REPORT OF GEOTECHNICAL INVESTIGATION

Proposed Classroom Building

Will Rogers Middle School

4110 West 154th Street, City of Lawndale, California

Prepared For:

Lawndale Unified School District
c/o The Toth Group
11301 Legacy Terrace
San Diego, California 92131

Project No. 7083.22 **November 18, 2022**

2860 Walnut Avenue, Signal Hill CA 90755 * Tel: (562) 426-7990 * Fax: (562) 426-1842

SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

November 18, 2022
Project No. 7083.22

Lawndale Elementary School District
c/o The Toth Group
11301 Legacy Terrace
San Diego, California 92131

Attention: Mr. Stephen M. Ford

Subject: **Report of Geotechnical Investigation**
Proposed New Classroom Building
Will Rogers Middle School
4110 West 154th Street, City of Lawndale, California

Ladies and Gentlemen:

Presented herewith is the Report of Geotechnical Investigation (the Soils Report) prepared by Associated Soils Engineering, Inc. (ASE) for the proposed new 2-story classroom building (the Building) to be located on the campus of Will Rogers Middle School, located at 4110 West 154th Street, in the City of Lawndale, California (the Site). This work was conducted in accordance with ASE's Proposal No. P22-114, dated August 4, 2022, which subsequently received your authorization.

The subject geotechnical investigation was planned and performed based on the relevant development information provided by your office. Provided information included Site Accessibility Plan, produced by Westberg & White, dated July 12, 2022, on which was shown the layout of the existing, demolished and new school buildings.

The purpose of this study was to evaluate the subsurface soils conditions at the Site, followed by assessment of site geologic/seismic hazards, performance of engineering analyses, and formulation/assembly of recommendations for the geotechnical design and construction pertinent to the Building. ASE's study has concluded that construction of the Building is geotechnically feasible provided that the recommendations and design guidelines with respect to ground preparation and foundation construction presented in the Soils Report are incorporated in the project plans and design, and implemented during construction. This Soils Report also presents 1) the findings of the geotechnical field investigation, 2) the summary of potential geological/seismic hazard assessment, and 3) the results of laboratory tests performed.

We at ASE appreciate the opportunity to provide our professional services on this important project, and look forward to assisting you during the construction phase of the Building.

If you have any questions or require additional information, please contact the undersigned.

Respectfully submitted,
ASSOCIATED SOILS ENGINEERING, INC.

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1.0 INTRODUCTION

This Soils Report presents the results of ASE's geotechnical investigation for the proposed new 2-story classroom building (the Building) to be located on the campus of Will Rogers Middle School, located at 4110 West 154th Street, in the City of Lawndale, California (the Site). The approximate location of the Site is shown on Figure 1, Site Location Map. The purpose of this investigation was to evaluate the general subsurface soil conditions at the Site and provide geotechnical recommendations for the design and construction of the Building. This Soils Report presents the summary of the data collected, the results of ASE's engineering evaluations/analyses, and the pertinent geotechnical conclusions and recommendations.

1.1 Project Outline

ASE understands that the following project information is applicable at the time of this Soils Report preparation.

1.1.1 Building/Development Scope:

Based on the provided plans and information, ASE understands that the Building development is to construct a new 2-story classroom building. The roughly "∟"-shaped Building has an approximate footprint of 18,344 square feet. Other appurtenant improvements are likely to include landscaping, utilities, hardscape, and accessibility upgrades.

1.1.2 Structural Loading for Geotechnical Analyses:

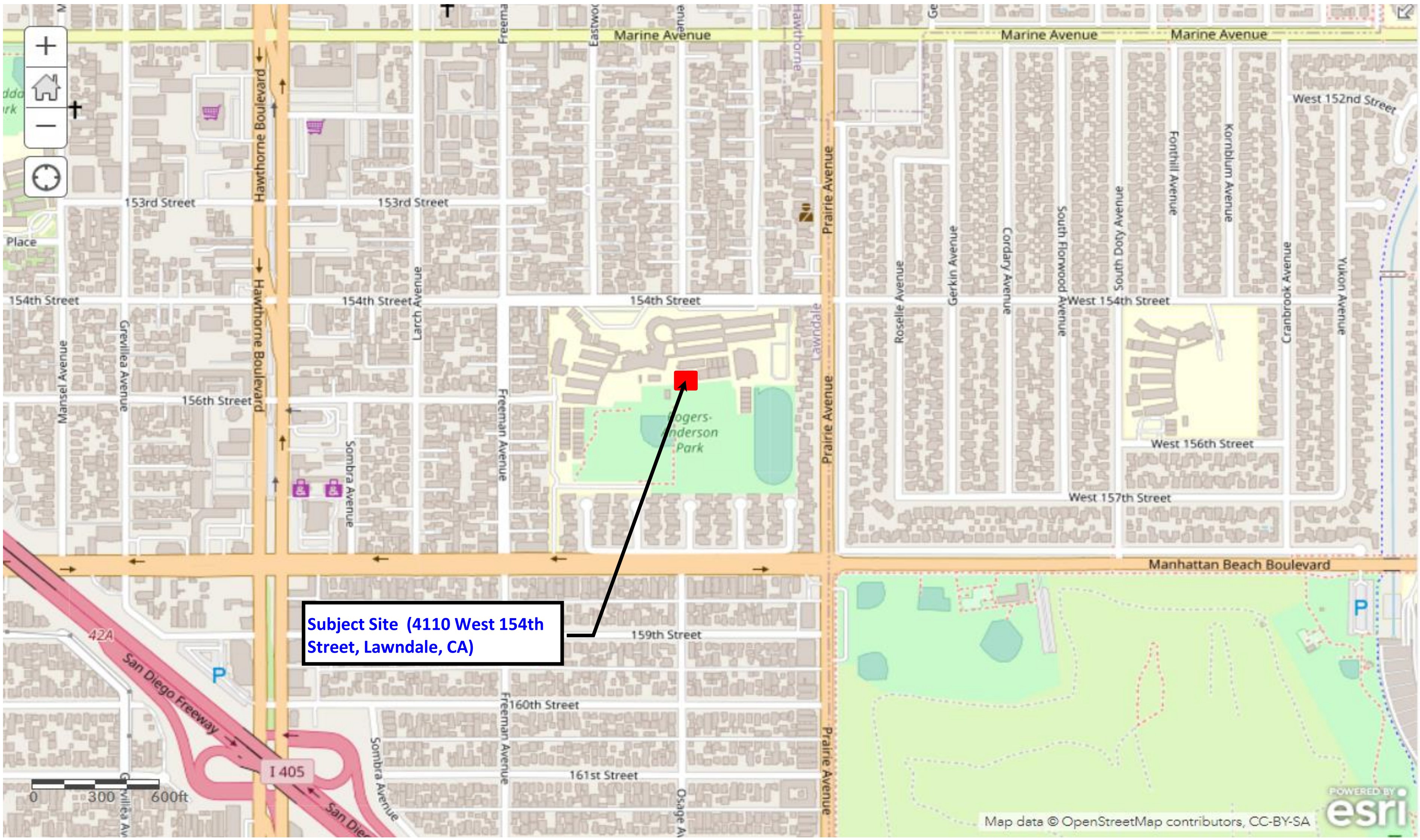
In the absence of structural loading information, ASE assumed that the Building will be supported by isolated pad footings and continuous spread footings, with maximum concentrated column load (dead plus live loads) on the order of 40 kips, and with a maximum line load (dead plus live loads) not exceeding 4,000 pounds per linear foot. Tolerable total and differential settlements resulted from the aforementioned structural loadings on the order of one (1) inch and one-half (1/2) inch, respectively, have also been assumed by ASE.

In anticipation of the presence of highly expansive soils, if the alternative post-tensioned (PT) slab is adopted, ASE has assumed a distributed load (D+L) on the order not exceeding 1,000 psf for our geotechnical analysis.

1.2 Scope of Exploration

In accomplishing the subject investigation, ASE's staff had performed the following geotechnical tasks:

A. Review of readily available background information, including in-house geotechnical data, geologic maps, seismic hazard maps, and literature relevant to the Site.



Associated Soils Engineering, Inc. **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	**Site Location Map**	Proj. Name:	**New Classroom Building at Will Rogers MS** **4110 West 154th Street, Lawndale, CA**	
		Figure 1	Proj. No.:	**7083.22**
			Date:	**November, 2022**

ASSOCIATED SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

B. A geotechnical site reconnaissance to observe the general surficial soil conditions at the Site and to select and mark boring locations, followed by notification to Underground Service Alert of the planned boring locations 72 hours prior to field exploration.

C. Field exploration consisting of drilling three (3) exploratory borings to depths ranging from 16 feet to 26 feet below respective existing grades. ASE staff logged and sampled representative soils encountered in each exploratory boring. Locations of the exploratory borings on site are shown on the Boring Location Plan, Plate A, in Appendix A.

D. Laboratory testing on retrieved representative soil samples for classification and for determination of pertinent engineering properties.

E. Engineering analyses of data obtained from literature review, site investigation and laboratory testing covering the following aspects:

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
- Assessment of geologic/seismic hazards based on the pertinent criteria required by the California Geological Survey (CGS) and the Division of the State Architect (DSA).
- Determination of the seismic design parameters in accordance with Chapters 16A and 18A of the California Building Code, 2019 Edition (2019 CBC; Reference 6), as well as the guidelines stipulated in the latest CGS Note 48 (Reference 26) and DSA IR A-4 (Reference 27).
- Evaluation of the suitability of on-site soils for foundation support, establishment of qualification criteria for on-site or imported fill material, and recommendations for site grading and subgrade preparation for the Building and other ancillary improvements.
- Recommendations for design of shallow footing foundations including minimum dimensions, allowable bearing capacity, estimated settlement, and lateral resistance.
- Recommendations for subgrade preparation and design parameters for alternative PT slab, slab-on-grade and flatwork.
- Recommendations for temporary excavation, shoring and trenching.
- Evaluation of the corrosion and expansion potential of the on-site materials.

G. Preparation of this Soils Report presenting the work performed and data acquired, as well as summarizing our conclusions and geotechnical recommendations for the design and construction of foundation supporting the Building.

Please note that ASE's geotechnical investigation did not include any evaluation or assessment of hazardous or toxic materials which may or may not exist on or beneath the Site. ASE does not consult in the field of potential site contamination/mitigation.

2.0 SITE AND SUBSURFACE CONDITIONS

2.1 Location, Boundary Conditions and Existing Development

The proposed classroom building (the Building) is to be located within the campus of Will Rogers Middle School, at 4110 154th Street, in the City of Lawndale, California.

The Building is to be located south-centrally within the campus, and immediately north of the sport fields. Existing school buildings are present to the immediate north. Existing gymnasium is present to the northeast.

The Building location is presently occupied by asphaltic concrete (AC) pavement. Three (3) existing portable classroom buildings are to be removed from the northwest corner of the Building footprint. The area is presently uniform and level.

2.2 Subsurface Conditions

2.2.1 Artificial Fill (af):

Artificial fill was encountered in the upper 2 and 3 feet in Borings B-1 and B-2, respectively, consisting mainly of clay with some sand in a soft to firm state. It is advisable the Owner/Architect/Civil Consultant should research available records to determine whether the encountered artificial fill was previously placed as a properly engineered fill. Until such records become available, the encountered artificial fill has been classified by ASE as "undocumented fill" and evaluated accordingly.

2.2.2 Older Alluvial Deposits (Qoa):

Native site soils consisting of older alluvial deposits were encountered in ASE's exploratory borings below the pavement section and artificial fill to the maximum explored depth of 26 feet. Per Reference 5, the older alluvial deposits are characterized as deposits associated with the elevated plain which consist of stiff to hard clay and medium dense to very dense sand, silty sand, clayey sand and silt. In specific, site alluvial soils predominantly contain silty clays, clays, and clayey silts.

Blow counts recorded from advancing Modified California barrel sampler empirically indicate that the native, cohesive, alluvial materials were generally moderately stiff to stiff in consistency and slightly moist to very moist in moisture content within the explored depths, at the time of ASE's site investigation.

More detailed descriptions of soils encountered and conditions observed during the subsurface exploration are shown in the Field Logs of Boring B-1 through B-3, in Appendix A, together with information including soil classifications, depths and types of soil samples, blow counts, field dry densities and moisture contents, and corresponding laboratory tests performed.

2.3 Groundwater and Caving

During field exploration, groundwater was encountered within Boring B-2 at a depth of 24 feet below existing grade. Published data in Reference 5 indicates a historic high groundwater contour of 30 feet in the Site vicinity. The Site elevation is approximately 44 feet above Mean Sea Level (MSL) per Google Earth.

Information from the State of California Water Resources Control Board Geotracker website (http://geotracker.waterboards.ca.gov) indicates that the groundwater elevation in groundwater monitoring well MW-5, located near the southwest corner of Hawthorne Boulevard and 154th Street (P & M #911 (Auto Max): 15407 South Hawthorne Boulevard - approximately 1/2 mile west of the Site), was 10.03 feet below grade on February 7, 2005. The ground surface elevation at this well location per Google Earth is approximately 52 to 53 feet above MSL, which is approximately 8 to 9 feet higher than site grades. The depth to groundwater for the most recent reading in this well (taken June 11, 2012) was 12.7 feet below well grade.

Please note that seasonal and long-term fluctuations in the groundwater may occur as a result of variations in subsurface conditions, rainfall, run-off conditions and other factors. Therefore, variations in groundwater levels from the short-term observations made in ASE's exploratory borings cannot be ruled out. Please note that ASE's exploratory borings were not meant for groundwater monitoring purpose.

While the use of hollow-stem augers during drilling precluded observation of potential caving conditions, caving and/or sloughing was not noticed during the extraction of auger stem at the completion of drilling operations. However, caving and/or soil sloughing could not be totally ruled out in excavations greater in dimension/depth than ASE's exploratory borings.

2.4 Utilities

No overhead or underground utilities were disturbed during the course of ASE's site exploration. However, overhead and underground utilities are present within the school campus, as well as along some site perimeters and nearby streets. Other utilities, though not known at the time of this report preparation, may be present on site, and should be located prior to the start of site grading work.

3.0 GEOLOGY

3.1 Regional Geologic Setting

The Site is located in the Central Block of the Los Angeles Basin. The Los Angeles Basin is a large northwest trending synclinal depression at the southern end of the Transverse Ranges and at the northwestern end of the Peninsula Range geomorphic Provinces of California. The Central Block is bounded by the active Newport-Inglewood Fault Zone (located 3.8 mile (6.1 km) southwest of the Site) and the active Whittier Fault Zone (approximately 19.9 miles (32.1 km) northeast of the Site).

3.2 Geologic and Soil Units

3.2.1 Late Pleistocene-Age Older Alluvial Deposits (Qoa):

The native soils encountered in every boring extending to the respective maximum explored depth consist predominantly of Late-Pleistocene Age older alluvial deposits. In accordance with Reference 5, soils within the unit of older alluvium deposits consist of firm to hard clay and loose dense to dense sand, silty sand, clayey sand and silt. In specific, on-site alluvial soils predominantly contain silty clays, clays, and clayey silts. Figure 2, Local Geologic Map, excerpt from Reference 5, shows Quaternary geology in the Site vicinity.

4.0 FAULTING AND SEISMICITY

Lawndale, like the rest of southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto, Newport-Inglewood and Whittier-Elsinore fault zones.

By the definition of CGS, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 1,600,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault Zones. The intent of the act is to require fault investigations on sites located within Special Studies Zones to preclude new construction of certain inhabited structures across the trace of active faults. The Site is not located within the Alquist-Priolo Earthquake Fault Zone. No evidence of active or potentially active faulting was observed during our investigation.

Several sources were researched for information pertaining to site seismicity. The majority of data was obtained from the program, EQFAULT, by Blake (2000) that allows for an estimation of peak horizontal ground acceleration (PGA) using a data file of approximately 150 digitized California faults. This program compiles information including the dominant type of faulting within a particular region, the maximum earthquake magnitude each fault is capable of generating, the estimated slip-rate for each fault, and the approximate location of the fault trace. Printouts of the results of the fault search for the Site are shown as Plates I-1 and I-2 in Appendix B. Regional Fault Map, Plate J-1, U.S. Geological Survey Quaternary Faults, in Appendix B, shows the major active faults in Southern California near the Site.

(Partial Extract of the Quaternary Geologic Map of the Inglewood 7.5-Minute Quadrangle,California Division of Mines and Geology,Seismic Hazard Zone Report 027, 1998)

LEGEND

■	Approximate Site Location
Qoa	Older Alluvium consisting of stiff to hard clay and medium dense to very dense sand, silty sand, clayey sand and silt.

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Project:	**New Classroom Building at Will Rogers MS 4110 West 154th Street, Lawndale, CA**		
Figure 2	**Local Geologic Map**		
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ASE has also searched the Center for Strong Motion data (CESMD) website (https://www.strongmotioncenter.org/cgi-bin/CESMD/iqrEventMap.pl) for historical strong earthquake events near the Site. Plate J-2 (2 sheets), Earthquakes with Strong Motion Records in CESMD, in Appendix B, shows the epicenter locations, occurrence dates and magnitudes of historical earthquakes greater than magnitude 3.5 within an area measuring roughly 60 miles x 80 miles with Lawndale at the center. Among all the significant historical earthquakes, the closest to the Site was the Whittier Earthquake of 6.1 Mw (moment magnitude per USGS) taking place on October 1, 1987.

4.1 Deterministic Analysis

The Site is likely to be subject to strong seismic ground shaking during the life of the project. Based on the referenced literature and deterministic analysis performed with the EQFAULT software, the Newport-Inglewood (L.A. Basin) Fault, approximately 3.8 miles (6.1 km) from the Site, would probably generate the most severe site ground motions. A Maximum Probable Earthquake (MPE), i.e. the maximum earthquake that is considered likely to occur during a 100-year time interval, of 7.1 Mw (moment magnitude as per USGS) has been assessed along the Newport-Inglewood (L.A. Basin) Fault. As shown on Plate I-2 in Appendix B, estimated PGA resulting from a MPE event on the Newport-Inglewood (L.A. Basin) Fault is on the order of 0.403g should this event occur at the fault's closest approach to the Site. Other nearby active faults include the Palos Verdes Fault and the Puente Hills Blind Thrust Fault, located approximately 4.9 miles (7.9 km) and 11.2 miles (18.0 km) away, respectively. In sum, approximately 45 active or potentially active faults have been found within 62 miles (100 km) of the Site.

4.2 Probabilistic Analysis

The seismicity of the Site was evaluated utilizing probabilistic analysis available from USGS Unified Hazard Tool (https://earthquake.usgs.gov/hazards/interactive/). The Maximum Probable Earthquake (MPE) and the Maximum Considered Earthquake (MCE) that carry 10 percent and 2 percent exceedance probabilities, respectively, in 50 years have been considered. Based on a typical damping ratio of 5% and a V_s^{30} value of 360 m/sec, corresponding with Site Class C/D boundary, nearest to the derived a V_s^{30} value of 387 m/sec from the "Set Site Parameters for Web Services"'' function as part of the "Hazard Spectrum Calculator (Local)" application available from the "OPENSHA" website, three spectral acceleration values representing peak ground acceleration (PGA), spectral acceleration for structural period of 0.2 second (Sa – 0.2 sec; typical of low-rise buildings) and spectral acceleration for structural period of 1.0 second (Sa – 1.0 sec; typical of multi-story buildings) have been analyzed and are tabulated below.

Seismic Acceleration Values from USGS Unified Hazard Tool [3]

Latitude	Longitude	V_s^{30} (m/sec)	Scenario	Acceleration (g)		
				PGA	Sa – 0.2 sec	Sa – 1.0 sec
N 33.8899°	W 118.3454°	360	MPE [1]	0.475	1.137	0.525
			MCE [2]	0.857	2.043	1.105

1. MPE scenario carries a 10% exceedance probability in 50 years.
2. MCE scenario carries a 2% exceedance probability in 50 years.
3. Edition: Dynamic: Conterminous U.S. 2014 (v4.2.0)

4.3 2019 CBC Seismic Design Parameters

The earthquake design requirements listed in 2019 CBC and other governing standards account for faults classified as "active", in accordance with the most recent fault listing as per the United States Geological Survey (USGS) or the CGS. The seismic design of the proposed structures should be implemented in accordance with the applicable provisions stipulated in 2019 CBC unless otherwise specified by the governing authority having jurisdiction over the project.

The 2019 CBC seismic design criteria for the Site based on a Site Class of "D" for the criterion based on site Vs value stipulated in Table 20.3-1 of ASCE 7-16 (Reference 13), a Risk Category II and a scenario of Risk-Targeted Maximum Considered Earthquake (MCE_R) that carries a 2% exceedance probability in 50 years had been determined utilizing the OSHPD Seismic Design Maps web-application (http://seismicmaps.org) and the criteria stipulated in Chapters 11 and 12 of Reference 13. Summaries of the seismic coefficients for the Site are tabulated below.

2019 CBC SEISMIC DESIGN PARAMETERS					
Site Latitude:	N 33.8899°	**Site Longitude:**	W 118.3454°	**Risk Category** [a]	II

Seismic Parameter	**Recommended Value**
Site Class [b]	D
Soil Profile Name [b]	Stiff Soil
Site Coefficient, Fa [c]	1.0
Site Coefficient, Fv [d]	1.7
0.2-Second Spectral Response Acceleration, S_S [e]	1.799g
1.0-Second Spectral Response Acceleration, S_1 [f]	0.635g
Adjusted 0.2-Second Spectral Response Acceleration, S_{MS} [g]	1.799g
Adjusted 1.0-Second Spectral Response Acceleration, S_{M1} [h]	1.079g
Design 0.2-Second Spectral Response Acceleration, S_{DS} [i]	1.199g
Design 1.0-Second Spectral Response Acceleration, S_{D1} [j]	0.720g
Long -Period Transition Period, T_L [k]	8 sec
Mapped MCE_G Geometric Mean Peak Ground Acceleration, PGA [l]	0.778g
Site Coefficient, F_{PGA} [m]	1.1
MCE_G Peak Ground Acceleration adjusted for Site Class Effect, PGA_M [n]	0.856g

Risk Category	**I or II or III**	**IV**
Seismic Design Category based on SD_1 [o]	D	D
Seismic Design Category based on SD_S [P]	D	D

a Per 2019 CBC Table 1604A.5
b Per 2019 CBC Section 1613A.2.2
c *Per 2019 CBC Table 1613A.2.3(1). Note: If simplified design procedure of Section 12.14 of ASCE 7-16 is adopted, the Fa value should be determined per Section 12.14.8.1 of ASCE 7-16 with no need for Fv, S_{MS}, S_{M1} values.*
d Per 2019 CBC Table 1613A.2.3(2), *provided Cs values are determined by Equations 12.8-2, 12.8-3 and 12.8-4 of ASCE 7-16.*
e Per 2019 CBC Figure 1613A.2.1(1)
f Per 2019 CBC Figure 1613A.2.1(2)
g Per 2019 CBC Equation 16A-36
h Per 2019 CBC Equation 16A-37
i Per 2019 CBC Equation 16A-38
j Per 2019 CBC Equation 16A-39
k Per ASCE 7-16 Figure 22-14
l Per ASCE 7-16 Figure 22-9
m Per ASCE 7-16 Table 11.8-1
n Per ASCE 7-16 Equation 11.8-1 = PGA x F_{PGA}
o Per 2019 CBC Section 1613A.2.5
p Per 2019 CBC Section 1613A.2.6

Please note that conformance to the 2019 CBC seismic design criteria does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not take place during the occurrence of a MCE_R event. The primary goal of seismic design is to protect life and not to avoid all damage, since such design may be economically prohibitive. Following a major earthquake, a building may be damaged beyond repair, yet not collapse. The Structural Consultant should review the pertinent parameters to evaluate the seismic design.

Per CGS Note 48 (Reference 26), seismic design parameters for Site Classes "D", "E" and "F" should be obtained from site-specific seismic hazard analysis unless exceptions stipulated in Section 11.4.8 of ASCE 7-16 are invoked. The values listed in the table on the previous page reflect such invocation of exceptions (see Footnotes c and d beneath the said table). If the structural design of the Building cannot be supported by the invoked exceptions, the Geotechnical Consultant should be contacted for performing additional, site-specific seismic hazard analysis such that values of site-specific seismic design parameters could be established.

5.0 GEOLOGIC HAZARDS

5.1 Surface Fault Rupture and Ground Shaking

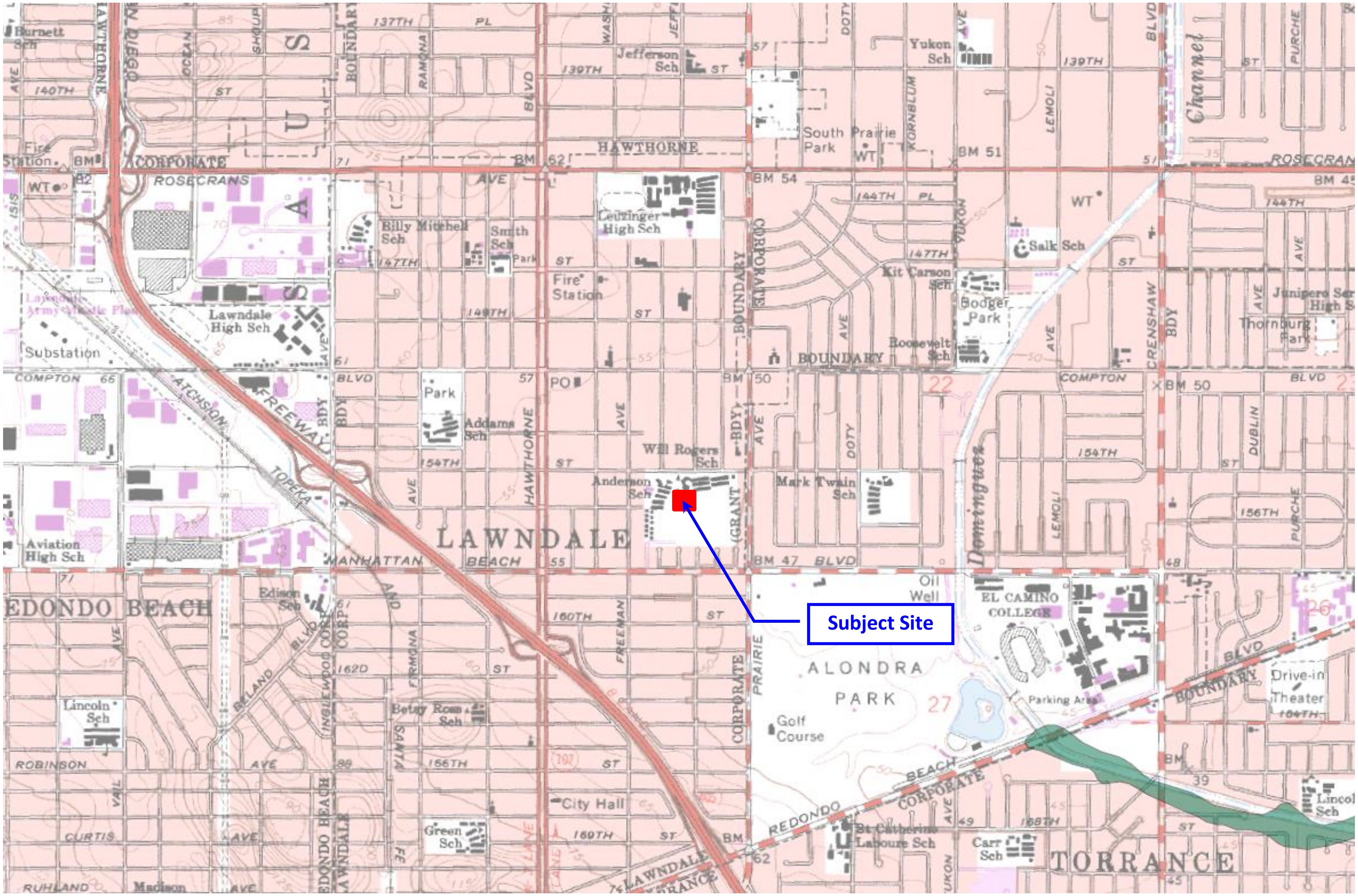
The Site is not located within an Alquist-Priolo Earthquake Fault Zone. No known active or potentially active faults are shown crossing the Site on published maps reviewed. No evidence for active faulting was encountered in the exploratory excavations performed during this evaluation. The risk of surface rupture at the Site is considered very low. However, being in close proximity to several known active and potentially active faults, severe ground shaking should be expected during the life of the proposed development.

5.2 Seismic Hazards

5.2.1 Liquefaction:

The term "liquefaction" describes a phenomenon in which a saturated cohesionless soil loses strength and acquires a degree of mobility as a result of strong ground shaking during an earthquake. The factors known to influence liquefaction potential include soil type and depth, grain size, relative density, groundwater level, degree of saturation, and both the intensity and duration of ground shaking. The soils to the maximum explored depth of 26 feet generally consist of moderately stiff to very stiff, cohesive, clayey soils. As evidenced in Figure 3, Local Seismic Hazard Map, the school campus is not within an area identified as having a potential for soil liquefaction when subject to a MPE or MCE event.

During ASE's field exploration, groundwater (possible perched) was encountered at a depth of 24 feet below existing grade in exploratory Boring B-2. Per Reference 5, the historic high groundwater in the vicinity of the Site is 30 feet below grade. According to the information available from the



(Partial Extract of the Earthquake Zones of Required Investigation Inglewood 7.5-Minute Quadrangle, California Geological Survey, dated 1999)

LEGEND

- Approximate Site Location
- Potential Landslide Hazard Area
- Potential Liquefaction Hazard Area
- Earthquake Fault Zone

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	Figure 3	**Local Seismic Hazard Map**	
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State of California Water Resources Control Board Geotracker website, historic high groundwater in a well located approximately 0.4 mile east of the Site is approximately 10.03 feet deep (with well grade 8 to 9 feet <u>higher</u> than the Site grade).

Considering that 1) groundwater (possibly perched) <u>was</u> encountered at a depth of 24 feet below existing grade, 2) historic high groundwater in a well near the vicinity of the subject Site is 10.03 feet deep based on ASE's groundwater monitoring data review, and 3) the soils to the maximum explored depth of 26 feet, and likely the same beyond, generally consist predominantly of moderately to very stiff clays, the likelihood of occurrence of seismically-induced liquefaction at the Site is deemed nil.

5.2.2 Seismic Settlements:

Ground accelerations emitted from a seismic event can cause densification of loose soils both above and below the groundwater table that may result in settlements on ground surface due to volumetric compression of soil mass. This phenomenon is often referred to as seismic settlement and commonly takes place in relatively clean sands, as well as soils with low plasticity and less fines. As the site soils encountered consist predominantly of stiff to hard clayey soils, the seismically-induced volumetric contraction of soils above the historic high ground water level (i.e. "dry" seismic settlement) upon the impact of MPE/MCE events is not anticipated to exceed 1/4 inch, with minimum differential seismic settlement.

5.2.3 Earthquake-Induced Landslides:

ASE's review of the same geohazard map that was based upon for the production of Figure 3 indicates that the Site <u>is not</u> located within an area identified as having a potential for earthquake-induced landslides. There is no indication that recent landslides or unstable slope conditions exist on or adjacent to the Site that would otherwise result in an obvious landslide hazard to the proposed development or adjacent properties. The potential for earthquake induced landslides at the Site in the future is considered nil.

5.2.4 Lateral Spreading:

Lateral spreading, a phenomenon associated with seismically-induced soil liquefaction, is a display of lateral displacement of soils due to inertial motion and lack of lateral support during or post liquefaction. It is typically exemplified by the formation of vertical cracks on the surface of liquefied soils, and usually takes place on gently sloping ground or level ground with nearby free surface such as drainage or stream channel. Since the Site has been evaluated in Section 5.2.1 above not to be susceptible to seismically-induced liquefaction, the potential for the occurrence of liquefaction-induced lateral spreading is deemed nil at the Site.

5.2.5 Tsunamis and Seiches:

Due to the elevation of the Site and absence of nearby waterfront, hazard from tsunami is considered very low.

Seiches are rhythmic movements of water within a lake or other enclosed or semi-enclosed body of water, generally caused by earthquakes. Since no lakes or other enclosed bodies of water lie on or near the Site, the hazard from seiches is not present at the Site.

5.2.6 Flood Hazards:

The Site was located on the ESRII/FEMA Hazard Awareness site. The Site is not located within the limits of the 100-year flood plain per FEMA Flood Insurance Rate Map (Map No. 06037C1790F, map revised September 26, 2008), and is located outside an area of 0.2-percent-annual-chance flood as shown on Figure 4, National Flood Hazard Layer FIRMette.

6.0 GEOTECHNICAL CONSIDERATIONS AND RECOMMENDATIONS

Based on the results of field exploration, laboratory testing, and engineering analysis, it is ASE's geotechnical opinion that the major geotechnical factors affecting the design and construction of the Building include the following:

1. Soil disturbances as a result of site demolition and clearing operations.
2. Presence of soils that exhibit "Very High" expansion potential at shallow depth that are likely heave or shrink significantly and unevenly upon saturation and drying, respectively, resulting in potentially excessive uneven displacement of overlying foundations, structural improvements, and flatworks.

In consideration of the above factors, it is ASE's opinion that the native soils underlying the Building construction will provide satisfactory bearing stratum for structure support provided that the foundation design accounts for the presence of "Very High" expansive subgrade soils. It is also ASE's opinion that overexcavation and backfilling with properly compacted suitable fill in the Building area, as recommended herein, will be essential to reduce unfavorable slab displacement as a consequence of settlement or heaves of underlying clayey soils. The grading recommendations provided herein should be reviewed when final project concept and grading plans become available. It is assumed that the proposed finish grades will be close to the existing site grades (± one foot).

Conventional foundations comprising continuous spread footings and isolated pad footings bearing on compacted blended or lime treated soils, together with slab-on-grade may be considered for structural support. In view of the presence of "Very High" expansive site soils, PT slab may be considered for structural support of the Building.

ASE#7083.22

National Flood Hazard Layer FIRMette FIGURE 4

FEMA

118°21'3"W 33°53'39"N

118°20'25"W 33°53'9"N

0 250 500 1,000 1,500 2,000 Feet

1:6,000

Basemap: USGS National Map: Orthoimagery: Data refreshed October, 2020

Legend

SEE FIS REPORT FOR DETAILED LEGEND AND INDEX MAP FOR FIRM PANEL LAYOUT

SPECIAL FLOOD HAZARD AREAS		Without Base Flood Elevation (BFE) *Zone A, V, A99*
		With BFE or Depth *Zone AE, AO, AH, VE, AR*
		Regulatory Floodway
OTHER AREAS OF FLOOD HAZARD		0.2% Annual Chance Flood Hazard, Areas of 1% annual chance flood with average depth less than one foot or with drainage areas of less than one square mile *Zone X*
		Future Conditions 1% Annual Chance Flood Hazard *Zone X*
		Area with Reduced Flood Risk due to Levee. See Notes. *Zone X*
		Area with Flood Risk due to Levee *Zone D*
OTHER AREAS	NO SCREEN	Area of Minimal Flood Hazard *Zone X*
		Effective LOMRs
		Area of Undetermined Flood Hazard *Zone D*
GENERAL STRUCTURES		Channel, Culvert, or Storm Sewer
		Levee, Dike, or Floodwall
OTHER FEATURES	B 20.2 17.5	Cross Sections with 1% Annual Chance Water Surface Elevation
	8	Coastal Transect
	513	Base Flood Elevation Line (BFE)
		Limit of Study
		Jurisdiction Boundary
		Coastal Transect Baseline
		Profile Baseline
		Hydrographic Feature
MAP PANELS		Digital Data Available
		No Digital Data Available
		Unmapped

N

The pin displayed on the map is an approximate point selected by the user and does not represent an authoritative property location.

This map complies with FEMA's standards for the use of digital flood maps if it is not void as described below. The basemap shown complies with FEMA's basemap accuracy standards

The flood hazard information is derived directly from the authoritative NFHL web services provided by FEMA. This map was exported on 11/18/2022 at 12:37 PM and does not reflect changes or amendments subsequent to this date and time. The NFHL and effective information may change or become superseded by new data over time.

This map image is void if the one or more of the following map elements do not appear: basemap imagery, flood zone labels, legend, scale bar, map creation date, community identifiers, FIRM panel number, and FIRM effective date. Map images for unmapped and unmodernized areas cannot be used for regulatory purposes.

6.1 Site Preparation

6.1.1 Existing Improvements:

Prior to grading operations, it will be necessary to remove designated existing construction, including any remaining buried obstructions, which may be in the areas of the Building. Structure removal should include foundations. Concrete flatwork and AC pavements should also be removed from the areas of proposed construction. Concrete and asphalt fragments from site demolition operations should be disposed of off-site, unless they can be stockpiled and processed to meet the specifications for Crushed Miscellaneous Base ("CMB"), Processed Miscellaneous Base ("PMB") or Pulverized Miscellaneous Base ("PMB") as outlined in Sections 200-2.4, 200-2.5 or 200-2.8, respectively, of the latest edition of the "Standard Specifications for Public Works Construction" (the Greenbook) and reused as approved fill or base material.

6.1.2 Surface Vegetation:

Surface vegetation should be stripped from areas of proposed construction. Stripping should penetrate six (6) inches into surface soils. Any soil contaminated with organic matter (such as root systems or strippings mixed into the soil) should be disposed of off-site or set aside for future use in non-structural landscaped areas. Removal of trees and shrubs should include rootballs and attendant root systems.

6.1.3 Underground Utilities:

Any underground utilities to be abandoned within the zone of proposed construction should be cut off a minimum of five (5) feet from the area of the new structures. The ends of cut-off lines should be plugged a minimum of five (5) feet with concrete exhibiting minimum shrinkage characteristics to prevent water migration to or from hollow lines. Capping of lines may also be required should the plug be subject to any line pressure. Alternatively, deep hollow lines may be left in place provided they are filled with concrete or 2-sack control density fill (slurry fill). No filled line should be permitted closer than two (2) feet from the bottom of future footings, unless it has been evaluated and approved by the Geotechnical Consultant.

However, local ordinances relative to abandonment of underground utilities, if more restrictive, will supersede the above minimum requirements.

6.2 Site Grading

In view of reducing the adverse effects associated with the development of excessive total or differential static settlement and/or heave underneath the Building construction, as well as to ensure uniform bearing competency for the foundation, and slabs, engineered improvements of subgrade soils are recommended as follows.

6.2.1 Undocumented Fill/Disturbed Native Soils:

Any undocumented fill soil encountered during site grading in the areas of the Building, as well as any native soils disturbed during demolition and clearing operations, should be excavated full depth under the observation and confirmation by the Geotechnical Consultant. Lateral extent of overexcavation beyond the perimeters of the Building, where possible, should be to a minimum distance equal to the depth of undocumented fill/disturbed soil encountered or two (2) feet, whichever is greater.

For other secondary improvements such as free-standing walls or hardscape, the lateral extent of removal should be to a minimum distance equal to the depth of undocumented fill/disturbed soils encountered or one (1) foot, whichever is greater.

The exposed excavation bottom should be scarified/reworked to a minimum one (1) foot depth and recompacted to a minimum 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content, prior to backfilling with approved soils as specified in Section 6.2.7.

6.2.2 Expansive Soils:

Laboratory test result on a near surface soils sample indicates a "Very High" soil expansion potential (i.e. Expansion Index, EI = 136 per ASTM D4829-19 Test Method) as defined in 2019 CBC. The recommendations of remedial grading and design of structural elements in contact with on-site soils should take the "Very High" expansion potential into consideration, as appropriate. However, for foundations and structural elements supported by approved fill materials complying with criteria stipulated in Section 6.2.6 below, or by recompacted site soils treated with quicklime as per Section 6.2.2 below, the very low or significantly reduced soils expansion potential is not anticipated to be of significant structural concern. Nonetheless, it may be desirable that the soil expansion potential be re-evaluated through additional testing during or after rough grading operations to verify the design adequacy of foundation or slab-on-grade against the re-tested soil expansion potential as heterogeneity within soil mass is not uncommon.

Lightly loaded structural elements such as shallow foundations and slabs are likely to undergo significant movements due to the "Very High" expansion potential of site clayey soils. Such magnitude of movement could potentially cause noticeable distress such as cracks, deformation and/or misalignments to the overlying foundations, slabs, or walls. It should be noted that design provisions, such as the use of “Very Low” to “Low” expansive fill or lime-treated soils beneath lightly loaded structural elements, adequate reinforcements, measures to minimize moisture fluctuations in foundation soils, deeper foundations or other measures, as presented in Sections 6.2 and 6.3 of this Soils Report, may help to alleviate the effects of “Very High” soils expansion on the foundations and structures but may not completely eliminate the problem.

6.2.3 Remedial Grading:

To provide acceptable support for slabs within the Building, it is recommended that on-site soils within the footprints of the building pads be overexcavated and removed uniformly to a minimum depth of five (5) feet below existing or finish grade, whichever is lower, and five (5) feet laterally beyond the Building footprint, and replaced with properly compacted fill such that the Building foundations are supported on a re-engineered, competent fill layer.

The upper three (3) feet of compacted fill should consist of "Very Low" to “Low” expansive, granular material (Expansion Index, "EI" per ASTM D4829-18 test procedures not greater than <u>50</u>), compacted to at least 90 percent relative compaction with minimum moisture content of two (2) percentage points <u>above</u> optimum moisture content. On-site clayey soils <u>at their present state</u> generally exhibit an EI far exceeding the preferred value and, thus, are not deemed suitable for re-use as fill within the upper three (3) feet from finish grade. As such, suitable import sources to be used as fill for the upper three (3) feet of compacted fill should be identified, pre-tested and pre-approved prior to the onset of site rough grading. Alternatively, blending “Very Low” expansive (EI ≤ 20) sandy soils, pulverized PCC or AC, or CAB material with the excavated on-site soils to lessen the resultant EI may be considered. The blended soil should comply with the fill criteria stipulated in Section 6.2.7 below. The blending should ensure a thorough mixture of various soils and materials, and should be subject to testing and approval by the Geotechnical Consultant prior to use.

A third alternative for remedial grading is to treat the overexcavated on-site soils with quicklime. To provide acceptable support for slabs within the Building, it is recommended that on-site soils within the footprints of the building pads be overexcavated and removed uniformly to a minimum depth of five (5) feet below existing or finish grade, whichever is lower, and five (5) feet laterally beyond the Building footprint and replaced with lime-treated, properly compacted fill such that the Building slabs are supported on a lime-treated, compacted fill layer. Due to the predominantly "CH" classification of on-site soils, by blending 5 to 8 percent of quicklime (by weight of dry on-site soils) into the overexcavated soils during grading and compaction it would chemically alter and stabilize the clayey soil into a less expansive state, thus providing a more stable subgrade for the support of shallow foundation and slabs. Guidelines and procedures for lime treatment of soils stipulated in Section 301-5 of the Greenbook should be followed if this alternative is to be adopted. For the determination of optimum lime content, it is advisable that laboratory mix design be performed by the Geotechnical Consultant prior to the proceeding of full-scale site grading.

Soils exposed at excavation bottoms to a depth of six (6) inches should be scarified, reworked and recompacted to exhibit a minimum 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content, prior to receiving fill placement. The exposed excavation bottom should be observed, tested, and approved by the Geotechnical Consultant prior to placing compacted fill.

In case of the presence of localized loose soils, the overexcavation needs to be deepened accordingly to delete the loose soil condition. However, this deepened overexcavation may be terminated when the exposed native, undisturbed soils exhibit a natural relative compaction greater than 85 percent, subject to the testing and inspection by the representative from the Geotechnical Consultant.

The Geotechnical Consultant should be provided with appropriate construction details and staking during grading to verify that depths and/or locations of the recommended overexcavation are adequate. For areas on site that grading recommendations stipulated in both Sections 6.2.1 and 6.2.3 apply, the more stringent grading criteria between the two sections should govern.

The depth of overexcavation should be reviewed by the Geotechnical Consultant during the actual construction. Any subsurface obstruction, buried structural elements, and unsuitable material encountered during grading, should be immediately brought to the attention of the Geotechnical Consultant for proper exposure, removal and processing, as recommended.

6.2.4 Pumping and Heaving of Subgrade Soils:

If site soils within the zone of recommended remedial grading are found to contain high moisture content at the time of remedial grading, pumping and/or heaving conditions may be experienced during site overexcavation and grading operations. When soils exhibiting high moisture content are encountered during site grading, the grading contractor may need to use track-mounted equipment in lieu of rubber tire mounted, wherever possible.

If excavated site soils are highly moist or wet at the time of site grading, the excavated soils should be evenly spread and left aerated for a period of time to reduce the moisture content prior to being re-used. If project schedule does not allow prolonged drying of any encountered wet site soils, alternative measures such as using or blending with site drier soils or imported “Very Low” to “Low” expansive (EI ≤ 50) fill, pulverized PCC or AC, or CAB material, if available, may be considered.

The Geotechnical Consultant should be consulted at the time of site grading for the assessment of such alternative measures.

When pumping and heaving conditions are observed with soils exposed at the overexcavation bottom, it will be necessary to deepen the overexcavation within the area of observed pumping and heaving an additional 12 to 18 inches, under the observation and evaluation of the Geotechnical Consultant's representative. The area of the deepened overexcavation should subsequently be backfilled with 3/4" to 1" open-graded crushed rocks complying with the criteria of Section 200-1.2 of the current edition of the Standard Specifications for Public Works Construction (the Greenbook) rammed tightly to form a stiffened platform to facilitate the ensuing fill placement and compaction. The crushed rock layer should be <u>underlain</u> and <u>overlain</u> by a layer of filtering geofabric (such as Mirafi 140N or equivalent) prior to and subsequent to the placement of crushed rocks, respectively. If necessary, alternative and site-specific recommendations for the stabilization of excavation bottoms may be provided by the Geotechnical Consultant based on a review of grading-exposed soil conditions.

6.2.5 Temporary Excavation:

Excavations of site soils 4 feet or deeper should be temporarily shored or sloped in accordance with Cal OSHA requirements.

a) Temporary Sloping:

In areas where excavations deeper than 4 feet are not adjacent to existing structures or public right-of-ways, sloping procedures may be utilized for temporary excavations. It is recommended that temporary slopes in both fill and native soils be graded no steeper than 3/4:1 (H:V) for excavations up to 10 feet in depth. The above temporary slope criteria is based on level soils conditions behind temporary slopes with no surcharge loading (structures, traffic) within a lateral distance behind the top of slope equivalent to the slope height.

It is recommended that excavated soils be placed a minimum lateral distance from top of slope equal to the height of slope. A minimum setback distance equivalent to the slope height should be maintained between the top of slope and heavy excavation/grading equipment.

Soil conditions should be reviewed by the Geotechnical Consultant as excavation progresses to verify acceptability of temporary slopes. Final temporary cut slope design will be dependent upon the soil conditions encountered, construction procedures and schedule.

b) Temporary Shoring:

Temporary shoring will be required for those excavations where temporary sloping as specified above is not feasible.

Temporary cantilever shoring, if used, should be designed to resist an active earth pressure of 53 pounds per cubic foot (pcf) equivalent fluid pressure (EFP) for level soil conditions behind shoring. The resultant lateral deflection of shoring and surficial settlement immediately behind shoring are estimated to be on the order of one (1) to one and one half (1 ½) percent of the shored excavation depth. Should this ground deformation be intolerable to the existing structure, ASE should be consulted for more detailed analysis and further recommendations.

Please note that, due to the presence of "Very High" expansive subgrade soils, temporary shoring could be subjected to excessively high lateral expansive pressure should the surrounding subgrade soils be subjected to prolonged moisture inundation from leaky plumbing or heavy rainfall. Should exposure to moisture inundation be a concern during the course of temporary excavation on site, alternative braced excavation may need to be considered and the Geotechnical Consultant should be consulted for pertinent recommendations.

The design of shoring should also include surcharge loading equivalent to one-third (1/3) of the loading of existing structures and anticipated traffic, including delivery and construction equipment, when such loading is within a distance from the shoring equal to the depth of excavation. In addition, a minimum uniform lateral pressure of 100 pounds per square foot (psf) in the upper ten (10) feet of shoring should be considered in the design when normal traffic is permitted within ten (10) feet of the shoring.

6.2.6 Exterior Slab-on-Grade/Flatwork/Pavement/Hardscape Support:

For the purpose of reducing future unsightly and uneven movements and cracks of any exterior slab-on-grade, concrete flatwork, or hardscape, it is recommended that subgrade soils to a minimum depth of eighteen (18) inches below the bottom of and eighteen (18) inches laterally beyond the footprint of exterior concrete slab-on-grade/concrete flatwork/pavement/hardscape should be overexcavated then backfilled and recompacted with suitable fill soils consisting of "Very Low" to "Low" expansive (EI ≤ 50), suitable site, import or blended material, compacted to at least 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content. Prior to placement of compacted fill, the upper six (6) inches of exposed native subgrade should be reworked to at least 90 percent relative compaction with a minimum moisture of two (2) percentage points above optimum moisture content. However, if undocumented fill is encountered in these areas, recommendations stipulated in Section 6.2.1 above should be complied with.

Alternatively, quicklime or cement treatment mentioned in Section 6.2.3 above may also be considered for the stabilization and re-engineering of the upper eighteen (18) inches of subgrade soils beneath exterior concrete slab-on-grade/concrete flatwork/ Pavement/hardscape. Should the Owner decide to use on-site clayey soils without either blending with "Very Low" to “Low” expansive import soils or treatment with quicklime for exterior concrete flatwork backfill, it is very likely that more frequent maintenance and repair of unsightly cracks and movements will be required due to the “Very High” expansive nature of site soils.

From geotechnical viewpoint, new landscape area with only softscape is not subject to subgrade preparation and remedial grading requirements mentioned in Sections 6.2.1, 6.2.3 and 6.2.6.

6.2.7 Suitable Soils and Imported Soils:

Unless otherwise noted, any soil re-used or imported as fill for the completion of subgrade preparation should consist of predominantly "Very Low" to “Low” expansive soils exhibiting an EI not greater than 50, and should be exhibiting a relatively uniform gradation, free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials. For the excavated on-site soils to be blended such that the resultant EI is not exceeding 50, a general rule-of-thumb would be blending one (1) part of excavated site soils with four (4) parts of imported “Very Low” (EI ≤ 20) expansive soils.

Unless otherwise approved by the Geotechnical Consultant, the imported fill materials should also comply with the following soil corrosivity criteria for the desired concrete and reinforcement protection.

Corrosivity Criteria for Select Fill and General Fill			
Soluble Sulfate (% by weight) (1)	**Soluble Chloride (ppm) (2)**	**Resistivity Value (ohm-cm) (3)**	**pH-Value (4)**
≤ 0.1	≤ 500	≥ 2000	7.0 ~ 8.8

(1) California Test Method 417. (2) California Test Method 422. (3) ASTM G187-18 Test Method. (4) California Test Method 532.

All blended material and imported fill soils should be examined by the Geotechnical Consultant’s representative, and tested as necessary for evaluating their suitability for use as fill prior to being hauled to the Site. Final acceptance of any imported soil will be based upon review and testing of the soil actually delivered to the Site. All blended soils to be used as fill must be tested and approved by the Geotechnical Consultant prior to being used for fill placement.

6.2.8 Backfilling and Compaction Requirements:

Existing site soils at their present state and composition, unless treated with quicklime or cement as per Section 6.2.3 above, blended with "Very Low" expansive (EI ≤ 20) soils, crushed AC/PCC or CAB material to reduce the resultant EI to ≤ 50, or other measures approved by the Geotechnical Consultant, are only considered suitable for re-use as fill during site grading at depth greater than three (3) feet below subgrade within the footprint of the Building, non-structural or landscape areas, and backfilling of utility trenches, provided they 1) free of debris, particles greater than 4 inches in maximum dimension, organic matter or other deleterious materials, 2) are not environmentally contaminated, and 3) adequately moisture conditioned to permit achieving the required compaction. No nesting of large particles (2 to 4-inch size) should be permitted during backfilling operations.

On-site soils, blended soils and import materials approved for use as fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to a minimum of two (2) percentage points above optimum moisture content for "Low" expansive import or blended material, as well as for untreated site clayey soils, and to a minimum of one (1) percentage point above optimum moisture content for "Very Low" expansive import material, and compacted to a minimum 90 percent relative compaction. Unless otherwise stated, the measurement of relative compaction in this report should always refer to ASTM D1557-12 Test Method.

6.2.9 Tests and Observations:

All subgrade preparation, compaction, and backfill operations should be performed under the observation of and testing by the Geotechnical Consultant's field representative. An adequate number of field tests should be taken to ensure compliance with this report and local ordinances. If it is determined during grading that site soils require overexcavation to greater depths for obtaining proper support for the proposed structures, this additional work should be performed in accordance with the recommendations of the Geotechnical Consultant.

6.3 Foundation Design

It is ASE's opinion that, if replacing the subgrade soils with "Very Low" to "Low" expansive (EI ≤ 50) imported or blended fill materials is not preferred, then PT slab foundation bearing on recompacted fill soils may be used to provide foundation support for the Building, provided that the site grading recommendations presented in Section 6.2 above are incorporated in project planning and design, and implemented during site construction. Alternatively, conventional continuous spread footings and isolated pad footings with slab-on-grade bearing on lime treated soils or imported/blended approved fill with EI ≤ 50 may be considered for foundation support of the Building.

Presented below are the recommended geotechnical design and construction criteria for PT slab foundation and shallow footing foundation and slab-on-grade.

6.3.1 Post-Tensioned (PT) Slabs for Support of the Building:

If adopted, PT slabs should be designed based on the latest PTI design method (Reference 31) which is stipulated in References 29 and 30. Both of References 29 and 30 are part of the PT-slab design procedure per Section 1808.6.2 of the 2019 CBC. The PT slabs should be designed for at least one (1.0) inch of differential settlement over the largest dimension of the Building.

Based on review of laboratory data for the on-site materials, the average soil modulus of subgrade reaction, K-value, to be used for design is 80 pounds per cubic inch (pci). The PT slabs may also be designed based on a surface soils bearing value of 700 psf.

PT slabs should have sufficient stiffness to resist differential movement of the corner, edge or center of slab due to non-uniform swell and shrinkage of subgrade soils and fluctuation of subgrade soil moisture content. Based on the specifications of the PTI method, 3rd Edition, the table on the next page presents suggested minimum coefficients to be used for soils with different degrees of expansion potential. For the encountered site soils, a "**Very High**" soil expansion category could be considered for PT slab design.

PTI METHOD (3RD EDITION) DESIGN PARAMETERS					
Thornswaite Index	-20	**Soil Fabric Factor, Ft**	1.0	**Equilibrium Suction, pF**	3.909
Surface Equilibrium Wet Suction, $(pF)_{wet}$	11.782 [a]		**Surface Equilibrium Dry Suction, $(pF)_{dry}$**		17.593 [a]
Soil Expansion Classification	**Very Low (EI ≤ 20)**	**Low (20 < EI ≤ 50)**	**Medium (50 < EI ≤90)**	**High (90 < EI ≤ 130)**	**Very High (EI > 130)**
(e_m) edge lift (ft) [b]	5.20 [c]	4.64 [c]	4.13 [c]	3.78 [c]	3.0
(e_m) center lift (ft) [b]	9.00 [c]	9.00 [c]	8.55 [c]	7.34 [c]	5.3
(y_m) edge lift (in)	0.25	0.62	1.11	1.61	1.66
(y_m) center lift (in)	0.10	0.25	0.43	0.60	0.75

a. Values per PTI recommendations.

b. Soil parameters such as Atterberg Limits and % passing #200 have been derived from typical values available from Day (2005), Bowles (1996), and PTI (2004).

c. Governed by soil aspect consideration.

The above tabulated coefficients are considered minimums and may not be adequate to represent worst-case conditions such as adverse drainage and/or improper landscaping and maintenance. The above parameters are applicable provided structures have gutters and downspouts and positive drainage is maintained away from structures.

Deepened footings/edges around the slab perimeter must be used to minimize non-uniform surface moisture migration (from an outside source) beneath the slab. An edge depth of at least 24 inches should be considered for soils with “Very High” EI’s and 18 inches for soils with “Medium” to “High” EI’s. The bottom of the deepened footing/edge should be designed to resist tension, using cable or reinforcement per the Structural Engineer.

The entirety of the Building interior slab should be underlain by a minimum 15-mil polyvinyl chloride membrane vapor barrier with a minimum overlap of 12 inches in all directions. The interior slab should be placed in direct contact with the vapor retarder, underlain by four (4) inches of 1/2” crushed rock as a capillary break as per Section 4.505.2.1 of the current CALGreen Code. The concrete slab shall consist of a concrete mix design which will address bleeding, shrinking and curling.

PT slabs often develop a “dishing” or “arching” characteristic due to the fluctuation of soil moisture content underlying the perimeter and center slab. All areas to receive concrete should be presaturated to a depth of 18 inches such that the soil within this zone is at optimum moisture or higher. The Geotechnical Engineer should verify the subgrade was presaturated within 24 hours of placing the moisture barrier.

6.3.2 Conventional Shallow Footing Foundation:

Presented below are the recommended geotechnical design and construction criteria for shallow footing foundation, provided grading and subgrade treatment recommendations presented in Section 6.2 above or incorporated in project planning and design, and implemented during site construction.

a) Minimum Footing Dimension and Reinforcement:

In order to mobilize sufficient soils bearing capacity supporting the new footings, it is recommended that the following tabulated minimum footing embedments, widths and reinforcements for various footing types be considered.

Minimum Footing Dimension & Reinforcement					
Continuous Spread Footing/Strip Footing			**Isolated Pad Footing**		
Depth (1) (in)	**Width (in)**	**Reinforcement**	**Depth (1) (in)**	**Width (in)**	**Reinforcement**
30	15	Four #4 bars – two near the top and two near the bottom	30	24 square	Four #4 bars – two near the top and two near the bottom, applied bi-axially

(1) Footing embedment measured from the nearest adjacent lowest soils grade.

The final foundation design details such as concrete strength, reinforcements, etc. should be established by the Structural Consultant.

b) Allowable Soils Bearing Capacity:

For footings complying with the minimum dimension requirements stipulated in Section 6.3.2a) above, the allowable soils bearing capacities, inclusive of both dead and live loads, should be as per tabulated below:

Allowable Soils Bearing Capacity (psf)		**Increase per 12-inch Increment in Footing Width (psf)**	**Increase per 12-inch Increment in Footing Depth (psf)**	**Maximum Composite Ceiling Value (psf)**
Continuous Spread Footing/Strip Footing	**Isolated Pad Footing**			
2,000	2,000	-	200	3,500

The above allowable bearing capacities may be increased by one-third (1/3) when subject to short-term, transient loading induced by wind or seismic activities.

c) Lateral Resistance:

Resistance to lateral loads can be assumed to be provided by passive lateral earth pressure and by friction acting on structural components in permanent contact with the subgrade soils.

For site preparation implemented as per recommended in the above Section 6.2.3, lateral resistance on the sides of foundations may be computed using a passive lateral earth pressure of 200 pcf EFP for footings embedded into approved compacted fill soils, subject to a maximum of 2,000 psf. An ultimate coefficient of friction on the order of 0.25 may also be used for structural dead load acting between the footing bottom and the supporting soils, regardless of the distance between new footing and existing building footing.

The passive lateral earth pressure above may be used in conjunction with the ultimate coefficient of friction in calculating composite lateral resistance, provided the passive lateral earth pressure value is reduced by one-third (1/3). The composite lateral resistance may be increased by one-third (1/3) under short term, transient wind or seismic loading.

d) Static Settlements/Heaves:

Total static settlements/heaves resulting from compression/expansion of subgrade soils for conventional footings designed and constructed in accordance with the above criteria, and supporting maximum provided dead plus live (D+L) column and wall loads mentioned in Section 1.1.2 above, are not anticipated to exceed one (1) inch, when founded on firm and unyielding subgrade materials prepared as per recommended in Section 6.2.3 above. A differential settlement/heave on the order of one-half (1/2) inch over a distance of 30 feet is anticipated between similarly loaded adjacent isolated pad footings, between isolated pad footings and continuous wall footings, and for continuous wall footings over a distance of approximately 30 feet. Please note that the Geotechnical Consultant should be contracted for further evaluation, as necessary, should final design structural loads exceed the maximum loads assumed in the above analyses by more than ten (10) percent.

6.3.3 Retaining Walls:

It is ASE's understanding that there is no retaining wall planned as part of the Building construction. If design or planning change requires the construction of retaining wall, ASE should be consulted for pertinent retaining wall design parameters and construction guidelines.

6.3.4 Footing/Foundation Observation:

All footing/foundation excavations should be observed by the Geotechnical Consultant's representative to verify minimum embedment depths and competency of bearing soils. Such observations should be made prior to placement of any reinforcing steel or concrete.

6.4 Slabs-On-Grade

Exterior concrete flatwork/hardscape should be supported on properly compacted soils as recommended in the preceding Section 6.2.6. The slab subgrade soils should also be proof-rolled just prior to construction to provide a firm, unyielding surface, especially if the subgrade has been disturbed or loosened by the passage of construction traffic. Final compaction and testing of slab subgrade should be performed just prior to placement of concrete.

Per Section 1808A.6.2 of the 2019 CBC, when the WRI/CRSI Design of Slab-on-Ground Foundations is adopted, the Effective Plasticity Index (EPI) tabulated below should be utilized for slab-on-grade design. The EPI defined as the weighted plasticity index (WPI) x Cs x Co. In addition to using site native soils, the preliminary EPI reflecting the alternative of using "Very Low" to "Low" expansive (EI ≤ 50) imported fill, blended soils, or lime-treated soils beneath slab-on-grade are also tabulated below. However, it is of essential importance that the EPI for final slab-on-grade design should be based on additional laboratory tests performed on soils samples retrieved from rough graded building pad.

Area	WPI	C_S (1)	C_0 (2)	EPI
Compacted native soils	40	1.0	1.0	40
With 3' of imported or blended "Very Low" to "Low" (EI ≤ 50) expansive fill soil from finish subgrade(3)	21	1.0	1.0	21
With 5' of lime-treated soils from finish subgrade (4)	16	1.0	1.0	16

(1) For essentially flat ground, C_s = 1.0.
(2) For over-consolidated foundation materials with unconfined compressive strength not exceeding 6000 psf, C_0 effect is deemed less significant.
(3) A PI of 10 is assumed for "Very Low" to "Low" expansive fill soil. Refer to Sections 6.2.3 and 6.2.6 for applicable "Low" expansive fill depths.
(4) A PI of 10 is assumed for lime-treated soils.

Structural Design of Concrete Slabs				
Area	**k-value (psi/in)**	**Allowable Bearing Capacity (psf)**	**Minimum Actual Slab Thickness (Inches)**	**Minimum Reinforcement**
Compacted native soils per Section 6.2.6; exterior slab-on-grade ONLY	80	600	5.0	No. 4 rebar spaced maximum 12 in. on centers each way
With 3' of imported or blended "Very Low" to "Low" (EI ≤ 50) expansive fill soil from finish subgrade	120	800	4.5	No. 3 rebar spaced max. 12 in. on centers each way
With 5' of lime-treated soils from finish subgrade	120	800	4.5	No. 3 rebar spaced max. 12 in. on centers each way

For structural design of concrete slabs, the moduli of subgrade, allowable bearing capacities may be used for slab various grading configurations as tabulated above. Slabs should be properly designed and reinforced for the construction and service loading conditions, as well as considering the soil expansion potential. To minimize slab distress due to soil expansion, geotechnically, it would be prudent to provide a minimum actual slab thickness with minimum reinforcements for various grading configurations as tabulated above. The structural details, such as slab thickness, concrete strength, amount and type of reinforcements, joint spacing, etc., should be established by the Structural Consultant in accordance with pertinent sections in 2019 CBC.

The entirety of any new slabs within the Building should be underlain by an impermeable vapor barrier as per stipulated in Section 6.3.1 above.

Exterior slabs should be properly jointed to limit the number of concrete shrinkage cracks. For long/thin sections, such as sidewalks, expansion or control joints should be provided at spacing intervals equal to the width of the section. Slabs between 5 and 10 feet in minimum dimension should have a control joint at centerline. Slabs greater than 10 feet in minimum dimension should have joints such that unjointed sections do not exceed 10 feet in maximum dimension. Where flatwork adjoins structures, it is recommended that a foam joint or similar expansion material be utilized. Joint depth and spacing should conform to the ACI recommendations. It is, however, cautioned that uneven heaving of exterior slabs may develop in the future when prolonged irrigation or seepage permeates the subgrade soil, especially in areas that expansive soil pockets exist due to inadequate control or inspection of earthwork construction.

6.5 Site Drainage

Per Section 1804A.4 of 2019 CBC, a minimum 5% descending gradient away from the Building for a minimum distance of 10 feet should be incorporated for earth grade placed adjacent to the foundation. This descending gradient may be reduced to 2% for any impervious areas, such as concrete paved walkways, within the 10-foot zone. For areas where the 10-foot drainage distance is not attainable, alternative measure such as concrete-lined swales having a minimum 2% gradient may be adopted to divert the water away from the Building, provided that a minimum 5% gradient is maintained in the distance between the building footprint and the diversion measure such as swales. For more specific site drainage guidelines, the Project Civil Consultant should refer to the pertinent sections in 2019 CBC.

Any planter areas to be placed adjacent to structure perimeters should be provided with impervious bottoms and a drainage pipe, or should be planted with drought-tolerant plants, to divert water away or minimize moisture infiltration from foundation and slab subgrade soils. Excessive moisture variations in site soils could result in significant volume changes and movement.

6.6 Soil Corrosivity Evaluation

Soils corrosivity tests were performed on representative samples of site soil. These tests are meant to determine the corrosive potential of on-site soils to proposed concrete foundations/flatwork and underground metal conduit. The soils corrosivity test results are presented in Appendix A.

6.6.1 Concrete Corrosion:

Disintegration of concrete may be attributed to the chemical reaction of soils sulfates and hydrated lime and calcium aluminate with the cement. The severity of the reaction resulting in expansion and disruption of the cement is primarily a function of the concentration of soluble sulfates and the water-cement ratio of the concrete.

A soluble sulfate content of 0.027% by weight has been recorded from testing per California Test Method (CTM) 417 conducted on on-site soils, as indicated in Appendix A. As per Table 19.3.1.1 of ACI 318-19, soils exhibiting soluble content less than 0.1% by weight are classified as having “S0” sulfate exposure category. As such, for structural features to be in direct contact with on-site soils, the special geotechnical requirements on the type of Portland cement or water cement ratio corresponding to the tested “S0” sulfate exposure category as per stipulated in Table 19.3.2.1 of ACI 318-19 should be considered.

6.6.2 Metal Corrosion:

In the evaluation of soil corrosivity to metal, the hydrogen ion concentrates (pH) and the electrical resistivity of the site and backfill soils are the principal variables in determining the service life of ferrous metal conduit. The pH of soil and water is a measure of acidity or alkalinity, while the resistivity is a measure of the soils resistance to the flow of electrical current.

Currently available design charts indicate that corrosion rates decrease with increasing resistivities and increasing alkalinities. It can also be noted that for alkaline soils, the corrosion rate is more influenced by resistivity than by pH.

The resistivity value of 695 ohm-cm per ASTM G187-18 Test Method coupled with a pH-value of 8.22 per CTM 643 classifies the on-site soils tested to be very corrosive to buried ferrous metals. Based on CTM 643, the year to perforation for 18-gauge steel in contact with soils of similar resistivity and pH-value is approximately 21 years for the very corrosive on-site soils. In lieu of additional testing, alternative piping materials, i.e. plastic piping, may be used instead of metal if longer service life is desired or required for utility pipes and fittings in direct contact with on-site soils. These resistivity values of on-site soils may also have implications to other building materials and depths of embedment for steel reinforcement, etc. It is recommended that a qualified corrosion consultant be engaged to review the building plans.

A soluble chloride content of 24 ppm was recorded in our laboratory tests per CTM 422. Per Caltrans guidelines and specifications (References 16 and 17 soils exhibiting soluble chloride contents exceeding 500 ppm are considered "corrosive". The soils are thus classified as "non-corrosive" per Caltrans criterion. In addition, special measure in terms of rebar protection against chloride corrosion under Exposure Class "C0" stipulated in Tables 19.3.1.1 and 19.3.2.1 of ACI 318-14 may be required as a result of the soluble chloride content tested. However, the compliance with the corrosivity criteria stipulated in Section 6.2.8 above will ensure that no other particular reinforcement protection measure will be needed for slab-on-grade in contact with import fill.

6.7 Utility Trenches

All trenches should be backfilled with approved fill material compacted to relative compaction of not less than 90 percent per ASTM D1557-12 Test Method. Care should be taken during backfilling to prevent utility line damage.

The on-site soils may be used for backfilling utility trenches from one (1) foot above the top of pipe to the surface, provided the material is free of organic matter and deleterious substances. Any soft and/or loose materials or fill encountered at pipe invert should be removed and replaced with properly compacted fill or adequate bedding material.

The on-site soils are not considered suitable for bedding or shading of utilities. Imported soils for pipe bedding should consist of non-expansive granular soils having a tested Sand Equivalent (SE) (per ASTM D 2419-14 Test Method testing) value not less than 30.

If sandy soils are used for trench backfill, the backfill should be topped with a minimum 2-foot thick cap of compacted fine-grained, cohesive soil. Also, a minimum 10-foot length of trench at the entrance and exit points of buildings should be backfilled with fine-grained soils to serve as a plug to prevent water migration into structure foundation support zones.

The walls of temporary construction trenches may not be stable when excavated nearly vertical due to the potential for caving. Shoring of excavation walls or flattening of slopes will be required if excavation depths greater than 4 feet are necessary. Trenches should be located so as not to impair the bearing capacity of soils or cause settlement under foundations. As a guide, trenches parallel to foundations should be clear of a 45-degree plane extending outward and downward from the edge of the foundations. All work associated with trenches, excavations and shoring must conform to the State of California Safety Code (CAL-OSHA).

6.8 Plan Review, Observations and Testing

Once foundation and grading plans are completed, they should be forwarded to the Geotechnical Consultant for review of conformance with the intent of these recommendations and criteria presented in the pertinent sections of this report.

All excavations should be observed by a representative of this office to verify minimum embedment depths, competency of bearing soils and that the excavations are free of loose and disturbed materials. Such observations should be made prior to placement of any fill, reinforcing steel or concrete. All grading and fill compaction should be performed under the observation of and testing by a Geotechnical Consultant or his representative.

7.0 CLOSURE

This report has been prepared for the exclusive use of **Lawndale Elementary School District** (the Client) and their subconsultants for use in design and construction of the Building. The report has not been prepared for use by other parties, and may not contain sufficient information for purposes of other parties.

The Client is responsible for ensuring the information and recommendations contained in this report are brought to the attention of the Owner or the other design consultants, incorporated into the project plans, and implemented by project contractors. This report should be named on project plans as a part of the project specifications.

We request and recommend notification should any of the following occur:

1. Final plans for site development indicate utilization of areas not originally proposed for construction.
2. Structural loading conditions vary from those utilized for evaluation and preparation of this report.
3. The site is not developed within 12 months following the date of this report.

If changes or delays do occur, this office should be notified and provided with finalized plans of site development for our review to enable us to provide the necessary recommendations for additional work and/or updating of the report. Any charges for such review and necessary recommendations would be at the prevailing rate at the time of performing review work.

The findings contained in this report are based upon our evaluation and interpretation of the information obtained from the limited number of test borings and the results of laboratory testing and engineering analysis. As part of the engineering analysis it has been assumed, and is expected, that the geotechnical conditions existing across the area of study are similar to those encountered in the test excavations. However, no warranty is expressed or implied as to the conditions at locations or depths other than those excavated. Should conditions encountered during construction differ significantly from those described in this report, this office should be contacted immediately for recommendations prior to continuation of work.

Our findings and recommendations were obtained in accordance with generally accepted current professional principles and local practice in geotechnical engineering and reflect our best professional judgment. We make no other warranty, either express or implied.

These recommendations are, however, dependent on the aforementioned assumption of uniformity and upon proper quality control of engineered fill and foundations. Geotechnical observations and testing should be provided on a continuous basis during grading at the site to confirm preliminary design assumptions and to verify conformance with the intent of our recommendations. If parties other than Associated Soils Engineering, Inc. are engaged to provide geotechnical services during construction, they must be informed that they will be required to assume complete responsibility for the geotechnical phase of the project by either concurring with the recommendations in this report or providing alternative recommendations.

This concludes our scope of services as indicated in our proposal dated August 4, 2022, however, our report is subject to review by the controlling authorities for the project. Any further geotechnical services that may be required of our office to respond to questions/comments of the controlling authorities after their review of the report will be performed on a time-and-expense basis as per our current fee schedule. We would not proceed with any response to report review comments/questions without authorization from your office.

We appreciate your business and are prepared to assist you with construction-related services.

APPENDIX A

The following Appendices contain the substantiating data and laboratory test results to complement the engineering evaluations and recommendations contained in the report.

Site Exploration

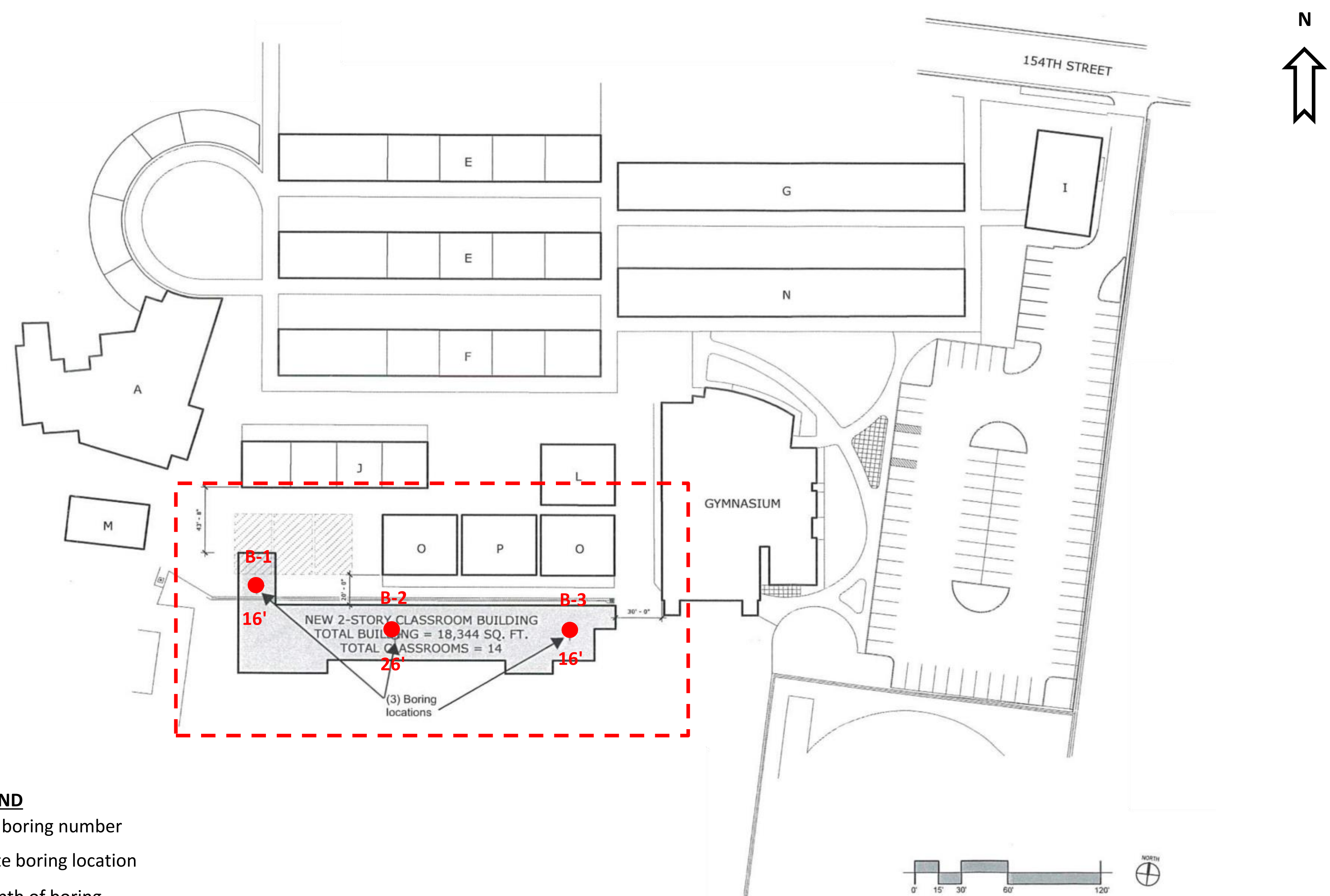
On October 14, 2022, field explorations were performed by drilling three (3) test borings at the approximate locations indicated on the attached Boring Location Plan, Plate A. The exploratory borings were drilled by Choice Drilling, Incorporated, utilizing a truck mounted, CME75 rotary drilling rig equipped with 8-inch diameter continuous flight, hollow-stem rotary augers. The borings extended to depths of 16 feet to 26 feet from the existing grades.

Continuous observations of the materials encountered in the borings were recorded in the field. The soils were classified in the field by visual and textural examination and these classifications were supplemented by obtaining bulk soil samples for future examination in the laboratory. Relatively undisturbed samples of soils were extracted in a Modified California barrel sampler lined with 2.416-inch diameter by one-inch high rings and tipped with tapered cutting shoe. All samples were secured in moisture-resistant bags immediately after retrieval from exploratory boring to minimize the loss of field moisture, followed by timely transportation to ASE's laboratory for ensuing testing. Upon completion of exploration, the borings were backfilled with excavated materials and compacted by tamping, with existing AC pavement patched with cold-patch asphalt.

Description of the soils encountered, depth of samples, field density and moisture content of tested samples, respective laboratory tests performed, as well as Modified California barrel sampler blow counts are presented in the attached Field Logs of Borings ("B" Plates).

Please note that the subsurface soils descriptions presented in the Field Logs of Borings have been interpreted from conditions exposed during the field investigation and/or information inferred from the reviewed geologic literature. As such, it is likely that not all of the subsurface conditions at the Site could be captured or represented. It is therefore essential that the Geotechnical Consultant's engineer or geologist should be on site during excavation/grading and pipeline construction such that information/recommendations deciphered during preliminary geotechnical investigation phase could be verified and amended, as appropriate

Plate A	Boring Location Plan
Plates B-1 through B-3	Field Logs of Borings



LEGEND

B-1 Designated boring number

Approximate boring location

16' Terminal depth of boring

Approximate site limit

NOT TO SCALE

Associated Soils Engineering, Inc.

2860 Walnut Avenue

Signal Hill, CA 90755

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Boring Location Plan

Project Name:	**New Classroom Building at Will Rogers MS 4110 West 154th Street, Lawndale, CA**
Plate A	
Proj. No.:	**7083.22**
Date:	**November, 2022**

FIELD LOG OF BORING B-1

Sheet 1 of 1

ASSOCIATED SOILS ENGINEERING, INC.

Project:	***Proposed Classroom Bldg.,Will Rogers Middle School-Lawndale***
Location:	***4110 West 154th Street***
Project No.	***7083.22***

Dates(s) Drilled:	**10/14/22**	Logged By:	**John Whitney**
Drilled By:	**Choice Drilling, Inc.**	Total Depth:	**16 feet**
Rig Make/Model:	**CME75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 inches**	Surface Elevation:	**N/A**

Comments: No groundwater encountered.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0			ASPHALTIC CONCRETE PAVEMENT: 3.0"				
			CL	AGGREGATE BASE MATERIAL: 4.0"				MAX DENSITY, REMOLD SHEAR, EXPANSION, ATTERBERG,
		13(Ring)	CH	CLAY AND SAND: FILL:Dark brown to light brown, soft to firm, moist to very moist, fine-grained sand	98.1	24.6		
			CH	CLAY: Native:Very dark gray to black, stiff, moist, plastic				
5	5	9/6"(Ring)		CLAY: Very dark grayish brown, stiff, moist	96.2	26.0		CONSOL, SHEAR
		16/6" (Ring)	CH	CLAY: Olive gray and white, moderately stiff to stiff, moist	105.6	21.1		
			ML	CLAYEY SILT: Olive brown to olive gray, stiff, slightly moist				
10	10	24/6" (Ring)			112.9	14.0		
		32/6" (Ring)	ML	CLAYEY SILT: Olive brown to olive gray, stiff, slightly moist, less clay	122.2	13.4		
			CH	CLAY WITH SILT: Olive brown, stiff, moist				
15	15	22/6" (Ring)			102.9	22.9		
		24/6" (Ring)	ML	CLAYEY SILT: Light olive brown,stiff, moist	108.7	17.0		

SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-2

Project: ***Prop. Classroom Bldg.,Will Rogers Middle School***

Location: ***4110 W. 154th St.-Lawndale*** Project No. ***7083.22***

Dates(s) Drilled:	**10/14/22**	Logged By:	**John Whitney**
Drilled By:	**Choice Drilling, Inc.**	Total Depth:	**26 feet**
Rig Make/Model:	**CME75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 inches**	Surface Elevation:	**N/A**

Comments: Groundwater at 24 feet.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0			ASPHALTIC CONCRETE PAVEMENT: 5.0"				
			CH	AGGREGATE BASE MATERIAL: 3.0"				
		9/6"(Ring)		CLAY: Very dark gray to very dark brown to black, stiff, plastic, moist	99.6	25.8		CONSOL
		15/6" (Ring)	CH		106.3	17.1		
				CLAY: Very dark grayish brown to dark yellowish brown, stiff, moist				
5	5	11/6" (Ring)		same as above, very dark grayish brown to black	100.8	23.4		
		19/6" (Ring)	CH	CLAY: Dark olive gray to olive, stiff, moist, kaolinite	104.3	22.0		
10	10	77(Ring)	CH-CL	CLAY WITH SILT: Light olive brown to light gray, stiff, moist	120.9	13.8		
15	15	39(Ring)	CL	SILTY CLAY: Light olive brown, stiff, moist	101.1	23.3		

SOILS ENGINEERING, INC.

FIELD LOG OF BORING B - 2

Project:	Prop. Classroom Bldg.,Will Rogers Middle School	
Location:	4110 W. 154th St.-Lawndale	Project No. 7083.22

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: BULK	DRIVE	TYPE, "N" or (Blows/ft.)	LITHOLOGY	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
						CH	CLAY: Light olive brown, moderately stiff, moist, trace silt				
20	20			33(Ring)				92.3	32.0		
25	25			36(Ring)			same as above, stiff, very moist	96.9	27.8		

FIELD LOG OF BORING B-3

Sheet 1 of 1

ASSOCIATED SOILS ENGINEERING, INC.

Project:	***Prop. Classroom Bldg.,Will Rogers Middle School***
Location:	***4110 W. 154th St.-Lawndale***
Project No.	***7083.22***

Dates(s) Drilled:	**10/14/22**	Logged By:	**John Whitney**
Drilled By:	**Choice Drilling, Inc.**	Total Depth:	**16 feet**
Rig Make/Model:	**CME75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-stem Auger**	Hammer Weight/Drop:	**140 Lb./±30 In.**
Hole Diameter:	**8 inches**	Surface Elevation:	**N/A**

Comments: No groundwater encountered.

DEPTH (Ft.)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE, "N" or (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (Pcf)	MOISTURE CONTENT (%)	WELL COMPLETION	OTHER TESTS
0	0			ASPHALTIC CONCRETE PAVEMENT: 6.0"				
				AGGREGATE BASE MATERIAL: 8.0"				
			CH	CLAY: FILL:Brown, olive, and dark brown, soft to firm, damp to moist				
		8/6"(Ring)	CH	CLAY: Native: Very dark brownish gray to black, stiff, moist	113.3	15.8		SHEAR
		10/6" (Ring)	CH		96.5	26.8		
				CLAY: Very dark gray to black, stiff, moist				
5	5	9/6"(Ring)		same as above,very dark grayish brown	100.9	25.1		CONSOL
		11/6" (Ring)	CH	CLAY: Olive brown, stiff, moist	113.1	8.3		
10	10	50(Ring)	ML/CL	CLAYEY SILT AND SILTY CLAY: Olive, moderately stiff to stiff, moist	113.1	16.7		
			CL	SILTY CLAY: Olive, stiff, moist				
15	15	17/6" (Ring)			106.3	17.1		
		26/6" (Ring)		CLAY WITH SILT: Olive, stiff, moist	97.8	24.6		

Laboratory Tests

After samples were visually classified in the laboratory, a testing program aimed at generating sufficient data for subsequent evaluation was established and implemented.

- Moisture Content and Density Tests

The undisturbed soil retained within the rings of the Modified California barrel sampler was tested in the laboratory to determine in-place dry density and moisture content. Test results are presented on the Field Logs of Borings ("B" Plates).

- Consolidation and Direct Shear Tests

Consolidation (ASTM D 2435-11 Test Method) and direct shear (ASTM D 3080-11 Test Method) tests were performed on selected relatively undisturbed and remolded samples to determine the settlement characteristics and shear strength parameters of various soil samples, respectively. The results of these tests are shown graphically on the appended "C" and "D" Plates.

- Atterberg Limits Tests

The Atterberg Limits (liquid limit-plastic limit and plasticity index) were determined on selected soils samples in accordance with ASTM D4318-10 Test Method, Method A (multi-point test), dry preparation procedures. The test results are as follows and are presented on the appended Plate E-1.

Sample ID	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plasticity Index, PI	Soil Classification
B-1 @ 1'-5'	54	15	39	CH

- Soil Corrosivity Tests

Tests of soluble sulfate and chloride contents were performed in accordance with CTM's 417 and 422, respectively, to assess the degree of corrosivity of the subgrade soils with regard to concrete and normal grade steel. Resistivity and ph-value tests were performed in accordance with ASTM G187-18 Test Method and CTM 643 to assess the degree of corrosivity of the subgrade soils with regard to ferrous metal piping. The test results are shown below.

Sample ID	Sulfate Content (1) (%)/ Exposure Category	Chloride Content (2) (ppm) / Exposure Category	Resistivity (3) (OHM-cm)/ Exposure Aggressiveness	Ph-Value(4)
B-1 @ 1'-5'	0.027/ S0	24 / C0	695 / Very Corrosive	8.22

(1) California Test Method 417. (2) California Test Method 422. (3) ASTM G187-18 Test Method. (4) California Test Method 643.

Laboratory Tests – continued

- Maximum Dry Density/Optimum Moisture Content Test

A maximum density test was conducted in accordance with ASTM D1557-12, Method A, using 5 equal layers, 25 blows each layer, 10-pound hammer, 18 inch drop in a 1/30 cubic foot mold. The results are as follows:

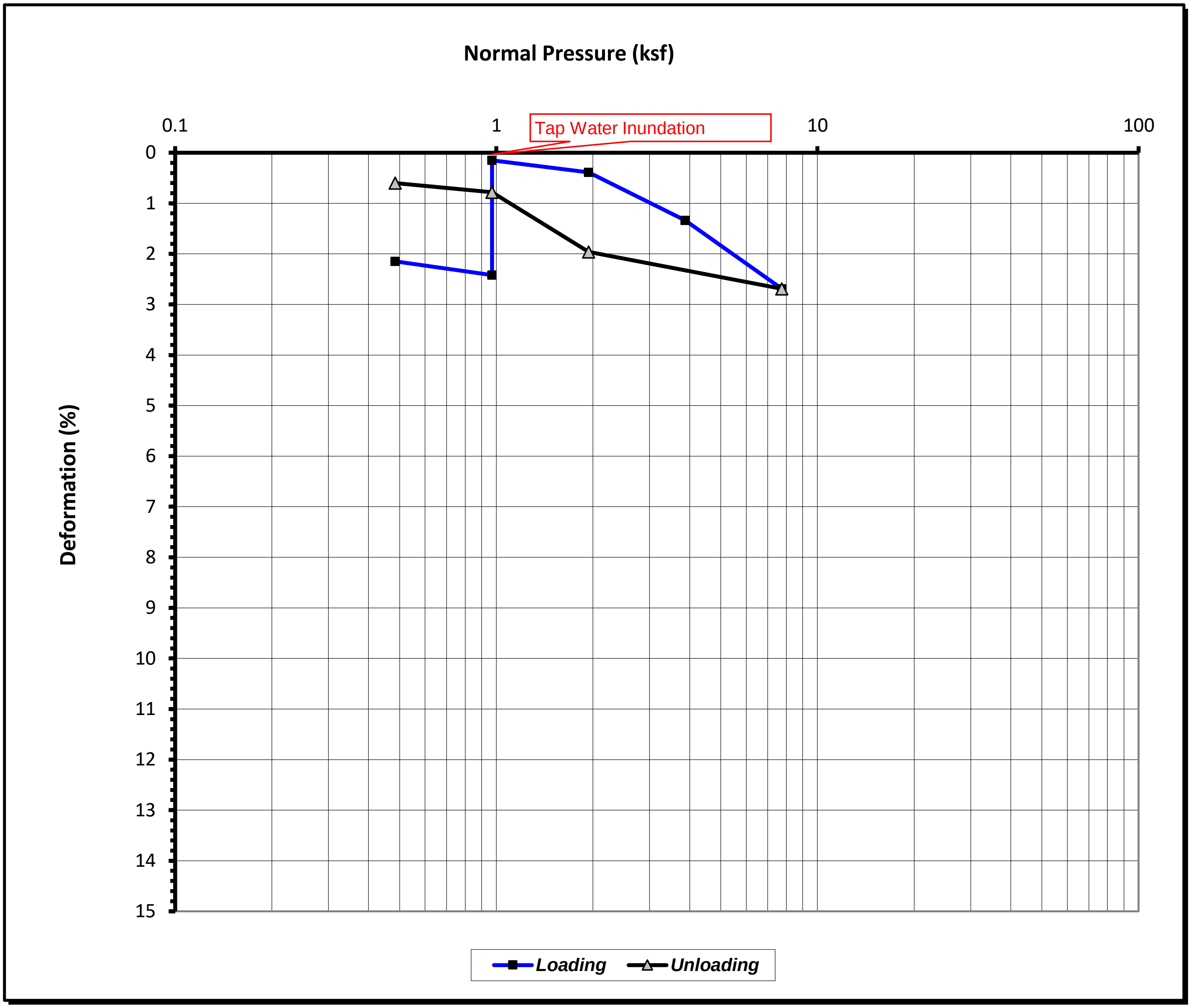
Sample ID	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Material Classification
B-1 @ 1'-5'	122.0	13.5	CH

- Expansion Test

An expansion test was performed on a soil sample to determine the swell characteristics. The expansion test was conducted in accordance with ASTM D4829-18 test procedures. The expansion sample was remolded to approximately 90 percent relative compaction at near optimum moisture content subjected to 144 pounds per square foot surcharge load and were saturated.

Sample ID	Molded Dry Density (pcf)	Molded Moisture Content (%)	% Saturation	Expansion Index (EI)	Expansion Classification
B-1 @ 1'-5'	106.6	13.1	61.1	136	Very High

Plates C-1 through C-3	Uni-axial Consolidation Test Results
Plates D-1 through D-3	Direct Shear Test Results
Plate E-1	Atterberg Limits Test Results



Sample Location	B-1 @ 5'	**Dry Density (pcf)**	105.6
		Moisture (%)	21.1
Sample Type	Ring		
Sample Description	Olive with Very Dark Grayish Brown Calcareous Clay		
Remark	Undisturbed sample		

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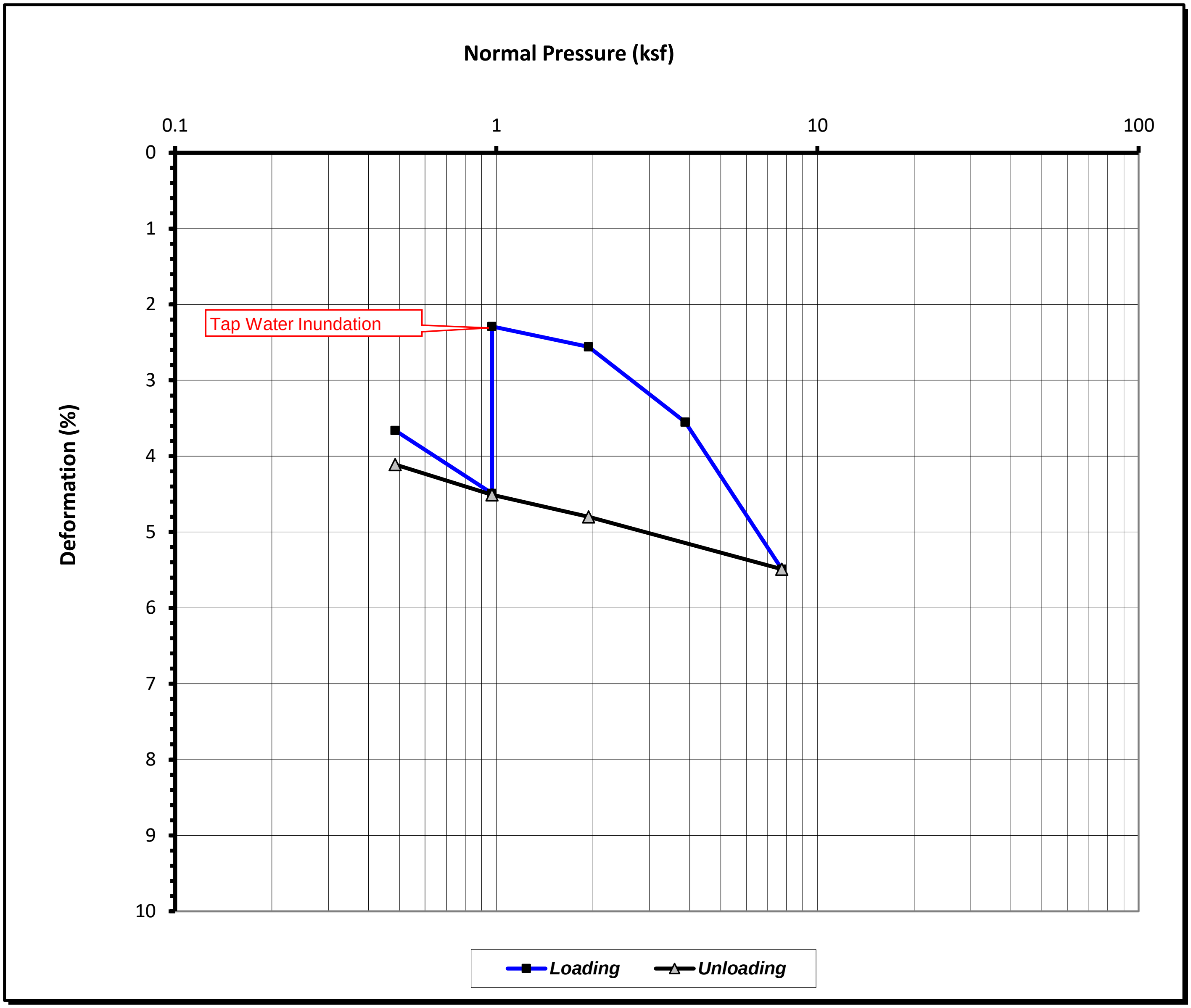
Project:	**Proposed New Classroom Building, Will Rogers Middle School, 4110 W. 154th St., Lawndale, CA**		
Plate C-1	**Result of Uniaxial Consolidation/Swelling Test of On-Site Soil (ASTM D2435-11 Test Method)**		
Project No.:	7083.22	**Date:**	November, 2022



Sample Location	B-2 @ 2'	**Dry Density (pcf)**	99.6
		Moisture (%)	25.8
Sample Type	Ring		
Sample Description	Black Clay		
Remark	Undisturbed sample		

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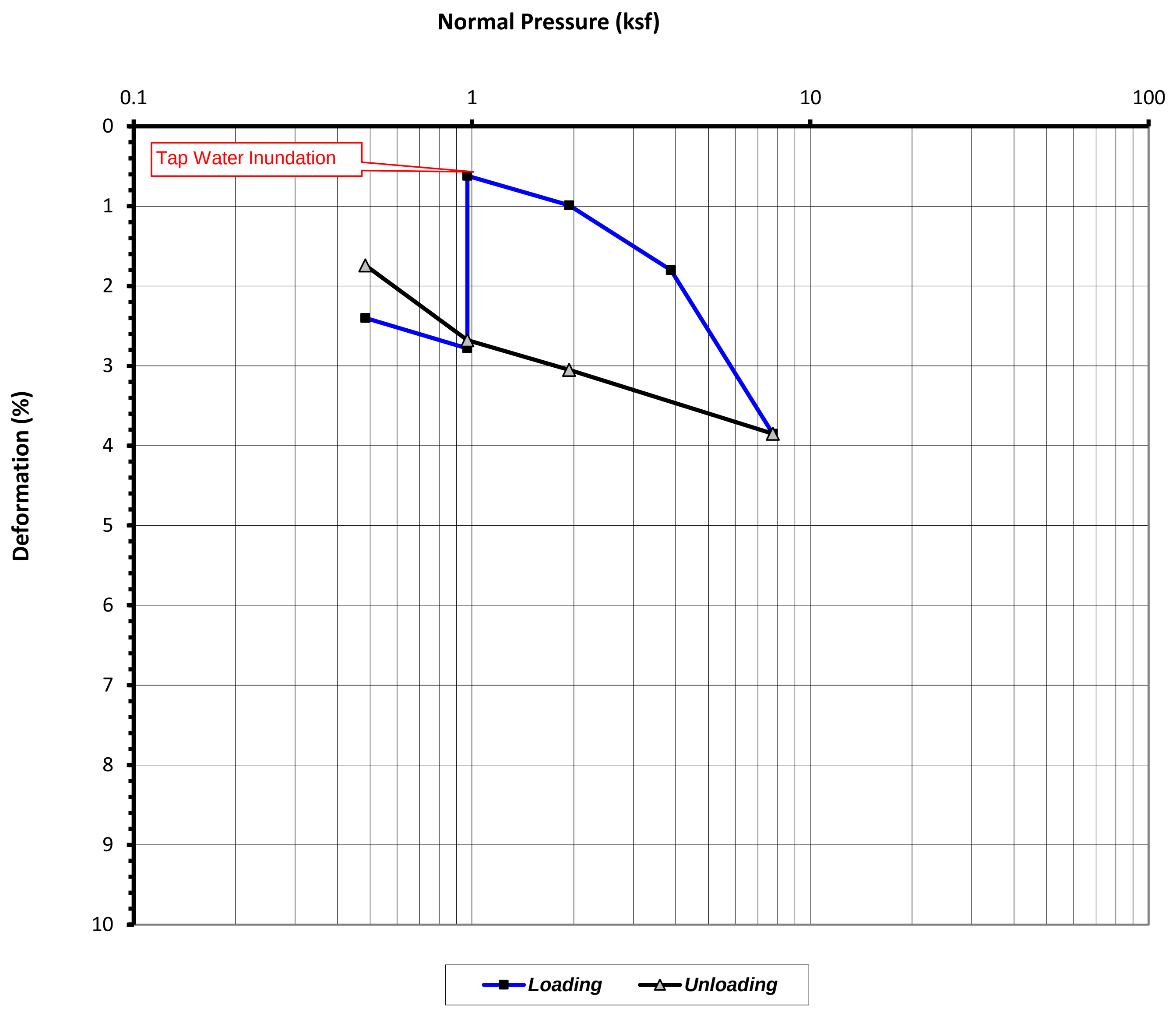
Project:	**Proposed New Classroom Building, Will Rogers Middle School, 4110 W. 154th St., Lawndale, CA**		
Plate C-2	**Result of Uniaxial Consolidation/Swelling Test of On-Site Soil (ASTM D2435-11 Test Method)**		
Project No.:	7083.22	**Date:**	November, 2022



Sample Location	B-3 @ 5'	**Dry Density (pcf)**	100.9
		Moisture (%)	25.1
Sample Type	Ring		
Sample Description	Very dark grayish brown to black and olive mottled Clay		
Remark	Undisturbed sample		

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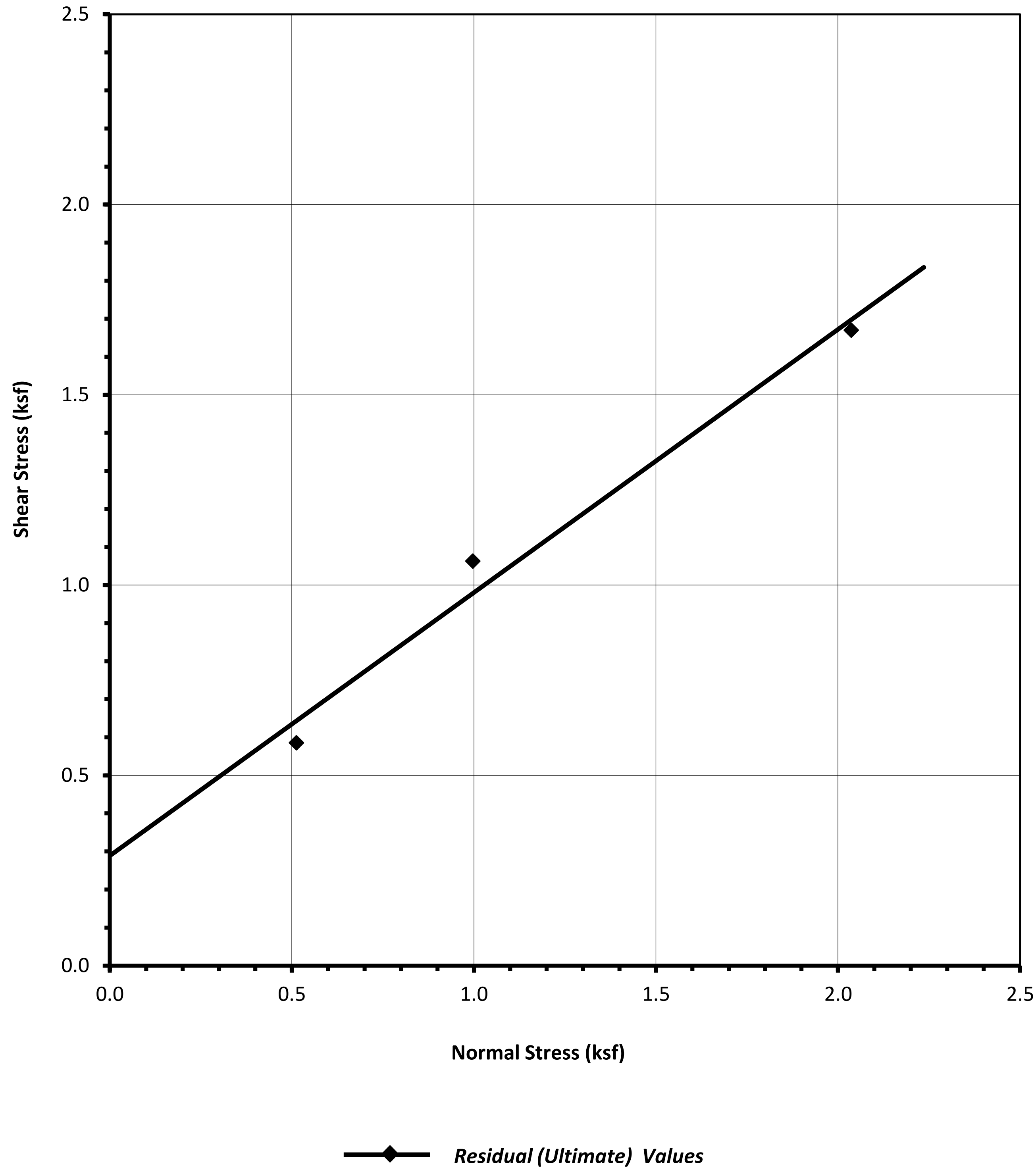
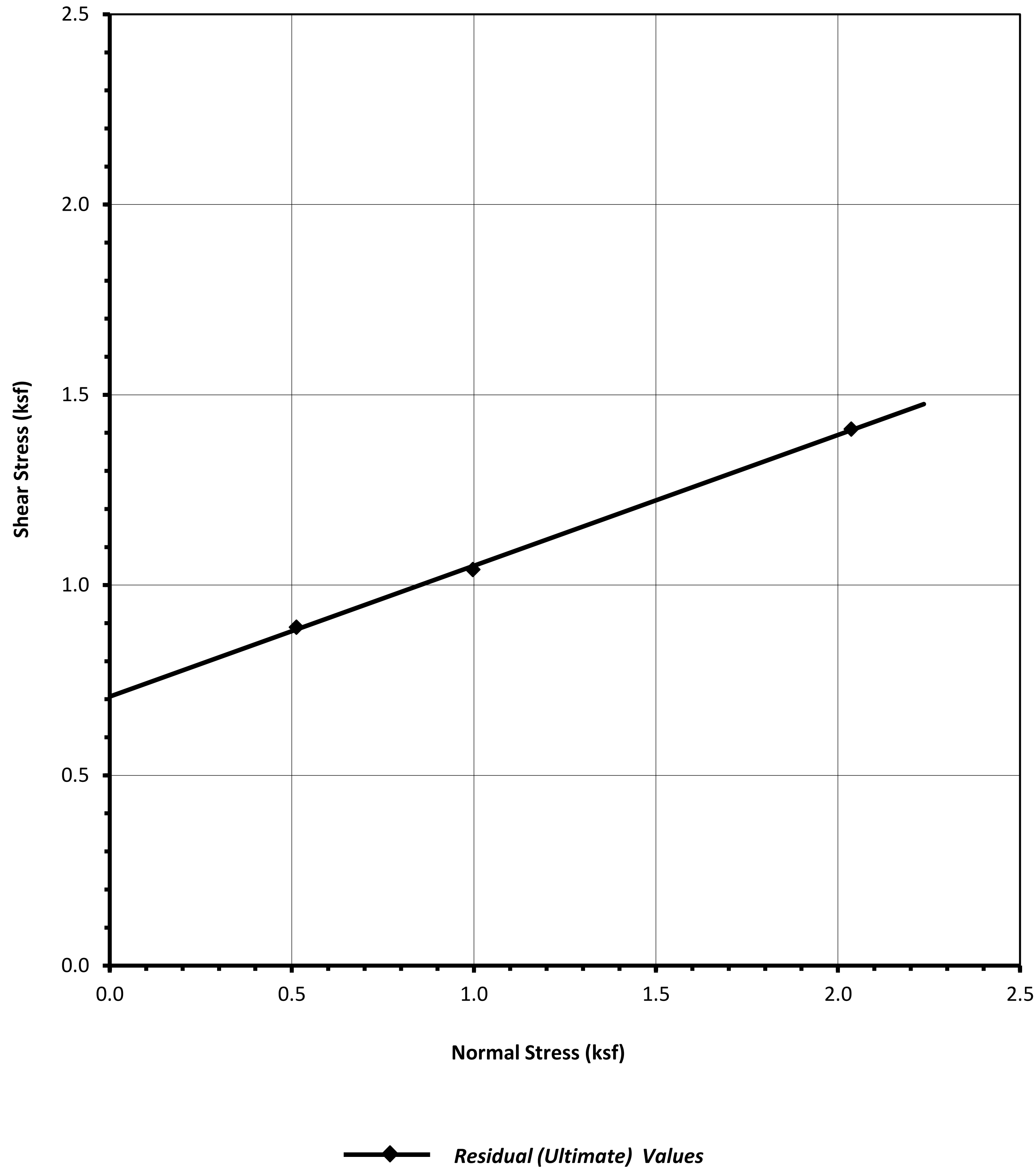
	Plate **C-3**	**Result of Uniaxial Consolidation/Swelling Test of On-Site Soil (ASTM D2435-11 Test Method)**		
	Project No.:	7083.22	**Date:**	November, 2022



Sample	*B-1 @ 5'*		**Dry Density (pcf)**	*96.2*
Description	*Mottled Olive with very dark grayish brown Clay, some calcareous inclusions*		**Moisture Content (%)**	*26.0*
ϕ-angle (degree)	*34.5*	**Ultimate**		
Cohesion (ksf)	*0.285*	**(Residual)**		

Associated Soils Engineering, Inc.	Project:	**New Classroom Building at Will Rogers MS 4110 West 154th Street, Lawndale, CA**
2860 Walnut Avenue Signal Hill, CA 90755	**Plate D-1**	**Direct Shear Test Result (ASTM D 3080-11 Test Method)**
Tel (562) 426-7990 Fax (562) 426-1842	Project No.: **7083.22**	Date: **November, 2022**



Sample	*B-3 @ 3'*		Dry Density (pcf)	*113.3*
Description	*Very dark grayish brown to black Clay*		Moisture Content (%)	*15.8*
ϕ-angle (degree)	*18.5*	Ultimate (Residual)		
Cohesion (ksf)	*0.705*			

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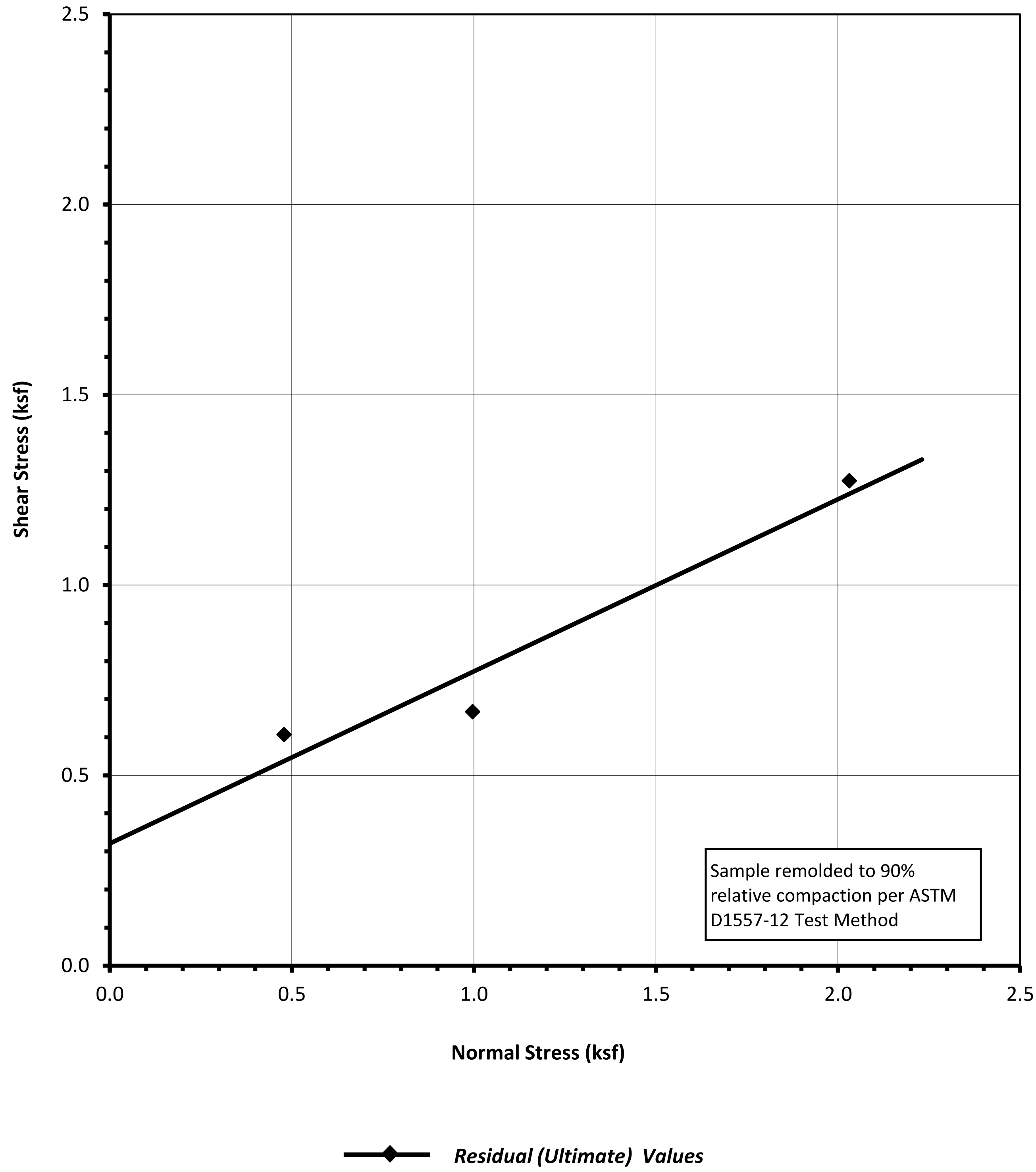
	Plate **D-2**	**Direct Shear Test Result** **(ASTM D 3080-11 Test Method)**		
	Project No.:	**7083.22**	Date:	**November, 2022**



Sample	*B-1@ 1'-5'*	Dry Density (pcf)	*109.8*
Description	*Black Clay with Sand*	Moisture Content (%)	*13.5*

ϕ-angle (degree)	*24.0*	Ultimate
Cohesion (ksf)	*0.320*	(Residual)

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Tel (562) 426-7990 Fax (562) 426-1842

Project:	**New Classroom Building at Will Rogers MS** **4110 West 154th Street, Lawndale, CA**		
Plate **D-3**	**Direct Shear Test Result** **(ASTM D 3080-11 Test Method)**		
Project No.:	**7083.22**	Date:	**November, 2022**

Associated Soils Engineering, Inc.

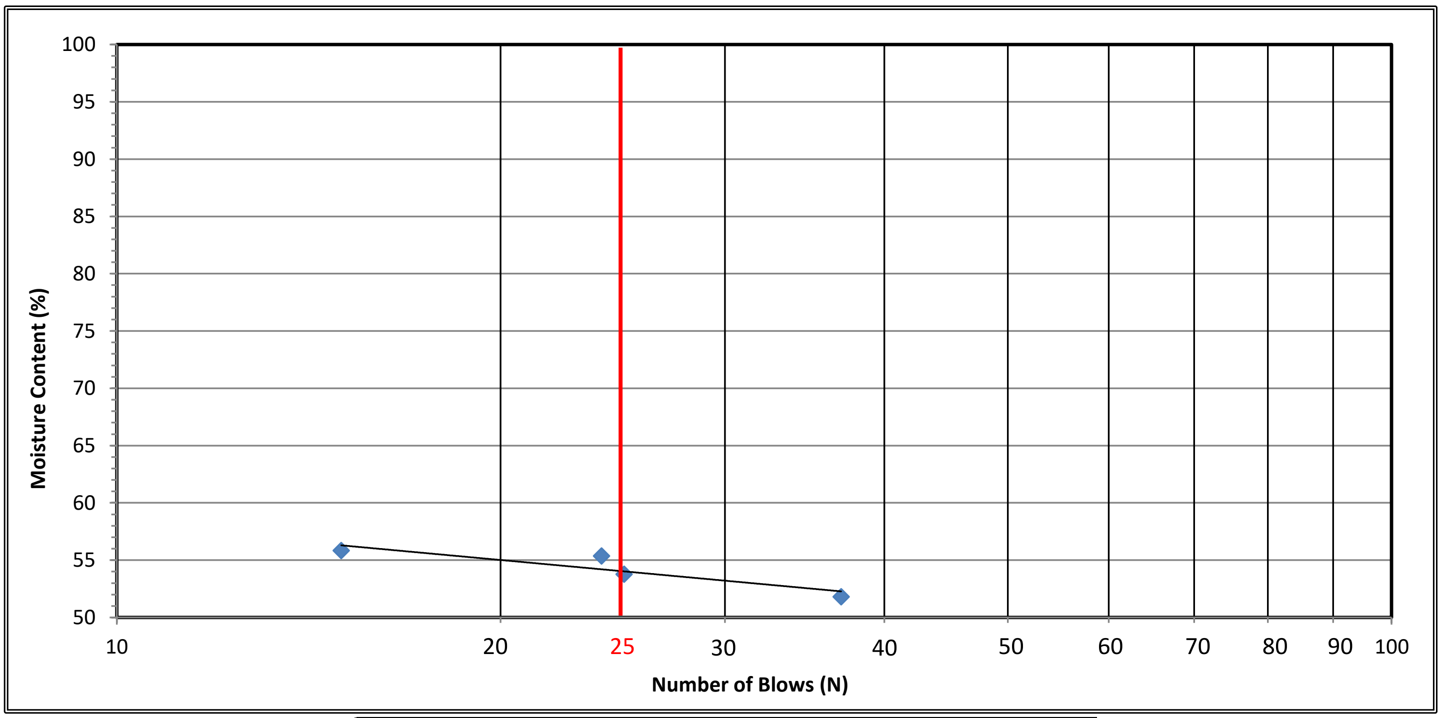
ATTERBERG LIMITS TEST

(ASTM D 4318-17e1 Test Method)

Project Name:	Proposed New Classroom Building at Will Rogers Middle School, 4110 West 154th Street, Lawndale, CA		
Project No.:	7083.22	**Sample ID:**	B-1 @ 2-5'
Visual Sample Description:	Very dark gray to black Clay		

Procedures Used			
	Wet Preparation - Multipoint - Wet	**X**	**Procedure A Multipoint Test**
X	**Dry Preparation - Multipoint - Dry**		**Procedure B One-Point Test**

	PLASTIC LIMIT		LIQUID LIMIT			
Test No.	1	2	1	2	3	4
Number of Blows (N)			37	25	15	24
Container No.	**b1**	**c1**	**A1**	**A2**	**C1**	**F**
Wet Wt. of Soil + Container (g)	14.26	14.40	15.19	17.32	15.36	11.89
Dry Wt. of Soil + Container (g)	13.01	13.12	13.76	15.18	13.64	10.55
Wt. of Container (g)	4.30	4.28	11.00	11.20	10.56	8.13
Moisture Content (%)	14.35	14.48	51.81	53.77	55.84	55.37



LL trendline = A · ln(x) + B	A =	-4.4500	x =	25
	B =	68.3450	LL =	***54***

Liquid Limit (LL) =	54
Plastic Limit (PL) =	14
Plasticity Index (PI) =	40
USCS Classification =	***CH***

"A" Line: PI = 0.73 · (LL-20)

"U" Line: Upper limit of plasticity per Casagrande,

PI = 0.9 · (LL-8)

For the tested LL:

PI at "A" Line =	24.8
PI at "U" Line =	41.4

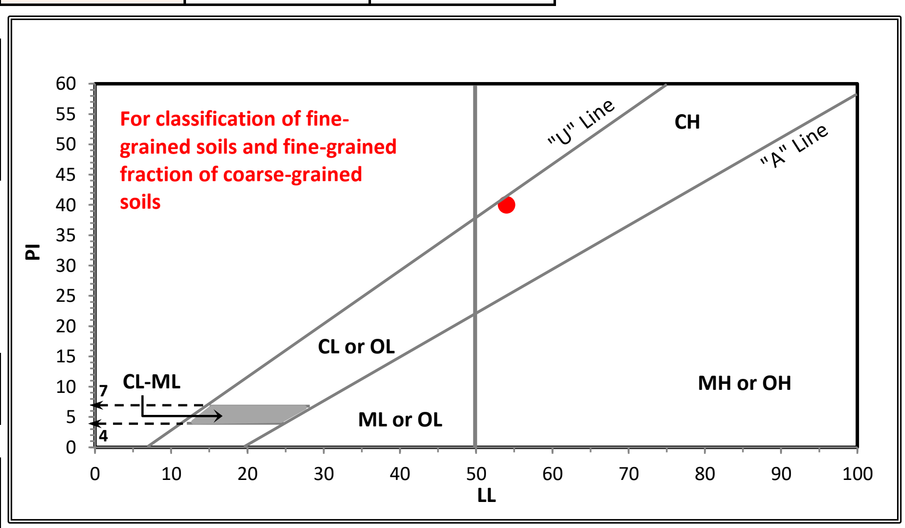


Plate No.:	**E-1**

APPENDIX B - SITE FAULTING/SEISMICITY DATA

Plates I-1 and I-2	EQFAULT – Deterministic Estimation of Peak Acceleration from Digitized Faults
Plate J-1	U.S. Geological Survey Quaternary Faults
Plate J-2 (2 Sheets)	Earthquakes with Strong Motion Records in CESMD

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***********************
*                     *
*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
***********************
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: 7083.22

DATE: 11-10-2022

JOB NAME: Proposed Classroom Building, Will Rogers Middle School
4110 West 154th Street, Lawndale, CA
CALCULATION NAME: east

FAULT-DATA-FILE NAME: C:\Program Files\EQFAULT1\Cgsflte.dat

SITE COORDINATES:
SITE LATITUDE: 33.8899
SITE LONGITUDE: 118.3454

SEARCH RADIUS: 62 mi

ATTENUATION RELATION: 20) Sadigh et al. (1997) Horiz. - Soil
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: clodis
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: Campbell SHR:
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: C:\Program Files\EQFAULT1\Cgsflte.dat

MINIMUM DEPTH VALUE (km): 0.0

EQFAULT SUMMARY

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
NEWPORT-INGLEWOOD (L.A.Basin)	3.8(6.1)	7.1	0.403	X
PALOS VERDES	4.9(7.9)	7.3	0.384	X
PUENTE HILLS BLIND THRUST	11.2(18.0)	7.1	0.298	IX
SANTA MONICA	12.4(19.9)	6.6	0.218	VIII
HOLLYWOOD	13.9(22.3)	6.4	0.172	VIII
UPPER ELYSIAN PARK BLIND THRUST	13.9(22.4)	6.4	0.171	VIII
MALIBU COAST	14.5(23.3)	6.7	0.199	VIII
RAYMOND	17.5(28.1)	6.5	0.148	VIII
VERDUGO	19.5(31.4)	6.9	0.167	VIII
WHITTIER	19.9(32.1)	6.8	0.120	VII
ANACAPA-DUME	21.1(33.9)	7.5	0.217	VIII
NORTHRIDGE (E. Oak Ridge)	23.1(37.2)	7.0	0.150	VIII
SIERRA MADRE	23.9(38.5)	7.2	0.163	VIII
SIERRA MADRE (San Fernando)	26.6(42.8)	6.7	0.105	VII
SAN JOAQUIN HILLS	27.2(43.8)	6.6	0.096	VII
CLAMSHELL-SAWPIT	28.1(45.2)	6.5	0.086	VII
SAN JOSE	28.6(46.1)	6.4	0.077	VII
SAN GABRIEL	29.8(48.0)	7.2	0.101	VII
SANTA SUSANA	30.5(49.1)	6.7	0.090	VII
NEWPORT-INGLEWOOD (Offshore)	32.2(51.8)	7.1	0.086	VII
CHINO-CENTRAL AVE. (Elsinore)	33.1(53.2)	6.7	0.081	VII
SIMI-SANTA ROSA	34.9(56.1)	7.0	0.094	VII
HOLSER	36.4(58.5)	6.5	0.062	VI
CUCAMONGA	38.9(62.6)	6.9	0.077	VII
OAK RIDGE (Onshore)	39.2(63.1)	7.0	0.082	VII
ELSINORE (GLEN IVY)	40.7(65.5)	6.8	0.052	VI
SAN CAYETANO	44.7(71.9)	7.0	0.069	VI
SAN ANDREAS - 1857 Rupture M-2a	46.9(75.5)	7.8	0.090	VII
SAN ANDREAS - Whole M-1a	46.9(75.5)	8.0	0.103	VII
SAN ANDREAS - Mojave M-1c-3	46.9(75.5)	7.4	0.068	VI
SAN ANDREAS - Cho-Moj M-1b-1	46.9(75.5)	7.8	0.090	VII
CORONADO BANK	49.2(79.1)	7.6	0.074	VII
OAK RIDGE(Blind Thrust Offshore)	51.9(83.6)	7.1	0.062	VI
CHANNEL IS. THRUST (Eastern)	53.3(85.7)	7.5	0.081	VII
SAN JACINTO-SAN BERNARDINO	53.9(86.8)	6.7	0.033	V
VENTURA - PITAS POINT	55.4(89.2)	6.9	0.048	VI
SAN ANDREAS - SB-Coach. M-1b-2	55.9(89.9)	7.7	0.068	VI
SAN ANDREAS - SB-Coach. M-2b	55.9(89.9)	7.7	0.068	VI
SAN ANDREAS - San Bernardino M-1	55.9(89.9)	7.5	0.059	VI
SAN ANDREAS - Carrizo M-1c-2	56.7(91.2)	7.4	0.054	VI
SANTA YNEZ (East)	57.1(91.9)	7.1	0.042	VI
CLEGHORN	58.3(93.9)	6.5	0.025	V
OAK RIDGE MID-CHANNEL STRUCTURE	59.0(94.9)	6.6	0.035	V
ELSINORE (TEMECULA)	59.7(96.1)	6.8	0.031	V
M.RIDGE-ARROYO PARIDA-SANTA ANA	61.8(99.5)	7.2	0.053	VI

****

-END OF SEARCH- 45 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS. THE NEWPORT-INGLEWOOD (L.A.Basin)FAULT IS CLOSEST TO THE SITE, ABOUT 3.8 MILES (6.1 km) AWAY.
LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4032 g

ASE#7083.22

PLATE J-1

U.S. Geological Survey Quaternary Faults

11/17/2022, 5:48:41 PM

Fault Areas

- Class B
- historic
- late Quaternary
- latest Quaternary
- middle and late Quaternary

National Database

- Historic (< 150 years), well constrained location
- Historic (< 150 years), moderately constrained location

1:288,895

0 2 4 8 mi

0 3.25 6.5 13 km

National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp.

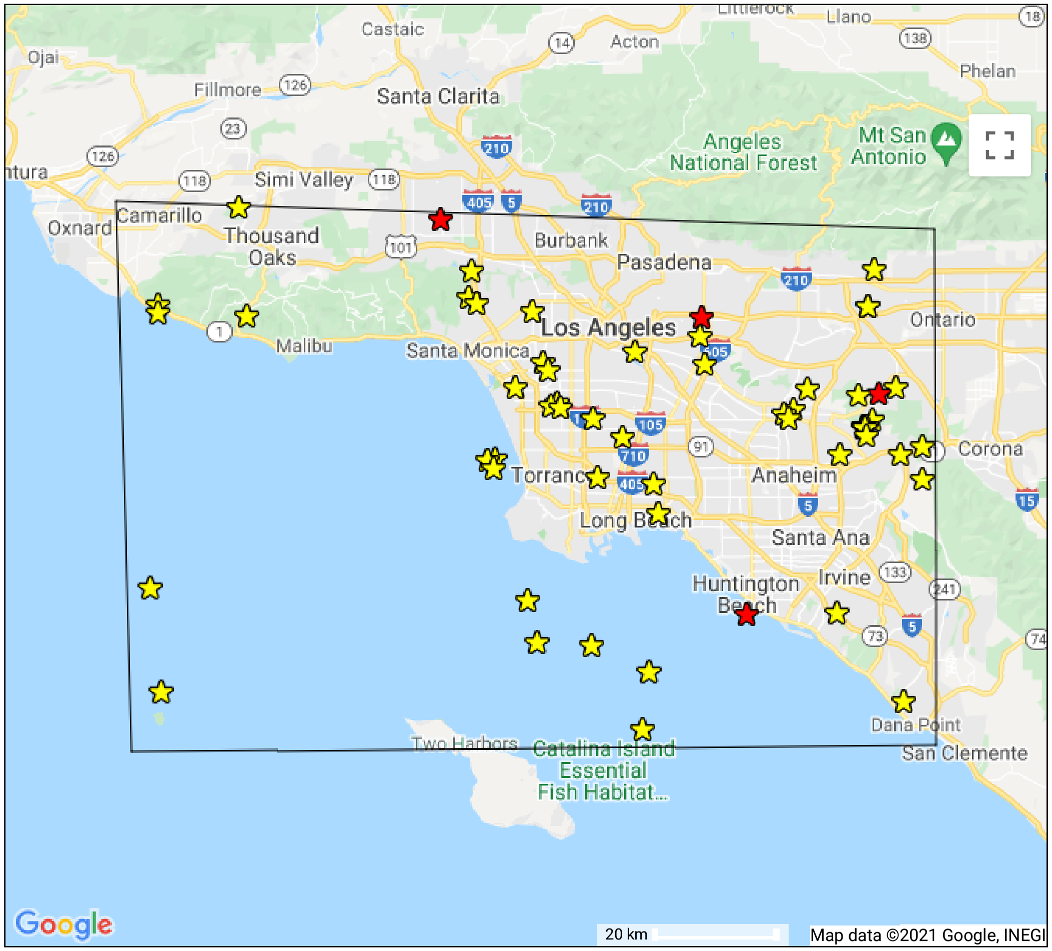
USGS

National Geographic, Esri, Garmin, HERE, UNEP-WCMC, USGS, NASA, ESA, METI, NRCAN, GEBCO, NOAA, increment P Corp. | USGS |

Earthquakes with Strong Motion Records in CESMD

Use polygon to display and list earthquakes within area of interest:

1. Start Polygon | 2. Close Polygon | 3. Show Result | Start Over



Help | View US Earthquakes

Significant Strong-Motion Earthquakes
Other

Hover over star to reveal earthquake name and information.

Note: Event Epicenters are approximate when viewed at high zoom levels.

Click earthquake for more information

CESMD

Click on an earthquake below to see details on map

Type to search events...

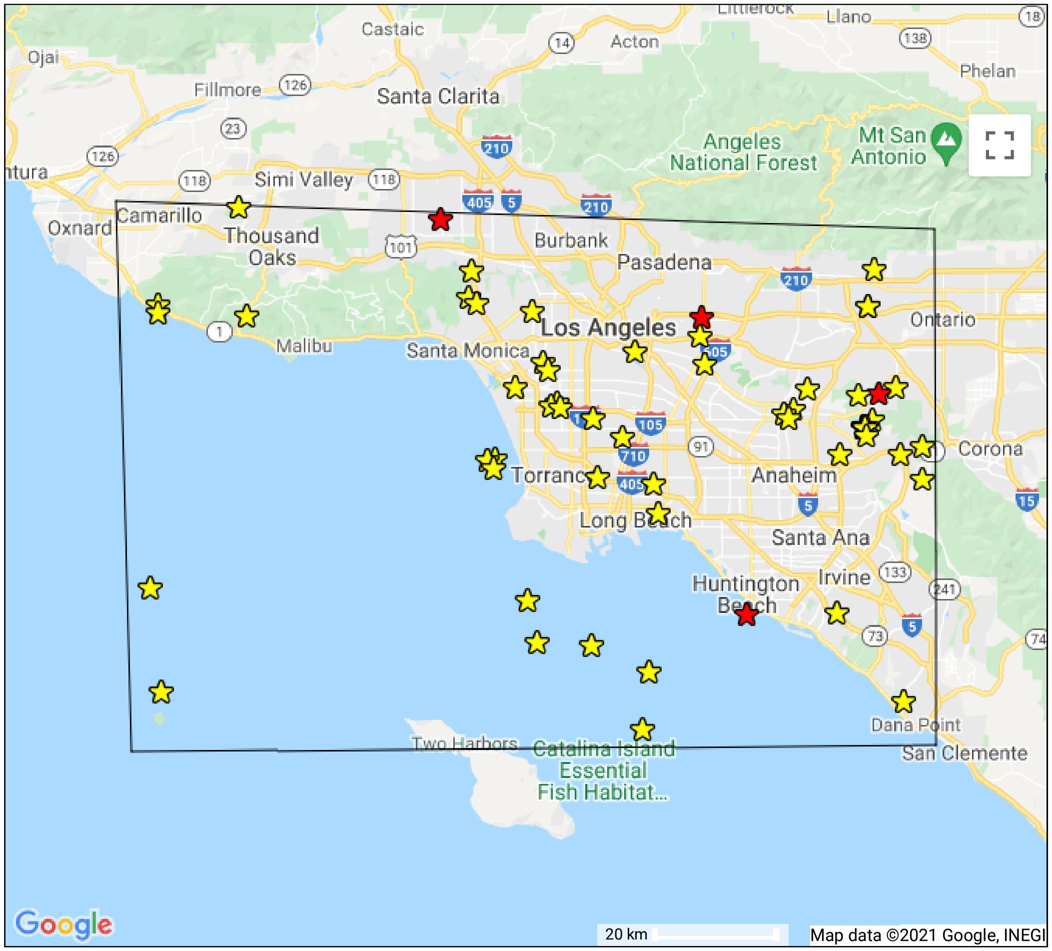
Long Beach, 10 Mar 1933, 6.4MW
Los Angeles, 02 Oct 1933, 5.4Ml
Los Angeles Airport, 25 Jul 2012, 3.7ML
Manhattan Beach, 07 Jun 2010, 3.7ML
Northridge, 17 Jan 1994, 6.4Ml
Pomona, 19 Sep 2013, 3.7ML
Pomona, 19 Sep 2013, 3.8ML
Pt. Fermin San Pedro, 15 May 2013, 4.0ML
Pt. Fermin San Pedro, 30 Dec 2014, 3.9ML
Rolling Hills, 30 May 2014, 3.8ML
Rowland Heights, 29 Mar 2014, 4.1MW
Santa Barbara Is., 24 Aug 2010, 4.0ML
Santa Barbara Is., 29 May 2012, 4.0ML
Seal Beach, 14 Sep 2019, 3.5MW
South El Monte, 18 Sep 2020, 4.5MW
Thousand Oaks, 13 Sep 2021, 3.6MW
View Park-Windsor Hills, 03 May 2015, 3.8ML
View Park-Windsor Hills, 22 Apr 2020, 3.7MW
Westlake Village, 01 May 2009, 4.4ML

Westwood, 18 Sep 2017, 3.6ML
Westwood Village, 01 Jun 2014, 4.2ML
Whittier, 01 Oct 1987, 6.1 ML
Whittier Narrows Area, 16 Mar 2010, 4.4ML
Willowbrook, 20 Jan 2021, 3.5ML
Yorba Linda Area, 23 Apr 2009, 4.0ML
Yorba Linda, 03 Apr 2019, 3.6ML
Yorba Linda, 03 Aug 2020, 3.5ML
Yorba Linda, 03 Sep 2002, 4.8 ML
Yorba Linda, 07 Aug 2012, 4.5ML
Yorba Linda, 08 Aug 2012, 4.5ML
Yorba Linda, 13 Jun 2012, 4.0ML
Yorba Linda, 29 Aug 2012, 4.1ML

Earthquakes with Strong Motion Records in CESMD

Use polygon to display and list earthquakes within area of interest:

1. Start Polygon | 2. Close Polygon | 3. Show Result | Start Over



Help | View US Earthquakes

Significant Strong-Motion Earthquakes
Other

Hover over star to reveal earthquake name and information.

Note: Event Epicenters are approximate when viewed at high zoom levels.

Click earthquake for more information

CESMD

Click on an earthquake below to see details on map

Type to search events...

- Choose an Earthquake in the selected Polygon -
Avalon, 26 Oct 2017, 3.7ML
Beverly Hills, 09 Sep 2001, 4.2 ML
Brea, 28 Mar 2014, 3.6ML
Camarillo, 12 Jun 2019, 3.7ML
Camarillo, 15 May 2010, 3.9ML
Carson, 17 Sep 2021, 4.3MW
Chino Hills Aftershock of 29 Jul 2008, 3.8ML
Chino Hills, 29 Jul 2008, 5.4Mw
Compton, 18 Oct 2019, 3.5ML
Corona, 15 Jan 2014, 3.6Ml
Corona, 13 Apr 2001, 3.6ML
Diamond Bar, 14 Dec 2001, 4.0 ML
East Los Angeles, 24 Oct 2021, 3.6MW
Encino, 17 Mar 2014, 4.4Mw
Hermosa Beach, 07 Jun 2010, 3.5ML
Hermosa Beach, 08 May 2021, 3.5ML
Inglewood Aftershock of 19 May 2009, 4.0ML
Inglewood Area, 17 May 2009, 4.7Mw
Irvine, 15 Sep 2011, 3.5ML
La Habra, 28 Mar 2014, 3.6ML
La Habra, 28 Mar 2014, 5.1Mw
La Verne, 28 Aug 2018, 4.4MW
Laguna Niguel, 23 Apr 2012, 3.9ML
Lennox, 05 Apr 2021, 4.0MW
Long Beach, 05 Nov 2010, 3.7ML
Long Beach, 10 Mar 1933, 6.4Mw
Los Angeles, 02 Oct 1933, 5.4Ml
Los Angeles Airport, 25 Jul 2012, 3.7ML
Manhattan Beach, 07 Jun 2010, 3.7ML
Northridge, 17 Jan 1994, 6.4Ml
Pomona, 19 Sep 2013, 3.7ML

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