HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT
PURCHASING DEPARTMENT
15959 E. GALE AVENUE • CITY OF INDUSTRY, CA 91745
(626) 933-3930

Joel Duarte
Director Purchasing and Warehouse

February 8, 2024

HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT
BID NUMBER 2023-24.09
HEAD START PORTABLE BUILDING & SITE IMPROVEMENTS WING LANE ES

ADDENDUM NO. 1

This Addendum Number One (#1) hereby makes changes to Bid #2023-24.09. Bidders shall acknowledge receipt of this Addendum in space provided on the Bid Form. Failure to acknowledge any addenda issued may subject Bidders to disqualification.

Included in this Bid Addenda No.1:

- Geotechnical Investigation Report, Converse Consultants, August 21, 2023

Sincerely,

Joel Duarte
Director of Purchasing and Warehouse
Hacienda La Puente Unified School District

Converse Consultants
Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

GEOTECHNICAL INVESTIGATION REPORT

WING LANE ELEMENTARY SCHOOL PORTABLES PROJECT

16605 WING LANE
VALINDA, CALIFORNIA 91744

CONVERSE PROJECT NO. 23-31-214-01

Prepared For:

HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT
15959 Gale Avenue
City of Industry, California 91745

Presented By:

CONVERSE CONSULTANTS
717 South Myrtle Avenue
Monrovia, California 91016
626-930-1200

August 21, 2023

Converse Consultants
Geotechnical Engineering, Environmental & Groundwater Science, Inspection & Testing Services

August 21, 2023

Mr. Leonard Hernandez
Executive Director of Facilities and Operations
Hacienda La Puente Unified School District
15959 Gale Avenue
City of Industry, California 91745

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Wing Lane Elementary School Portables Project
16605 Wing Lane
Valinda, California 91744
Los Angeles County Assessor Parcel Number (APN): 8745-014-900
Converse Project No. 23-31-214-01

Dear Mr. Hernandez:

Enclosed is the Geotechnical Investigation Report prepared by Converse Consultants (Converse) for the Wing Lane Elementary School Portables Project located within the existing Wing Lane Elementary School campus in Valinda, California.

The purpose of the study was to investigate the geotechnical site conditions and provide recommendations for the installation of a new parking lot, the addition of two new shade structures and two playground structures, and a new 48' x 40' portable building on raised wood foundations with ramp access.

Based on our field exploration, laboratory testing, geologic evaluation, and geotechnical analysis, the site is suitable from a geotechnical standpoint for this project, provided our conclusions and recommendations are implemented during design and construction.

We appreciate the opportunity to be of continued service to Hacienda La Puente Unified School District (HLPUSD). If you should have any questions, please do not hesitate to contact us at (626) 930-1200.

Sincerely,

CONVERSE CONSULTANTS

Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F. ASCE
Senior Vice President/Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
KATHIRAVETPILLAI SIVATHASAN
No. 2708
Exp. 06/30/2024
GEOTECHNICAL
STATE OF CALIFORNIA

717 South Myrtle Avenue, Monrovia, California 91016
Telephone: (626) 930-1200 ♦ Facsimile: (626) 930-1212 ♦ www.converseconsultants.com

PROFESSIONAL CERTIFICATION

This report for the Wing Lane Elementary School Portables Project located within the existing Wing Lane Elementary School campus in Valinda, California., has been prepared by the staff of Converse under the professional supervision of the individuals whose seals and signatures appear hereon.

The findings, recommendations, specifications, or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of Southern California. There is no warranty, either expressed or implied.

In the event that changes to the property occur, or additional, relevant information about the property is brought to our attention, the conclusions contained in this report may not be valid unless these changes and additional relevant information are reviewed, and the recommendations of this report are modified or verified in writing.

Babak Abbasi, PhD, PE
Project Engineer

LICENSED PROFESSIONAL ENGINEER
BABAK ABBASI
No. 94467
Exp. 3/31/2023
CIVIL
STATE OF CALIFORNIA

Derek A. Sobol, EIT
Project Engineer

Robert L. Gregorek II, PG, CEG
Senior Geologist

REGISTERED GEOLOGIST
ROBERT L. GREGOREK II
No. 1257
CERTIFIED ENGINEERING GEOLOGIST
Exp. 8/31/23
STATE OF CALIFORNIA

Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F. ASCE
Senior Vice President/Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
KATHIRAVETPILLAI SIVATHASAN
No. 2708
Exp. 06/30/2024
GEOTECHNICAL
STATE OF CALIFORNIA

EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions and recommendations as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed project is located at 16605 Wing Lane, in Valinda, Los Angeles County, California. The subject site is relatively flat graded with surface elevations of approximately 365 feet relative to mean-sea-level (MSL). The site is bounded by Residential Housing to the north, Valinda Avenue to the east, Wing Lane to the south, and N Brigita avenue to the west. The site coordinates are: North latitude: 34.0297, West longitude: -117.9281 degrees.

- A total of four (4) exploratory borings (BH-1 through BH-4) were drilled within the project sites on August 3, 2023. Three of the borings (BH-1 through BH-3) were advanced using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from approximately 26.5 feet to 51.5 feet below the existing ground surface (bgs). BH-4 was advanced using a 4-inch hand auger to depth of 5 feet bgs. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils.

- The project site is not located within a currently designated State of California Earthquake Fault Zone for surface fault rupture. No surface faults are known to project through or towards the site.

- The site is located within a potential liquefaction zone per the State of California Seismic Hazard Zones Map for the Baldwin Park Quadrangle as shown in Figure No. 6, *Seismic Hazard Zones Map*. The soil encountered during our subsurface exploration shows that in-situ soil type is Clay (CL) to Sandy Clay(CL). The liquefaction induced settlement is expected to be low.

- Groundwater was encountered during our subsurface exploration at approximately 22.7 feet. Based on review of Historically Highest Groundwater Map, Plate No. 1.2, in the Seismic Hazard Zone report for the Baldwin Park 7.5-minute Quadrangle (1998), the historically highest groundwater level contours in the vicinity of the site are interpreted to be approximately 20 feet below ground surface.

- The earth materials encountered during our investigation consist of existing fill soils placed during previous site grading operations and natural alluvial soils to a maximum depth explored of 51.5 feet bgs. The fill soils encountered consist primarily of sandy clay topsoil, filled with grass roots to a depth of approximately 5 feet. The alluvial soil deposits below the fill consist of lean clay and sandy clay.

Sampling blow-counts correlate from stiff to very stiff conditions near surface, and generally become softer with depth.

- In general, the pH value, chloride content, and concentrations of water-soluble sulfates saturated resistivity of the site soils are in the non-corrosive range for cement. The saturated resistivity of the site soils is in corrosive range to ferrous metals.

- The earth materials at the site should be excavatable with conventional heavy-duty earth moving equipment. Earthwork should be performed with suitable equipment for clayey materials.

- Shallow foundations are considered suitable for structure support provided the recommendations in this report are incorporated into the project plans and specifications and are followed during site construction. Note that two feet of non-expansive soils are recommended below concrete pads, footings, asphalt driving surfaces, sidewalks and other improvements not supported on cast-in-drilled-hole footings.

- For non-building structures (e.g., shade structures, signs, fence walls, short retaining walls, etc.), cast-in-drilled-hole footings can be used.

Results of our investigation indicate that the site is suitable from a geotechnical standpoint for the proposed development, provided that the recommendations contained in this report are incorporated into the design and construction of the project.

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1.0 INTRODUCTION

This report contains the findings and recommendations of our geotechnical study performed at the site of the Wing Lane Elementary School Portables Project located within the existing Wing Lane Elementary School campus in Valinda, California., as shown on Figure No. 1, *Site Location Map*.

The purpose of the study was to evaluate the subsurface soil conditions and provide geotechnical recommendations and design recommendations for the design and construction of the proposed project, consistent with the 2022 edition of California Building Code (CBC), Title 24, Chapter 16; Earthquake Design, Chapter 18A, Foundation and Retaining Wall; Appendix Chapter 33, Excavation and Grading; Part 1' section 4-317 (e) and CGS Note 48-Checklist for the review of Geologic/Seismic Reports for California Public Schools, Hospitals and Essential Services Buildings for new and existing (retrofit/modernization) buildings.

This report is written for the project described herein and is intended for use solely by HLPUSD, Wing Lane Elementary School, and its design team. It should not be used as a bidding document but may be made available to potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

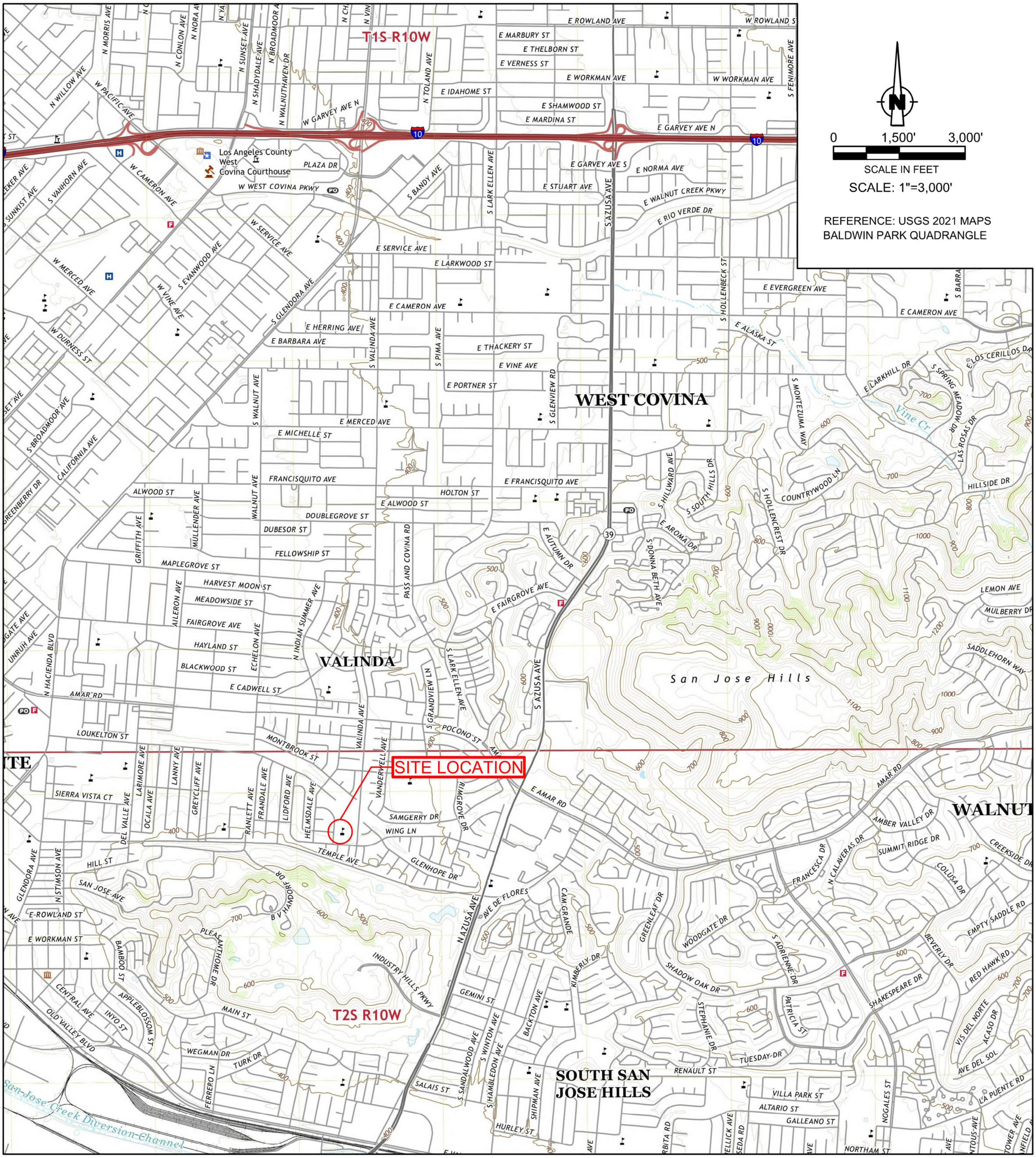
2.0 SITE AND PROJECT DESCRIPTION

The proposed project is located at 16605 Wing Lane, La Puente, In Los Angeles County, California. The site is bounded by Residential Housing, to the north, Valinda Avenue to the east, Wing Lane to the south, and N Brigita avenue to the west. The site coordinates are: North latitude: 34.0297, West longitude: -117.9281 degrees.

We understand that the proposed project entails the installation a new parking lot, the addition of two new shade structures and two playground structures and a new 48' x 40' portable building on raised wood foundations with ramp access, the project is located within the existing Wing Lane Elementary School campus as shown on Figure No. 2 *Approximate Boring Location Map.* The structural loads are anticipated to be low to moderate.

3.0 SCOPE OF WORK

The scope of our work included a site reconnaissance, subsurface exploration with soil sampling, laboratory testing, engineering analyses, and preparation of this report.



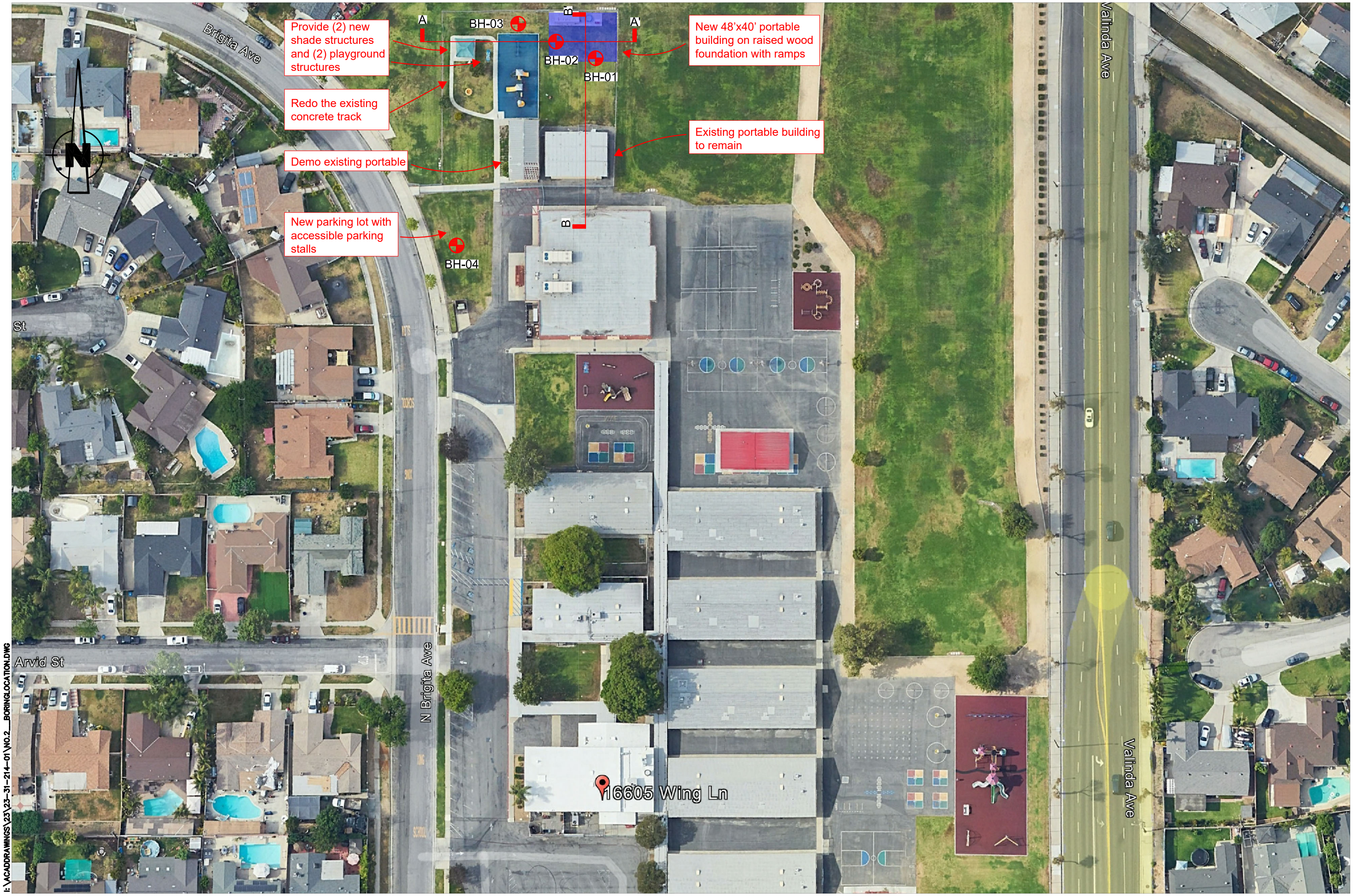
SITE LOCATION MAP

Wing Lane Elementary School
16605 Wing Lane
Valinda, California 91744

Project No.
23-31-214-01

Converse Consultants

Figure No.
1



Boring Location Map

Converse Consultants

Wing Lane Elementary School
16605 Wing Lane
Valinda, California 91744

Project No.	Figure No.
22-31-214-01	2

3.1 Site Reconnaissance

During the site reconnaissance on July 21, 2023, the surface conditions were noted, and the locations of the borings were determined so that drill rig access to all the locations was available. The borings were located using existing boundary features as a guide and should be considered accurate only to the degree implied by the method used. Underground Service Alert (USA) of Southern California was notified of our proposed drilling locations at least 48 hours prior to initiation of the subsurface field work.

3.2 Subsurface Exploration

A total of four (4) exploratory borings (BH-1 through BH-4) were drilled within the project sites on August 3, 2023. Three of the borings (BH-1 through BH-3) were advanced using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from approximately 26.5 feet to 51.5 feet below the existing ground surface (bgs). BH-4 was advanced using a 4-inch hand auger to depth of 5 feet bgs. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

California Modified Sampler ring samples, Standard Penetration Test samples, and bulk soil samples were obtained for laboratory testing. Standard Penetration Tests (SPTs) were performed in selected borings at selected intervals using a standard split-barrel sampler (1.4 inches inside diameter and 2.0 inches outside diameter). Borings terminated at a depth less than 10 feet below existing ground surface were backfilled with soil cuttings, tamped and capped to match surface conditions. Borings extending into groundwater or deeper than 10 feet were backfilled with cement grout and capped to match surface conditions.

The approximate locations of the exploratory borings are shown in Figure No. 2, *Approximate Boring Location Map*. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

3.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in classification and to evaluate relevant engineering properties. The tests performed included:

- In Situ Moisture Contents and Dry Densities (ASTM Standard D2216)
- Grain-Size Analysis (ASTM D422)
- Passing Sieve No. 200 (ASTM D1140)
- Direct Shear (ASTM Standard D3080)

- Maximum dry density and optimum-moisture content relationship (ASTM Standard D1557)
- Consolidation (ASTM Standard D2435)
- Soil Corrosivity Tests (Caltrans 643, 422, 417, and 532)

For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*. For *in-situ* moisture and density data, see the Logs of Borings in Appendix A, *Field Exploration*.

3.4 Engineering Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated. This report was prepared to provide the findings, conclusions and recommendations developed during our investigation and evaluation.

4.0 GEOLOGIC CONDITIONS

4.1 Regional Geologic Setting

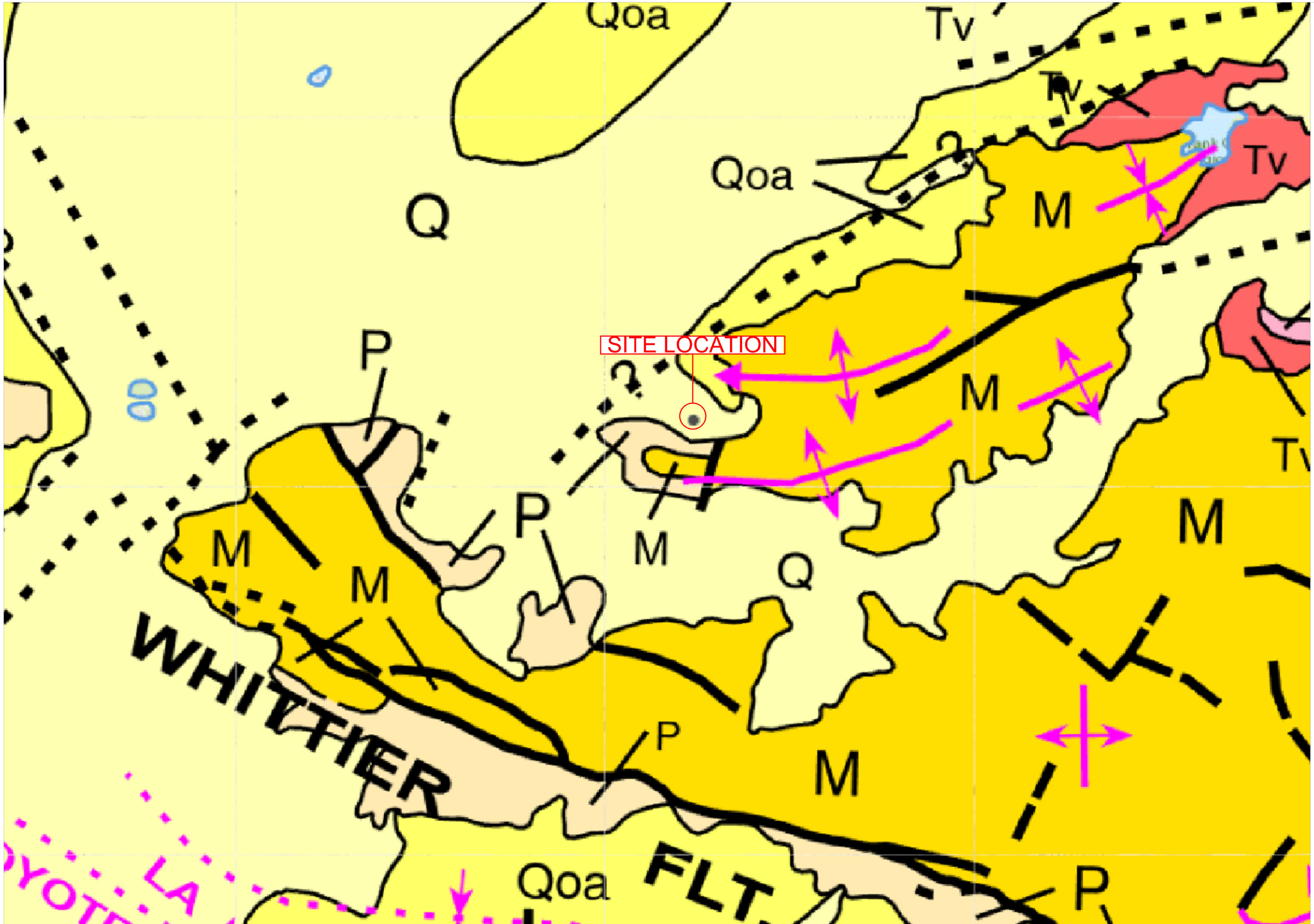
The project site lies in within the central portion of the San Gabriel Valley along the southern margin of the Transverse Ranges geomorphic province of California and along the northern margin of the Peninsular Ranges geomorphic province.

The central San Gabriel Valley is situated at the junction of the two major convergent fault systems. The first group includes the northwest-trending high angle strike slip faults of the San Andreas system projecting from the northern terminus of the Peninsular Ranges province. Faults in this group include the Palos Verdes, Newport-Inglewood, and Whittier-Elsinore fault zones. The second group includes the east-west trending low angle reverse or reverse-oblique faults bounding the south margin of the Traverse Ranges Province. Faults in this group include the Malibu-Santa Monica, Hollywood, Raymond and Sierra Madre fault zones. The seismic hazard for the San Gabriel Valley and vicinity is high.

Figure No. 3, *Regional Geologic Map*, has been prepared to show the location of the existing Wing Lane Elementary School with respect to regional geology of the San Gabriel Valley and vicinity. The nearby Elysian Park Fault and Raymond Fault and other active local and regional faults were included as active faults modeled for the probabilistic seismic hazard analysis.

4.2 Subsurface Profile of Project Site

The earth materials encountered during our investigation consist of existing fill soils placed during previous site grading operations and natural alluvial soils to a maximum depth explored of 51.5 feet bgs. The fill soils encountered consists primarily of sandy clay, silty sands and sands. The alluvial soil deposits below the fill consist of silty sands, sands,



QUATERNARY DEPOSITS

Qs	Extensive marine and nonmarine sand deposits, generally near the coast or desert playas
Q	Alluvium, lake, playa, and terrace deposits; unconsolidated and semi-consolidated
Qls	Selected large landslides
Qg	Glacial till and moraines. Found at high elevations mostly in the Sierra Nevada and Klamath Mountains
Qoa	Older alluvium, lake, playa, and terrace deposits
QPc	Pleistocene and/or Pliocene sandstone, shale, and gravels deposits; mostly loosely consolidated

QUATERNARY VOLCANIC ROCKS

Qrv	Recent (Holocene) volcanic flow rocks; minor pyroclastic deposits
Qrv^{p}	Recent (Holocene) pyroclastic and volcanic mudflow deposits
Qv	Quaternary volcanic flow rocks; minor pyroclastic deposits
Qv^{p}	Quaternary pyroclastic and volcanic mudflow deposits

TERTIARY SEDIMENTARY ROCKS

Tc	Undivided Tertiary nonmarine sandstone, shale, conglomerate, breccia, and ancient lake deposits
P	Pliocene marine sandstone, siltstone, shale, and conglomerate; mostly moderately consolidated
M	Miocene marine sandstone, shale, siltstone, conglomerate, and breccia; moderately to well consolidated
Mc	Miocene nonmarine sandstone, shale, conglomerate, and fanglomerate; moderately to well consolidated
Ø	Oligocene marine sandstone, shale, and conglomerate; mostly well consolidated
Øc	Oligocene nonmarine sandstone, shale, and conglomerate; mostly well consolidated
E	Eocene marine shale, sandstone, conglomerate, and minor limestone; mostly well consolidated
Ec	Eocene nonmarine sandstone, shale, and conglomerate; moderately to well consolidated
Ep	Paleocene marine sandstone, shale, and conglomerate; mostly well consolidated

SYMBOL EXPLANATION

Contact between geologic units - approximately located

?

Fault traces - solid where well located, dashed where approximately located or inferred, dotted where concealed, and queried where continuation or existence is uncertain. Ball and bar on downthrown side (relative or apparent). Arrows indicate direction of lateral movement (relative or apparent)

?

Thrust fault (barbs on upper plate)

Regional strike and dip of stratified rocks

Regional strike and dip of stratified rocks (overturned)

N

NOT TO SCALE

REGIONAL GEOLOGIC MAP

Converse Consultants

Wing Lane Elementary School
16605 Wing Lane
Valinda, California 91744

Project No. 23-31-214-01

Figure No. 3

I:\ACADDRAWINGS\23\23-31-214-01\NO.3 GEOLOGICAL MAP.DWG and clays. Sampling blow-counts correlate to loose to moderately dense conditions near surface, and generally become denser with depth.

4.3 Groundwater

Groundwater was encountered during our subsurface exploration at approximately 22.7 feet below ground surface. Based on review of Historically Highest Groundwater Map, Plate No. 1.2, in the Seismic Hazard Zone report for the Baldwin Park 7.5 Minute Quadrangle, the historically highest groundwater level contours in the vicinity of the site are interpreted to be approximately 20 feet below ground surface.

The groundwater level beneath the site can vary depending upon the seasonal precipitation and groundwater basin activities including recharge, storage and pumping occurring in the general site vicinity. Zones of perched groundwater may be present within the near-surface deposits due to local conditions, storm water recharge or during rainy seasons.

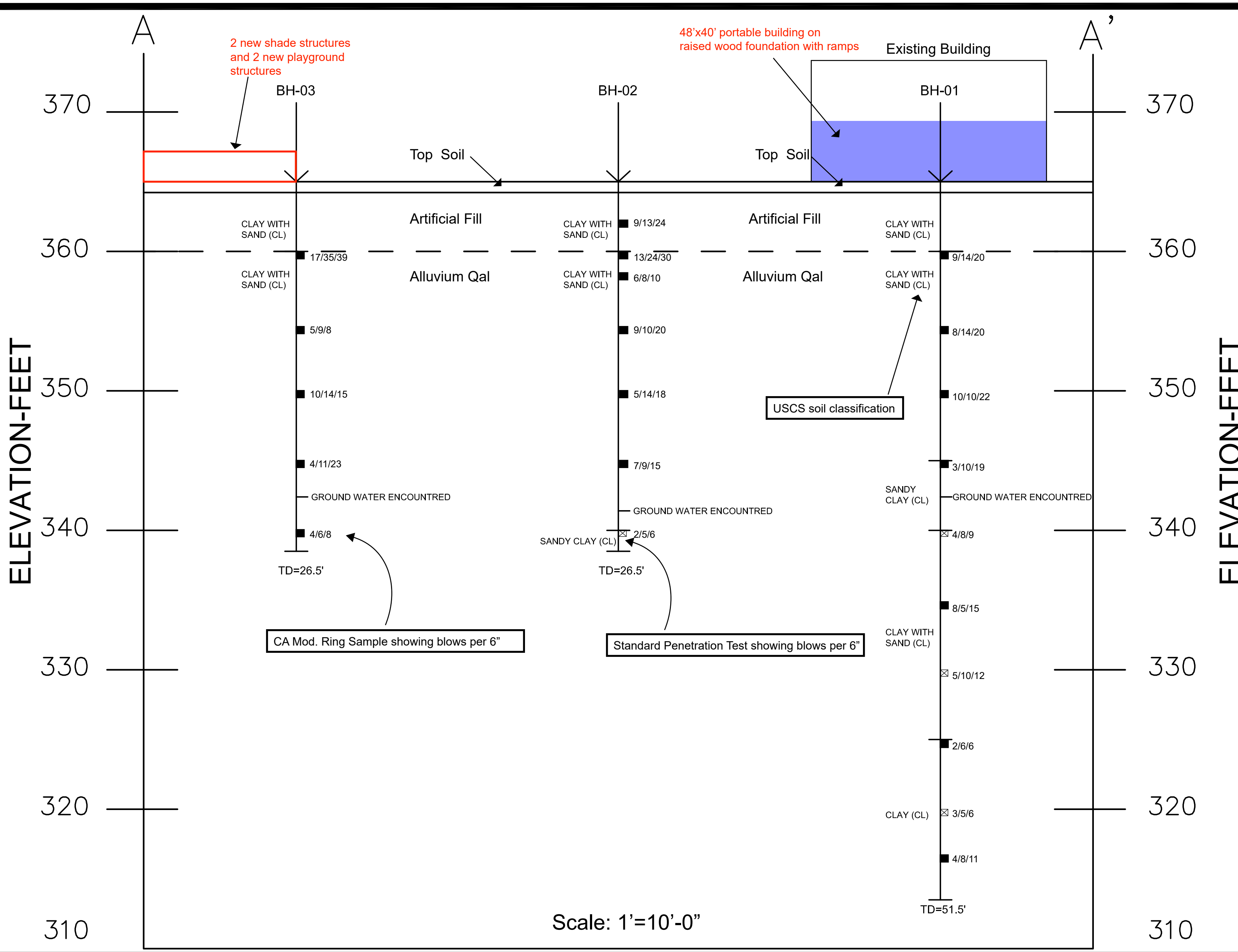
4.4 Subsurface Variations

Based on results of the subsurface exploration and our experience, some variations in the continuity and nature of subsurface conditions within the project site should be anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If, during construction, subsurface conditions differ significantly from those presented in this report, this office should be notified immediately so that recommendations can be modified, if necessary.

A detailed description of the earth materials encountered during our field exploration is presented in Appendix A*, Field Exploration.* Figure No. 4, *Geologic Cross Section A-A'*, and Figure No. 4a, *Geologic Cross Section B-B'* are provided to illustrate current site conditions by using exploratory borings from the current study drilled on August 3, 2023.

5.0 FAULTING AND SEISMIC HAZARDS

Geologic hazards are defined as geologically related conditions that may present a potential danger to life and property. Typical geologic hazards in Southern California include earthquake ground shaking, fault surface rupture, liquefaction and seismically induced settlement, lateral spreading, landslides, earthquake induced flooding, tsunamis and seiches, and volcanic eruption hazard. Results of a site-specific evaluation for each type of possible seismic hazards are discussed in the following sections.



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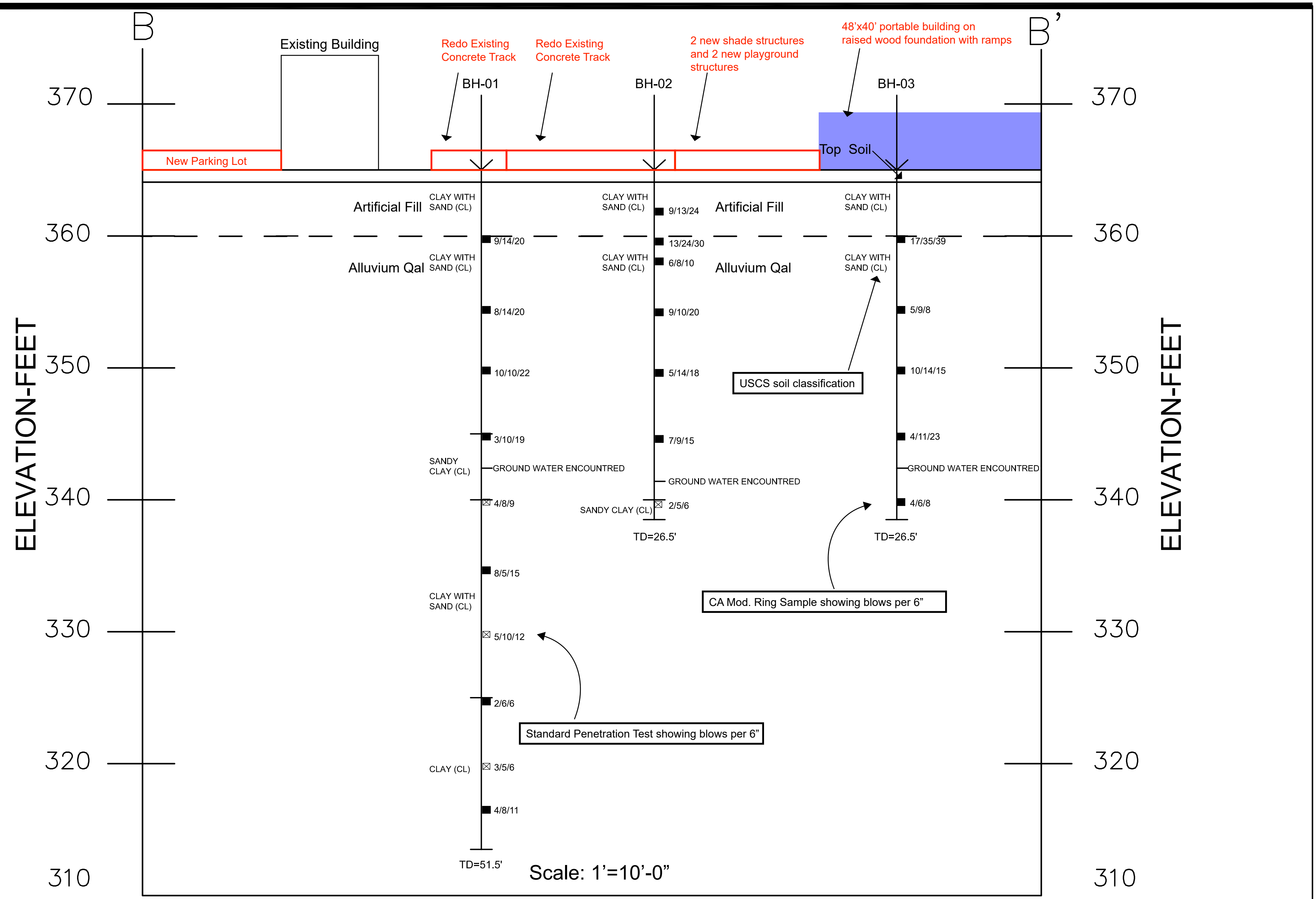
GEOLOGIC CROSS SECTION A-A'

Converse Consultants

Wing Lane Elementary School
16605 Wing Lane
Valinda, California 91744

Project No.	Figure No.
23-31-214-01	4



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GEOLOGIC CROSS SECTION B–B'

Converse Consultants

Wing Lane Elementary School
16605 Wing Lane
Valinda, California 91744

Project No.	Figure No.
23-31-214-01	4a

5.1 Seismic Characteristics of Nearby Faults

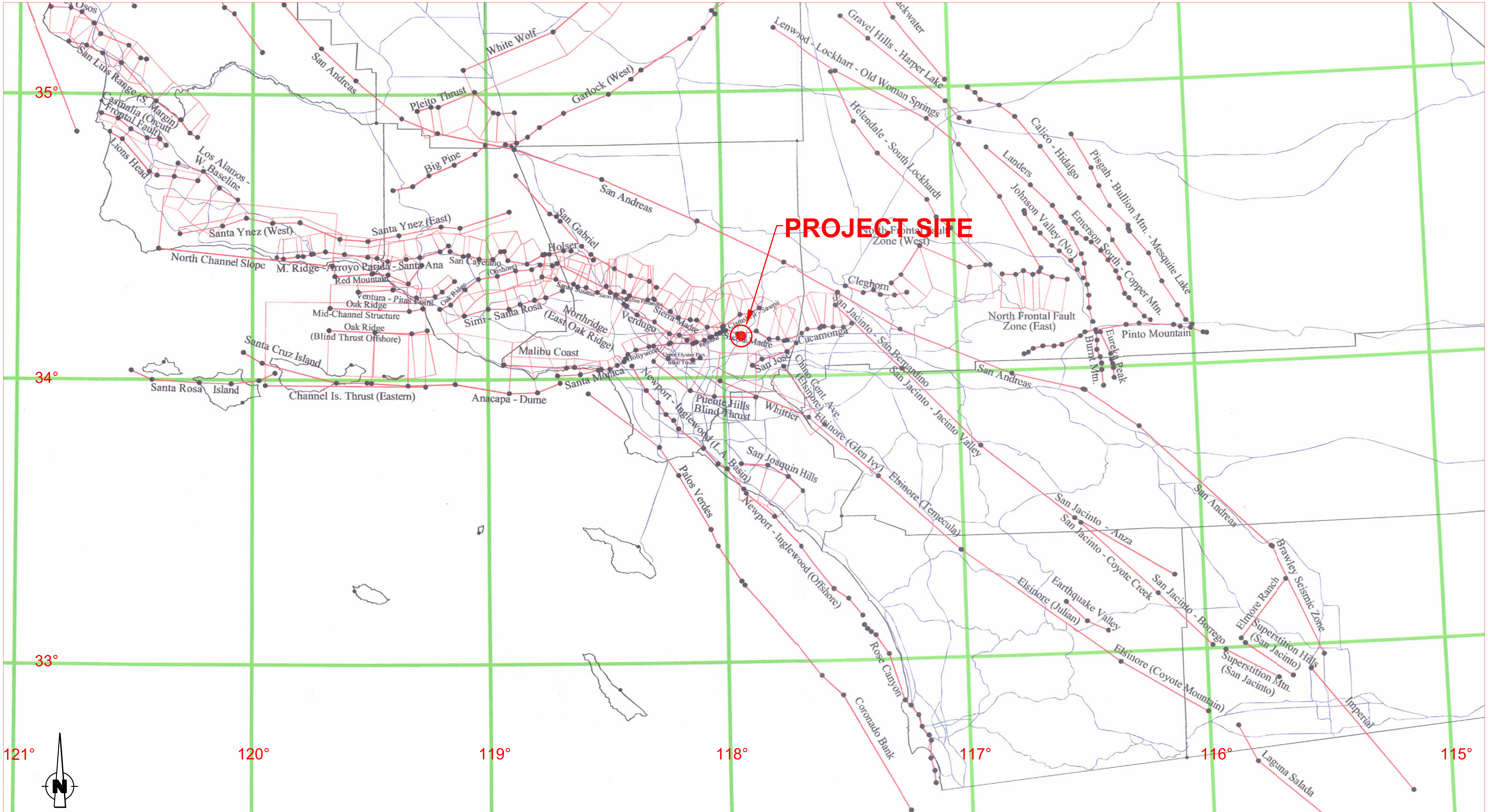
The proposed project site is situated in a seismically active region. As is the case for most areas of Southern California, ground-shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site. Review of recent seismological and geophysical publications indicates that the seismic hazard for the project site is high.

The project site is not located within a currently designated State of California Earthquake Fault Zone for surface fault rupture. No surface faults are known to project through or towards the site. The closest known faults to the project site with mapped surface traces are the Elysian Park (Upper) Fault (approximately 2.5 kilometers) and the Raymond Fault (approximately 4.6 kilometers). The Elsinore Fault, Verdugo Fault, and Sierra Madre (Central) Fault along with other regional faults were included as capable faults modeled for the probabilistic seismic hazard analysis for the site. The approximate locations of these local active faults with respect to the project site are shown on Figure No. 5, *Southern California Regional Fault Map*.

There are a number of regional fault systems, which could produce ground shaking at the site during a major earthquake. Table No. 1, *Summary of Regional Faults,* shows the location of the known most capable faults with respect to the site within 50 kilometers. The data presented below was calculated using the National Seismic Hazard Maps Database (USGS, 2008) and other published geologic data.

Table No. 1, Summary of Regional Faults

Fault Name and Section	Approximate * Distance to Site (kilometers)	Max. Moment Magnitude (M_{max})	Slip Rate (mm/yr)
Elysian Park (Upper)	2.54	6.70	1.3
Raymond	4.62	6.80	1.5
Elsionore;W	4.90	7.03	2.5
Verdugo	7.19	6.90	0.5
Sierra Madre	7.57	7.20	2.0
Sierra Madre Connected	7.57	7.30	2.0
Puente Hills (LA)	7.81	7.00	0.7
Clamshell-Sawpit	8.36	6.70	0.5
Puente Hills (Santa Fe Springs)	9.54	6.70	0.7
San Jose	10.31	6.70	0.5
Hollywood	10.66	6.70	1.0
Puente Hills (Coyote Hills)	11.94	6.90	0.7
Santa Monica Connected alt 2	13.43	7.40	2.4



REFERENCE: PORTION OF CGS 2002 CALIFORNIA FAULT MODEL MODIFIED FOR USE WITH FRISKSP AND EQFAULT BY THOMAS F. BLAKE, AUGUST 2004

FAULT SOURCES

BLIND THRUST FAULT, POLYGONS INDICATE RUPTURE PLANES AND DIP DIRECTION

SOUTHERN CALIFORNIA REGIONAL FAULT MAP

Converse Consultants

Wing Lane Elementary School
16605 Wing Lane
Valinda, California 91744

Project No.	23-31-214-01
Figure No.	5
Date	AUGUST 2023

Fault Name and Section	Approximate * Distance to Site (kilometers)	Max. Moment Magnitude (M_{max})	Slip Rate (mm/yr)
Newport Inglewood Connected alt 2	16.68	7.50	1.3
Newport-Inglewood, alt 1	17.01	7.20	1.0
Newport Inglewood Connected alt 1	17.01	7.50	1.3
Chino, alt 2	17.94	6.80	1.0
Chino, alt 1	17.98	6.70	1.0
Cucamonga	19.19	6.70	5.0
Sierra Madre (San Fernando)	20.06	6.70	2.0
Santa Monica Connected alt 1	20.42	7.30	2.6
Santa Monica, alt 1	20.42	6.60	1.0
San Gabriel	21.60	7.30	1.0
Palos Verdes Connected	24.65	7.70	3.0
Palos Verdes	24.65	7.30	3.0
Northridge	24.96	6.90	1.5
San Joaquin Hills	26.51	7.10	0.5
Malibu Coast, alt 2	27.05	7.00	0.3
Malibu Coast, alt 1	27.05	6.70	0.3
Anacapa-Dume, alt 2	28.65	7.20	3.0
Santa Susana, alt 1	30.85	6.90	5.0
Elsinore;GI	31.34	6.89	5.0
San Jacinto;SBV	32.20	7.06	6.0
Newport-Inglewood (Offshore)	33.71	7.00	N/A
S. San Andreas;NSB	33.77	6.86	22
Anacapa-Dume, alt 1	37.13	7.20	3.0
Holser, alt 1	37.17	6.80	0.4
Cleghorn	37.86	6.80	3.0
Simi-Santa Rosa	40.18	6.90	1.0
Oak Ridge (Onshore)	44.52	7.20	4.0
Oak Ridge Connected	44.52	7.40	3.6
San Jacinto;SJV	47.10	7.04	N/A
Elsinore;T	47.14	7.07	5.0
San Cayetano	47.81	7.20	6.0
S. San Andreas;SSB	48.15	6.95	16
North Frontal (West)	48.27	7.20	1.0

* (Source: https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/)

5.2 Surface Fault Rupture

The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. The Alquist-Priolo Earthquake Fault Zoning Act requires the California Geological Survey to zone "active faults" within the State of California. An "active fault" has exhibited surface displacement with Holocene time (within the last 11,000 years) hence constituting a potential hazard to structures that may be located across it. Based on a review of existing geologic information, no known active faults project through or toward the site. The potential for surface rupture resulting from the movement of the nearby major faults is considered very low.

5.3 Liquefaction and Seismically-Induced Settlement

Liquefaction is the sudden decrease in strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within 50 feet of the ground surface.

The site is located within a potential liquefaction zone per the State of California Seismic Hazard Zones Map for the Baldwin Park 7.5-minute quadrangle as shown in Figure No. 6, *Seismic Hazard Zones Map*. However, due to clayey nature of in-situ soil the liquefaction induced settlement is expected to be low.

5.4 Lateral Spreading

Seismically induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography at the project site and in the immediate vicinity of the site is relatively flat, with no significant nearby slopes or embankments. Under these circumstances, the potential for lateral spreading at the subject site is considered low.

5.5 Seismically-Induced Slope Instability

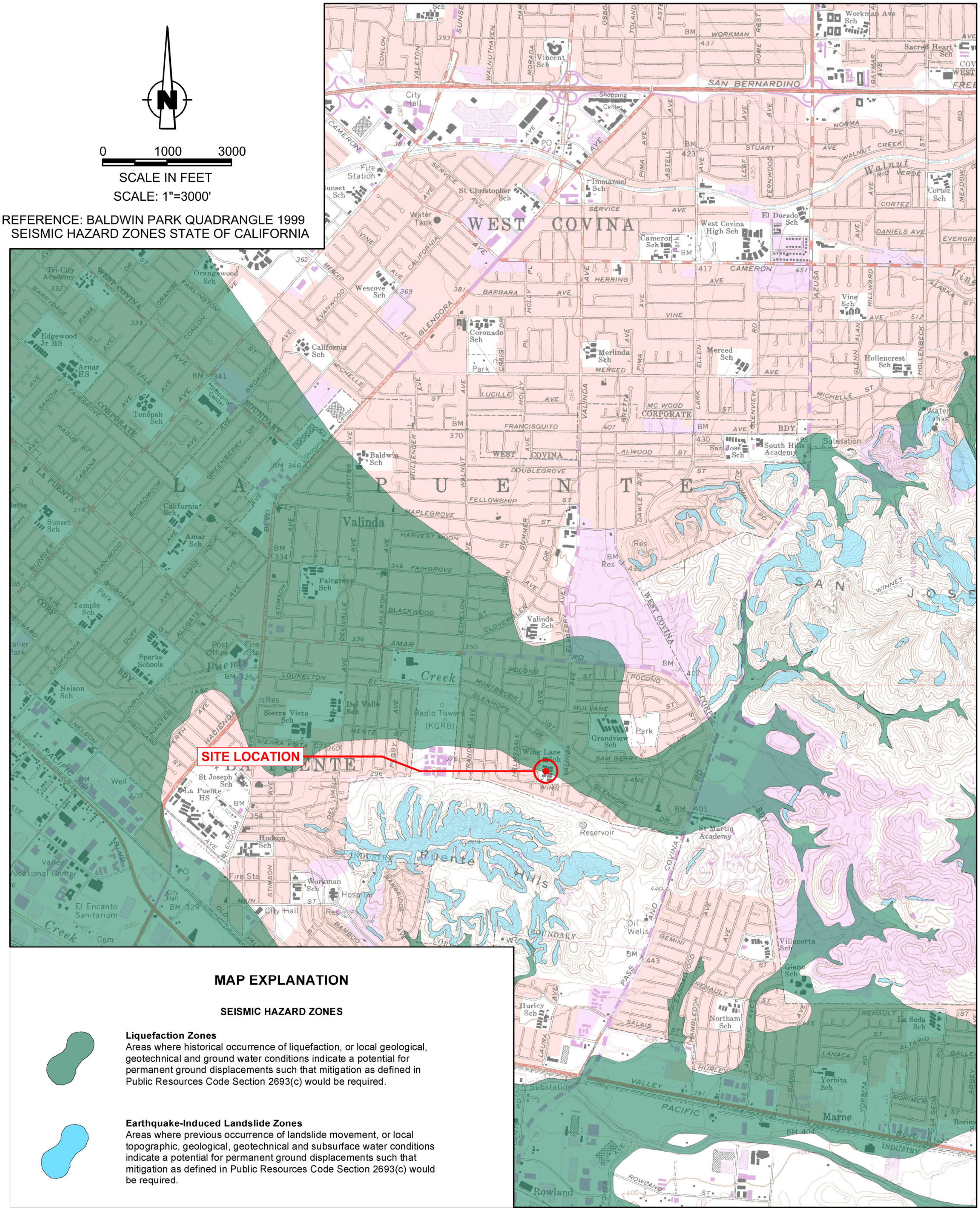
Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The project site is relatively flat. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered to be very low.

0 1000 3000

SCALE IN FEET

SCALE: 1"=3000'

REFERENCE: BALDWIN PARK QUADRANGLE 1999
SEISMIC HAZARD ZONES STATE OF CALIFORNIA



MAP EXPLANATION

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

SEISMIC HAZARD ZONES MAP

Converse Consultants	**Wing Lane Elementary School** 16605 Wing Lane Valinda, California 91744	**Project No.** 23-31-214-01	**Figure No.** 6

I:\ACADDRAWINGS\23\23-31-214-01\NO.6_7 SEISMIC HAZARD MAP.DWG

5.6 Earthquake-Induced Flooding

Review of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM), Map Number 06037C1700F, effective date September 26, 2008, from the Map Service Center (MSC) viewer, indicates that the site is designated as Zone "X", "Area of minimal flood hazard".

5.7 Tsunami and Seiches

Tsunamis are seismic sea waves generated by fault displacement or major ground movement. Based on the location of the site from the ocean (approximately 23 miles southwest of the site), tsunamis do not pose a hazard. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Based on site location away from lakes and reservoirs, seiches pose a very low hazard.

5.8 Volcanic Eruption Hazard

There are no known volcanoes near the site. According to Jennings (1994), the nearest potential hazards from future volcanic eruptions is the Amboy Crater-Lavic Lake area located in the Mojave Desert more than 120 miles northeast of the site. Volcanic eruption hazards are not present.

6.0 SEISMIC ANALYSIS

6.1 CBC Seismic Design Parameters

General seismic parameters based on the 2022 California Building Code and ASCE 7-16 with Supplement 1 are calculated using the ATC hazard, *Seismic Design by Location* website application and the site coordinates (North latitude: 34.0297, West longitude: -117.9281 degrees). The seismic parameters are presented below.

Table No. 2, CBC Seismic Design Parameters

Seismic Parameter	Value
Site Class	D
Mapped Short Period (0.2-sec) Spectral Response Acceleration, S_S	1.732 g
Mapped 1-second Spectral Response Acceleration, S_1	0.620 g
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.7
MCE 0.2-sec Period Spectral Response Acceleration, S_{MS}	1.732 g
MCE 1-second Period Spectral Response Acceleration, S_{M1}	1.581*
Design Spectral Response Acceleration for Short Period, S_{DS}	1.155 g

Seismic Parameter	Value
Design Spectral Response Acceleration for 1-second Period, S_{D1}	1.054*
TL	8
PGA_M	0.811
Seismic Design Category	D

*Per ASCE 7-16 Supplement 3-Section 11.4.8, the S_{M1} and S_{D1} values listed in this table are increased by 50%.

Provided the structure has fundamental period of vibration less than 0.5 s, site-specific seismic parameters may be used for site class F.

6.2 Site-Specific Seismic Parameters

Site-specific acceleration parameters were evaluated in accordance with the seismic provisions in Section 21 of ASCE 7-16 guidelines with Supplement 3 (ASCE, 2016), which were adopted in the 2022 California Building Code. These parameters were determined for the site coordinates from the boring data using the online calculator developed by the Utilization of Ground Motion Simulation (UGMS) committee of the Southern California Earthquake Center (SCEC). The recommended site-specific risk-targeted Maximum Considered Earthquake (MCE_R) and design response spectra are presented in Appendix D, *Seismic Hazard Analysis Results*. The following table summarizes the recommended 2022 CBC site-specific seismic design parameters calculated using the UGMS online tool.

Table No. 3, 2022 CBC Site-Specific Seismic Parameters

Seismic Parameter	Value
(1) MCE_R (5%, damped) Spectral response acceleration for short periods adjusted for site class, S_{MS}	2.141 g
(1)MCE_R (5% damped) spectral response acceleration at 1-second period adjusted for site class, S_{M1}	1.319 g
Design spectral response acceleration (5% damped) at short periods, S_{DS}	1.427 g
Design Spectral response acceleration (5% damped) at 1-second period, S_{D1}	0.879 g
Site-Modified Peak Ground Acceleration, MCE_G PGA	0.806 g

Site-specific parameters were determined based on the estimated average shear wave velocity of the site in the upper 30 meters (100 feet), V_{s30} of 278.0 m/sec (850 ft/sec), which calculated using the SPTPROP software (InfraGEO, 2020) based on the correlation with SPT blow counts by Brandenberg, Bellana and Shantz (2010). Extrapolation of estimated shear wave velocities from 50-ft depth to 100-ft depth was performed using the method proposed by Boore (2004). The Modified California Sampler blow counts were converted to equivalent SPT blow counts by multiplying the value by 0.65 to account for end-area effects.

A seismic deaggregation analysis conducted using the USGS Unified Hazard online tool shows the magnitude 6.26 event located approximately 5.4 miles (8.7 km) from the project site contributes the most to the seismic hazard at the project site.

7.0 EARTHWORK RECOMMENDATIONS

7.1 General Evaluation

Site earthwork recommendations provided in this section are based on our experience with similar projects and our evaluation of this study. Based on our understanding of the proposed project and the results of our field exploration, laboratory testing, and analysis of subsurface conditions at the site, we anticipate that the main earthwork activities associated with construction will be remedial grading (over-excavation), foundation excavations and trench excavation/backfill for utilities.

On-Site soils are not considered suitable for re-use as in the upper 2 feet of finished grade for support slabs, foundations, walkways and pavements. Excavated site soils, free of deleterious materials and rock particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill at depths greater than 2 feet below foundations, slabs, walkways and pavements. Any import fill should be tested and approved by geotechnical engineer or their representative. Any import fill should have an expansion potential less than 20. All compacted fill soils should be observed and tested by a Converse representative in accordance with the specifications presented in this section.

7.2 Over-Excavation

Prior to the start of construction, all loose soil, fill, and soil disturbed during demolition should be removed to firm and unyielding native material or compacted fill.

Due to the undocumented fills encountered at the site, we recommend over-excavation for structure footings to be at least five (5) feet below the existing grade or 3 feet below bottom of footing, whichever is deeper. Deeper removal and re-compaction will be needed if firm soil conditions are not exposed on the excavation bottom. Over-excavation should extend at least five (5) feet laterally beyond the limits of perimeter footings where feasible. The on-site soil is not considered suitable for re-use in the upper 2 feet of finished grade for support of slabs, foundations, walkways, and pavements. Due to high expansion potential of in-situ soils, Converse recommends the upper 2 feet of finished grade to be replaced and compacted with import fill with an expansion potential less than 20.

Over-excavation and re-compaction for retaining walls, if any, should be three (3) feet below bottom of footings and should extend three (3) feet laterally beyond the retaining wall area. The upper 24-inches of site soils should be removed and re-compacted in areas of sidewalks and surface parking. The upper 18 inches of soil should be processed and compacted in field areas. The over-excavation should extend two (2) feet laterally beyond the sidewalk and surface parking areas. If loose, disturbed, or otherwise unsuitable materials are encountered at the bottom of excavation, deeper removal will be required until firm native soils are encountered.

Excavation activities should not disturb adjacent utilities or undermine any adjacent buildings and structures to remain. Existing utilities should be removed and adequately capped at the project boundary line or salvaged/rerouted as designed.

The actual depth of removal should be based on recommendations and observation made during grading. Therefore, some variations in the depth and lateral extent of over-excavation recommended in this report should be anticipated.

7.3 Structural Preparation

All exposed subgrade soil surfaces should be observed by a geotechnical engineer or their representative prior to placement of fill or base materials. The exposed subgrade should be scarified at least 6 inches, moisture conditioned as needed to near-optimum moisture content, mixed and compacted to 90 percent relative compaction. The upper 12 inches of subgrade below new pavement should be compacted to 95 percent relative compaction.

If loose, yielding soil conditions are encountered at the excavation bottom, the following options can be considered:

a. Over-excavate until a firm bottom is reached.

b. Over-excavate an additional 18 inches deep, and then place at least 18-inch-thick compacted base material (CAB or equivalent) to bridge the soft bottom. Base materials should be compacted to at least 95% relative compaction.

c. Over-excavate an additional 18-inches deep, and then place a layer of geotextile (i.e., Mirafi HP570, or equivalent), then place 18-inch-thick compacted base material (CAB or equivalent) to bridge the soft bottom. Base should be compacted to at least 95% relative compaction. An additional layer of geotextile may be needed on top of the compacted base materials depending on the actual site conditions.

7.4 Engineered Fill

All engineered fill should be placed on competent, scarified and compacted bottom as evaluated by the geotechnical engineer and in accordance with the specifications presented in this section. Excavated site soils, free of deleterious materials and rock particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill. Any proposed import fill should be evaluated and approved by geotechnical engineer or their representatives prior to import to the site. Import fill material should have an expansion index less than 20.

Prior to compaction, fill materials should be thoroughly mixed, and moisture conditioned within three (3) percent of the optimum moisture content for granular soils and to approximately three (3) percent above the optimum moisture for fine-grained soils. Fill soils shall be evenly spread in maximum 8-inch lifts, watered, or dried as necessary, mixed, and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the geotechnical engineer. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D2922 test method.

7.5 Excavatability

Based on our field exploration, the earth materials at the site may be excavated with conventional heavy-duty earth moving and trenching equipment. (The onsite materials may contain demolition debris and gravel and/or cobbles. Earthwork should be performed with suitable equipment and methods for removal of debris from the engineered fill.)

7.6 Expansive Soil

One (1) expansion index test was performed in accordance with ASTM Standard D4829 for the upper five (5) feet of soil. Results of the test indicated an expansion index of 105 in BH-1 which corresponds to high expansion potential. Mitigation for expansive soil is necessary. The on-site soil materials will be mixed during the grading and the expansion potential might change. Therefore, the potential expansion index of site soils should be tested and verified after the grading of new slabs, foundations and pavements. New slabs, foundations, sidewalks and pavements should not be placed directly on on-site or native expansive subgrade soils, otherwise they may likely crack over time.

7.7 Trench Zone Backfill

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

Trench excavations to receive backfill shall be free of trash, debris, grass, roots or other unsatisfactory materials at the time of backfill placement. Excavated on-site soils free of oversize particles, defined as larger than one (1) inch in maximum dimension in the upper 12 inches of subgrade soils and larger than three (3) inches in the largest dimension in the trench backfill below, and deleterious matter after proper processing may be used to backfill the trench zone. Imported trench backfill, if used, should be approved by the project soils consultant prior to delivery at the site. No more than 30 percent of the backfill volume should be larger than ¾ inch in the largest dimension.

Trench backfill shall be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D2922 test method. At least the upper twelve (12) inches of trench underlying pavements should be compacted to at least 95 percent of the laboratory maximum dry density.

Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to within three (3) percent of optimum moisture content and then placed in horizontal layers if the expansion index is less than or equal to 30. Should the expansion index be greater than 30, backfill materials shall be brought to approximately three (3) percent above optimum moisture content. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer shall be evenly spread, moistened, or dried as necessary, and then tamped or rolled until the specified density has been achieved.

The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent. Observation and field tests should be performed by geotechnical engineer or their representatives during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained. It should be the responsibility of the contractor to maintain safe conditions during cut and/or fill operations. Trench backfill shall not be placed, spread, or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

Imported soils, if any, used as compacted trench backfill should be predominantly granular and meet the following criteria:

- Expansion Index less than 20
- Free of all deleterious materials
- Contain no particles larger than 3 inches in the largest dimension

- Contain less than 30 percent by weight retained on ¾-inch sieve
- Contain at least 15 percent fines (passing #200 sieve)
- Have a Plasticity Index of 10 or less

Any import fill should be tested and approved by the geotechnical representative prior to delivery to the site.

7.8 Shrinkage and Subsidence

Soil shrinkage and/or bulking as a result of remedial grading depends on several factors including the depth of over-excavation, and the grading method and equipment utilized, and average relative compaction. For preliminary estimation, bulking and shrinkage factors for various units of earth material at the site may be taken as presented below:

- The approximate shrinkage factor for the undocumented fill soils is estimated to range from ten (10) to fifteen (15) percent.

- The approximate shrinkage factor for the native alluvial soils is estimated to range from five (5) to ten (10) percent.

- For estimation purposes, ground subsidence may be taken as 0.1 feet as a result of remedial grading.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

The various design recommendations provided in this section are based on the assumptions that in preparing the site, the earthwork and site grading recommendations provided in this report will be followed.

8.0 DESIGN RECOMMENDATIONS

Based on the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, and understanding of the planned site development, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications, and are followed during site construction.

Remedial grading consisting of over-excavation and re-compaction and is required for the surficial soils to provide structural support of appurtenant improvements.

Shallow foundations can be used to support the proposed buildings with 3 feet of over-excavation and re-compaction below the bottom of the footings. For other non-building structures such as retaining walls, shallow foundation may be used.

8.1 Shallow Foundations

8.1.1 Vertical Capacity

We recommend continuous and square footings be founded at least 18 inches below lowest adjacent final grade entirely into compacted fill. A minimum footing width of 24 inches is recommended for square footings and 18 inches for continuous footings. The allowable bearing value for footings with above minimum sizes founded on compacted fill and competent native soils may be designed for a net bearing pressure of 2,000 pounds per square foot (psf) for dead-plus-live-loads. The net allowable bearing pressure can be increased by 200 psf for each additional foot of excavation depth and by 200 psf for each additional foot of excavation width up to a maximum value of 3,000 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity.

8.1.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.3 may be assumed with normal dead load forces. An allowable passive earth pressure of 200 psf per foot of depth up to a maximum of 1,500 psf may be used for footings poured against properly compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

8.1.3 Static Settlement

The static settlement of structures supported on continuous and/or spread footings founded on compacted fill and native soil will depend on the actual footing dimensions and the imposed vertical loads. Based on the maximum allowable net bearing pressures presented above, static settlement is anticipated to be less than 1.0 inch. Differential settlement is expected to be up to one-half of the total settlement over a 30-foot span.

8.1.4 Dynamic Increases

Bearing values indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by 33% for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by 33% for lateral loading due to wind or seismic forces.

8.2 Cast-In-Drilled-Hole Pile Foundations for Shade and other Non-building Structures

The planned shade and other non-building structures (e.g., lighting for parking lot, walkway, and court, fence walls, signs, etc.) may be supported on a Cast-In-Drilled-Hole (CIDH) pile foundation provided the following recommendations are incorporated into design and construction.

8.2.1 Vertical Capacity

CIDH piles should be at least 18-inches in diameter and can be designed for an allowable skin friction of 100 psf against the perimeter of pile. The diameter and length of CIDH pile shall be determined by the structural engineer based on design loads. The uplift capacities can be taken as one-half of compressive capacities for pile design.

8.2.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.30 may be assumed with normal dead load forces. An allowable passive earth pressure of 200 psf per foot of depth up to a maximum of 1,500 psf may be used for foundations poured against compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

For ground surface restrained by concrete slab, the passive resistance may be calculated from the ground surface. For unrestrained ground condition, the passive resistance of the upper one (1) foot of earth material should be neglected in design.

8.2.3 Settlement

Based on the maximum allowable net vertical capacity presented above, static settlement is anticipated to be less than 0.5 inch.

8.3 Slabs-on-Grade

Slabs-on-grade should have a minimum thickness of five (5) inches nominal for support of normal ground-floor live loads. Minimum reinforcement for slabs-on-grade should be No. 4 reinforcing bars, spaced at 18 inches on-center each way. The thickness and reinforcement of more heavily loaded slabs will be dependent upon the anticipated loads and should be designed by a structural engineer. A static modulus of subgrade reaction equal to 100 pounds per square inch per inch may be used in structural design of concrete slabs-on-grade.

It is critical that the exposed subgrade soils should not be allowed to desiccate prior to the slab pour. Care should be taken during concrete placement to avoid slab curling. Slabs should be designed and constructed as promulgated by the ACI and Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

In areas where a moisture-sensitive floor covering (such as vinyl tile or carpet) is used, a minimum 10-mil-thick moisture retarder/barrier between the bottom of slab and subgrade that meets the performance criteria of ASTM E1745 Class A material. Retarder/barrier sheets should be overlapped a minimum of six inches and should be taped or otherwise sealed per the product specifications.

8.4 Modulus of Subgrade Reaction

For the subject project, design of the structures supported on compacted fill subgrade prepared in accordance with the recommendations provided in this report may be based on a soil modulus of subgrade reaction of (k_s) of 100 pounds per square inch per inch.

8.5 Lateral Earth Pressure

Although not anticipated, the following provisional design values may be used for any utility vaults and/or walls below grade that are less than 8 feet high.

The earth pressure behind any buried wall depends primarily on the allowable wall movement, type of soil behind the wall, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following earth pressures are recommended for vertical walls with no hydrostatic pressure.

Table No. 4, Lateral Earth Pressures for Retaining Wall Design

Backfill Slope (H:V)	Cantilever Wall Equivalent Fluid Pressure (pcf)	Restrained Wall (psf)
Level	55 (triangular pressure distribution)	75 (triangular pressure distribution)

The recommended lateral pressures assume that the walls are fully back-drained to prevent build-up of hydrostatic pressure. Adequate drainage could be provided by means of permeable drainage materials wrapped in filter fabric installed behind the walls. The drainage system should consist of perforated pipe surrounded by a minimum one (1) square feet per lineal feet of free draining, uniformly graded, 3/4-inch washed, crushed aggregate, and wrapped in filter fabric such as Mirafi 140N or equivalent. The filter fabric should overlap approximately 12 inches or more at the joints. The subdrain pipe should consist of perforated, four-inch diameter, rigid ABS (SDR-35) or Schedule 40 PVC Pipe, or equivalent, with perforations placed down. Alternatively, a prefabricated drainage composite system such as the Miradrain G100N or equivalent can be used. The subdrain should be connected to solid pipe outlets, with a maximum outlet spacing of 100 feet. Waterproofing membranes should be added to the subterranean wall levels for moisture sensitive areas to mitigate moisture migration through the walls.

In addition, walls with inclined backfill should be designed for an additional equivalent fluid pressure of one (1) pound per cubic foot for every two (2) degrees of slope inclination. Walls subjected to surcharge loads located within a distance equal to the height of the wall should be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained or restrained walls, respectively. These values are applicable for backfill placed between the wall stem and an imaginary plane rising 45 degrees from below the edge (heel) of the wall footings.

Retaining walls taller than 6 feet should be designed to resist additional earth pressure caused by seismic ground shaking based on CBC latest edition. A seismic earth pressure of 29H (psf), based on an inverted triangular distribution, can be used for design of wall.

8.6 Soil Corrosivity Evaluation

Based on our review of soil corrosivity test results (see Appendix B), the soluble sulfate concentration, pH, and chloride content are not in the corrosive range to concrete in accordance with the Caltrans Corrosive Guidelines (2012). The minimum saturated resistivity is in the corrosive range to ferrous metal. Mitigation measures to protect concrete in contact with the soils are not anticipated.

A corrosion engineer may be consulted for appropriate mitigation procedures and construction design, if needed. General considerations for corrosion mitigation measures may include the following:

- Steel and wire concrete reinforcement should have at least three inches of concrete cover where cast against soil, unformed.
- Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

The test results presented herein are considered preliminary. If advanced corrosivity study is desired by the design team, a corrosion engineer can be consulted for appropriate mitigation procedures and construction design.

8.7 Flexible Pavement

The flexible pavement structural section design recommendations were performed in accordance with the method contained in the *CALTRANS Highway Design Manual*, Chapter 630, without the factor of safety. No specific traffic study was performed to determine the Traffic Index (TI) for the proposed project, therefore a wide range of TI values were evaluated.

Due to various earth materials encountered at the site, flexible pavement structural section recommendations are prepared for both subgrade soils. We recommend that the project structural engineer consider the traffic loading conditions at various locations and select the appropriate pavement sections from the following table:

Table No. 5, Flexible Pavement Structural Sections

Design R-value	Design TI	Asphalt Concrete (AC) Over Aggregate Base (AB) Structural Sections		Full AC Structural Section
		AC (inches)	AB (inches)	AC (inches)
8	4	***3.0***	***5.5***	***5.0***
	5	***4.0***	***7.0***	***6.5***
	6	***5.0***	***9.0***	***8.0***
	7	***6.0***	***11.0***	***9.5***
	8	***7.0***	***13.0***	***11.0***
	9	***8.0***	***14.5***	***12.5***

Base material shall conform to requirements for Crushed Miscellaneous Base (CMB) or equivalent and should be placed in accordance with the requirements of the Standard Specifications for Public Works Construction (SSPWC, latest Edition). Asphaltic materials should conform to Section 203-1, "Paving Asphalt," of the Standard Specifications for Public Works Construction (SSPWC, latest Edition) and should be placed in accordance with Section 302-5, "Asphalt Concrete Pavement," of the SSPWC, 2012 edition.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

8.8 Rigid Pavement

Rigid pavement design recommendations were provided in accordance with the Portland Cement Association's (PCA) Southwest Region Publication P-14, Portland Cement Concrete Pavement (PCCP) for Light, Medium and Heavy Traffic Rigid Pavement. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table:

Table No. 6, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
8	5.0	7.5
	6.0	8.0
	7.0	8.0
	8.0	8.5
	9.0	8.5

The above pavement section is based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,750 psi. The third point method of testing beams should be used to evaluate modulus of rupture. The concrete mix design should contain a minimum cement content of 5.5 sacks per cubic yard. Recommended maximum and minimum values of slump for pavement concrete are 3.0 inches to 1.0 inch, respectively.

Transverse contraction joints should not be spaced more than 10 feet and should be cut to a depth of 1/4 the thickness of the slab. Longitudinal joints should not be spaced more than 12 feet apart. A longitudinal joint is not necessary in the pavement adjacent to the curb and gutter section.

Prior to placement of concrete, at least the upper 12.0 inches of subgrade soils below rigid pavement sections should be compacted to at least ninety-five percent (95%) relative compaction as defined by the ASTM D 1557 standard test method.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into pavement base and/or subgrade.

8.9 Site Drainage

Adequate positive drainage should be provided away from the structures to prevent ponding and to reduce percolation of water into structural backfill. We recommend that the landscape area immediately adjacent to the foundation shall be designed sloped away from the building with a minimum 5% slope gradient for at least 10 feet measured perpendicular to the face of the wall. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building per 2022 CBC.

Planters and landscaped areas adjacent to the building perimeter should be designed to minimize water infiltration into the subgrade soils. Gutters and downspouts should be installed on the roof, and runoff should be directed to the storm drain through non-erosive devices. Lower-level walkways and open patio areas may require special drainage provisions and sump pumps to provide suitable drainage.

9.0 CONSTRUCTION RECOMMENDATIONS

9.1 General

Site soils should be excavatable using conventional heavy-duty excavating equipment. Temporary sloped excavation is feasible if performed in accordance with the slope ratios provided in Section 10.2, *Temporary Excavations*. Existing utilities should be accurately located and either protected or removed as required. For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor as necessary, to protect the workers in the excavation.

9.2 Temporary Excavations

Based on the materials encountered in the exploratory borings, sloped temporary excavations may be constructed according to the slope ratios presented in Table No. 7, *Slope Ratios for Temporary Excavation*. Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand should be constructed at a flatter gradient than presented in the following table:

Table No. 7, Slope Ratios for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 4	Vertical
4 – 8	1: 1
8+	1.5: 1

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to minimize raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction equipment, should not be placed within five (5) feet of the unsupported excavation edge. The above maximum slopes are based on a maximum height of six (6) feet of stockpiled soils placed at least five (5) feet from the excavation edge.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

If the excavation occurs near existing structures, special construction considerations would be required during excavation to protect these existing structures during construction. The proposed excavation should not cause loss of bearing and/or lateral supports of the existing structures.

9.3 Slot Cut Recommendations

Temporary excavations during possible improvements should not extend below a 1:1 horizontal:vertical/ (H:V) plane extending beyond and down from the bottom of the existing utility lines or structures. The remedial grading excavations should not cause loss of bearing and/or lateral support for adjacent utilities or structures.

If remedial grading excavations extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of adjacent off-site utility lines or structure foundations, shoring or slot cutting shall be employed. “A-B-C” slot cuts exposing native sandy soils may be excavated with maximum 8 feet wide and 8 feet depth sections to prevent the existing utility lines or off-site structures from becoming unstable. Backfill should be accomplished in the shortest period of time possible and in alternating sections.

The ABC slot cutting method for retaining walls could be a possible option as an alternative to shoring for excavation less than 8 feet or with cohesive soils. In general, for structures it is not recommended for slot cutting if the height of excavation exceeds more than 8 feet or into sandy soils and with surcharging load.

9.4 Geotechnical Services During Construction

This report has been prepared to aid in the foundation plans and specifications, and to assist the architect, civil and structural engineers in the design of the proposed structures. It is recommended that this office be provided an opportunity to review final design drawings and specifications to verify that the recommendations of this report have been properly implemented.

Footing excavations should be observed by geotechnical engineer or their representative prior to placement of steel and concrete so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials. Trench backfill should be placed and compacted with observation and field density testing provided by this office.

During construction, the geotechnical engineer and/or their authorized representatives should be present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc. This firm does not practice or consult in the field of safety engineering. We do not direct the contractor's operations and cannot be responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the contractor. The contractor should notify the Owner if he considers any recommended actions presented herein to be unsafe.

10.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field and laboratory investigations, combined with an interpolation and extrapolation of soil conditions between and beyond boring locations. If conditions encountered during construction appear to be different from those shown by the borings, this office should be notified.

Design recommendations given in this report are based on the assumption that the earthwork and site grading recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the final site grading and actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

11.0 REFERENCES

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Appendix A

Field Exploration

APPENDIX A: FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface exploration program. During the site reconnaissance, the surface conditions were noted, and the approximate locations of the borings were determined. The exploratory borings were approximately located using existing boundary and other features as a guide and should be considered accurate only to the degree implied by the method used. The various field study methods performed are discussed below.

Exploratory Borings

A total of four (4) exploratory borings (BH-1 through BH-4) were drilled within the project sites on August 16, 2023. Three of the borings (BH-1 through BH-3) were advanced using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from approximately 26.5 feet to 51.5 feet below the existing ground surface (bgs). BH-4 was advanced using a 4-inch hand auger to depth of 5 feet bgs. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. Where appropriate, field descriptions and classifications have been modified to reflect laboratory test results.

Ring samples of the subsurface materials were obtained at frequent intervals in the exploratory borings using a drive sampler (2.4-inches inside diameter and 3.0-inches outside diameter) lined with sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches, using an automatic hammer. Samples are retained in brass rings (2.4-inches inside diameter and 1.0-inch in height). The central portion of the samples were retained and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Blow counts for each sample interval are presented on the logs of borings. Bulk samples of typical soil types were also obtained.

Standard Penetration Test (SPT) was also performed using a standard split-barrel sampler (1.4-inches inside diameter and 2.0-inches outside diameter). The mechanically driven hammer for the SPT sampler was 140 pounds, falling 30 inches for each blow. The recorded blow counts for every six inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings in the “BLOWS" column. The standard penetration test was performed in accordance with the ASTM Standard D1586 test method.

It should be noted that the exact depths at which material changes occur cannot always be established accurately. Changes in material conditions that occur between driven samples are indicated in the logs at the top of the next drive sample. A key to soil symbols and terms is presented as Drawing Nos. A-1a and A-1b, *Unified Soil Classification and Key to Boring Log Symbols*. The log of the exploratory boring is presented in Drawing Nos. A-2 through A-5, *Log of Borings*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
				CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

FIELD AND LABORATORY TESTS

C	Consolidation (ASTM D 2435)
CL	Collapse Potential (ASTM D 4546)
CP	Compaction Curve (ASTM D 1557)
CR	Corrosion, Sulfates, Chlorides (CTM 643-99; 417; 422)
CU	Consolidated Undrained Triaxial (ASTM D 4767)
DS	Direct Shear (ASTM D 3080)
EI	Expansion Index (ASTM D 4829)
M	Moisture Content (ASTM D 2216)
OC	Organic Content (ASTM D 2974)
P	Permeablility (ASTM D 2434)
PA	Particle Size Analysis (ASTM D 6913 [2002])
PI	Liquid Limit, Plastic Limit, Plasticity Index (ASTM D 4318)
PL	Point Load Index (ASTM D 5731)
PM	Pressure Meter
PP	Pocket Penetrometer
R	R-Value (CTM 301)
SE	Sand Equivalent (ASTM D 2419)
SG	Specific Gravity (ASTM D 854)
SW	Swell Potential (ASTM D 4546)
TV	Pocket Torvane
UC	Unconfined Compression - Soil (ASTM D 2166)
	Unconfined Compression - Rock (ASTM D 7012)
UU	Unconsolidated Undrained Triaxial (ASTM D 2850)
UW	Unit Weight (ASTM D 2937)
WA	Passing No. 200 Sieve (ASTM D1140)

BORING LOG SYMBOLS

SAMPLE TYPE

- STANDARD PENETRATION TEST Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method
- DRIVE SAMPLE 2.42" I.D. sampler (CMS).
- DRIVE SAMPLE No recovery
- BULK SAMPLE
- GROUNDWATER WHILE DRILLING
- GROUNDWATER AFTER DRILLING

DRILLING METHOD SYMBOLS

Auger Drilling | Mud Rotary Drilling | Dynamic Cone or Hand Driven | Diamond Core

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

CONSISTENCY OF COHESIVE SOILS

Descriptor	Unconfined Compressive Strength (tsf)	SPT Blow Counts	Pocket Penetrometer (tsf)	CA Sampler	Torvane (tsf)	Field Approximation
Very Soft	<0.25	< 2	<0.25	<3	<0.12	Easily penetrated several inches by fist
Soft	0.25 - 0.50	2 - 4	0.25 - 0.50	3 - 6	0.12 - 0.25	Easily penetrated several inches by thumb
Medium Stiff	0.50 - 1.0	5 - 8	0.50 - 1.0	7 - 12	0.25 - 0.50	Can be penetrated several inches by thumb with moderate effort
Stiff	1.0 - 2.0	9 - 15	1.0 - 2.0	13 - 25	0.50 - 1.0	Readily indented by thumb but penetrated only with great effort
Very Stiff	2.0 - 4.0	16 - 30	2.0 - 4.0	26 - 50	1.0 - 2.0	Readily indented by thumbnail
Hard	>4.0	>30	>4.0	>50	>2.0	Indented by thumbnail with difficulty

APPARENT DENSITY OF COHESIONLESS SOILS

Descriptor	SPT N_{60}- Value (blows / foot)	CA Sampler
Very Loose	<4	<5
Loose	4- 10	5 - 12
Medium Dense	11 - 30	13 - 35
Dense	31 - 50	36 - 60
Very Dense	>50	>60

MOISTURE

Descriptor	Criteria
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

PERCENT OF PROPORTION OF SOILS

Descriptor	Criteria
Trace (fine)/ Scattered (coarse)	Particles are present but estimated to be less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%
Mostly	50 to 100%

SOIL PARTICLE SIZE

Descriptor		Size
Boulder		> 12 inches
Cobble		3 to 12 inches
Gravel	Coarse	3/4 inch to 3 inches
	Fine	No. 4 Sieve to 3/4 inch
Sand	Coarse	No. 10 Sieve to No. 4 Sieve
	Medium	No. 40 Sieve to No. 10 Sieve
	Fine	No. 200 Sieve to No. No. 40 Sieve
Silt and Clay		Passing No. 200 Sieve

PLASTICITY OF FINE-GRAINED SOILS

Descriptor	Criteria
Nonplastic	A 1/8-inch thread cannot be rolled at any water content.
Low	The thread can barely be rolled, and the lump cannot be formed when drier than the plastic limit.
Medium	The thread is easy to roll, and not much time is required to reach the plastic limit; it cannot be rerolled after reaching the plastic limit. The lump crumbles when drier than the plastic limit.
High	It takes considerable time rolling and kneading to reach the plastic limit. The thread can be rerolled several times after reaching the plastic limit. The lump can be formed without crumbling when drier than the plastic limit.

CEMENTATION/ Induration

Descriptor	Criteria
Weak	Crumbles or breaks with handling or little finger pressure.
Moderate	Crumbles or breaks with considerable finger pressure.
Strong	Will not crumble or break with finger pressure.

NOTE: This legend sheet provides descriptions and associated criteria for required soil description components only. Refer to Caltrans Soil and Rock Logging, Classification, and Presentation Manual (2010), Section 2, for tables of additional soil description components and discussion of soil description and identification.

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Project Name
Wing Lane Elementary School
16605 Wing Lane, Valinda, California 91744

Project No.
23-31-214-01

Drawing No.
A-1b

Project ID: 138-01.GPJ; Template: LOG

Log of Boring No. BH-01

Dates Drilled: 8/3/2023 | Logged by: Derek Sobol | Checked By: Babak Abbasi

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 365 | Depth to Water (ft): 22.7

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. 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.

Depth (ft)	Description	Samples: Drive	Samples: Bulk	Blows	Moisture (%)	Dry Unit Wt. (pcf)	Other
0	**TOPSOIL AND GRASS, SIGNIFICANT ROOT SYSTEMS**						
1.5	**FILL (Af):** **CLAY WITH SAND (CL):** trace sand, very stiff, trace roots, dark brown.		X				PA, EI
5	**ALLUVIUM (Qal):** **CLAY WITH SAND (CL):** medium plasticity, slightly damp, little sand, very stiff, dark brown.	X		9/14/20	28	87	wa(fc=82.5%)
10		X		8/14/20	20	104	
15		X		10/10/22	22	102	
20	**SANDY CLAY (CL):** medium plasticity, some sand, slightly damp, very stiff, dark brown.	X		3/10/19	19	109	
22.7	- at 22.7': groundwater enctountered.						
25	**CLAY WITH SAND (CL):** medium plasticity, little sand, wet, stiff to very stiff, dark brown.	X		4/8/9			
30		X		8/5/15	34	89	

Log of Boring No. BH-01

Dates Drilled: 8/3/2023 Logged by: Derek Sobol Checked By: Babak Abbasi

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 365 Depth to Water (ft): 22.7

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. 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.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		CLAY WITH SAND (CL): medium plasticity, little sand, wet, very stiff.			5/10/12			
40		**CLAY (CL):** medium to high plasticity, few sand, wet, trace sand, stiff, dark brown.			2/6/6	31	91	
45					3/5/6			
50					4/8/11	27	98	

End of boring at 51.5 feet below ground surface.
Groundwater encountered at 22.7 feet below ground surface.
Borehole backfilled with cement grout on 8/3/2023.

Log of Boring No. BH-02

Dates Drilled: 8/3/2023 | Logged by: Derek Sobol | Checked By: Babak Abbasi

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 365 | Depth to Water (ft): 22.7

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. 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.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOP SOIL AND GRASS, SIGNIFICANT ROOT SYSTEMS						
		FILL (Af): **CLAY WITH SAND (CL):** medium plasticity, damp, very stiff, dark brown.			9/13/24	16	115	CR
5		**ALLUVIUM (Qal):** **CLAY WITH SAND (CL):** medium plasticity, few sand, very stiff, dark brown.			13/24/30	17	78	DS, C
					6/8/10	22	99	
10					9/10/20	19	111	wa(fc=74.2%)
15					5/14/18	23	103	
20					7/9/15	23	104	
		- at 22.7': groundwater encountered.						
25		**SANDY CLAY (CL):** low plasticity, some sand, stiff, dark brown.			2/5/6			

End of boring at 26.5 feet below ground surface.
Groundwater encountered at 22.7 feet below ground surface.
Borehole backfilled with cement grout on 8/3/2023.

Log of Boring No. BH-03

Dates Drilled: 8/3/2023 Logged by: Derek Sobol Checked By: Babak Abbasi

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 365 Depth to Water (ft): 22.6

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. 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.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOP SOIL AND GRASS						
		FILL (Af): **CLAY WITH SAND (CL):** medium plasticity, damp, dark brown.						CP
5		**ALLUVIUM (Qal):** **CLAY WITH SAND (CL):** medium plasticity, little sand, very stiff, dark brown.	■		17/35/39	21	110	
10			■		5/9/8	19	111	wa(fc=75.4%)
15			■		10/14/15	21	106	
20			■		4/11/23	22	104	
		- at 22.6': groundwater encountered.						
25			■		4/6/8			
		End of boring at 26.5 feet below ground surface. Groundwater encountered at 22.6 feet below ground surface. Borehole backfilled with cement grout on 8/3/2023.						

Converse Consultants

Project Name
Wing Lane Elementary School
16605 Wing Lane, Valinda, California 91744

Project No. **23-31-214-00**

Figure No. **A-4**

Log of Boring No. BH-04

Dates Drilled: 8/3/2023 Logged by: Derek Sobol Checked By: Babak Abbasi

Equipment: Hand Auger Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 365 Depth to Water (ft): N/A

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. 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.	SAMPLES DRIVE	SAMPLES BULK	BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		TOPSOIL AND GRASS, SIGNIFICANT ROOT SYSTEMS						
		FILL (Af): **CLAY WITH SAND (CL):** trace sand, very stiff, trace roots, dark brown.						R
5		End of boring at 5 feet below ground surface. No groundwater encountered. Borehole backfilled with soil trimmings on 8/3/2023.						

Converse Consultants

Project Name
Wing Lane Elementary School
16605 Wing Lane, Valinda, California 91744

Project No. **23-31-214-00**

Figure No. **A-5**

Appendix B

Laboratory Testing Program

APPENDIX B: LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical requirements of the project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the laboratory tests conducted for this project.

Moisture Content and Dry Density

Results of moisture content and dry density tests performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure of the *in-situ* dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Soil Corrosivity

One (1) representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including chloride concentrations, and soluble sulfate. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EGL in Arcadia, California. The test results received from EGL are included in the following table:

Table No. B-1, Summary of Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) (%)	Saturated Resistivity (Caltrans 643) Ohm-cm
BH-2	1.5-5	7.37	140	0.005	680

Percent Finer Than Sieve No. 200

The percent finer than sieve No. 200 test was performed on three (3) selected soil samples to aid in the classification of the on-site soils and to estimate other engineering parameters. Testing was performed in general accordance with the ASTM Standard D1140 test method. The test results are presented in the boring logs.

Table No. B-2, Summary of Percent Passing Sieve #200 Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Passing Sieve No. 200
BH-1	10.0-11.5	Clay with sand (CL)	82.5%
BH-2	10.0-11.5	Clay with sand (CL)	74.2%
BH-3	10.0-11.5	Clay with sand (CL)	74.5%

R-value

One (1) representative bulk soil sample was tested for resistance value (R-value) in accordance with ASTM D2844-18 Standard. This test is designed to provide a relative measure of soil strength for use in pavement design. The test results are shown in the following table:

Table No. B-3, R-value Test Result

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-4	1.5-5	Clay with sand (CL)	8

Expansion Index Test

One (1) representative bulk sample was tested to evaluate the expansion potential of material encountered at the site. The test was conducted in accordance with ASTM D4829 Standard. Test results are presented in the following table:

Table No. B-4, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-1	1.5-5	Clay with sand (CL)	105.0	High

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on one (1) selected sample. Testing was performed in general accordance with the ASTM Standard C136 test method. Grain-size curve is shown in Drawing No. B-1, *Grain Size Distribution Results*.

Table No. B-5, Summary of Grain Size Distribution Test Results

Boring No.	Depth (feet)	Soil Classification	% Gravel	% Sand	%Silt	%Clay
BH-1	1.5-5	Clay with sand (CL)	0.0	20.6	79.4	

Maximum Dry Density Test

One (1) laboratory maximum dry density-moisture content relationship test was performed on a representative bulk sample of the upper 5 feet of soil material. The testing was conducted in accordance with ASTM Standard D1557 laboratory procedure. The test result is presented on Drawing No. B-2, *Moisture-Density Relationship Results*.

Table No. B-6, Summary of Moisture-Density Relationship Results

Boring No.	Depth (feet)	Soil Description	Optimum Moisture (%)	Maximum Density (lb/cft)
BH-3	1.5-5	Clay with sand (CL)	11.5	121.0

Direct Shear

Direct shear test was performed on one (1) relatively undisturbed samples at soaked moisture conditions. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The sand samples were then sheared at a constant strain rate of 0.025 inch/minute. Shear deformation was recorded until a maximum of about 0.250-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawing No. B-3, *Direct Shear Test Results,* and the following table:

Table No. B-7, Summary of Direct Shear Test Results

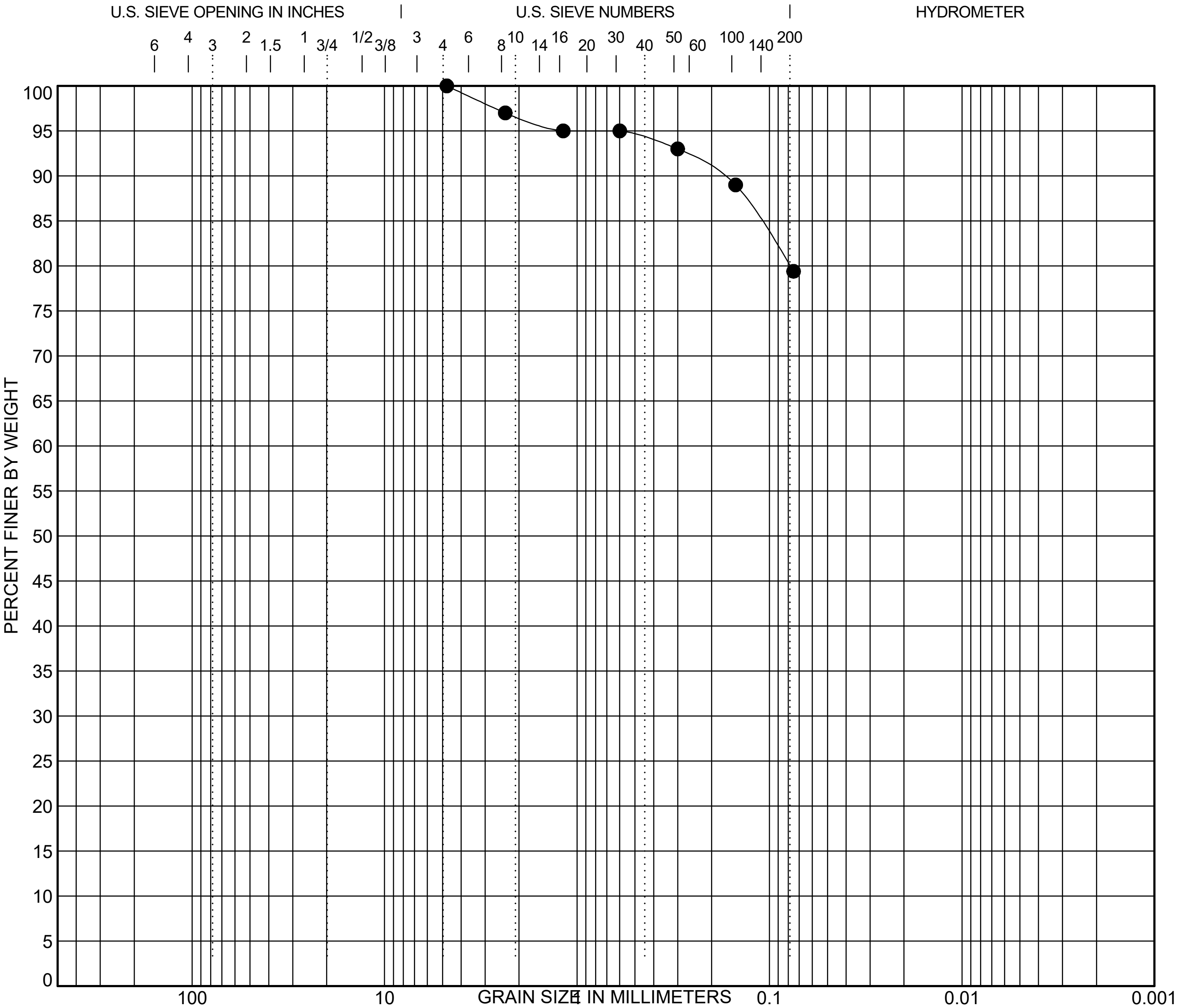
Boring No.	Depth (feet)	Soil Classification	Ultimate Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-2	5.0-6.5	Clay with sand (CL)	25	950

Consolidation Test

Consolidation test was performed on one (1) relatively undisturbed sample. Data obtained from this test was used to evaluate the settlement characteristics of the foundation soils under load. Preparation for this test involved trimming the sample and placing the 1-inch-high brass ring into the test apparatus, which contained porous stones, both top and bottom, to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. The sample was tested at field and submerged conditions. The test results, including sample density and moisture content, are presented in Drawing No. B-4, *Consolidation Test Results*.

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period of time.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-01**	**5**	**SANDY CLAY (CL)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-01**	**5**	**4.75**				**0.0**	**20.6**	**79.4**	

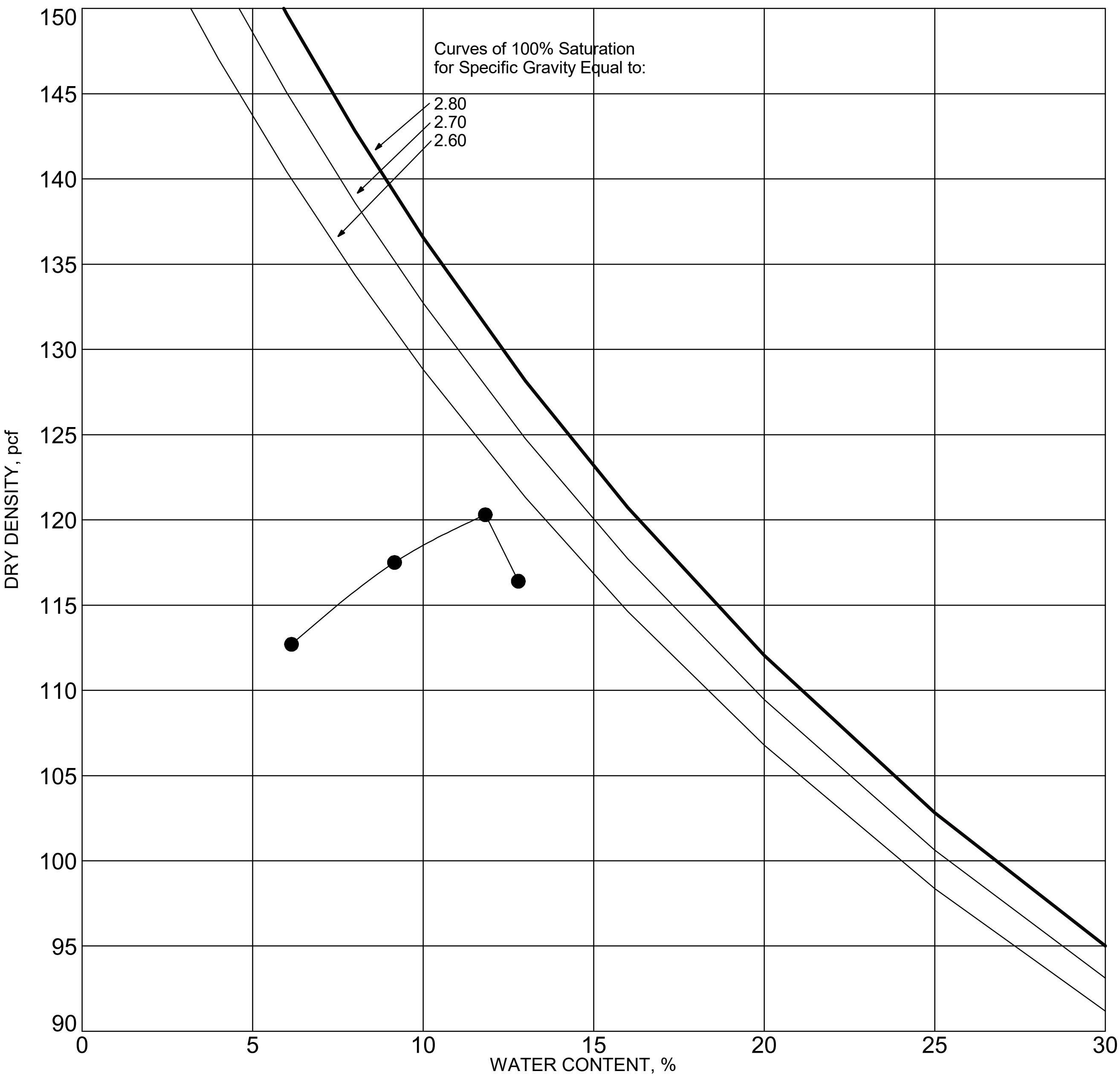
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Project Name
Wing Lane Elementary School
16605 Wing Lane, Valinda, California 91744

Project No. **23-31-214-00**

Drawing No. **B-1**



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-03	5	CLAY (CL)	D1557 Method B	11.5	121

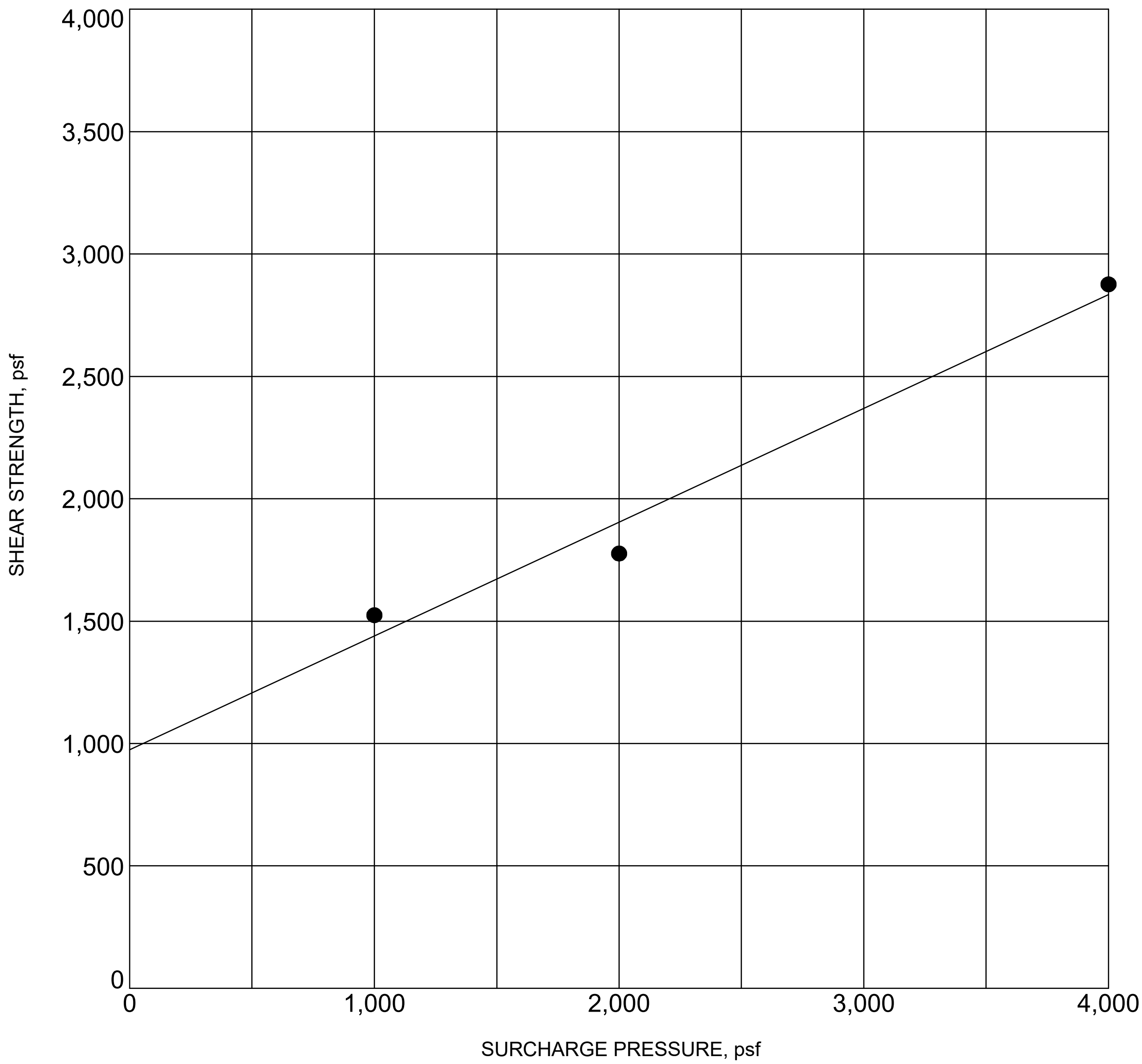
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Project Name
Wing Lane Elementary School
16605 Wing Lane, Valinda, California 91744

Project No.
23-31-214-00

Drawing No.
B-2



BORING NO. :	**BH-02**	DEPTH (ft) :	**5-6.5**
DESCRIPTION :	**CLAY WITH SAND (CL)**		
COHESION (psf) :	**950**	FRICTION ANGLE (degrees):	**25**
MOISTURE CONTENT (%) :	**17**	DRY DENSITY (pcf) :	**78**

NOTE: Ultimate Strength.

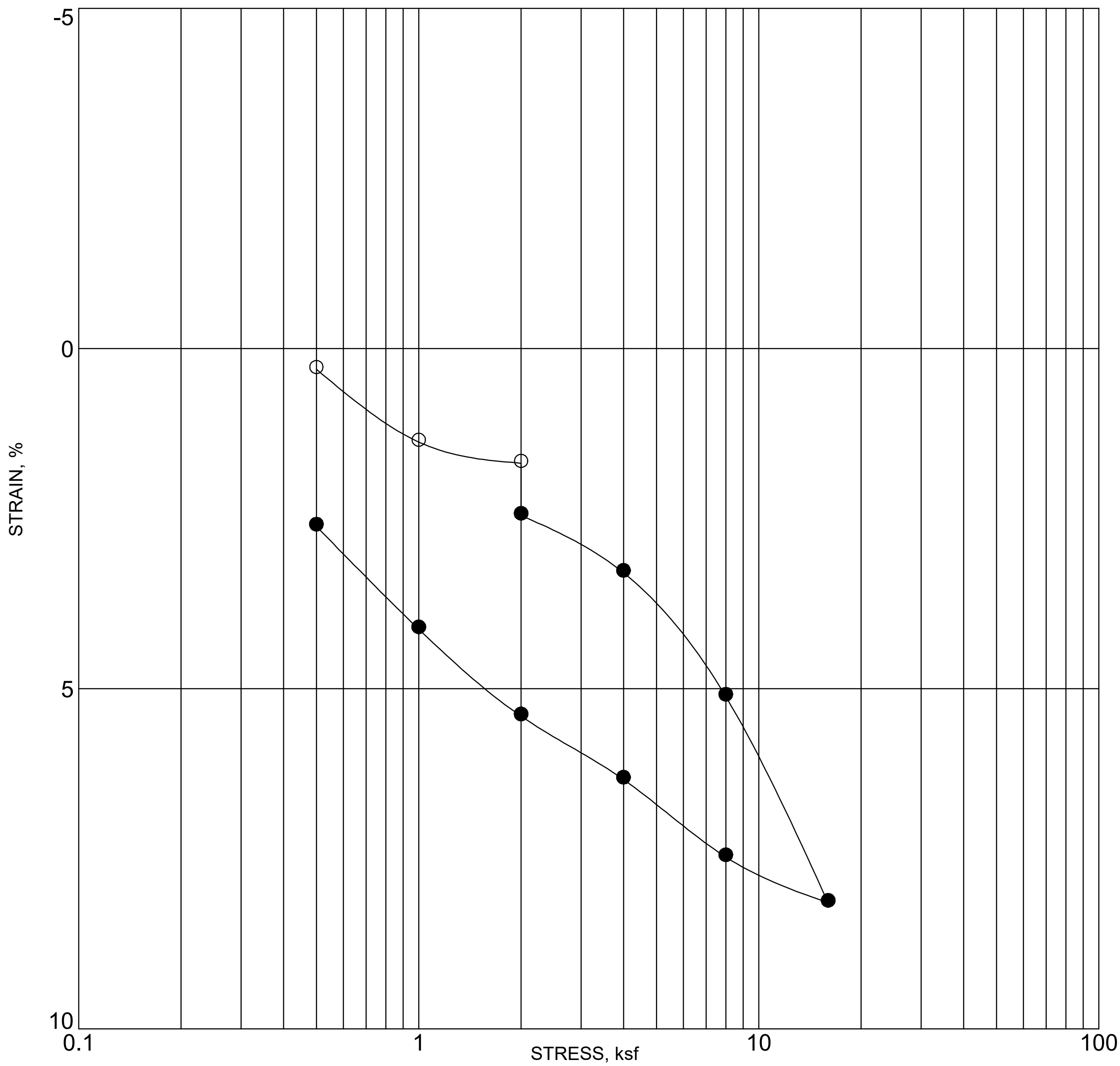
DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
Wing Lane Elementary School
16605 Wing Lane, Valinda, California 91744

Project No.
23-31-214-00

Drawing No.
B-3



BORING NO. :	BH-02	DEPTH (ft) :	5-6.5
DESCRIPTION :	CLAY WITH SAND (CL)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	17	78	41	1.08
FINAL				

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
Wing Lane Elementary School
16605 Wing Lane, Valinda, California 91744

Project No. **23-31-214-00**

Drawing No. **B-4**

Appendix C

Earthwork Specifications

APPENDIX C: EARTHWORK SPECIFICATIONS

Scope of Work

The work includes all labor, supplies and construction equipment required to construct the building pads in a good, workman-like manner, as shown on the drawings and herein specified. The major items of work covered in this section include the following:

- Site Inspection
- Authority of Geotechnical Engineer
- Site Clearing
- Excavations
- Preparation of Fill Areas
- Placement and Compaction of Fill
- Observation and Testing

Site Inspection

1. The Contractor shall carefully examine the site and make all inspections necessary, in order to determine the full extent of the work required to make the completed work conform to the drawings and specifications. The Contractor shall satisfy himself as to the nature and location of the work, ground surface and the characteristics of equipment and facilities needed prior to and during prosecution of the work. The Contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications must be brought to the owner's attention in order to clarify the exact nature of the work to be performed.

2. This G*eotechnical Study Report* by Converse Consultants may be used as a reference to the surface and subsurface conditions on this project. The information presented in this report is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the boring logs. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information shall not relieve the Contractor from performing such independent investigation and evaluation to satisfy himself as to the nature of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

Authority of the Geotechnical Engineer

1. The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.

2. As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed, and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material; and (c) have the authority to approve or reject soils proposed for use in building areas.

3. The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the Contractor and (c) the matters of compensation.

Site Clearing

1. Clearing and grubbing shall consist of the removal from building areas to be graded of all existing structures, pavement, utilities, and vegetation.

2. Organic and inorganic materials resulting from the clearing and grubbing operations shall be hauled away from the areas to be graded.

Excavations

1. Based on observations made during our field explorations, the surficial soils can be excavated with conventional earthwork equipment.

Preparation of Fill Areas

1. All organic material, organic soils, incompetent alluvium, undocumented fill soils and debris should be removed from the proposed building areas.

2. Due to the undocumented fills encountered at the site, we recommend over-excavation for structure footings to be at least five (5) feet below the existing grade or 3 feet below bottom of footing, whichever is deeper. Deeper removal and re-compaction will be needed if firm soil conditions are not exposed on the excavation bottom. Over-excavation should extend at least five (5) feet laterally beyond the limits of perimeter footings where feasible. The on-site soil is not considered suitable for re-use in the upper 2 feet of finished grade for support of slabs, foundations, walkways, and pavements. Due to high expansion potential of in-situ soils, Converse recommends the upper 2 feet of finished grade to be replaced and compacted with import fill with an expansion potential less than 20.

3. Excavation activities should not disturb existing utilities, buildings, and remaining structures. Existing utilities should be removed and adequately capped at the project boundary line or salvaged/rerouted so as not to interfere with the proposed improvements if possible.

4. The subgrade in all areas to receive fill shall be scarified to a minimum depth of six inches, the soil moisture adjusted within three (3) percent above optimum, and then compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method.

5. Compacted fill may be placed on native soils that have been properly scarified and re-compacted as discussed above.

6. All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

Placement and Compaction of Fill

1. Compacted fill placed for the support of footings, slabs-on-grade, exterior concrete flatwork, and driveways will be considered structural fill. Structural fill may consist of approved on-site soils at depths greater than 2 feet below foundations, slabs, sidewalks or pavements. Imported fill that meets the criteria indicated below may be used as structural fill.

2. Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer shall be placed in layers on approved earth materials. Soils used as compacted structural fill shall have the following characteristics:

3. All fill soil particles shall not exceed three (3) inches in nominal size and shall be free of organic matter and miscellaneous inorganic debris and inert rubble.

4. Imported fill materials shall have an Expansion Index (EI) less than 20. All imported fill should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM Standard D1557) at about three percent above optimum moisture.

5. Fill soils shall be evenly spread in maximum 8-inch lifts, watered, or dried as necessary, mixed, and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

6. All fill placed at the site shall be compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. The

on-site soils shall be moisture conditioned at approximate three (3) percent above the optimum moisture content.

7. Representative samples of materials being used as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 compaction method.

8. Fill materials shall not be placed, spread, or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations shall not resume until the Geotechnical Engineer approves the moisture and density conditions of the previously placed fill.

9. It shall be the Grading Contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with job specifications, all berms have been properly constructed, and all associated drainage devices meet the requirements of the Civil Engineer.

Trench Backfill

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

1. Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

2. Trench backfill shall be compacted to a minimum relative compaction of 90 percent as per ASTM Standard D1557 test method.

3. Rocks larger than one inch should not be placed within 12 inches of the top of the pipeline or within the upper 12 inches of pavement or structure subgrade. No more than 30 percent of the backfill volume shall be larger than 3/4-inch in largest dimension. Rocks shall be well mixed with finer soil.

4. The pipe design engineer should select bedding material for the pipe. Bedding materials generally should have a Sand Equivalent (SE) greater than or equal to 30, as determined by the ASTM Standard D2419 test method.

5. Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density

specified herein. The backfill materials shall be brought to between optimum and three percent above optimum, then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight inches. Each layer shall be evenly spread, moistened, or dried as necessary, and then tamped or rolled until the specified density has been achieved.

6. The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

7. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.

8. Observation and field tests should be performed by geotechnical representative during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

9. It should be the responsibility of the Contractor to maintain safe conditions during cut and/or fill operations.

10. Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

Observation and Testing

1. During the progress of grading, the Geotechnical Engineer will provide observation of the fill placement operations.

2. Field density tests will be made during grading to provide an opinion on the degree of compaction being obtained by the contractor. Where compaction of less than specified herein is indicated, additional compactive effort with adjustment of the moisture content shall be made as necessary, until the required degree of compaction is obtained.

3. A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.

Appendix D

Seismic Hazard Analysis Results

Site Class Determination

SIMPLIFIED EVALUATION OF SITE CLASS AND GEOTECHNICAL DESIGN PARAMETERS USING STANDARD PENETRATION TEST (SPT) DATA

PROJECT INFORMATION

Project Name	Wing Lane Elementary School
Project No.	23-31-214-01
Project Location	16605 Wing Lane, Valinda, California 91744
Analyzed By	B. Abbasi
Reviewed By	S. Sivathasan

GENERAL INPUT DATA

Analysis Description	
Boring ID No.	BH-1
Ground Surface Elevation	365.00 feet
Proposed Grade Elevation	365.00 feet
Total Unit Weight of New Fill	115.00 pcf
Borehole Diameter	8.00 inches
Hammer Weight	140.00 pounds
Hammer Drop	30.00 inches
Hammer Efficiency Ratio, ER	80.00 %
Hammer Dist. to Ground Surface	5.00 feet
Groundwater Depth During Test	22.70 feet

SPT BLOW COUNT AND RELATIVE DENSITY

- Based on the recommendations by Idriss and Boulanger (2008), the normalized SPT blow count is defined as $(N_1)_{60} = N_{60}\,C_N$

where $N_{60} = N_{field}\,C_E\,C_B\,C_R\,C_S$

and the relative density of granular soils is estimated as

$D_r = 15\,[(N_1)_{60}]^{0.5}$ in percent

SHEAR WAVE VELOCITY AND SITE CLASSIFICATION

- Shear wave velocities are estimated based on empirical correlations with SPT N_{60} values for various soil types, as derived by Brandenberg, Bellana and Shantz (2010) from regression analyses.
- Site classification is analyzed using the method by Boore (2004).

Ave. Shear Wave Velocity (Top Depth d), $V_{s,d}$ = 236.50 m/s

Ave. Shear Wave Velocity (Top 30 m), $V_{s,30} = 10^{\,a + b \log (V_{s,d})}$

where a = 0.01389

b = 1.02370

Coefficients a and b vary with depth, as derived by Boore (2004).

Computed $V_{s,30}$ = 278.0 m/s

Site Class = D

SOIL STRENGTH AND DEFORMATION MODULUS PARAMETERS

- For granular soils, the effective peak friction angle, ϕ', is estimated from correlations with the normalized SPT blow count, $(N_1)_{60}$ from Bowles (1996) and recommended adjustments from Caltrans Geotechnical Manual (2014).
- For cohesive soils, the undrained shear strength, S_u, is based on field measurements with torvane or pocket penetrometer. When only SPT values are available, S_u is estimated using the correlation chart with $(N_1)_{60}$ value provided in the Caltrans Geotechnical Manual (2014).
- Modulus of Elasticity, E_s, values for granular soils and cohesive soils are estimated from correlations with SPT N_{60} and undrained shear strength, S_u, respectively summarized by Bowles (1996).
- Shear Modulus, $G = E_s / [3\,(1 - 2\mu)]$ and Bulk Modulus, $K = E_s / [2\,(1 + \mu)]$ based on theory of elasticity where μ is the Poisson's ratio of the soil. Typical values of Poisson's ratio are estimated from various references.

REFERENCES:

1. AASHTO, 1988. Manual on Subsurface Investigations.
2. Boore, D.M., 2004. "Estimating Vs(30) (or NEHRP Site Classes) from shallow velocity models (depths < 30 m)", Bulletin of Seismological Society of America, 94(2), pp. 591-597.
3. Brandenberg, S.J., Bellana, N. and Shantz, T., 2010. "Shear Wave Velocity as a Statistical Function of Standard Penetration Test Resistance and Vertical Effective Stress at Caltrans Bridge Sites," PEER Report 201/03.
4. FHWA, 2002. Subsurface Investigations Reference Manual, Geotechnical Site Characterization.
5. Idriss, I.M. and Boulanger, R.W., 2008, "Soil Liquefaction During Earthquakes", EERI Monograph MNO-12.

INPUT SOIL PROFILE DATA

Depth to Top of Soil Layer (feet)	Depth to Bottom of Soil Layer (feet)	Material Type USCS Group Symbol (ASTM D2487)	Total Soil Unit Weight γ_t (pcf)	Type of Soil Sampler	Field Blow Count N_{field} (blows/ft)	Pocket Penetrometer Shear Test Results PP (tsf)	Torvane Shear Test Results TV (tsf)
0.00	5.00	CL	111.3	MCal	34.0		
5.00	10.00	CL	124.8	MCal	32.0		
10.00	15.00	CL	124.4	MCal	29.0		
15.00	20.00	CL	129.7	MCal	32.0		
20.00	25.00	CL	125.0	MCal	29.0		
25.00	30.00	CL	119.3	SPT1	17.0		
30.00	35.00	CL	119.3	MCal	23.0		
35.00	40.00	CL	119.2	SPT1	22.0		
40.00	45.00	CL	122.0	MCal	12.0		
45.00	48.00	CL	124.4	SPT1	11.0		
48.00	51.50	CL	124.4	MCal	16.0		

ESTIMATED GEOTECHNICAL DESIGN PARAMETERS

Bottom of Soil Layer Elevation (feet)	Soil Depth During Test (feet)	SPT Corr. For Vert. Stress C_N	SPT Corr. For Hammer Energy C_E	SPT Corr. For Borehole Size C_B	SPT Corr. For Rod Length C_R	SPT Corr. For Sampling Method C_S	Corrected SPT Blow Count N_{60}	Normalized SPT Blow Count $(N_1)_{60}$	Relative Density D_r (%)	Shear Wave Velocity V_s (ft/s)	Effective Peak Friction Angle ϕ' (deg)	Undrained Shear Strength S_u (ksf)	Apparent Density / Soil Consistency Description FHWA (2002) and AASHTO (1988)	Poisson's Ratio μ	Modulus of Elasticity E_s (ksf)	Shear Modulus G (ksf)	Bulk Modulus K (ksf)
360.00	2.50	1.700	1.333	1.150	0.750	0.650	25.4	43.2		574.01		3.10	Very Stiff Clay	0.45	2,250.87	7,502.91	776.16
355.00	7.50	1.518	1.333	1.150	0.800	0.650	25.5	38.7		692.44		3.11	Very Stiff Clay	0.45	2,261.42	7,538.00	780.00
350.00	12.50	1.158	1.333	1.150	0.850	0.650	24.6	28.4		750.10		2.99	Very Stiff Clay	0.45	2,161.22	7,204.00	745.00
345.00	17.50	0.970	1.333	1.150	0.950	0.650	30.3	29.4		834.32		3.72	Very Stiff Clay	0.45	2,767.68	9,226.00	954.00
340.00	22.50	0.862	1.333	1.150	0.950	0.650	27.5	23.7		847.77		3.36	Very Stiff Clay	0.45	2,467.09	8,224.00	851.00
335.00	27.50	0.807	1.333	1.150	0.950	1.000	24.8	20.0		846.13		3.02	Very Stiff Clay	0.45	2,181.91	7,273.00	752.00
330.00	32.50	0.772	1.333	1.150	1.000	0.650	22.9	17.7		843.39		2.78	Very Stiff Clay	0.45	1,987.19	6,624.00	685.00
325.00	37.50	0.741	1.333	1.150	1.000	1.000	33.7	25.0		934.12		4.16	Hard Clay	0.45	6,236.08	20,787.00	2,150.00
320.00	42.50	0.713	1.333	1.150	1.000	0.650	12.0	8.5		745.25		1.39	Stiff Clay	0.35	827.01	919.00	306.00
317.00	46.50	0.692	1.333	1.150	1.000	1.000	16.9	11.7		814.50		2.02	Very Stiff Clay	0.45	1,346.25	4,488.00	464.00
313.50	49.75	0.676	1.333	1.150	1.000	0.650	15.9	10.8		810.29		1.90	Stiff Clay	0.35	1,248.90	1,388.00	463.00

Mapped Seismic Parameters

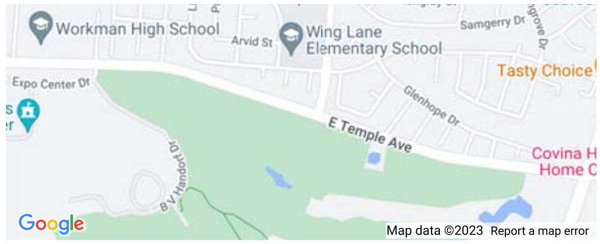
⚠ This is a beta release of the new ATC Hazards by Location website. Please contact us with feedback.

ℹ The ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

ATC Hazards by Location

Search Information

Coordinates:	34.0297, -117.9281
Elevation:	364 ft
Timestamp:	2023-08-21T16:18:56.815Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D



Basic Parameters

Name	Value	Description
S_S	1.732	MCE_R ground motion (period=0.2s)
S_1	0.62	MCE_R ground motion (period=1.0s)
S_{MS}	1.732	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.155	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.91	Coefficient of risk (0.2s)
CR_1	0.905	Coefficient of risk (1.0s)
PGA	0.737	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.811	Site modified peak ground acceleration
T_L	8	Long-period transition period (s)
SsRT	1.732	Probabilistic risk-targeted ground motion (0.2s)
SsUH	1.904	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.306	Factored deterministic acceleration value (0.2s)
S1RT	0.62	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.685	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.722	Factored deterministic acceleration value (1.0s)
PGAd	0.923	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Please note that the ATC Hazards by Location website will not be updated to support ASCE 7-22. Find out why.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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Seismic Hazard Deaggregation

U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Please also see the new USGS Earthquake Hazard Toolbox for access to the most recent NSHMs for the conterminous U.S. and Hawaii.

Input

Edition

Dynamic: Conterminous U.S. 2014 (update) (4.2.0)

Latitude

Decimal degrees

34.0297

Longitude

Decimal degrees, negative values for western longitudes

-117.9281

Site Class

259 m/s (Site class D)

Spectral Period

Peak Ground Acceleration

Time Horizon

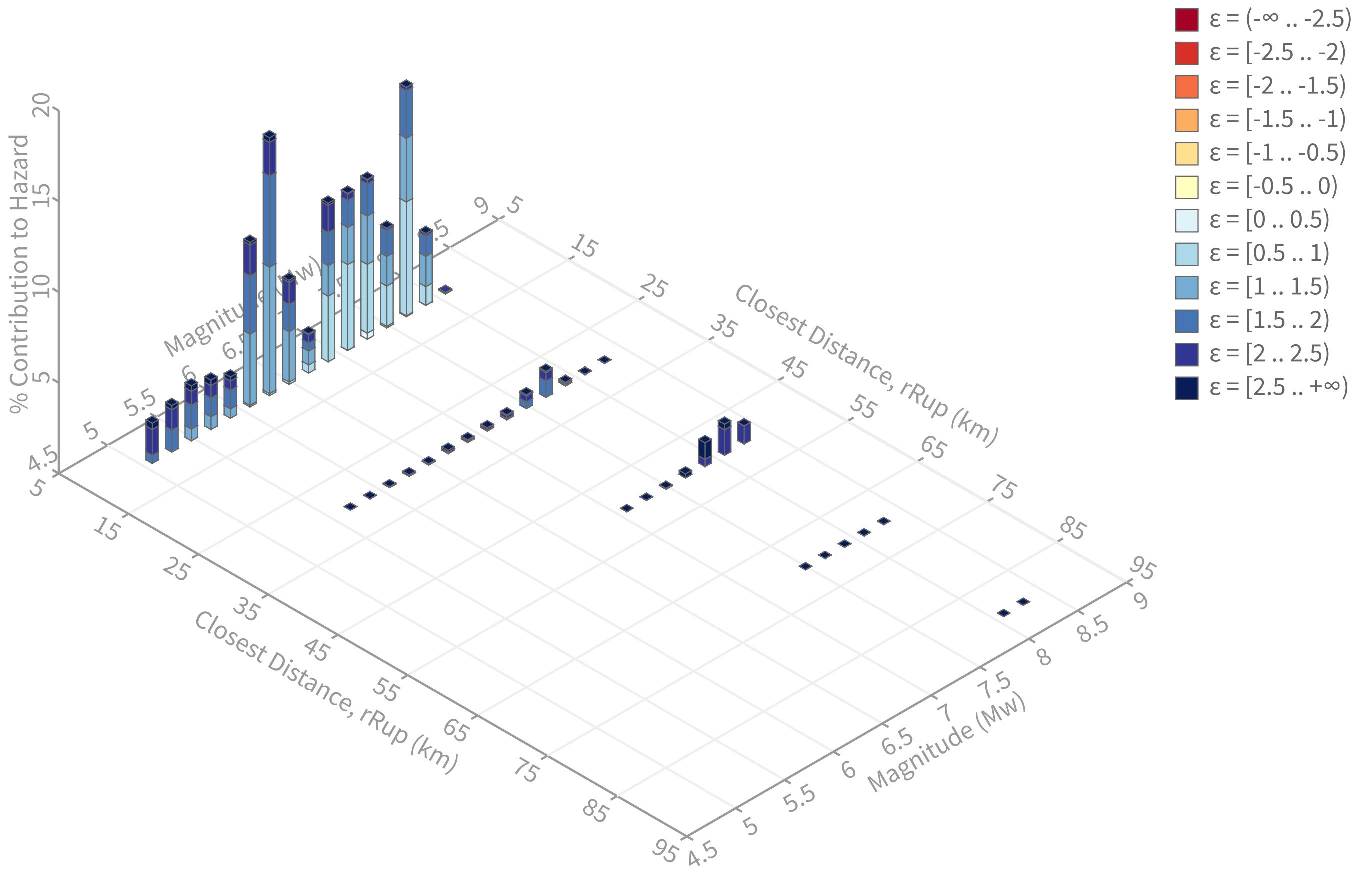
Return period in years

2475

^ Deaggregation

Component

Total


% Contribution to Hazard
Closest Distance, rRup (km)
Magnitude (Mw)
ε = (-∞ .. -2.5)
ε = [-2.5 .. -2)
ε = [-2 .. -1.5)
ε = [-1.5 .. -1)
ε = [-1 .. -0.5)
ε = [-0.5 .. 0)
ε = [0 .. 0.5)
ε = [0.5 .. 1)
ε = [1 .. 1.5)
ε = [1.5 .. 2)
ε = [2 .. 2.5)
ε = [2.5 .. +∞)

Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.78268891 g

Recovered targets

Return period: 2917.8423 yrs
Exceedance rate: 0.00034271901 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.09 %

Mean (over all sources)

m: 6.83
r: 12.14 km
ε_0: 1.48 σ

Mode (largest m-r bin)

m: 6.26
r: 8.67 km
ε_0: 1.51 σ
Contribution: 14.21 %

Mode (largest m-r-ε_0 bin)

m: 6.26
r: 7.64 km
ε_0: 1.14 σ
Contribution: 6.97 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
ε4: [-1.0 .. -0.5)
ε5: [-0.5 .. 0.0)
ε6: [0.0 .. 0.5)
ε7: [0.5 .. 1.0)
ε8: [1.0 .. 1.5)
ε9: [1.5 .. 2.0)
ε10: [2.0 .. 2.5)
ε11: [2.5 .. +∞]

Deaggregation Contributors

Source Set ↳ Source	Type	r	m	ε_0	lon	lat	az	%
UC33brAvg_FM31	System							41.44
Whittier alt 1 [6]		7.65	6.63	1.26	117.961°W	33.966°N	203.07	11.44
Whittier alt 1 [5]		8.02	7.32	0.90	117.931°W	33.957°N	182.02	6.05
San Jose [2]		4.85	6.98	0.95	117.881°W	34.043°N	71.23	4.65
Sierra Madre [2]		14.40	7.68	1.58	117.923°W	34.159°N	1.95	2.73
Puente Hills [1]		9.46	7.15	0.79	117.957°W	33.944°N	195.36	2.59
San Andreas (Mojave S) [12]		44.26	8.07	2.43	117.711°W	34.385°N	26.71	2.02
Chino alt 1 [0]		14.27	6.44	2.07	117.752°W	34.028°N	90.79	1.92
Compton [0]		18.76	7.24	1.46	118.112°W	33.746°N	208.31	1.87
Raymond [0]		16.25	7.19	1.94	117.991°W	34.166°N	339.13	1.05
UC33brAvg_FM32	System							39.56
Whittier alt 2 [5]		7.84	7.20	1.08	117.953°W	33.963°N	197.40	12.10
San Jose [2]		4.85	6.98	0.95	117.881°W	34.043°N	71.23	4.53
Richfield [1]		11.57	6.17	1.35	117.870°W	33.882°N	161.93	4.44
Puente Hills (Coyote Hills) [0]		9.98	7.13	0.86	117.868°W	33.919°N	155.71	3.37
Sierra Madre [2]		14.40	7.69	1.58	117.923°W	34.159°N	1.95	2.74
San Andreas (Mojave S) [12]		44.26	8.07	2.43	117.711°W	34.385°N	26.71	2.02
Compton [0]		18.76	7.30	1.43	118.112°W	33.746°N	208.31	2.01
Chino alt 2 [0]		15.78	6.83	1.98	117.751°W	34.030°N	89.86	1.25
Puente Hills (LA) [0]		16.62	7.17	1.76	118.116°W	33.990°N	255.90	1.16
Raymond [0]		16.25	7.18	1.95	117.991°W	34.166°N	339.13	1.14
UC33brAvg_FM32 (opt)	Grid							9.53
PointSourceFinite: -117.928, 34.070		6.84	5.62	1.65	117.928°W	34.070°N	0.00	1.95
PointSourceFinite: -117.928, 34.070		6.84	5.62	1.65	117.928°W	34.070°N	0.00	1.95
PointSourceFinite: -117.928, 34.097		8.73	5.73	1.89	117.928°W	34.097°N	0.00	1.01
PointSourceFinite: -117.928, 34.097		8.73	5.73	1.89	117.928°W	34.097°N	0.00	1.01
UC33brAvg_FM31 (opt)	Grid							9.47
PointSourceFinite: -117.928, 34.070		6.88	5.59	1.67	117.928°W	34.070°N	0.00	2.04
PointSourceFinite: -117.928, 34.070		6.88	5.59	1.67	117.928°W	34.070°N	0.00	2.04

Site-Specific Spectral Accelerations

AN NSF+USGS CENTER

Site-Specific MCE_R & Design Response Spectral Accelerations

Wing Lane Elementary School

Input Parameters

Coordinates	34.030, -117.928
V_{S30}	278 m/s

Values used in Computation

V_{S30}	278 m/s
Z1.0	250 m
Z2.5	2550 m

Calculated Results

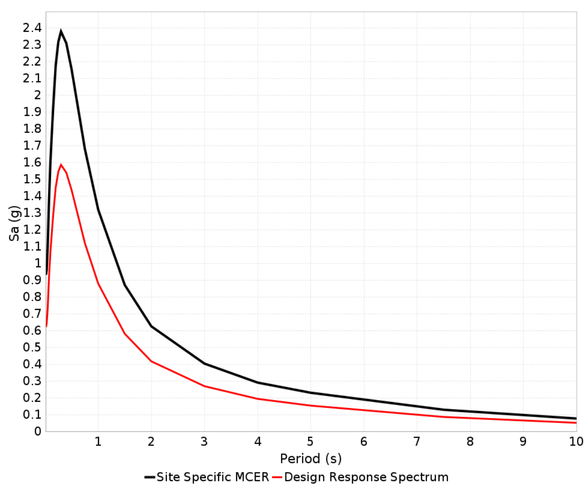
Site-Specific Design Parameters

S_{DS}	1.427	S_{MS}	2.141
S_{D1}	0.879	S_{M1}	1.319

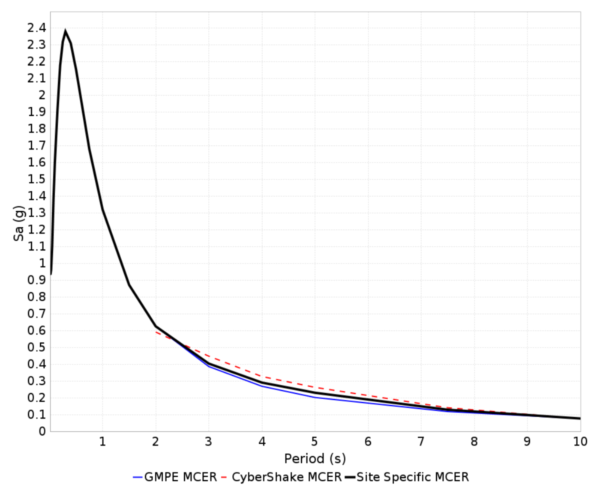
MCE_G Peak Ground Acceleration (Sect. 21.5)

PGA_M 0.806 g

MCE_R Response Spectra



MCE_R Response Spectra Comparison



Site-Specific MCE_R & Design Response Spectral Accelerations

MCE_R Response Spectrum Table

Period (s)	GMPE Sa (g)	CyberShake Sa (g)	Site-Specific MCE_R Sa* (g)
0.01	0.936		0.936
0.02	0.937		0.937
0.03	0.961		0.961
0.05	1.106		1.106
0.075	1.383		1.383
0.1	1.600		1.600
0.15	1.915		1.915
0.2	2.178		2.178
0.25	2.319		2.319
0.3	2.379		2.379
0.4	2.309		2.309
0.5	2.153		2.153
0.75	1.679		1.679
1.0	1.319		1.319
1.5	0.871		0.871
2.0	0.625	0.591	0.625
3.0	0.386	0.447	0.403
4.0	0.268	0.326	0.290
5.0	0.202	0.262	0.230
7.5	0.118	0.141	0.129
10.0	0.076	0.076	0.076

* Site-Specific MCER response spectrum obtained using obtained weighted geometric averaging procedure. See User Guide: https://data2.scec.org/ugms-mcerGM-tool_v18.4/guide

Site-Specific MCE_R & Design Response Spectral Accelerations

Important Note

The site-specific, design response spectral acceleration, Sa, returned by this tool for user-specified inputs, must be compared to the minimum Sa requirement described in Section 21.3 of ASCE 7-16 (second and third paragraphs). This minimum Sa is computed as 80% of the design response spectrum derived from the SDS, SD1, and TL values obtained from the ASCE tool at https://asce7hazardtool.online/. The larger of the site-specific Sa and the 80% minimum Sa at each period, T, is the final design response spectral acceleration. This final Sa x 1.5 is the final MCER response spectral acceleration.

About UGMS

The UGMS MCER tool was developed by the SCEC Committee for Utilization of Ground Motion Simulations (or "UGMS Committee") from research supported by the Southern California Earthquake Center (SCEC). SCEC is funded by NSF Cooperative Agreement EAR-1033462 & USGS Cooperative Agreement G12AC20038. For more information on the UGMS Committee, visit https://www.scec.org/research/ugms.

User Guide

Background

The paper Site-Specific MCE_R Response Spectra for Los Angeles Region based on 3-D Numerical Simulations and the NGA West2 Equations (https://www.scec.org/publication/8024) provides background information and the method used to obtain the site-specific MCE_R response spectra.

Abstract

The Utilization of Ground Motion Simulation (UGMS) committee of the Southern California Earthquake Center (SCEC) developed site-specific, risk-targeted Maximum Considered Earthquake (MCE_R) response spectra for the Los Angeles region. The long period ($T \geq$ 2-sec) MCE_R response spectra were computed as the weighted average of MCE_R spectral accelerations derived from (1) 3-D numerical ground-motion simulations using the CyberShake computational platform, and (2) empirical ground-motion prediction equations (GMPEs) from the Pacific Earthquake Engineering Research (PEER) Center NGAWest2 project. The short period ($T <$ 2- sec) MCE_R response spectra were computed exclusively from the NGAWest2 GMPEs. A web- based lookup tool was also developed so users can obtain the MCE_R response spectrum for a specified latitude and longitude and for a specified site class or 30-m average shear-wave velocity, VS30. The tool provides acceleration ordinates of the MCE_R response spectrum at 21 natural periods in the 0 to 10-sec band.

Citation

Crouse, C., Jordan, T. H., Milner, K. R., Goulet, C. A., Callaghan, S., & Graves, R. W. (2018, 06). Site-Specific MCER Response Spectra for Los Angeles Region based on 3-D Numerical Simulations and the NGA West2 Equations. Presentation at 11th National Conference in Earthquake Engineering.

Important Note. The site-specific, design response spectral acceleration, S_a, returned by this tool for user-specified inputs, must be compared to the minimum S_a requirement described in Section 21.3 of ASCE 7-16 (second and third paragraphs). This minimum S_a is computed as 80% of the design response spectrum derived from the S_{DS}, S_{D1}, and T_L values obtained from the ASCE tool at https://asce7hazardtool.online/ (https://asce7hazardtool.online/). The larger of the site-specific S_a and the 80% minimum S_a at each period, T, is the final design response spectral acceleration. This final S_a x 1.5 is the final MCE_R response spectral acceleration.

1.0 Introduction.

The Application Page (/ugms-mcerGM-tool_v18.4/) is where the user specifies the inputs required to obtain the site-specific MCE_R response spectrum and site-specific design response spectrum (=2/3 x MCE_R spectrum), subject to the 80% minimum requirement described in the **Important Note** above.

2.0 Instructions for Specification of Inputs.

2.1 Site Location.

The user can either (1) locate the site on the map by using the cursor and zoom-in (+) or zoom-out (-) feature, or (2) enter the latitude and longitude in the boxes to the left of the map. The map, which initially shows the rectangular region where the lookup tool is valid, allows the user to zoom-in to the streets bounding the site of interest, once the general location within the region is identified. After the site is found, a left click of the mouse brings a pop-up with the site coordinates. The user can click on the box, “Use this point” or exit by clicking on the “x” in the upper right corner of the pop-up and search again.

If instead the user enters the latitude and longitude, the longitude must be a negative number. If the user only knows the site address, then the corresponding latitude and longitude can be obtained using the ASCE Lookup Tool (https://asce7hazardtool.online/)

2.2 Site Geotechnical Classification.

Three options are provided: Site Class, Vs30 (m/s), or Unknown (Vs30 estimated from Wills et al., 2015). Clicking on the circle to the left enables the option.

2.2.1 Site Class. The drop-down menu of site classes appears by clicking on the Select box. The nomenclature following the site class letter designation is identical to that in the ASCE lookup tool.

2.2.2 Vs30 (m/s). This parameter is the average shear-wave velocity in the upper 30 meters at the site (See Chapter 20 of ASCE 7-16 for equation to compute the value). Enter the number in the Value box.

2.2.3. Unknown (Vs30 estimated from Wills et al., 2015). This option is used when the user does not yet know the site class or Vs30 value, but wants a preliminary estimate of the MCE_R response spectrum for the site. In this case the tool selects a Vs30 value based on the method in Wills et al. (2015). See citation for Wills et al. in the References section of Crouse et al. (2018) (https://www.scec.org/publication/8024).

After the appropriate site classification option is selected, click on the "Compute Response Spectra" box at the bottom of the page to view the output page.

3.0 Description of Outputs.

The output page opens to a one-page Summary. Below the page header to the left of the map appears the row “Summary Detailed Download all”. The last two are links to additional information. All three are described below.

3.1 Summary.

The Input Parameters (site coordinates & classification) specified by the user are displayed under “Summary”. If the Unknown site classification option was selected, the value of Vs30 used by the tool is displayed under “Values used in Computation”.

The MCE_R and Design Earthquake (=2/3 MCE_R) response spectral parameters (S_{MS}, S_{M1}, S_{DS}, S_{D1}) and the MCE_G parameter (PGA_M) returned by the tool are listed under Computed Results. The site-specific MCE_R response spectrum is listed and plotted under MCE_R Response Spectrum.

The user must check these results against the 80% minimum requirement in the **Important Note** at the top of this User Guide. The web link to the ASCE web tool (https://asce7hazardtool.online/ (https://asce7hazardtool.online/)) at the bottom of the page can be used to extract the response spectra derived from Chapter 11 of ASCE 7-16 that is required for this check.

3.2 Detailed.

The Detailed output page provides values of parameters (Vs30 and basin depth terms, Z1.0 & Z2.5) the tool used to obtain the MCE_R response spectrum. [Note: Z1.0 & Z2.5 are the depths beneath the site to the tops of the layers with shear-wave velocities of 1.0 km/s and 2.5 km/s, respectively.]

The left hand plot under MCE_R Response Spectrum provides the site-specific MCE_R and Design Earthquake response spectra. The right hand plot provides the same MCE_R response spectrum and the MCE_R response spectra obtained from the ground-motion prediction equations (GMPE) and the 3-D numerical simulations (CyberShake). See Crouse et al. (2018) (https://www.scec.org/publication/8024) for details on the derivation of the MCE_R response spectrum from these GMPE-based and CyberShake-based MCE_R response spectra. The table below the plots provides the spectral acceleration values for the three curves in the right hand plot.

3.3 Download all.

Clicking on this link gives the user the option to download a ZIP file containing the Summary and Detailed output pages in PDF format, and an Excel CSV (comma separated values) file with the tabular MCE_R and Design response spectral data.