

Appendices

Appendix H Geotechnical Investigation Report

Appendices

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Converse Consultants

Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

GEOTECHNICAL INVESTIGATION REPORT

LIBRARY AND CLASSROOM RENOVATIONS PROJECT

GRANT ELEMENTARY SCHOOL

2368 PEARL STREET

SANTA MONICA, LOS ANGELES COUNTY, CALIFORNIA 90405

CONVERSE PROJECT No. 21-31-226-02

Prepared For:

SANTA MONICA-MALIBU UNIFIED SCHOOL DISTRICT

Mr. Michael Burke

Architect/Design Manager

Facility Improvement Projects

2828 4th Street

Santa Monica, California 90405

Presented By:

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December 9, 2021



Converse Consultants

75 Years of Dedication in Geotechnical Engineering & Consulting, Environmental & Groundwater Science,
Materials Testing & Inspection Services

December 9, 2021

Mr. Michael Burke
Architect/Design Manager
Facility Improvement Projects
Santa Monica-Malibu Unified School District
2828 4th Street
Santa Monica, California 90405

Subject: **GEOTECHNICAL INVESTIGATION REPORT**
Library and Classroom Renovations Project
Grant Elementary School
2368 Pearl Street, Santa Monica, Los Angeles County, California 90405
Santa Monica-Malibu Unified School District
Converse Project No. 21-31-226-02

Dear Mr. Burke:

Enclosed is the Geotechnical Investigation Report prepared by Converse Consultants (Converse) for the Library and Classroom Renovations Project within Grant Elementary School in Santa Monica, California. The purpose of the investigation was to evaluate the site conditions and generate a report for design purposes, consistent with the 2019 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.

Based on our field exploration, laboratory testing, geological evaluation, and geotechnical analysis, the proposed project is considered feasible from a geotechnical standpoint, provided the recommendations presented in this report are implemented during design and construction.

We appreciate the opportunity to be of continued service to the Santa Monica Malibu Unified School District. 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



PROFESSIONAL CERTIFICATION

This report for the Library and Classroom Renovation Project located within Grant Elementary School in Santa Monica, 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.

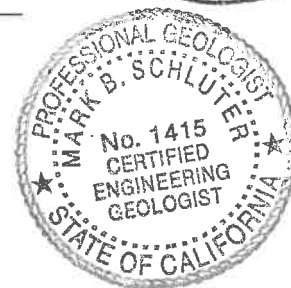
In the event that changes to the property occur, or additional, relevant information about the property is brought to our attention, the conclusions contained in this report may not be valid unless these changes and additional relevant information are reviewed, and the recommendations of this report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



Edmond V. Babayan, PE
Project Engineer



Mark B. Schluter, PG, CEG, CHG
Senior Engineering Geologist



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



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 will focus upon the redevelopment of the Grant Elementary School campus consisting of the renovation of an existing library and classroom building along with additional modifications to an existing outdoor landscaped garden area.
- The project site is located within the existing Grant Elementary School at 2368 Pearl Street in the City of Santa Monica, Los Angeles County, California. The proposed areas of development within the school are relatively level with no pronounced changes in grade elevations. The topography of the overall school site gradually descends towards the south with surface elevations ranging from approximately 153 feet to 147 feet relative to mean-sea-level (MSL). The site is bounded by Pearl Street to the north, 24th Court to the east, single-family residential properties to the south, and Pearl Place to the south. The site coordinates are: North latitude: 34.01886 degrees, West longitude: 118.46135 degrees.
- A total of three (3) exploratory borings (BH-1 through BH-3) were manually excavated on November 11, 2021 using 4-inch diameter hand-auger equipment. The borings were advanced to depths ranging from 6 to 21 feet below the existing ground surface (bgs). Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils.
- The geologic materials encountered during site exploration consist of artificial fill and natural alluvial soils. The artificial fill was identified within all of the brings to depths ranging from 2 to 3 feet below the existing ground surface. The fill materials encountered consisted primarily of silty sand with minor clay. The alluvial soils below the fill consist of poorly graded silty sand with trace amounts of clay.
- Groundwater was not encountered during subsurface exploration to a maximum excavated depth of 21 feet bgs. Based on review of Historically Highest Groundwater Map, Plate No. 1.2, in the Seismic Hazard Zone report for the Beverly Hills 7.5-Minute Quadrangle, Los Angeles County, California, published by the California Department of Conservation, Division of Mines and Geology (CDMG) in 1998, the historic high groundwater level of about 40 feet below the ground surface was noted from wells and soil borings information in the vicinity of the site.



- 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 not located within a mapped potential liquefaction zone per the State of California Earthquake Zones of Required Investigation Map for the Beverly Hills Quadrangle (2018) as shown in Figure No. 7, *Seismic Hazard Zones Map*.
- The surficial site soils exhibit a “very low” expansive potential. Mitigation for expansive soil is not considered necessary for slabs, foundations and pavements.
- The pH value, chloride content, and concentration of water-soluble sulfates of the site soils obtained from corrosion testing indicate that the soils are in the non-corrosive range. The saturated resistivity of the site soils is in the 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 gravelly materials.
- Excavated on-site materials should be cleaned of all organics, deleterious materials, debris and oversize rocks and then moisture conditioned and mixed for re-use as compacted fill. All fill placed at the project site should be compacted to at least 90 percent of the laboratory maximum dry densities as determined by ASTM Standard D1557 test method unless a higher compaction is specified herein.
- Shallow foundations including continuous and isolated footings are considered suitable for structural support provided the recommendations in this report are incorporated into the project plans and specifications and are followed during construction.
- For miscellaneous structures (e.g. signs, perimeter fencing, short retaining walls, etc.), shallow foundations bearing upon newly compacted fill or competent native soils may be utilized.

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.

If any changes are made to the proposed development, as understood and outlined in this report, the conclusions and recommendations contained in this report may not be valid unless these changes have been reviewed, and the recommendations of this report are modified or verified in writing.



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Appendix A	Field Exploration
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1.0 INTRODUCTION

This report contains the findings and recommendations of the geotechnical investigation performed at the site of the Library and Classroom Renovation Project located at Grant Elementary School in Santa Monica, California, as shown on Figure No. 1, *Site Location Map*.

The purpose of the investigation was to evaluate the subsurface soil conditions and provide geotechnical recommendations for the design and construction of the proposed project, consistent with the 2019 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 the Santa Monica Malibu Unified School District, Grant 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 project site is located within Grant Elementary School at 2368 Pearl Street in the City of Santa Monica, California. The proposed areas of development within the school are relatively level with no pronounced changes in grade elevations. The topography of the overall site as gradually descends towards the south with surface elevations ranging from approximately 153 feet to 147 feet relative to mean-sea-level (MSL). The site is bounded by Pearl Street to the north, 24th Court to the east, single-family residential properties to the south, and Pearl Place to the south. The site coordinates are North latitude: 34.01886 degrees, West longitude: 118.46135 degrees.

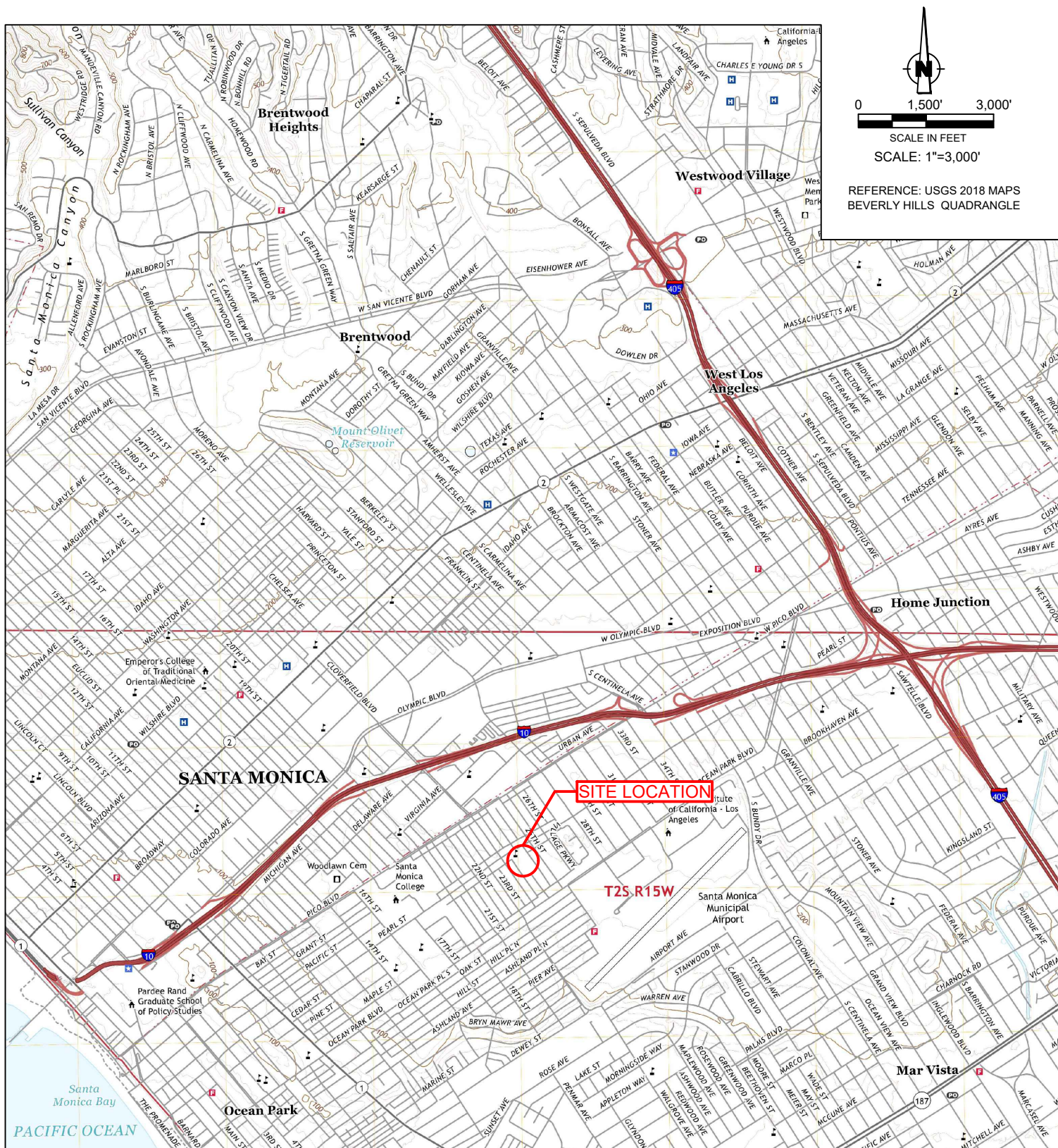
The proposed project will focus upon the redevelopment of the Grant Elementary School buildings consisting of the renovation of an existing library and classroom building along with additional modifications to an existing outdoor landscaped garden area as shown on Figure No. 2, *Approximate Boring Locations*. The structural loads are anticipated to be low.

3.0 SCOPE OF WORK

The scope of our work for the investigation included a site reconnaissance, subsurface exploration laboratory testing, engineering analyses, and preparation of this report.



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SITE LOCATION MAP

Library and Classroom Renovations Project
Grant Elementary School
2368 Pearl Street, Santa Monica,
Los Angeles County, California 90405
Santa Monica-Malibu Unified School District

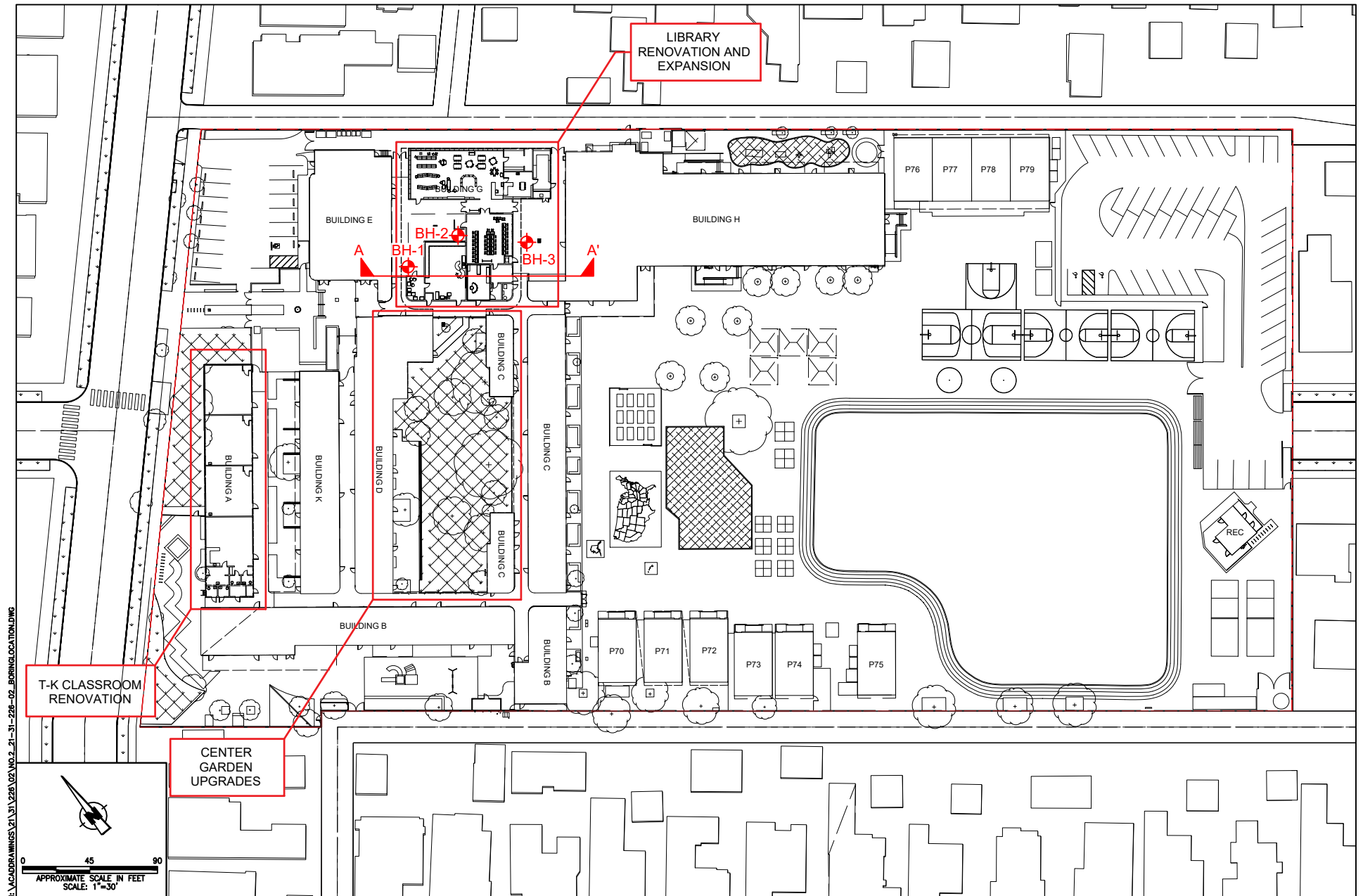
Project No.
21-31-226-02

Figure No.

1



Converse Consultants



3.1 Site Reconnaissance

During site reconnaissance conducted on November 2, 2021, the surface conditions were noted, and the locations of the borings were determined. 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 exploration locations at least 48 hours prior to initiation of the subsurface field work.

3.2 Subsurface Exploration

A total of three (3) exploratory borings (BH-1 through BH-3) were conducted within the project site on November 11, 2021. Initially, it was planned for the borings to be drilled within the locations of the proposed improvements utilizing a track-mounted limited access drilling rig. However, due to the limited space at the site, the drilling contractor could not access the boring locations. As a result, the borings were advanced using manual hand auger equipment. The maximum depth of exploration was initially set for 30 feet below the existing ground surface (bgs). This depth could not be reached due to the site access restrictions for the drilling rig. Thus, the exploration depths ranged from 6 to 21 feet below the existing ground surface using 4-inch diameter hand augers. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soil conditions. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

Undisturbed ring samples and representative bulk samples of the on-site soils were obtained for laboratory testing. Once the explorations were completed, the borings were backfilled with soil cuttings, tamped and capped to match surface conditions. The approximate locations of the exploratory borings are shown in Figure No. 2, *Approximate Boring Locations*.

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)
- Expansion Index (ASTM Standard D4829)



- Collapse Potential (ASTM Standard D5333)
- Soil Corrosivity Tests (Caltrans 643, 422, 417, and 532)
- R-value (California Test 301)

For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*.

3.4 Engineering Analyses and Report

Data obtained from the exploratory fieldwork and laboratory testing were analyzed and evaluated. Geotechnical analyses of the compiled data were performed, and this report was prepared to present the findings, conclusions and recommendations for the proposed project.

4.0 GEOLOGIC CONDITIONS

4.1 Regional Geology

The site is located in the northwest portion of the Los Angeles Basin along the northern boundary of the Peninsular Ranges geomorphic province and the southern margin of the Transverse Ranges geomorphic province. The Los Angeles basin is situated at the junction of two major convergent fault systems. The first group includes the northwest-trending high angle strike slip faults of the San Andreas system projecting northward from the northern terminus of the Peninsular Ranges province. Faults in this group include the Palos Verdes, Newport-Inglewood, Whittier-Elsinore, San Jacinto and San Andreas fault zones. The second group includes the east-west trending low angle reverse or reverse-oblique faults bounding the southern margin of the Transverse Ranges province. Faults in this group include the Malibu-Santa Monica, Hollywood, Raymond and Sierra Madre fault zones. The Los Angeles basin is bounded by active faults on all sides and is underlain by buried thrust fault zones. The seismic hazard for the Los Angeles basin is high.

The Los Angeles basin is a subsiding structural depression that has accumulated as much as 10,000 feet of sediments since Middle Miocene time, about 10 million years ago. Older sediments have been indurated by deep burial and deformed by folding and faulting. The geologic map of the site vicinity is shown on Figure No. 3, *Regional Geologic Map*. At the site location, it is estimated that pre-Quaternary sedimentary formations lie at least 350 to 400 feet below ground surface. Overlying those formations are younger relatively less deformed, sub-horizontally stratified, Quaternary aged sediments varying from marine sands, wind-blown dune deposits to alluvial sands, gravely sands, silts and clays. The shallowest sediments in the site vicinity are Quaternary (< 11,000 years old) fan deposits (CDMG, 1998) that were laid down by streams flowing south from the Santa Monica Mountains.





LEGEND

SURFICIAL SEDIMENTS
Unconsolidated detrital sediments; generally undisturbed
at Artificial cut and fill
Qa/Qbs Beach sand
Qa Alluvial gravel, sand and silt-clay, derived mostly from Santa Monica Mountains; includes gravel and sand of stream channels

LANDSLIDE DEBRIS
Unconsolidated detrital sediments; generally undisturbed
Qls Landslide debris

OLDER SURFICIAL SEDIMENTS
Unconsolidated to weakly consolidated alluvial sediments, dissected where elevated; late Pleistocene age
Qae Alluvial fan sediments of granitic sand at West Hollywood, derived from adjacent Santa Monica Mountains
Qop Paleosol in Baldwin Hills (Fox Hills residual paleosol of Weber et al. 1982), gray to rusty brown, sandy, locally pebbly, moderately indurated "hardpan" on Qoa
Qoa Older alluvium of gray to light brown pebble-gravel, sand, silt and clay derived from Santa Monica Mountains; slightly consolidated; in Baldwin Hills designated as Baldwin Hills sandy gravel by Weber et al. 1982, where it is much dissected and eroded

UNCONFORMITY
in mountain areas

SHALLOW MARINE SEDIMENTS
Shallow marine and some alluvial detrital sediments, weakly consolidated, dissected where elevated; Pleistocene age
Qom Marine deposits of Hoots 1931: light gray to light brown sand, pebbly sand gravel and silt; probably in part non-marine alluvium
Qsp San Pedro Sand (of Wright 1987; Culver Sand of Weber et al. 1982, in Baldwin Hills): light gray to light brown sand, fine to coarse grained or pebbly, locally contains shell fragments
Ql Inglewood Formation (of Wright 1987; Weber et al. 1982, in Baldwin Hills): light gray, friable; fine grained sandstone and interbedded soft gray siltstone; base not exposed

UNNAMED SHALE
(Upper member of Modelo Formation of Hoots 1931; upper Modelo Formation of Durrell 1954; equivalent to Sisquoc Formation of Dibblee 1989, in Ventura Basin)
Marine clastic and biogenic; late Miocene age ("Delmontian" and upper Mohnian Stages)
Tud Soft, white-weathering diatomaceous shale to diatomite, bedded, soft, semi-punky, porous; grades laterally westward into Tush

MONTEREY FORMATION
(Lower member of Modelo Formation of Hoots 1931; Lower Modelo Formation of Durrell 1954; equivalent to Monterey Formation of Dibblee 1989, in Ventura Basin)
Marine biogenic; mostly late Miocene age (Mohnian, possibly late Luisian Stage)
Tm White-weathering, thin bedded, platy dark brown siliceous shale; moderately hard; porcelaneous, locally brittle and cherty in lower part
Tmss Tan to light gray semi-friable bedded sandstone; locally includes interbedded shale; deposited as deep sea fans (Tarzana Fan of Sullwold 1960); locally on south side of mountains, basal sandstone member consists of hard pebbly graywacke of slate on Santa Monica Slate
Tmu Mostly shale similar to Tm but includes interbedded tan friable sandstone similar to Tmss
Tmc Gray claystone at base, includes minor gray sandstone; crumbly where weathered

UNCONFORMITY

UPPER TOPANGA FORMATION
(Of Durrell 1954; part of Topanga Formation of Hoots 1931; may be equivalent to Calabasas Formation of Yerkes and Campbell 1979)
Marine clastic; middle Miocene age
Ttush Gray micaceous clay shale
Ttucg Light gray conglomerate of pebbles and cobbles of granitic and metamorphic rocks and of subangular dense black andesite (derived from Tva) in soft sandy matrix
Ttusi Mostly gray micaceous clay shale or claystone, crumbly where weathered, and thin interbeds of light gray to tan semi-friable sandstone; locally includes pebbly sandstone and lenses of pebble-cobble conglomerate; poorly exposed
Ttusc Light gray sandstone and pebble-cobble conglomerate

NOT TO SCALE

HOLOCENE

QUATERNARY

PLEISTOCENE

REGIONAL GEOLOGIC MAP

4.2 Subsurface Profile

The geologic materials encountered during exploration consist of existing fill and natural alluvial soils. The fill soils consist primarily of silty sands with minor clay fines. The existing fill encountered within the borings extended to depths of approximately 2 to 3 feet below the ground surface and were likely placed during previous site grading. The alluvial soil deposits below the fill consist of poorly graded silty sand. Figure No. 4, *Geologic Cross Section A-A'*, has been drawn across a portion of the project site to illustrate the subsurface conditions. A detailed description of the earth materials encountered during exploration is presented in Appendix A, *Field Exploration*

4.3 Groundwater

Groundwater was not encountered during our subsurface exploration to a maximum excavated depth of 21 feet bgs. Based on review of Historically Highest Groundwater Map, Plate No. 1.2, in the Seismic Hazard Zone report for the Beverly Hills 7.5-Minute Quadrangle, Los Angeles County, California, published by the California Department of Conservation, Division of Mines and Geology (CDMG) in 1998, the historically high groundwater level in the vicinity of the site is approximately 40 feet below the ground surface

Groundwater conditions below any given site may vary depending on numerous factors including seasonal rainfall, local irrigation, stormwater recharge, groundwater basin activities including recharge and pumping (extraction) occurring in the general site vicinity.

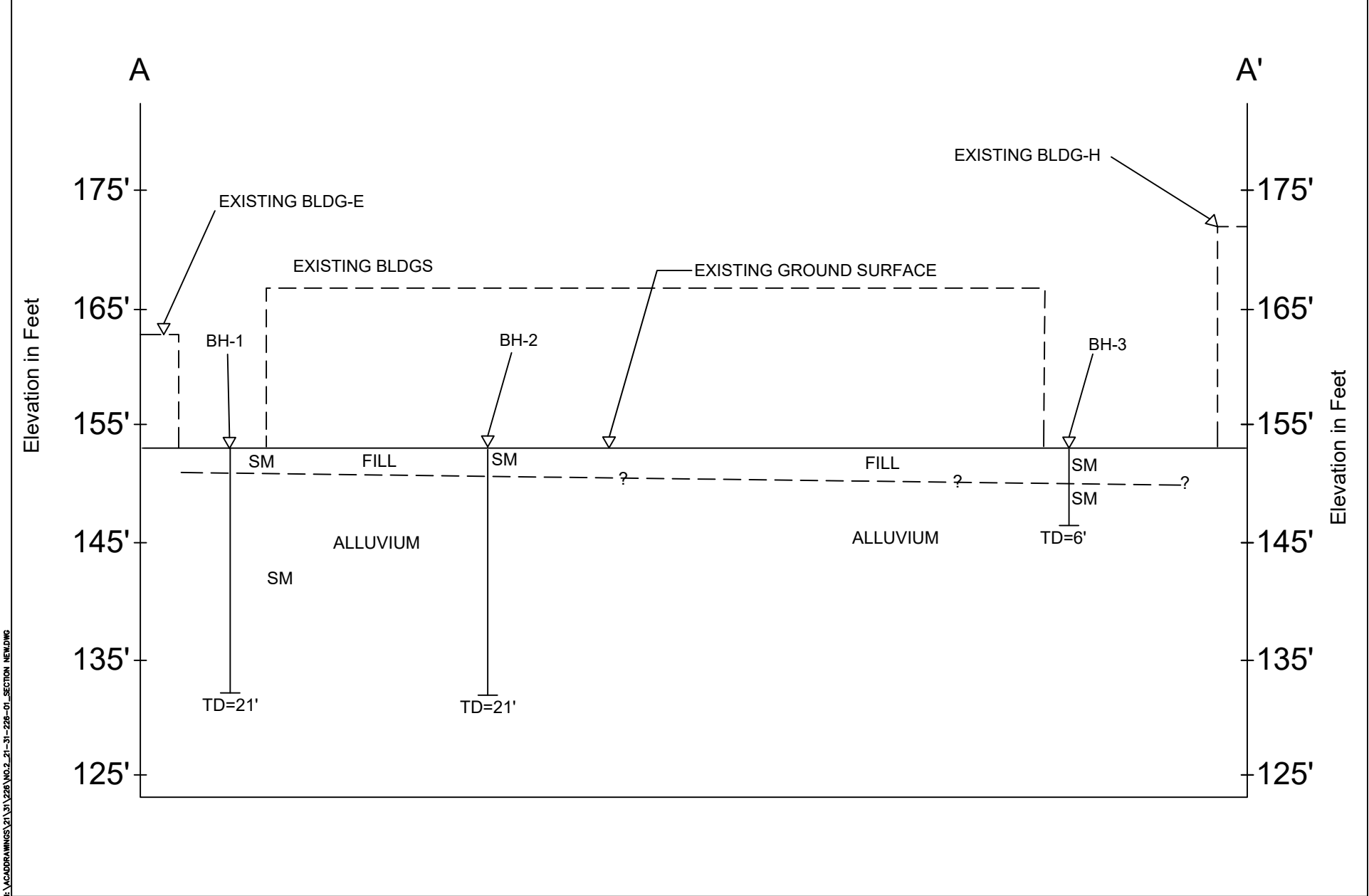
4.4 Subsurface Variations

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 materials 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.

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.





GEOLOGIC CROSS-SECTION A-A'

Results of a site-specific evaluation for each type of possible seismic hazard is discussed in the following sections.

5.1 Seismic Characteristics of Nearby Faults

The 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 designed service life of the proposed development, 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.

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)
Santa Monica Connected alt 2	1.59	7.3	2.4
Santa Monica Connected alt 1	2.19	7.4	2.6
Santa Monica, alt 1	2.19	6.6	1.0
Malibu Coast, alt 1	6.00	6.7	0.3
Malibu Coast, alt 2	6.00	7.0	0.3
Newport Inglewood Connected alt 2	7.14	7.5	1.3
Newport-Inglewood, alt 1	7.14	7.2	1.0
Newport Inglewood Connected alt 1	7.14	7.5	1.3
Anacapa-Dume, alt 2	8.39	7.2	3.0
Hollywood	8.83	6.7	1.0
Palos Verdes Connected	10.33	7.7	3.0
Puente Hills (LA)	11.76	7.0	0.7
Elysian Park (Upper)	18.40	6.7	1.3
Anacapa-Dume, alt 1	21.96	7.2	3.0
Verdugo	24.25	6.9	0.5
Raymond	24.76	6.8	1.5
Sierra Madre Connected	30.38	7.3	2.0
Sierra Madre (San Fernando)	30.38	6.7	2.0
Sierra Madre	30.38	7.2	2.0
Santa Susana, alt 1	32.16	6.9	5.0
Puente Hills (Santa Fe Springs)	32.32	6.5	0.7



Fault Name and Section	Approximate * Distance to Site (kilometers)	Max. Moment Magnitude (M_{max})	Slip Rate (mm/yr)
Northridge	33.14	6.9	1.5
San Gabriel	37.12	7.3	1.0
Elsinore;W	38.27	7.0	2.5
Simi-Santa Rosa	38.31	6.9	1.0
Puente Hills (Coyote Hills)	41.06	6.9	0.7
Holser, alt 1	44.90	6.8	0.4
Clamshell-Sawpit	46.14	6.7	0.5
Oak Ridge (Onshore)	48.45	7.2	4.0
Oak Ridge Connected	48.45	7.4	3.6

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

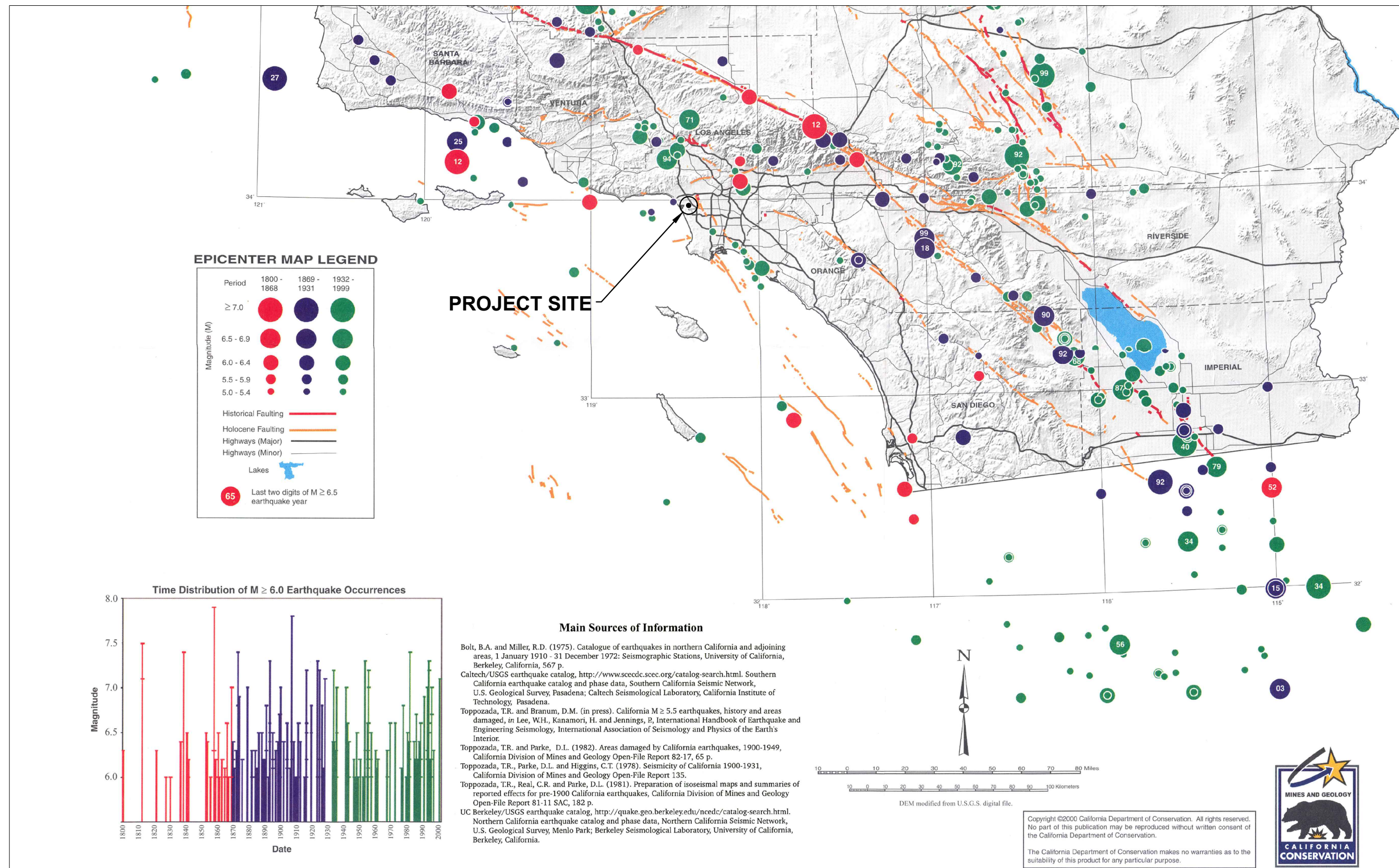
5.2 Surface Fault Rupture

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. Structures are required to be set-back at least 50 feet from an active fault. The active fault set-back distance is measured perpendicular from the dip of the fault plane. Based on a review of existing geologic information, no known active faults project through or toward the site. The closest known fault to the project site with mapped surface traces is the Santa Monica Fault (Approximately 1.3 miles to the northwest and north). The potential for surface rupture resulting from the movement of the nearby major faults is considered very low. The approximate locations of local and regional active faults with respect to the project site are shown on Figure No. 6, *Southern California Regional Fault Map*.

5.3 Liquefaction and Seismically-Induced Settlement

Liquefaction is the sudden decrease in the strength and stiffness of unconsolidated, saturated cohesionless soils typically resulting from seismic ground shaking. For soils to liquefy, the intensity and duration of the seismically induced cyclic loading must be sufficient to increase the excess pore water pressures to such an extent that the effective stresses on the soil particles reduces to zero. If liquefaction is initiated, the saturated soils behave temporarily as a viscous fluid and, consequently, lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content. Liquefaction potential has been found to be the greatest where the groundwater level and loose cohesionless soils occur within 50 feet of the ground surface.





REFERENCE: PORTION OF EPICENTERS AND AREAS DAMAGED BY M≥5 CALIFORNIA EARTHQUAKES, 1800-1999
CALIFORNIA DEPARTMENT OF CONSERVATION, MAP SHEET 49 DATED 2000.

EPICENTER MAP OF SOUTHERN CALIFORNIA EARTHQUAKES (1800-1999)



Converse Consultants

H-18

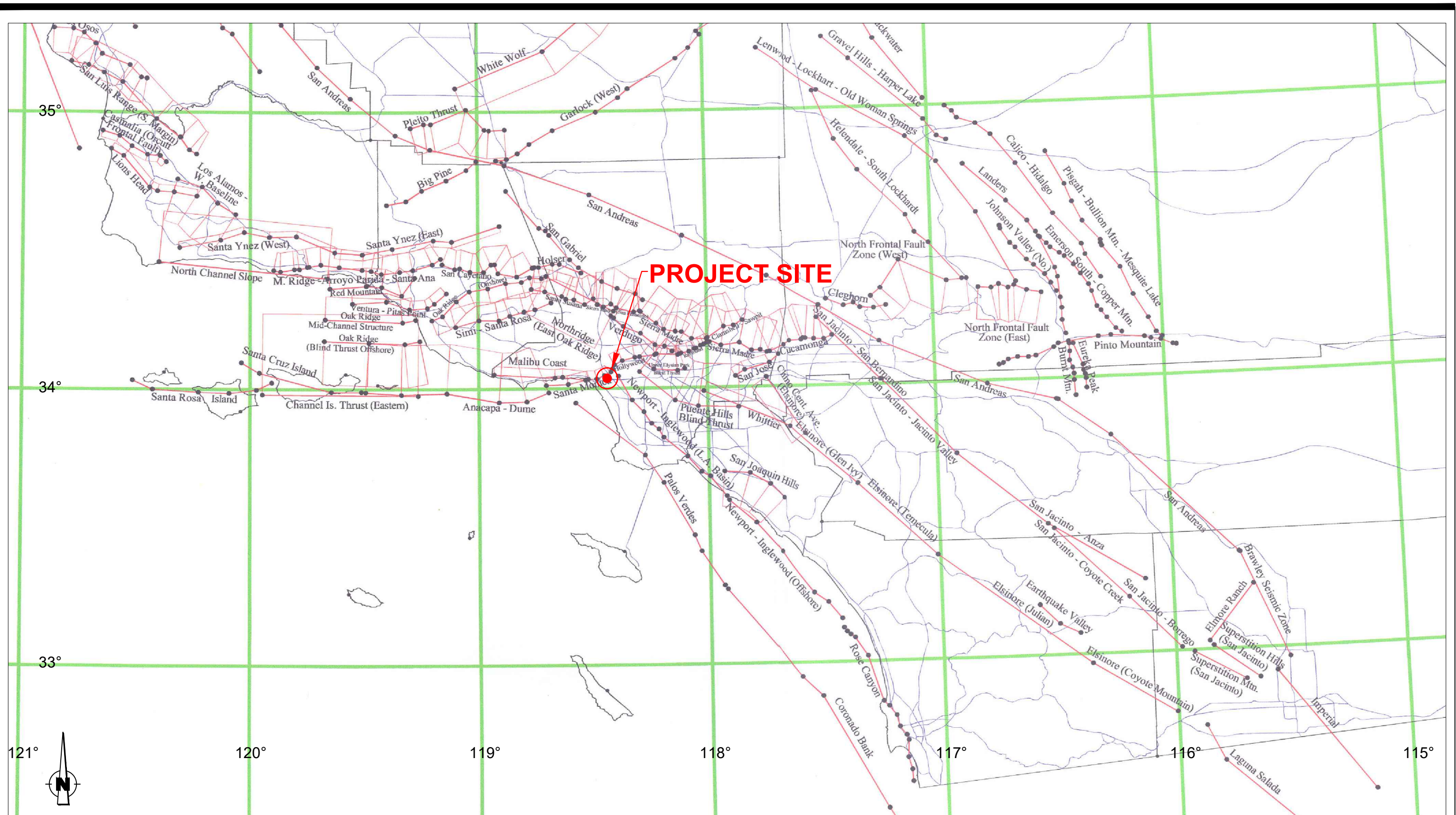
Library and Classroom Renovations Project
Grant Elementary School
2368 Pearl Street, Santa Monica,
Los Angeles County, California 90405
Santa Monica-Malibu Unified School District

Project No.

21-31-226-02

Figure No.

5



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

Library and Classroom Renovations Project
Grant Elementary School
2368 Pearl Street, Santa Monica,
Los Angeles County, California 90405
Santa Monica-Malibu Unified School District

Project No.
21-31-226-02
Figure No.
6
Date
NOV. 2021

The site is not located within a mapped potential liquefaction zone per the State of California Seismic Hazard Zones Map for the Beverly Hills Quadrangle (2018) as shown in Figure No. 7, *Seismic Hazard Zones Map*. Due to the relatively dense nature of the underlying soils and the depths to historic high groundwater levels, the potential for liquefaction is considered 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 negligible.

5.5 Seismically-Induced Slope Instability

Seismically induced landslides and other slope failures are common occurrences during or soon after earthquakes. The topography at the project site and in the immediate vicinity of the site is relatively level, with no significant nearby slopes. Based on the Zones of Required Investigation for the Beverly Hills Quadrangle (1999 & 2018) developed by the California Geological Survey, the site is not located within a mapped landslide zone. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered to be negligible.

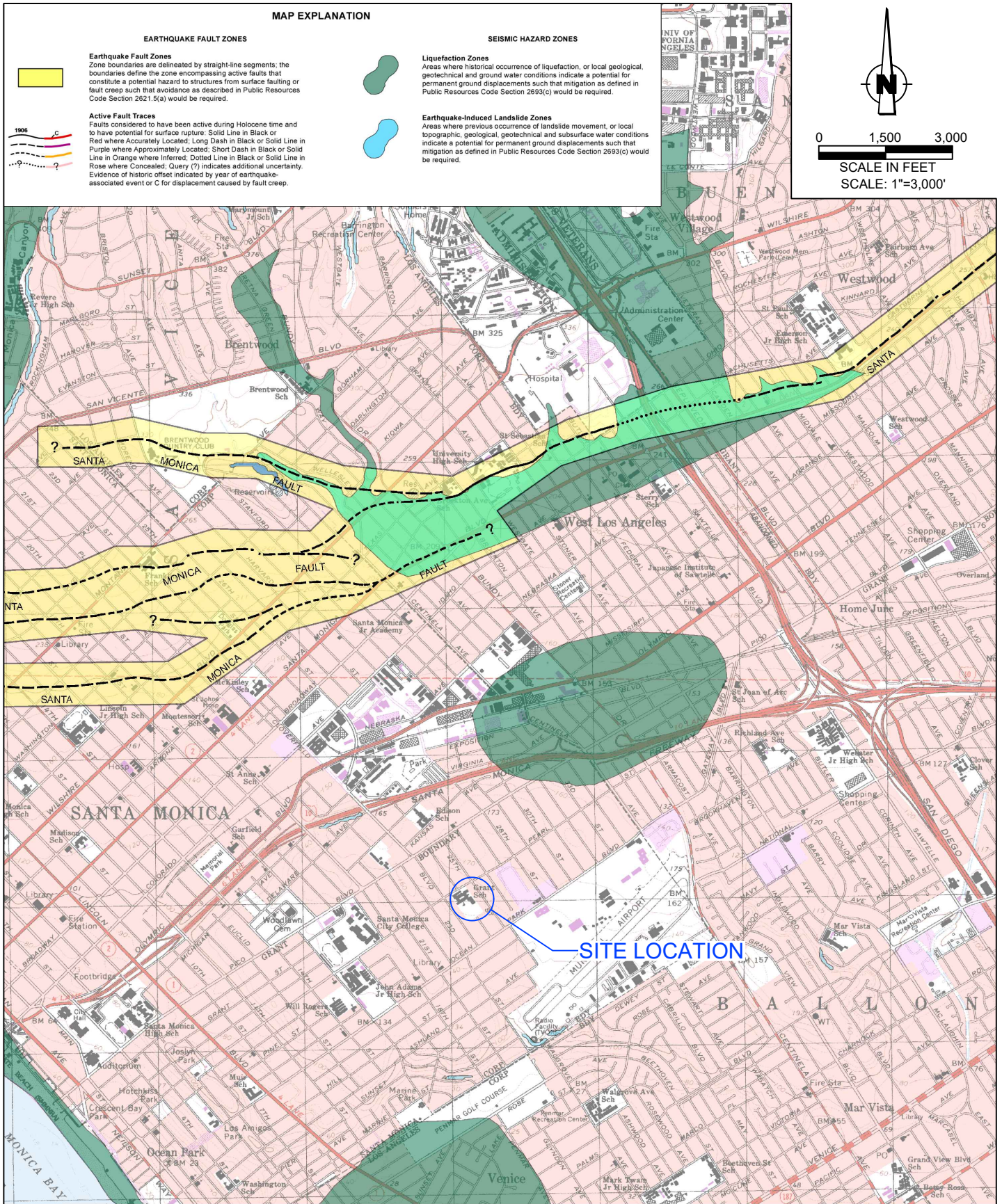
5.6 Earthquake-Induced Flooding

Review of the Federal Emergency Management Agency (FEMA) Flood Insurance Rate Map (FIRM), Map Number 06037C1589G, effective date April 21, 2021, from the Map Service Center (MSC) viewer, indicates that the site is designated as Zone "X", "Area of minimal flood hazard". The potential of earthquake induced flooding of the subject site is considered to be very low.

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 2.0 miles west of the site) and site surface elevation, tsunamis do not pose a hazard. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Based on the site's location relative to lakes, reservoirs and other water-retaining structures, seiches do not pose a hazard.





CGS-BEVERLY HILLS QUAD EZRIM 2018

SEISMIC HAZARD ZONES MAP

5.8 Volcanic Eruption Hazard

There are no known volcanoes near the site. According to Jennings (1994), the nearest potential hazard 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 mapped seismic parameters based on the 2019 California Building Code and ASCE 7-16 with Supplement 1 were calculated using the ATC Hazards by Location web resource application and the site coordinates (North latitude: 34.0188 degrees, West longitude: 118.4613 degrees). The seismic parameters are presented below.

Table No. 2, CBC Mapped Seismic Design Parameters

Seismic Parameter	Value
Site Class	Default (D)
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_s	1.927g
Mapped 1-second Spectral Response Acceleration, S_1	0.686 g
Site Coefficient, F_a	1.2
Site Coefficient, F_v^*	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	2.312 g
MCE 1-second period Spectral Response Acceleration, S_{M1}^*	1.166 g
Design Spectral Response Acceleration for short period, S_{DS}	1.541 g
Design Spectral Response Acceleration for 1-second period, S_{D1}^*	0.777 g
Site-Modified Peak Ground Acceleration, $MCE_G PGA_M$	0.986 g

*See requirements of ASCE 7-16 Section 11.4.8

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 (ASCE, 2016), which were adopted in the 2019 California Building Code. These parameters were determined for the site coordinates and seismic site classification Site Class D 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 2019 CBC site-specific seismic design parameters calculated using the UGMS online tool.



Table No. 3, 2019 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}	2.402 g
⁽¹⁾ MCE _R (5% damped) spectral response acceleration at 1-second period adjusted for site class, S _{M1}	1.819 g
Design spectral response acceleration (5% damped) at short periods, S _{DS}	1.601 g
Design Spectral response acceleration (5% damped) at 1-second period, S _{D1}	1.212 g
Site-Modified Peak Ground Acceleration, MCE _G PGA _M	0.900 g

A seismic deaggregation analysis conducted using the USGS Unified Hazard online tool shows a magnitude 7.31 event located approximately 5.30 miles (8.54 km) from the project site contributes the most to the strong ground shaking 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 the findings from the investigation. 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 and re-compaction), foundation excavations and trench excavation/backfill for utilities.

Excavated site soils, free of grass, roots, organic matter, deleterious materials and rock particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill. 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 and re-compaction for structural footings to be at least five (5) feet below the existing grade or 3 feet below the bottom of footings, whichever is deeper. Deeper removal and re-compaction will be needed if firm soil conditions are not exposed on the excavation bottom. Over-excavation and re-compaction should extend at least five (5) feet laterally



beyond the limits of newly constructed footings where feasible. The on-site soil is considered suitable for re-use as compacted fill provided it is cleaned of organics (e.g. roots, grass, tree stumps, vegetation), debris, deleterious materials and rocks larger than 3-inches.

Over-excavation and re-compaction for retaining walls, if any, should be two (2) feet below the bottom of footings and should extend two (2) feet laterally beyond the footings of the retaining wall. 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 where space permits. If loose, disturbed, or otherwise unsuitable materials are encountered at the bottom of excavations, deeper removals 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 and be protected in place. Existing utilities should be removed and adequately capped and sealed 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, base materials, or slabs. The exposed subgrade should be scarified at least 6 inches, moisture conditioned as needed to near-optimum moisture content, mixed and compacted to at least 90 percent relative compaction. The upper 12-inches of soil subgrade below new pavement should be compacted to at least 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 fills should be placed on competent, scarified and compacted bottom as evaluated by the geotechnical engineer or his representatives 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 approximate 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 fills, 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 Soils

One (1) expansion index test was performed in accordance with ASTM Standard D4829 for the upper five (5) feet of the on-site soils. Results of the test indicated an expansion index of $EI=0$ in BH-1 which corresponds to very low expansion potential. Mitigation for expansive soil is not necessary based on the soil materials encountered. The on-site soil materials will be mixed during the grading and the expansion potential might change. Therefore, the expansion potential of site soils should be verified after the grading for slabs, foundations and pavement placed directly on expansive subgrade soil will likely crack over time.

During construction, grading will mix and relocate the site soils, and additional fill soils may be added. The expansion potential of the finish-grade soils should be tested at the completion of grading. Shallow foundations should be designed to accommodate the



anticipated soil expansion. Any proposed imported fill should have an expansion index less than 20 and should be evaluated and approved by Converse prior to import to the site.

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, 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 $\frac{3}{4}$ 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 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 is required for the surficial soils to provide structural support of appurtenant improvements.

Shallow foundations can be used to support the proposed building renovations with over-excavation and re-compaction of five (5) feet below the existing grade or 3 feet below bottom of footings, whichever is deeper. Flexible utility connections can be used to minimize the damage on utilities during seismic events. For miscellaneous, non-building structures such as retaining walls, shallow foundations may be used.

8.1 Shallow Foundations

8.1.1 Vertical Capacity

The proposed structural renovations to the library and classroom buildings may be supported by conventional shallow foundations. We recommend continuous and isolated footings be founded at least 18 inches below lowest adjacent final grade entirely into compacted fill or undisturbed native soil. A minimum width of 24 inches is recommended for isolated footings and 18 inches for continuous footings. The allowable bearing value for footings with above minimum sizes founded on compacted fill or undisturbed native soils may be designed for a net bearing pressure of 3,000 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 100 psf for each additional foot of excavation width up to a maximum value of 4,000 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 230 psf per foot of depth up to a maximum of 2,000 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 isolated footings founded on compacted fill or 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 values 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 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 125 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 15-mil-thick moisture retarder/barrier between the bottom of slab and subgrade that meets the performance criteria of ASTM E1745 Class A material should be implemented. Retarder/barrier sheets should be overlapped a minimum of six inches and should be taped or otherwise sealed per the product specifications.

8.3 Modulus of Subgrade Reaction

For the proposed development, 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 125 pounds per square inch per inch.

8.4 Lateral Earth Pressure

Although not anticipated, the following provisional design values may be used for any utility vaults or below grade walls 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 Equivalent Fluid Pressure (pcf)
Level	40 (triangular pressure distribution)	60 (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 may consist of a 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.

8.5 Soil Corrosivity Evaluation

The results of corrosion testing performed on a representative sample of the site soils was evaluated for corrosivity evaluation with respect to common construction materials such as concrete and steel (if present). The test results are presented in Appendix B, *Laboratory Testing Program* and are discussed below.



The results of the pH test indicate the soil has pH of 7.18, which should not have an adverse effect on buried concrete or metal.

The sulfate content of the sampled soil corresponds to American Concrete Institute (ACI) exposure class S0 for this sulfate concentration (ACI 318-14, Table 19.3.1.1). No concrete type restrictions are specified for exposure class S0 (ACI 318-14, Table 19.3.2.1). A minimum compressive strength of 2,500 psi is recommended.

We anticipate that concrete structures, if any, will be exposed to moisture from precipitation and irrigation. Based on the alignments location and the results of chloride testing of the site soils, we do not anticipate concrete structures will be exposed to external sources of chlorides, such as deicing chemicals, salt, brackish water, or seawater. ACI specifies exposure class C1 where concrete is exposed to moisture, but not to external sources of chlorides (ACI 318-14, Table 19.3.1.1). ACI provides concrete design recommendations in ACI 318-14, Table 19.3.2.1, including a compressive strength of at least 2,500 psi and a maximum chloride content of 0.3 percent.

According to Romanoff, 1957, the following table provides general guideline of soil corrosion based on electrical resistivity.

Table No. 5, Correlation Between Resistivity and Corrosion

Soil Resistivity (ohm-cm) per Caltrans CT 643	Corrosivity Category
Over 10,000	Mildly corrosive
2,000 – 10,000	Moderately corrosive
1,000 – 2,000	corrosive
Less than 1,000	Severe corrosive

The measured value of the minimum electrical resistivity of the sample when saturated was 1,700 ohm-cm for the site. This indicates that the soils tested are corrosive to ferrous metals in contact with the soil. Converse does not practice in the area of corrosion consulting. If needed, a qualified corrosion consultant should provide appropriate corrosion mitigation measures for any ferrous metals in contact with the site soils. 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.



8.6 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.

Laboratory testing was performed to determine the Resistance value (R-value) of the in-situ subgrade soils. Flexible pavement structural section recommendations have been developed for subgrade soils with a design R-value of 40. 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. 6, 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)
40	4.0	3.0	3.0	4.0
	5.0	3.0	4.0	4.5
	6.0	4.0	4.0	5.0
	7.0	4.0	4.5	6.5
	8.0	5.0	5.0	7.5
	9.0	6.0	6.0	9.0

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, 2018 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.

8.7 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. 7, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Sections (inches)
40*	4.0	6.0
	5.0	6.5
	6.0	6.5
	7.0	7.0
	8.0	7.5
	9.0	8.0

The above pavement sections are based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,000 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 three inches to one inch, respectively.

Transverse expansion/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 inches of subgrade soils below rigid pavement sections should be compacted to at least 95 percent 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.

8.8 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 2019 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 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 excavating equipment. Temporary sloped excavations are feasible if performed in accordance with the slope ratios provided in Section 9.2, *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.

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. 8, *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. 8, 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 excavations occur near existing structures, special considerations would be required during excavation to protect these existing structures during construction. The proposed



excavations should not cause loss of bearing and/or lateral supports of the existing structures.

9.3 Slot Cut Recommendations

Temporary excavations during construction should not extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of any 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 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 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.

9.4 Shoring Design

Temporary shoring may be required where open sloped excavations will not be feasible due to unstable soils or due to nearby existing structures or facilities. Temporary shoring may consist of conventional soldier piles and lagging. Shoring for utility trenches may be laterally supported by cross bracing or may be cantilevered. Drilled excavations for soldier piles may require the use of drilling fluids or casing to prevent caving and to maintain an opened hole for pile installation.

The active earth pressure behind any shoring depends primarily on the allowable movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressures.

The lateral earth pressures to be used in the design of shoring for each segment is presented in the following table.

Table No. 9, Lateral Earth Pressures for Temporary Shoring

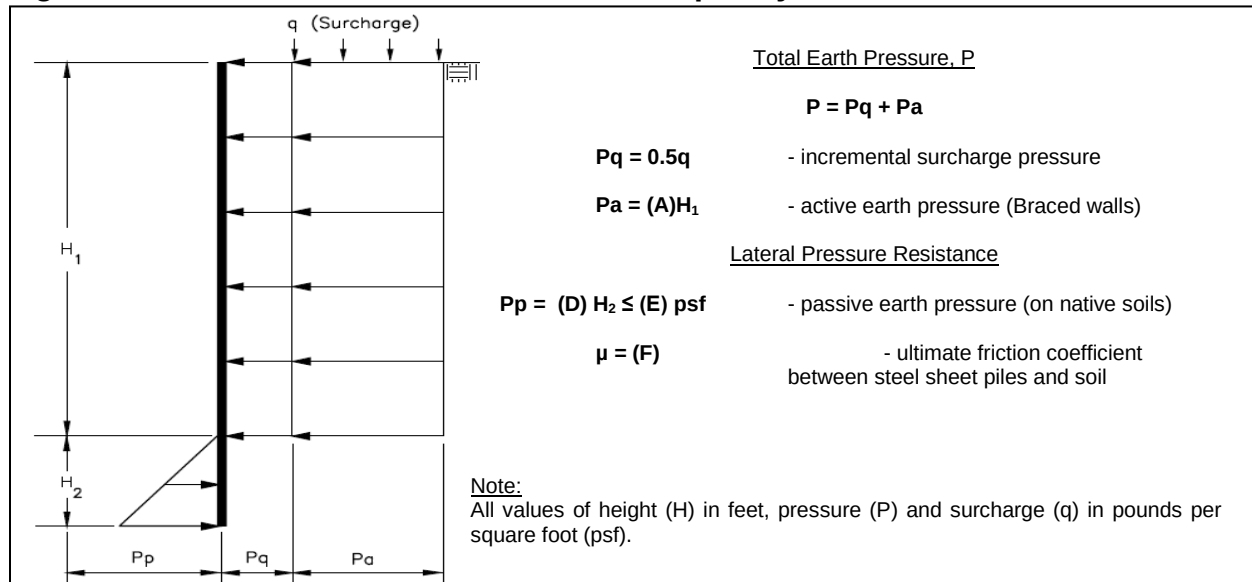
Soil Parameters*	Values
Active Earth Pressure (Braced Shoring) (psf) (A)	40
Active Earth Pressure (Cantilever Shoring) (psf) (B)	45
At-Rest Earth Pressure (Cantilever Shoring) (psf) (C)	60
Passive earth pressure (psf per foot of depth) (D)	230
Maximum allowable passive pressure against native soils (psf) (E)	2,000
Coefficient of friction between soldier or sheet pile and native soils, fs (F)	0.30

* Parameters A through F are used in Figures No. 8 and 9.



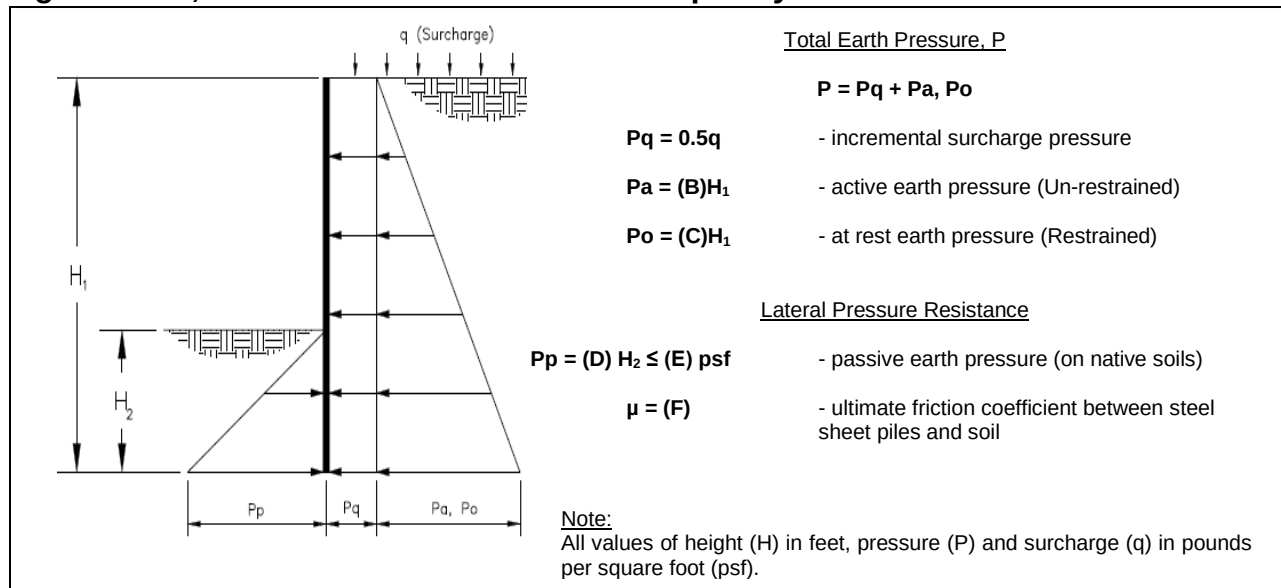
Restrained (braced) shoring systems should be designed based on Figure No. 8, *Lateral Earth Pressures for Temporary Braced Excavation* to support a uniform rectangular lateral earth pressure.

Figure No. 8, Lateral Earth Pressures for Temporary Braced Excavation



Unrestrained (cantilever) shoring consisting of soldier piles spaced at least two diameters on-center, can be designed based on Figure No. 9, *Lateral Earth Pressures on Temporary Cantilever Wall*.

Figure No. 9, Lateral Earth Pressures on Temporary Cantilever Wall



The provided pressures assume no hydrostatic pressures. If hydrostatic pressures are allowed to build up, the incremental earth pressures below the ground-water level should be reduced by 50 percent and added to hydrostatic pressure for total lateral pressure.

Passive resistance includes a safety factor of 1.5. The upper 1 foot for passive resistance should be ignored unless the surface is confined by a pavement or slab.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation. As previously mentioned, all shoring should be designed and installed in accordance with state and federal safety regulations.

The contractor should have provisions for shoring pile removal. All voids resulting from removal of shoring should be filled. The method for filling voids should be selected by the contractor, depending on construction conditions, void dimensions, and available materials. The acceptable materials, in general, should be non-deleterious, and able to flow into the voids created by shoring removal (e.g. concrete slurry, "pea" gravel, etc.).

Excavations for any utilities should not extend below a 1:1 Horizontal: Vertical (H:V) plane extending from the bottom of any existing structures, utility lines or streets. Any proposed excavation should not cause loss of bearing and/or lateral supports of the existing utilities or streets.

If the excavation extends below a 1:1 (H:V) plane extending from the bottom of the existing structures, utility lines or streets, a maximum of 10 feet of slope face parallel to the existing improvement should be exposed at a time to reduce the potential for instability. Backfill should be accomplished in the shortest period of time and in alternating sections.

9.5 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 a 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 field exploration method performed are discussed below.

Exploratory Borings

A total of three (3) exploratory borings (BH-1 through BH-3) were conducted within the project site on November 11, 2021. Initially, it was planned for the borings to be drilled within the locations of the proposed improvements utilizing a track-mounted limited access drilling rig. However, due to the limited space at the site, the drilling contractor could not access the boring locations. As a result, the borings were advanced using manual hand auger equipment. The maximum depth of exploration was initially set for 30 feet below the existing ground surface (bgs). This depth could not be reached due to the access restrictions for the drilling rig. Thus, the exploration depths ranged from 6 to 21 feet below the existing ground surface using 4-inch diameter hand augers. Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soil conditions. 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 within the borings using a hand-driven sampler lined with sample rings. Samples were 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. Bulk samples of the upper 5 feet of surficial materials were also obtained from each boring.

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 to A-1b, *Unified Soil Classification Chart*. The log of the exploratory boring is presented in Drawing Nos. A-2 through A-4, *Log of Borings*.



SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
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
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	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
		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
	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

BORING LOG SYMBOLS

DRILLING METHOD SYMBOLS			
	Auger Drilling		Mud Rotary Drilling
	Dynamic Cone or Hand Driven		Diamond Core

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	Permeability (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)

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

SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS



Converse Consultants

Project Name

Library & Classroom Renovations Project
Grant Elementary School
2368 Pearl Street
Santa Monica, California 90405
For: Santa Monica-Malibu Unified School District

Project No. 21-31-226-02

Drawing No. A-1a

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. 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.

SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS



Converse Consultants

Project Name
Library & Classroom Renovations Project
Grant Elementary School
2368 Pearl Street
Santa Monica, California 90405
For: Santa Monica-Malibu Unified School District

Project No. Drawing No.
21-31-226-02 A-1b

Log of Boring No. BH-1

Dates Drilled: 11/11/2021 Logged by: Flora Delgado Checked By: E.Babayan

Equipment: HAND AUGER Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 153 Depth to Water (ft, bgs): NOT ENCOUNTERED

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 Test Pit 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		BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
			DRIVE	BULK				
		CONCRETE GROUND COVER 6 INCHES						
		ARTIFICIAL FILL (Af):						
		SILTY SAND (SM): fine to medium-grained, little clay, medium dense, moist, dark reddish brown.						
5		ALLUVIUM (Qal):						
		SILTY SAND (SM): fine to medium-grained, trace clay, dense, moist, brown.				9	87.8	WA, fc=42.7%
10						8	119.3	WA, fc=29.7%
15						10	108.5	
20		-very dense				8	103.4	
		End of boring at 21 feet below ground level. No groundwater encountered. Borehole backfilled with soil cuttings and patched with concrete on 11/11/21.						



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Library & Classroom Renovations Project
Grant Elementary School
2368 Pearl Street
Santa Monica, California 90405
For: Santa Monica-Malibu Unified School District

Project No. 21-31-226-02 Drawing No. A-2

Log of Boring No. BH-2

Dates Drilled: 11/11/2021 Logged by: Flora Delgado Checked By: E.Babayan

Equipment: HAND AUGER Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 153 Depth to Water (ft, bgs): NOT ENCOUNTERED

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 Test Pit 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		BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
			DRIVE	BULK				
		ASPHALT GROUND COVER~3 INCHES						
		ARTIFICIAL FILL (Af):						
		SILTY SAND (SM): fine to medium-grained, little clay, medium dense, moist, brown.						
5		ALLUVIUM (Qal):						
		SILTY SAND (SM): fine to coarse-grained, trace clay, dense, moist, brown.				10	123	
10						8	109	
15						7	109.5	DS
20		-very dense				8	106.4	
		End of boring at 21 feet below ground level. Groundwater not encountered. Backfilled with soil cuttings and patched with asphalt on 11/11/21.						



Converse Consultants

Library & Classroom Renovations Project
Grant Elementary School
2368 Pearl Street
Santa Monica, California 90405
For: Santa Monica-Malibu Unified School District

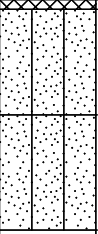
Project No. 21-31-226-02 Drawing No. A-3

Log of Boring No. BH-3

Dates Drilled: 11/11/2021 Logged by: Flora Delgado Checked By: E.Babayan

Equipment: HAND AUGER Driving Weight and Drop: N/A

Ground Surface Elevation (ft): 153 Depth to Water (ft, bgs): NOT ENCOUNTERED

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 Test Pit 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		BLOWS	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
			DRIVE	BULK				
5		ASPHALT GROUND COVER~3 INCHES						
		ARTIFICIAL FILL (Af): SILTY SAND (SM): little clay, medium dense, moist, dark brown.						
		ALLUVIUM (Qal): SILTY SAND (SM): fine to medium-grained, little clay, medium dense, moist,.				8	98.3	CL
		End of boring at 6 feet below ground level. Groundwater not encountered. Borehole backfilled with soil cuttings and patched with asphalt on 11/11/21.						



Converse Consultants

Library & Classroom Renovations Project
Grant Elementary School
2368 Pearl Street
Santa Monica, California 90405
For: Santa Monica-Malibu Unified School District

Project No. 21-31-226-02 Drawing No. A-4

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*.

Expansion Index Test

One (1) representative soil 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-1, Summary of Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-1	0-5	Silty Sand (SM)	0	Very Low

Soil Corrosivity

One (1) representative soil sample was tested to determine minimum electrical resistivity, pH, and chemical content, including chloride concentrations, and water-soluble sulfates. 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 are summarized in the following table:

Table No. B-2, 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	0-5	7.18	200	0.009	1,700



R-Value

One representative bulk soil sample was tested for resistance value (R-value) in accordance with California Test Method CT301. This test is designed to provide a relative measure of soil strength for use in pavement design. The test result is presented in the table below.

Table No. B-3, Summary of R-Value Test Result

Boring No.	Depth (feet)	Soil Classification	Measured R-Value
BH-3	0-5	Silty Sand (SM)	40

Percent Finer Than Sieve No. 200

The percent finer than sieve No. 200 test was performed on two (2) 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-4, Summary of Percent Passing Sieve #200 Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Passing Sieve No. 200
BH-1	5-6.5	Silty Sand (SM)	42.7
BH-1	10-11.5	Silty Sand (SM)	29.7

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on one (1) selected bulk 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	0-5	Silty Sand (SM)	0.0	58	42	

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-2	0-5	Silty Sand (SM)	8.0	133

Direct Shear

Direct shear testing was performed on relatively undisturbed soil samples obtained from a certain depth range. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus, inundated with water, 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-1	10-11.5	Silty Sand (SM)	29	170

Collapse

To evaluate the moisture sensitivity (collapse/swell potential) of the encountered soils, one collapse test was performed in accordance with the ASTM Standard D4546 laboratory procedure. The sample was loaded to approximately 2 kips per square foot (ksf), allowed to stabilize under load, and then submerged. The test results are presented in the following table.

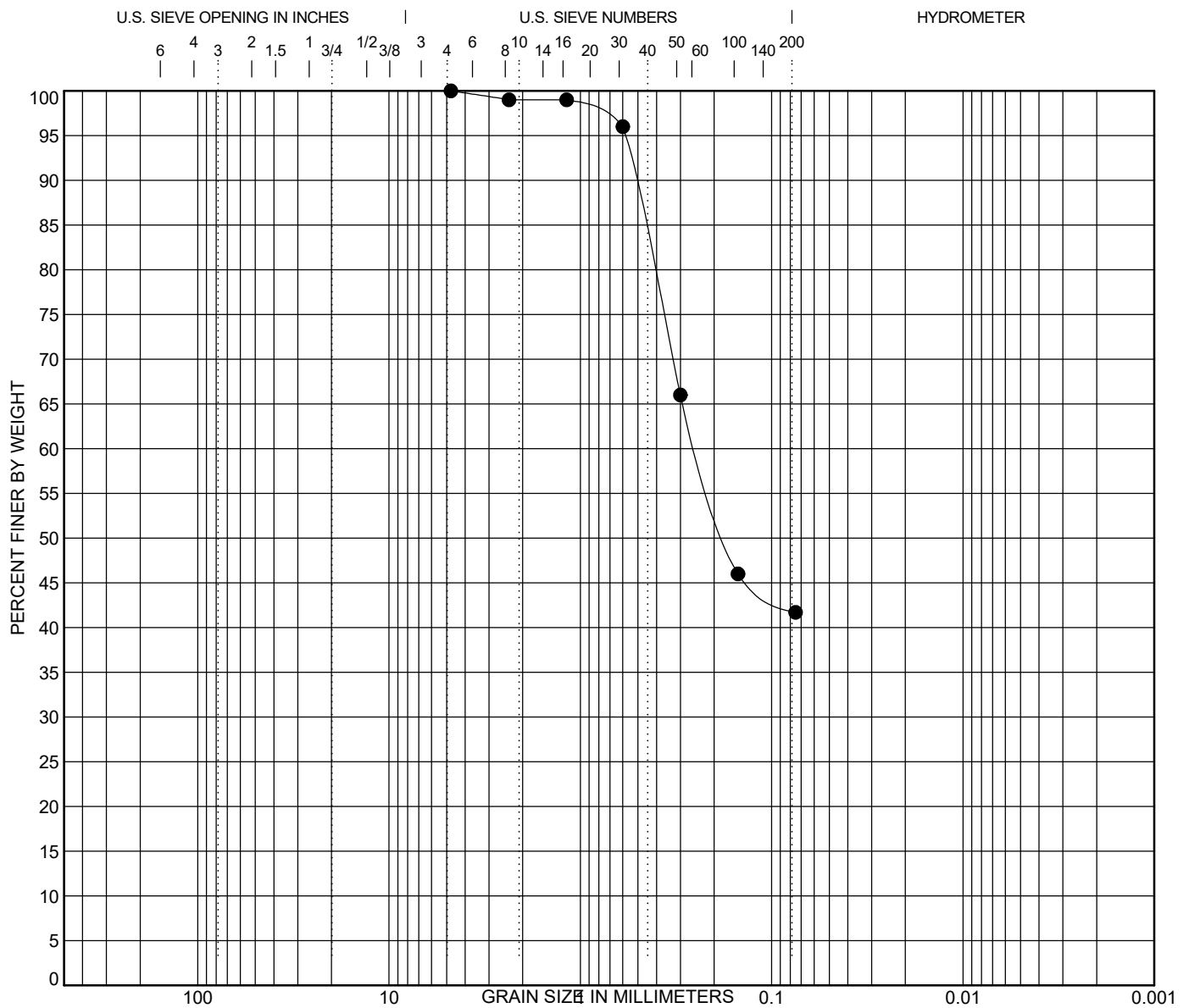
Table No. B-8, Summary of Collapse Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Swell (+) Percent Collapse (-)	Collapse Potential
BH-3	5	Silty Sand (SM)	-0.9	Slight

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.

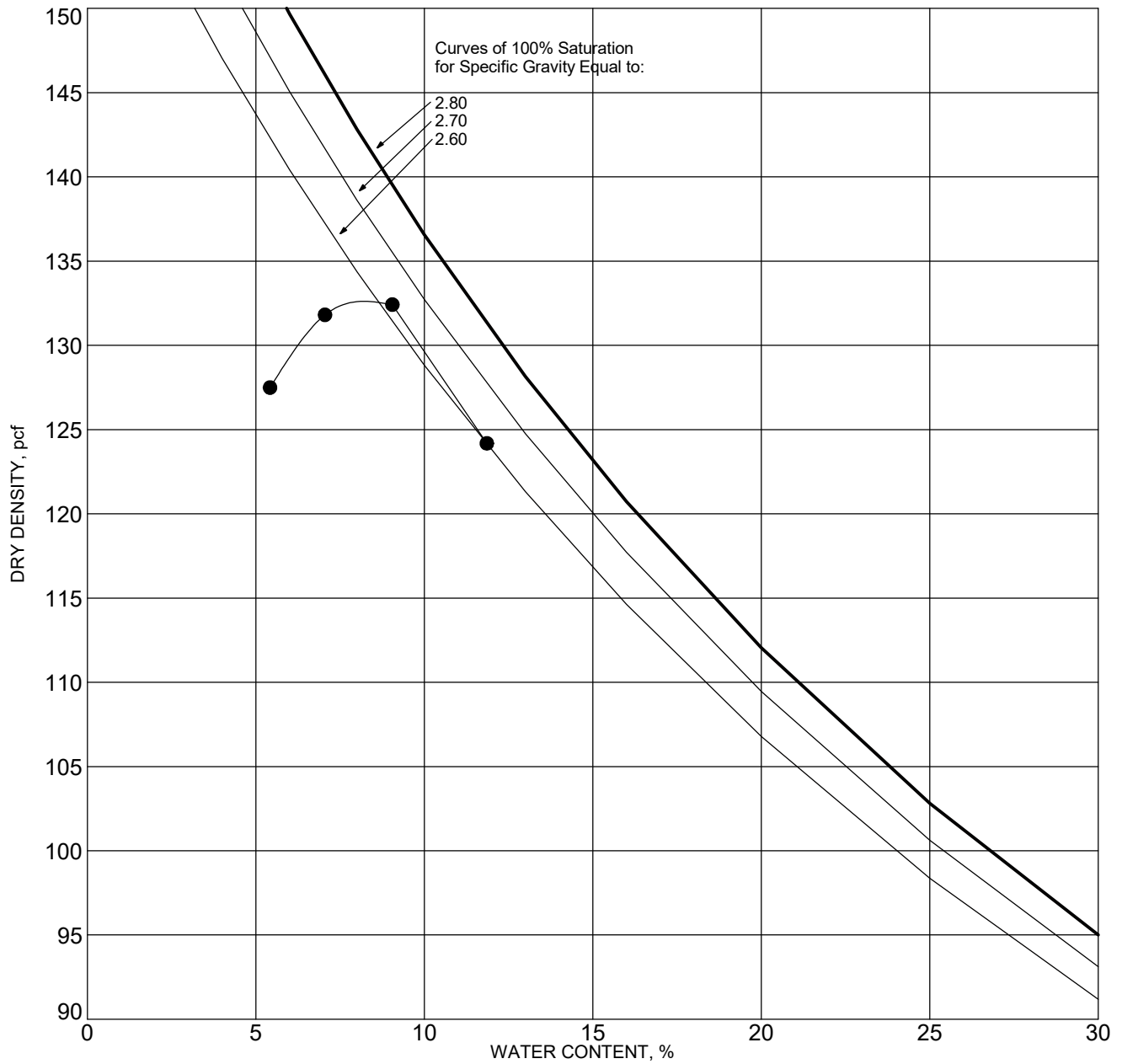




COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Boring No.		Depth (ft)	Description					LL	PL	PI	Cc	Cu
●	BH-1	0-5	Silty Sand									
Boring No.		Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay		
●	BH-1	0-5	4.75	0.244			0.0	58.3	41.7			

GRAIN SIZE DISTRIBUTION RESULTS



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-2	0-5	Silty Sand with trace Clay (SM)	D1557 Method B	8	133

MOISTURE-DENSITY RELATIONSHIP RESULTS

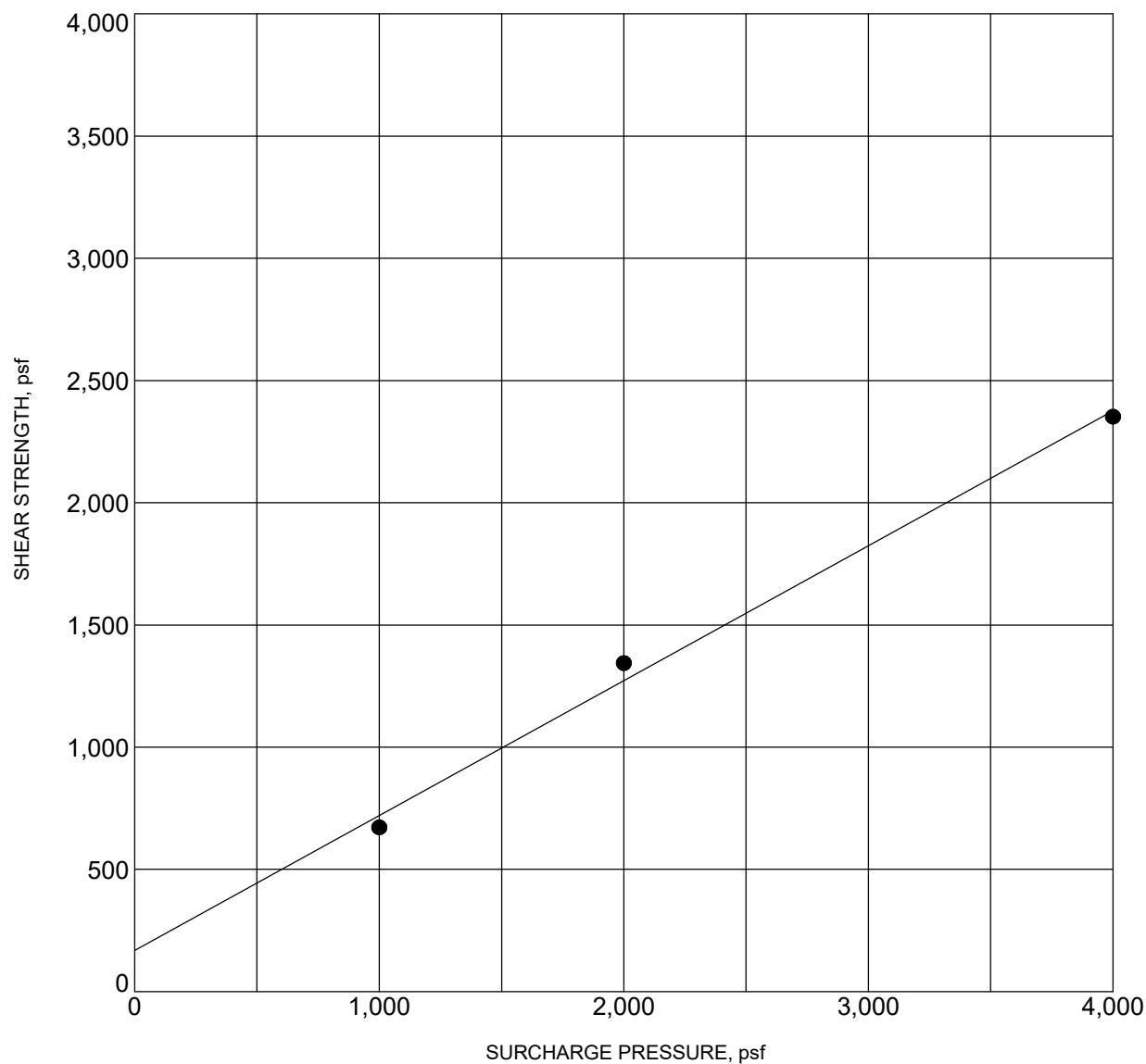


Converse Consultants

Project Name
Library and Classroom Renovations Project
Grant Elementary School
2368 Pearl Street, Santa Monica,
Los Angeles County, California 90405
Santa Monica-Malibu Unified School District

Project No.
21-31-226-02

Drawing No.
B-2



BORING NO. :	BH-1	DEPTH (ft) :	10
DESCRIPTION :	Silty Sand		
COHESION (psf) :	170	FRICTION ANGLE (degrees):	29
MOISTURE CONTENT (%) :	8	DRY DENSITY (pcf) :	108.5

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS



Converse Consultants

Project Name
Library & Classroom Renovations Project
Grant Elementary School
2368 Pearl Street
Santa Monica, California 90405
For: Santa Monica-Malibu Unified School District

Project No.
21-31-226-02

Figure No.
B-3

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 *Geotechnical 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. In order to provide uniform support for the new structures, the minimum depth of over-excavation should be five (5) feet below the existing grade, or 3 feet below proposed shallow foundations whichever is deeper. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. The actual depth of removal should be determined based on observations made during grading. Over-excavation should extend a least five (5) feet beyond the limits of footings, or equal distance of over-excavation depth, whichever is greater, or as limited by the existing structures. 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 as



designed for sidewalks and flatwork area, at least the upper 24 inches of existing soils should be scarified and recompacted to at least 90 percent of compaction. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. The excavation should be extended to at least 12 inches beyond the driveway and flatwork limit where space is permitted.

3. 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.
4. Compacted fill may be placed on native soils that have been properly scarified and re-compacted as discussed above.
5. 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 or imported fill that meets the criteria indicated below.
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 to 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



Mapped Seismic Parameters





Search Information

Address: 2368 Pearl St, Santa Monica, CA 90405, USA

Coordinates: 34.0188621, -118.4613494

Elevation: 155 ft

Timestamp: 2021-12-07T17:15:26.328Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: III

Site Class: D-default



Basic Parameters

Name	Value	Description
S_S	1.927	MCE_R ground motion (period=0.2s)
S_1	0.686	MCE_R ground motion (period=1.0s)
S_{MS}	2.312	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.541	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	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.908	Coefficient of risk (0.2s)
CR_1	0.903	Coefficient of risk (1.0s)
PGA	0.822	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.986	Site modified peak ground acceleration

H-62

T _L	8	Long-period transition period (s)
SsRT	1.927	Probabilistic risk-targeted ground motion (0.2s)
SsUH	2.121	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	2.458	Factored deterministic acceleration value (0.2s)
S1RT	0.686	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.76	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.829	Factored deterministic acceleration value (1.0s)
PGAd	0.994	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.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey [Seismic Design Web Services](#).

While the information presented on this website is believed to be correct, ATC and its sponsors and contributors assume no responsibility or liability for its accuracy. The material presented in the report should not be used or relied upon for any specific application without competent examination and verification of its accuracy, suitability and applicability by engineers or other licensed professionals. ATC does not intend that the use of this information replace the sound judgment of such competent professionals, having experience and knowledge in the field of practice, nor to substitute for the standard of care required of such professionals in interpreting and applying the results of the report provided by this website. Users of the information from this website assume all liability arising from such use. Use of the output of this website does not imply approval by the governing building code bodies responsible for building code approval and interpretation for the building site described by latitude/longitude location in the report.

Seismic Hazard Deaggregation



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.

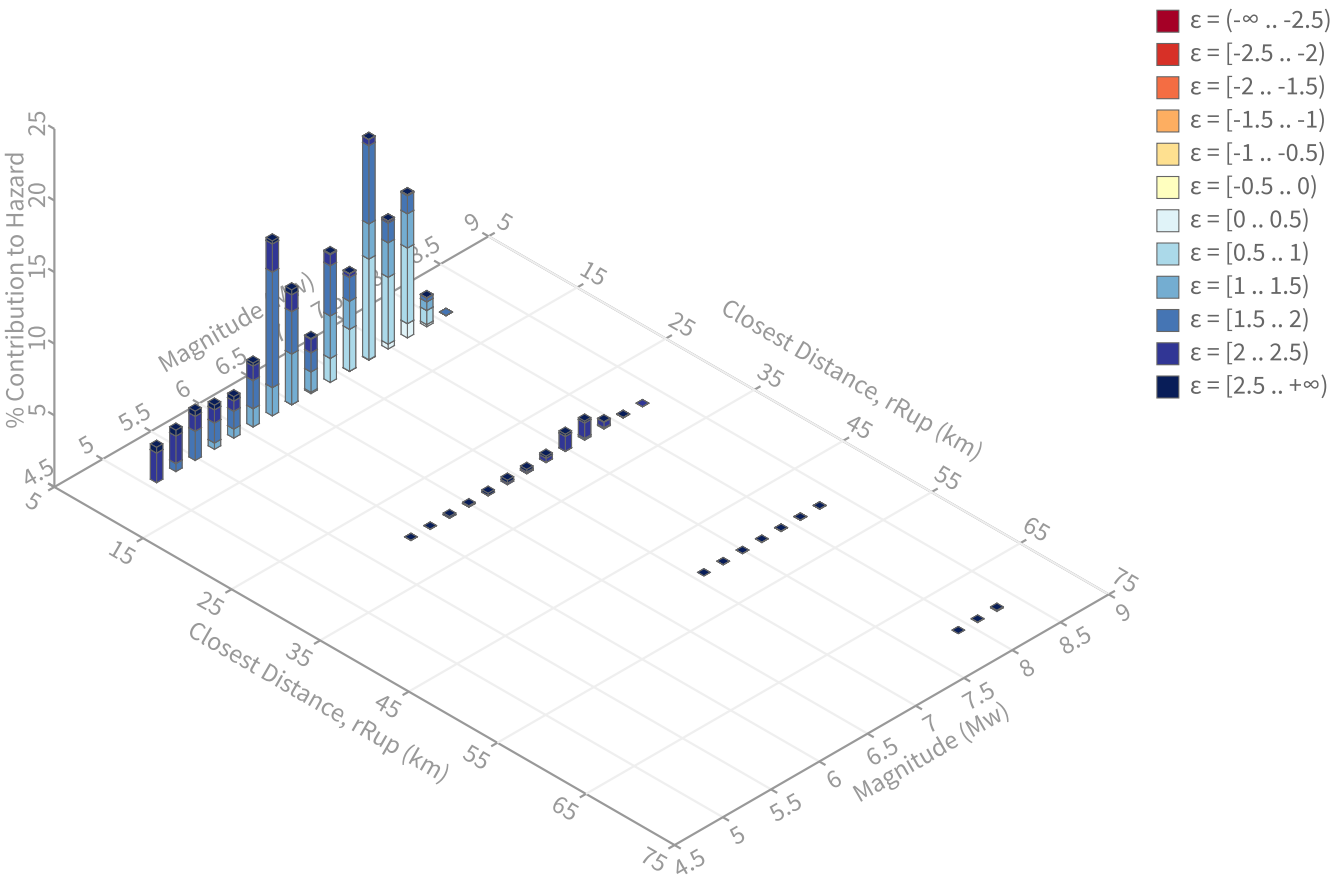
^ Input

Edition Dynamic: Conterminous U.S. 2014 (u...	Spectral Period Peak Ground Acceleration
Latitude Decimal degrees 34.0188621	Time Horizon Return period in years 2475
Longitude Decimal degrees, negative values for western longitudes -118.4613494	
Site Class 259 m/s (Site class D)	

^ Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr⁻¹
PGA ground motion: 0.84625274 g

Recovered targets

Return period: 2966.8851 yrs
Exceedance rate: 0.00033705383 yr⁻¹

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.06 %

Mean (over all sources)

m: 6.81
r: 8.78 km
ε₀: 1.51 σ

Mode (largest m-r bin)

m: 7.31
r: 8.54 km
ε₀: 1.21 σ
Contribution: 15.4 %

Mode (largest m-r-ε₀ bin)

m: 6.35
r: 7.78 km
ε₀: 1.78 σ
Contribution: 8.07 %

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							36.85
	Santa Monica alt 1 [0]		3.22	7.18	0.98	118.470°W	34.042°N	342.50	10.53
	Compton [4]		10.05	7.39	0.88	118.556°W	33.944°N	226.39	5.91
	Newport-Inglewood alt 1 [8]		7.36	6.64	1.68	118.389°W	34.044°N	67.51	5.74
	Palos Verdes [15]		10.39	7.02	1.74	118.551°W	33.963°N	233.19	4.94
	Compton [3]		10.10	7.35	0.91	118.533°W	33.925°N	212.38	1.27
	Malibu Coast alt 1 [0]		6.05	6.35	1.66	118.525°W	34.031°N	282.56	1.14
	San Pedro Escarpment [1]		8.77	7.60	0.79	118.655°W	33.915°N	237.25	1.13
UC33brAvg_FM32		System							36.75
	Santa Monica alt 2 [2]		2.89	7.16	0.97	118.460°W	34.043°N	3.08	8.06
	Newport-Inglewood alt 2 [8]		7.31	6.72	1.63	118.390°W	34.043°N	67.64	4.62
	Hollywood [2]		8.27	6.97	1.58	118.422°W	34.084°N	26.30	4.59
	Palos Verdes [15]		10.39	7.02	1.81	118.551°W	33.963°N	233.19	4.53
	Malibu Coast alt 2 [0]		6.27	7.49	1.02	118.525°W	34.033°N	285.06	3.88
	Compton [4]		10.05	7.47	0.86	118.556°W	33.944°N	226.39	2.97
	Compton [3]		10.10	7.26	0.91	118.533°W	33.925°N	212.38	1.90
UC33brAvg_FM31 (opt)		Grid							13.75
	PointSourceFinite: -118.461, 34.059		6.61	5.75	1.69	118.461°W	34.059°N	0.00	3.01
	PointSourceFinite: -118.461, 34.059		6.61	5.75	1.69	118.461°W	34.059°N	0.00	3.01
	PointSourceFinite: -118.461, 34.077		7.79	5.81	1.85	118.461°W	34.077°N	0.00	1.68
	PointSourceFinite: -118.461, 34.077		7.79	5.81	1.85	118.461°W	34.077°N	0.00	1.68
UC33brAvg_FM32 (opt)		Grid							12.65
	PointSourceFinite: -118.461, 34.059		6.59	5.77	1.68	118.461°W	34.059°N	0.00	2.59
	PointSourceFinite: -118.461, 34.059		6.59	5.77	1.68	118.461°W	34.059°N	0.00	2.59
	PointSourceFinite: -118.461, 34.077		7.77	5.83	1.84	118.461°W	34.077°N	0.00	1.51
	PointSourceFinite: -118.461, 34.077		7.77	5.83	1.84	118.461°W	34.077°N	0.00	1.51

Site-Specific Spectral Accelerations



Site-Specific MCE_R & Design Response Spectral Accelerations

Grant Elementary School: Library and Classroom Renovation Project

Input Parameters

Coordinates 34.019, -118.461
Site Class D - Default (see Section 11.4.3)

Values used in Computation

V_{S30} N/A
Z1.0 650 m
Z2.5 5000 m

Calculated Results

Site-Specific Design Parameters

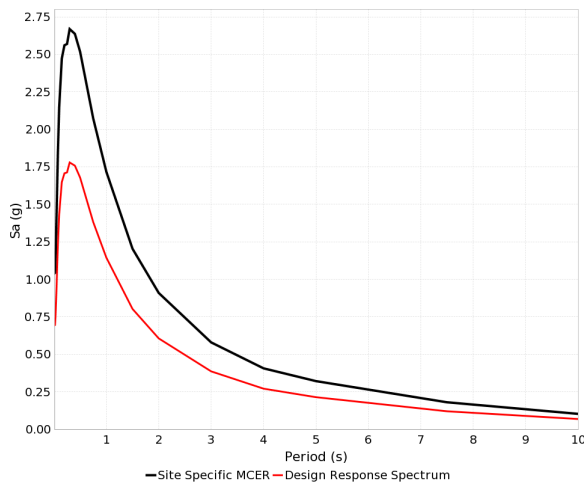
S_{DS} 1.601 S_{MS} 2.402
 S_{D1} 1.212 S_{M1} 1.819



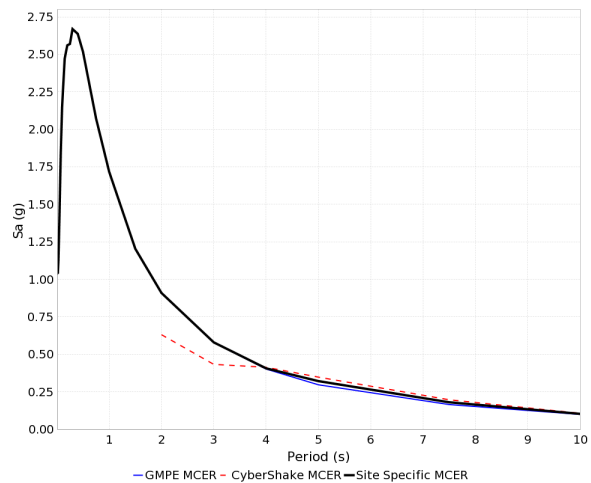
MCE_G Peak Ground Acceleration (Sect. 21.5)

PGA_M 0.900 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	1.040		1.040
0.02	1.043		1.043
0.03	1.122		1.122
0.05	1.417		1.417
0.075	1.841		1.841
0.1	2.146		2.146
0.15	2.472		2.472
0.2	2.561		2.561
0.25	2.570		2.570
0.3	2.669		2.669
0.4	2.637		2.637
0.5	2.517		2.517
0.75	2.073		2.073
1.0	1.718		1.718
1.5	1.204		1.204
2.0	0.909	0.631	0.909
3.0	0.580	0.433	0.580
4.0	0.402	0.414	0.407
5.0	0.297	0.348	0.321
7.5	0.165	0.197	0.180
10.0	0.100	0.107	0.103

* 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, 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 SDS, SD1, and TL values obtained from the ASCE tool at <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 $S_a \times 1.5$ is the final MCE_R response spectral acceleration.

About UGMS

The UGMS MCE_R 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 $S_a \times 1.5$ is the final MCE_R response spectral acceleration.

1.0 Introduction.

The Application Page ([/ugms-mcerGM-tool_v18.4/](/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 ($=2/3 \times MCE_R$ 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/>) 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.