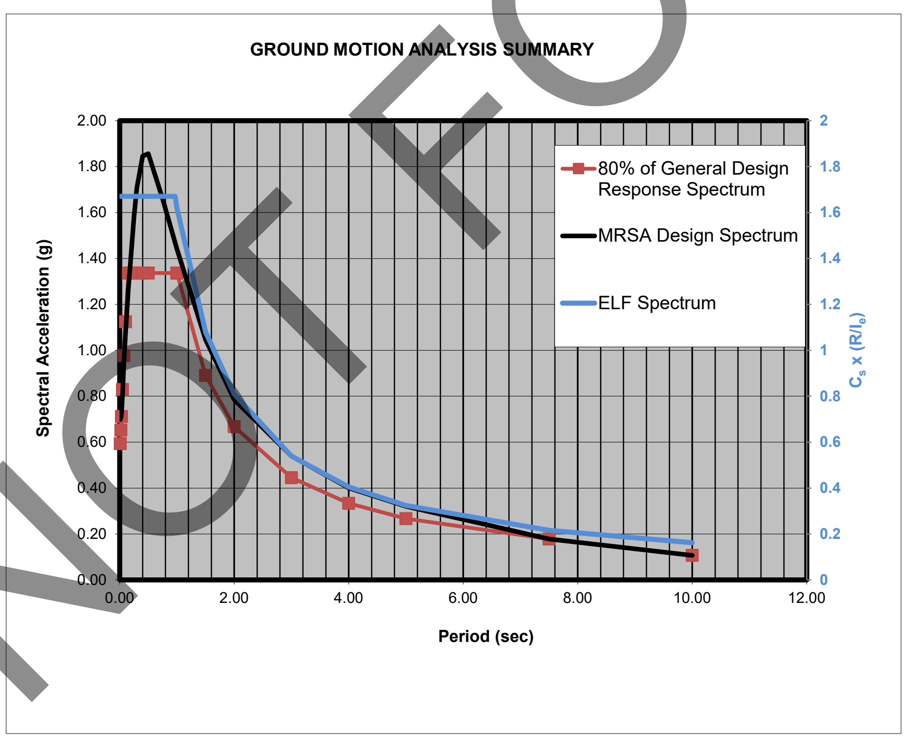
Highest value of S_a for any period exceeding 0.2 sec.=	1.86
90%of Highest Value =	1.67
80% 0f Mapped S_{DS}=	1.34
Maximum TXSa from T=1s-5s =	1.62
80% of Mapped S_{D1}=	0.91

S_{DS}=	1.67
S_{D1}=	1.62
Ts =	0.97

S_{MS}=	2.506
S_{M1}=	2.429

PGA Determination:

Site Coefficient F_{PGA}=	1.1	
Mapped PGA=	1.04	Figure 22-7
PGA_M =	1.14	g
Deterministic PGA =	0.95	g
Probabilistic PGA =	1.24	g
Lesser of Deterministic/Probabilistic =	0.95	g
80% of $PGA_{M=}$	0.92	g
MCE_G PGA=	0.95	g



SUMMARY OF SITE SPECIFIC GROUND MOTION HAZARD ANALYSIS DATA

1	2	3		4	5	6	7	8	9	10	11	12
Period (sec)	Mapped MCE$_R$ Spectrum	Mapped Design Spectrum	**Period (sec)**	Risk Coefficient C$_R$	Scaled MCE$_R$ Deterministic Spectrum	Probabilistic MCE$_R$ Spectrum	Probabilistic w/Risk Coeffcicent C$_R$	84th Percentile Deterministic Spectrum	2/3 Site Specific MCE$_R$ Spectrum	80% of General Design Spectrum	Site Specific MCE$_R$ Spectrum	Design Response Spectrum
0.01	1.00	0.67	**0.01**	0.905	1.05	1.25	1.25	1.05	0.70	0.57	1.05	0.70
0.14	2.51	1.67	**0.02**	0.905	1.06	1.26	1.26	1.06	0.71	0.61	1.06	0.71
0.20	2.51	1.67	**0.03**	0.905	1.09	1.31	1.31	1.09	0.72	0.65	1.09	0.72
0.68	2.51	1.67	**0.05**	0.905	1.21	1.52	1.52	1.21	0.81	0.74	1.21	0.81
0.70	2.43	1.62	**0.08**	0.905	1.42	1.88	1.88	1.42	0.95	0.84	1.42	0.95
0.80	2.13	1.42	**0.10**	0.905	1.61	2.36	2.36	1.61	1.07	0.94	1.61	1.07
0.90	1.89	1.26	**0.15**	0.905	1.90	2.49	2.49	1.90	1.27	1.14	1.90	1.27
1.00	1.70	1.14	**0.20**	0.905	2.13	2.73	2.73	2.13	1.42	1.34	2.13	1.42
1.10	1.55	1.03	**0.25**	0.904	2.37	2.96	2.96	2.37	1.58	1.34	2.37	1.58
1.20	1.42	0.95	**0.30**	0.902	2.56	3.13	3.13	2.56	1.71	1.34	2.56	1.71
1.30	1.31	0.87	**0.40**	0.900	2.77	3.23	3.23	2.77	1.85	1.34	2.77	1.85
1.40	1.22	0.81	**0.50**	0.897	2.78	3.17	3.17	2.78	1.86	1.34	2.78	1.86
1.50	1.14	0.76	**0.75**	0.891	2.49	2.73	2.73	2.49	1.66	1.34	2.49	1.66
1.60	1.06	0.71	**1.00**	0.884	2.16	2.30	2.30	2.16	1.44	1.34	2.16	1.44
1.70	1.00	0.67	**1.50**	0.884	1.57	1.58	1.58	1.57	1.05	0.89	1.57	1.05
1.80	0.95	0.63	**2.00**	0.884	1.18	1.18	1.18	1.18	0.79	0.67	1.18	0.79
1.90	0.90	0.60	**3.00**	0.884	0.84	0.81	0.81	0.84	0.54	0.45	0.81	0.54
2.00	0.85	0.57	**4.00**	0.884	0.63	0.60	0.60	0.63	0.40	0.33	0.60	0.40
3.00	0.57	0.38	**5.00**	0.884	0.49	0.48	0.48	0.49	0.32	0.27	0.48	0.32
4.00	0.43	0.28	**7.50**	0.884	0.26	0.26	0.26	0.26	0.17	0.18	0.27	0.18
5.00	0.34	0.23	**10.00**	0.884	0.15	0.16	0.16	0.15	0.10	0.11	0.16	0.11
7.50	0.23	0.15										
10.00	0.14	0.09										

APPENDIX C

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April 22, 2025
Project No. S168-193

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

Attention: Tony Finaldi, Architect

Subject: Plan and Specification Review
San Bernardino County Fire Station No. 227

Reference: Geotechnical Investigation Report, Proposed Fire Station 227, NWC Genevieve Street N. and W. 38th Street, San Bernardino, California, prepared by Inland Foundation Engineering, Inc., dated August 20, 2024, Project No. S168-193

Dear Mr. Finaldi:

Inland Foundation Engineering, Inc. (IFE) has reviewed the following plans and specifications for conformance with the recommendations in the above-referenced geotechnical investigation report.

- Plans for San Bernardino County Fire Station #227, Project # 10.10.1202 (99 sheets), prepared by STK Architecture, Inc., dated December 2024, Project No. 374-193-24

- Technical Specifications, SBC Fire Department , Station No. 227 (521 pages), prepared by STK Architecture, Inc., dated March 2025

Our review indicates the plans and specifications have been prepared in accordance with our recommendations and we take no exception to their approval.

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

Sincerely,
INLAND FOUNDATION ENGINEERING, INC.

REGISTERED PROFESSIONAL ENGINEER
ALLEN D. EVANS
No. 2060
GEOTECHNICAL
STATE OF CALIFORNIA

ADE No. 2060

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

ADE:sd

REVIEWED FOR CODE COMPLIANCE

These plans and documents have been reviewed and found to be in compliance with the applicable code requirements of the jurisdiction. Issuance of a permit is recommended subject to approval by other departments and any noted conditions. The stamping of these plans shall not be held to permit or be an approval of any violation of applicable codes and standards to relieve the owner, design professional of record or contractor of compliance with the applicable codes and standards. Plan review of documents does not authorize construction to proceed in violation of any federal, state or local regulations.

BUREAU VERITAS NORTH AMERICA, INC.

SIGNATURE: Bureau Veritas

DATE: 5/30/2025