ASSOCIATED SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

REPORT OF GEOTECHNICAL INVESTIGATION

Proposed Community Hub Building at Santa Ana Zoo

1801 East Chestnut Avenue

City of Santa Ana, California

Prepared For:

LOC Architects
3203 East 4th Street
Los Angeles, California 90063

Project No. 7160.23 **January 23, 2024**

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SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

January 22, 2024
Project No. 7160.23

LOC Architects
3203 East 4th Street
Los Angeles, California 90063

Attention: Poonam Sharma

Subject: **REPORT OF GEOTECHNICAL INVESTIGATION AND FIELD INFILTRATION TESTING**
Proposed Community Hub Building at Santa Ana Zoo
1801 East Chestnut Avenue, Santa Ana, 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 community hub building (the Building) in the Santa Ana Zoo, located at 1801 East Chestnut Avenue in the City of Santa Ana, California. This work was conducted in accordance with ASE's Proposal No. P23-041, dated March 17, 2024, which subsequently received your authorization.

The subject geotechnical investigation was planned and performed based on the relevant project information provided by your office, which included request for proposal, prepared by City of Santa Ana, detailing the project scope of work. Also provided were plans entitled "Topographic Survey", prepared by kpff, dated September 29, 2023 and "Implementation Plan Summary" (3 sheets) prepared by CLR Design, dated February 2018. The plans show the existing site layout and proposed future zoo developments, respectively.

The purpose of this study was to evaluate the subsurface soils conditions at the Sites, 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 criteria 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, 3) the results of laboratory tests performed, and 4) the measured results from on-site percolation testing and the calculated infiltration rates.

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

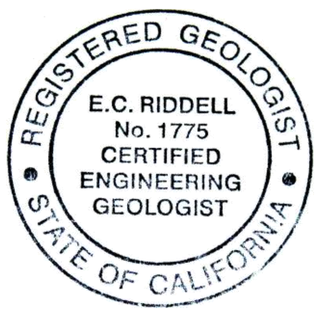
If you have any questions or require additional information, please contact the undersigned.

Respectfully submitted,

ASSOCIATED SOILS ENGINEERING, INC.

Torin Ng, EIT
Project Engineer

Lawrence J.D. Chang, P.E, G.E.
Geotechnical Engineer, RGE 2881



Edward C. (Ted) Riddell, P.G.
Engineering Geologist, CEG 1775

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1.0 INTRODUCTION

This Soils Report presents the results of ASE's geotechnical investigation for the proposed community hub building (the Building), located within the Santa Ana Zoo, at 1801 East Chestnut Avenue, in the City of Santa Ana, 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, and the results of ASE's engineering evaluations/analyses, which provide the basis for the formulation of relevant geotechnical conclusions and recommendations.

1.1 Project Outline

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

1.1.1 Building/Development Scope:

Based on the provided information, ASE understands that the Building will be located along the east perimeter of the zoo, west of the overflow parking lot and north of lawn and proposed entry village. The Building will be approximately 6000 square feet in planar dimension for education, containing flex classrooms and a new holding building for ambassador animals. The other appurtenant improvements are likely to include signage, landscape, pavement, utilities and hardscape.

1.1.2 Structural Loading for Geotechnical Analyses:

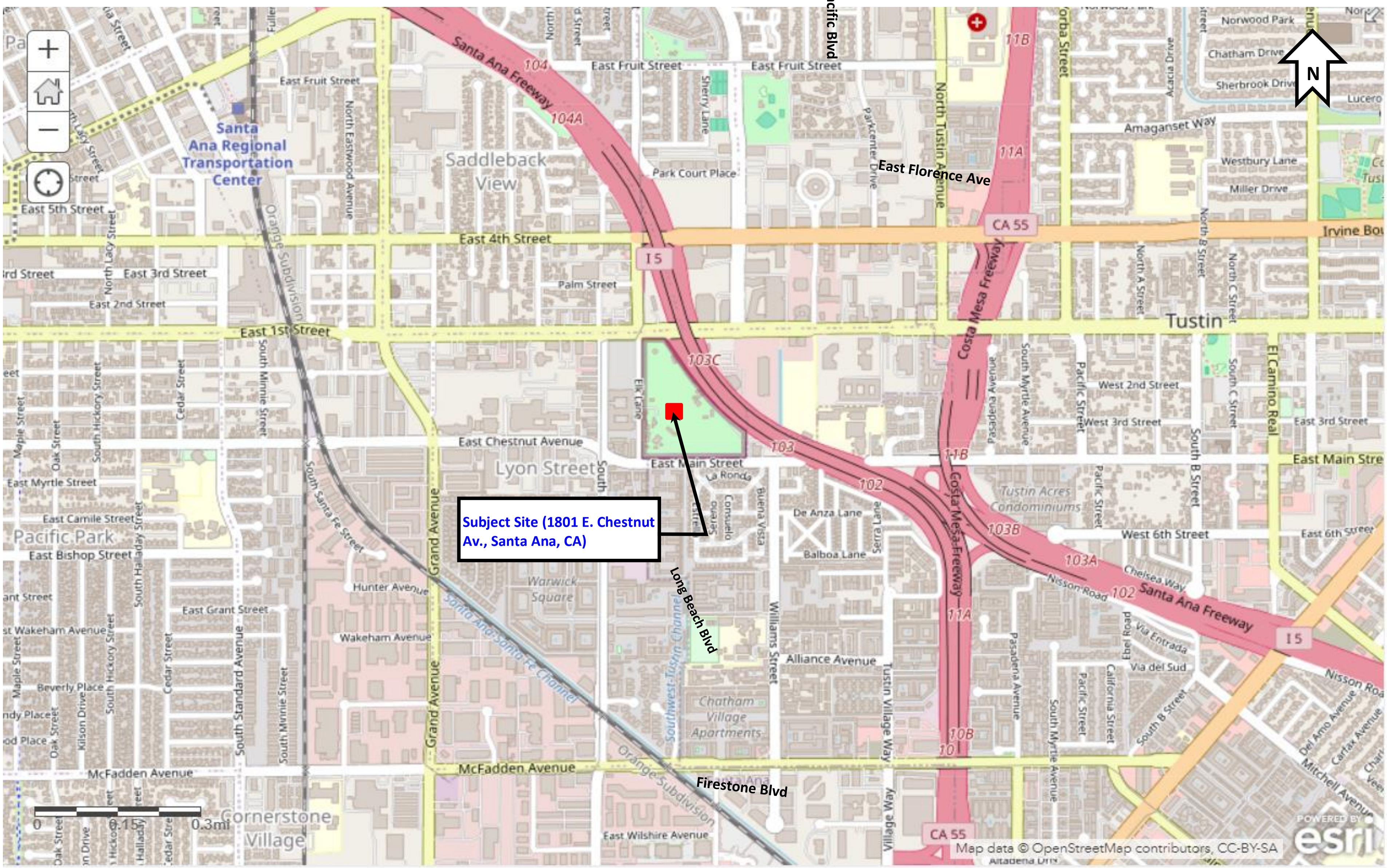
In the absence of structural loading information and for the purpose of foundation analysis, ASE has assumed that the Building will have a maximum concentrated column load (D+L) on the order of 40 kips when supported by isolated pad footings or have a maximum line load (D+L) not exceeding 3,000 pounds per linear foot (plf) when supported on continuous spread footings.

For any new secondary structural improvements (i.e. site walls, trash enclosures, signs, etc.), ASE has assumed a maximum concentrated column load (D+L) on the order of 10 kips when supported by isolated pad footings or a maximum line load (D+L) not exceeding 1,500 pounds per linear foot (plf) when supported on continuous spread footings.

Tolerable total and differential static settlements resulted from the above structural loadings on the order of one-half (1/2) inch and one-quarter (1/4) inch over a 30-foot distance, respectively, have been considered by ASE.

1.2 Scope of Investigation

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



Associated Soils Engineering, Inc. **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	**Site Location Map**	Proj. Name:	**Proposed Community Hub Building** **1801 E. Chestnut Av., Santa Ana,CA**
		Figure 1	Proj. No.: **7160.23**
			Date: **January, 2024**

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

B. A geotechnical site reconnaissance to observe the general Site conditions and to select/mark boring locations, followed by 72-hour notification to Underground Service Alert prior to field investigation.

C. Field exploration consisting of drilling two (2) exploratory borings to depths of 25 feet 11 inches and 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 and Percolation Test Location Plan, Plate A, in Appendix A.

D. Field percolation testing at two (2) percolation test borings location (i.e. Percolation Borings B-P1 and B-P2, with approximate locations shown on the Boring and Percolation Test Location Plan, Plate A, in Appendix A) to measure infiltration rates of site soils as part of the requirements for the planning and design of on-site stormwater Low Impact Development (LID) facilities.

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

F. Engineering analyses of data obtained from literature review, the 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 and quantification of geologic/seismic hazards based on the pertinent criteria required by the California Geological Survey (CGS).
- Determination of the seismic design parameters in accordance with Chapters 16 and 18 of the California Building Code, 2022 Edition (2022 CBC; Reference 5).
- Evaluation of the suitability of on-site soils for foundation support, followed by establishment of qualification criteria for on-site or imported fill material, and recommendations for site remedial grading and subgrade preparation for the Building.
- Recommendations of suitable foundation systems including conventional shallow footing foundations, covering minimum dimensions, allowable bearing capacity, estimated settlement, and lateral resistance.
- Recommendations for subgrade preparation and design parameters for slab-on-grade and flatwork support.
- Recommendations for temporary excavation, shoring and trenching.
- Evaluation of the corrosion and expansion potential of the on-site materials.

- Computation of design infiltration rates of site soils required for on-site stormwater low impact development (LID) system planning and design.

G. Preparation of this Soils Report presenting the works performed, the data acquired, and our conclusions and geotechnical recommendations for the design and construction of the Building. Also presented are the recommended design infiltration rates for on-site stormwater LID system planning and design.

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 and Surface Conditions

The Building are to be located within the Santa Ana Zoo at 1801 East Chestnut Avenue in the City of Santa Ana.

The Site is bound north and northeast by East 1st Street and the Santa Ana Freeway, respectively. Zoo Lane and East Main Street bound the Site to the west and south, respectively.

The Building will be located easterly within the existing Santa Ana Zoo facility. The area of the Building is presently occupied by open unpaved entry walkway and turf lawn, and is generally uniform and level. Existing zoo buildings throughout the Site are generally one to two-story in height. Animal enclosures, small bushes and trees are present throughout the Site. Asphaltic concrete (AC) paved parking lots and access roads are present to the south and east of the Site.

2.2 Subsurface Conditions

2.2.1 Artificial Fill (af):

Artificial fill was not observed in any of ASE's exploratory borings and percolation test borings but may be present at other areas of the site, or could be encountered during site grading, subject to the observation and confirmation of the Geotechnical Consultant.

2.2.2 Younger Fan Deposits (Qyfa):

Native site soils consisting of Holocene-age younger fan deposits were encountered from surface to the maximum explored depth of 26 feet in ASE's exploratory boring B-1. Per Reference 4, the younger fan deposits are characterized as unconsolidated sand, sandy silt, and silt of the Santa Ana River, Santiago Creek and Peters Creek. In specific, the on-site fan deposits materials consist of sandy silts, silty sands, and gravel. Figure 2, Local Geologic Map, excerpted from Reference 2, shows geologic material distribution in the vicinity of the Site.

Blow counts recorded from advancing Modified California barrel sampler empirically indicate that the granular, sandy strata of site native alluvial soils were in a medium dense to very dense condition, whereas the fine-grained, cohesive strata encountered (i.e. sandy silts) were generally in a firm to stiff condition. Site subsurface soils were, in general, in a dry to moist condition 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" Plates) and Field Logs of Percolation Boring ("B-P" Plates) in Appendix A, together with information of soil classifications, depths and types of soil samples, blow counts, field dry densities and moisture contents, and corresponding laboratory tests performed.

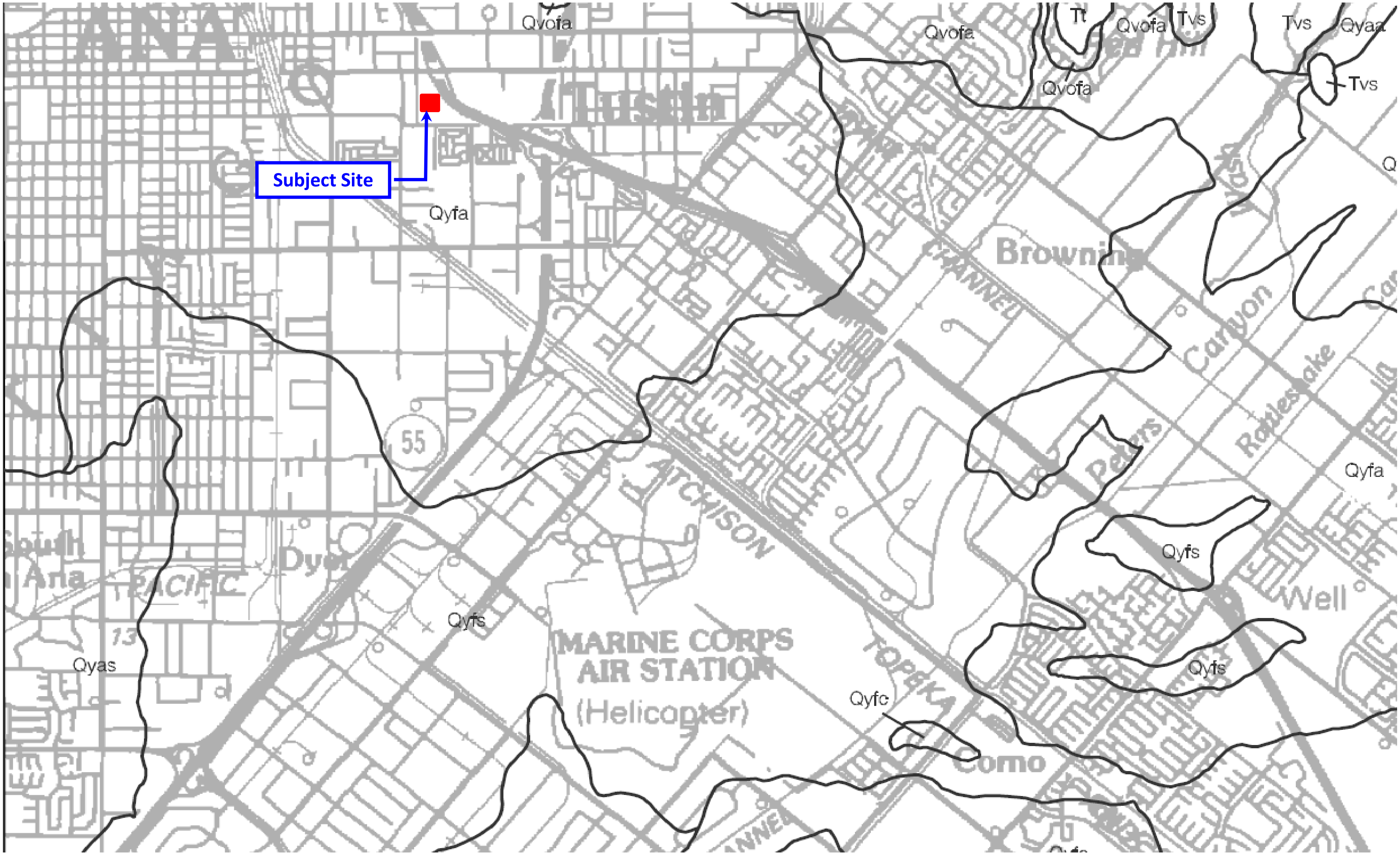
2.3 Groundwater and Caving

During field exploration, groundwater was not encountered in ASE's exploratory borings to the maximum explored depth of 26 feet in Boring B-1. Published groundwater data in Reference 4 indicates that the historic high groundwater levels in the vicinity of the Sites are approximately 40 feet deep. A search on Google Earth indicates that the Site is approximately 129 to 132 feet above Mean Sea Level (MSL).

Information available 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 B-22, (UNOCAL #4991: 1601 East 1st Street – located approximately 1/4 mile northwest of the Site), was 63.4 below grade on May 29, 2007, which was the most recent reading in this well. The ground surface elevation at this well location (taken from Google Earth images) is approximately 131 feet above MSL, which is approximately the same as the Site grades.

Additional information available from the same Geotracker website indicates that the groundwater elevation in groundwater monitoring well MW-7, (Thrift Oil $377/ ARCO #7741: 324 South Grand Avenue – located approximately 1/2 mile west of the Site), was 64.57 below grade on June 22, 2005, which was the most recent groundwater reading in the well. The ground surface elevation at this well location (taken from Google Earth images) is approximately 122 feet above MSL, which is approximately 7 to 10 feet lower Site grades.

Generally, 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 notes that ASE's exploratory borings were not meant for groundwater monitoring.



(Partial Extract of the Quaternary Geologic Map of the Santa Ana 7.5-Minute Quadrangle,California Division of Mines and Geology,Seismic Hazard Zone Report 012, 1998)

LEGEND		
	■	Approximate Site Location
	Qyfa	Younger Fan Deposits: unconsolidated sand, sandy silt and silt of Santa Ana River, Santiago Creek, and Peters Creek

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Project:	**Proposed Community Hub Building 1801 E. Chestnut Av., Santa Ana,CA**		
Figure 2	**Local Geologic Map**		
Proj. No.:	**7160.23**	Date:	**January, 2024**

The use of hollow-stem augers during drilling precluded observation of potential caving conditions which may have otherwise occurred in an uncased hole. Caving and/or sloughing were not measured during the extraction of auger stem at the completion of boring operations. However, caving and/or soil sloughing may be likely in excavations greater in dimension than our exploratory borings.

2.4 Utilities

No overhead or underground utilities were encountered or disturbed during the course of ASE's on-site exploration. However, underground utilities servicing the existing buildings may be present on site, and should be located and incorporated into site development plans accordingly.

3.0 FAULTING AND SEISMICITY

Santa Ana, 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 subject Site is not located within the Alquist-Priolo Earthquake Fault Zone. Additionally, the site is not located within a seismic hazard zone per CGS's mapping.

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, and the approximate location of the fault trace. Printouts of the Site fault search results are shown on Plates I-1 and I-2 in Appendix B.

3.1 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 San Joaquin

Hills Fault, approximately 3.9 miles (6.3 km) from the Site, would probably generate the most severe 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 6.6 Mw (moment magnitude as per USGS) has been assessed along the San Joaquin Hills Fault. As shown on Plate I-2 in Appendix B, estimated PGA resulting from a MPE event on the San Joaquin Hills Fault is on the order of 0.457g should this event occur at the fault's closest approach to the Site. Other nearby active faults include the Newport-Inglewood (L.A.Basin) Fault and the Newport-Inglewood (Offshore) Fault, located approximately 9.6 miles (15.5 km) and 11.3 miles (18.2 km) away, respectively. In sum, 40 active or potentially active faults have been identified within 62 miles (100 km) of the Site.

3.2 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 259 m/sec, corresponding with "Site Class D", nearest to the derived V_s^{30} value of 294 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's Unified Hazard Tool [3]

Latitude	Longitude	V_s^{30} (m/sec)	Scenario	Acceleration (g)		
				PGA	Sa – 0.2 sec	Sa – 1.0 sec
N 33.7429°	W 117.8417°	259	MPE [1]	0.3837	0.9436	0.5290
			MCE [2]	0.6370	1.5084	0.9860

1. MPE scenario carries a 10% exceedance probability in 50 years.
2. MCE scenario carries a 2% exceedance probability in 50 years.
3. Edition: Conterminous U.S. 2014 (update) (4.2.0)

3.3 2022 CBC Seismic Design Parameters

The earthquake design requirements listed in 2022 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 2022 CBC unless otherwise specified by the governing authority having jurisdiction over the project.

The 2022 CBC seismic design criteria for the Site based on a Site Class of "D", determined based on the inferred V_s^{30} value of 294 m/sec shown in Section 3.2 above and the criteria of 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 5, including Supplements 1 and 3. Summaries of the seismic coefficients for the Site are tabulated below.

2022 CBC SEISMIC DESIGN PARAMETERS					
Site Latitude:	N 33.7429°	**Site Longitude:**	W 117.8417°	**Risk Category** [a]	II
Seismic Parameter				**Recommended Value**	
Site Class [b]				D	
Soil Profile Name [b]				Stiff Soil Profile	
Site Coefficient, Fa [c]				1.0	
Site Coefficient, Fv [d]				1.842	
0.2-Second Spectral Response Acceleration, S_S [e]				1.284g	
1.0-Second Spectral Response Acceleration, S_1 [f]				0.458g	
Adjusted 0.2-Second Spectral Response Acceleration, S_{MS} [g]				1.284g	
Adjusted 1.0-Second Spectral Response Acceleration, S_{M1} [h]				0.844g	
Design 0.2-Second Spectral Response Acceleration, S_{DS} [i]				0.856g	
Design 1.0-Second Spectral Response Acceleration, S_{D1} [j]				0.562g	
Long -Period Transition Period, T_L [k]				8 sec	
Mapped MCE_G Geometric Mean Peak Ground Acceleration, PGA [l]				0.538g	
Site Coefficient, F_{PGA} [m]				1.1	
MCE_G Peak Ground Acceleration adjusted for Site Class Effect, PGA_M [n]				0.592g	
Risk Category			**I or II or III**		**IV**
Seismic Design Category based on S_1 [o]			N/A		N/A
Seismic Design Category based on SD_S [p]			D		D
Seismic Design Category based on SD_1 [q]			D		D

a Per 2022 CBC Table 1604.5
b Per 2022 CBC Section 1613.2.2
c Per 2022 CBC Table 1613.2.3(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 F_v, S_{MS}, S_{M1} values.*
d Per 2022 CBC Table 1613.2.3(2), *provided C_s values are determined by Equations 12.8-2, 12.8-3 and 12.8-4 of ASCE 7-16.*
e Per 2022 CBC Figure 1613.2.1(1)
f Per 2022 CBC Figure 1613.2.1(2)
g Per 2022 CBC Equation 16-20
h Per 2022 CBC Equation 16-21
i Per 2022 CBC Equation 16-22
j Per 2022 CBC Equation 16-23
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 2022 CBC Section 1613.2.5
p Per 2022 CBC Table 1613.2.5(1)
q Per 2022 CBC Table 1613.2.5(2)

Please note, 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 above reflect invocation of such exceptions (see Footnotes c and d beneath the said table). Please note that, as S_1 value is greater than 0.2, should exception per Item 1 in Section 11.4.8 of

ASCE 7-16, Supplement 3 be applied, a 50% increase on the S_{M1} value derived from Equation 11.4-2 of ASCE 7-16 (or Equation 16-21 of 2022 CBC) is required. The increased S_{M1} value should then serve as the basis for the derivation of S_{D1} value for structural design. Please note that the S_{M1} and S_{D1} values listed in the table on the preceding page do not reflect such 50% increase. 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 design parameters could be established.

Please also note that conformance to the 2022 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.

4.0 GEOLOGIC HAZARDS

4.1 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.

4.2 Seismic Hazards

4.2.1 Liquefaction:

As evidenced in Figure 3, Local Seismic Map, the Site is not within an area identified by CGS as having a potential for soil liquefaction when subject to an MPE event.

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 primarily consist of dense to very dense granular, sandy soils.

During ASE's field exploration, groundwater was not encountered to the maximum explored depth of 26 feet. Per Reference 4, historic high groundwater contour in the vicinity of the Site is approximately 40 feet below grade. Additionally, the groundwater levels in groundwater monitoring wells closest to the Site were in excess of 60 feet below well grades, as per reviewed from the State Geotracker well records.

Considering that: 1) site subsurface soils have been classified as Holocene-age younger fan deposits consisting of dense to very dense granular, sandy soils within the maximum explored depth of 26 feet and likely beyond; 2) a PGA_M of 0.592g from 2022 CBC seismic design criteria; 3) the historic high groundwater level is 40 feet deep per CGS and likely exceeding 60 feet deep per nearby groundwater monitoring well data; and 4) an earthquake magnitude of 6.6 Mw derived per EQFAULT software, the potential for the occurrence of seismically-induced liquefaction at the Site has been assessed to be nil, per the criteria stipulated in SP 117A (Reference 2).

4.2.2 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. Although the earth materials on site include medium dense to very dense granular, sandy soils within the maximum depth explored, and are considered non-liquefiable as per stated in Section 4.2.1 above, they may still undergo minor seismically- induced volumetric densification above groundwater level upon a MCE event.

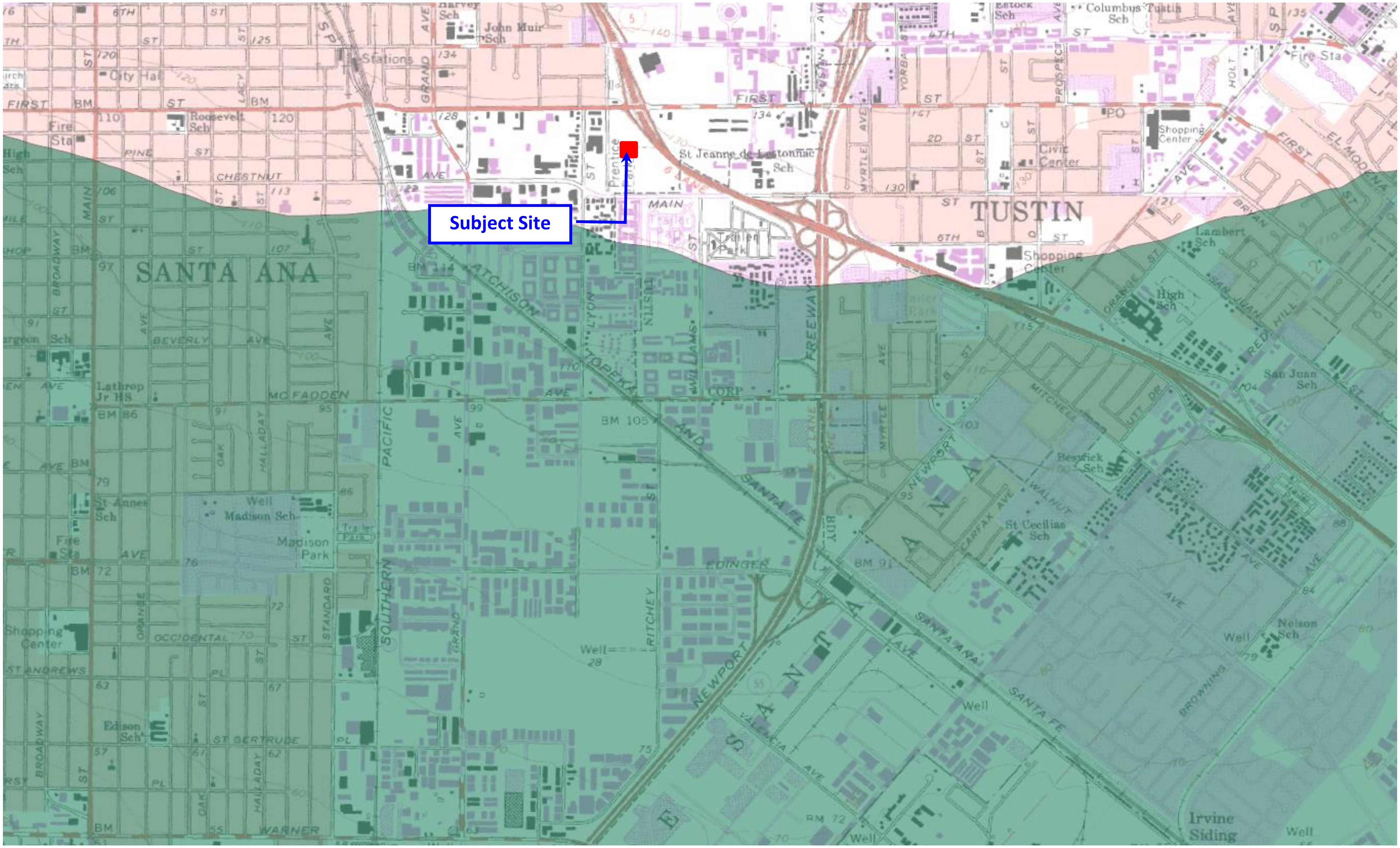
The settlement of site granular, sandy materials in their present state as a result of seismically-induced densification (i.e. “dry” seismic settlement) is estimated to be less than one-half (1/2) inch. Such magnitude of seismically-induced dry soil settlement is expected to affect relatively large area. Thus, the corresponding differential settlement over short distances is likely to be negligible.

4.2.3 Earthquake-Induced Landslides:

As evidenced in Figure 3, the Site is not located within an area identified as having a potential for earthquake-induced landslides. There is lack of significant relief on or adjacent to the Site. During ASE’s field investigation, there was no indication that recent landslides or unstable slope conditions exist on or adjacent to the Site that would otherwise result in a landslide hazard to the Building or adjacent properties. The potential for earthquake-induced landslides at the Site is considered nil.

4.2.4 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



(Partial Extract of the Earthquake Zones of Required Investigation Santa Ana 7.5-Minute Quadrangle, California Geological Survey, dated 2016)

LEGEND

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

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Project:	**Proposed Community Hub Building 1801 E. Chestnut Av., Santa Ana,CA**
Figure 3	**Local Seismic Hazard Map**
Proj. No.:	**7160.23** — Date: **January, 2024**

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 there is no liquefaction potential at the Site, as per discussed in Section 4.2.1 above, the potential for the occurrence of lateral spreading is also nil at the Site.

4.2.5 Tsunamis and Seiches:

Due to the elevation of the Site and absence of nearby waterfront, hazard from a 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 bodies of water lie on or near the Site, the hazard from seiches is not present at the Site.

4.2.6 Flood Hazards:

The Site was located on the ESRII/FEMA Hazard Awareness site, as shown on Figure 4, National Flood Hazard Layer FIRMette. The Site is not located within the limits of the 100-year flood plain per FEMA Flood Insurance Rate Map (Map No. 06059C0277J, map revised December 3, 2009), and is located outside an area of 0.2-percent-annual-chance flood.

5.0 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, clearing and excavation operations.
2. Presence of loose, low density soils within the zone of foundation bearing strata.
3. Excavation and construction of new footings/foundations located adjacent to or near existing building foundation that might undermine stability. Therefore, it is of essential importance that the embedment depth of any new footing planned next to the existing footing be the same as the embedment depth of the existing footing. This will ensure that: a) no soils beneath the existing footing would be undermined resulting in the bearing support to the existing footing being compromised, and b) no undesirable surcharge would be imposed on the existing footing from and adjoining new footing.

In consideration of the above factors, it is ASE's opinion that overexcavation and backfilling with properly compacted fill in the areas of the Building, as recommended herein, will be essential to reduce unfavorable static settlements of underlying soils, and to provide satisfactory bearing stratum for the Building. The

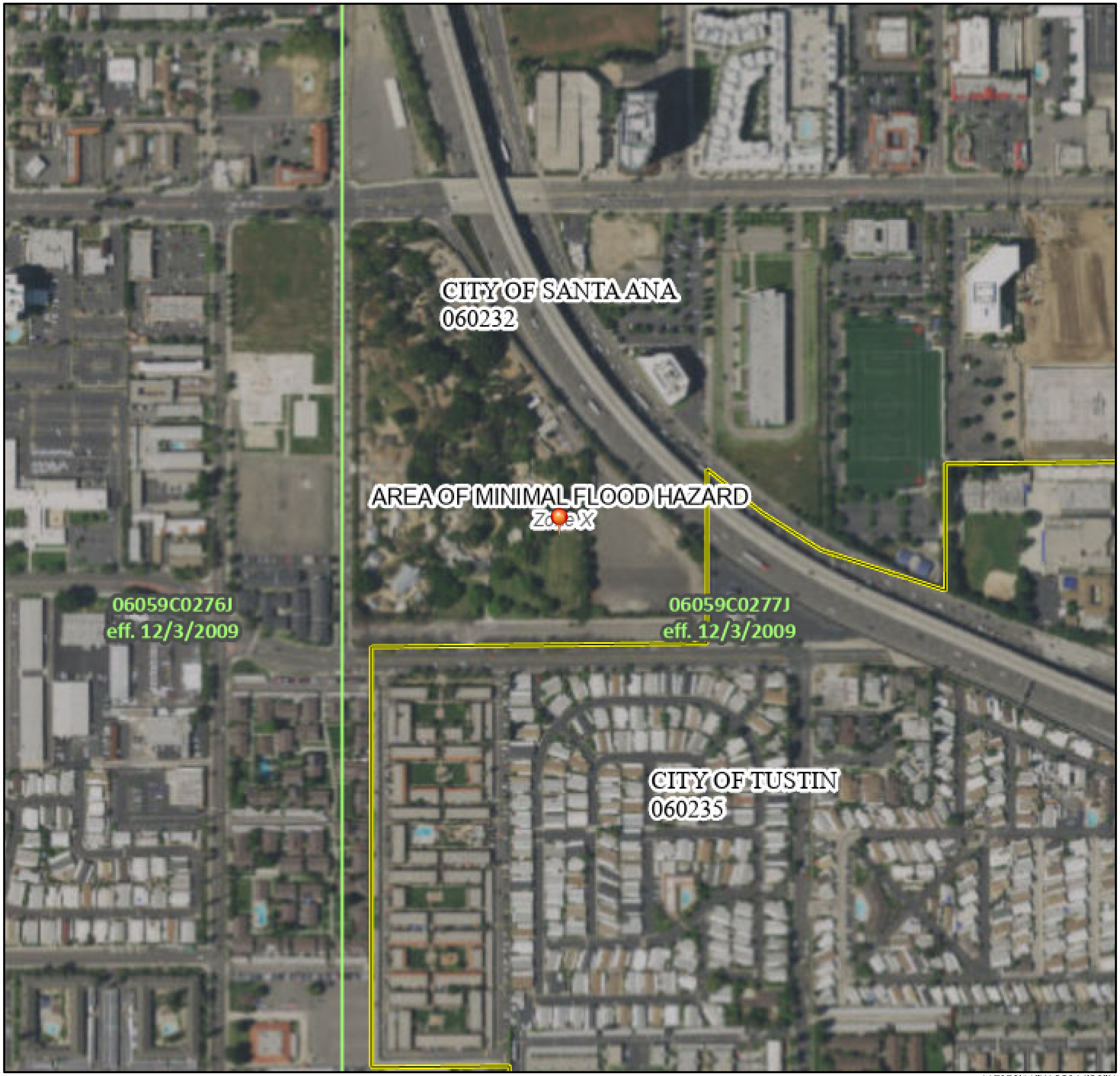
ASE#7160.23

National Flood Hazard Layer FIRMette

FIGURE 4

FEMA

117°50'49"W 33°44'50"N



117°50'11"W 33°44'20"N

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

1:6,000

Basemap Imagery Source: USGS National Map 2023

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
- 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 1/4/2024 at 2:51 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.

grading recommendations provided herein should be reviewed when final project concept and grading plans are available. It is assumed that the proposed finish grades will be close to the existing site grades ($\pm$one foot).

5.1 Site Preparation

5.1.1 Existing Improvements:

Prior to grading operations, it will be necessary to remove any existing improvements, including any remaining buried obstructions, which may be in the areas of the Building. Structure removal should include foundations. Concrete flatwork and asphalt 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.

5.1.2 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.

5.1.3 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 Building. 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.

Please note that local ordinances relative to abandonment of underground utilities, if more restrictive, should be complied with.

5.2 Site Grading

In minimizing the potential adverse effects associated with the development of excessive total or differential settlement underneath the Building, as well as to ensure uniform bearing competency for the foundations and slabs, preparation of on-site soils is recommended in the following sections.

5.2.1 Undocumented Fill/Disturbed Native Soils:

Although not encountered in ASE's exploratory borings, any undocumented fill soil, if 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 Building perimeters, where possible, should be to a minimum distance equal to the depth of undocumented fill/disturbed soil encountered or four (4) 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 eighteen (18) inches, whichever is greater.

The exposed excavation bottom should be scarified/reworked to a minimum one (1) foot depth and recompacted to at least 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 5.2.6.

5.2.2 Expansive Soils:

Laboratory testing result on a near surface site soil sample indicates "Low" soil expansion potential (i.e. Expansion Index, EI = 31 per ASTM D4829-21 Test Method) as defined in Table 1 of ASTM D4829-21 Test Method and Section 1803.5.3 of 2022 CBC. Lightly loaded structural elements such as shallow foundations and slabs could undergo noticeable movements, at time unevenly, in areas underlain by soils with "Low" expansion potential. It should be noted that design provisions such as increased reinforcements, deeper foundations or other measures discussed in this Soils Report may help to alleviate the undesirable effects of "Low" soils expansion on the slabs and structures but may not completely eliminate the problem.

It is recommended that the soil expansion potential be re-evaluated through additional testing during or after rough grading operations to verify the design adequacy of footing foundation and lab-on-grade against the re-tested soil expansion potential as heterogeneity within soil mass is not uncommon.

5.2.3 Remedial Grading:

a) Proposed Building and Ancillary Improvements:

To provide competent bearing support for the proposed restroom building and to reduce potential static settlements, it is recommended that on-site soils within the footprint of the restroom building be overexcavated and removed uniformly to a minimum depth of four (4) feet below existing grade or finish grade, whichever is lower, and replaced with properly compacted fill such that the building foundation is supported on a re-engineered, compacted fill layer. The excavation bottom should be near uniform. The overexcavation should extend laterally to a minimum distance of four (4) feet beyond the restroom perimeters, wherever possible. The fill should be compacted to at least 90 percent relative compaction with minimum moisture content of two (2) percentage points above optimum moisture content. On-site subgrade soils at their present state generally exhibit an EI within the preferred value and, thus, are deemed suitable for re-use as fill.

For foundations supporting ancillary improvements, it is recommended that on-site soils within the footprints of the ancillary improvements be overexcavated and removed uniformly to a minimum depth of three (3) feet below existing grade or finish grade, whichever is lower, and replaced with properly compacted fill such that the foundations for ancillary improvements are supported on a re-engineered, compacted fill layer. The excavation bottoms should be near uniform. The overexcavation should extend laterally to a minimum distance of three (3) feet beyond structure perimeters, where possible.

Soils exposed at excavation bottoms to a depth of one (1) foot 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.

b) Exterior Slab-on-Grade/Concrete Flatwork/Hardscape/Pavement Support:

For the purpose of reducing future unsightly and uneven movements and cracks of any exterior slab-on-grade, concrete flatwork, hardscape, or pavement, 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/hardscape/ pavement should be overexcavated then backfilled and recompacted with suitable fill soils consisting of “Very Low” to “Low” expansive site or import material (EI ≤ 35), 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.

Please note that if undocumented fill is encountered in any area to receive remedial grading discussed in Sections 5.2.3.a) or b) above, recommendations stipulated in Section 5.2.1 above, if more stringent, should be complied with. If all undocumented fill is not removed full depth in areas of exterior slab-on-grade, concrete flatwork, hardscape or pavement, the Owner should be made aware that more frequent maintenance and/or repair will likely be required. Please also note that, from geotechnical viewpoint, new landscape area with only softscape is not subject to subgrade preparation and remedial grading requirements mentioned in Sections 5.2.1 and 5.2.3.

In case of the presence of localized loose soils, the overexcavations need 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 testing and inspection by the representative from the Geotechnical Consultant.

The Geotechnical Consultant's field representative 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 5.2.1 and 5.2.3 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.

5.2.4 Temporary Excavation:

Excavations of site soils deeper than 4 feet 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 of 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

1:1.5 (H:V) for excavations up to 10 feet in depth. The above temporary slope criteria are 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 <u>40</u> 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.

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

c) Slot Cutting Adjacent to Existing Structure Foundations:

Prior to any excavation, the footings of the existing structure should be researched as it could compromise the stability of foundation when excavating site soils immediately next to or below any existing footing foundation. "A-B-C" slot cutting grading procedures may be utilized to accomplish the required overexcavation for areas adjacent to existing building foundation or improvements that might otherwise be undermined by the grading operation on the subject Site. As a general guideline, slot cutting would be necessary for overexcavation located within a lateral distance from the existing structure or public right-of-ways equivalent to one (1) times the excavation depth.

While the maximum width and sequence of slot-cuts should be evaluated in the field during grading operations based on conditions exposed during initial site grading adjacent to the existing structure, for preliminary planning purpose, the width per slot should not exceed six (6) feet. Increase of length per cut slot is possible upon inspection and evaluation of actual exposed slot cut condition by the Geotechnical Consultant during site grading. Care shall be exercised such that no soil is removed from underneath any existing shallow foundation.

5.2.5 Suitable Soils and Imported Soils:

Any fill used for the completion of subgrade preparation for the building pad areas of the Building should consist of predominantly "Very Low" to "Low" expansive material exhibiting an EI not greater than 35, 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.

Unless otherwise approved by the Geotechnical Consultant, the imported or blended fill materials should also comply with the following soil corrosivity criteria:

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 G187-23 Test Method. (4) California Test Method 532.

Imported fill soils or base materials should be examined by a representative of the Geotechnical Consultant 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 material to be used as fill must be tested and approved by the Geotechnical Consultant prior to being used for fill placement.

5.2.6 Backfilling and Compaction Requirements:

Existing site soils at their present state and composition, unless indicated otherwise, are considered suitable for re-use as fill during site grading, 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 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, 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(2021) Test Method.

5.2.7 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.

5.3 Foundation Design

Provided that the site grading recommendations presented in Section 5.2.3a above are incorporated in project planning and design, and implemented during site construction, it is ASE's opinion that the Building could be supported by conventional footing foundation.

5.3.1 Conventional Shallow Footing Foundation:

a) Minimum Footing Dimension and Reinforcement:

In order to mobilize sufficient soils bearing capacity supporting the new footings for the Building or any ancillary improvement (i.e. site walls, trash enclosures, signs, etc.), 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 (2)**	**Depth (1) (in)**	**Width (in)**	**Reinforcement (2)**
18	15	Four #4 bars – two near the top and two near the bottom	18	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.
(2) Based strictly from geotechnical point of view.

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 5.3.2a) 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	100	300	3,000

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

For any new footings that are within a lateral distance from any existing building footing equal to the depth of the new footing (D), the following tabulated reduction factors should be applied to the corresponding allowable soils bearing capacity values.

Lateral Distance between New Footing and Existing Building Footing expressed in Fraction of the New Footing Depth, D	≥ 1D	½D	0
Reduction Factor To Allowable Soils Bearing Capacity [a]	1.0	0.75	0.5

a. Interpolation may be used for deriving reduction factor for other distance value.

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 5.2, 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 0.35 may also be used for structural dead load acting between the footing bottom and the supporting soils, regardless of the lateral distance between new footing and existing building footing.

The above passive lateral earth pressure 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.

For any new footing that is within a lateral distance from any existing building footing equal to two (2) times the depth of the new footing (D), the following tabulated reduction factors should be applied to the corresponding passive lateral earth pressure values for the sides of the new footing that are facing the existing building footing.

Lateral Distance between New Footing and Existing Building Footing expressed in Fraction of the New Footing Depth, D	$\geq$ 2D	1D	0
Reduction Factor To Passive Lateral Earth Pressure [a]	1.0	0.5	0

a. Interpolation may be used for deriving reduction factor for other distance value.

The above passive lateral earth pressure 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:

Total static settlements resulting from compression of subgrade soils for conventional footings designed and constructed in accordance with the criteria discussed in preceding Sections 5.3.2.a), b), and c) and supporting maximum assumed dead plus live (D+L) column and wall loads mentioned in Section 1.1.2 above, are not anticipated to exceed 1/2 inch, upon implementation of site preparation as per recommended in Section 5.2 above. A differential settlement/heaves on the order of 1/4 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 be reminded that the Geotechnical Consultant should be contracted for further evaluation and recommendations, as necessary, should final design structural loads exceed the maximum loads assumed in the above analyses by more than ten (10) percent.

5.3.2 Retaining Walls:

Cantilevered retaining walls should be designed for an “active” lateral earth pressure value tabulated on the next page for approved granular backfill soils or site soils and level backfill conditions, whereas an “At-rest” lateral earth pressure value for approved granular backfill or site soils and level backfill conditions tabulated on the next page should be used for top-restrained retaining walls. Retaining walls subject to uniform surcharge loads should be designed for an additional uniform lateral pressure equal to one-third (1/3) and one-half (1/2) of the anticipated surcharge pressure over the full retained height of the retaining wall (measuring from the top of wall to the heel of wall footing) for cantilevered and top-restrained wall fixity conditions, respectively, as shown in Figure 5, Nearby Building Surcharge Consideration and Retaining Wall Drainage Details.

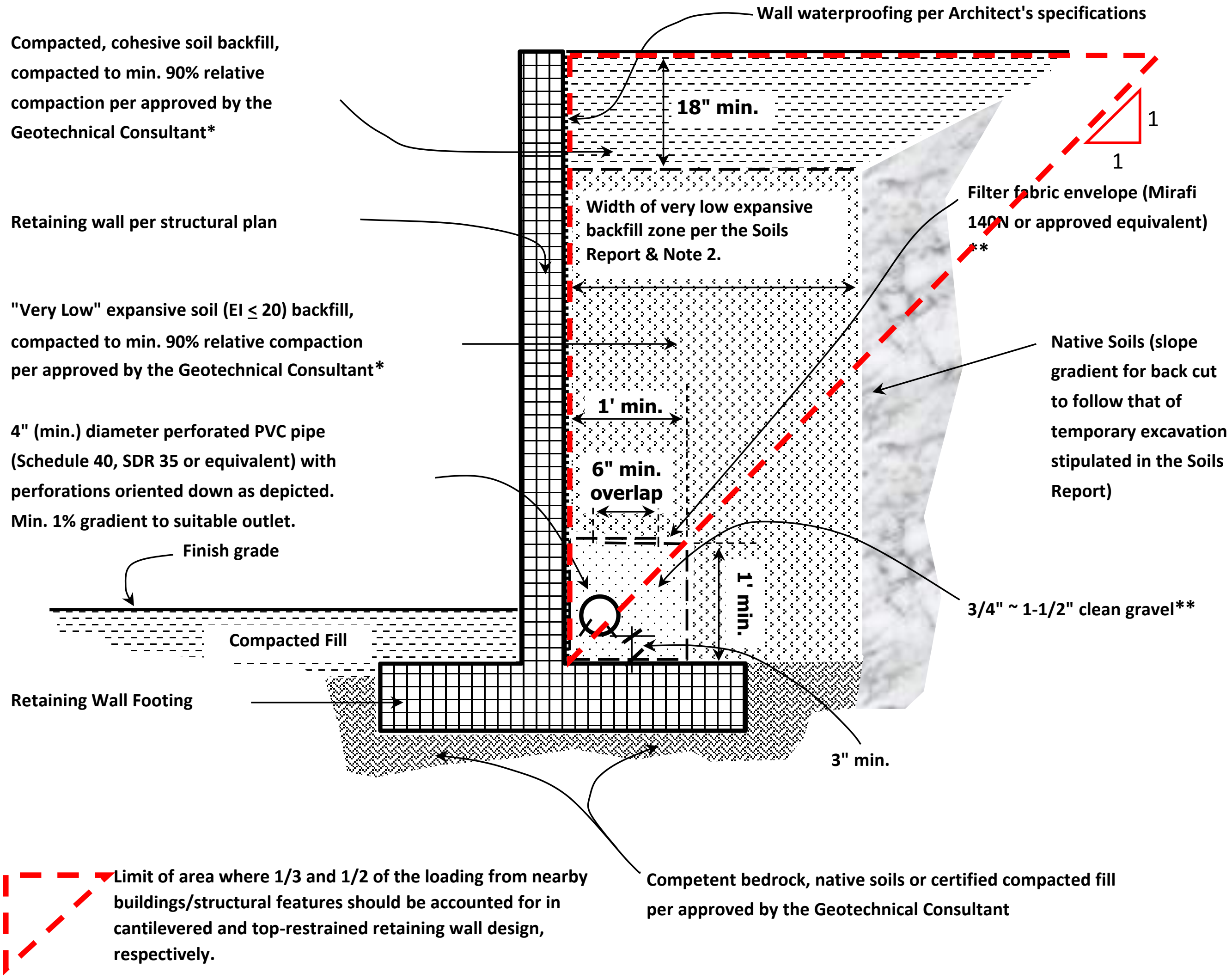
Any retaining wall with a retained height exceeding six (6) feet should additionally be designed to resist seismic lateral earth pressure. The Geotechnical Consultant should be consulted if this condition exists, or if the local governing agency requires the retaining wall to be designed for seismic lateral earth pressure regardless of the retained height. Footings should be reinforced as recommended to by Structural Consultant. Appropriate back drainage should be provided to avoid excessive build-up of hydrostatic wall pressures.

Retaining Wall Design Parameter	Value
Allowable Soils Bearing Capacity	2,000 psf (1)(2)
Active Pressure [site soils backfill: level]	40 pcf EFP (3)
At-rest Pressure [site soils backfill: level]	60 pcf EFP (3)
Passive Pressure (per foot of depth)	200 pcf EFP, subject to a ceiling value of 2,000 psf(4)
Coefficient of Friction	0.35 (4)
Minimum Footing Depth	18 inches
Minimum Footing Width	15 inches
Minimum Reinforcement	Four No. 4 rebar - 2 near top and 2 near bottom

(1) Based on compliance with earthwork recommendations per Section 5.2 of this Soils Report.
(2) Allowable soils bearing capacity increase for larger retaining wall footings should be as per Section 5.3.2 b).
(3) Design values assuming a drained condition with “Very Low” to “Low” expansive materials (EI ≤ 35) within the backfill zone and no surcharge loading conditions.
(4) Passive lateral resistance may be combined with frictional resistance provided the passive lateral earth pressure is reduced by 1/3. See Section 5.3.1c.

The Geotechnical Consultant should be on-site during temporary back cut and retaining wall construction to inspect and evaluate the stability of cuts and, if necessary, to provide additional remedial or mitigative recommendations.

Preferably, the backfill should consist of approved “Very Low” expansive material (i.e. EI ≤ 20) and should be compacted to a minimum relative compaction of 90 percent. The width of the “Very Low” expansive backfill zone should be a minimum of one (1) foot measured from the rear side of the stem of the retaining wall, or the space between the rear side of the stem and the heel of the retaining wall, or one-half (1/2) of the retained height of the retaining wall, whichever is greater. Flooding or jetting of backfill should not be permitted. Granular backfill should be capped with 18 inches (minimum) of relatively impervious fill to seal the backfill and prevent saturation. Figure 5 illustrates the general configuration and requirements for retaining wall drainage. Should any conflict noticed between recommendations stated in this report and those shown in Figure 5, the fore should govern. Other retaining wall drainage alternatives such as CONTECH C-Drain or



SPECIFICATIONS FOR CALTRANS CLASS II PERMEABLE MATERIAL	
U.S. STANDARD SIEVE SIZE	% PASSING
1"	100
3/4"	90 ~ 100
3/8"	40 ~ 100
No. 4	25 ~ 40
No. 8	18 ~ 33
No. 30	5 ~ 15
No. 50	0 ~ 7
No. 200	0 ~ 3
Sand Equivalent > 75	

* Based on ASTM D-1557-12

** If Caltrans Class II permeable material (see gradation to left) is used in place of 3/4" ~ 1-1/2" gravel, filter fabric may be deleted. Caltrans Class 2 permeable material should be compacted to minimum 90 percent relative compaction. Unless otherwise specified, a minimum of 1 cubic foot of gravel should be used for each 1 foot run of drain.

Note 1: Composite drainage products such as Contech C-Drain, Miradrain or J-Drain may be used as alternative to gravel or Class II. Installation should be performed in accordance with manufacturer's specifications.

Note 2: Width of " Very Low" expansion backfill equals 1/2 of retained height, or distance from back of wall to heel of footing, whichever is greater.

Schematic Not To Scale

ASSOCIATED ASE SOILS ENGINEERING, INC.
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Tel (562) 426-7990 Fax (562) 426-1842

Project:	Proposed Community Hub Building 1801 E. Chestnut Av., Santa Ana,CA		
Figure 5	Nearby Building Surcharge Consideration & Retaining Wall Drainage Details		
Proj. No.:	7160.23	Date:	January, 2024

MIRADRAIN may be considered but should first be reviewed and approved by the Geotechnical Consultant prior to implementation.

Should the space behind the new retaining wall be too tight to implement the above recommended backfill effort, as an alternative, 2-sack control density fill (slurry fill) may be used in lieu of regular soil backfill, provided that the integrity and functionality of wall backdrain is protected and maintained. It should be noted that the use of heavy compaction equipment in close proximity to retaining structures can result in wall pressures exceeding design values and corresponding wall movement greater than that normally associated with the development of active or at-rest conditions. In this regard, the contractor should take appropriate precautions during the backfill placement.

5.3.3 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.

5.4 Slabs-On-Grade

For structural design of concrete slabs, a modulus of subgrade reaction ("k-value") on the order of 100 pounds per square inch per inch ("psi/in") and an allowable bearing capacity of 850 psf may be used for slab constructed on recompacted site soils. Interior and exterior slabs should be properly designed and reinforced for the construction and service loading conditions. To minimize static slab distress, geotechnically, it would be prudent to provide minimum actual slab thickness of four (4) inches with minimum reinforcements consisting of number 4 reinforcing bars spaced maximum 12 inches on centers each way, placed at mid-slab, for exterior slabs and Building interior slab-on-grade, when supported by compacted fill. The final 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 2022 CBC.

The entirety of any new slabs within the Building should be underlain by an impermeable vapor barrier (minimum 15-mil-thick visqueen). A minimum 12-inch overlap between visqueen sheets should be ensured during placement. All visqueen sheets should be puncture free prior to slab construction, and should be sandwiched top and bottom by two (2) inches of clean sand (Sand Equivalent (SE) ≥ 30 per ASTM D2419-22 Test Method). Alternatively, as per stipulations in Section 5.2.3.2 of ACI 302.1R-15, for slabs in moisture-sensitive areas, the concrete slabs should be poured directly on a moisture barrier consisting of 15-mil Stego Wrap Vapor Barrier/Retarder that is in turn underlain by 4" of ½" to 3/8" open graded gravel or crushed rock complying with the criteria stipulated in Section 200-1.2 of the current Greenbook. The concrete slab shall consist of a concrete mix design which will address bleeding, shrinking and curling.

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.

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.

5.5 Asphaltic Concrete (AC) Flexural Pavement Design

The finish subgrade within the new pavement areas on Site is anticipated to be underlain by compacted structural fill as per stipulated in Section 5.2.3b) above. For preliminary pavement design purposes, a conservative R-Value of 25 has been utilized considering the observed heterogenic silty sand and sandy silt site soils. The AC pavement analyses were performed based on procedures stipulated in the current edition of the Caltrans Highway Design Manual. By assuming compliance with site preparation recommendations aforementioned in Sections 5.1.1 through 5.2.6, ASE recommends that the following AC pavement structural sections be considered. However, local government authority should be consulted for minimum pavement section requirements and, if more stringent than that recommended by ASE, be complied with.

Traffic Index (TI)	**Pavement Section Alternatives**			**Remark**
	AC (1) (inches)	**AB** (2) (inches)	**Full Depth AC** (3) (inches)	
4.5	3.0	5.0	5.5	For auto parking stalls.
5.5	3.0	8.5	7.0	For auto circulation aisles.
	4.0	6.0		
7.0	4.0	11.0	9.5	For fire lanes and truck access ways/entry and exits.
	5.0	9.0		

(1) Asphaltic Concrete
(2) CAB per the Greenbook Section 200-2.2, compacted at least 95% relative compaction
(3) Upper 6 inches of subgrade soils to be compacted to a minimum 95% relative compaction

It is recommended that R-Value testing be performed on representative soil samples after subgrade preparation on the upper 2 feet to confirm/modify applicability of the above pavement sections.

The aggregate base should conform to the Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB) requirements per Sections 200-2.2 and 200-2.4 of the Greenbook, respectively. The base course should be compacted to a minimum relative compaction of 95% at a minimum of one (1) percentage point above the optimum moisture content. Field testing should be used to verify compaction, aggregate gradation, and compacted thickness.

The AC pavement should be compacted to 95% of the unit weight as tested in accordance with the Hveem procedure per the latest CTM 304 procedures. The AC material shall conform to Type III, Class C2 or C3, of the Greenbook. All subgrade and aggregate base materials should be proof-rolled by heavy rubber tire equipment to verify that the subgrade and base grade are in a firm and non-yielding condition.

All AC laydown operations should be performed under the observation of and testing by the Geotechnical Consultant's field representative. An adequate number of field density tests should be taken to ensure compliance with this report and local ordinances. New AC should be examined by a representative of the Geotechnical Consultant, and tested as necessary to ensure that they meet the recommended quality specifications prior to being hauled to the site. Final acceptance of any AB material or AC will be based upon review and testing of the material actually delivered to the site. AC delivered to the site should be tested as necessary for quality assurance and relative compaction determination during laydown.

If the paved areas are to be used during construction, or if the type and frequency of traffic is greater than assumed in the design, the pavement section should be re-evaluated for the anticipated traffic.

5.6 Portland Cement Concrete (PCC) Pavements

The PCC pavement sections tabulated below are based on load safety factors of 1.0 and 1.1, and a modulus of subgrade reaction ("k" Value) of 100 pounds per cubic inch for site soils compacted as subgrade material, and the design procedures presented in the Portland Cement Association bulletin "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984. A design service life of 20 years was assumed for the design of the Portland cement concrete pavement section.

Concrete Flexural Strength (psi) (1)	Pavement Thickness (in) (2), (4)	Pavement Thickness (in) (3), (4)
600	6.0	7.0
650	5.5	6.5

(1) Represents 90-day flexural strength. Based on Figure 10 of Reference 5, concrete with 28-day unconfined compressive strength values of 4000 to 4500 psi typically correlates to 90-day flexural strength values of 600 and 650 psi, respectively.
(2) Load Safety Factor = 1.0 (Auto Parking Stalls)
(3) Load Safety Factor = 1.1 (Fire Lanes/Truck Traffic Areas/Entry and Exits)
(4) Assumes no PCC shoulder or curb.

The Structural Consultant should establish the design details of the concrete pavement section, including reinforcements, concrete strength, and joint and load transfer requirements.

It is recommended that edges of concrete pavements which are not adjacent to existing buildings, or are adjacent to planter areas, be downturned a minimum of 12 inches or be constructed with curbing to prevent water infiltration to subgrade soils. If edges are downturned or curbing is constructed, the above pavement thicknesses should be decreased by 1/2 inch.

The upper one (1) foot of exposed subgrade soils beneath concrete pavements should be further compacted to a minimum 95 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content. Subgrade soils should exhibit a firm, unyielding surface in addition to the recommended compaction. Final compaction and testing of pavement subgrade should be performed just prior to placement of aggregate base and/or concreting. Other pertinent subgrade preparation measures stipulated in the "Thickness Design for Concrete Highway and Street Pavements" (EB109.01P), 1984, or required by the jurisdictional municipal authorities should be followed accordingly.

5.7 Site Drainage

Per Section 1804.4 of 2022 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 the 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 Civil Consultant should refer to the pertinent sections in 2022 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.

5.8 Soil Corrosivity Evaluation

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

5.8.1 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 0.002% 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.

5.8.2 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 1,953 ohm-cm per ASTM G187-23 Test Method coupled with a pH-value of 8.22 per CTM 643 classifies the on-site soils tested to be "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 32 years for the "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 14 ppm was recorded in our laboratory tests per CTM 422. Per Caltrans guidelines and specifications (References 21 and 22), 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 318-19 may be required as a result of the soluble chloride content tested. However, the compliance with the corrosivity criteria stipulated in Section 5.2.8 above will ensure that no other particular reinforcement protection.

5.9 Utility Trenches

All trenches should be backfilled with approved fill material compacted to relative compaction of not less than 90 percent. 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 may be considered suitable for bedding or shading of utilities. Site or imported soils for pipe bedding should consist of non-expansive granular soils. Bedding materials should consist of sand with a tested Sand Equivalent, SE, value (ASTM D 2419-22 Test Method) 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 structures 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 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).

5.10 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.

6.0 FIELD PERCOLATION TEST DATA

Initial seepage rates obtained during the "Reading Time Interval Test" in Borings B-P1 and B-P2 after overnight pre-soaking indicated the time interval between readings should be 10 minutes maximum, i.e. the "Sandy Soil" category. The percolation tests were therefore performed using the sandy soil method (i.e. one hour test maximum) for Borings B-P1 and B-P2, in accordance with the State of California Regional Water Quality Board Technical Guidance Document Appendices (Appendix VII) procedures modified to test the cross sectional zone of typical soils within the level of anticipated storm water infiltration (e.g. approximately 5 feet to 10 feet and 1 foot to 5 feet below existing grade for Borings B-P1 and B-P2, respectively).

Field percolation testing was conducted on December 2, 2023. Stabilized field percolation test data indicates preadjusted percolation test rates ranging from 0.690 to 5.714 minutes per inch (mpi) for clean water at the locations of Borings B-P2 and B-P1, respectively. Field percolation test data is presented on the attached Plates H-1 and H-2 in Appendix A.

Tabulated below are the results of percolation testing conducted at the locations of Boring B-P1 and B-P2, including the infiltration rate derived from the Porchet Method of Percolation Rate Conversion procedures outlined in Appendix VII of the Technical Guidelines Document Appendices.

Boring No.	Test Depth (ft)	Percolation Test Rate (Minutes/Inch)	Infiltration Rate* (Inches/Hour)
B-P1	5-10	0.690	0.570
B-P2	1-5	5.714	5.559

*Infiltration Rate derived from Porchet Method Conversion from Percolation Rate using a Factor of Safety of 3.

The rates presented on the previous page are anticipated to be the fastest rates that can be absorbed by the site soils at the boring locations. However, with time and depending on the degree of saturation of soils and other factors, the percolation rate may reduce which is typical for sewage disposal or stormwater dispersal fields. Per Appendix VII, the results of the field percolation testing (i.e. measured infiltration rate greater than 0.3 inch per hour) indicate that site soils only at Boring B-P1 location are deemed suitable for the planning and installation of an on-site stormwater LID system within the approximate upper five to ten (5-10) feet from existing grade.

Please be informed that during installation of on-site stormwater dispersal system, the following factors should be noted:

- The degree of compactive effort in the upper 1 to 1.5 feet of soils above any filter material should be between 90 and 92 percent relative compaction. As any greater compactive efforts in the soil strata of water retention system construction may cause the percolation rates to reduce substantially, it is not advisable to impose significant structural loading in these areas, from a geotechnical viewpoint.

- The rate of water transmission from the filter material to the soil will be limited the porosity characteristics of the fabric wrap around the filter material.

7.0 CLOSURE

This report has been prepared for the exclusive use of the **LOC Architects** (the Client) and their design consultants for use in the design and construction of the proposed community hub 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 or its representatives are responsible for ensuring the information and recommendations contained in this report are brought to the attention of the project engineers and architects, incorporated into the project plans, and implemented by project contractors. This report should be reflected on project grading 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 Soil 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 March 17, 2023, 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 at ASE 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

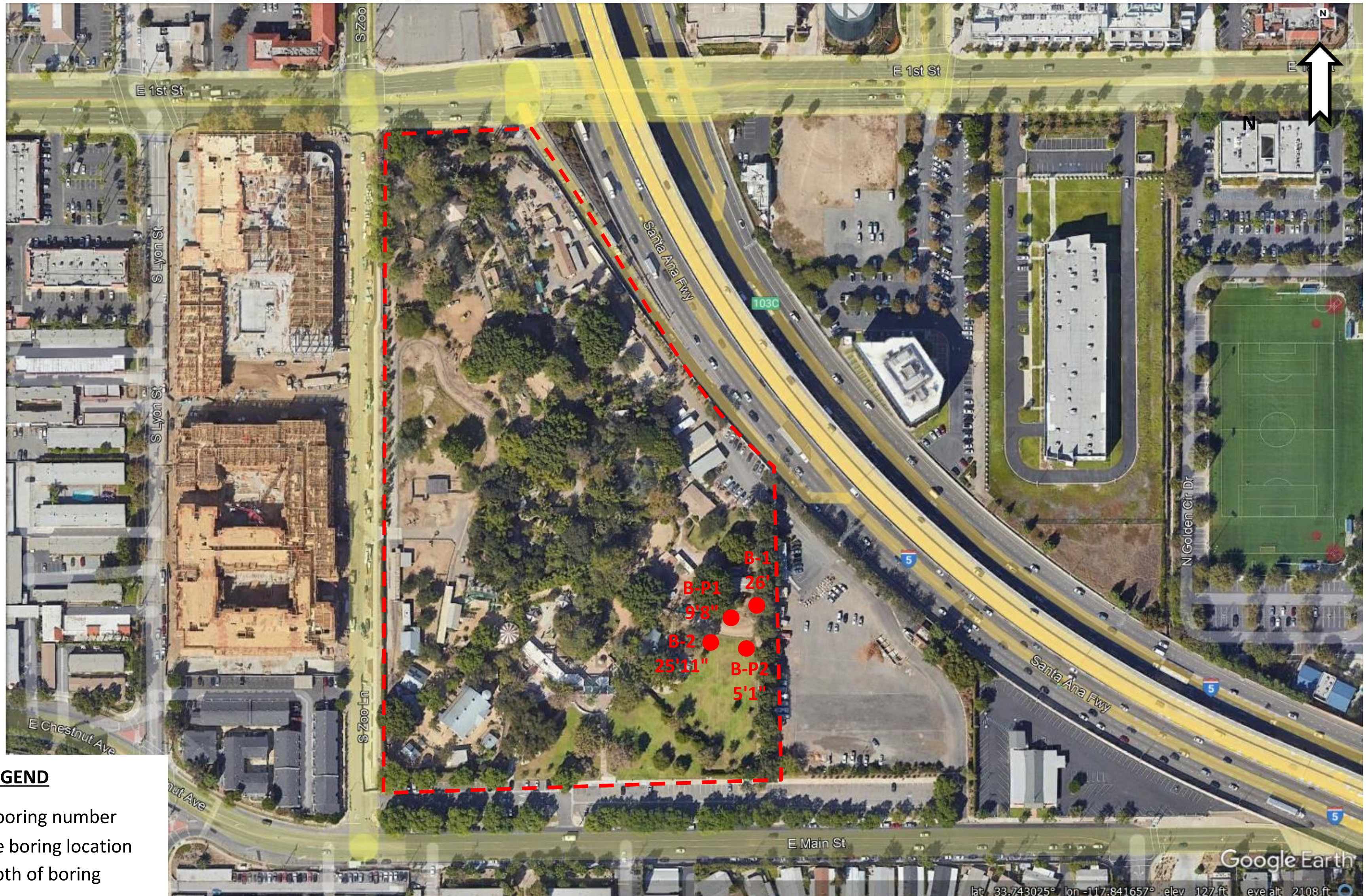
On December 1, 2023, field explorations were performed by drilling two (2) test and two (2) percolation borings at the approximate locations indicated on the attached Boring and Percolation Test Location Plan, Plate A. The exploratory borings were drilled by Alroy Drilling Services utilizing a truck mounted CME75, rotary drilling rig equipped with 8-inch diameter continuous flight, hollow-stem rotary augers. The borings extended to depths ranging from 5 feet 1 inches 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.

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 / Field Logs of Percolation Boring ("B" Plates).

The subsurface soils descriptions presented in the Field Logs of Boring / Field Logs of Percolation Boring 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 be on site during grading and foundation construction such that information/recommendations deciphered during preliminary geotechnical investigation phase could be verified and, if necessary, amended as appropriate.

Plate A	Boring Location Plan
Plates B-1 and B-2	Field Logs of Boring
Plates B-P1 and B-P2	Field Logs of Percolation Boring



LEGEND

B-1 Designated boring number

● Approximate boring location

26' Terminal depth of boring

Approximate Site limits

NOT TO SCALE

Associated Soils Engineering, Inc. **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	**Boring and Percolation Test Location Plan**	Project Name:	**Proposed Community Hub Building 1801 E. Chestnut Av., Santa Ana,CA**	
		Plate A	Proj. No.:	**7160.23**
			Date:	**January, 2024**

ASSOCIATED SOILS ENGINEERING, INC. Consulting Geotechnical Engineers

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-1

Project: **Proposed Community Hub Building**

Location: **1801 E. Chestnut Av., Santa Ana,CA** Project No. **7160.23**

Date(s) Drilled:	**12/1/2023**	Logged By:	**Ted Riddell**
Drilled By:	**Alroy Drilling Services**	Total Depth:	**26 feet**
Rig Make/Model:	**CME75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./±30In.**
Hole Diameter:	**8 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (FT)	ELEVATION (MSL)	SAMPLE INTERVALS: Bulk	SAMPLE INTERVALS: Drive	TYPE "N" (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS
0					SM	SILTY SAND: Medium brown, damp, fine-grained sand			MAX DENSITY, REMOLD SHEAR, EXPANSION, CORROSIVITY
3				41 (Ring)	SM-ML	SILTY SAND TO SANDY SILT: Medium brown, damp, dense, very fine to fine-grained sand, sinkhole pores	107.8	11.5	SHEAR
5				82 (Ring)		same as above	112.1	4.2	SHEAR
7				80 (Ring)	SM	SILTY SAND: Medium brown, damp, dense, fine-grained sand	102.0	7.2	
10				61 (Ring)	SM	SILTY SAND: Medium brown to yellowish brown, dry to damp, dense, fine to medium-grained sand, some pebbles	110.5	2.5	
15				88/11"(Ring)	SM	SILTY SAND: Light medium brown, damp, dense, fine-grained sand	112.6	8.5	

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF BORING B-1

Project: **Proposed Community Hub Building**

Location: **1801 E. Chestnut Av., Santa Ana,CA** Project No. **7160.23**

DEPTH (FT)	ELEVATION (MSL)	SAMPLE INTERVALS Bulk	Drive	TYPE "N" (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS
20				50/5"(Ring)	SM/GP	SILTY SAND AND GRAVEL: Medium brown with gray gravel, dry to damp, dense, fine to medium-grained sand	108.3	3.1	
25				93/10"(Ring)	SM	SILTY SAND: Light medium brown, damp, dense to very dense, fine-grained sand	109.7	8.8	
30									
35									

DRILLING NOTE: Bottom of hole at 26 feet. No groundwater encountered.

FIELD LOG OF BORING B-2

ASSOCIATED SOILS ENGINEERING, INC.

Project:	**Proposed Community Hub Building**
Location:	**1801 E. Chestnut Av., Santa Ana,CA**
Project No.	**7160.23**

Date(s) Drilled:	**12/1/2023**	Logged By:	**Ted Riddell**
Drilled By:	**Alroy Drilling Services**	Total Depth:	**25 feet 11 inches**
Rig Make/Model:	**CME75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./±30In.**
Hole Diameter:	**8 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (FT)	ELEVATION (MSL)	SAMPLE INTERVALS: TYPE "N" (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS
0			SM	SILTY SAND: Medium brown, damp to moist, fine-grained sand			
3		43(Ring)	SM	SILTY SAND: Medium brown, damp to moist, dense, very fine to fine-grained sand	109.4	8.4	CONSOL
7		35(Ring)	SM	SILTY SAND: Medium brown, dampto moist, medium dense to dense, fine to medium-grained sand	107.1	9.3	
10		33(Ring)	SM	SILTY SAND: Medium brown, moist, medium-dense, fine to medium-grained sand, some pebbles, trace clay	114.9	9.7	
15		37/11"(Ring)	SM	SILTY SAND WITH CLAY: Medium brown, damp to moist, dense, fine-grained sand	120.0	8.6	

FIELD LOG OF BORING B-2

SOILS ENGINEERING, INC.

Project: **Proposed Community Hub Building**

Location: **1801 E. Chestnut Av., Santa Ana,CA**

Project No. **7160.23**

DEPTH (FT)	ELEVATION (MSL)	SAMPLE INTERVALS Bulk	Drive	TYPE "N" (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS
20				81/12"(Ring)	SM	SILTY SAND: Medium brown, Damp to moist, dense, fine-grained sand	115.0	8.4	
25				90/11"(Ring)	SM	SILTY SAND: Medium brown, damp to moist, dense, fine-grained sand, some clay	116.7	2.3	
30									
35									

DRILLING NOTE: Bottom of hole at 25 feet 11 inches. No groundwater encountered.

ASSOCIATED SOILS ENGINEERING, INC.

FIELD LOG OF PERCOLATION BORING B-P1

Sheet 1 of 1

Project:	**Proposed Community Hub Building**		
Location:	**1801 E. Chestnut Av., Santa Ana,CA**	Project No.	**7160.23**

Date(s) Drilled:	**12/1/2023**	Logged By:	**Ted Riddell**
Drilled By:	**Alroy Drilling Services**	Total Depth:	**9 feet 8 inches**
Rig Make/Model:	**CME75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./±30In.**
Hole Diameter:	**8 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (FT)	ELEVATION (MSL)	SAMPLE INTERVALS: Bulk	SAMPLE INTERVALS: Drive	SAMPLE INTERVALS: TYPE "N" (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS
0					SM	SILTY SAND: Medium-brown, damp, fine-grained sand			
3				45(Ring)	SM	SILTY SAND: Medium brown, damp, medium dense, fine-grained sand	106.0	8.5	
5									PERCOLATION
7				86(Ring)	SM	SILTY SAND: Medium brown, damp, fine-grained sand	112.3	5.4	
10									
15									

NOTE: 10 feet (5' solid and 5' slotted) PVC pipe place in boring with annular area backfilled with pea gravel to surface. Two (2.0) inches of pea gravel placed at bottom of pipe. Percolation test performed after overnight presoaking.

FIELD LOG OF PERCOLATION BORING B-P2

ASSOCIATED SOILS ENGINEERING, INC.

Project:	**Proposed Community Hub Building**
Location:	**1801 E. Chestnut Av., Santa Ana,CA**
Project No.	**7160.23**

Date(s) Drilled:	**12/1/2023**	Logged By:	**Ted Riddell**
Drilled By:	**Alroy Drilling Services**	Total Depth:	**5 feet 1 inch**
Rig Make/Model:	**CME75**	Hammer Type:	**Automatic**
Drilling Method:	**Hollow-Stem Auger**	Hammer Weight/Drop:	**140 Lb./±30In.**
Hole Diameter:	**8 inches**	Surface Elevation:	**N/A**

Comments: Groundwater not encountered. Backfill not determined.

DEPTH (FT)	ELEVATION (MSL)	SAMPLE INTERVALS: Bulk	Drive	TYPE "N" (Blows/ft.)	USCS	GEOTECHNICAL DESCRIPTION	DRY DENSITY (pcf)	MOISTURE CONTENT (%)	OTHER TESTS
0					SM	SILTY SAND: Medium brown, damp to moist, medium dense, fine-grained sand			PERCOLATION
3			■	30(Ring)		same as above	93.1	19.0	
5									
10									
15									

NOTE: 5 feet (5'0") slotted PVC pipe place in boring with annular area backfilled with pea gravel to surface. Two (2.0) inches of pea gravel placed at bottom of pipe. Percolation test performed after overnight presoaking.

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 / Field Logs of Percolation Borings (see attached "B" Plates).

- Consolidation and Direct Shear Tests

Consolidation (ASTM D 2435-11(2020) Test Method) and direct shear (ASTM D3080-23 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.

- Soil Corrosivity

Tests of soluble sulfate and chloride contents were performed in accordance with the latest edition of California Test Methods 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 the latest edition of ASTM G187-23 Test Method and California Test Method 643, respectively, to assess the degree of corrosivity of the subgrade soils with regard to ferrous metal piping. The test results are presented below.

Sample ID	Sulfate Content [1] (%)/ Exposure Category	Chloride Content [2] (ppm) / Exposure Category	Resistivity [3] (OHM-cm)/ Degree of Corrosivity	Ph-Value[3]
B-1 @ 1'-5'	0.002 / S0	14 / C0	1,953 / Corrosive	8.22

1. California Test Method 417 2. California Test Method 422 3. ASTM G187-23 Test Method 4. California Test Method 643

- Maximum Dry Density/Optimum Moisture Content Test

A maximum density test was conducted in accordance with ASTM D1557-12(2021) Test Method, 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'	126.0	10.5	SM-ML

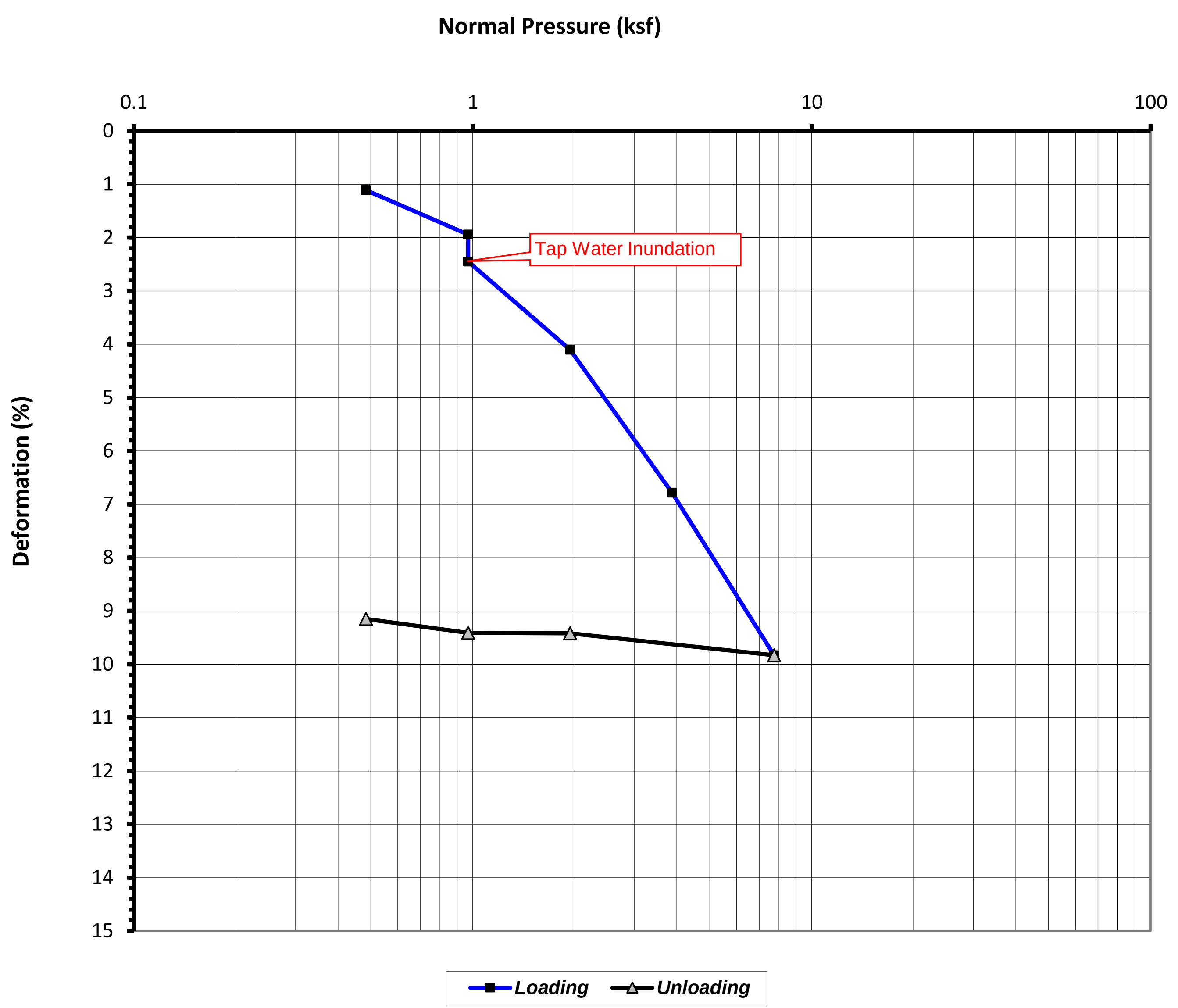
Laboratory Tests – continued

- 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 D4829-21 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'	108.9	9.6	49.0	31	Low

Plates C-1 through C-3	Uni-axial Consolidation Test Results
Plates D-1 through D-3	Direct Shear Test Results
Plates H-1 and H-2	Percolation Data Sheet



Sample Location	B-2 @ 3'	**Dry Density (pcf)**	109.4
		Moisture (%)	8.4
Sample Type	Ring		
Sample Description	Silty Very Fine Sand		
Remark	Undisturbed sample		

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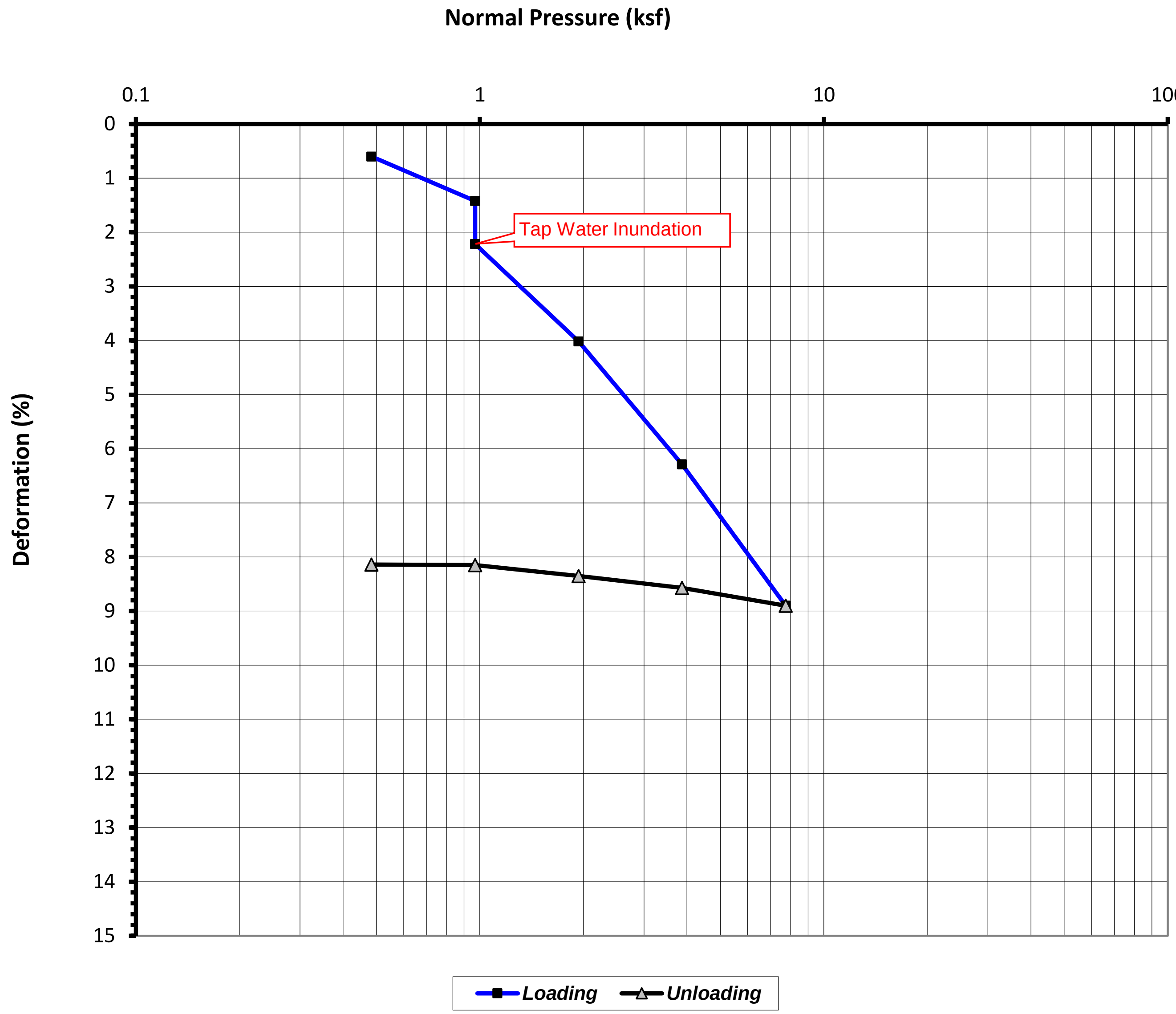
Tel (562) 426-7990 Fax (562) 426-1842

Project:	**Proposed Community Hub Building 1801 E. Chestnut Av., Santa Ana,CA**		
Plate	**Result of Uniaxial Consolidation/Swelling Test of On-Site Soil (ASTM D2435-11 Test Method)**		
C-1			
Project No.:	7160.23	**Date:**	January, 2024



Sample Location	B-2 @ 7'
Sample Type	Ring
Sample Description	Silty Sand
Remark	Undisturbed sample

Dry Density (pcf)	107.1
Moisture (%)	9.3

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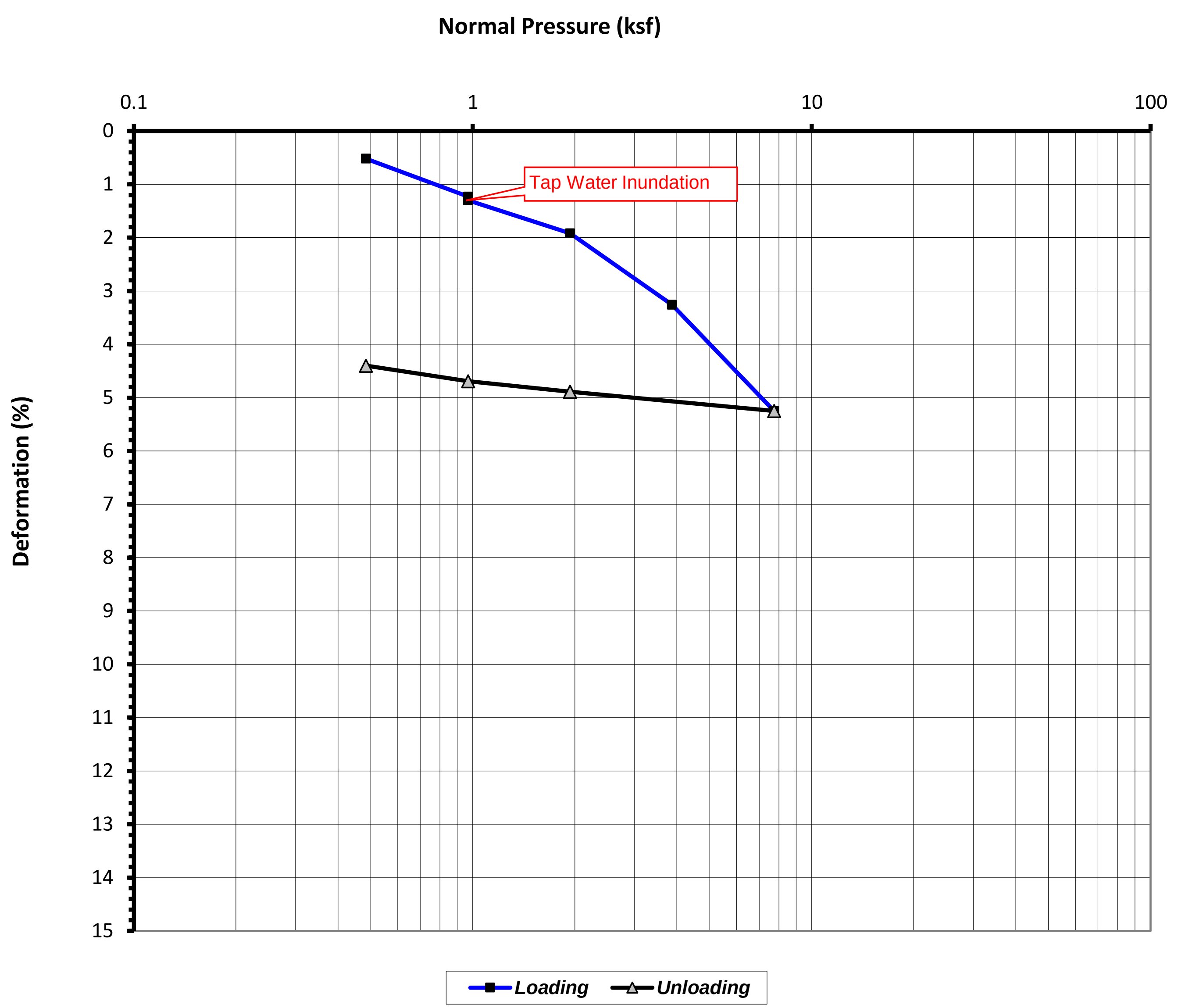
Project:	**Proposed Community Hub Building** **1801 E. Chestnut Av., Santa Ana,CA**		
Plate **C-2**	**Result of Uniaxial Consolidation/Swelling Test of On-Site Soil** **(ASTM D2435-11 Test Method)**		
Project No.:	7160.23	**Date:**	January, 2024



Sample Location	B-2 @ 10'	**Dry Density (pcf)**	114.9
		Moisture (%)	9.7
Sample Type	Ring		
Sample Description	Silty Sand, trace clay		
Remark	Undisturbed sample		

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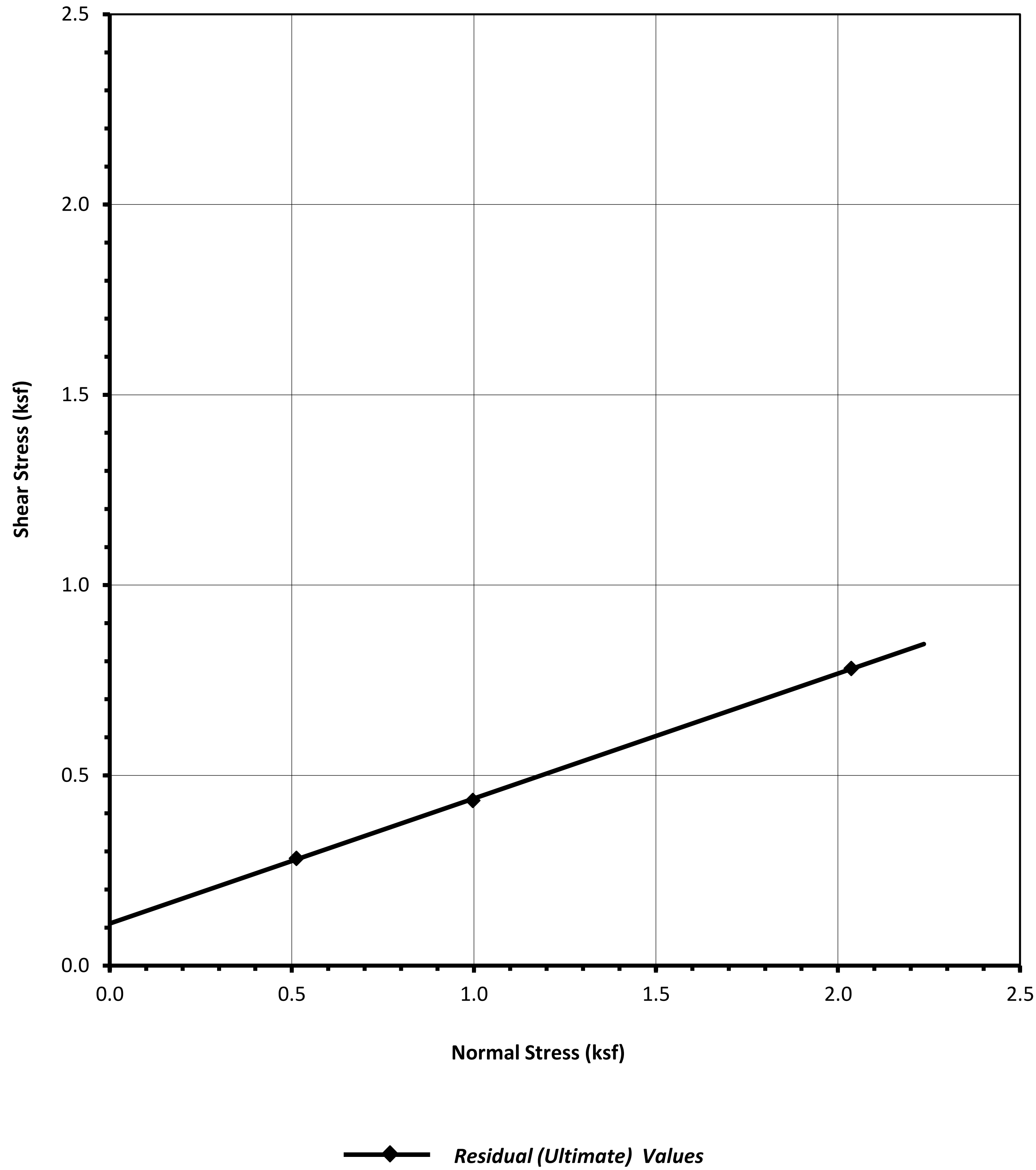
Project:	**Proposed Community Hub Building 1801 E. Chestnut Av., Santa Ana,CA**		
Plate	**Result of Uniaxial Consolidation/Swelling Test of On-Site Soil (ASTM D2435-11 Test Method)**		
C-3			
Project No.:	7160.23	**Date:**	January, 2024



Sample	*B-1 @ 3'*	Dry Density (pcf)	*107.8*
Description	*Silty Very Fine Sand to Sandy Silt*	Moisture Content (%)	*4.5*

ϕ-angle (degree)	*18.0*	Ultimate
Cohesion (ksf)	*0.110*	(Residual)

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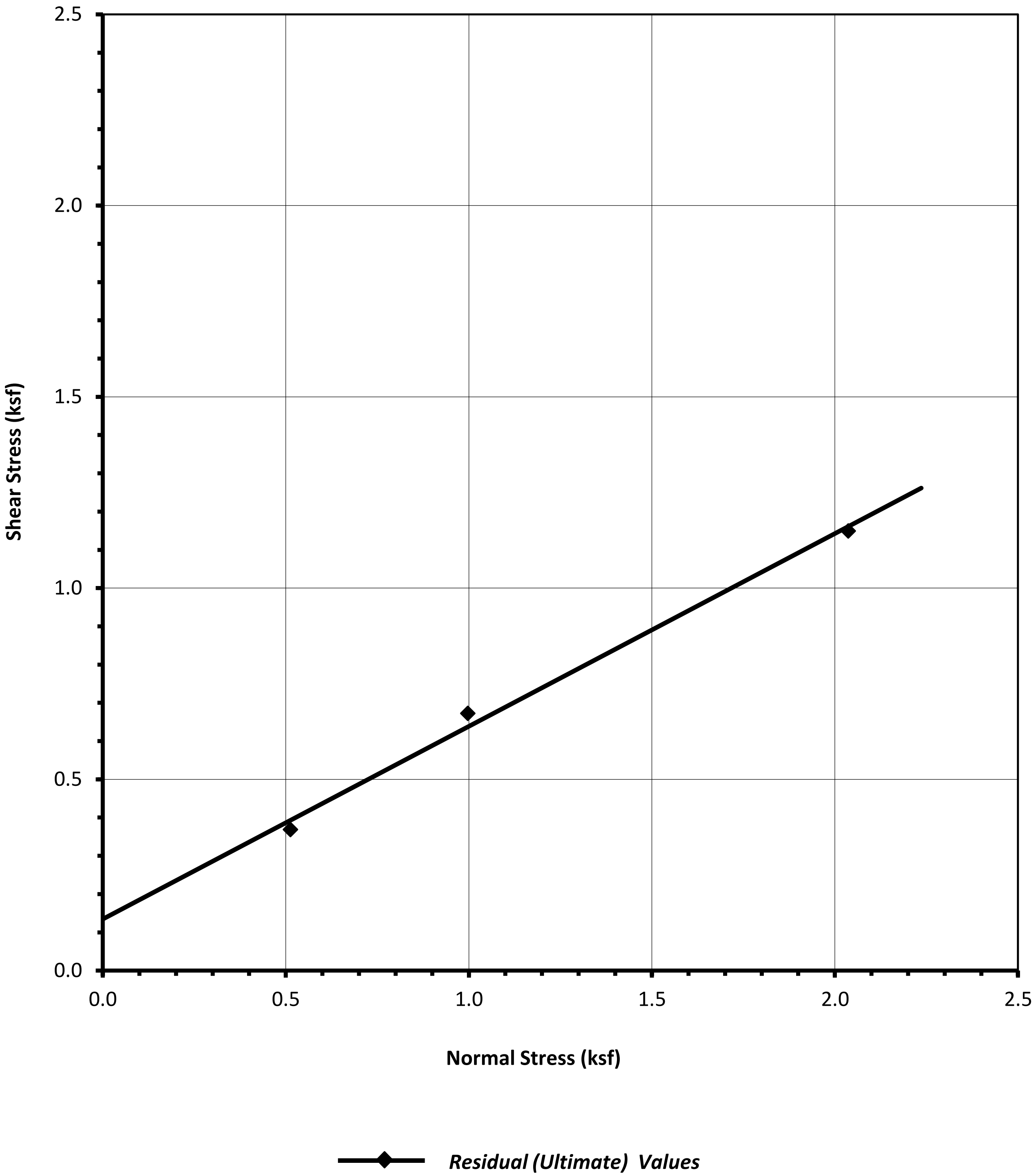
Project:	**Proposed Community Hub Building** **1801 E. Chestnut Av., Santa Ana,CA**		
Plate **D-1**	**Direct Shear Test Result** **(ASTM D 3080-11 Test Method)**		
Project No.:	**7160.23**	Date:	**January, 2024**



Sample	*B-1 @ 5'*	Dry Density (pcf)	*112.1*
Description	*Silty Very Fine Sand to Sandy Silt*	Moisture Content (%)	*4.2*

ϕ-angle (degree)	*26.5*	Ultimate (Residual)
Cohesion (ksf)	*0.130*	

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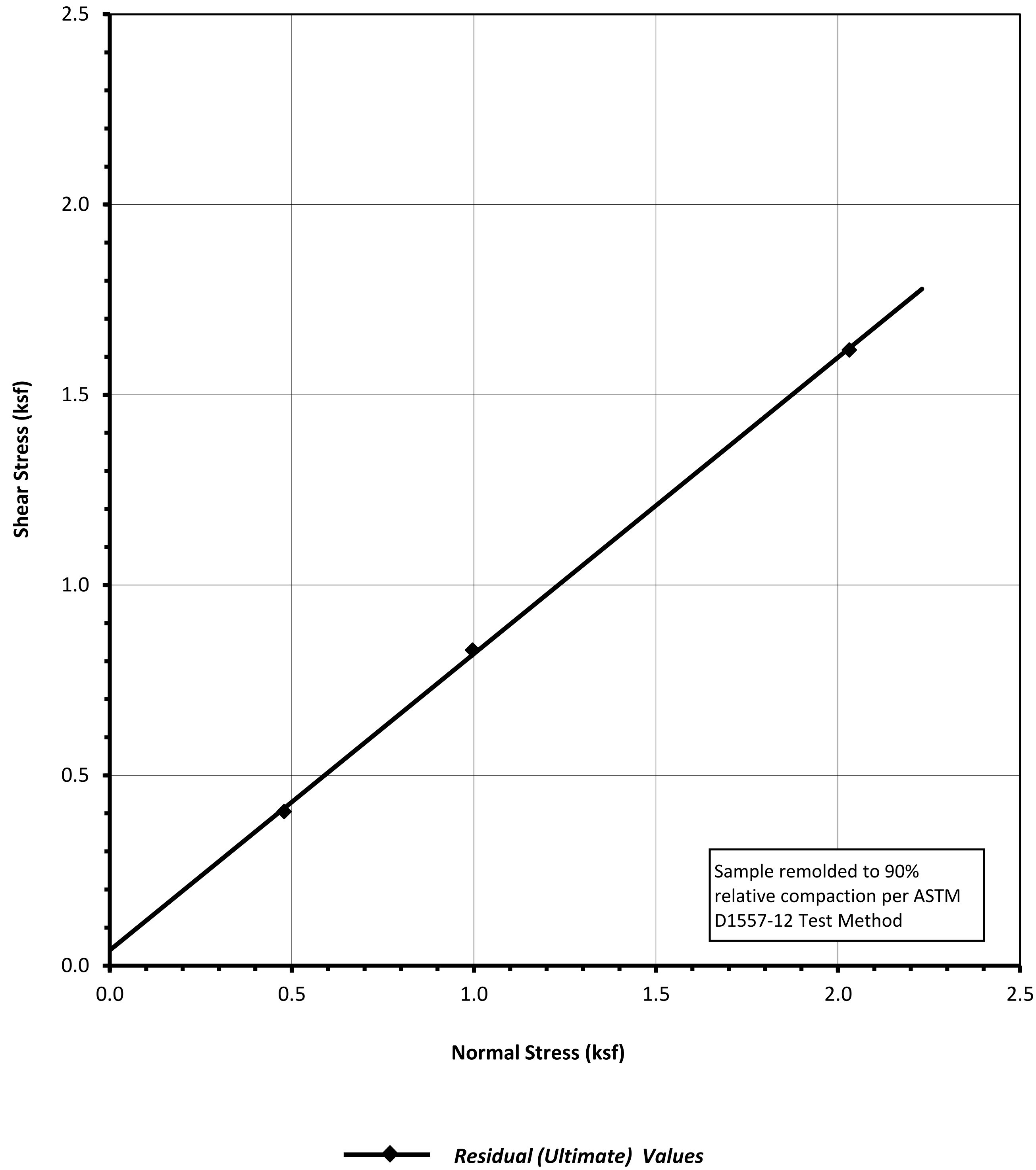
	Plate **D-2**	**Direct Shear Test Result** **(ASTM D 3080-11 Test Method)**		
	Project No.:	**7160.23**	Date:	**January, 2024**



Sample	*B-1 @ 0'-5'*	**Dry Density (pcf)**	*113.4*
Description	*Silty Fine Sand*	**Moisture Content (%)**	*10.5*

ϕ-angle (degree)	*37.5*	Ultimate
Cohesion (ksf)	*0.040*	(Residual)

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Project:	**Proposed Community Hub Building** **1801 E. Chestnut Av., Santa Ana,CA**		
Plate D-3	**Direct Shear Test Result** **(ASTM D 3080-11 Test Method)**		
Project No.:	**7160.23**	Date:	**January, 2024**

PERCOLATION DATA SHEET

Project: On-Site Storm Water Dispersal Job No.: 7160.23

Proposed Community Hub Building

1801 East Chestnut Avenue, Santa Ana, CA

Test Hole No.: B-P1 Date Excavated: 12/1/2023 Depth of Test Hole: 9' 8"

Soil Classification: Silty Fine Sand Presoak: √

Percolation Tested By: JC Date: 12/2/2023 Note: 2.0" Gravel at Bottom of Pipe

SANDY SOIL CRITERIA TEST

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)
1	7:22 7:28	6	-9.0	-18.0	9.0
2	7:30 7:34	4	-9.0	-15.0	6.0

USE ~~NORMAL~~ SANDY (CROSS ONE) SOIL CRITERIA

Time	Time Interval (Min.)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Percolation Rate (Min./Inches)
7:36 7:46	10	10	-9	-23.0	14.0	0.714
7:47 7:57	10	20	-9	-24.0	15.0	0.667
7:58 8:08	10	30	-9	-27.0	18.0	0.556
8:09 8:19	10	40	-9	-24.0	15.0	0.667
8:20 8:30	10	50	-9	-21.25	12.25	0.816
8:31 8:41	10	60	-9	-23.5	14.5	0.690
8:42 8:52	10	10	-9	-23.0	14.0	0.714

PLATE H-1

PERCOLATION DATA SHEET

Project: On-Site Storm Water Dispersal Job No.: 7160.23

Proposed Community Hub Building

1801 East Chestnut Avenue, Santa Ana, CA

Test Hole No.: B-P2 Date Excavated: 12/1/2023 Depth of Test Hole: 5' 1"

Soil Classification: Silty Sand Presoak: √

Percolation Tested By: JC Date: 12/2/2023 Note: 2.0" Gravel at Bottom of Pipe

SANDY SOIL CRITERIA TEST

Trial No.	Time	Time Interval (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)
1	7:40 7:59	19	-4.0	-10.0	6.0
2	8:00 8:22	22	-4.0	-10.0	6.0

USE ~~NORMAL~~ SANDY (CROSS ONE) SOIL CRITERIA

Time	Time Interval (Min.)	Total Elapsed Time (Min.)	Initial Water Level (Inches)	Final Water Level (Inches)	Δ In Water Level (Inches)	Percolation Rate (Min./Inches)
8:28 8:38	10	10	-4.0	-6.5	2.5	4.000
8:39 8:49	10	20	-4.0	-6.25	2.25	4.444
8:50 9:00	10	30	-4.0	-5.75	1.75	5.714
9:01 9:11	10	40	-4.0	-5.5	1.5	6.667
9:12 9:22	10	50	-4.0	-5.5	1.5	6.667
9:23 9:33	10	60	-4.0	-5.75	1.75	5.714

PLATE H-2

APPENDIX B - SITE FAULTING AND SEISMIC HAZARD DATA

Plates I-1 and I-2	EQFAULT – Deterministic Estimation of Peak Acceleration from Digitized Faults within 100 km-radius from the Site

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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: 7160.23

DATE: 01-23-2024

JOB NAME: Proposed Community Hub Building

CALCULATION NAME: SantaAnaZoo

FAULT-DATA-FILE NAME: C:\Program Files (x86)\EQFAULT1\CGSFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 33.7429
SITE LONGITUDE: 117.8417

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 (x86)\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.
SAN JOAQUIN HILLS	3.9(6.3)	6.6	0.457	X
NEWPORT-INGLEWOOD (L.A.Basin)	9.6(15.5)	7.1	0.257	IX
NEWPORT-INGLEWOOD (Offshore)	11.3(18.2)	7.1	0.230	IX
WHITTIER	11.6(18.7)	6.8	0.197	VIII
PUENTE HILLS BLIND THRUST	13.2(21.2)	7.1	0.264	IX
ELSINORE (GLEN IVY)	14.0(22.6)	6.8	0.168	VIII
CHINO-CENTRAL AVE. (Elsinore)	14.7(23.6)	6.7	0.197	VIII
SAN JOSE	20.6(33.1)	6.4	0.114	VII
PALOS VERDES	20.7(33.3)	7.3	0.155	VIII
SIERRA MADRE	26.8(43.2)	7.2	0.145	VIII
UPPER ELYSIAN PARK BLIND THRUST	27.0(43.4)	6.4	0.083	VII
CUCAMONGA	27.2(43.8)	6.9	0.118	VII
ELSINORE (TEMECULA)	29.2(47.0)	6.8	0.079	VII
RAYMOND	30.0(48.2)	6.5	0.079	VII
CLAMSHELL-SAWPIT	31.4(50.5)	6.5	0.075	VII
VERDUGO	32.2(51.9)	6.9	0.097	VII
CORONADO BANK	33.1(53.3)	7.6	0.116	VII
HOLLYWOOD	34.2(55.1)	6.4	0.061	VI
SAN JACINTO-SAN BERNARDINO	38.0(61.2)	6.7	0.053	VI
SAN JACINTO-SAN JACINTO VALLEY	39.8(64.0)	6.9	0.058	VI
SANTA MONICA	39.8(64.1)	6.6	0.059	VI
SAN ANDREAS - SB-Coach. M-1b-2	42.5(68.4)	7.7	0.094	VII
SAN ANDREAS - Whole M-1a	42.5(68.4)	8.0	0.114	VII
SAN ANDREAS - SB-Coach. M-2b	42.5(68.4)	7.7	0.094	VII
SAN ANDREAS - San Bernardino M-1	42.5(68.4)	7.5	0.082	VII
SAN ANDREAS - 1857 Rupture M-2a	42.6(68.6)	7.8	0.100	VII
SAN ANDREAS - Mojave M-1c-3	42.6(68.6)	7.4	0.077	VII
SAN ANDREAS - Cho-Moj M-1b-1	42.6(68.6)	7.8	0.100	VII
MALIBU COAST	44.4(71.5)	6.7	0.055	VI
CLEGHORN	44.9(72.2)	6.5	0.036	V
SIERRA MADRE (San Fernando)	45.2(72.7)	6.7	0.054	VI
SAN GABRIEL	47.0(75.7)	7.2	0.059	VI
NORTHRIDGE (E. Oak Ridge)	48.1(77.4)	7.0	0.063	VI
ROSE CANYON	49.2(79.1)	7.2	0.056	VI
NORTH FRONTAL FAULT ZONE (West)	51.0(82.1)	7.2	0.068	VI
ANACAPA-DUME	51.8(83.3)	7.5	0.083	VII
SAN JACINTO-ANZA	53.1(85.5)	7.2	0.050	VI
ELSINORE (JULIAN)	53.9(86.8)	7.1	0.046	VI
SANTA SUSANA	55.0(88.5)	6.7	0.041	V
HOLSER	60.7(97.7)	6.5	0.031	V

-END OF SEARCH- 40 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN JOAQUIN HILLS FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 3.9 MILES (6.3 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.4572 g

APPENDIX C - LIST OF REFERENCES

1. Blake, T.F., 2000, EQFAULT, A Computer Program for the Deterministic Predication of Peak Horizontal Acceleration from Digitized California Faults.

2. Guidelines for Evaluating and Mitigating Seismic Hazards in California, 2008, Special Publication 117A, California Geological Survey.

3. California Geological Survey, 2016, Seismic Hazard Zones Official Map, Santa Ana Quadrangle.

4. California Geological Survey, 1998, Seismic Hazard Zone Report 012, Seismic Hazard Zone Report for the Santa Ana 7.5-Minute Quadrangle, Orange County, California.

5. California Building Code, 2022 Edition: Sacramento, CA, California Building Standards Commission, 2 Volumes.

6. Legg, M.R., Borrero, J.C., and Synolakis, C.E., 2003, Evaluation of Tsunami Risk to Southern California Coastal Cities, The 2002 NEHRP Professional Fellowship Report, PF 2002-11, Earthquake Engineering Research Institute.

7. Soil Mechanics Design Manual 7.1 (NAVFAC DM-7.1), 1982, Department of the Navy, Naval Facilities Engineering Command, p. 347.

8. Foundation and Earth Structures Design Manual 7.2 (NAVFAC DM-7.2), 1982, Department of the Navy, Naval Facilities Engineering Command.

9. Stewart, J.P., Whang, D.H., Moyneur, M., and Duku, P., 2004, Seismic Compression of As-Compacted Fill Soils with Variable Level of Fines Content and Fines Plasticity, CUREE Publication No. EDA-05, 101p.

10. Tokimatsu, A.M. and Seed, H.B., 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, Vol. 113, No. 8, p. 861-878.

11. 2008 Interactive Deaggregations, Hazards Program, United States Geological Survey, website: http:/eqint.cr.usgs.gov/deaggint/2008/dndez.php.

12. Krinitzsky, Ellis L., Gould, James P. and Edinger, Peter H., 1993, Fundamentals of Earthquake Resistant Construction, John Wiley & Sons, Inc.

13. Minimum Design Loads for Buildings and Other Structures, 2016, American Society of Civil Engineers, ASCE Standard 7-16.

14. Building Code Requirements for Structural Concrete (ACI 318-19) and Commentary, 2019, published by American Concrete Institute, Farmington Hill, MI, 623p.

15. Civil Engineering Pavements, Design Manual 5.4 (DM 5.4), 1979, Department of the Navy, Naval Facilities Engineering Command, 60 pp.

16. California Geological Survey, Revised 1994, Fault Rupture Hazard Zones in California, Special Publication 42.

APPENDIX C - LIST OF REFERENCES - continued

17. Federal Emergency Management Agency, 2009, National Flood Insurance Program, Flood Insurance Rate Map, Los Angeles County, California and Incorporated Areas, Map Number 06059C0277J, effective date December 3.

18. Youd, T.L., and Idriss, I.M., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geo-Environmental Engineering, ASCE, Vol. 127, No. 4, p. 297-313.

19. Recommended Procedures for Implementation of DMG Special Publication 117: Guidelines for Analyzing and Mitigating Liquefaction in California, 1999, Southern California Earthquake Center, University of Southern California.

20. Seed, R.B., Cetin, K.O., Moss, R.E.S., Kammerer, A.M., Wu, J., Pestana, J.M., Riemer, M.F., Sancio, R.B., Kayen, R.E., and Faris, A., 2003, Recent Advances in Soils Liquefaction Engineering: A Unified and Consistent Framework, Keynote Presentation, 26th Annual ASCE Los Angeles Geotechnical Spring Seminar.

21. Corrosion Guidelines, Version 3.2, May 2021, Published by California Department of Transportation (Caltrans), 44p.

22. Bridge Design Specifications, September 2003, Published by California Department of Transportation (Caltrans).

23. Cetin, K.O., Youd, L.T., Seed, R.B., Bray, J.D., Stewart, J.P., Durgunoglu, T.H., Lettis, W., and Yilmaz, T.M., 2004, Liquefaction-induced Lateral Spreading at Izmit Bay During the Kocaeli (Izmit)-Turkey Earthquake, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 12, p. 1300-1313.

24. Settlement Analysis, 1994, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 9, published by American Society of Civil Engineers, New York, NY, p. 136.

25. Winterkorn, H.F., ad Fag, H.Y., 1976, Foundation Engineering Handbook: New York, NY, Van Nostrand Reinhold, 751p.

26. Cao, T., Bryant, W.A. Rowshandel, B., Branum, D., and Wills, C.J., 2003, the Revised 2002 California Probabilistic Seismic Hazard Maps, California Geological Survey.

27. Peterson, M.D., Bryant, W.A., Cramer, C.H., Cao, T., Reichle, M., Frankel, A.D., Lienkaemper, J.J., McCrory, P.A., and Schwartz, D.P., 1996, Probabilistic Seismic Hazard Assessment for the State of California, California Department of Conservation, Division of Mines and Geology, Open-File Report 96-706.

28. Wood, N.J., Ratliff, J., and Peters, J., Community Exposure to Tsunami Hazards in California, 2013, Scientific Investigations Report 2012-5222, published by U.S. Geological Survey.