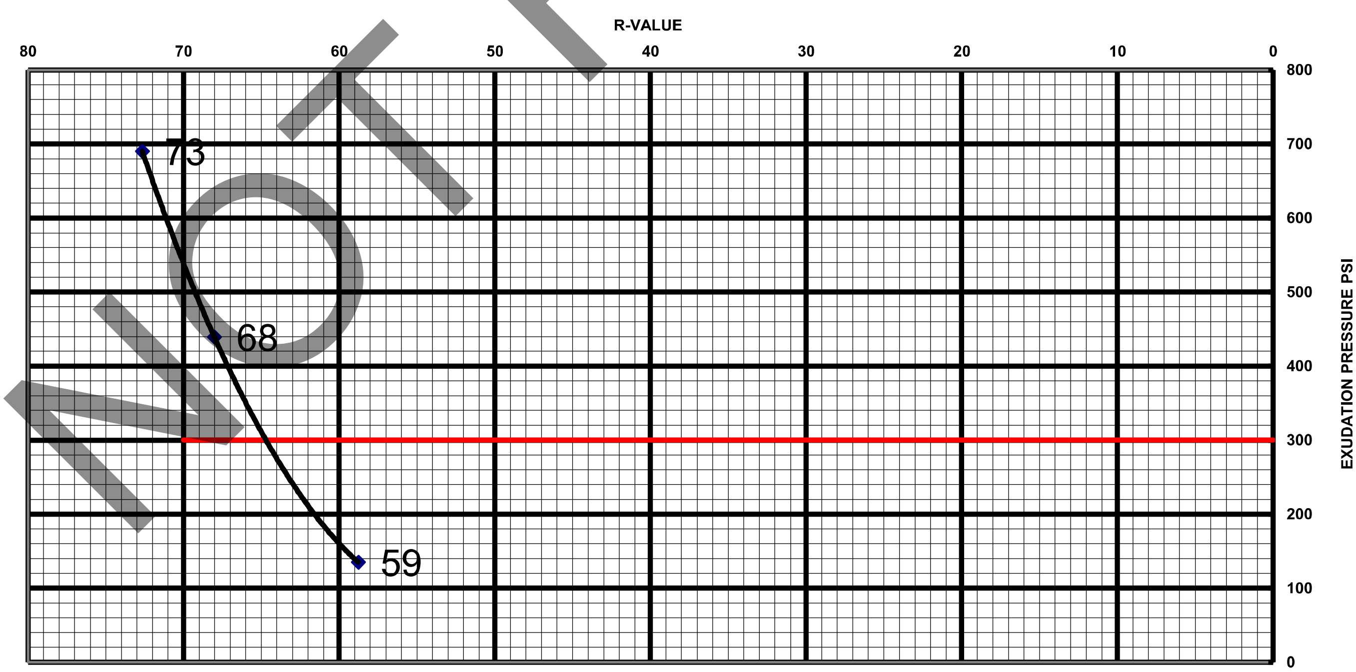
Job No. CB191148
Date. 8/8/2024

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).

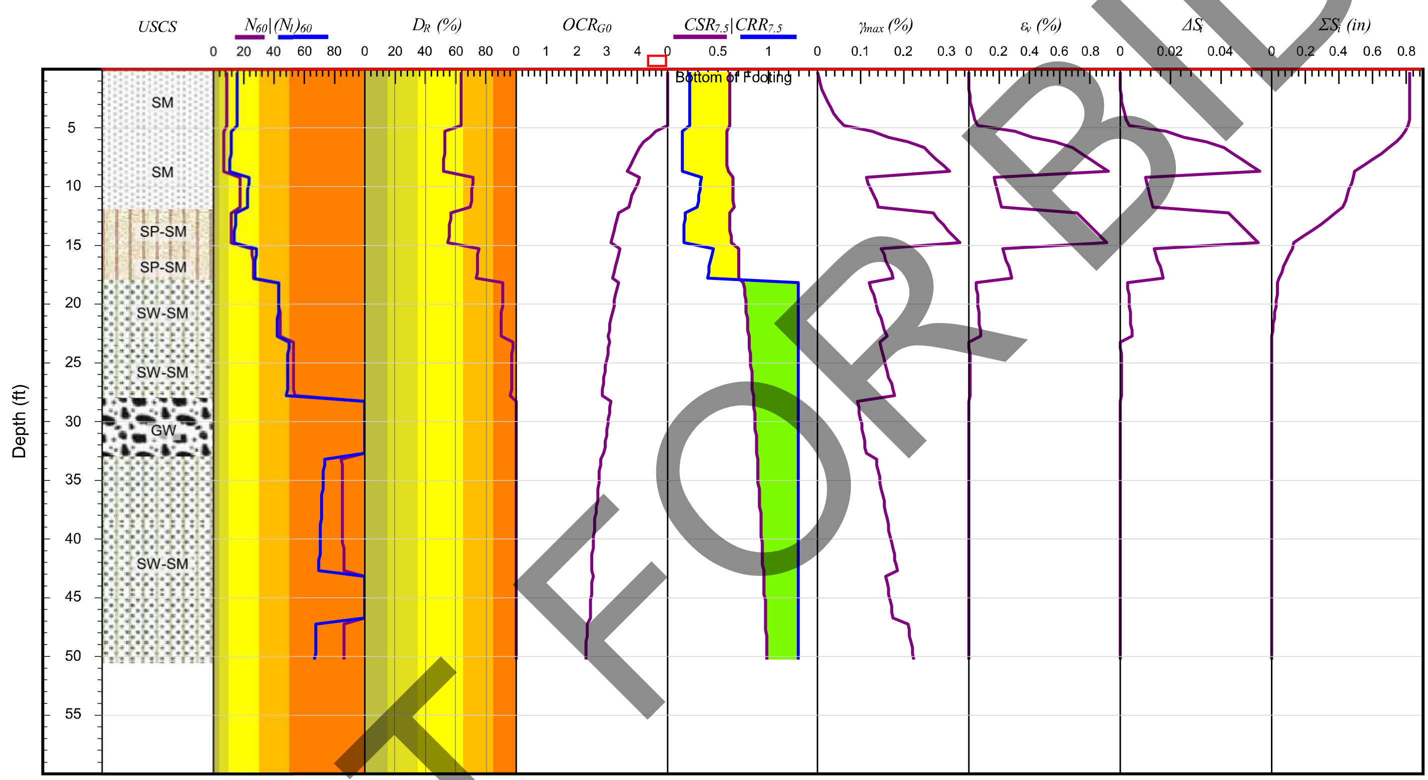
C_R = rod length correction factor. GeoSuite® applies a C_R factor for each layer in the soil profile using the values in Table 5.2 of the 1999 SCEC guidelines, and assuming a rod stick up length (above the ground surface) of 3 feet.

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

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

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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.

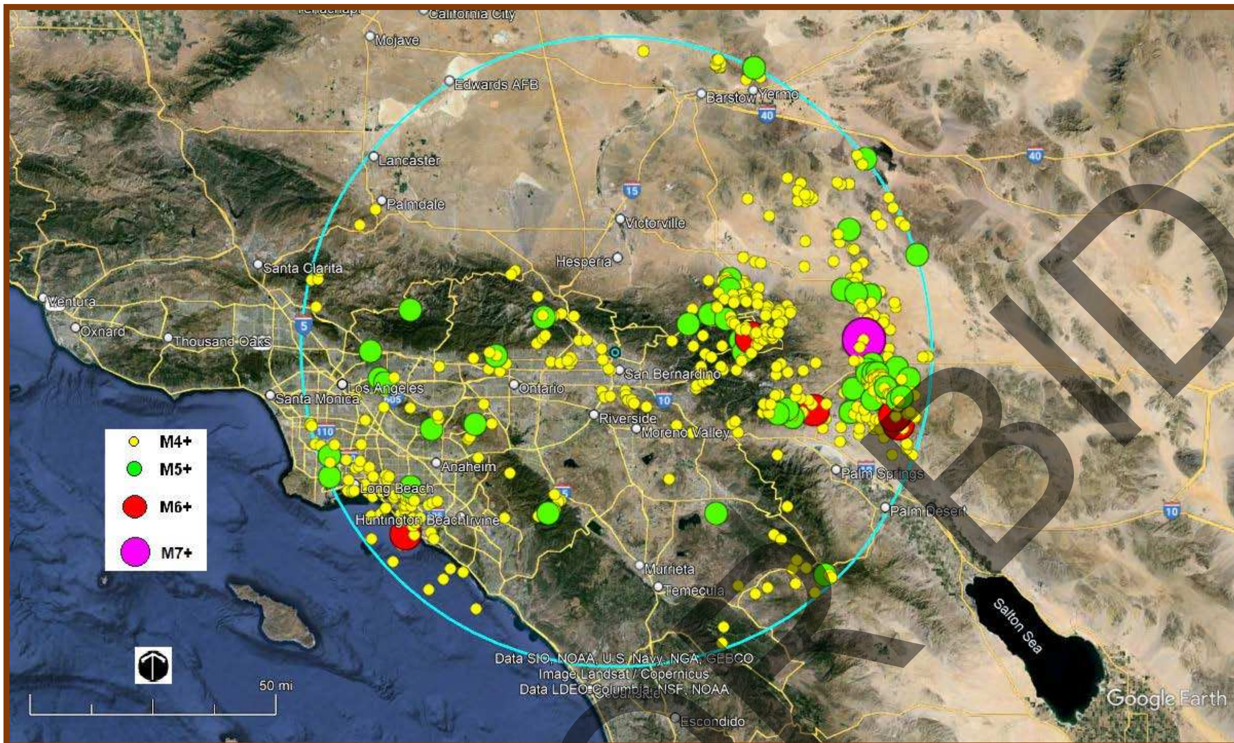


FIGURE 1- Earthquake Epicenter Map showing events of M4.0+ within a 100-kilometer radius.

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.

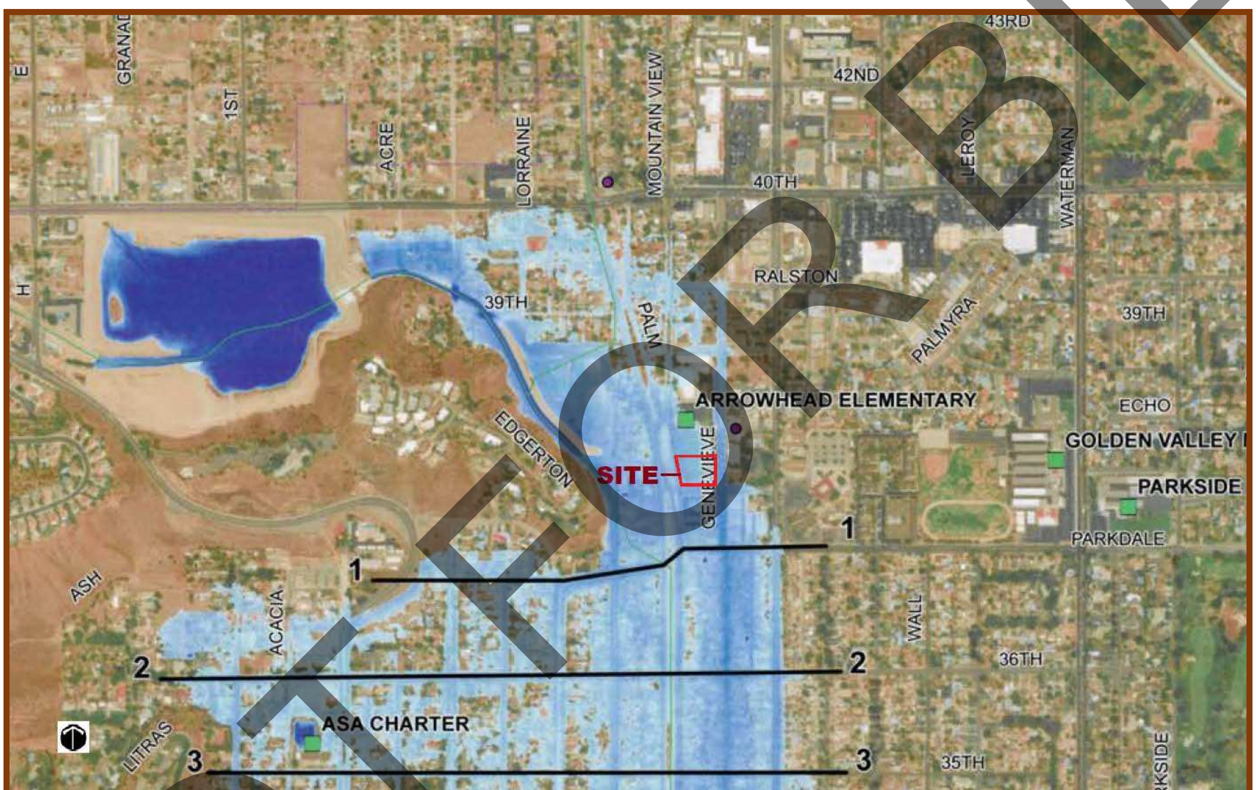


FIGURE 2- Dam Inundation Map (San Bernardino County, 2018); flooding shown as blue shading.

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.

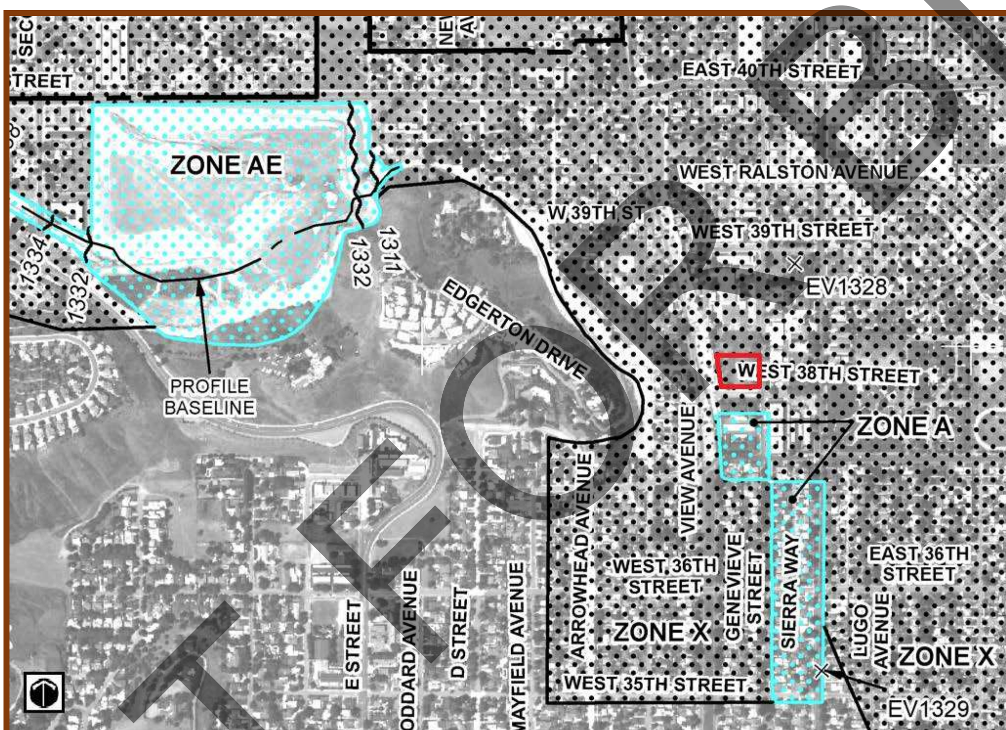


FIGURE 2- FEMA Flood Zone Map; Site boundary approximated by red outline.

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

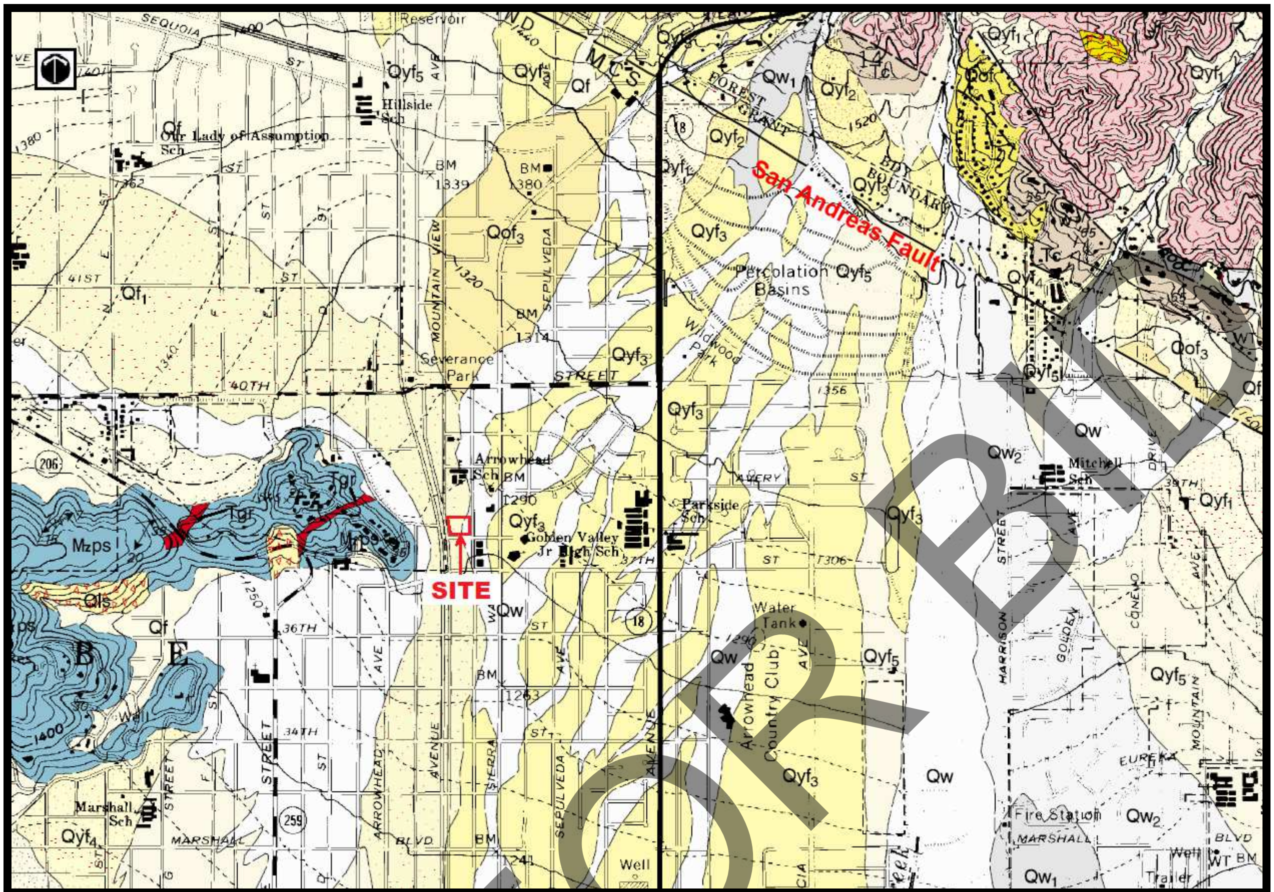
Our conclusions and recommendations are based on a review of available existing published geologic/seismic data and the provided site-specific subsurface exploratory boring logs. No subsurface exploration was performed by this firm for this evaluation. We make no warranty, either express or implied. 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

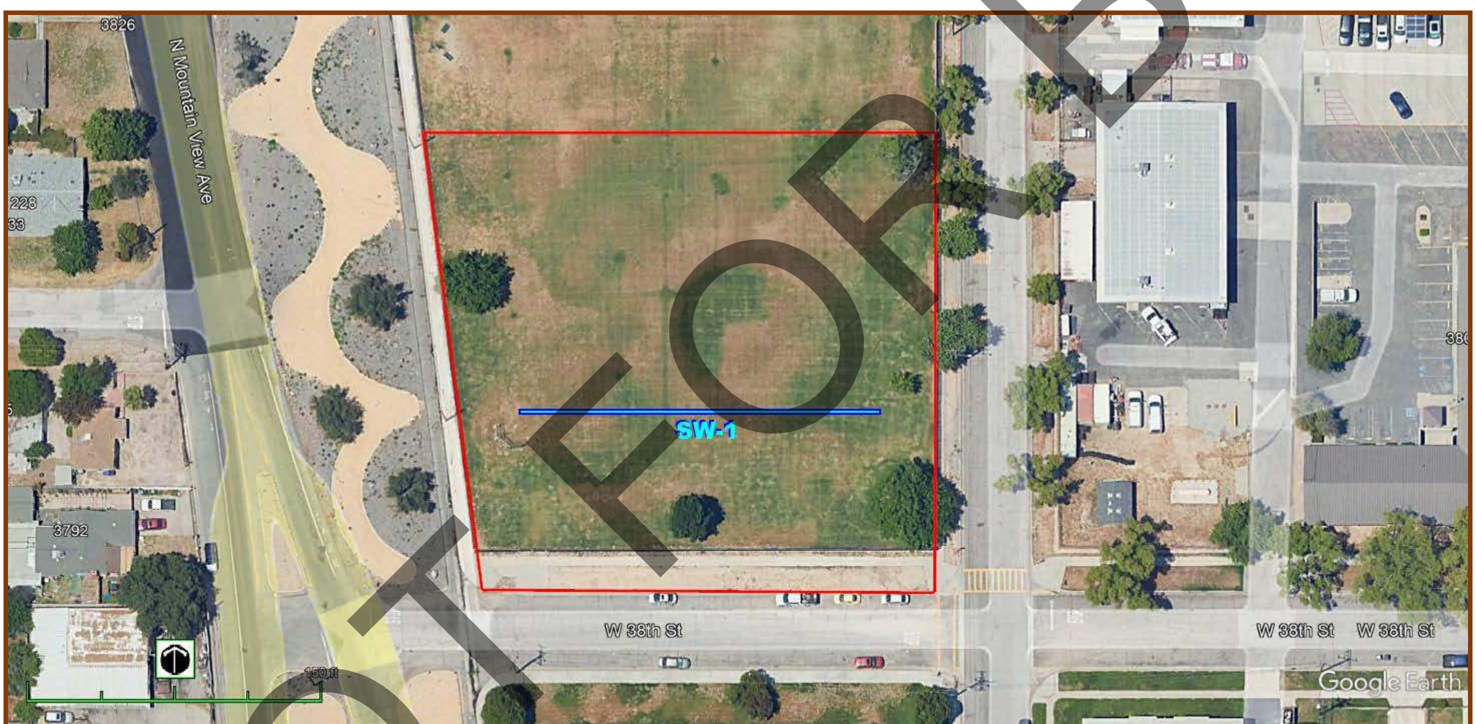


BASE MAP: Miller et al. (2001), U.S.G.S., Open File Report 01-131, Scale 1: 24,000, Site outlined in red.

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.

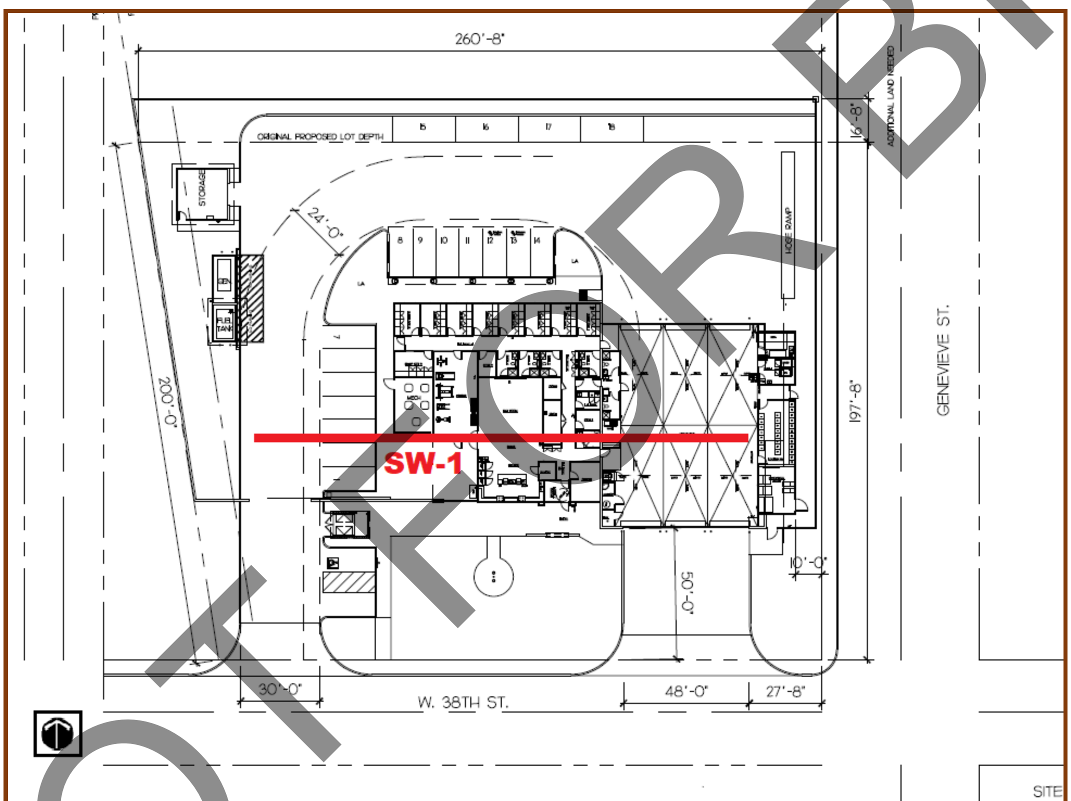
GOOGLE™ EARTH IMAGERY MAP



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

PROJECT NO. 244073-1 **PLATE 2**

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).

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

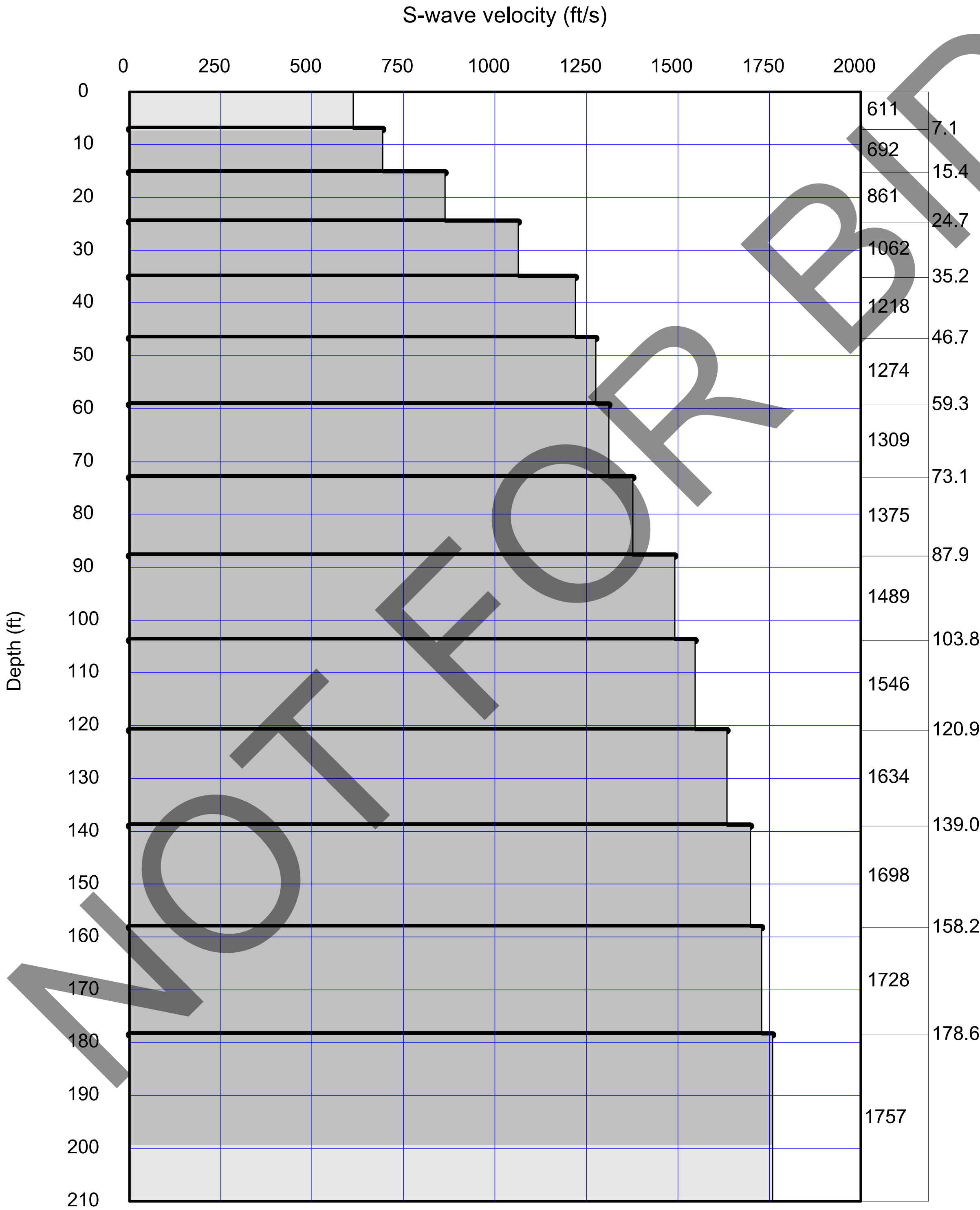
Where d1, d2, d3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n. The detailed shear-wave model displays these calculated layer boundaries/depths and associated velocities (feet/second) for the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

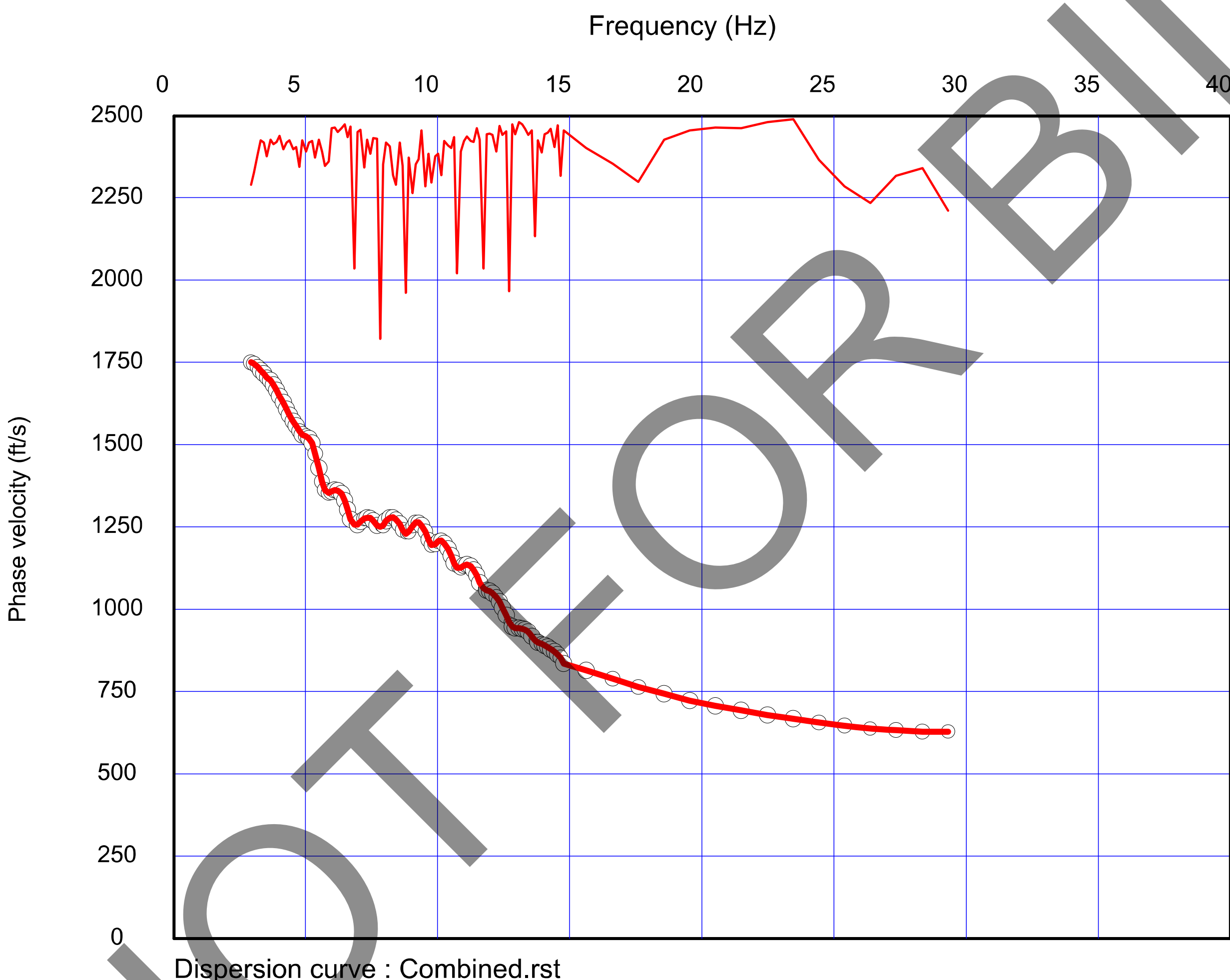
View looking east along Seismic Line SW-1.

SEISMIC LINE SW-1
SHEAR-WAVE MODEL



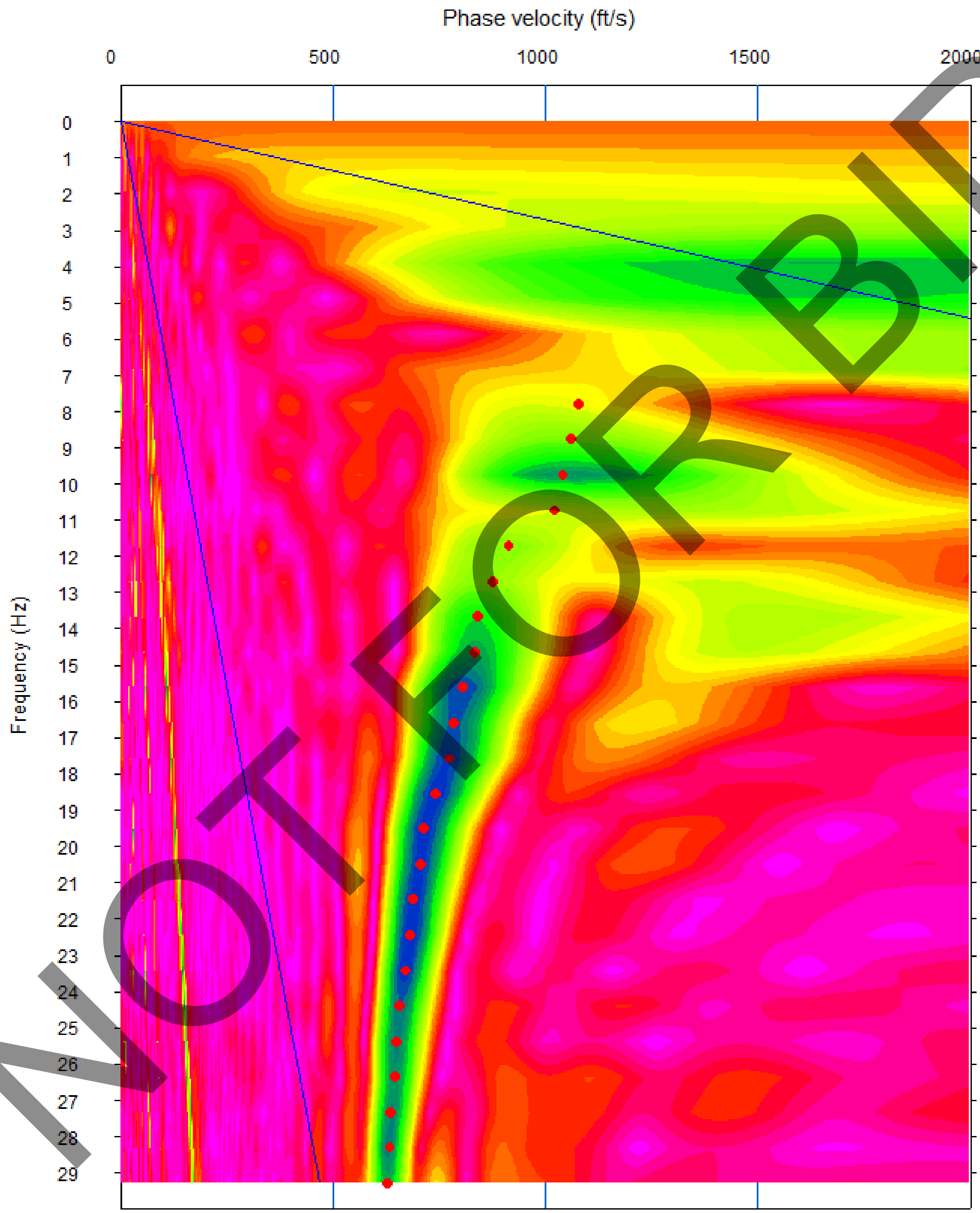
S-wave velocity model (inverted): Final.rst **Average Vs 100ft = 1075.1 ft/sec**

SEISMIC LINE SW-1



COMBINED DISPERSION CURVE

SEISMIC LINE SW-1



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE