ASSOCIATED SOILS ENGINEERING, INC.
Consulting Geotechnical Engineers

REPORT OF GEOTECHNICAL INVESTIGATION

Proposed Corona South Mall Improvements

725 South Main Street, City of Corona, California

Prepared For:

IDS Group Inc.

1 Peters Canyon Road, Suite 130
Irvine, California 92602

Project No. 7232.25.1 **August 12, 2025**

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

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Consulting Geotechnical Engineers

August 12, 2025
Project No. 7232.25.1

IDS Group, Inc.
1 Peters Canyon Road, Suite 130
Irvine, California 92602

Attention: Mr. Silman Ruiz,
Assistant Project Manager

Subject: **REPORT OF GEOTECHNICAL INVESTIGATION**
Proposed Corona South Mall Improvements
725 South Main Street, City of Corona, California

Ladies and Gentlemen:

Presented herewith is the Report of Geotechnical Investigation (the Soils Report) prepared by Associated Soils Engineering, Inc. (ASE) as part of the proposed renovations and additions (the Remodel/Addition) to the South Corona Mall, with the subject building located at 725 South Main Street, in the City of Corona, California (the Site). This work was conducted in accordance with ASE's Proposal No. P25-041, dated April 2, 2025, which subsequently received your authorization.

The subject geotechnical investigation was planned and performed based on the relevant project information provided by your office, including preliminary plans prepared by KES Technologies, Inc.

The purpose of this investigation was to explore the subsurface soils conditions at the Site, followed by assessment of site geologic/seismic hazards, performance of engineering analyses, and formulation/assembly of recommendations for the geotechnical design and construction pertinent to the Remodel/Addition. ASE's study has concluded that construction of the Remodel/Addition 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, and 3) the results of laboratory tests performed.

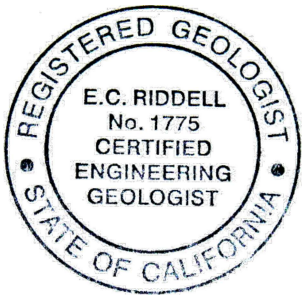
We at ASE appreciate the opportunity to provide our professional services on this important project and look forward to assisting you during construction phase of the Remodel/Addition.

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

Respectfully submitted,
ASSOCIATED SOILS ENGINEERING, INC.

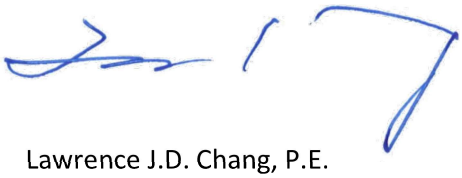






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1.0 INTRODUCTION

This Soils Report presents the results of ASE's geotechnical investigation for the proposed renovations, improvements and additions (the Remodel/Addition) to the existing building in the southwest corner of the Corona South Mall, located at 725 South Main Street, in the City of Corona, California (the Site). The approximate location of the Site is shown on Figure 1, Site Location Map. The purpose of this investigation was to explore the subsurface soils conditions at the Site, to assess potential site geologic/seismic hazards, to perform geotechnical engineering analyses, and to formulate/assemble geotechnical recommendations for subgrade preparation and foundation design and construction pertinent to the Remodel/Addition. This Soils Report presents the summary of collected field and laboratory testing data, the results of ASE's assessments of potential geological/seismic hazards and engineering evaluations/analyses, and the geotechnical conclusions and recommendations for site grading and foundation design and construction.

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:

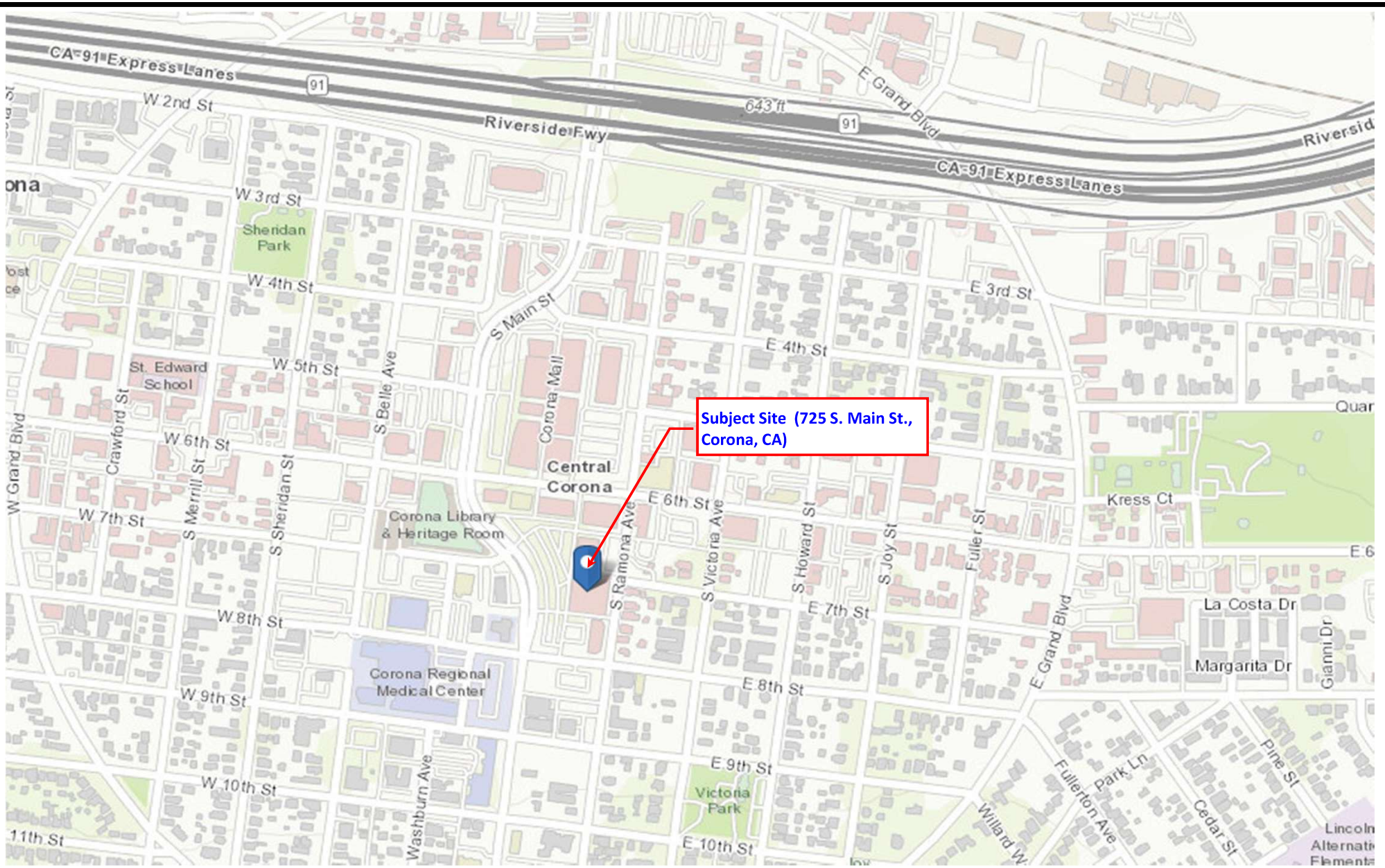
Based on the provided information, ASE understands that the Remodel/Addition includes possible additions to, remodeling of, and retrofitting, the existing building. Appurtenant construction is assumed to consist of landscaping, walls, fencing, hardscaping, and utilities.

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 new or renovated structure will have a maximum concentrated column load (D+L) on the order of 80 kips when supported by isolated pad footings or have a maximum line load (D+L) not exceeding 4,000 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 15 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.



Associated Soils Engineering, Inc. **2860 Walnut Avenue** **Signal Hill, CA 90755** **Tel (562) 426-7990 Fax (562) 426-1842**	**Site Location Map**	Proj. Name:	**Prop. Corona South Mall Improvements** **725 South Main Street, Corona, CA**
		Figure 1	Proj. No.: **7232.25.1**
			Date: **August, 2025**

1.2 Scope of Investigation

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

A. Review of readily available background information, including in-house geotechnical data, 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 of Southern California (the DigAlert) prior to field investigation.

C. Field exploration consisting of drilling three (3) exploratory borings (Borings B-1 through B-3) to a maximum depth of 26.5 feet below respective existing grades. ASE staff logged and sampled representative soils encountered in each exploratory boring. Locations of the exploratory borings on site are shown on the Boring Location Plan, Plate A, in Appendix A.

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

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

- Evaluation of general subsurface conditions and description of types, distribution, and engineering characteristics of subsurface materials.
- Assessment 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 provisions in Chapters 16 and 18 of the California Building Code, 2022 Edition (2022 CBC; Reference 5) and seismic retrofitting stipulations in ASCE 41-17 (Reference 27).
- Recommendations of subgrade preparation for the Remodel/Addition and other ancillary improvements together with evaluation of the suitability of on-site soils for foundation support and establishment of qualification criteria for on-site or imported fill material.
- Recommendations of suitable foundation systems consisting of conventional 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.
- Recommendations for temporary excavation, shoring and trenching.
- Evaluation of the corrosion and expansion potential of the on-site materials.

F. 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 Remodel/Addition.

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, Surface Conditions and Existing Development

The Remodel/Addition is to be located on the existing building in the southeast corner of the mall located at 725 South Main Street, in the City of Corona, California. The building is 33,826 square feet and proposed to remain to be remodeled/retrofitted.

The Site, which is bound by South Main Street to the West, and Ramona Ave to the East. To the north is another building and on the South there is a parking lot with a Starbucks on the western side and another building with multiple stores on the east side. Immediately west of the building exists a parking lot before South Main Street.

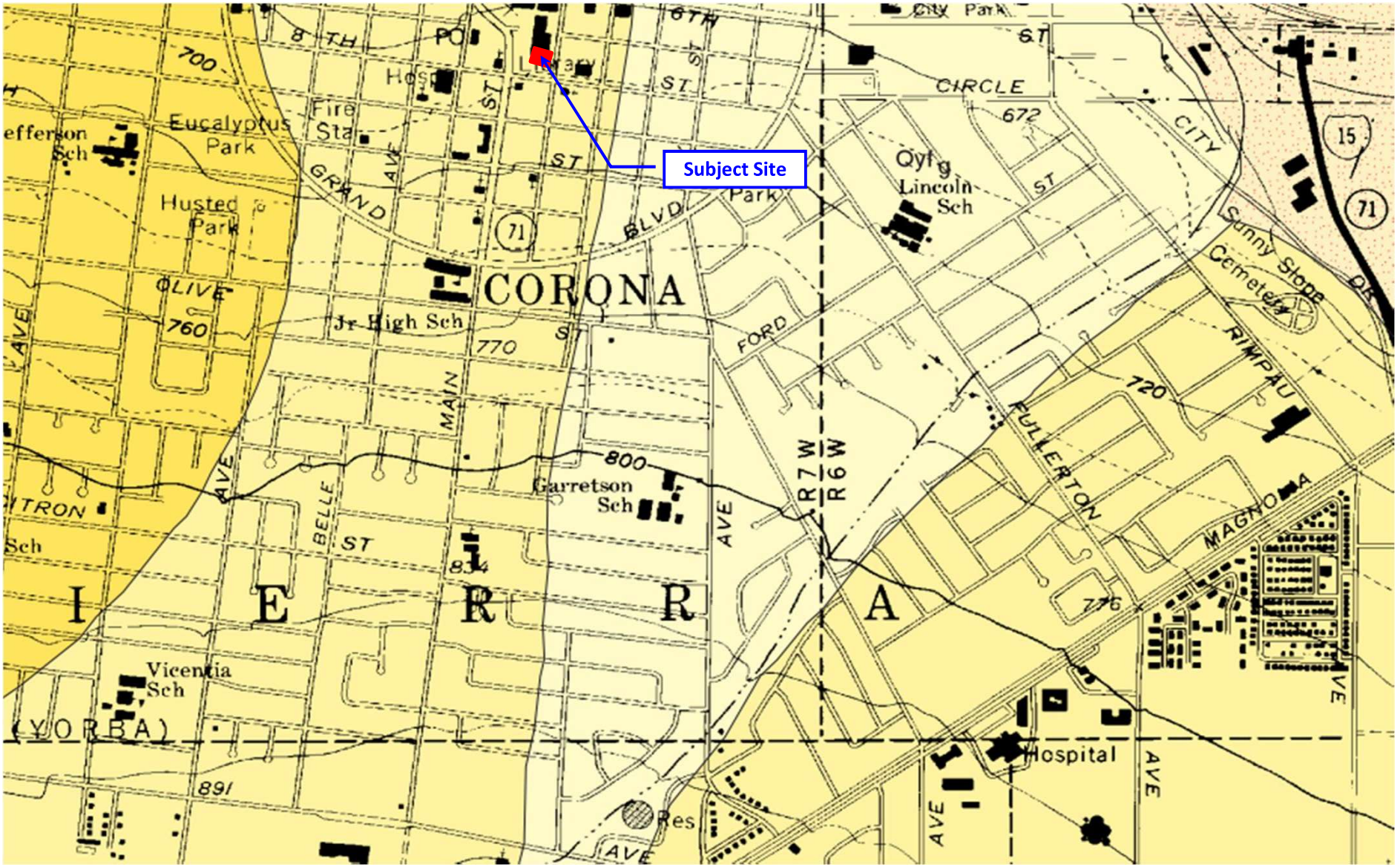
2.2 Subsurface Conditions

2.2.1 Artificial Fill (af):

Artificial fill was not observed in any of ASE's exploratory borings but may be present at other areas of the Site, or could be encountered during site grading. Artificial fill 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 Young Alluvial Fan Deposits, Unit 1 (Qyf1):

Native site soils consisting of Holocene- to late Pleistocene-age young alluvial fan deposits, Unit 1 were encountered in each of exploratory borings to the maximum terminal explored depth of 26.5 feet. Per Reference 3, materials of the young alluvial fan deposits, Unit 1 consist of pale-gray, unconsolidated, cobble- to granule- sized gravel and are restricted to single fan bisected by younger Qyf fan emanating from Main Street and Eagle Canyons. In specific, the encountered on-site alluvial deposits predominantly consist of silty sands with varying amounts of rock, gravel and silty clay. Figure 2, Local Geologic Map, excerpted from Reference 3, shows geologic material distribution in the vicinity of the Site.



(Partial Extract of the Geologic Map of the Corona South 7.5' Qusdrangle, Riverside and Orange Counties, California, Version 1.0 - USGS Open File Report 02-21)

LEGEND

Approximate Site Location

Qyf Young alluvial fan deposits (Holocene and late Pleistocene)—Gray-hued sand and cobble- and gravel-sand deposits derived from lithic diverse sedimentary units.

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Project:	**Prop. Corona South Mall Improvements 725 South Main Street, Corona, CA**		
Figure 2	**Local Geologic Map**		
Proj. No.:	**7232.25.1**	Date:	**August, 2025**

Blow counts recorded from advancing Standard Penetration Test (SPT) sampler and Modified California barrel sampler empirically indicate that the granular, sandy strata of site native soils are in a medium dense to very dense condition, whereas the fine-grained, cohesive strata (i.e. silts/clays) exhibit stiff to very stiff consistencies. Site subsurface soils were, in general, in a damp to very 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) 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 within the explored depth of 26.5 feet in Borings B-1 through B-3. A search on Google Earth indicates that the subject Site is approximately 690 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 historic high groundwater elevation in groundwater monitoring well SB-1, Well Number T0606588396 (SHELL Lincoln Pomona at 111 North Lincoln Avenue, Corona, CA 92882), located approximately one mile northwest of the Site, was 96.5 feet below grade on February 25, 2003. The ground surface elevation at this well location (taken from Google Earth images) is approximately 225 ft AMSL. The most recent reading in this well (taken January 30, 2007) was 102.11 feet below well grade.

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 from our observations may occur, as ASE's exploratory borings were not installed for long-term groundwater monitoring purpose.

Caving and/or sloughing were not observed and/or measured during the drilling and sampling operations performed with manual equipment. However, caving and/or soil sloughing cannot be ruled out in excavations greater in dimension/depth than ASE's exploratory borings.

2.4 Utilities

No overhead or underground were disturbed during the course of ASE's site exploration. Underground lines are present which service the site structures and are along site bordering streets. Irrigation lines are present in landscaping areas. Other utilities, though not known at the time of this report preparation, may be present on site, and should be located and incorporated into site development plans accordingly.

3.0 FAULTING AND SEISMICITY

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

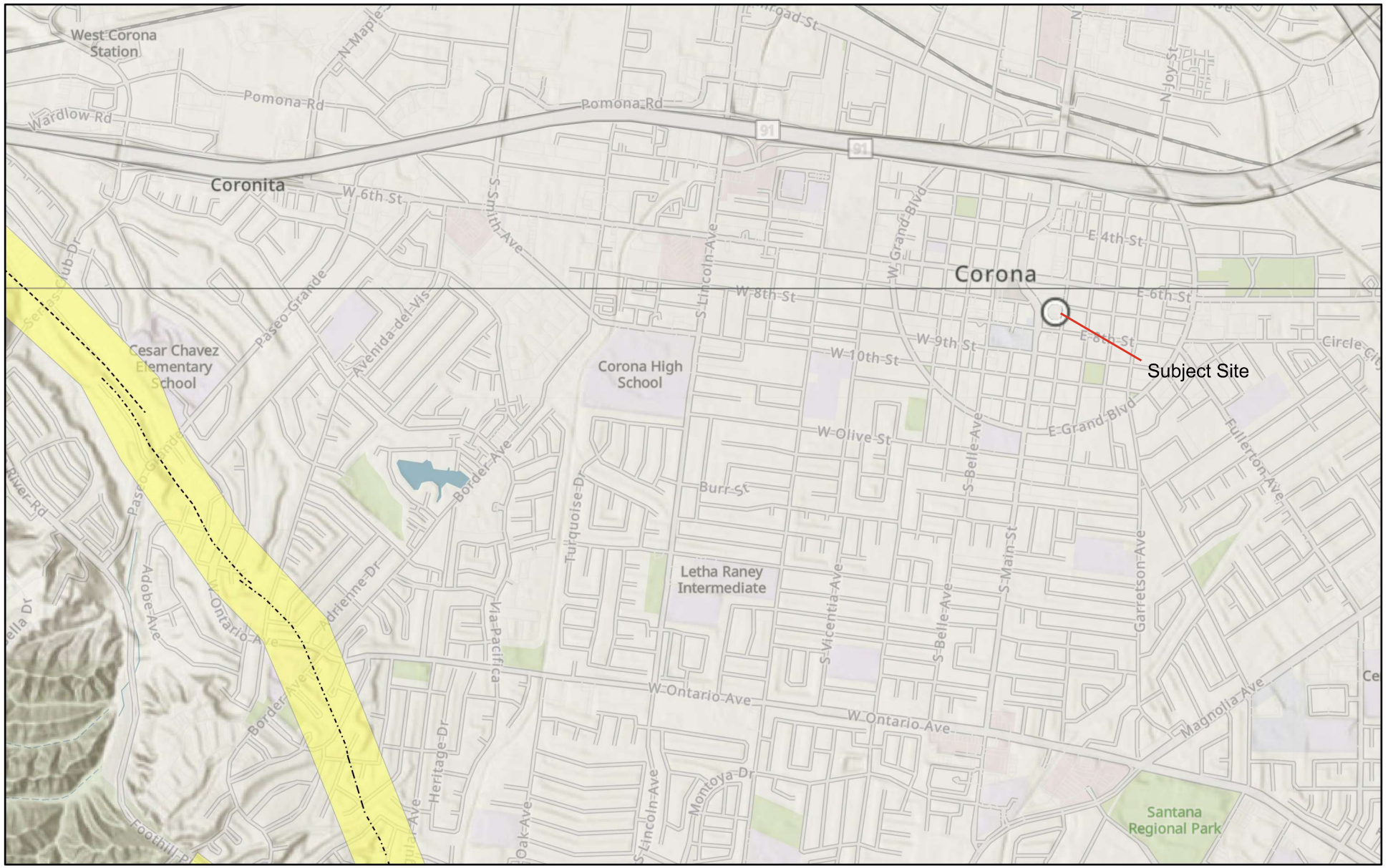
By the definition of CGS, an active fault is one which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). The CGS has defined a potentially active fault as any fault which has been active during the Quaternary Period (approximately the last 1,600,000 years). These definitions are used in delineating Earthquake Fault Zones as mandated by the Alquist-Priolo Geologic Hazard Zones Act of 1972 and as subsequently revised in 1997 as the Alquist-Priolo Earthquake Fault Zoning Act and Earthquake Fault Zones. The intent of the act is to require fault investigations on sites located within Special Studies Zones to preclude new construction of certain inhabited structures across the trace of active faults. The Site is not located within the Alquist-Priolo Earthquake Fault Zone and the Site similarly is not located within a seismic hazard zone per CGS's mapping, as shown in Figure 3, Local Fault Rupture Hazard Map.

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 Chino – Central Ave (Elsinore) Fault, approximately 2.0 miles (3.2 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 6.7 Mw (moment magnitude as per USGS) has been assessed along the Chino – Central Ave (Elsinore) Fault. As shown on Plate I-2 in Appendix B, estimated PGA resulting from a MPE event on the Chino – Central Ave (Elsinore) Fault is on the order of 0.582 should this event occur at the fault's closest approach to the Site. Other nearby active faults include the Elsinore (Glen Ivy) Fault and Whittier Fault, located approximately 3.7 miles (6.0 km) and 4.2 miles (6.8 km) away, respectively. In sum, 41 active or potentially active faults have been identified within 62 miles (100 km) of the Site.

Figure 3 Local Fault Rupture Hazard Map



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CGS Alquist Priolo Fault Traces

Approximately Located

Inferred

Concealed

CGS Alquist-Priolo Fault Zones

CGS SHZ Unevaluated Areas

1:30,585

0 0.2 0.4 0.8 mi

0 0.35 0.7 1.4 km

Esri, NASA, NGA, USGS, FEMA, Sources: Esri, TomTom, Garmin, FAO, NOAA, USGS, © OpenStreetMap contributors, and the GIS User Community,

ASE #7232.25.1

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 360 m/sec, corresponding with Site Class "C/D boundary", nearest to the inferred V_s^{30} value of 351.9 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 33.8740°	W 117.5658°	360.0	MPE [1]	0.5884	1.4017	0.6979
			MCE [2]	1.0119	2.3973	1.3718

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 and ASCE 41-17 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 according to the inferred Vs value discussed in Section 3.2 above and the Vs value-based criterion stipulated in Table 20.3-1 of ASCE 7-16 (Reference 13), a Risk Category II and a scenario of Risk-Targeted Maximum Considered Earthquake (MCE_R) that carries a 2% exceedance probability in 50 years had been determined utilizing the "ASCE Hazard Tool" web-application (https://ascehazardtool.org/) and the criteria stipulated in Chapters 11 and 12 of ASCE 7-16, including Supplements 1 and 3. Summaries of the seismic coefficients for the Site are tabulated on next page.

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 on next page 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 below do not reflect such 50% increase. If the structural design of the Remodel & Addition 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.

2022 CBC SEISMIC DESIGN PARAMETERS					
Site Latitude:	N 33.8740°	**Site Longitude:**	W 117.5658°	**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]				1.7	
0.2-Second Spectral Response Acceleration, S_S [e]				2.065g	
1.0-Second Spectral Response Acceleration, S_1 [f]				0.783g	
Adjusted 0.2-Second Spectral Response Acceleration, S_{MS} [g]				2.065g	
Adjusted 1.0-Second Spectral Response Acceleration, S_{M1} [h]				1.331g	
Design 0.2-Second Spectral Response Acceleration, S_{DS} [i]				1.376g	
Design 1.0-Second Spectral Response Acceleration, S_{D1} [j]				0.887g	
Long -Period Transition Period, T_L [k]				8 sec	
Mapped MCE_G Geometric Mean Peak Ground Acceleration, PGA [l]				0.866g	
Site Coefficient, F_{PGA} [m]				1.1	
MCE_G Peak Ground Acceleration adjusted for Site Class Effect, PGA_M [n]				0.953g	
Risk Category			**I or II or III**		**IV**
Seismic Design Category based on S_1 [o]			E		F
Seismic Design Category based on SD_S [p]			N/A		N/A
Seismic Design Category based on SD_1 [q]			N/A		N/A

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)

Additionally, the seismic design criteria for the Site for BSE-2N, BSE-1N, BSE-2E, and BSE-1E Hazard Levels, have been analyzed utilizing the "ASCE Hazard Tool" web-application (https://ascehazardtool.org/) per the criteria of Basic Performance Objective Equivalent to New Building Standards (BPON) and Basic

Performance Objective for Existing Buildings (BPOE) stipulated in ASCE 41-17 (Reference 27). Summaries of the relevant ASCE 41-17 seismic coefficients for the Site are tabulated below.

ASCE41-17 SEISMIC DESIGN PARAMETERS							
Site Latitude:	N 33.8740°	**Site Longitude:**	W 117.5658°	**Risk Category** [a]	II		
Site Class [b]	D						
Hazard Level:	BSE-2N	BSE-1N	BSE-2E	BSE-1E			
% Exceedance Probability in 50 Years:	2	10	5	20			
Seismic Parameter	**Recommended Value**						
Factored Uniform-Hazard Spectral Acceleration, SSUH	2.422	N/A	N/A	N/A			
Coefficient of Risk (0.2s), CRS [a]	0.915	N/A	N/A	N/A			
Probabilistic Risk-Targeted Ground Motion (0.2s) (g), SSRT	2.215	N/A	N/A	N/A			
Factored Deterministic Acceleration Value (0.2s) (g), SSD	2.065	N/A	N/A	N/A			
Factored Uniform-Hazard Spectral Acceleration, S1UH	0.866	N/A	N/A	N/A			
Coefficient of Risk (1.0s) CR1 [b]	0.904	N/A	N/A	N/A			
Probabilistic Risk-Targeted Ground Motion (1.0s) (g), S1RT	0.783	N/A	N/A	N/A			
Factored Deterministic Acceleration Value (1.0s) (g), S1D	0.809	N/A	N/A	N/A			
Site Coefficient, Fa [c]	1.000	N/A	1.000	1.152			
Site Coefficient, Fv [d]	1.700	N/A	1.700	2.023			
0.2-Second Spectral Response Acceleration, S_S [e]	2.065	N/A	1.747	0.870			
1.0-Second Spectral Response Acceleration, S_1 [f]	0.783	N/A	0.605	0.289			
Adjusted 0.2-Second Spectral Response Acceleration, S_{XS} [g]	2.065	1.376	1.747	1.002			
Adjusted 1.0-Second Spectral Response Acceleration, S_{X1} [h]	1.331	0.887	1.028	0.584			
Long -Period Transition Period, T_L [l]	8 sec						

a Per ASCE 7-16 Figure 22-18.
b Per ASCE 7-16 Figure 22-19.
c Per ASCE 7-16 Table 11-4.1.
d Per ASCE 7-16 Table 11-4.2.
e Per ASCE 7-16 Section11.4.2.
f Per ASCE 41-17 Equation 2-1 =F_aS_S
g Per ASCE 41-17 Equation 2-2 = F_vS_1
h Per ASCE 7-16 Figure 22-14.
g Per ASCE 41-17 Equation 2-2 = F_vS_1

Please note that conformance to the 2022 CBC and ASCE 41-17 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 strong seismic 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 nor a Fault Hazard Management Zone, as shown in Figure 3, Local Fault Rupture Hazard Map. 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 3a, Local Liquefaction Hazard Map, the Site has been mapped by the City of Corona to have a "Low" liquefaction susceptibility. 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 26.5 feet generally consist of dense to very dense granular, sandy soils and stiff to very stiff fine-grained, silty/clayey soils.

While no groundwater was encountered within the maximum explored depth of 26.5 feet in Borings B-1 through B-3 during ASE's site exploration, the historic high groundwater level in the Site vicinity has been shown to be greater than 50 feet deep, as per discussed in Section 2.3 above.

Considering that: 1), site subsurface soils consist predominantly of dense to very dense granular, silty sand with rock and gravel and trace silty clay to the maximum explored depth of 26.5 feet and likely beyond; 2) a PGA_M of 0.953g from 2022 CBC seismic design criteria; 3) the historic high groundwater level is greater than 50 feet deep; and 4) an earthquake magnitude of 7.1 Mw derived per EQFAULT software, the potential for the occurrence of seismically-induced liquefaction at the Site has been assessed to be very low, 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.

While the site soils encountered consist predominantly of medium dense to very dense silty sand with rock and gravel and stiff to very stiff silty clay that are considered non-liquefiable as per discussed in Section 4.2.1 above, settlement of on-site soils above groundwater level as a result of seismically-induced densification (i.e. "dry" seismic settlement) has been calculated to be 1.23 inches for the average soil profile encountered in Borings B-1 through B-3, as shown on Plate M-1,

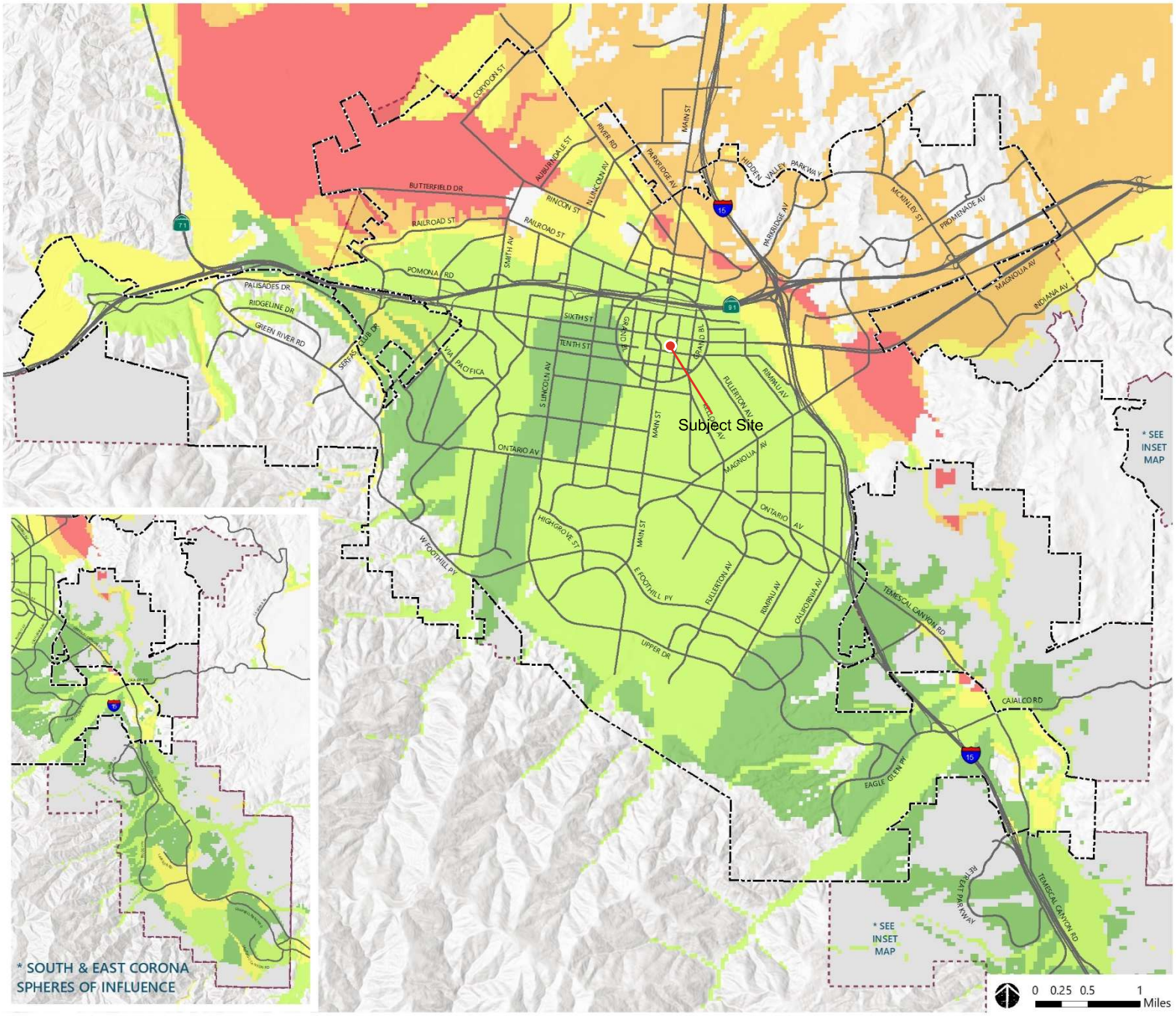


Figure 5-2
Liquefaction Hazards

Legend

Susceptibility

- Very High
- High
- Moderate
- Low
- Very low
- City Boundary
- Sphere of Influence Areas

Source:
County of Riverside, 2017

CORONA General Plan

ASE #7232.25.1

Figure 3a Local Liquefaction Hazard Map

Dynamic Settlement of Dry Soil, in Appendix B, in accordance with the procedures recommended by Krinitsky et.al. (Reference 12). With the implementation of relevant subgrade preparation measures discussed in the sub-sections of Section 5.2 below, the "dry" dynamic settlement is anticipated to be reduced to 0.71 inch, also shown on Plate M-1. Due to the generally uniform geologic nature of subgrade soils in Site vicinity, such magnitude of "dry" seismic settlement is anticipated to be relatively uniform across the Remodel/Addition footprint with minimum differential "dry" seismic settlement.

4.2.3 Earthquake-Induced Landslides:

There is no indication that recent landslides or unstable slope conditions exist on or adjacent to the project Site that would otherwise result in an obvious landslide hazard to the proposed Remodel/Addition or adjacent properties. Due to the lack of significant unretained relief on or adjacent to the Site, the potential for earthquake induced landslides in the future 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 liquefaction. It is typically exemplified by the formation of vertical cracks on the surface of liquefied soils, and usually takes place on gently sloping ground or level ground with nearby free surface such as drainage or stream channel. Since the potential for the occurrence of seismically-induced liquefaction at the Site has been assessed to be very low in Section 4.2.1 above, the potential for the occurrence of liquefaction-induced lateral spreading is also nil on 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.

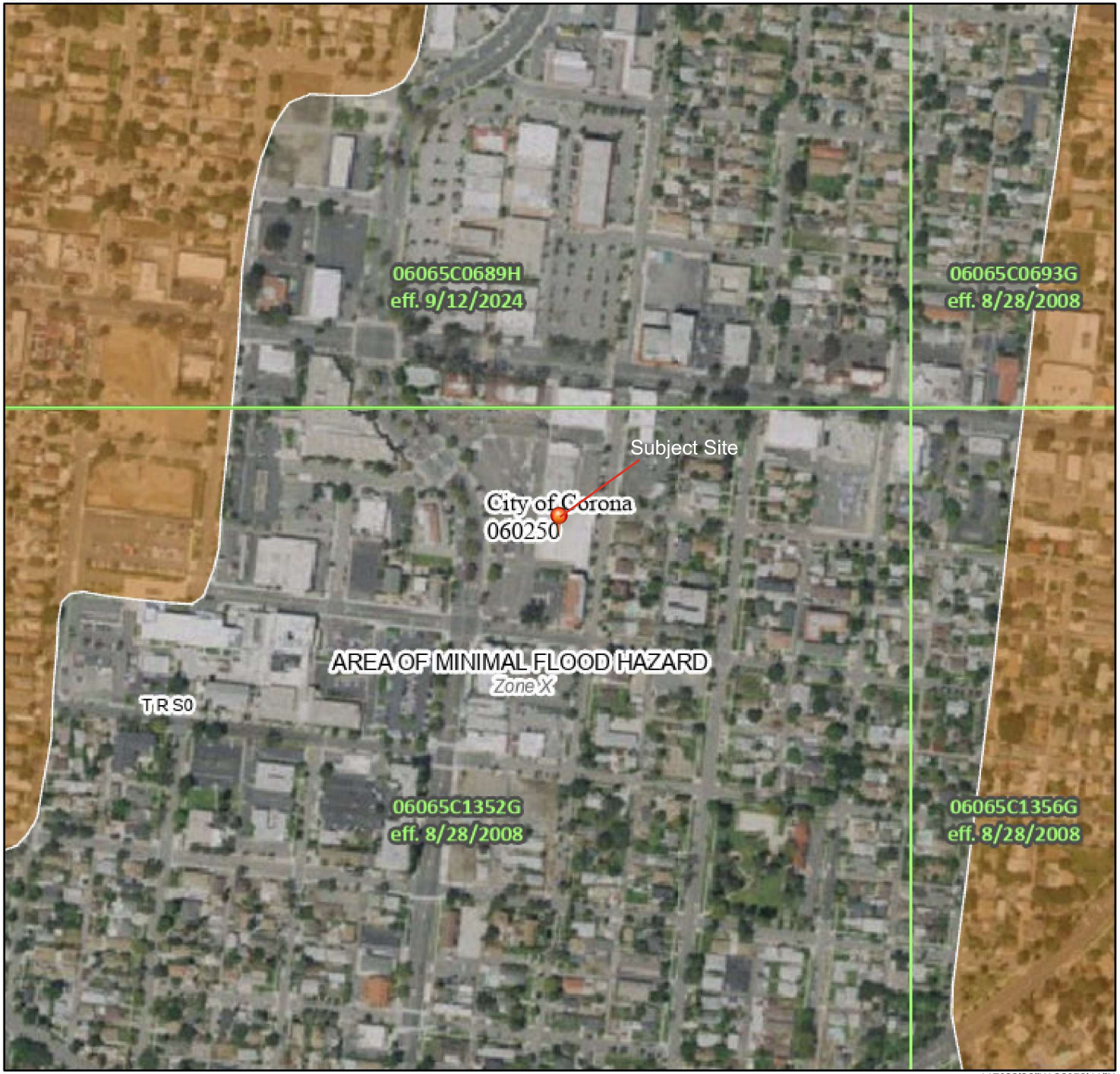
4.2.6 Flood Hazards:

Per 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. 0605C1352G, map revised August 28, 2008).

ASE #7232.25.1

National Flood Hazard Layer FIRMette-Figure 4

FEMA



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
- Coastal Transect
- 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 8/8/2025 at 11:34 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.

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 Remodel/Addition include the following:

1. Soil disturbances as a result of site demolition, clearing and excavation operations.
2. Localized presence of loose, low-density soils within the zone of foundation bearing strata that could undergo static settlement and dynamic settlement upon loading and shaking.
3. Presence of varying and heterogeneous soil types within the Remodel/Addition footprint and appurtenant improvements.

In consideration of the above factors, it is ASE's opinion that overexcavation and backfilling with properly compacted fill within the footprint of the Remodel/Addition, as recommended herein, will be essential to reduce unfavorable settlements of underlying soils, and to provide satisfactory bearing stratum for the foundations and slabs of the Remodel/Addition. The grading recommendations provided herein should be reviewed when final project concept and grading plans are available.

Upon completion of relevant subgrade preparation measures discussed in all sub-sections of Section 5.2 below, conventional footing foundations and slabs-on-grade bearing on approved compacted fill soils or base is deemed suitable for the support of the Remodel/Addition.

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.

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 Remodel/Addition. 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 Remodel/Addition, 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:

Any undocumented fill soil, if encountered during site grading in the areas of the Remodel/Addition, 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 building/foundation 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, trash enclosure, or hardscape, the lateral extent of removal should be to a minimum distance equal to the depth of undocumented fill/disturbed soils encountered or two (2) feet, whichever is greater.

Subsequent to the removal of artificial fill and/or disturbed native soils, the exposed removal 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 Sections 5.2.5 and 5.2.6 below.

5.2.2 Expansive Soils:

Laboratory test results on a near surface soil sample indicate a "Low" soil expansion potential (i.e. Expansion Index, EI = 26 per ASTM D4829-21 Test Method) as per defined in Table 1 of ASTM D4829-21 Test Method and Section 1803.5.3 of 2022 CBC. As such, noticeable movement of overlying lightly loaded foundations, slabs, and utilities arising from prolonged moisture inundation to site soils with the tested "Low" soil expansion at shallow depth could not be ruled out.

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" soil expansion on the slabs and structures but may not completely eliminate the problem, especially in areas exposed to potential moisture inundation.

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) Remodel/Addition Structures:

To provide acceptable support for building foundations and slabs, it is recommended that on-site soils within the footprint of the Remodel/Addition pad area, be overexcavated and removed uniformly to a minimum depth of (4) feet below existing grade, or two (2) feet below the bottom of the lowest footing, whichever is lower, and replaced with properly compacted fill such that the Remodel/Addition structure foundations and slabs 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 building/foundation perimeters, wherever is possible.

The fill should be compacted to at least 90 percent relative compaction with a minimum moisture content of two (2) percentage points above optimum moisture content. The excavation bottom should be trimmed neat and devoid of any loose material. Prior to placement of compacted fill, the exposed subgrade to a minimum depth of one (1) foot should be scarified and reworked to at least 90 percent relative compaction with a minimum moisture of two (2) percentage points above optimum moisture content.

The compacted fill underlying the Remodel/Addition foundations should consist of "Very Low" to “Low” expansive, granular material (EI ≤ 30). On-site subgrade soils at their present state generally exhibit an acceptable EI and, thus, are deemed suitable for re-use as compacted fill. However, additional testing should be performed to verify the EI’s of excavated site soils during subgrade preparation.

b) Secondary Structural Improvements/ Slab-on-Grade/Concrete Flatwork/Concrete Driveway/ Hardscape/Pavement Support:

For the purpose of reducing future unsightly and uneven movements and cracks of any slab-on-grade, concrete flatwork, hardscape, or pavement, it is recommended that subgrade soils to a minimum depth of two (2) feet below the bottom of and two (2) feet laterally beyond the footprint of concrete slab-on-grade/concrete flatwork/concrete driveway/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 ≤ 30), 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 areas to receive remedial grading discussed in Sections 5.2.3.a) or b) above, all undocumented fill should be removed full depth under the observation of the Geotechnical Consultant’s representatives, followed by backfilled with compacted fill with suitable moisture conditioning to the recommended percent relative compaction as per mentioned in the preceding paragraphs. Please also note that, from geotechnical viewpoint, new landscape area with only softscape is not subject to the remedial grading requirements mentioned in Sections 5.2.1 and 5.2.3.

c) Special Notes of Remedial Grading:

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 the 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 the current Cal OSHA requirements.

a) Temporary Sloping:

In areas where excavations deeper than 4 feet are not adjacent to existing structures and/or public right-of-ways, sloping procedures may be utilized for temporary excavations. It is recommended that temporary slopes in both fill and native soils be graded no steeper than 1:1 (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 41 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 (1-1⁄2) 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 eight (8) 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 on the Site should consist of predominantly “Very Low” to “Low” expansive, granular material exhibiting an EI not greater than 30, 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 the applicable minimum percent relative compaction stipulated in corresponding sections of this Soils Report. 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

5.3.1 Conventional Shallow Footing Foundation:

a) Minimum Footing Dimension and Reinforcement:

In order to mobilize sufficient soils bearing capacity supporting any new footing foundation, it is recommended that the minimum footing embedments, widths and reinforcements for various footing types tabulated on next page 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)	Max. Composite Ceiling Value (psf)
Continuous Spread Footing/Strip Footing	Isolated Pad Footing			
2,500	2,500	200	400	5,000

The allowable bearing capacities tabulated on 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 footing equal to the depth of the new footing (D), the following tabulated reduction multipliers should be applied to the corresponding allowable soils bearing capacity values.

Lateral Distance between New Footing and Existing Footing expressed in Fraction of the New Footing Depth, D	≥ 1D	½D	0
Reduction Multiplier 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 241 pcf EFP for footings embedded into approved compacted fill soils, subject to a maximum

of 2,410 psf. An ultimate coefficient of friction on the order of 0.4 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.

For any new footing that is within a lateral distance from any existing footing equal to two (2) times the depth of the new footing (D), the following tabulated reduction multipliers 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 Footing expressed in Fraction of the New Footing Depth, D	≥ 2D	1D	0
Reduction Multiplier 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) 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/heave 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:

Retaining walls should be adequately designed to resist the lateral earth pressures and the anticipated construction and service loadings. The earth pressure acting on retaining walls depends primarily on allowable wall movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. As listed in the table on next page, cantilevered retaining walls with on-site soil backfill and level conditions should be designed to resist "active" lateral earth

pressure values, whereas the "at-rest" lateral earth pressure should be used for design of top-restrained retaining walls.

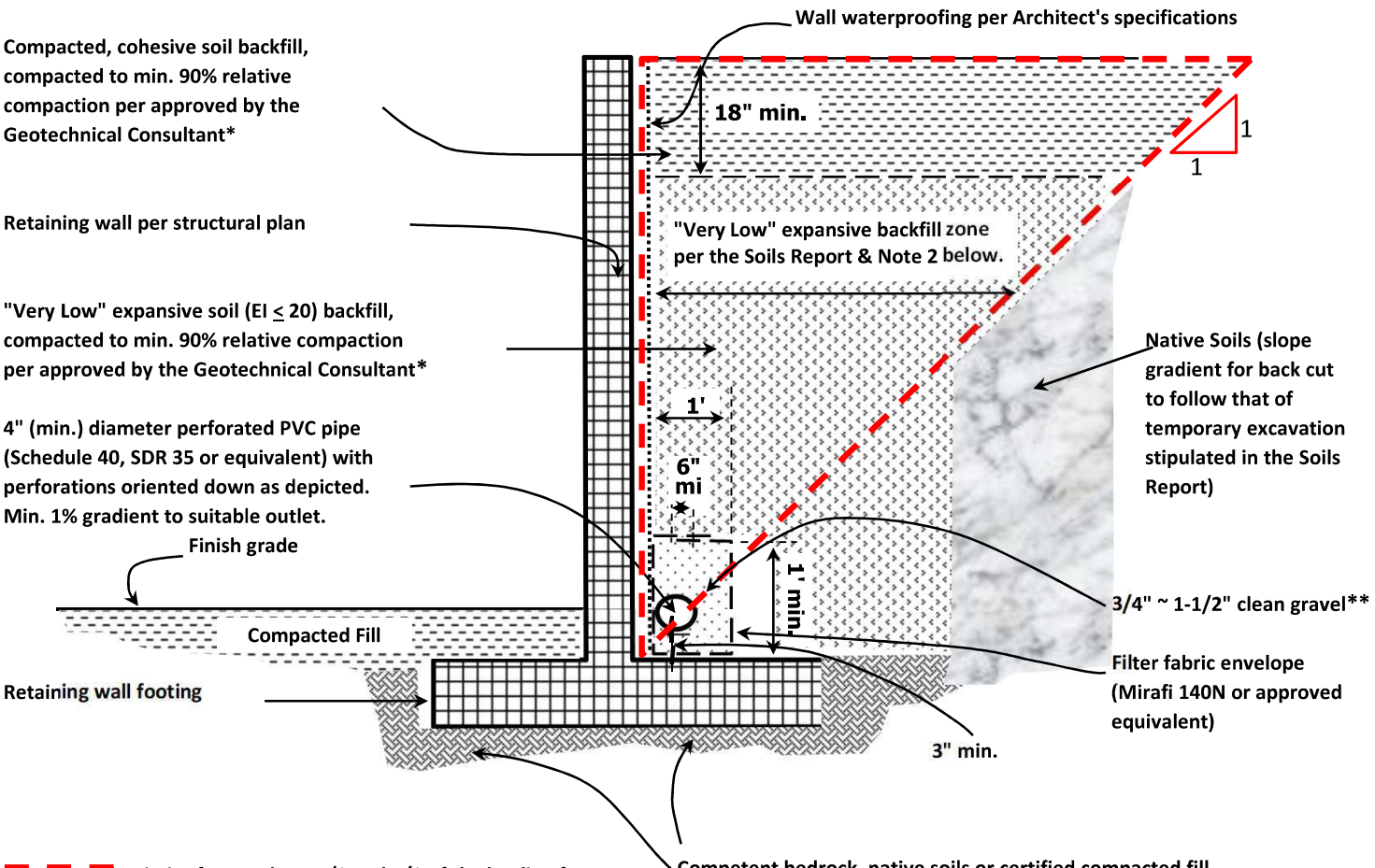
Design Parameter	Values	
Allowable Soils Bearing Capacity	2,500 (1)	
Increase in Bearing Capacity for each 12-inch increment in Footing Depth / Width	400 psf/200 psf, subject to a composite ceiling value of 5,000 psf (1)	
Active Pressure (level backfill with on-site soils or equivalent)	41 pcf EFP (2)	
At-rest Pressure (level backfill with on-site soils or equivalent)	62 pcf EFP (2)	
Passive Pressure (per foot of depth)	241 pcf-EFP, subject to a ceiling value of 2,410 psf (3)	
Ultimate Coefficient of Friction	0.4 (3)	
Resultant Seismic Lateral Earth Pressure (lb/ft) / Point of Application	**Active**	15 H^2 / 1/3H from wall footing bottom
	At-rest	24 H^2 / 1/3H from wall footing bottom
Minimum Footing Depth	18 inches	
Minimum Footing Width	15 inches	
Minimum Footing Reinforcement	Four #4 bars – two near the top and two near the bottom	

(1) Based on compliance with pertinent earthwork recommendations in Section 5.2;
(2) Design values assuming a drained condition within the backfill zone and no surcharge loading conditions;
(3) Passive lateral resistance may be combined with frictional resistance provided the passive lateral resistance value is reduced by one-third (1/3).

Cantilevered and top-restrained wet well, manhole and/or substructure 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, respectively, as depicted in Figure 5, Nearby Building Surcharge Consideration & Retaining Wall Drainage Details. Foundations should be reinforced as recommended by the Structural Consultant with appropriate back drainage provided to avoid excessive build-up of hydrostatic wall pressures, if drainage outlet is practically available.

For the design of wet well, manhole and/or substructure walls exceeding six (6) feet in retained height that need to account for seismic lateral earth pressure, ASE recommends that the active and at-rest seismic lateral earth pressure values and distribution tabulated above, derived based on the guidelines stipulated in LA County Admin. Manual S004.0 (Reference 28), with calculations shown on Plate LP-1, Calculations of Lateral Earth Pressure Values, in Appendix B, should be considered for cantilevered and top-restrained wet well, manhole and/or substructure walls, respectively.

It is preferred that backfill of wet well, manhole and/or substructure walls should consist of approved "Very Low" to "Low" expansive (EI ≤ 30) material and should be compacted to a minimum relative compaction of 90 percent. 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 wet well, manhole and/or substructure walls backdrainage. Other retaining wall drainage alternatives such as



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-1557-02

** 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.

Note2: Min. Extent of " Very Low" expansion backfill zone should be as per indicated by the red-dotted wedge area behind retaining wall.

Schematic Not To

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Project:	Prop. Corona South Mall Improvements 725 South Main Street, Corona, CA		
Figure 5	Nearby Building Surcharge Consideration & Retaining Wall Drainage Details		
Proj. No.:	7232.25.1	Date:	August, 2025

CONTECH C-Drain system or MIRADRAIN may be considered but should first be reviewed and approved by the Geotechnical Consultant prior to implementation.

Should the space behind the new wet well, manhole and/or substructure walls be too tight to implement the above recommended backfill effort, as an alternative, 1.5-sack control density fill may be used in lieu of regular soil backfill, provided that the integrity and functionality of wall backdrain is protected and maintained. The Geotechnical Consultant should be on-site during slope cutting and wet well, manhole and/or substructure wall construction to inspect the exposed slope conditions, to evaluate the stability of slope cuts and, if necessary, to provide additional remedial or mitigative recommendations.

It should be noted that the use of heavy compaction equipment in close proximity to wet well, manhole and/or substructure walls 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/flatworks, a modulus of subgrade reaction ("k-value") on the order of 120 pounds per square inch per inch ("psi/in") and an allowable bearing capacity of 900 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 thicknesses of four (4) inches with minimum reinforcement consisting of number 3 reinforcing bars spaced maximum 12 inches on centers each way, placed at mid-slab, 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.

Per the stipulations in Section 5.2.3.2 of ACI 302.1R-15, for slabs in moisture-sensitive areas, it is recommended that the concrete slabs 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. A minimum 12-

inch overlap between moisture barrier layers should be ensured during placement, if applicable. All moisture barrier layers should be puncture free prior to slab construction. 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 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.

5.6 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.

5.6.1 Concrete Corrosion:

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

A soluble sulfate content of 0.003% 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.

5.6.2 Metal Corrosion:

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

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

The resistivity value of 2,154 ohm-cm per ASTM G187-23 Test Method coupled with a pH-value of 7.28 per CTM 643 classifies the on-site soils tested to be "Moderately 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 "Moderately 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 might be advisable that a qualified corrosion consultant be engaged to review the building plans.

A soluble chloride content of 7 ppm was recorded in our laboratory tests per CTM 422. Per Caltrans guidelines and specifications (References 20 and 21), 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 5.2.5 above will ensure that no other particular reinforcement protection.

5.7 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.

Soils for pipe bedding should consist of non-expansive granular soils having a tested Sand Equivalent, SE, value (ASTM D 2419-22 Test Method) not less than 30. Should on-site soils be considered for bedding or shading of utilities, satisfactory SE testing result should be obtained by the Geotechnical Consultant prior to application.

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).

5.8 Plan Review, Observations and Testing

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

All excavations should be observed by a representative of this office to verify minimum embedment depths, competency of bearing soils and that the excavations are free of loose and disturbed materials.

Such observations should be made prior to placement of any fill, reinforcing steel or concrete. All grading and fill compaction should be performed under the observation of and testing by a Geotechnical Consultant or his representative.

6.0 CLOSURE

This report has been prepared for the exclusive use of **IDS Group Inc.** (the Client) and their design consultants for use in the remodel of the proposed 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.
4. Change of ownership of property that would render the proposed development indicated in this report irrelevant or significantly different.

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 April 2, 2025, 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

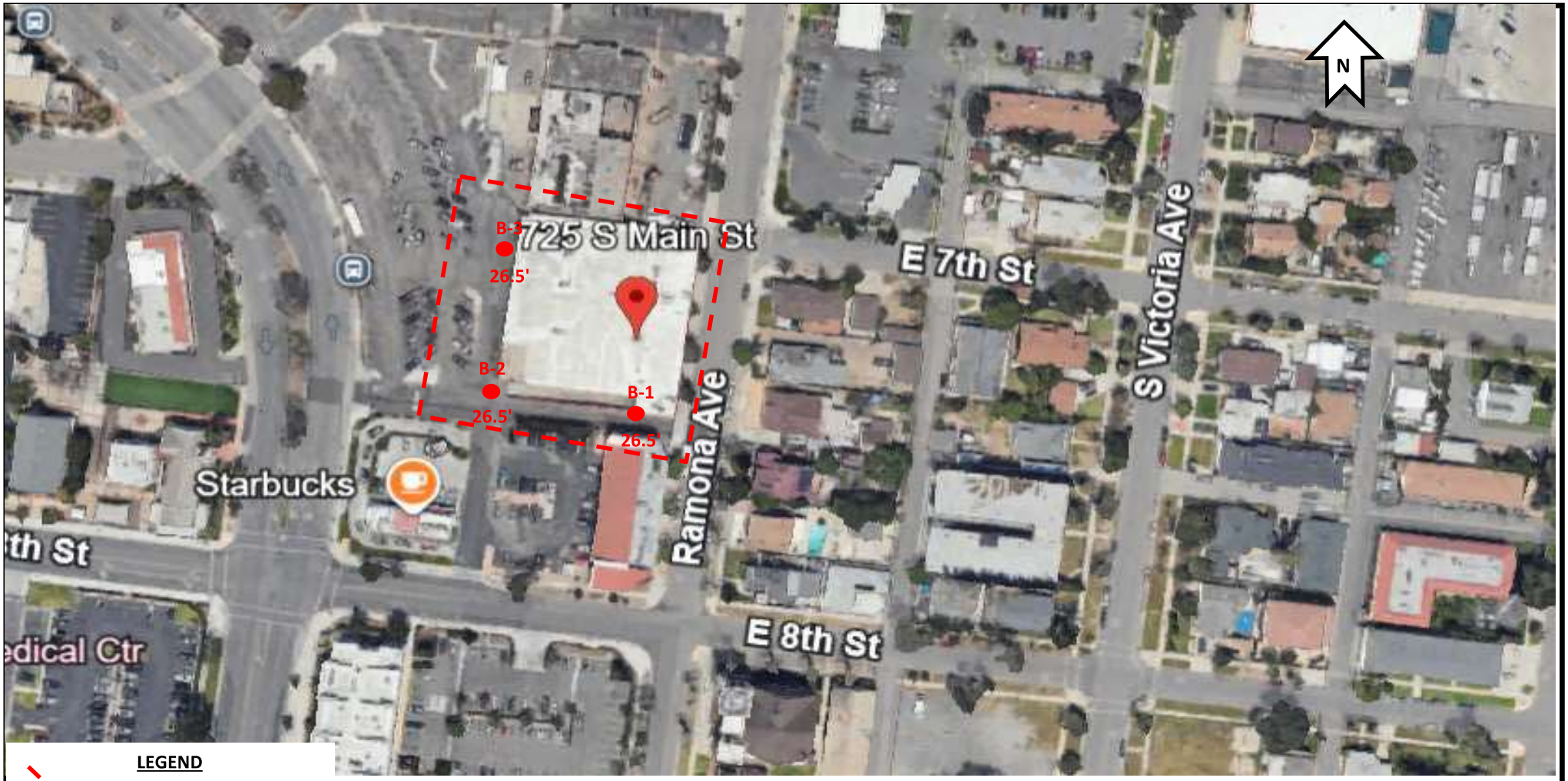
On April 24, 2025, field explorations were performed by drilling three (3) test borings at the approximate locations indicated on the attached Boring Location Plan, Plate A. The exploratory borings were drilled by Choice Drilling, Inc. utilizing a truck mounted CME55, rotary drilling rig equipped with 8-inch diameter continuous flight, hollow-stem rotary augers. All borings extended to a terminal explored depth of 26 feet 6 inches below 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. Additional samples were obtained in a Standard Penetration sampler in accordance with specification outlined in ASTM D1586-18e1 Test Method. 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 Standard Penetration Test ("N" Valves) and Modified California barrel sampler blow counts are presented in the attached Field Logs of Boring ("B" Plates). The United Soil Classification System (USCS) and symbols of soil samples adopted in the Field Logs of Boring are shown in Exhibit 1.

The subsurface soils descriptions presented in the Field Logs of 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 Map
Exhibit 1	United Soils Classification System and Symbols of Soil Samples
Plates B-1 through B-3	Field Logs of Boring



LEGEND

- Approximate Site Limits
- B-1 Designated boring number
- Approximate boring location
- 26.5' Terminal depth of boring

NOT TO SCALE

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Boring Location Plan

Project Name:	**Corona South Mall Improvements 725 South Main Street, Corona, CA 92882**	
Plate A	Proj. No.:	**7232.25.1**
	Date:	**August, 2025**

Associated Soils Engineering, Inc.

Exhibit 1

UNIFIED SOIL CLASSIFICATION SYSTEM

(excerpted from Caltrans Geotechnical Manual)

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOILS (more than 50% of material is larger than No. 200 sieve size.)			
GRAVELS More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravels (Less than 5% fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels, gravel-sand mixtures, little or no fines
	Gravels with fines (More than 12% fines)	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
SANDS 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sands (Less than 5% fines)	SW	Well-graded sands, gravelly sands, little or no fines
		SP	Poorly graded sands, gravelly sands, little or no fines
	Sands with fines (More than 12% fines)	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS (50% or more of material is smaller than No. 200 sieve size.)			
SILTS AND CLAYS Liquid limit less than 50%		ML	Inorganic silts and very fine sands, rock flour, silty of clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silty clays of low plasticity
SILTS AND CLAYS Liquid limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS		PT	Peat and other highly organic soils

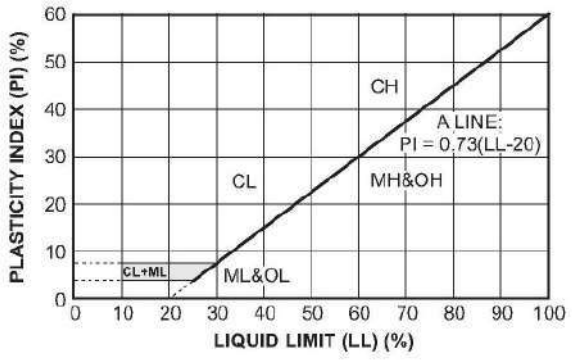
LABORATORY CLASSIFICATION CRITERIA

GW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above "A" line with P.I. greater than 7	
SW	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{D_{30}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for GW	
SM	Atterberg limits below "A" line or P.I. less than 4	Limits plotting in shaded zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols.
SC	Atterberg limits above "A" line with P.I. greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent ..... GW, GP, SW, SP
More than 12 percent ..... GM, GC, SM, SC
5 to 12 percent ..... Borderline cases requiring dual symbols

PLASTICITY CHART



Symbols of Soil Samples

Bulk Sample | Shelby Tube Sample | Bag Sample | SPT Sample | Moldified Cal Ring Sample

DRILLED BY: Choice Drilling (Javi)	DATE DRILLED: 4/24/25	LOGGED BY: RN
DRILLING EQUIPMENT: CME75 drill rig w/ 8" HS auger	SURFACE ELEVATION: approx. 680' AMSL	GROUNDWATER DEPTH: NA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE TYPE	BLOW COUNT (BLOWS/6")	WATER CONTENT (%)	DRY DENSITY (PCF)	LL / PL / EI	OTHER LAB TESTS
SOIL DESCRIPTIONS	USCS	SOIL TYPE							
AC: 3", AB: 6.5", poor conditioned asphalt, ravelling and longitudinal cracking			0						MDD-OMC,DS,Hydro, Atterberg,Corrosivity
Silty sand with gravel, loose, slightly damp, medium to dark brown	AS SM		GWT not encountered 5		10/6/8	5.9	114.3		In-situ DD-MC,DS
Silty sand with gravel, loose, slightly damp, medium to dark brown	SM				8/11/13	4.3	107.3		In-situ DD-MC
Silty fine sand with gravel, loose, slightly damp, medium brown	SM		10		20/32/40	6.8	117.7		In-situ DD-MC
Silty fine sand with gravel, loose, damp, medium brown	SM		15		4/5/6	9.6			In-situ MC
Sandy clay with rock, medium stiff, damp, medium brown	SC		20		27/50-6"	4.0	127.2		In-situ DD-MC
Sandy clay with rock, medium stiff, damp, medium brown	SC		25		3/4/4	13.8			In-situ MC
Boring completed at depth of 26.5', no groundwater encountered, bored hole backfilled with cutting			30 35						

File: C:\Superlog4\PROJECT\7232.25.log Date: 5/29/2025

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FIELD LOG OF BORING

Corona South Mall Improvements
601&725 South Main St, Corona, CA 92882

PROJECT NO.	DATE	BORING NO.
7232.25	5/15/25	**B-1**

DRILLED BY: Choice Drilling (Javi)	DATE DRILLED: 4/24/25	LOGGED BY: RN
DRILLING EQUIPMENT: CME75 drill rig w/ 8" HS auger	SURFACE ELEVATION: approx. 680' AMSL	GROUNDWATER DEPTH: NA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE TYPE	BLOW COUNT (BLOWS/6")	WATER CONTENT (%)	DRY DENSITY (PCF)	LL / PL / EI	OTHER LAB TESTS
SOIL DESCRIPTIONS	USCS	SOIL TYPE							
AC: 4.5", AB: 3.5", poor conditioned asphalt, transverse and longitudinal cracking Clayey sand with rocks, loose, slightly damp, dark brown	SM		0		3/2/3	11.5	113.7		In-situ DD-MC
Silty sand with gravel and rocks, slightly damp to dry, loose, medium brown to dark brown	SM		5		20/32/45	3.7	122.4		In-situ DD-MC,DS
Silty sand with gravel and rocks, slightly damp to dry, loose, medium brown to dark brown	SM		10		6/10/13	4.7	110.3		In-situ DD-MC
Silty sand with gravel and rocks, damp, medium dense, medium brown	SM		15		6/13/19	6.0			In-situ MC
Sandy clay with rocks, medium dense, damp, medium brown	SC		20		40/50-5"	8.9	105.7		In-situ DD-MC
Sandy clay with rock, firm, damp, medium brown	SC		25		7/13/20	12.1			In-situ MC
Boring completed at depth of 26.5', no groundwater encountered, bored hole backfilled with cutting			30						
			35						

GWT not encountered

File: C:\Superlog4\PROJECT\7232.25.log Date: 5/29/2025

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FIELD LOG OF BORING

Corona South Mall Improvements
601&725 South Main St, Corona, CA 92882

PROJECT NO.	DATE	BORING NO.
7232.25	5/15/25	**B-2**

DRILLED BY: Choice Drilling (Javi)	DATE DRILLED: 4/24/25	LOGGED BY: RN
DRILLING EQUIPMENT: CME75 drill rig w/ 8" HS auger	SURFACE ELEVATION: approx. 680' AMSL	GROUNDWATER DEPTH: NA

DESCRIPTION AND CLASSIFICATION			DEPTH (FEET)	SAMPLE TYPE	BLOW COUNT (BLOWS/6")	WATER CONTENT (%)	DRY DENSITY (PCF)	LL / PL / EI	OTHER LAB TESTS
SOIL DESCRIPTIONS	USCS	SOIL TYPE							
Silty sand with rocks, loose, slightly damp, medium to dark brown	SM		0		15/18/30	7.3	107.7		In-situ DD-MC
Silty clayey sand with rocks and gravel, loose, damp, medium to dark brown	SM		5		10/19/22	4.8	98.6		In-situ DD-MC
Coarse sand with rocks, trace silt, loose, damp, medium to dark brown	SP		10		12/16/22	4.3	111.3		In-situ DD-MC
Sandy clay with rocks, medium firm, damp, medium brown	CL		15		3/3/5	12.7			In-situ MC
Sandy clay, lots of large rocks, dense, damp, medium brown with gray rock	CL		20		50-6"	1.9	121.0		In-situ DD-MC
Sandy clay with rocks, stiff, damp, medium brown	CL		25		5/6/9	11.4			In-situ MC
Boring completed at depth of 26.5', no groundwater encountered, bored hole backfilled with cutting			30						
			35						

GWT not encountered

File: C:\Superlog4\PROJECT\7232.25.log Date: 5/29/2025

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FIELD LOG OF BORING

Corona South Mall Improvements
601&725 South Main St, Corona, CA 92882

PROJECT NO.	DATE	BORING NO.
7232.25	5/15/25	**B-3**

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 Shelby tubes and 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 Boring (see attached "B" Plates).

- Direct Shear Tests

Direct shear (ASTM D3080-11 Test Method) tests were performed on selected relatively undisturbed and remolded samples to determine the shear strength parameters of various soil samples. The results of these tests are shown graphically on the appended "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'	0.003 / S0	7 / C0	2,154/ Moderately Corrosive	7.28

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'	138.0	7.5	Silty Sand with Gravel and Clay

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.

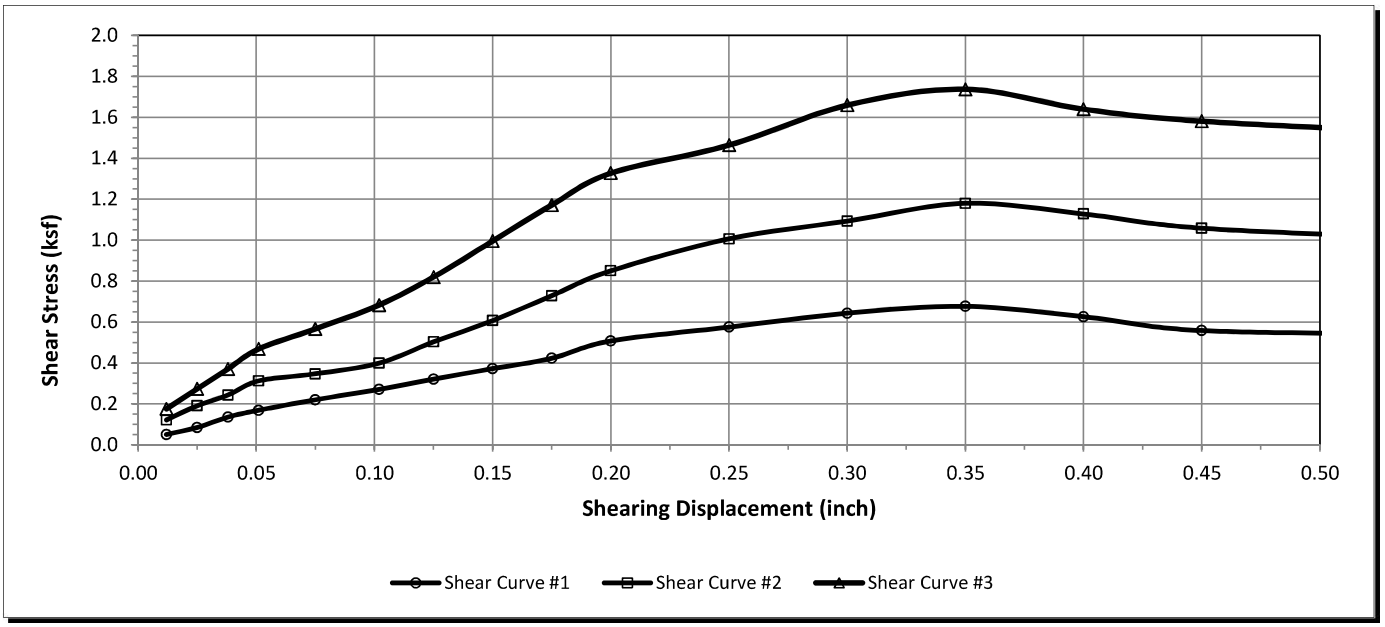
Sample ID	Molded Dry Density (pcf)	Molded Moisture Content (%)	% Saturation	Expansion Index (EI)	Expansion Classification
B-1 @ 1”-5’	114.2	10.4	59.1	26	Low

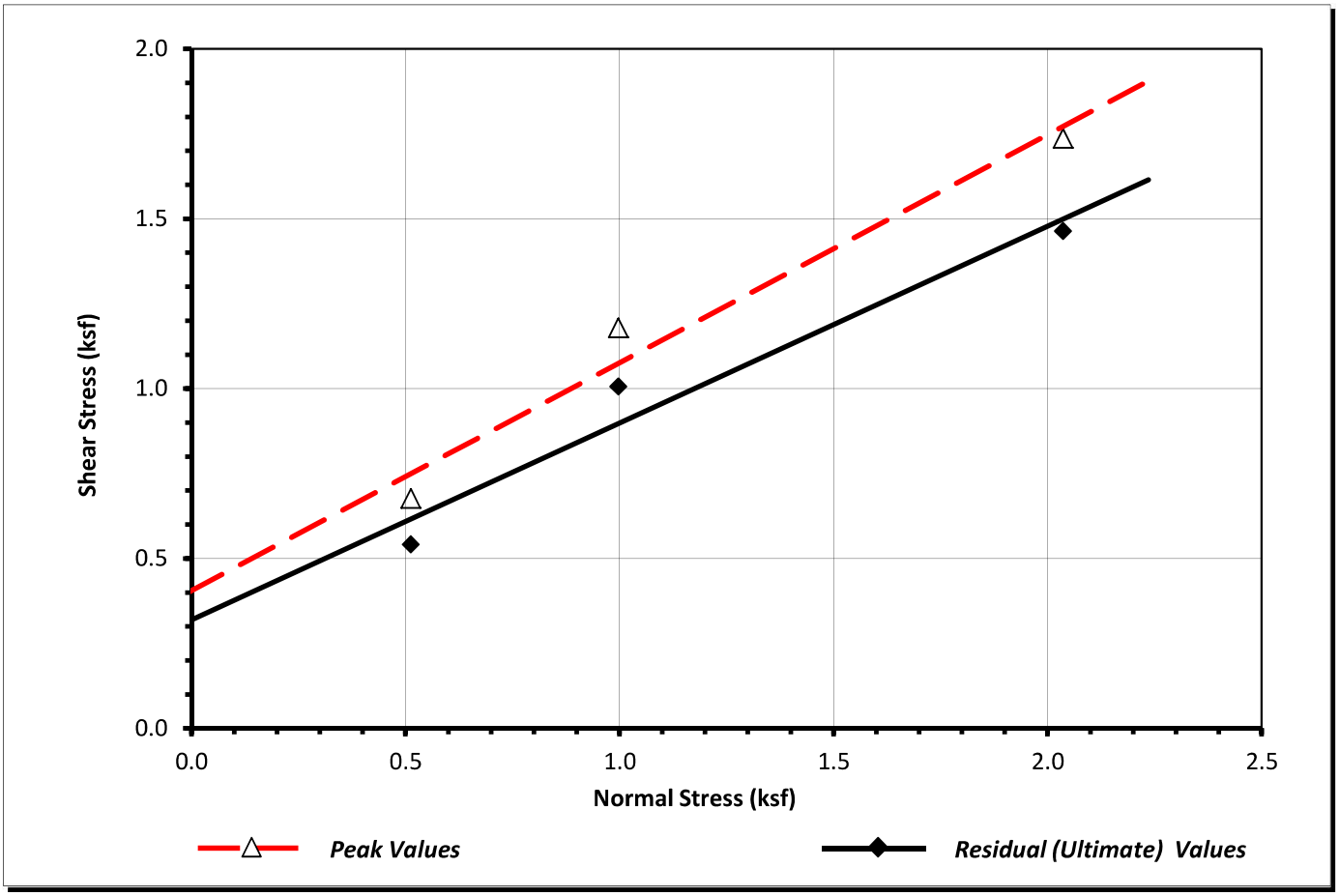
- Atterberg Limits Test

The Atterberg Limits (liquid limit-plastic limit and plasticity index) were determined on a selected soils sample in accordance with ASTM D4318—17e1 Test Method, Method A (multi-point test), dry preparation procedures. The test results are tabulated below.

Sample ID	Liquid Limit, LL (%)	Plastic Limit, PL (%)	Plasticity Index, PI	Soil Classification
B-1 @ 1’ - 5’	24	17	7	ML

Plates D-1 through D-3 Direct Shear Test Results





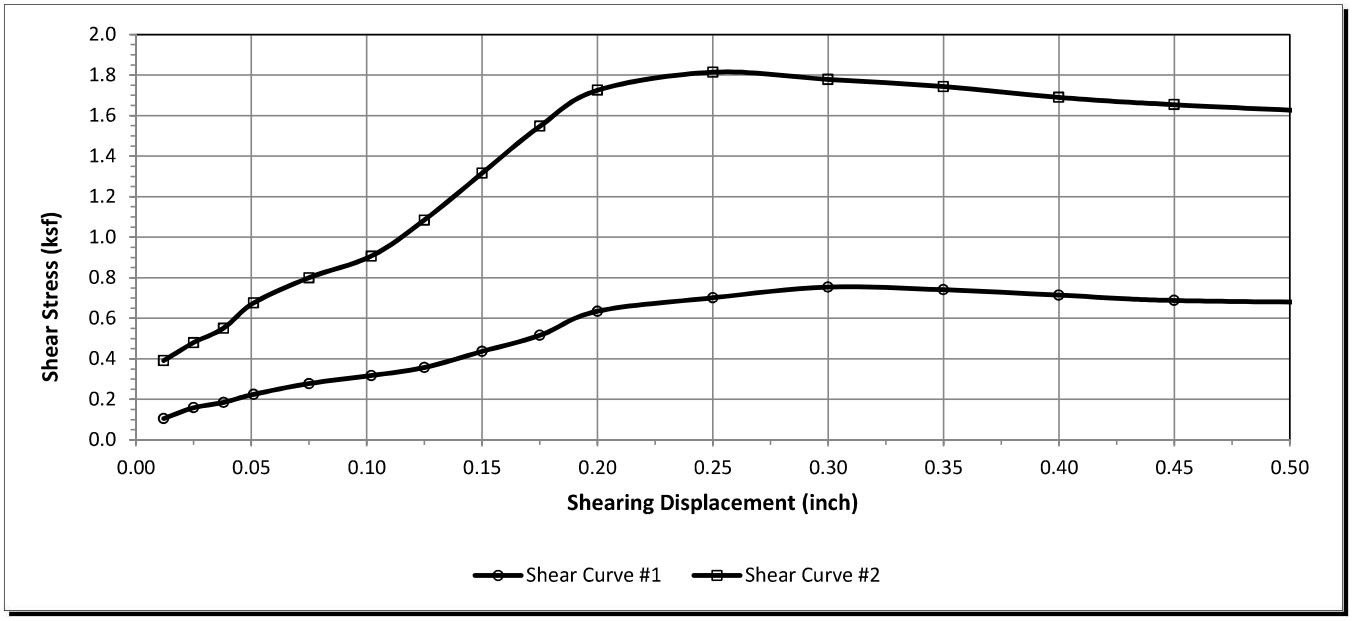
Sample	*B-1 @ 3'*			Dry Density (pcf)	*114.3*
Description	*Dark brown silty sand w/ small rock & fine gravel & trace silty clay*			Moisture Content (%)	*5.9*
ϕ-angle (degree)	*30.1*	>	Ultimate (Residual)	*33.9*	> Peak
Cohesion (ksf)	*0.319*			*0.405*	

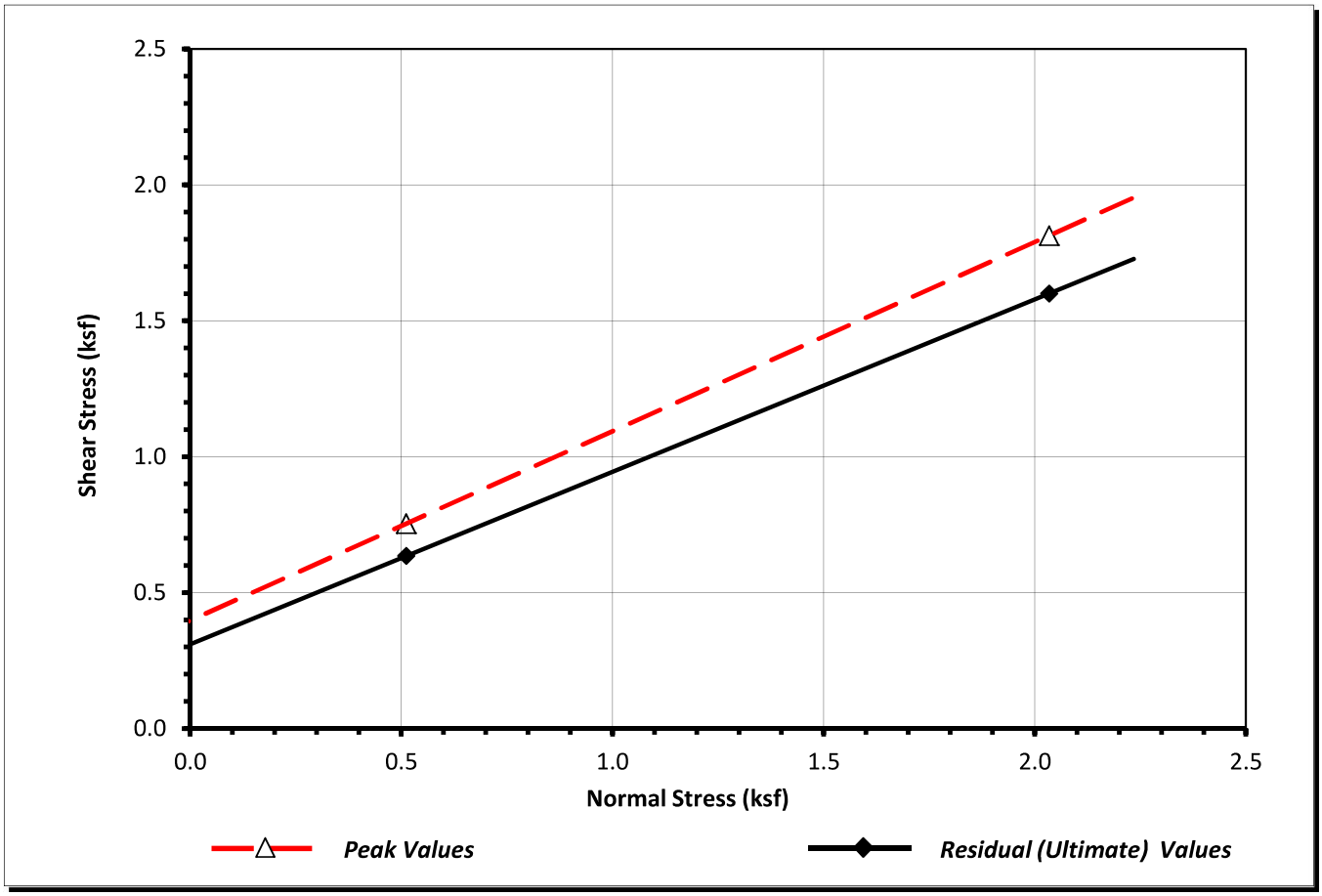
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Project:	**Prop. Corona South Mall Improvements 725 South Main Street, Corona, CA**		
Plate D-1	**Direct Shear Test Result (ASTM D 3080-23 Test Method)**		
Project No.:	**7232.25.1**	Date:	**August, 2025**





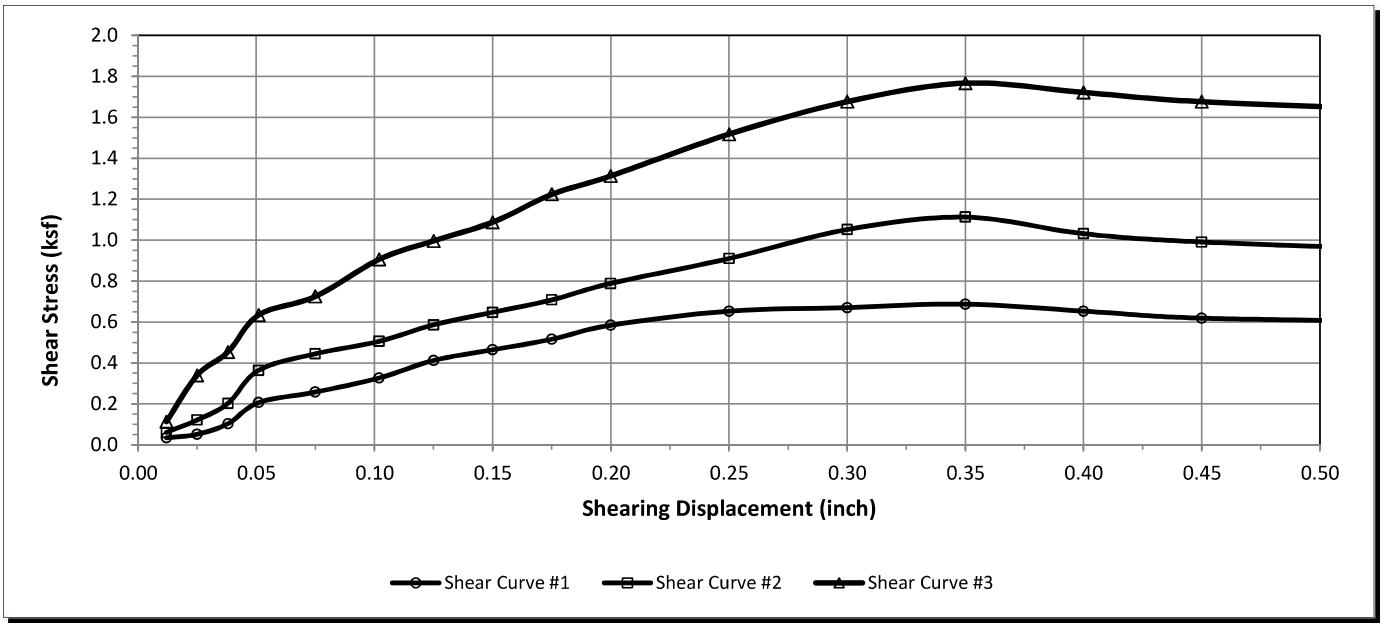
Sample	*B-2 @ 7'*			Dry Density (pcf)	*122.4*
Description	*Medium brown silty sand w/ small rock & fine gravel*			Moisture Content (%)	*3.7*
ϕ-angle (degree)	*32.4*	Ultimate (Residual)	*34.8*	Peak	
Cohesion (ksf)	*0.310*		*0.398*		

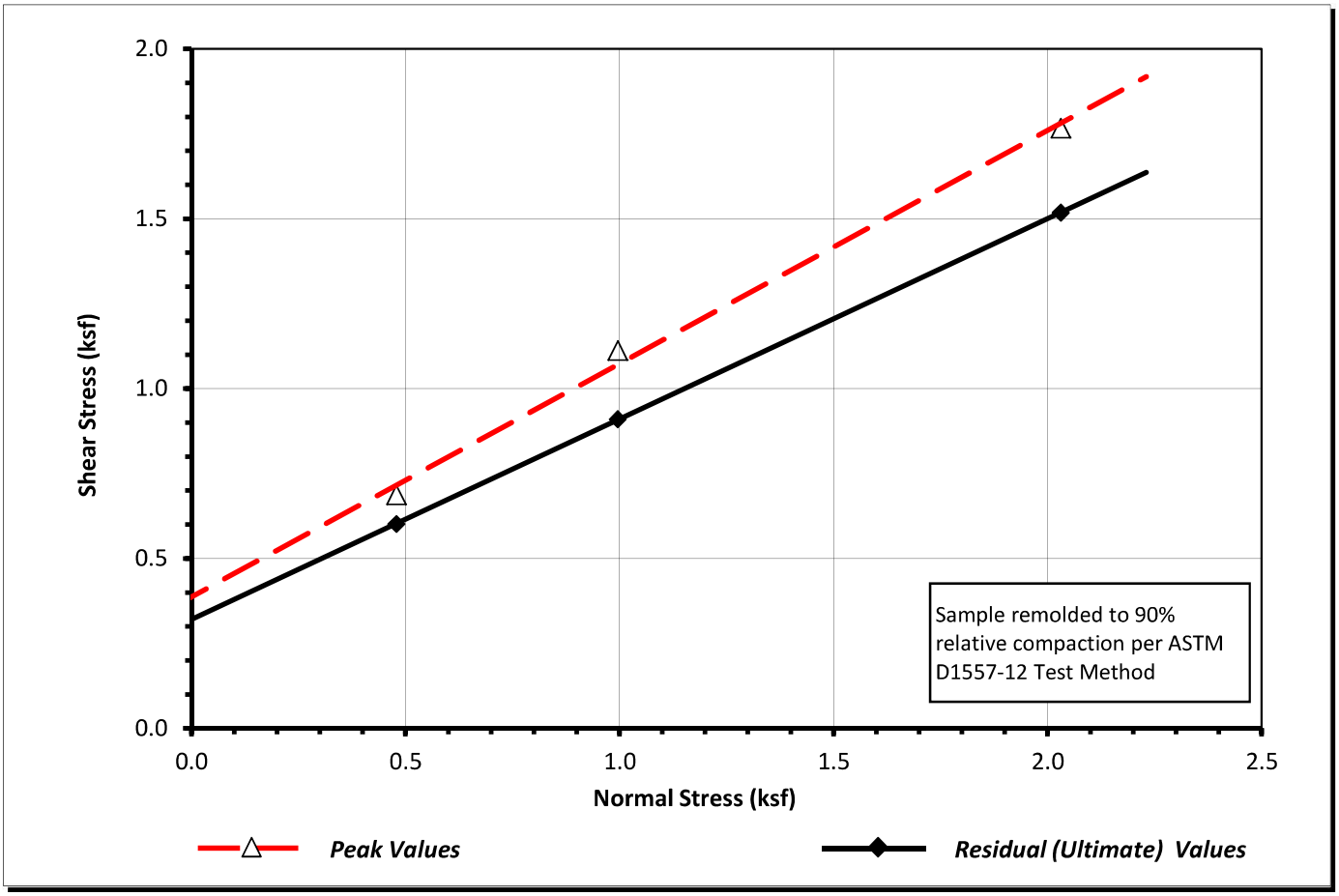
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Project:	**Prop. Corona South Mall Improvements 725 South Main Street, Corona, CA**		
Figure D-2	**Direct Shear Test Result (ASTM D 3080-23 Test Method)**		
Proj. No.:	**7232.25.1**	Date:	**August, 2025**





Sample	*B-1 @ 1-5'*			Dry Density (pcf)	*124.2*
Description	*Light olive brown silty sand w/ gravel and trace silty clay*			Moisture Content (%)	*7.5*
ϕ-angle (degree)	*30.5*	>	Ultimate (Residual)	*34.5*	> Peak
Cohesion (ksf)	*0.321*			*0.387*	

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Project:	**Prop. Corona South Mall Improvements 725 South Main Street, Corona, CA**		
Plate D-3	**Direct Shear Test Result (ASTM D 3080-23 Test Method)**		
Project No.:	**7232.25.1**	Date:	**August, 2025**

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
Plate M-1	Dynamic Settlement of Dry Soil
Plate LP-1	Calculations of Lateral Earth Pressure Values

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*                    *
*    E Q F A U L T   *
*                    *
*    Version 3.00    *
*                    *
**********************

      DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS


JOB NUMBER: 7232.25.1
                                                DATE: 08-08-2025

JOB NAME: Corona South Mall Improvements
          725 South Main Street, Corona, CA 92882
CALCULATION NAME: Corona South Mall Improvements

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

SITE COORDINATES:
   SITE LATITUDE:  33.8740
   SITE LONGITUDE:  117.5658

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.
CHINO-CENTRAL AVE. (Elsinore)	2.0(3.2)	6.7	0.582	X
ELSINORE (GLEN IVY)	3.7(6.0)	6.8	0.381	X
WHITTIER	4.2(6.8)	6.8	0.362	IX
SAN JOSE	17.3(27.9)	6.4	0.137	VIII
PUENTE HILLS BLIND THRUST	18.0(28.9)	7.1	0.203	VIII
SAN JOAQUIN HILLS	18.5(29.8)	6.6	0.148	VIII
CUCAMONGA	19.6(31.6)	6.9	0.166	VIII
SIERRA MADRE	19.9(32.0)	7.2	0.195	VIII
ELSINORE (TEMECULA)	20.3(32.6)	6.8	0.118	VII
SAN JACINTO-SAN BERNARDINO	20.4(32.8)	6.7	0.110	VII
SAN JACINTO-SAN JACINTO VALLEY	21.5(34.6)	6.9	0.118	VII
SAN ANDREAS - SB-Coach. M-1b-2	26.4(42.5)	7.7	0.153	VIII
SAN ANDREAS - SB-Coach. M-2b	26.4(42.5)	7.7	0.153	VIII
SAN ANDREAS - San Bernardino M-1	26.4(42.5)	7.5	0.137	VIII
SAN ANDREAS - Whole M-1a	26.4(42.5)	8.0	0.179	VIII
NEWPORT-INGLEWOOD (L.A.Basin)	27.7(44.5)	7.1	0.102	VII
NEWPORT-INGLEWOOD (Offshore)	27.9(44.9)	7.1	0.101	VII
SAN ANDREAS - Cho-Moj M-1b-1	29.0(46.6)	7.8	0.149	VIII
SAN ANDREAS - 1857 Rupture M-2a	29.0(46.6)	7.8	0.149	VIII
SAN ANDREAS - Mojave M-1c-3	29.0(46.6)	7.4	0.118	VII
CLAMSHELL-SAWPIT	30.0(48.2)	6.5	0.079	VII
CLEGHORN	30.2(48.6)	6.5	0.061	VI
RAYMOND	31.5(50.7)	6.5	0.074	VII
UPPER ELYSIAN PARK BLIND THRUST	33.5(53.9)	6.4	0.063	VI
NORTH FRONTAL FAULT ZONE (West)	33.7(54.3)	7.2	0.113	VII
VERDUGO	38.0(61.2)	6.9	0.079	VII
SAN JACINTO-ANZA	38.3(61.7)	7.2	0.075	VII
PALOS VERDES	38.9(62.6)	7.3	0.080	VII
HOLLYWOOD	41.7(67.1)	6.4	0.047	VI
CORONADO BANK	46.7(75.1)	7.6	0.079	VII
ELSINORE (JULIAN)	46.7(75.2)	7.1	0.055	VI
PINTO MOUNTAIN	50.1(80.6)	7.2	0.054	VI
NORTH FRONTAL FAULT ZONE (East)	50.1(80.7)	6.7	0.047	VI
SIERRA MADRE (San Fernando)	50.3(80.9)	6.7	0.047	VI
SANTA MONICA	50.6(81.4)	6.6	0.043	VI
SAN GABRIEL	51.2(82.4)	7.2	0.053	VI
ROSE CANYON	52.6(84.6)	7.2	0.051	VI
HELENDALE - S. LOCKHARDT	52.6(84.7)	7.3	0.055	VI
NORTHRIDGE (E. Oak Ridge)	55.9(89.9)	7.0	0.052	VI
MALIBU COAST	56.5(91.0)	6.7	0.040	V

-END OF SEARCH- 41 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE CHINO-CENTRAL AVE. (Elsinore) FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 2.0 MILES (3.2 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.5816 g

DYNAMIC SETTLEMENT OF DRY SOIL

(Per procedures stipulated in Krinitzsky et al, 1984)

PGA (g) = 0.9530

O/X Depth (ft) * = 5.0 (* Incl. of scarification/reworking depth)

Boring No. *Avg. of B-1 thru B-3*

Layer Depth (ft)		Layer Thickness (ft)	Graded Layer ?	N	a_p/g	ΔH/H (%)	Dynamic Dry Settlement (in)	
Top	Bottom						Present	Post-Grading
0.00	5.00	5.00	y	14.0	0.9530	0.875	0.52	0.00
5.00	10.00	5.00	n	29.0	0.9530	0.240	0.14	0.14
10.00	15.00	5.00	n	28.0	0.9530	0.250	0.15	0.15
15.00	20.00	5.00	n	17.0	0.9530	0.549	0.33	0.33
20.00	25.00	5.00	n	60.0	0.9530	0.000	0.00	0.00
25.00	26.50	1.50	n	18.0	0.9530	0.475	0.09	0.09
Total Dynamic Dry Settlement (in)						Present State	1.23	---
						Post-Grading	---	0.71

Calculations of Lateral Earth Pressure Values

γ (pcf)	127.20	S_{DS}	1.376			
PGA (= S_{DS}/2.5) (g)	0.550		K_h	0.468	0.017	A4
Φ_P (degree)	34.40		K_a	0.320	0.249	X
Φ_U (degree)	31.00		K_v	0.312		
β (degree)	0.00	(level)				
	26.50	(2H:1V)				
δ (degree)	0.00					
θ (degree)	0.00					
C (psf)	316.00					
Coefficient of Friction		0.60	use =>	0.40		

Avg. of Plates D-1 thru D-3

Project No.: 7232.25.1

Date: 8/12/2025

	Static (pcf EFP)			Seismic (lb/ft)[1]		
At-Rest (Top-Restrained)	K_o	$P_{o\ (cal'c)}$	$P_{o\ (report)}$	$\Delta P_{OE\ (cal'c)}$[2]	$\Delta P_{OE\ (report)}$[2]	Point of Application of Resultant Seismic Lateral Force
Level (β = 0°)	0.485	61.69	62.00	23.80	24.00 H^2	1/3H from Ret. Wall Footing Bottom
Sloping 2:1 (β = 26.5°)	0.485	77.02	78.00			
Active (Cantilevered)	K_A	$P_{A\ (cal'c)}$	$P_{A\ (report)}$	$\Delta P_{AE\ (cal'c)}$[3, 4]	$\Delta P_{AE\ (report)}$[3, 4]	
Level (β = 0°)	0.320	40.72	41.00	14.70	15.00 H^2	1/3H from Ret. Wall Footing Bottom
Sloping 2:1 (β = 26.5°)	0.500	63.57	64.00	24.50	25.00 H^2	1/3H from Ret. Wall Footing Bottom
Passive	K_P	$P_{P\ (cal'c)}$	$P_{P\ (report)}$			
Level (β = 0°)	3.124	397.38	241.00			
Sloping 2:1 (β = 26.5°)	7.987	1015.88	616.00			

1. Per LA County Admin. Manual S004.0 with Rev. dated 1/6/2020.
2. For Level backfill; $\Delta P_{OE} = 1/2\gamma H^2(0.68PGA/g)$
3. For Level backfill; $\Delta P_{AE} = 1/2\gamma H^2(0.42PGA/g)$
4. For Sloping backfill not steeper than 2:1 (H:V) & displacement not more than 5 cm; $\Delta P_{AE} = 1/2\gamma H^2(0.70PGA/g)$

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