



GEOTECHNICAL INVESTIGATION

LAKEWOOD CIVIC PARK
4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

SEPTEMBER 17, 2024
PROJECT NO. W1974-06-01

PREPARED FOR:
Dahlin Group, Inc.
Pleasanton, California



Project No. W1974-06-01
September 17, 2024

Dahlin Group, Inc.
5865 Owens Drive
Pleasanton, CA 94588

Subject: GEOTECHNICAL INVESTIGATION
LAKEWOOD CIVIC PARK
4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

Ladies and Gentlemen:

In accordance with your authorization of our proposal dated July 22, 2023, we have performed a geotechnical investigation for proposed Lakewood Civic Park planned for 4900 Clark Avenue in the City of Lakewood, California. The accompanying report presents the findings of our study and our conclusions and recommendations pertaining to the geotechnical aspects of the proposed design and construction. Based on the results of our investigation, it is our opinion that the site can be developed as proposed, provided the recommendations in this report are followed and implemented during design and construction.

If you have any questions regarding this report, or if we may be of further service, please contact the undersigned.

Very truly yours,

GEOCON WEST, INC.



Andrew Sover,
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(email) Addressee



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GEOTECHNICAL INVESTIGATION

1. PURPOSE AND SCOPE

This report presents the results of a geotechnical investigation for the proposed Lakewood Civic Park planned for 4900 Clark Avenue in the City of Lakewood, California (see Vicinity Map, Figure 1). The purpose of the investigation was to evaluate subsurface soil and geologic conditions underlying the site and, based on conditions encountered, to provide conclusions and recommendations pertaining to the geotechnical aspects of proposed design and construction.

The scope of this investigation included a site reconnaissance, field exploration, laboratory testing, engineering analysis, and the preparation of this report. The site was explored on August 2, 2024, by excavating five 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine to depths between 5½ and 63½ feet beneath the ground surface. Additionally, percolation testing was performed in four of the borings (B-1 through B-4) to determine the infiltration rate of site soils. The approximate locations of the exploratory borings are depicted on the Site Plan (see Figure 2). A detailed discussion of the field investigation, including boring logs, is presented in Appendix A.

Laboratory tests were performed on selected soil samples obtained during the investigation to determine pertinent physical and chemical soil properties. Appendix B presents a summary of the laboratory test results.

The recommendations presented herein are based on analysis of the data obtained during the investigation and our experience with similar soil and geologic conditions. References reviewed to prepare this report are provided in the *List of References* section.

If project details vary significantly from those described herein, Geocon should be contacted to determine the necessity for review and possible revision of this report.

2. SITE AND PROJECT DESCRIPTION

The subject site is located at 4900 Clark Avenue in the City of Lakewood, California. The northern half of the site is an asphalt parking lot and the southern half of the site is a vacant lot. The site is bounded a drive aisle and a public library to the north, by an asphalt parking lot and Civic Center Way to the east, by Del Amo Boulevard to the south, and by Clark Avenue to the west. The site is relatively level and surface water drainage at the site appears to flow southeast to the city streets.

Based on the information provided by the Client, it is our understanding that two conceptual plans are being considered for the proposed park. Option 1 includes flatwork and landscape improvements, a restroom and storage building, play area, artistic wall, and trash enclosure. Option 2 depicts two large shade structures over synthetic turf, two stages, a restroom, paved dining area and food truck parking/serving zone. The northeastern parking lot will be preserved to serve either park option. Based on the current site grades with respect to the surrounding roadways, we expect the improvements will be constructed near current site elevations. Plans depicting the proposed improvements are provided on the Site Plan (see Figure 2).

Based on the preliminary nature of the design at this time, wall and column loads were not available. It is anticipated that column loads for the proposed structures will be up to 25 kips, and wall loads will be up to 2 kips per linear foot.

Once the design phase and foundation loading configuration proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Any changes in the design, location or elevation of any structure, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

3. GEOLOGIC SETTING

The site is located in the Los Angeles Basin. The Los Angeles Basin is a coastal plain between the Santa Monica Mountains to the north, the Elysian Hills and Repetto Hills to the northeast, the Puente Hills and Whittier Fault to the east, the Palos Verdes Peninsula and Pacific Ocean on the west and south, and the Santa Ana Mountains and San Joaquin Hills on the southeast. Regionally, the site is located within the Peninsular Ranges geomorphic province, characterized by northwest-trending physiographic and geologic structures such as the nearby Whittier Fault and Newport-Inglewood Fault Zone.

4. SOIL AND GEOLOGIC CONDITIONS

Based on published geologic maps of the area, the site is underlain by younger alluvium and Terrace Deposits/Palos Verdes Sand at depth (Johns, et.al, 1954) most recently referred to as Older Alluvium. The terrace deposits were not encountered in our borings. Artificial fill is present across the site from the previous development. The younger alluvium consists of sandy silt, clay, silt and poorly graded sand. Detailed stratigraphic profiles of the materials encountered at the site are provided on the boring logs in Appendix A.

4.1 Artificial Fill

Artificial fill was encountered in our explorations to a maximum depth of 3 feet below existing ground surface. The artificial fill generally consists of brown sandy silt and can be characterized as slightly moist and soft. The fill is likely the result of past grading or construction activities at the site. Deeper fill may exist between excavations and in other portions of the site that were not directly explored.

4.2 Younger Alluvium

Holocene-age alluvium was encountered beneath the fill in each boring to the maximum depth explored of 63½ feet below existing ground surface. The alluvium generally consists of brown to grey brown interbedded silt, clay, silty sand and poorly graded sand. The alluvial soils are characterized as moist to wet and soft to hard or medium dense to dense.

5. GROUNDWATER

Review of the Seismic Hazard Zone Report for the Long Beach Quadrangle (California Division of Mines and Geology [CDMG], 1998) indicates the historically highest groundwater level in the area is approximately 20 feet beneath the ground surface. Groundwater information presented in this document is generated from data collected in the early 1900's to the late 1990s. Based on current groundwater basin management practices, it is unlikely that groundwater levels will ever exceed the historic high levels.

Groundwater was encountered at a depth of 31 feet below the ground surface during site exploration. The groundwater measurements were performed in a manner that is typical of geotechnical exploration and should not be interpreted as representing a fully equalized water level; the depth to water at the time of construction may be higher or lower than what was observed in the boreholes

Based on the depth of groundwater encountered in our borings and the depth of proposed construction, static groundwater is neither expected to be encountered during construction, nor have a detrimental effect on the project. However, it is not uncommon for groundwater levels to vary seasonally or for groundwater seepage conditions to develop where none previously existed (especially in impermeable fine-grained soils which are heavily irrigated or after seasonal rainfall), groundwater seepage levels encountered during construction may be actually higher than those encountered during our investigation. In addition, recent requirements for stormwater infiltration could result in shallower seepage conditions in the region. Proper surface drainage of irrigation and precipitation will be critical for future performance of the project. Recommendations for drainage are provided in the Surface Drainage section of this report (see Section 7.16).

6. GEOLOGIC HAZARDS

6.1 Surface Fault Rupture

The numerous faults in Southern California include Holocene-active, pre-Holocene, and inactive faults. The criteria for these major groups are based on criteria developed by the California Geological Survey (CGS, formerly known as CDMG) for the Alquist-Priolo Earthquake Fault Zone Program (CGS, 2018). By definition, a Holocene-active fault is one that has had surface displacement within Holocene time (about the last 11,700 years). A pre-Holocene fault has demonstrated surface displacement during Quaternary time (approximately the last 1.6 million years) but has had no known Holocene movement. Faults that have not moved in the last 1.6 million years are considered inactive.

The site is not within a state-designated Alquist-Priolo Earthquake Fault Zone (CGS, 1986) for surface fault rupture hazards. However, the Los Alamos Fault terminates immediately southeast of the site. No Holocene-active or pre-Holocene faults with the potential for surface fault rupture are known to pass directly beneath the site. Therefore, the potential for surface rupture due to faulting occurring beneath the site during the design life of the proposed development is considered low. However, the site is located in the seismically active Southern California region and could be subjected to moderate to strong ground shaking in the event of an earthquake on one of the many active Southern California faults. The faults in the vicinity of the site are shown in Figure 3, Regional Fault Map.

The closest surface trace of an active fault to the site is the Los Alamitos Fault located approximately 50 feet southeast of the site (Jennings & Bryant, 2010). Other nearby active faults are the Newport-Inglewood Fault, the Palos Verdes Fault, the Whittier Fault, and the Hollywood Fault, located approximately 3.5 miles southwest, 11 miles northeast, 11.5 miles to the southwest, and 20 miles to the northwest of the site, respectively (USGS, 2006; Ziony and Jones, 1989). The active San Andreas Fault Zone is located approximately 45 miles northeast of the site (USGS, 2006; Ziony and Jones, 1989).

Several buried thrust faults, commonly referred to as blind thrusts, underlie the Southern California area at depth. These faults are not exposed at the ground surface and are typically identified at depths greater than 3.0 kilometers. The October 1, 1987 M_w 5.9 Whittier Narrows earthquake and the January 17, 1994 M_w 6.7 Northridge earthquake were a result of movement on the Puente Hills Blind Thrust and the Northridge Thrust, respectively. The Puente Hills Blind Thrust underlies the site at depth. These thrust faults and others in the Los Angeles area are not exposed at the surface and do not present a potential surface fault rupture hazard at the site; however, these deep thrust faults are considered active features capable of generating future earthquakes that could result in moderate to significant ground shaking at the site.

6.2 Seismicity

As with all of Southern California, the site has experienced historic earthquakes from various regional faults. The seismicity of the region surrounding the site was formulated based on research of an electronic database of earthquake data. The epicenters of recorded earthquakes with magnitudes equal to or greater than 5.0 in the site vicinity are depicted on Figure 4, Regional Seismicity Map. A partial list of moderate to major magnitude earthquakes that have occurred in the Southern California area within the last 100 years is included in the following table.

LIST OF HISTORIC EARTHQUAKES

Earthquake (Oldest to Youngest)	Date of Earthquake	Magnitude	Distance to Epicenter (Miles)	Direction to Epicenter
Long Beach	March 10, 1933	6.4	19	SE
Tehachapi	July 21, 1952	7.5	94	NW
San Fernando	February 9, 1971	6.6	42	NNW
Whittier Narrows	October 1, 1987	5.9	15	NNE
Sierra Madre	June 28, 1991	5.8	30	NNE
Landers	June 28, 1992	7.3	100	ENE
Big Bear	June 28, 1992	6.4	79	ENE
Northridge	January 17, 1994	6.7	34	NW
Hector Mine	October 16, 1999	7.1	118	ENE
Ridgecrest China Lake Fault	July 5, 2019	7.1	136	NNE

The site could be subjected to strong ground shaking in the event of an earthquake. However, this hazard is common in Southern California and the effects of ground shaking can be minimized if the proposed structures are designed and constructed in conformance with current building codes and engineering practices.

6.3 Seismic Design Criteria

The following table summarizes the site-specific design criteria obtained from the 2022 California Building Code (CBC; Based on the 2021 International Building Code [IBC] and ASCE 7-16), Chapter 16 Structural Design, Section 1613 Earthquake Loads. The data was calculated using the online application U.S. Seismic Design Maps, provided by the Structural Engineers Association of California (SEAOC). The short spectral response uses a period of 0.2 second. We evaluated the Site Class based on the discussion in Section 1613.2.2 of the 2022 CBC and Table 20.3-1 of ASCE 7-16. The values presented on the following page are for the risk-targeted maximum considered earthquake (MCE_R).

2022 CBC SEISMIC DESIGN PARAMETERS

Parameter	Value	2022 CBC Reference
Site Class	D	Section 1613.2.2
MCE _R Ground Motion Spectral Response Acceleration – Class B (short), S _s	1.565g	Figure 1613.2.1(1)
MCE _R Ground Motion Spectral Response Acceleration – Class B (1 sec), S ₁	0.559g	Figure 1613.2.1(2)
Site Coefficient, F _A	1	Table 1613.2.3(1)
Site Coefficient, F _V	1.741*	Table 1613.2.3(2)
Site Class Modified MCE _R Spectral Response Acceleration (short), S _{MS}	1.565g	Section 1613.2.3 (Eqn 16-20)
Site Class Modified MCE _R Spectral Response Acceleration – (1 sec), S _{M1}	0.973g*	Section 1613.2.3 (Eqn 16-21)
5% Damped Design Spectral Response Acceleration (short), S _{DS}	1.043g	Section 1613.2.4 (Eqn 16-22)
5% Damped Design Spectral Response Acceleration (1 sec), S _{D1}	0.649*	Section 1613.2.4 (Eqn 16-23)
*Per Supplement 3 of ASCE 7-16, a ground motion hazard analysis (GMHA) shall be performed for projects on Site Class “D” sites with 1-second spectral acceleration (S₁) greater than or equal to 0.2g, which is true for this site. However, Supplement 3 of ASCE 7-16 provides an exception stating that that the GMHA may be waived provided that the parameter SM1 is increased by 50% for all applications of SM1. The values for parameters SM1 and SD1 presented above have not been increased in accordance with Supplement 3 of ASCE 7-16.		

The table below presents the mapped maximum considered geometric mean (MCE_G) seismic design parameters for projects located in Seismic Design Categories of D through F in accordance with ASCE 7-16.

ASCE 7-16 PEAK GROUND ACCELERATION

Parameter	Value	ASCE 7-16 Reference
Mapped MCE _G Peak Ground Acceleration, PGA	0.671g	Figure 22-9
Site Coefficient, F _{PGA}	1.1	Table 11.8-1
Site Class Modified MCE _G Peak Ground Acceleration, PGA _M	0.738g	Section 11.8.3 (Eqn 11.8-1)

Deaggregation of the MCE peak ground acceleration was performed using the USGS online Unified Hazard Tool, 2014 Conterminous U.S. Dynamic edition (v4.2.0). The result of the deaggregation analysis indicates that the predominant earthquake contributing to the MCE peak ground acceleration is characterized as a 7.3 magnitude event occurring at a hypocentral distance of 10.67 kilometers from the site.

Conformance to the criteria in the above tables for seismic design does not constitute any kind of guarantee or assurance that significant structural damage or ground failure will not occur if a large earthquake occurs. The primary goal of seismic design is to protect life, not to avoid all damage, since such design may be economically prohibitive.

6.4 Liquefaction Potential

Liquefaction is a phenomenon in which loose, saturated, relatively cohesionless soil deposits lose shear strength during strong ground motions. Primary factors controlling liquefaction include intensity and duration of ground motion, gradation characteristics of the subsurface soils, in-situ stress conditions, and the depth to groundwater. Liquefaction is typified by a loss of shear strength in the liquefied layers due to rapid increases in pore water pressure generated by earthquake accelerations.

The current standard of practice, as outlined in the “Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California” and “Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California” requires liquefaction analysis to a depth of 50 feet below the lowest portion of the proposed structure. Liquefaction typically occurs in areas where the soils below the water table are composed of poorly consolidated, fine- to medium-grained, primarily sandy soil. In addition to the requisite soil conditions, the ground acceleration and duration of the earthquake must also be of a sufficient level to induce liquefaction.

The State of California Seismic Hazard Zone Map for the Long Beach Quadrangle (CDMG, 1999) indicates that the site is located in an area designated as having a potential for liquefaction.

The historic high groundwater level in the site vicinity is at a depth of 20 feet beneath the ground surface. Groundwater was encountered in boring B5 at an approximate depth of 31 feet beneath the ground surface.

Liquefaction analysis of the soils underlying the site was performed using the 1996 NCEER method of analysis with the updates by Youd et al. (2001). This semi-empirical method is based on a correlation between values of Standard Penetration Test (SPT) resistance and field performance data.

Screening criteria developed by Bray and Sancio (2006) characterize fine-grained soils which are not susceptible to liquefaction as soils with a plasticity index (PI) that is greater than 18 or with a saturated moisture content that is less than 80 percent of the liquid limit. In order to apply the screening criteria, laboratory testing was performed to evaluate the Atterberg Limits of select soil samples. Laboratory test results used for the screening criteria are presented as Figure B5.

The liquefaction analysis was performed for the Maximum Considered Earthquake level by using a groundwater table of 20 feet below the ground surface, a magnitude 7.3 earthquake, and a peak horizontal acceleration of 0.738g (PGA_M). The enclosed liquefaction analysis, included herein for boring B5, indicates that the alluvium below the historic high groundwater level could be susceptible to approximately 1.6 inches of liquefaction settlement during Maximum Considered Earthquake ground motion (see enclosed calculation sheets, Figures 5 and 6).

6.5 Seismically-Induced Settlement

Dynamic compaction of dry and loose sands may occur during a major earthquake. Typically, settlements occur in thick beds of such soils. The seismically-induced settlement calculations were performed in accordance with the American Society of Civil Engineers, Technical Engineering and Design Guides as adapted from the US Army Corps of Engineers, No. 9.

The calculations provided herein for boring B1 indicates that the soil above the historic high groundwater level of 20 feet could be susceptible to approximately 0.35 inches of settlement as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M). Calculation of the anticipated seismically-induced settlements is provided as Figure 7.

6.6 Slope Stability

The topography at the site is relatively level and the topography in the immediate site vicinity slopes very gently to the south-southwest. The Seismic Hazards Zones Map of the Long Beach Quadrangle (CGS, 1999). Additionally, the site is not within an area identified as having a potential for seismic slope instability (CGS 1999). There are no known landslides near the site, nor is the site in the path of any known or potential landslides. Therefore, the potential for slope stability hazards to adversely affect the proposed development is considered low.

6.7 Earthquake-Induced Flooding

Earthquake-induced flooding is inundation caused by failure of dams or other water-retaining structures due to earthquakes. Based on a review of the California Department of Water Resources Dam Breach Inundation Map Web Publisher, the site is not located within a potential inundation area for an earthquake-induced dam failure. Therefore, the probability of earthquake-induced flooding is considered very low.

6.8 Tsunamis, Seiches, and Flooding

The site is not located within a coastal area. Therefore, tsunamis are not considered a significant hazard at the site.

Seiches are large waves generated in enclosed bodies of water in response to ground shaking. No major water-retaining structures are located immediately up gradient from the project site. Therefore, flooding resulting from a seismically induced seiche is considered unlikely.

The site is within an area of reduced flood risk due to levee (Zone X) as defined by the Federal Emergency Management Agency (FEMA, 2008).

6.9 Oil Fields & Methane Potential

Based on a review of the California Geologic Energy Management Division (CalGEM) Well Finder Website, the site is not located within an oil field and oil or gas wells are not documented within ½-mile of the site (CalGEM, 2024). However, due to the voluntary nature of record reporting by the oil well drilling companies, wells may be improperly located or not shown on the location map and undocumented wells could be encountered during construction. Any wells encountered during construction will need to be properly abandoned in accordance with the current requirements of CalGEM.

Since the site is not located within the boundaries of a known oil field, the potential for the presence of methane or other volatile gases at the site is considered low. However, should it be determined that a methane study is required for the proposed development it is recommended that a qualified methane consultant be retained to perform the study and provide mitigation measures as necessary.

6.10 Subsidence

Subsidence occurs when a large portion of land is displaced vertically, usually due to the withdrawal of groundwater, oil, or natural gas. Soils that are particularly subject to subsidence include those with high silt or clay content. The site is located within an area of known ground subsidence due to groundwater pumping (USGS, 2024). Ground subsidence typically over a region and may not necessarily result in differential settlement within relatively small areas such as the subject site.

7. CONCLUSIONS AND RECOMMENDATIONS

7.1 General

- 7.1.1 It is our opinion that neither soil nor geologic conditions were encountered during the investigation that would preclude the construction of the proposed development provided the recommendations presented herein are followed and implemented during design and construction.
- 7.1.2 Up to 3 feet of existing artificial fill was encountered during the site investigation. The existing fill encountered is believed to be the result of past grading and construction activities at the site. Deeper fill may exist in other areas of the site that were not directly explored. It is our opinion that the existing fill, in its present condition, is not suitable for direct support of proposed foundations or slabs. The existing fill and site soils are suitable for re-use as engineered fill provided the recommendations in the *Grading* section of this report are followed (see Section 7.4).
- 7.1.3 The enclosed seismically-induced settlement analyses indicate that the site soils could be susceptible to up to approximately 2 inches of total settlement as a result of the Maximum Considered Earthquake peak ground acceleration (PGA_M). The resulting differential settlement at the ground surface is anticipated to be less than 1 inch over a distance of 30 feet. The grading and foundation design recommendations presented herein are intended to reduce the effects of settlement on proposed improvements.
- 7.1.4 Based on these considerations, it is recommended that proposed habitable structures be supported on a conventional foundation system deriving support in a blanket of newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer prior to placement of steel or concrete. Recommendations for the *Foundation Design* are provided in Sections 7.5 and 7.6.

- 7.1.4 As a minimum, it is recommended that the upper 3 feet of existing earth materials within proposed habitable structure footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any deeper fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. Recommendations for earthwork are provided in the Grading section of this report (see Section 6.4).
- 7.1.6 All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon) prior to the placement of bedding materials, fill, gravel, steel, or concrete.
- 7.1.5 The upper alluvial soils at the site are currently very moist and the grading contractor should be aware that the existing soils are currently near or slightly above optimum moisture content. Conditions could change seasonally. If the soils are in excess of 3 percent above optimum moisture content at the time of construction the soils will likely require some spreading and drying activities in order to achieve proper compaction. Additionally, stabilization of the excavation bottom may be required prior to the placement and compaction of engineered fill.
- 7.1.6 It is anticipated that stable excavations for the recommended grading associated with the proposed structures can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures may be necessary in order to maintain lateral support of offsite improvements. Excavation recommendations are provided in the *Temporary Excavations* section of this report (Section 7.13).
- 7.1.7 It should be noted that implementation of the recommendations presented herein is not intended to completely prevent damage to the structure during the occurrence of strong ground shaking as a result of nearby earthquakes. It is intended that the structure be designed in such a way that the amount of damage incurred as a result of strong ground shaking be minimized.
- 7.1.8 It is recommended that flexible utility connections be considered for all rigid utilities to minimize or prevent damage to utilities from minor differential movements.

- 7.1.9 Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied-in to the proposed structure, may be supported on conventional foundations bearing on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils at or below a depth of 2½ feet and should be deepened as necessary to maintain a minimum of 12-inch embedment into recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved in writing by a Geocon representative.
- 7.1.9 Cast-in-place friction piles may be utilized for support of proposed lighting improvements, as well as proposed shade structures, provided foundations derive support in the competent alluvium found at or below a depth of 3 feet. Recommendations for friction pile design are provided in Section 6.7 of this report.
- 7.1.10 Where new paving is to be placed, it is recommended that all existing fill soils and soft alluvial soils be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing fill in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable soils may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of soil should be scarified and properly compacted for paving support. Paving recommendations are provided in the *Preliminary Pavement Recommendations* section of this report (see Section 7.12).
- 7.1.11 Based on the results of the percolation testing, stormwater infiltration is not considered feasible for this project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency. Discussion on the percolation test results are provided in Section 7.15.
- 7.1.12 Once the design and foundation loading configuration for the proposed structures proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary. Based on the final foundation loading configurations, the potential for settlement should be re-evaluated by this office.

7.1.13 Any changes in the design, location or elevation, as outlined in this report, should be reviewed by this office. Geocon should be contacted to determine the necessity for review and possible revision of this report.

7.1.14 The most recent ASTM standards apply to this project and must be utilized, even if older ASTM standards are indicated in this report.

7.2 Soil and Excavation Characteristics

7.2.1 The in-situ soils can be excavated with moderate effort using conventional excavation equipment. Some caving should be anticipated in unshored excavations, especially where granular soils are encountered.

7.2.2 It is the responsibility of the contractor to ensure that all excavations and trenches are properly shored and maintained in accordance with applicable OSHA rules and regulations to maintain safety and maintain the stability of existing adjacent improvements.

7.2.3 All onsite excavations must be conducted in such a manner that potential surcharges from existing structures, construction equipment, and vehicle loads are resisted. The surcharge area may be defined by a 1:1 projection down and away from the bottom of an existing foundation or vehicle load. Penetrations below this 1:1 projection will require special excavation measures such as sloping or shoring. Excavation recommendations are provided in the *Temporary Excavations* section of this report (see Section 7.13).

7.2.4 The upper five feet of existing site soils encountered during the investigation are considered to have a “low” expansive potential ($EI = 27$) and are classified as “expansive” in accordance with the 2022 California Building Code (CBC) Section 1803.5.3. The recommendations presented herein assume that foundations and slabs as well as new paving and hardscape will derive support in these materials.

7.3 Minimum Resistivity, pH, and Water-Soluble Sulfate

7.3.1 Potential of Hydrogen (pH) and resistivity testing as well as chloride content testing were performed on representative samples of soil to generally evaluate the corrosion potential to surface utilities. The tests were performed in accordance with California Test Method Nos. 643 and 422 and indicate that the soils are considered “severely corrosive” with respect to corrosion of buried ferrous metals on site. The results are presented in Appendix B (Figure B9) and should be considered for design of underground structures. Due to the corrosive potential of the soils, it is recommended that PVC, ABS or other approved plastic piping be utilized in lieu of cast-iron when in direct contact with the site soils.

- 7.3.2 Laboratory tests were performed on representative samples of the on-site soil to measure the percentage of water-soluble sulfate content. Results from the laboratory water-soluble sulfate tests are presented in Appendix B (Figure B9) and indicate that the on-site soil possess a sulfate exposure class of “S0” to concrete structures as defined by 2022 CBC Section 1904 and ACI 318-19 Chapter 19.
- 7.3.3 Geocon West, Inc. does not practice in the field of corrosion engineering and mitigation. If corrosion sensitive improvements are planned, it is recommended that a corrosion engineer be retained to evaluate corrosion test results and incorporate the necessary precautions to avoid premature corrosion of buried metal pipes and concrete structures in direct contact with the soils.

7.4 Grading

- 7.4.1 Grading is anticipated to include preparation of building pads, excavation of site soils for the foundations and utility trenches, placement of backfill for trenches, and preparation of paving and hardscape subgrade.
- 7.4.2 A preconstruction conference should be held at the site prior to the beginning of excavation operations with the owner, contractor, civil engineer, geotechnical engineer, and building official in attendance. Special soil handling requirements can be discussed at that time.
- 7.4.3 Earthwork should be observed, and compacted fill tested by representatives of Geocon West, Inc. The existing fill and alluvial soil encountered during exploration are suitable for re-use as an engineered fill, provided any encountered oversize material (greater than 6 inches) and any encountered deleterious debris are removed.
- 7.4.4 Grading should commence with the removal of all existing vegetation and existing improvements from the area to be graded. Deleterious debris such as wood and root structures should be exported from the site and should not be mixed with the fill soils. Asphalt and concrete should not be mixed with the fill soils unless approved by the Geotechnical Engineer. All existing underground improvements planned for removal should be completely excavated and the resulting depressions properly backfilled in accordance with the procedures described herein. Once a clean excavation bottom has been established it must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.).

- 7.1.4 As a minimum, it is recommended that the upper 3 feet of existing earth materials within proposed habitable structure footprint areas be excavated and properly compacted for foundation and slab support. Deeper excavations should be conducted as needed to remove any deeper fill or soft soils as necessary at the direction of the Geotechnical Engineer (a representative of Geocon). The excavation should extend laterally a minimum distance of 3 feet beyond the building footprint areas, including building appurtenances, or a distance equal to the depth of fill below the foundation, whichever is greater. The limits of existing fill and/or soft soil removal will be verified by the Geocon representative during site grading activities. All excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).
- 7.4.5 Based on the conditions encountered during site exploration, areas of soft and overly moist soils may be encountered during grading operations. These conditions may change seasonally. If the soils are more than 3 percent above the optimum moisture content at the time of construction the soils will likely require some spreading and drying activities in order to achieve proper compaction.
- 7.4.6 Prior to placing fill, a stable excavation bottom must be established. In areas where the subgrade is saturated or soft, proper compaction will likely not be possible or achieved in a timely manner without introducing stabilization measures. If subgrade stabilization is required at the excavation bottom, rubber tire equipment should not be allowed in the excavation bottom until it is stabilized or extensive soil disturbance could result. It is suggested that excavation and grading be performed during the summer season to promote moisture control of the soils. In addition, the use of track equipment should be used to minimize disturbance to the soils at the excavation bottom.
- 7.4.7 Bottom stabilization, if necessary, may be achieved placing a thin lift of 3- to 6-inch-diameter crushed angular rock into the soft excavation bottom. The use of crushed concrete will also be acceptable. The crushed rock should be spread thinly across the excavation bottom and pressed into the soils by track rolling or wheel rolling with heavy equipment. It is very important that voids between the rock fragments are not created so the rock must be thoroughly pressed or blended into the soils. All subgrade soils must be properly compacted and proof-rolled in the presence of the Geotechnical Engineer (a representative of Geocon West, Inc.).

- 7.4.8 An additional method of subgrade stabilization would be to place a minimum 12 inches thick layer of aggregate base over Tensar InterAx NX850 geogrid or equivalent extruded (nonwoven) geotextile. The Tensar geogrids should be installed taught and should overlap in accordance with the manufacturer's recommendations. Prior to placing the geogrid, excessively soft or wet materials should be removed and the resulting excavation bottom should be free of loose material. Non-vibratory compaction methods should be used for compaction of the base material. The aggregate base should be compacted to a dry density of at least 95 percent of the laboratory maximum density near the optimum moisture. If pumping of the subgrade continues, a thicker layer of aggregate base may be placed. It is very important that subgrade stabilization be performed uniformly across the entire excavation bottom.
- 7.4.9 Stable excavations for the recommended grading can be achieved with sloping measures. However, if excavations in close proximity to an adjacent property line and/or structure are required, special excavation measures such as slot cutting may be necessary in order to maintain lateral support of the existing offsite improvements. Excavation recommendations are provided in the Temporary Excavations section of this report (Section 7.13).
- 7.4.10 All fill and backfill soils should be placed in horizontal loose layers approximately 6 to 8 inches thick, moisture conditioned to near to slightly above optimum moisture content, and properly compacted to a minimum 90 percent of the maximum dry density in accordance with ASTM D 1557 (latest edition).
- 7.4.11 Although not anticipated for this project, all imported fill shall be observed, tested, and approved by Geocon West, Inc. prior to bringing soil to the site. Import fill should consist of the characteristics presented in the table below.

SUMMARY OF IMPORT FILL RECOMMENDATIONS

Soil Characteristic	Values
Expansion Potential	"Low" (Expansion Index of 50 or less)
Particle Size	Maximum Dimension Less Than 6 Inches
	Free of Debris
Corrosivity	Less Detrimental Than Existing Onsite Soils

- 7.4.12. Where new paving is to be placed, it is recommended that all existing fill and soft alluvium be excavated and properly compacted for paving support. As a minimum, the upper 12 inches of soil should be scarified, moisture conditioned to near to slightly above optimum moisture content, and compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Paving recommendations are provided in *Preliminary Pavement Recommendations* section of this report (see Section 7.12).
- 7.4.13. Foundations for small outlying structures, such as block walls up to 6 feet high, planter walls or trash enclosures, which will not be tied to the proposed building, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and proper compaction cannot be performed, foundations may derive support directly in the undisturbed alluvial soils at and below a depth of 2½ feet below the existing ground surface and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials. If the soils exposed in the excavation bottom are soft or loose, compaction of the soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative.
- 7.4.14. Utility trenches should be properly backfilled in accordance with the following requirements. The pipe should be bedded with clean sands (Sand Equivalent greater than 30) to a depth of at least 1 foot over the pipe, and the bedding material must be inspected and approved in writing by the Geotechnical Engineer (a representative of Geocon). The use of gravel is not acceptable unless used in conjunction with filter fabric to prevent the gravel from having direct contact with soil. The remainder of the trench backfill may be derived from onsite soil or approved import soil, compacted as necessary, until the required compaction is obtained. The use of minimum 2-sack slurry as backfill is also acceptable. Prior to placing any bedding materials or pipes, the excavation bottom must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon).
- 7.4.15. All trench and foundation excavation bottoms must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing bedding materials, fill, steel, gravel, or concrete.

7.5 Shrinkage

- 7.5.1 Shrinkage results when a volume of material removed at one density is compacted to a higher density. A shrinkage factor of between 10 and 18 percent should be anticipated when excavating and compacting the upper few feet of existing earth materials on the site to an average relative compaction of 92 percent.
- 7.4.2 If import soils will be utilized in the building pad, the soils must be placed uniformly and at equal thickness at the direction of the Geotechnical Engineer (a representative of Geocon West, Inc.). Soils can be borrowed from non-building pad areas and later replaced with imported soils.

7.6 Conventional Foundation Design

- 7.6.1 Once the foundation loading configuration for the proposed structures proceeds to a more finalized plan, the recommendations within this report should be reviewed and revised, if necessary.
- 7.6.2 A conventional shallow spread foundation system may be utilized for support of the proposed structures provided foundations derive support in a blanket of newly placed engineered fill. All foundation excavations must be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon), prior to placing steel or concrete.
- 7.6.3 Conventional shallow spread foundations for proposed structures should consist of continuous strip footings and/or isolated spread footings and should be designed using the parameters in the table on the following page.

SUMMARY OF FOUNDATION RECOMMENDATIONS

Parameter	Value
Minimum Continuous Foundation Width	12 Inches
Minimum Isolated Foundation Width	24 Inches
Minimum Foundation Depth	18 Inches Below Lowest Adjacent Grade & 12 Inches into Recommended Bearing Material
Minimum Steel Reinforcement	4 No. 4 Bars, 2 Top and 2 Bottom
Allowable Bearing Capacity – Continuous Foundation	2,000 psf
Allowable Bearing Capacity – Isolated Foundation	2,500 psf
Bearing Capacity Increase	250 psf per Foot of Width
	600 psf per Foot of Depth
Maximum Allowable Bearing Capacity	3,500 psf
Estimated Total Settlement	½ Inch
Estimated Differential Settlement	¼ Inch over 20 Feet

- 7.6.4 The above foundation dimensions and minimum reinforcement recommendations are based on soil conditions and building code requirements only and are not intended to be used in lieu of those required for structural purposes.
- 7.6.5 The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.
- 7.6.6 The enclosed liquefaction analyses indicate that the site soils could be susceptible to up to approximately 2 inches of total settlement as a result of the MCE peak ground acceleration (PGA_M). The resulting differential settlement at the ground surface is anticipated to be less than 1 inch over a distance of 30 feet. These settlements are in addition to the static settlements indicated in the table above and must be considered in the structural design.
- 7.6.7 Once the design and foundation loading configurations for the proposed structures proceeds to a more finalized plan, the estimated settlements presented in this report should be reviewed and revised, if necessary. If the final foundation loading configurations are greater than the assumed loading conditions, the potential for settlement should be reevaluated by this office.
- 7.6.8 No special subgrade presaturation is required prior to placement of concrete. However, the moisture in the foundation subgrade should be sprinkled as necessary to maintain a moist condition at the time of concrete placement.

- 7.6.9 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated. If unanticipated soil conditions are encountered, foundation modifications may be required.
- 7.6.10 This office should be provided with a copy of the final construction plans so that the excavation recommendations presented herein could be properly reviewed and revised if necessary.

7.7 Miscellaneous Foundations

- 7.7.1 Foundations for small outlying structures, such as block walls up to 6 feet in height, planter walls or trash enclosures, which will not be tied to the proposed structure, may be supported on conventional foundations deriving support on a minimum of 12 inches of newly placed engineered fill which extends laterally at least 12 inches beyond the foundation area. Where excavation and compaction cannot be performed, foundations may derive support directly in the competent undisturbed alluvial soils at and below a depth of 2½ feet and should be deepened as necessary to maintain a minimum 12-inch embedment into the recommended bearing materials.
- 7.7.2 If the soils exposed in the excavation bottom are soft, compaction of the soft soils will be required prior to placing steel or concrete. Compaction of the foundation excavation bottom is typically accomplished with a compaction wheel or mechanical whacker and must be observed and approved by a Geocon representative. Miscellaneous foundations may be designed for a bearing value of 1,500 psf and should be a minimum of 12 inches in width, 18 inches in depth below the lowest adjacent grade and 12 inches into the recommended bearing material. The allowable bearing pressure may be increased by up to one-third for transient loads due to wind or seismic forces.
- 7.7.3 Foundation excavations should be observed and approved in writing by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to the placement of reinforcing steel and concrete to verify that the excavations and exposed soil conditions are consistent with those anticipated.

7.8 Lateral Design

- 7.8.1 Resistance to lateral loading may be provided by friction acting at the base of foundations, slabs and by passive earth pressure. An allowable coefficient of friction of 0.35 may be used with the dead load forces in the newly placed engineered fill or alluvial soils.
- 7.8.2 Passive earth pressure for the sides of foundations and slabs poured against newly placed engineered fill or undisturbed alluvial soils may be computed as an equivalent fluid having a density of 230 pounds per cubic foot (pcf) with a maximum earth pressure of 2,300 psf. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third. A one-third increase in the passive value may be used for wind or seismic loads.

7.9 Friction Pile Foundations – Light Standards

- 7.9.1 Typical light standards are between 10 and 15 feet in height and are supported on pile foundations. Cast-in-place friction piles may be utilized for support of the proposed lighting poles as well as proposed shade structures provided foundations derive support in or the competent alluvium found at and below a depth of 3 feet.
- 7.9.2 Friction piles should be a minimum of 18 inches in diameter and should be embedded a minimum of 6 feet into the recommended bearing materials (undisturbed alluvium). Where not protected by pavement, the upper two feet of soil should be ignored when calculating axial and lateral capacity.
- 7.9.3 Friction piles may be designed based on a skin friction capacity of 100 pounds per square foot and do not require the complete removal of all loose earth materials from the bottom of the excavation since the end-bearing capacity is not being considered for design. However, a cleanout of the excavation bottom will be required. Piles may be assumed fixed at an embedment depth of 5 feet below the ground surface. Single pile uplift capacity can be taken as 60 percent of the allowable downward capacity. The allowable downward capacity and allowable uplift capacity may be increased by one-third when considering transient wind or seismic loads.

- 7.9.4 For design purposes, an allowable passive value for the soils may be assumed to be 230 psf per foot with a maximum earth pressure of 2,300 psf. To develop the full lateral value, provisions should be implemented to assure firm contact between the piles and the engineered fill and underlying alluvium. A one-third increase in the passive value may be used for wind or seismic loads. The allowable capacity may be doubled for isolated piles spaced more than three times the diameter on-center.
- 7.9.5 All drilled pile excavations should be continuously observed by personnel of this firm to verify adequate penetration into the recommended bearing materials. The capacity presented is based on the strength of the soils. The compressive and tensile strength of the pile sections should be checked to verify the structural capacity of the piles.

7.10 Pile Installation

- 7.10.1 Caving may occur in granular soils and the contractor should have casing available prior to commencement of pile excavation. When casing is used, extreme care should be employed so that the pile is not pulled apart as the casing is withdrawn. At no time should the distance between the surface of the concrete and the bottom of the casing be less than 5 feet. Continuous observation of the drilling and pouring of the piles by the Geotechnical Engineer (a representative of Geocon West, Inc.), is required.
- 7.10.2 Groundwater was encountered during exploration at a depth of 31 feet beneath the ground surface. Groundwater is not expected to be encountered during construction. However, should groundwater or seepage be encountered during pile installation, the contractor should be prepared. Piles placed below the water level require the use of a tremie to place the concrete into the bottom of the hole. A tremie should consist of a rigid, water-tight tube having a diameter of not less than 6 inches with a hopper at the top. The tube should be equipped with a device that will close the discharge end and prevent water from entering the tube while it is being charged with concrete. The tremie should be supported so as to permit free movement of the discharge end over the entire top surface of the work and to permit rapid lowering when necessary to retard or stop the flow of concrete. The discharge end should be closed at the start of the work to prevent water entering the tube and should be entirely sealed at all times, except when the concrete is being placed. The tremie tube should be kept full of concrete. The flow should be continuous until the work is completed and the resulting concrete seal should be monolithic and homogeneous. The tip of the tremie tube should always be kept about 5 feet below the surface of the concrete and definite steps and safeguards should be taken to insure that the tip of the tremie tube is never raised above the surface of the concrete.

- 7.10.3 A special concrete mix should be used for concrete to be placed below water. The design should provide for concrete with an unconfined compressive strength psi of 1,000 pounds per square inch (psi) over the initial job specification. An admixture that reduces the problem of segregation of paste/aggregates and dilution of paste should be included. The slump should be commensurate to any research report for the admixture, provided that it should also be the minimum for a reasonable consistency for placing when water is present.
- 7.10.4 Closely spaced piles should be drilled and filled alternately, with the concrete permitted to set at least eight hours before drilling an adjacent hole. Unless the holes are fully cased from top to bottom, it is not recommended that holes be left open overnight.

7.11 Concrete Slabs-on-Grade

- 7.11.1 Concrete slabs-on-grade subject to vehicle loading should be designed in accordance with the recommendations in the Preliminary Pavement Recommendations section of this report (Section 7.12).
- 7.11.2 Subsequent to the recommended grading, concrete slabs-on-grade for structures, not subject to vehicle loading, should be a minimum of 4 inches thick and minimum slab reinforcement should consist of No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Steel reinforcing should be positioned vertically near the slab midpoint.

- 7.11.3 Slabs-on-grade at the ground surface that may receive moisture-sensitive floor coverings or may be used to store moisture-sensitive materials should be underlain by a vapor retarder placed directly beneath the slab. The vapor retarder and acceptable permeance should be specified by the project architect or developer based on the type of floor covering that will be installed. The vapor retarder selection and design should be consistent with the guidelines presented in Section 9.3 of the American Concrete Institute's (ACI) Guide for Concrete Slabs that Receive Moisture-Sensitive Flooring Materials (ACI 302.2R-06) as well as ASTM E1745 and should be installed in general conformance with ASTM E 1643 (latest edition) and the manufacturer's recommendations. A minimum thickness of 15 mils extruded polyolefin plastic is recommended; vapor retarders which contain recycled content or woven materials are not recommended. The vapor retarder should have a permeance of less than 0.01 perms demonstrated by testing before and after mandatory conditioning is recommended. The vapor retarder should be installed in direct contact with the concrete slab with proper perimeter seal. If the California Green Building Code requirements apply to this project, the vapor retarder should be underlain by 4 inches of clean aggregate. It is important that the vapor retarder be puncture resistant since it will be in direct contact with angular gravel. As an alternative to the clean aggregate suggested in the Green Building Code, it is our opinion that the concrete slab-on-grade may be underlain by a vapor retarder over 4-inches of clean sand (sand equivalent greater than 30), since the sand will serve a capillary break and will minimize the potential for punctures and damage to the vapor barrier.
- 7.11.4 For seismic design purposes, a coefficient of friction of 0.35 may be utilized between concrete slabs and subgrade soils without a moisture barrier, and 0.15 for slabs underlain by a moisture barrier.
- 7.11.5 Exterior slabs for walkways and flatwork, not subject to traffic loads, should be at least 4 inches thick and reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions, positioned near the slab midpoint. Prior to construction of slabs, the upper 12 inches of subgrade should be moistened to near to slightly above optimum moisture content and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition). Crack control joints should be spaced at intervals not greater than 10 feet and should be constructed using saw-cuts or other methods as soon as practical following concrete placement. Crack control joints should extend a minimum depth of one-fourth the slab thickness. Construction joints should be designed by the project structural engineer.

- 7.11.6 The moisture content of the slab subgrade should be maintained and sprinkled as necessary to maintain a moist condition as would be expected in any concrete placement.
- 7.11.7 The recommendations of this report are intended to reduce the potential for cracking of slabs due to settlement. However, even with the incorporation of the recommendations presented herein, foundations, stucco walls, and slabs-on-grade may exhibit some cracking due to minor soil movement and/or concrete shrinkage. The occurrence of concrete shrinkage cracks is independent of the supporting soil characteristics. Their occurrence may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement and curing, and by the placement of crack control joints at periodic intervals, in particular, where re-entrant slab corners occur.

7.12 Preliminary Pavement Recommendations

- 7.12.1 Where new paving is to be placed, it is recommended that all existing fill and soft alluvium materials be excavated and properly compacted for paving support. The client should be aware that excavation and compaction of all existing artificial fill and soft alluvium in the area of new paving is not required; however, paving constructed over existing uncertified fill or unsuitable alluvium material may experience increased settlement and/or cracking, and may therefore have a shorter design life and increased maintenance costs. As a minimum, the upper 12 inches of paving subgrade should be scarified, moisture conditioned to near to slightly above optimum moisture content, and properly compacted to at least 95 percent relative compaction, as determined by ASTM Test Method D 1557 (latest edition).
- 7.12.2 The following pavement sections are based on a R-Value of 25. Once site grading activities are complete an R-Value should be obtained by laboratory testing to confirm the properties of the soils serving as paving subgrade, prior to placing pavement.
- 7.12.3 The Traffic Indices listed below are estimates. Geocon does not practice in the field of traffic engineering. The actual Traffic Index for each area should be determined by the project civil engineer. If pavement sections for Traffic Indices other than those listed below are required, Geocon should be contacted to provide additional recommendations. Pavement thicknesses were determined following procedures outlined in the *California Highway Design Manual* (Caltrans). It is anticipated that the majority of traffic will consist of automobile and large truck traffic.

PRELIMINARY PAVEMENT DESIGN SECTIONS

Location	Estimated Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base (inches)
Automobile Parking and Driveways	4.0	3.0	4.0
Trash Truck & Fire Lanes	7.0	4.0	12.0

- 7.12.4 Asphalt concrete should conform to Section 203-6 of the *“Standard Specifications for Public Works Construction”* (Green Book). Class 2 aggregate base materials should conform to Section 26-1.02A of the *“Standard Specifications of the State of California, Department of Transportation”* (Caltrans). The use of Crushed Miscellaneous Base (CMB) in lieu of Class 2 aggregate base is acceptable. Crushed Miscellaneous Base should conform to Section 200-2.4 of the *“Standard Specifications for Public Works Construction”* (Green Book).
- 7.12.5 Unless specifically designed and evaluated by the project structural engineer, where exterior concrete paving will be utilized for support of vehicles, it is recommended that the concrete be a minimum of 6 inches of concrete reinforced with No. 3 steel reinforcing bars placed 18 inches on center in both horizontal directions. Concrete paving supporting vehicular traffic should be underlain by a minimum of 4 inches of aggregate base and a properly compacted subgrade. The subgrade and base material should be compacted to 95 percent relative compaction as determined by ASTM Test Method D 1557 (latest edition).
- 7.12.6 The performance of pavements is highly dependent upon providing positive surface drainage away from the edge of pavements. Ponding of water on or adjacent to the pavement will likely result in saturation of the subgrade materials and subsequent cracking, subsidence and pavement distress. If planters are planned adjacent to paving, it is recommended that the perimeter curb be extended at least 12 inches below the bottom of the aggregate base to minimize the introduction of water beneath the paving.

7.13 Temporary Excavations

- 7.13.1 Excavations less than 5 feet in height are anticipated for grading and construction activities. The excavations are expected to expose artificial fill and alluvial soils, which may be subject to caving where granular soils are exposed. Temporary vertical excavations up to 5 feet in height may be attempted where loose soils or caving sands are not present, and where excavations are not surcharged by adjacent traffic or structures.

- 7.13.2 Vertical excavations greater than 5 feet or where surcharged by existing structures will require sloping or shoring measures in order to provide a stable excavation. Where sufficient space is available, temporary unsurcharged embankments could be sloped back at a uniform 1:1 slope gradient or flatter up to a maximum of 8 feet in height. A uniform slope does not have a vertical portion.
- 7.13.3 Where temporary construction slopes are utilized, the top of the slope should be barricaded to prevent vehicles and storage loads at the top of the slope within a horizontal distance equal to the height of the slope. If the temporary construction slopes are to be maintained during the rainy season, berms are suggested along the tops of the slopes where necessary to prevent runoff water from entering the excavation and eroding the slope faces. The soils exposed in the cut slopes should be inspected during excavation by our personnel and the contractor's competent person so that modifications of the slopes can be made if variations in the soil conditions occur. All excavations should be stabilized within 30 days of initial excavation.

7.14 Stormwater Infiltration

- 7.14.1 During the site exploration, borings B1, B2, B3 and B4 were used to perform percolation testing. The borings were excavated to depths of approximately 5 to 10 feet below ground surface, as indicated in the table below. Slotted casing was placed in the borings and the borings were then filled with water to pre-saturate the soils. The casing was refilled with water and percolation test readings were performed after repeated flooding of the cased excavation.
- 7.14.2 Based on the test results, the field-measured percolation rate and the design infiltration rate are provided in the following table. The Reduction Factor (Rf), to convert the field-measured percolation rate to an infiltration rate, is also shown in the table below. This value has been calculated in accordance with the Small Diameter Boring Infiltration Test Procedure in the County of Los Angeles Department of Public Works GMED *Guidelines for Design, Investigation, and Reporting Low Impact Development Stormwater Infiltration* (June 2021). Calculations of the percolation rate, reduction factor, and infiltration rate are provided on Figures 8 through 11.

Boring	Infiltration Depth (ft)	Field Measured Infiltration Rate (in/hour)	Reduction Factor (Rf)	Design Infiltration Rate (in / hour)
B1	5-10	0.11	3.0	0.04
B2	2-5	0.04	3.0	0.01
B3	2-5	0.06	3.0	0.02
B4	2-5	0.01	3.0	0.0

7.14.3 The results of the percolation testing indicated that the infiltration rate within the alluvial soils is less than the accepted minimum accepted infiltration rate of 0.3 inches per hour. Therefore, based on the results of percolation testing performed at the site, a stormwater infiltration system is not recommended for this project. It is recommended that stormwater be retained, filtered, and discharged in accordance with the requirements of the local governing agency.

7.15 Surface Drainage

7.15.1 Proper surface drainage is critical to the future performance of the project. Uncontrolled infiltration of irrigation excess and storm runoff into the soils can adversely affect the performance of the planned improvements. Saturation of a soil can cause it to lose internal shear strength and increase its compressibility, resulting in a change in the original designed engineering properties. Proper drainage should be maintained at all times.

7.15.2 All site drainage should be collected and controlled in non-erosive drainage devices. Drainage should not be allowed to pond anywhere on the site, and especially not against any foundation or retaining wall. The site should be graded and maintained such that surface drainage is directed away from structures in accordance with 2022 CBC 1804.4 or other applicable standards. In addition, drainage should not be allowed to flow uncontrolled over any descending slope. Discharge from downspouts, roof drains and scuppers are not recommended onto unprotected soils within 5 feet of the building perimeter. Planters which are located adjacent to foundations should be sealed to prevent moisture intrusion into the soils providing foundation support. Landscape irrigation is not recommended within 5 feet of the building perimeter footings except when enclosed in protected planters.

7.15.3 Positive site drainage should be provided away from structures, pavement, and the tops of slopes to swales or other controlled drainage structures.

- 7.15.4 Landscaping planters immediately adjacent to paved areas are not recommended due to the potential for surface or irrigation water to infiltrate the pavement's subgrade and base course. Either a subdrain, which collects excess irrigation water and transmits it to drainage structures, or impervious above-grade planter boxes should be used. In addition, where landscaping is planned adjacent to the pavement, it is recommended that consideration be given to providing a cutoff wall along the edge of the pavement that extends at least 12 inches below the base material.

7.16 Plan Review

- 7.16.1 Grading, foundation, and shoring plans should be reviewed by the Geotechnical Engineer (a representative of Geocon West, Inc.), prior to finalization to verify that the plans have been prepared in substantial conformance with the recommendations of this report and to provide additional analyses or recommendations.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

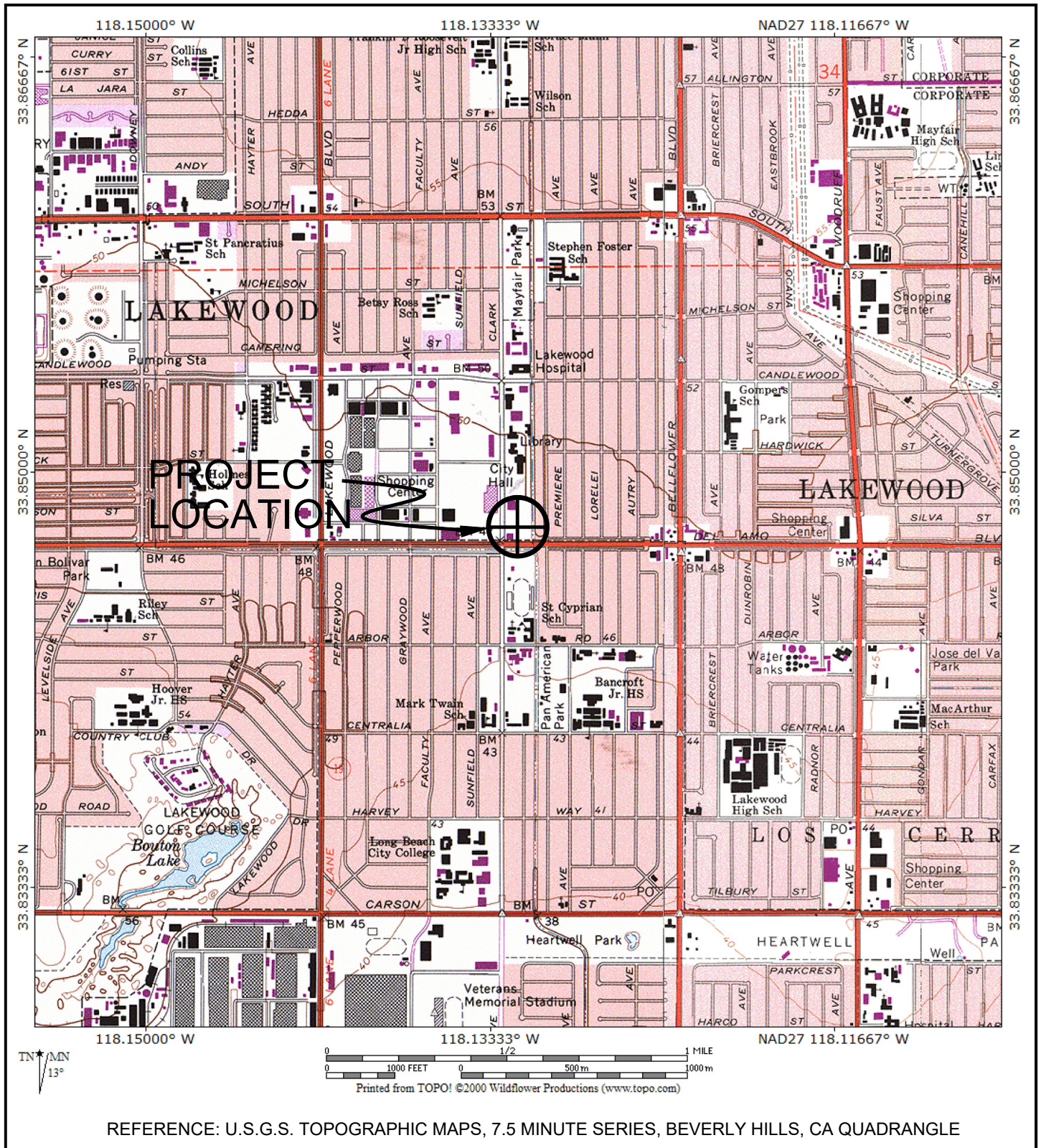
1. The firm that performed the geotechnical investigation for the project should be retained to provide testing and observation services during construction to provide continuity of geotechnical interpretation and to check that the recommendations presented for geotechnical aspects of site development are incorporated during site grading, construction of improvements, and excavation of foundations. If another geotechnical firm is selected to perform the testing and observation services during construction operations, that firm should prepare a letter indicating their intent to assume the responsibilities of project geotechnical engineer of record. A copy of the letter should be provided to the regulatory agency for their records. In addition, that firm should provide revised recommendations concerning the geotechnical aspects of the proposed development, or a written acknowledgement of their concurrence with the recommendations presented in our report. They should also perform additional analyses deemed necessary to assume the role of Geotechnical Engineer of Record.
2. The recommendations of this report pertain only to the site investigated and are based upon the assumption that the soil conditions do not deviate from those disclosed in the investigation. If any variations or undesirable conditions are encountered during construction, or if the proposed construction will differ from that anticipated herein, Geocon Incorporated should be notified so that supplemental recommendations can be given. The evaluation or identification of the potential presence of hazardous or corrosive materials was not part of the scope of services provided by Geocon Incorporated.
3. This report is issued with the understanding that it is the responsibility of the owner or his representative to ensure that the information and recommendations contained herein are brought to the attention of the architect and engineer for the project and incorporated into the plans, and the necessary steps are taken to see that the contractor and subcontractors carry out such recommendations in the field.
4. The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

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VICINITY MAP

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

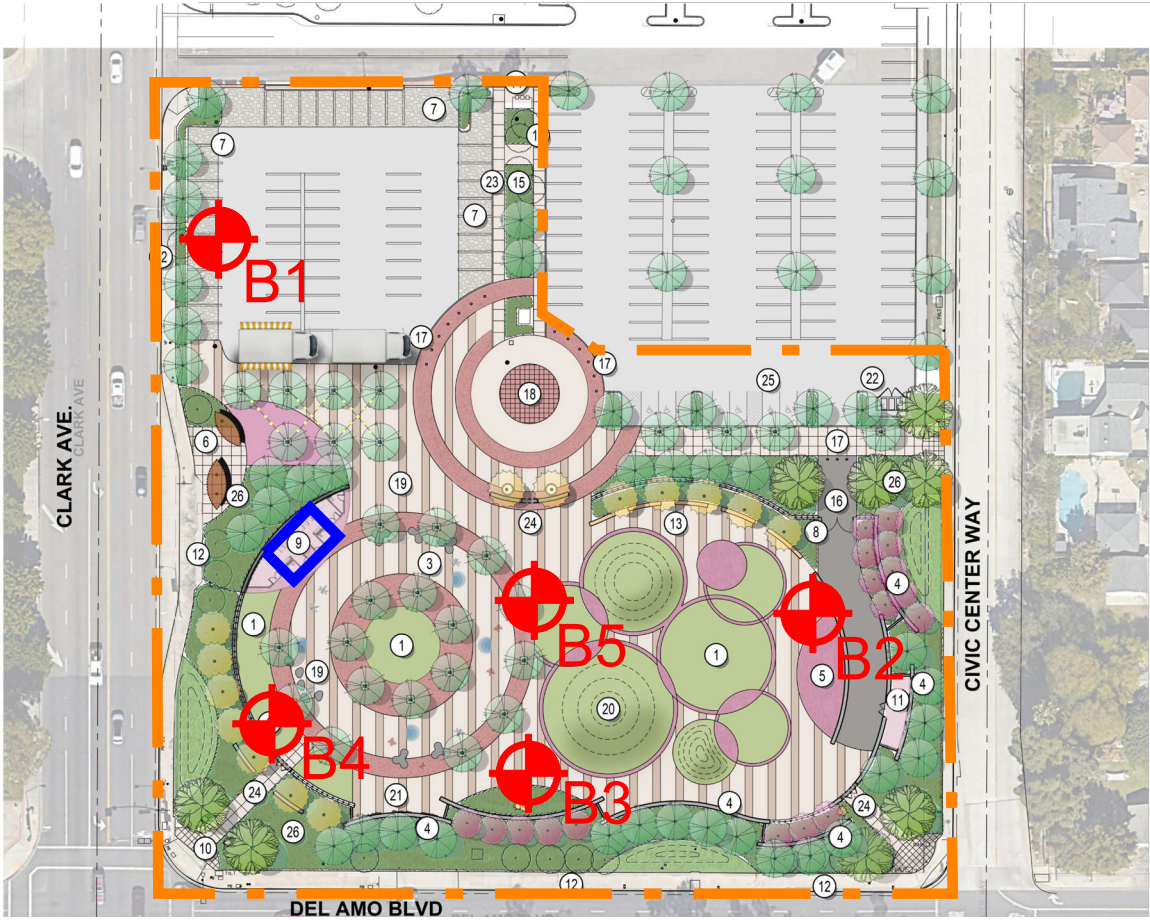
SEPT 2024

PROJECT NO. W1974-06-01

FIG. 1

LEGEND

-  B5
- Approximate Boring Locations
- 
- Approximate Project Limits
- 
- Proposed Restroom



Plan Option 1

- LEGEND
- 1

FLAT SYNTHETIC TURF

2

CHRISTMAS TREE AREA

3

ACCENT PAVING

4

10' HIGH SOUND WALL

5

MOBILE STAGE (14' X 36")

6

BUS SHELTER AND SHADE AREA

7

PERMEABLE PAVING

8

6' HIGH WROUGHT IRON FENCE WITH MOW CURB

9

RESTROOM

10

PARK ENTRY SIGN

11

STORAGE ROOM

12

EXISTING STREET TREES

13

SEAT WALLS
- 14

EXISTING TRASH ENCLOSURE

15

EXISTING EUCALYPTUS TREES

16

SERVICE ACCESS WITH LOCKING GATE

17

BOLLARDS

18

REMOVABLE CHRISTMAS TREE

19

ARTISTIC PLAY FEATURE

20

MOUNDED SYNTHETIC TURF FOR STAGE SEATING AND PLAY

21

CHILDREN'S ARTISTIC WALL

22

PROPOSED TRASH ENCLOSURE

23

FORMAL ACCESS THROUGH EXISTING PARKING LOT

24

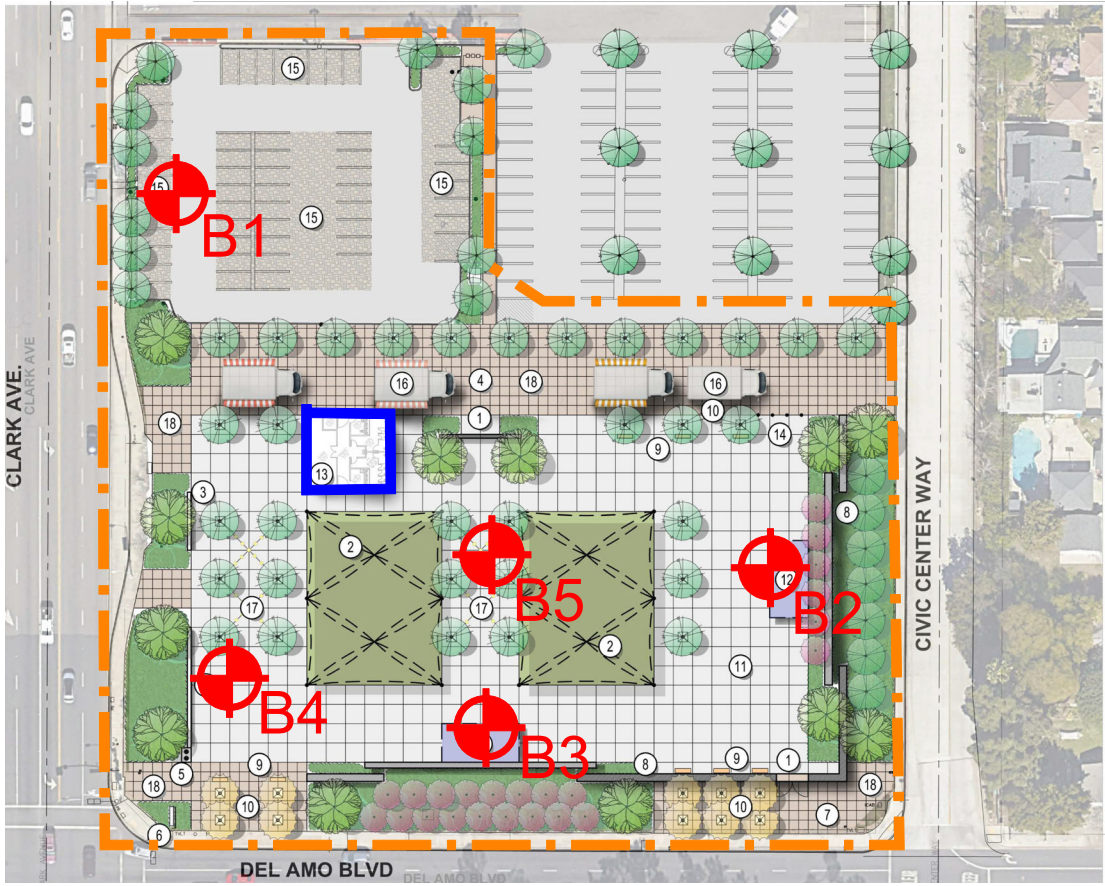
VERTICAL GATEWAY FEATURE

25

PROPOSED PARKING

26

LOW WATER TREES BUFFER AND PLANTING



Plan Option 2

- LEGEND
- 1

GATEWAY ARCH

2

SHADE OVER SYNTHETIC TURF

3

LOW WALL

4

CHRISTMAS TREE AREA

5

FLAG LOCATION

6

PARK SIGN

7

OPEN TO STREET

8

SOUND WALL

9

CONCRETE BENCH (TYP.)
- 10

TREE WELL (TYP.)

11

10'X10' PAVING JOINT GRID

12

STAGE LOCATION

13

RESTROOM

14

REMOVABLE BOLLARDS FOR TRUCK ACCESS

15

PERMEABLE PAVING

16

FOOD TRUCK PARKING

17

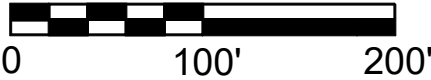
BISTRO LIGHTING OPTIONAL DINING AREA

18

5'X5' PAVING JOINT GRID



NORTH



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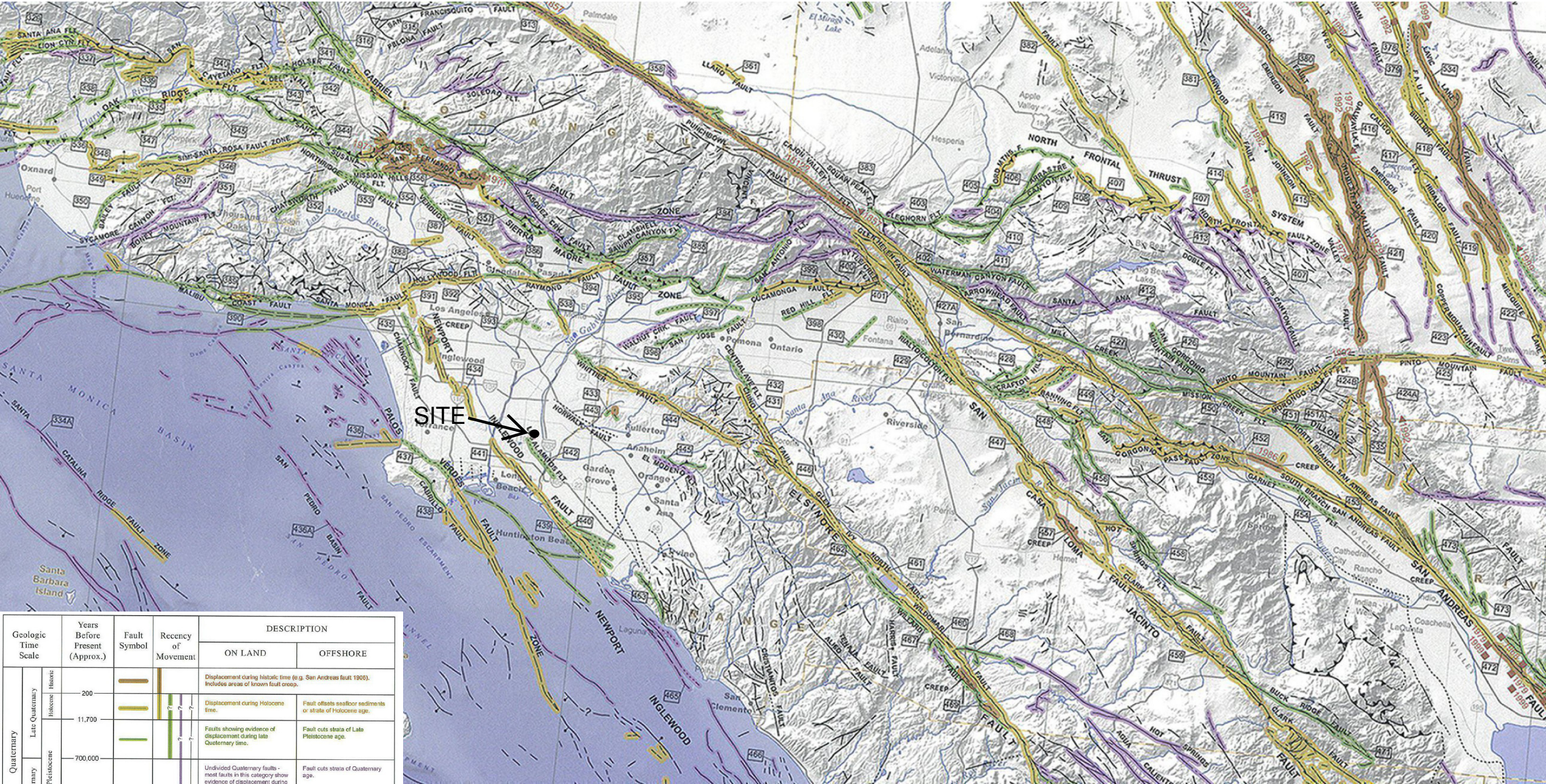
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SITE PLAN

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

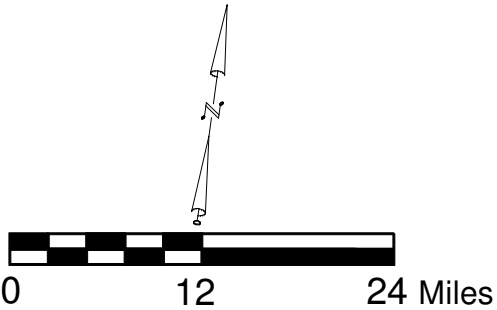
SEPT 2024 PROJECT NO. W1974-06-01 FIG. 2

Reference: Jennings, C.W. and Bryant, W. A., 2010, Fault Activity Map of California, California Geological Survey Geologic Data Map No. 6.



Geologic Time Scale	Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
				ON LAND	OFFSHORE
Quaternary	Late Quaternary Holocene	—	—	Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
	11,700	—	—	Displacement during Holocene time.	Fault offsets seafloor sediments or strata of Holocene age.
	700,000	—	—	Faults showing evidence of displacement during late Quaternary time.	Fault cuts strata of Late Pleistocene age.
Early Quaternary Pleistocene	1,600,000	—	—	Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age.	Fault cuts strata of Quaternary age.
Pre-Quaternary	4.5 billion (Age of Earth)	—	—	Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive.	Fault cuts strata of Pliocene or older age.

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.



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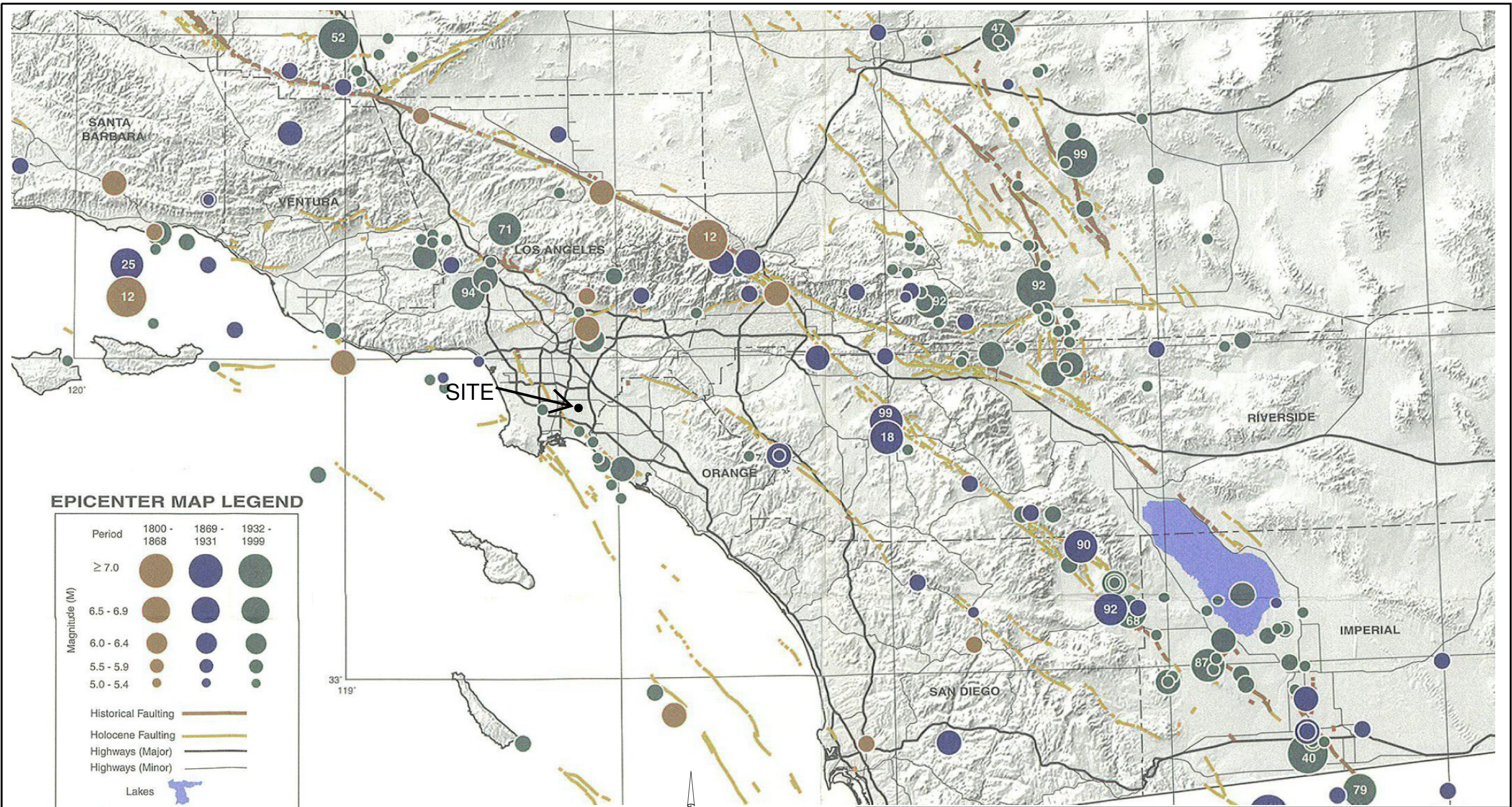
REGIONAL FAULT MAP

4900 CLARK AVENUE!
LAKEWOOD, CALIFORNIA

SEPT 2024

PROJECT NO. W1974-06-01

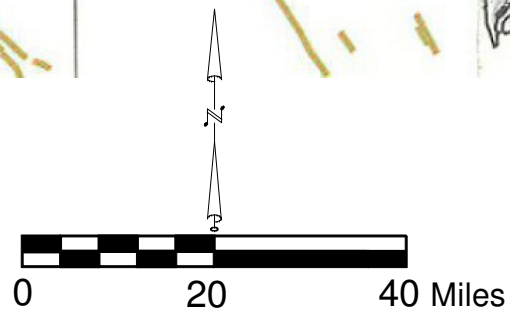
FIG. 3



EPICENTER MAP LEGEND

Period	1800 - 1868	1869 - 1931	1932 - 1999
≥ 7.0			
6.5 - 6.9			
6.0 - 6.4			
5.5 - 5.9			
5.0 - 5.4			
Historical Faulting			
Holocene Faulting			
Highways (Major)			
Highways (Minor)			
Lakes			
	Last two digits of M ≥ 6.5 earthquake year		

Reference: Toppozada, T., Branum, D., Petersen, M., Hallstrom, C., Cramer, C., and Reichle, M., 2000, Epicenters and Areas Damaged by M $\geq$ 5 California Earthquakes, 1800 - 1999, California Geological Survey, Map Sheet 49.



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REGIONAL SEISMICITY MAP

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

SEPT 2024

PROJECT NO. W1974-06-01

FIG.4

EMPIRICAL ESTIMATION OF LIQUEFACTION POTENTIAL MAXIMUM CONSIDERED EARTHQUAKE

NCEER (1996) METHOD W 2001 UPDATES

EARTHQUAKE INFORMATION:

Earthquake Magnitude:	7.30
Peak Horiz. Acceleration PGA_M (g):	0.738
Magnitude Scaling Factor:	1.071
Historic High Groundwater:	20.0
Groundwater Depth During Exploration:	31.0

ENERGY & ROD CORRECTIONS:

Energy Correction (CE) for N60:	1.25
Rod Len.Corr.(CR) (0=no or 1=yes):	1
Bore Dia. Corr. (CB):	1.00
Sampler Corr. (CS):	1.20
Use Ksigma (0=no or 1=yes):	1

Boring ID: B5

LIQUEFACTION CALCULATIONS:

Unit Wt. Water (pcf): 62.4															
Depth to Base (ft)	Total Unit Wt. (pcf)	Water (0 or 1)	Field SPT (N)	Depth of SPT (ft)	Liq.Sus. (0 or 1)	-200 (%)		Est. Dr (%)	CN Factor	Corrected (N1)60cs	Eff. Unit Wt. (psf)	Resist. CRR 7.5	rd Factor	Induced CSR	Liquefac. Safe.Fact.
1.0	125.0	0	6.0	1.0	1	50	*	59	1.700	18.8	125.0	0.201	1.000	0.480	--
2.0	125.0	0	6.0	2.0	1	50	*	58	1.700	18.8	125.0	0.201	0.998	0.479	--
3.0	125.0	0	6.0	3.0	1	50	*	56	1.700	18.8	125.0	0.201	0.996	0.478	--
4.0	125.0	0	6.0	4.0	1	50	*	55	1.700	18.8	125.0	0.201	0.994	0.477	--
5.0	125.0	0	6.0	5.0	1	50	*	53	1.700	18.8	125.0	0.201	0.991	0.476	--
6.0	125.0	0	6.0	6.0	1	50	*	52	1.700	18.8	125.0	0.201	0.989	0.474	--
7.0	125.0	0	6.0	7.0	1	50	*	51	1.603	18.0	125.0	0.192	0.987	0.473	--
8.0	125.0	0	6.0	8.0	1	50	*	50	1.492	17.1	125.0	0.182	0.985	0.472	--
9.0	125.0	0	6.0	9.0	1	50	*	49	1.402	16.4	125.0	0.174	0.982	0.471	--
10.0	125.0	0	8.0	12.5	1	50	*	53	1.326	19.3	125.0	0.207	0.980	0.470	--
11.0	125.0	0	8.0	12.5	1	50	*	53	1.261	18.6	125.0	0.199	0.978	0.469	--
12.5	125.0	0	8.0	12.5	1	50	*	53	1.192	17.9	125.0	0.190	0.975	0.468	--
13.0	125.0	0	8.0	12.5	1	50	*	53	1.168	17.6	125.0	0.187	0.973	0.467	--
14.0	125.0	0	8.0	12.5	1	50	*	53	1.112	17.0	125.0	0.181	0.972	0.466	--
15.0	125.0	0	8.0	12.5	1	50	*	53	1.073	16.6	125.0	0.176	0.970	0.465	--
16.0	125.0	0	10.0	17.5	0	88			1.038	21.0	125.0	~	0.967	0.464	~
17.0	125.0	0	10.0	17.5	0	88			1.006	20.5	125.0	~	0.965	0.463	~
18.0	125.0	0	10.0	17.5	0	88			0.977	20.0	125.0	~	0.963	0.462	~
19.0	125.0	0	10.0	17.5	0	88			0.950	19.6	125.0	~	0.961	0.461	~
20.0	125.0	0	10.0	17.5	0	88			0.926	19.3	125.0	~	0.958	0.460	~
21.0	125.0	1	10.0	17.5	0	88			0.903	18.9	62.6	~	0.956	0.464	~
22.5	125.0	1	10.0	17.5	0	88			0.876	18.5	62.6	~	0.952	0.476	~
23.0	125.0	1	14.0	22.5	0	55			0.866	25.3	62.6	~	0.949	0.480	~
24.0	125.0	1	14.0	22.5	0	55			0.843	24.7	62.6	~	0.947	0.491	~
25.0	125.0	1	14.0	22.5	0	55			0.826	24.3	62.6	~	0.944	0.498	~
26.0	125.0	1	14.0	22.5	0	55			0.809	23.9	62.6	~	0.940	0.505	~
27.5	125.0	1	14.0	22.5	0	55			0.790	23.5	62.6	~	0.935	0.513	~
28.0	125.0	1	18.0	27.5	1	52		66	0.783	29.8	62.6	0.457	0.931	0.515	0.81
29.0	125.0	1	18.0	27.5	1	52		66	0.766	29.3	62.6	0.425	0.928	0.523	0.73
30.0	125.0	1	18.0	27.5	1	52		66	0.752	28.9	62.6	0.404	0.923	0.528	0.68
31.0	125.0	1	23.0	32.5	1	22		73	0.743	32.0	62.6	Inf.	0.918	0.532	Non-Liq.
32.0	125.0	1	23.0	32.5	1	22		73	0.737	31.7	62.6	Inf.	0.912	0.535	Non-Liq.
33.0	125.0	1	23.0	32.5	1	22		73	0.731	31.5	62.6	Inf.	0.907	0.538	Non-Liq.
34.0	125.0	1	23.0	32.5	1	22		73	0.725	31.3	62.6	Inf.	0.900	0.541	Non-Liq.
35.0	125.0	1	23.0	32.5	1	22		73	0.720	31.1	62.6	Inf.	0.894	0.543	Non-Liq.
36.0	125.0	1	23.0	32.5	1	22		73	0.714	30.9	62.6	Inf.	0.887	0.544	Non-Liq.
37.5	125.0	1	23.0	32.5	1	22		73	0.707	30.6	62.6	Inf.	0.878	0.545	Non-Liq.
38.0	125.0	1	17.0	37.5	1	3		61	0.705	18.0	62.6	0.192	0.870	0.543	0.31
39.0	125.0	1	17.0	37.5	1	3		61	0.698	17.8	62.6	0.190	0.864	0.545	0.30
40.0	125.0	1	17.0	37.5	1	3		61	0.693	17.7	62.6	0.188	0.855	0.544	0.30
41.0	125.0	1	17.0	37.5	1	3		61	0.688	17.6	62.6	0.187	0.846	0.543	0.29
42.0	125.0	1	17.0	37.5	1	3		61	0.683	17.4	62.6	0.185	0.837	0.542	0.29
43.0	125.0	1	20.0	42.5	1	6		64	0.679	20.5	62.6	0.222	0.828	0.540	0.34
44.0	125.0	1	20.0	42.5	1	6		64	0.674	20.3	62.6	0.220	0.818	0.537	0.34
45.0	125.0	1	20.0	42.5	1	6		64	0.670	20.2	62.6	0.218	0.808	0.535	0.34
46.0	125.0	1	11.0	47.5	0	89			0.665	18.2	62.6	~	0.798	0.532	~
47.5	125.0	1	11.0	47.5	0	89			0.660	18.1	62.6	~	0.786	0.528	~
48.0	125.0	1	11.0	47.5	0	89			0.658	18.0	62.6	~	0.775	0.522	~
49.0	125.0	1	11.0	47.5	0	89			0.652	17.9	62.6	~	0.768	0.521	~
50.0	125.0	1	11.0	47.5	0	89			0.648	17.8	62.6	~	0.757	0.517	~
51.0	125.0	1	33.0	52.5	1	15	*	78	0.644	35.9	62.6	Inf.	0.747	0.513	Non-Liq.
52.5	125.0	1	33.0	52.5	1	15	*	78	0.639	35.7	62.6	Inf.	0.735	0.508	Non-Liq.
53.0	125.0	1	33.0	52.5	1	15	*	78	0.637	35.6	62.6	Inf.	0.725	0.502	Non-Liq.
54.0	125.0	1	33.0	52.5	1	15	*	78	0.632	35.3	62.6	Inf.	0.717	0.501	Non-Liq.
55.0	125.0	1	33.0	52.5	1	15	*	78	0.629	35.1	62.6	Inf.	0.708	0.496	Non-Liq.
56.0	125.0	1	33.0	52.5	1	15	*	78	0.625	34.9	62.6	Inf.	0.698	0.492	Non-Liq.
57.0	125.0	1	33.0	52.5	1	15	*	78	0.621	34.7	62.6	Inf.	0.689	0.488	Non-Liq.
58.0	125.0	1	33.0	52.5	1	15	*	78	0.618	34.5	62.6	Inf.	0.680	0.484	Non-Liq.
59.0	125.0	1	33.0	52.5	1	15	*	78	0.614	34.4	62.6	Inf.	0.671	0.480	Non-Liq.
60.0	125.0	1	33.0	52.5	1	15	*	78	0.611	34.2	62.6	Inf.	0.663	0.476	Non-Liq.
61.0	125.0	1	33.0	52.5	1	15	*	78	0.607	34.0	62.6	Inf.	0.655	0.472	Non-Liq.
62.0	125.0	1	33.0	52.5	1	15	*	78	0.604	33.8	62.6	Inf.	0.647	0.468	Non-Liq.
63.5	125.0	1	33.0	52.5	1	15	*	78	0.600	33.6	62.6	Inf.	0.638	0.463	Non-Liq.

* Indicates Assumed Value

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LIQUEFACTION POTENTIAL - MAXIMUM CONSIDERED EARTHQUAKE

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

SEPT 2024

PROJECT NO. W1974-06-01

FIG. 5

LIQUEFACTION SETTLEMENT ANALYSIS

MAXIMUM CONSIDERED EARTHQUAKE

EARTHQUAKE INFORMATION:

Earthquake Magnitude:

PGA _M (g):	0.738
-----------------------	-------

Historic High Groundwater:	20.0
----------------------------	------

Groundwater @ Exploration:	31.0
----------------------------	------

Boring ID: B5

DEPTH TO BASE	BLOW COUNT N	WET DENSITY (PCF)	TOTAL STRESS O (TSF)	EFFECT STRESS O' (TSF)	REL. DEN. Dr (%)	ADJUST BLOWS (N1)60	Tav/σ' _o	LIQUEFACTION SAFETY FACTOR	VOL. STRAIN [e _v] (%)	EQ. SETTLE. Pe (in.)
1.0	6	125.0	0.031	0.031	59	19	0.480	--	0.00	0.00
2.0	6	125.0	0.094	0.094	58	19	0.480	--	0.00	0.00
3.0	6	125.0	0.156	0.156	56	19	0.480	--	0.00	0.00
4.0	6	125.0	0.219	0.219	55	19	0.480	--	0.00	0.00
5.0	6	125.0	0.281	0.281	53	19	0.480	--	0.00	0.00
6.0	6	125.0	0.344	0.344	52	19	0.480	--	0.00	0.00
7.0	6	125.0	0.406	0.406	51	18	0.480	--	0.00	0.00
8.0	6	125.0	0.469	0.469	50	17	0.480	--	0.00	0.00
9.0	6	125.0	0.531	0.531	49	16	0.480	--	0.00	0.00
10.0	8	125.0	0.594	0.594	53	19	0.480	--	0.00	0.00
11.0	8	125.0	0.656	0.656	53	19	0.480	--	0.00	0.00
12.5	8	125.0	0.734	0.734	53	18	0.480	--	0.00	0.00
13.0	8	125.0	0.766	0.766	53	18	0.480	--	0.00	0.00
14.0	8	125.0	0.844	0.844	53	17	0.480	--	0.00	0.00
15.0	8	125.0	0.906	0.906	53	17	0.480	--	0.00	0.00
16.0	10	125.0	0.969	0.969		21	0.480	--	0.00	0.00
17.0	10	125.0	1.031	1.031		20	0.480	--	0.00	0.00
18.0	10	125.0	1.094	1.094		20	0.480	--	0.00	0.00
19.0	10	125.0	1.156	1.156		20	0.480	--	0.00	0.00
20.0	10	125.0	1.219	1.219		19	0.480	--	0.00	0.00
21.0	10	125.0	1.281	1.266		19	0.486	--	0.00	0.00
22.5	10	125.0	1.359	1.305		18	0.500	--	0.00	0.00
23.0	14	125.0	1.391	1.320		25	0.505	--	0.00	0.00
24.0	14	125.0	1.469	1.360		25	0.518	--	0.00	0.00
25.0	14	125.0	1.531	1.391		24	0.528	--	0.00	0.00
26.0	14	125.0	1.594	1.422		24	0.538	--	0.00	0.00
27.5	14	125.0	1.672	1.461		23	0.549	--	0.00	0.00
28.0	18	125.0	1.703	1.477	66	30	0.553	0.81	0.75	0.05
29.0	18	125.0	1.781	1.516	66	29	0.564	0.73	0.75	0.09
30.0	18	125.0	1.844	1.547	66	29	0.572	0.68	0.75	0.09
31.0	23	125.0	1.906	1.579	73	32	0.579	Non-Liq.	0.00	0.00
32.0	23	125.0	1.969	1.610	73	32	0.587	Non-Liq.	0.00	0.00
33.0	23	125.0	2.031	1.641	73	31	0.594	Non-Liq.	0.00	0.00
34.0	23	125.0	2.094	1.673	73	31	0.601	Non-Liq.	0.00	0.00
35.0	23	125.0	2.156	1.704	73	31	0.607	Non-Liq.	0.00	0.00
36.0	23	125.0	2.219	1.735	73	31	0.613	Non-Liq.	0.00	0.00
37.5	23	125.0	2.297	1.774	73	31	0.621	Non-Liq.	0.00	0.00
38.0	17	125.0	2.328	1.790	61	18	0.624	0.31	1.70	0.10
39.0	17	125.0	2.406	1.829	61	18	0.631	0.30	1.70	0.20
40.0	17	125.0	2.469	1.860	61	18	0.637	0.30	1.70	0.20
41.0	17	125.0	2.531	1.892	61	18	0.642	0.29	1.70	0.20
42.0	17	125.0	2.594	1.923	61	17	0.647	0.29	1.70	0.20
43.0	20	125.0	2.656	1.954	64	20	0.652	0.34	1.40	0.17
44.0	20	125.0	2.719	1.986	64	20	0.657	0.34	1.40	0.17
45.0	20	125.0	2.781	2.017	64	20	0.662	0.34	1.40	0.17
46.0	11	125.0	2.844	2.048		18	0.666	~	0.00	0.00
47.5	11	125.0	2.922	2.087		18	0.672	~	0.00	0.00
48.0	11	125.0	2.953	2.103		18	0.674	~	0.00	0.00
49.0	11	125.0	3.031	2.142		18	0.679	~	0.00	0.00
50.0	11	125.0	3.094	2.173		18	0.683	~	0.00	0.00
51.0	33	125.0	3.156	2.205	78	36	0.687	Non-Liq.	0.00	0.00
52.5	33	125.0	3.234	2.244	78	36	0.691	Non-Liq.	0.00	0.00
53.0	33	125.0	3.266	2.259	78	36	0.693	Non-Liq.	0.00	0.00
54.0	33	125.0	3.344	2.299	78	35	0.698	Non-Liq.	0.00	0.00
55.0	33	125.0	3.406	2.330	78	35	0.701	Non-Liq.	0.00	0.00
56.0	33	125.0	3.469	2.361	78	35	0.705	Non-Liq.	0.00	0.00
57.0	33	125.0	3.531	2.392	78	35	0.708	Non-Liq.	0.00	0.00
58.0	33	125.0	3.594	2.424	78	35	0.711	Non-Liq.	0.00	0.00
59.0	33	125.0	3.656	2.455	78	34	0.714	Non-Liq.	0.00	0.00
60.0	33	125.0	3.719	2.486	78	34	0.717	Non-Liq.	0.00	0.00
61.0	33	125.0	3.781	2.518	78	34	0.720	Non-Liq.	0.00	0.00
62.0	33	125.0	3.844	2.549	78	34	0.723	Non-Liq.	0.00	0.00
63.5	33	125.0	3.922	2.588	78	34	0.727	Non-Liq.	0.00	0.00

TOTAL SETTLEMENT = 1.6 inches

GEOCON
WEST, INC.



ENVIRONMENTAL GEOTECHNICAL MATERIALS
500 NORTH VICTORY BOULEVARD - BURBANK, CA 91502
PHONE: 818-841-8388 FAX: 818-841-1704

DRAFTED BY: ACS

CHECKED BY: JTA

**LIQUEFACTION INDUCED SETTLEMENT
- MAXIMUM CONSIDERED EARTHQUAKE**

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

SEPT 2024

PROJECT NO. W1974-06-01

FIG. 6

MCE EARTHQUAKE INFORMATION

Earthquake Magnitude	7.30
Peak Horiz. Acceleration (g)	0.738

Boring ID: B5

Depth of Base of Strate (ft)	Thickness of Layer (ft)	Depth of Mid-point of Layer (ft)	Soil Unit Weight (pcf)	Overburden Pressure at Mid-point (tsf)	Mean Effective Pressure at Mid-point (tsf)	Average Cyclic Shear Stress (Ts1)	Field SPT [N]	Correction Factor [Ccr]	Relative Density [Dr] (%)	Correction Factor [Cn]	Corrected [N]60	rd Factor	Maximum Shear Mod. [Gmax] (tsf)	[yeff]*[Geff] (tsf)	yeff Shear Strain	[yeff]*100%	Volumetric Strain M7.5 [E]5 (%)	Number of Strain Cycles [Nc]	Corrected Vol. Strains [Ec]	Estimated Settlement [S] (inches)
1.0	1.0	0.5	125.0	0.03	0.02	0.015	6	1.25	59.3	1.7	18.8	1.0	171.9	8.63E-05	1.80E-04	0.016	1.73E-02	13.3	1.84E-02	0.00
2.0	1.0	1.5	125.0	0.09	0.06	0.045	6	1.25	57.6	1.7	18.8	1.0	297.7	1.47E-04	2.30E-04	0.023	2.48E-02	13.3	2.35E-02	0.01
3.0	1.0	2.5	125.0	0.16	0.10	0.075	6	1.25	56.0	1.7	18.8	1.0	384.4	1.86E-04	1.70E-04	0.017	1.83E-02	13.3	1.74E-02	0.00
4.0	1.0	3.5	125.0	0.22	0.15	0.105	6	1.25	54.5	1.7	18.8	1.0	454.8	2.15E-04	8.10E-04	0.081	8.74E-02	13.3	8.29E-02	0.02
5.0	1.0	4.5	125.0	0.28	0.19	0.135	6	1.25	53.2	1.7	18.8	1.0	515.7	2.40E-04	8.10E-04	0.081	8.74E-02	13.3	8.29E-02	0.02
6.0	1.0	5.5	125.0	0.34	0.23	0.164	6	1.25	51.9	1.7	18.8	1.0	570.1	2.60E-04	4.50E-04	0.045	4.86E-02	13.3	4.61E-02	0.01
7.0	1.0	6.5	125.0	0.41	0.27	0.194	6	1.25	50.8	1.6	18.0	1.0	611.0	2.81E-04	4.50E-04	0.045	5.11E-02	13.3	4.85E-02	0.01
8.0	1.0	7.5	125.0	0.47	0.31	0.224	6	1.25	49.7	1.5	17.1	1.0	645.2	3.02E-04	1.00E-03	0.100	1.21E-01	13.3	1.15E-01	0.03
9.0	1.0	8.5	125.0	0.53	0.36	0.253	6	1.25	48.7	1.4	16.4	1.0	676.9	3.20E-04	1.00E-03	0.100	1.27E-01	13.3	1.21E-01	0.03
10.0	1.0	9.5	125.0	0.59	0.40	0.282	8	1.25	52.8	1.3	19.3	1.0	756.5	3.14E-04	1.00E-03	0.100	1.04E-01	13.3	9.89E-02	0.02
11.0	1.0	10.5	125.0	0.66	0.44	0.311	8	1.25	52.8	1.3	18.6	1.0	785.6	3.28E-04	1.00E-03	0.100	1.09E-01	13.3	1.03E-01	0.02
12.5	1.5	11.8	125.0	0.73	0.49	0.347	8	1.25	52.8	1.2	17.9	1.0	819.9	3.44E-04	1.00E-03	0.100	1.14E-01	13.3	1.09E-01	0.04
13.0	0.5	12.8	125.0	0.80	0.53	0.376	8	1.25	52.8	1.2	17.6	1.0	849.8	3.54E-04	7.10E-04	0.071	8.27E-02	13.3	7.85E-02	0.01
14.0	1.0	13.5	125.0	0.84	0.57	0.398	8	1.25	52.8	1.1	17.0	1.0	864.4	3.63E-04	7.10E-04	0.071	8.62E-02	13.3	8.18E-02	0.02
15.0	1.0	14.5	125.0	0.91	0.61	0.426	8	1.25	52.8	1.1	16.6	1.0	888.4	3.73E-04	7.10E-04	0.071	8.88E-02	13.3	8.43E-02	0.02
16.0	1.0	15.5	125.0	0.97	0.65	0.454	10	1.25		1.0	19.3	1.0	993.3	3.51E-04	7.10E-04	0.071	6.70E-02	13.3	6.39E-02	0.02
17.0	1.0	16.5	125.0	1.03	0.69	0.482	10	1.25		1.0	20.5	1.0	1016.8	3.58E-04	7.10E-04	0.071	6.90E-02	13.3	6.54E-02	0.02
18.0	1.0	17.5	125.0	1.09	0.73	0.509	10	1.25		1.0	20.0	1.0	1039.4	3.66E-04	7.10E-04	0.071	7.08E-02	13.3	6.72E-02	0.02
19.0	1.0	18.5	125.0	1.16	0.77	0.536	10	1.25		1.0	19.6	1.0	1061.4	3.72E-04	7.10E-04	0.071	7.26E-02	13.3	6.89E-02	0.02
20.0	1.0	19.5	125.0	1.22	0.82	0.563	10	1.25		0.9	19.3	1.0	1082.6	3.78E-04	7.10E-04	0.071	7.43E-02	13.3	7.05E-02	0.02
21.0	1.0	20.5	125.0	1.28	0.86	0.590	10	1.25		0.9	18.9	1.0	1103.2	3.84E-04	7.10E-04	0.071	7.60E-02	13.3	7.21E-02	0.00
22.5	1.5	21.8	125.0	1.36	0.91	0.623	10	1.25		0.9	18.5	1.0	1128.1	3.90E-04	7.10E-04	0.071	7.80E-02	13.3	7.40E-02	0.00
23.0	0.5	22.8	125.0	1.42	0.95	0.649	14	1.25		0.9	25.3	0.9	1280.0	3.54E-04	7.10E-04	0.071	5.37E-02	13.3	5.09E-02	0.00
24.0	1.0	23.5	125.0	1.47	0.98	0.668	14	1.25		0.8	24.7	0.9	1291.5	3.58E-04	7.10E-04	0.071	5.51E-02	13.3	5.23E-02	0.00
25.0	1.0	24.5	125.0	1.53	1.03	0.693	14	1.25		0.8	24.3	0.9	1311.5	3.61E-04	5.20E-04	0.052	4.12E-02	13.3	3.90E-02	0.00
26.0	1.0	25.5	125.0	1.59	1.07	0.718	14	1.25		0.8	23.9	0.9	1330.9	3.65E-04	5.20E-04	0.052	4.19E-02	13.3	3.98E-02	0.00
27.5	1.5	26.8	125.0	1.67	1.12	0.748	14	1.25		0.8	23.5	0.9	1354.6	3.69E-04	5.20E-04	0.052	4.29E-02	13.3	4.07E-02	0.00
28.0	0.5	27.8	125.0	1.73	1.16	0.772	18	1.25	66.5	0.8	29.8	0.9	1494.5	3.41E-04	5.20E-04	0.052	3.22E-02	13.3	3.05E-02	0.00
29.0	1.0	28.5	125.0	1.78	1.19	0.790	18	1.25	66.5	0.8	29.3	0.9	1505.2	3.44E-04	5.20E-04	0.052	3.29E-02	13.3	3.12E-02	0.00
30.0	1.0	29.5	125.0	1.84	1.24	0.814	18	1.25	66.5	0.8	28.9	0.9	1524.1	3.48E-04	5.20E-04	0.052	3.35E-02	13.3	3.18E-02	0.00
31.0	1.0	30.5	125.0	1.91	1.28	0.837	23	1.25	72.5	0.7	32.0	0.9	1603.0	3.35E-04	5.20E-04	0.052	2.96E-02	13.3	2.81E-02	0.00
32.0	1.0	31.5	125.0	1.97	1.32	0.859	23	1.25	72.5	0.7	31.7	0.9	1625.2	3.39E-04	5.20E-04	0.052	2.99E-02	13.3	2.84E-02	0.00
33.0	1.0	32.5	125.0	2.03	1.36	0.881	23	1.25	72.5	0.7	31.5	0.9	1646.9	3.37E-04	5.20E-04	0.052	3.02E-02	13.3	2.86E-02	0.00
34.0	1.0	33.5	125.0	2.09	1.40	0.903	23	1.25	72.5	0.7	31.3	0.9	1668.1	3.38E-04	5.20E-04	0.052	3.04E-02	13.3	2.88E-02	0.00
35.0	1.0	34.5	125.0	2.16	1.44	0.924	23	1.25	72.5	0.7	31.1	0.9	1689.0	3.39E-04	5.20E-04	0.052	3.07E-02	13.3	2.91E-02	0.00
36.0	1.0	35.5	125.0	2.22	1.49	0.945	23	1.25	72.5	0.7	30.9	0.9	1709.5	3.39E-04	5.20E-04	0.052	3.09E-02	13.3	2.93E-02	0.00
37.5	1.5	36.8	125.0	2.30	1.54	0.971	23	1.25	72.5	0.7	30.6	0.9	1734.5	3.40E-04	5.20E-04	0.052	3.12E-02	13.3	2.96E-02	0.00
38.0	0.5	37.8	125.0	2.36	1.58	0.991	17	1.25	60.8	0.7	18.0	0.9	1472.1	4.05E-04	8.10E-04	0.081	9.21E-02	13.3	8.74E-02	0.00
39.0	1.0	38.5	125.0	2.41	1.61	1.005	17	1.25	60.8	0.7	17.8	0.9	1482.1	4.06E-04	8.10E-04	0.081	9.31E-02	13.3	8.83E-02	0.00
40.0	1.0	39.5	125.0	2.47	1.65	1.024	17	1.25	60.8	0.7	17.7	0.9	1497.6	4.06E-04	8.10E-04	0.081	9.39E-02	13.3	8.91E-02	0.00
41.0	1.0	40.5	125.0	2.53	1.70	1.043	17	1.25	60.8	0.7	17.6	0.8	1512.8	4.07E-04	8.10E-04	0.081	9.47E-02	13.3	8.99E-02	0.00
42.0	1.0	41.5	125.0	2.59	1.74	1.061	17	1.25	60.8	0.7	17.4	0.8	1527.8	4.07E-04	8.10E-04	0.081	9.55E-02	13.3	9.06E-02	0.00
43.0	1.0	42.5	125.0	2.66	1.78	1.079	20	1.25	64.0	0.7	20.5	0.8	1631.7	3.85E-04	5.20E-04	0.052	5.05E-02	13.3	4.79E-02	0.00
44.0	1.0	43.5	125.0	2.72	1.82	1.097	20	1.25	64.0	0.7	20.3	0.8	1647.0	3.85E-04	5.20E-04	0.052	5.09E-02	13.3	4.83E-02	0.00
45.0	1.0	44.5	125.0	2.78	1.86	1.114	20	1.25	64.0	0.7	20.2	0.8	1662.1	3.84E-04	5.20E-04	0.052	5.14E-02	13.3	4.87E-02	0.00
46.0	1.0	45.5	125.0	2.84	1.91	1.130	11	1.25		0.7	18.2	0.8	1622.1	3.97E-04	5.20E-04	0.052	5.84E-02	13.3	5.54E-02	0.00
47.5	1.5	46.8	125.0	2.92	1.96	1.150	11	1.25		0.7	18.1	0.8	1640.9	3.97E-04	5.20E-04	0.052	5.88E-02	13.3	5.57E-02	0.00
48.0	0.5	47.8	125.0	2.98	2.00	1.166	11	1.25		0.7	18.0	0.8	1657.1	3.96E-04	5.20E-04	0.052	5.89E-02	13.3	5.59E-02	0.00
49.0	1.0	48.5	125.0	3.03	2.03	1.177	11	1.25		0.7	17.9	0.8	1666.8	3.96E-04	1.00E-02	1.000	1.14E+00	13.3	1.08E+00	0.00
50.0	1.0	49.5	125.0	3.09	2.07	1.192	11	1.25		0.6	17.8	0.8	1681.4	3.95E-04	1.00E-02	1.000	1.15E+00	13.3	1.09E+00	0.00
51.0	1.0	50.5	125.0	3.16	2.11	1.206	33	1.25	78.4	0.6	35.9	0.7	2144.6	3.12E-04	1.00E-02	1.000	4.95E-01	13.3	4.70E-01	0.00
52.5	1.5	51.8	125.0	3.23	2.17	1.224	33	1.25	78.4	0.6	35.7	0.7	2165.8	3.11E-04	1.00E-02	1.000	5.00E-01	13.3	4.74E-01	0.00
53.0	0.5	52.8	125.0	3.30	2.21	1.237	33	1.25	78.4	0.6	35.6	0.7	2184.6	3.10E-04	1.00E-02	1.000	5.01E-01	13.3	4.76E-01	0.00
54.0	1.0	53.5	125.0	3.34	2.24	1.247	33	1.25	78.4	0.6	35.3	0.7	2194.9	3.10E-04	1.00E-02	1.000	5.06E-01	13.3	4.80E-01	0.00
55.0	1.0	54.5	125.0	3.41	2.28	1.260	33	1.25	78.4	0.6	35.1	0.7	2211.2	3.10E-04	1.00E-02	1.000	5.09E-01	13.3	4.83E-01	0.00
56.0	1.0	55.5	125.0	3.47	2.32	1.272	33	1.25	78.4	0.6	34.9	0.7	2227.4	3.09E-04	1.00E-02	1.000	5.12E-01	13.3	4.86E-01	0.00
57.0	1.0	56.5	125.0	3.53	2.37	1.284	33	1.25	78.4	0.6	34.7	0.7	2243.3	3.08E-04	1.00E-02	1.000	5.16E-01	13.3	4.89E-01	0.00
58.0	1.0	57.5	125.0	3.59	2.41	1.296	33	1.25	78.4	0.6	34.5	0.7	2259.0	3.08E-04	1.00E-02	1.000	5.19			

BORING PERCOLATION TEST FIELD LOG

Date: <u>Friday, August 2, 2024</u> Project Number: <u>W1974-06-01</u> Project Location: <u>4900 Clark Avenue</u> Earth Description: <u>ML-CL</u> Tested By: <u>RA</u> Liquid Description: <u>Water</u> Measurement Method: <u>Sounder</u>	Boring/Test Number: <u>Boring B1</u> Diameter of Boring: <u>8</u> inches Diameter of Casing: <u>2</u> inches Depth of Boring: <u>10</u> feet Depth to Invert of BMP: <u>5</u> feet Depth to Water Table: <u>32</u> feet Depth to Initial Water Depth (d_i): <u>45</u> inches Start Time for Pre-Soak: <u>7:00 AM</u> Start Time for Standard: <u>8:00 AM</u> Water Remaining in Boring (Y/N): <u>Yes</u> Standard Time Interval Between Readings: <u>30 min</u>
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Reading Number	Start Time (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	8:27 AM	8:57 AM	30	3.6	
2	8:58 AM	9:28 AM	30	3.2	
3	9:23 AM	9:53 AM	30	2.2	
4	10:04 AM	10:34 AM	30	2.2	
5	10:35 AM	11:05 AM	30	2.2	
6	11:06 AM	11:36 AM	30	2.2	Stabilized Readings
7	11:37 AM	12:07 PM	30	2.2	Achieved with Readings
8	12:08 PM	12:38 PM	30	2.3	6, 7, and 8

MEASURED PERCOLATION RATE AND DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
 Test Section Height, h: 75 inches

Test Section Surface Area, $A = 2\pi rh + \pi r^2$

A = 1935 in²

Discharged Water Volume, $V = \pi r^2 \Delta d$

Percolation Rate = $\left(\frac{V/A}{\Delta T} \right)$

Reading 6 V = 109 in³
 Reading 7 V = 109 in³
 Reading 8 V = 115 in³

Percolation Rate = 0.11 inches/hour
 Percolation Rate = 0.11 inches/hour
 Percolation Rate = 0.12 inches/hour

Measured Percolation Rate = 0.11 inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1
 Site Variability, RF_v = 1
 Long Term Siltation, RF_s = 1

Total Reduction Factor, $RF = RF_t + RF_v + RF_s$

Total Reduction Factor = 3

Design Infiltration Rate

Design Infiltration Rate = 0.04 inches/hour

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CHECKED BY: JTA

PERCOLATION TEST RESULTS AND CALCULATIONS

4900 CLARK AVENUE
 LAKEWOOD, CALIFORNIA

SEPT 2024

PROJECT NO. W1974-06-01

FIG. 8

BORING PERCOLATION TEST FIELD LOG

Date: <u>Friday, August 2, 2024</u> Project Number: <u>W1974-06-01</u> Project Location: <u>4900 Clark Avenue</u> Earth Description: <u>ML-CL</u> Tested By: <u>RA</u> Liquid Description: <u>Water</u> Measurement Method: <u>Sounder</u>	Boring/Test Number: <u>Boring B2</u> Diameter of Boring: <u>8</u> inches Diameter of Casing: <u>2</u> inches Depth of Boring: <u>5</u> feet Depth to Invert of BMP: <u>2</u> feet Depth to Water Table: <u>32</u> feet Depth to Initial Water Depth (d_i): <u>17.76</u> inches Start Time for Pre-Soak: <u>7:15 AM</u> Start Time for Standard: <u>8:15 AM</u> Water Remaining in Boring (Y/N): <u>Yes</u> Standard Time Interval Between Readings: <u>30 min</u>
---	--

Reading Number	Start Time (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	8:32 AM	9:02 AM	30	0.0	
2	9:02 AM	9:32 AM	30	0.5	
3	9:34 AM	10:04 AM	30	0.8	
4	10:06 AM	10:36 AM	30	0.5	
5	10:40 AM	11:10 AM	30	0.4	
6	11:10 AM	11:40 AM	30	0.5	Stabilized Readings
7	11:40 AM	12:10 PM	30	0.5	Achieved with Readings
8	12:10 PM	12:40 PM	30	0.5	6, 7, and 8

MEASURED PERCOLATION RATE AND DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
 Test Section Height, h: 42.24 inches

Test Section Surface Area, $A = 2\pi rh + \pi r^2$

A = 1112 in²

Discharged Water Volume, $V = \pi r^2 \Delta d$

Percolation Rate = $\left(\frac{V/A}{\Delta T} \right)$

Reading 6 V = 24 in³
 Reading 7 V = 24 in³
 Reading 8 V = 24 in³

Percolation Rate = 0.04 inches/hour
 Percolation Rate = 0.04 inches/hour
 Percolation Rate = 0.04 inches/hour

Measured Percolation Rate = 0.04 inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1
 Site Variability, RF_v = 1
 Long Term Siltation, RF_s = 1

Total Reduction Factor, $RF = RF_t + RF_v + RF_s$

Total Reduction Factor = 3

Design Infiltration Rate

Design Infiltration Rate = 0.01 inches/hour

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PERCOLATION TEST RESULTS AND CALCULATIONS

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 LAKEWOOD, CALIFORNIA

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CHECKED BY: JTA

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PROJECT NO. W1974-06-01

FIG. 9

BORING PERCOLATION TEST FIELD LOG

Date: <u>Friday, August 2, 2024</u> Project Number: <u>W1974-06-01</u> Project Location: <u>4900 Clark Avenue</u> Earth Description: <u>ML-CL</u> Tested By: <u>RA</u> Liquid Description: <u>Water</u> Measurement Method: <u>Sounder</u>	Boring/Test Number: <u>Boring B3</u> Diameter of Boring: <u>8</u> inches Diameter of Casing: <u>2</u> inches Depth of Boring: <u>5</u> feet Depth to Invert of BMP: <u>2</u> feet Depth to Water Table: <u>32</u> feet Depth to Initial Water Depth (d_i): <u>18.6</u> inches
Start Time for Pre-Soak: <u>7:30 AM</u> Start Time for Standard: <u>8:30 AM</u>	Water Remaining in Boring (Y/N): <u>Yes</u> Standard Time Interval Between Readings: <u>30 min</u>

Reading Number	Start Time (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	8:48 AM	9:18 AM	30	0.6	
2	9:18 AM	9:48 AM	30	1.3	
3	9:48 AM	10:18 AM	30	1.2	
4	10:18 AM	10:48 AM	30	0.7	
5	10:48 AM	11:18 AM	30	0.8	
6	11:18 AM	11:48 AM	30	0.6	Stabilized Readings
7	11:48 AM	12:18 PM	30	0.6	Achieved with Readings
8	12:18 PM	12:48 PM	30	0.7	6, 7, and 8

MEASURED PERCOLATION RATE AND DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
 Test Section Height, h: 41.4 inches

Test Section Surface Area, $A = 2\pi rh + \pi r^2$

A = 1091 in²

Discharged Water Volume, $V = \pi r^2 \Delta d$

Percolation Rate = $\left(\frac{V/A}{\Delta T} \right)$

Reading 6 V = 30 in³
 Reading 7 V = 30 in³
 Reading 8 V = 36 in³

Percolation Rate = 0.06 inches/hour
 Percolation Rate = 0.06 inches/hour
 Percolation Rate = 0.07 inches/hour

Measured Percolation Rate = 0.06 inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1
 Site Variability, RF_v = 1
 Long Term Siltation, RF_s = 1

Total Reduction Factor, $RF = RF_t + RF_v + RF_s$

Total Reduction Factor = 3

Design Infiltration Rate

Design Infiltration Rate = 0.02 inches/hour

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PERCOLATION TEST RESULTS AND CALCULATIONS

4900 CLARK AVENUE
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PROJECT NO. W1974-06-01

FIG. 10

BORING PERCOLATION TEST FIELD LOG

Date: <u>Friday, August 2, 2024</u> Project Number: <u>W1974-06-01</u> Project Location: <u>4900 Clark Avenue</u> Earth Description: <u>ML-CL</u> Tested By: <u>RA</u> Liquid Description: <u>Water</u> Measurement Method: <u>Sounder</u>	Boring/Test Number: <u>Boring B4</u> Diameter of Boring: <u>8</u> inches Diameter of Casing: <u>2</u> inches Depth of Boring: <u>5</u> feet Depth to Invert of BMP: <u>2</u> feet Depth to Water Table: <u>32</u> feet Depth to Initial Water Depth (d_i): <u>9.6</u> inches Start Time for Pre-Soak: <u>8:00 AM</u> Start Time for Standard: <u>9:00 AM</u> Water Remaining in Boring (Y/N): <u>Yes</u> Standard Time Interval Between Readings: <u>30 min</u>
---	--

Reading Number	Start Time (hh:mm)	Time End (hh:mm)	Elapsed Time Δtime (min)	Water Drop During Standard Time Interval, Δd (in)	Soil Description Notes Comments
1	9:15 AM	9:45 AM	30	0.6	
2	9:45 AM	10:15 AM	30	0.6	
3	10:15 AM	10:45 AM	30	0.2	
4	10:45 AM	11:15 AM	30	0.4	
5	11:15 AM	11:45 AM	30	0.5	
6	11:45 AM	12:15 PM	30	0.1	Stabilized Readings
7	12:15 PM	12:45 PM	30	0.1	Achieved with Readings
8	12:45 PM	1:15 PM	30	0.0	6, 7, and 8

MEASURED PERCOLATION RATE AND DESIGN INFILTRATION RATE CALCULATIONS*

* Calculations Below Based on Stabilized Readings Only

Boring Radius, r: 4 inches
 Test Section Height, h: 50.4 inches

Test Section Surface Area, $A = 2\pi rh + \pi r^2$

A = 1317 in²

Discharged Water Volume, $V = \pi r^2 \Delta d$

Percolation Rate = $\left(\frac{V/A}{\Delta T} \right)$

Reading 6 V = 6 in³
 Reading 7 V = 6 in³
 Reading 8 V = 0 in³

Percolation Rate = 0.01 inches/hour
 Percolation Rate = 0.01 inches/hour
 Percolation Rate = 0.00 inches/hour

Measured Percolation Rate = 0.01 inches/hour

Reduction Factors

Small Diameter Boring, RF_t = 1
 Site Variability, RF_v = 1
 Long Term Siltation, RF_s = 1

Total Reduction Factor, $RF = RF_t + RF_v + RF_s$

Total Reduction Factor = 3

Design Infiltration Rate

Design Infiltration Rate = 0.00 inches/hour

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PERCOLATION TEST RESULTS AND CALCULATIONS

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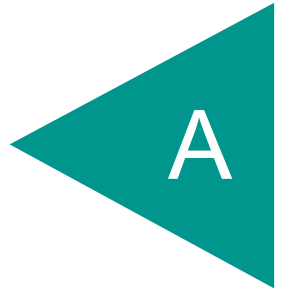
SEPT 2024

PROJECT NO. W1974-06-01

FIG. 11

APPENDIX

A



APPENDIX A

FIELD INVESTIGATION

The site was explored on August 2, 2024, by excavating five 8-inch-diameter borings using a truck-mounted hollow-stem auger drilling machine to depths between 5½ and 63½ feet beneath the ground surface. Representative and relatively undisturbed samples were obtained by driving a 3-inch, O. D., California Modified Sampler into the “undisturbed” soil mass with blows from a 140-pound auto-hammer falling 30 inches. The California Modified Sampler was equipped with 1-inch by 2³/₈-inch diameter brass sampler rings to facilitate soil removal and testing. Bulk samples were also obtained.

The soil conditions encountered in the borings were visually examined, classified and logged in general accordance with the Unified Soil Classification System (USCS). The logs of the borings are presented on Figures A1 through A5. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained. The logs also include our interpretation of the conditions between sampling intervals. Therefore, the logs contain both observed and interpreted data. We determined the lines designating the interface between soil materials on the logs using visual observations, penetration rates, excavation characteristics and other factors. The transition between materials may be abrupt or gradual. Where applicable, the logs were revised based on subsequent laboratory testing. The approximate locations of the borings are shown on Figure 2.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 1	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.) -- DATE COMPLETED 08/02/2024 EQUIPMENT HOLLOW STEM AUGER BY: ACS			
					MATERIAL DESCRIPTION			
0	BULK 0-5'				ASPHALT: 4" BASE: 5.5" ARTIFICIAL FILL Sandy Silt, soft, slightly moist, brown, fine-grained, trace clay.			
2	B1@2.5'					7	89.3	26.3
4	B1@5'		ML		YOUNGER ALLUVIUM Sandy Silt, firm, slightly moist, brown, fine-grained, some clay.			
6	B1@7.5'				- soft, light brown	15	90.0	28.6
8	B1@10'				- stiff	10	87.3	33.5
10					23	97.1	27.0	
					Total depth of boring: 10.5 feet Fill to 3 feet. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped. Patched with asphalt. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A1,
Log of Boring 1, Page 1 of 1

W1974-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED.
IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 2 ELEV. (MSL.) - - DATE COMPLETED 08/02/2024 EQUIPMENT HOLLOW STEM AUGER BY: ACS	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
2	B2@2.5'				ARTIFICIAL FILL Sandy Silt, stiff, slightly moist, brown, fine-grained, trace clay.	22	99.7	18.8
4	B2@5'			ML	YOUNGER ALLUVIUM Sandy Silt, stiff, slightly moist, brown, fine-grained, trace clay.	25	100.6	17.0
Total depth of boring: 5.5 feet Fill to 3 feet. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.								

Figure A2,
Log of Boring 2, Page 1 of 1

W1974-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS		... SAMPLING UNSUCCESSFUL		... STANDARD PENETRATION TEST		... DRIVE SAMPLE (UNDISTURBED)
		... DISTURBED OR BAG SAMPLE		... CHUNK SAMPLE		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED.
IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 3			PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.)	--	DATE COMPLETED			
					08/02/2024					
					EQUIPMENT HOLLOW STEM AUGER BY: ACS					
					MATERIAL DESCRIPTION					
0										
2										
	B3@2.5'				ARTIFICIAL FILL Sandy Silt, firm, slightly moist, brown, fine-grained, trace clay.			17	91.5	25.4
4										
	B3@5'			ML	YOUNGER ALLUVIUM Sandy Silt, slightly moist, brown, fine-grained, trace clay.			16	96.0	23.5
					Total depth of boring: 5.5 feet Fill to 3 feet. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.					

Figure A3,
Log of Boring 3, Page 1 of 1

W1974-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS		... SAMPLING UNSUCCESSFUL		... STANDARD PENETRATION TEST		... DRIVE SAMPLE (UNDISTURBED)
		... DISTURBED OR BAG SAMPLE		... CHUNK SAMPLE		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED.
IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 4 ELEV. (MSL.) - - DATE COMPLETED 08/02/2024 EQUIPMENT HOLLOW STEM AUGER BY: ACS	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
0					MATERIAL DESCRIPTION			
0-5'	BULK				ARTIFICIAL FILL Sandy Silt, firm, slightly moist, brown, fine-grained, trace clay.			
2								
B4@2.5'					- asphalt	49	118.7	8.0
4				ML	YOUNGER ALLUVIUM Sandy Silt, stiff, slightly moist, brown, fine-grained, trace clay.			
B4@5'						22	92.3	30.1
					Total depth of boring: 5.5 feet Fill to 3 feet. No groundwater encountered. Percolation testing performed. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A4,
Log of Boring 4, Page 1 of 1

W1974-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS		... SAMPLING UNSUCCESSFUL		... STANDARD PENETRATION TEST		... DRIVE SAMPLE (UNDISTURBED)
		... DISTURBED OR BAG SAMPLE		... CHUNK SAMPLE		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED.
IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 5 ELEV. (MSL.) - - DATE COMPLETED 08/02/2024 EQUIPMENT HOLLOW STEM AUGER BY: ACS	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					MATERIAL DESCRIPTION			
0	BULK 0-5'				ARTIFICIAL FILL Sandy Silt, soft, slightly moist, brown, fine-grained, trace clay.			
2	B5@2.5'				YOUNGER ALLUVIUM Sandy Silt, soft, slightly moist, brown, fine-grained, trace clay.	6		21.3
4	B5@5'				- firm	17	99.9	17.1
6								
8	B5@7.5'			ML	- soft, increase in fine-grained	8		17.6
10	B5@10'				- firm	26	90.7	28.1
12								
14	B5@12.5'				- soft, dark brown, increase in clay	8		28.7
16	B5@15'				Silty Clay, firm, slightly moist, dark brown, trace fine-grained sand.	19	93.2	27.4
18	B5@17.5'			CL	- soft, gray to brown	10		20.1
20	B5@20'				- stiff, gray	31	114.1	18.5
22								
24	B5@22.5'				Sandy Clay, firm, slightly moist, grayish brown, fine-grained, trace clay.	14		17.8
26	B5@25'			CL	- stiff, light brown	32	105.1	23.0
28	B5@27.5'			ML	Sandy Silt, firm, slightly moist, grayish brown, fine-grained.	18		19.9

Figure A5,
Log of Boring 5, Page 1 of 3

W1974-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED.
IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 5		PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
					ELEV. (MSL.)	DATE COMPLETED			
						08/02/2024			
					EQUIPMENT HOLLOW STEM AUGER BY: ACS				
					MATERIAL DESCRIPTION				
30	B5@30'			SM	Silty Sand, medium dense, moist, grayish brown, fine-grained.		36	117.2	17.2
32	B5@32.5'				- groundwater encountered		23		23.2
34	B5@35'				- dense, decrease in silt		64	108.2	19.9
36									
38	B5@37.5'			SP	Sand, poorly graded, medium dense, wet, grayish brown, fine- to medium-grained.		17		19.7
40	B5@40'				- no recovery		11		
42	B5@42.5'						20		20.3
44									
46	B5@45'			CL	Clay, soft, wet, brown.		50	109.7	17.2
48	B5@47.5'						11		34.2
50	B5@50'								
52	B5@52.5'			SM	Silty Sand, dense, wet, brown, fine-grained.		43	116.3	19.1
54							33		16.9
56									
58	B5@57.5'				- fine- to medium-grained		41		19.9

Figure A5,
Log of Boring 5, Page 2 of 3

W1974-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS		... SAMPLING UNSUCCESSFUL		... STANDARD PENETRATION TEST		... DRIVE SAMPLE (UNDISTURBED)
		... DISTURBED OR BAG SAMPLE		... CHUNK SAMPLE		... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED.
IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

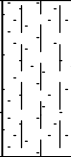
DEPTH IN FEET	SAMPLE NO.	LITHOLOGY	GROUNDWATER	SOIL CLASS (USCS)	BORING 5 ELEV. (MSL.) -- DATE COMPLETED <u>08/02/2024</u> EQUIPMENT <u>HOLLOW STEM AUGER</u> BY: <u>ACS</u>	PENETRATION RESISTANCE (BLOWS/FT*)	DRY DENSITY (P.C.F.)	MOISTURE CONTENT (%)
60					MATERIAL DESCRIPTION			
62	B5@62.5'			SM	- very dense	72		20.1
					Total depth of boring: 63.5 feet Fill to 2 feet. Groundwater encountered at 31 feet. Percolation testing performed. Backfilled with soil cuttings and tamped. *Penetration resistance for 140-pound hammer falling 30 inches by auto-hammer. NOTE: The stratification lines presented herein represent the approximate boundary between earth types; the transitions may be gradual.			

Figure A5,
Log of Boring 5, Page 3 of 3

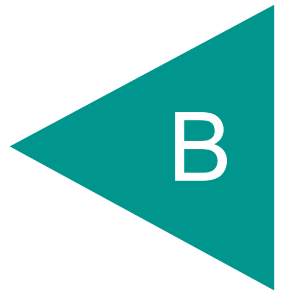
W1974-06-01 BORING LOGS.GPJ

SAMPLE SYMBOLS	 ... SAMPLING UNSUCCESSFUL	 ... STANDARD PENETRATION TEST	 ... DRIVE SAMPLE (UNDISTURBED)
	 ... DISTURBED OR BAG SAMPLE	 ... CHUNK SAMPLE	 ... WATER TABLE OR SEEPAGE

NOTE: THE LOG OF SUBSURFACE CONDITIONS SHOWN HEREON APPLIES ONLY AT THE SPECIFIC BORING OR TRENCH LOCATION AND AT THE DATE INDICATED.
IT IS NOT WARRANTED TO BE REPRESENTATIVE OF SUBSURFACE CONDITIONS AT OTHER LOCATIONS AND TIMES.

APPENDIX

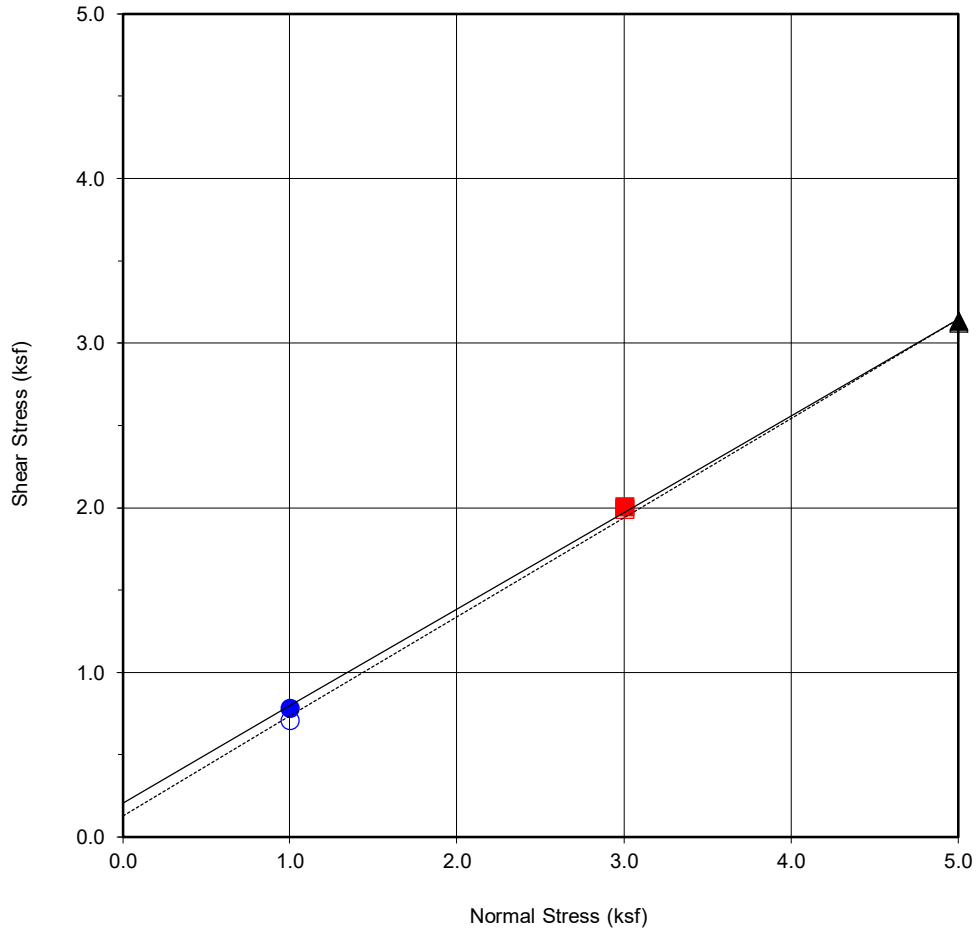
B



APPENDIX B

LABORATORY TESTING

We performed laboratory tests in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. We tested selected soil samples for in-place dry density/moisture content, expansion index, water-soluble sulfate, pH, resistivity, water-soluble chloride ion content, consolidation, and direct shear strength. The in-place dry density and moisture content of the samples tested are presented on the boring logs, Appendix A.



Boring No.	B5
Sample No.	B5@5'
Depth (ft)	5'
<u>Sample Type:</u>	RING

<u>Soil Identification:</u>		
Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	208	30
Ultimate	131	31

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.78	■ 2.00	▲ 3.13
Shear Stress @ End of Test (ksf)	○ 0.71	□ 1.99	△ 3.12
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	28.5	27.5	26.3
Initial Dry Density (pcf)	97.1	96.4	92.9
Initial Degree of Saturation (%)	104.4	99.3	87.2
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	30.2	28.4	27.7



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: ACS

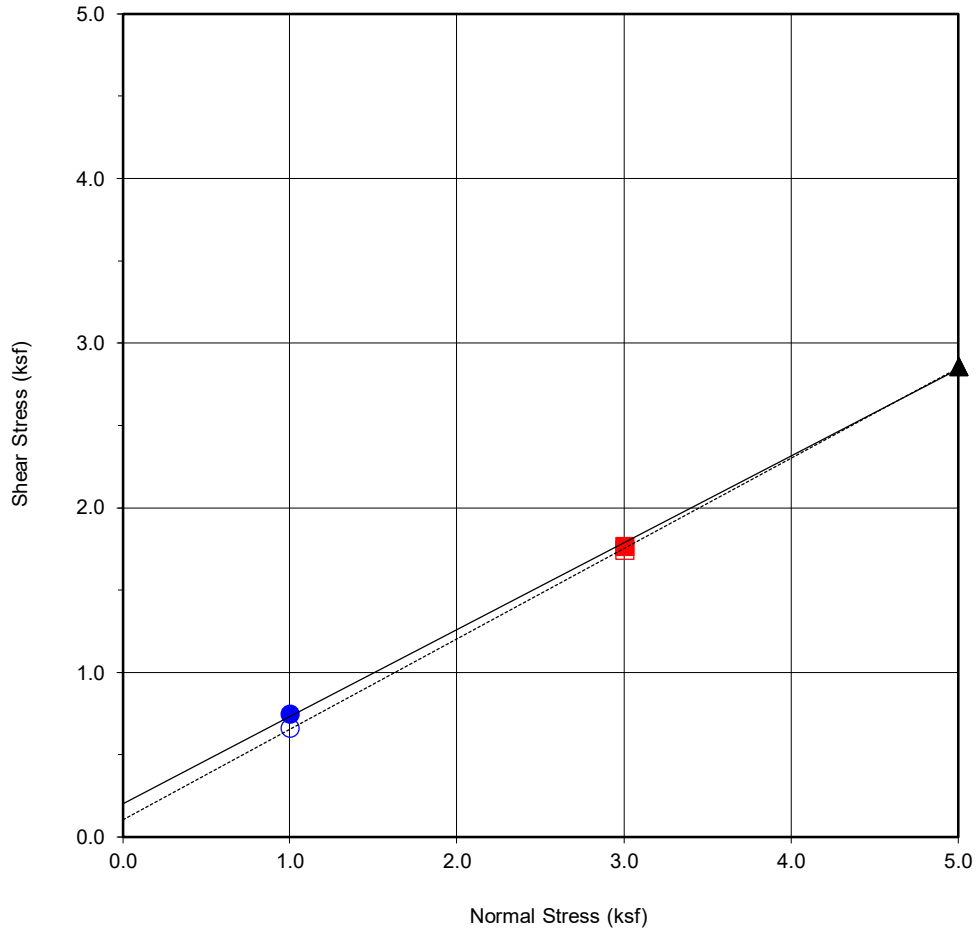
Project No.:

W1974-06-01

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

September 2024

Figure B1



Boring No.	B5
Sample No.	B5@10'
Depth (ft)	10'
Sample Type:	RING

Soil Identification:		
Sandy Silt (ML)		
Strength Parameters		
	C (psf)	ϕ (°)
Peak	204	28
Ultimate	105	29

Normal Stress (kip/ft ²)	1	3	5
Peak Shear Stress (kip/ft ²)	● 0.74	■ 1.76	▲ 2.86
Shear Stress @ End of Test (ksf)	○ 0.66	□ 1.74	△ 2.86
Deformation Rate (in./min.)	0.05	0.05	0.05
Initial Sample Height (in.)	1.0	1.0	1.0
Ring Inside Diameter (in.)	2.375	2.375	2.375
Initial Moisture Content (%)	28.7	27.1	36.2
Initial Dry Density (pcf)	93.3	90.9	89.2
Initial Degree of Saturation (%)	96.2	85.6	109.8
Soil Height Before Shearing (in.)	1.2	1.2	1.2
Final Moisture Content (%)	33.6	33.7	31.5



DIRECT SHEAR TEST RESULTS

Consolidated Drained ASTM D-3080

Checked by: ACS

Project No.:

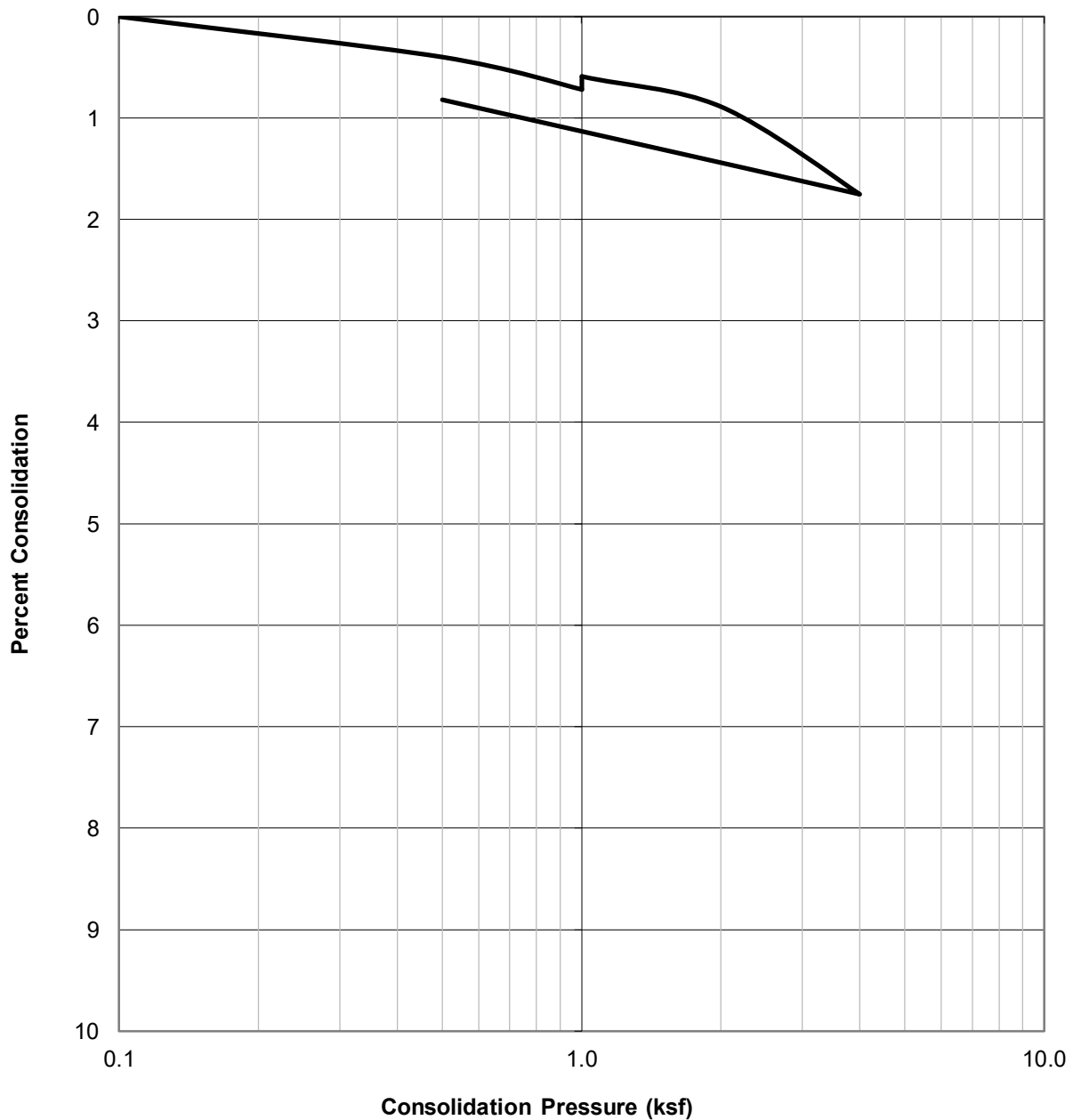
W1974-06-01

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Figure B2

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@5'	Sandy Silt (ML)	100.7	20.9	25.1



GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

Checked by: ACS

Project No.:

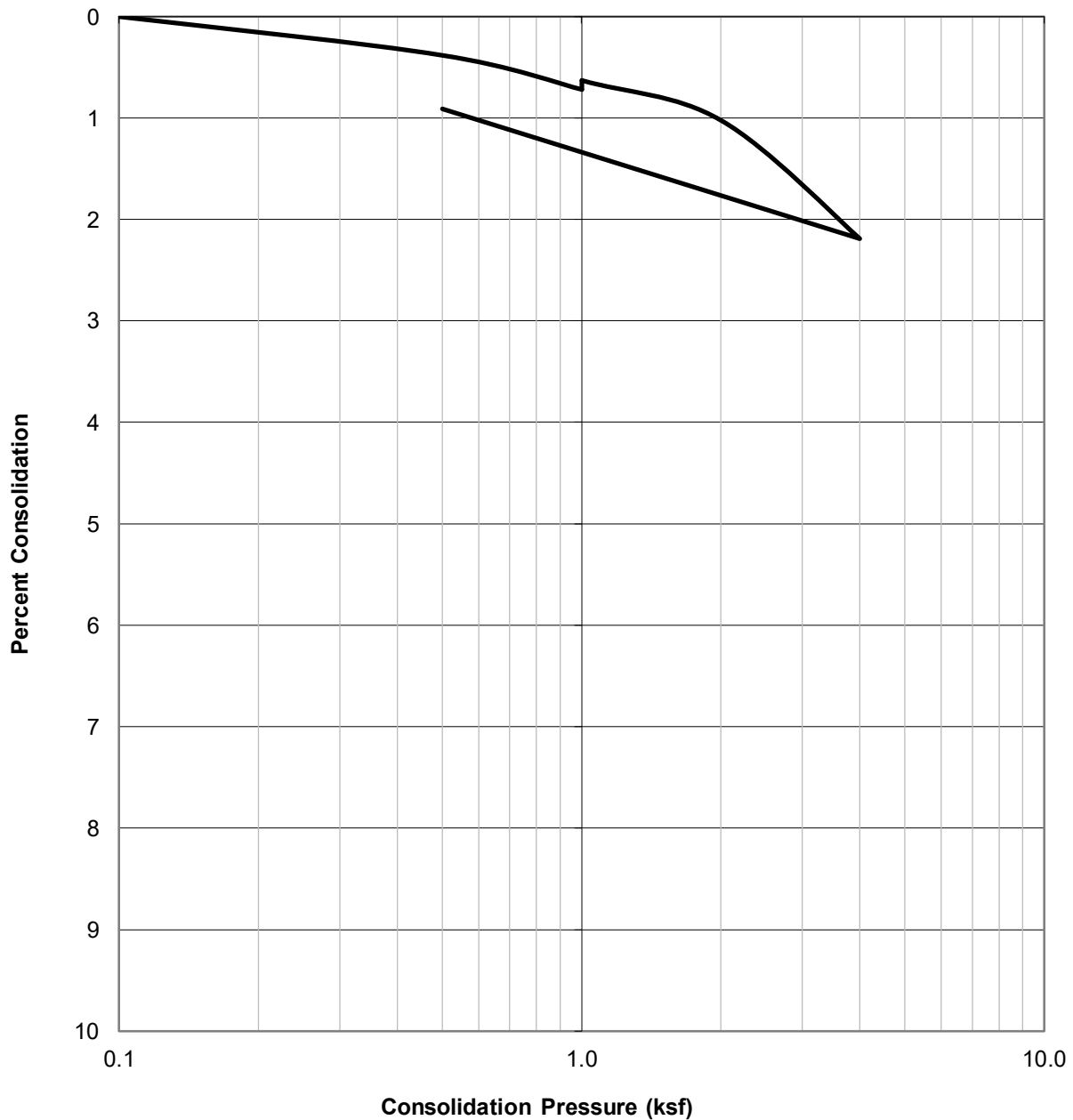
W1974-06-01

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

September 2024

Figure B3

WATER ADDED AT 1.0 KSF



SAMPLE ID.	SOIL TYPE	DRY DENSITY (PCF)	INITIAL MOISTURE (%)	FINAL MOISTURE (%)
B5@10'	Sandy Silt (ML)	90.0	31.7	33.3



GEOCON

CONSOLIDATION TEST RESULTS

ASTM D-2435

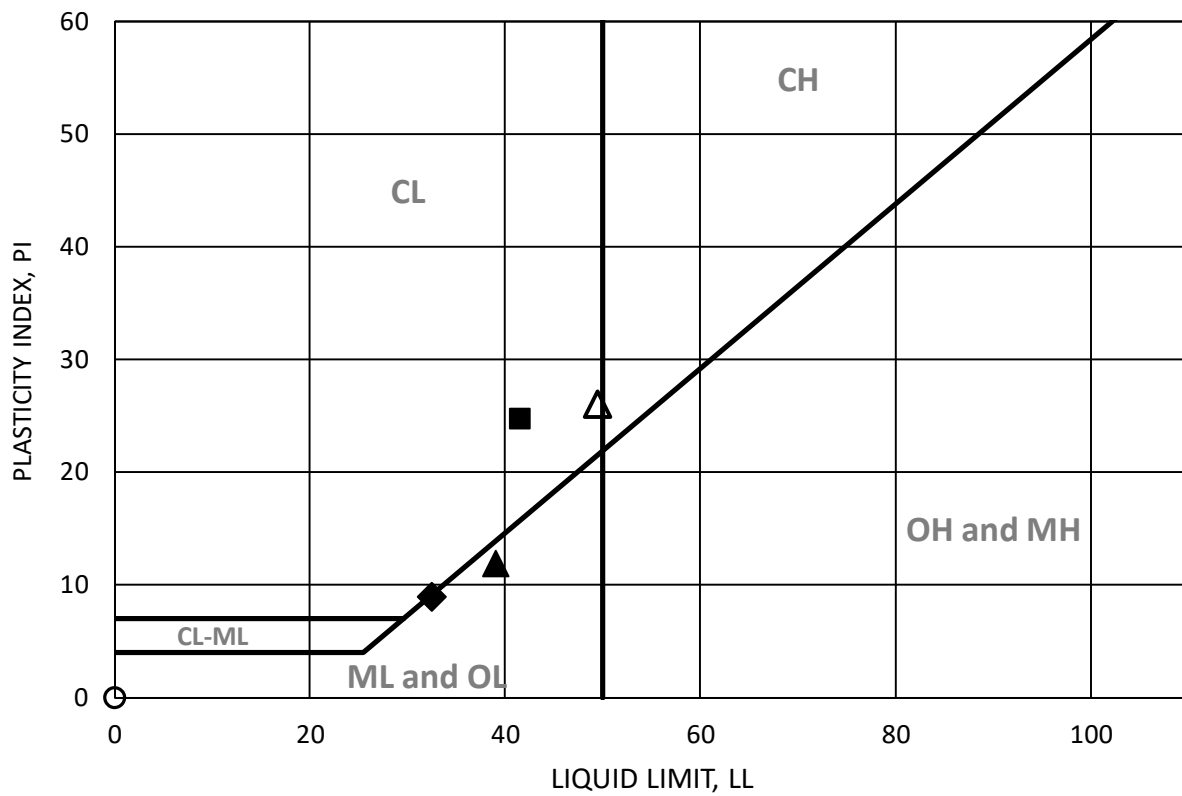
Checked by: ACS

Project No.: W1974-06-01

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

September 2024

Figure B4



SYMBOL	BORING	DEPTH (ft)	LL	PL	PI	MOISTURE CONTENT AT SATURATION	SOIL BEHAVIOR
■	B5	17.5'	41.5	17	25	23	CL
◆	B5	22.5'	33	24	9		CL
▲	B5	27.5'	39	27	12		ML
●	B5	32.5'	N/P	N/P	N/P		N/P
□	B5	37.5'	N/P	N/P	N/P		N/P
◇	B5	42.5'	N/P	N/P	N/P		N/P
△	B5	47.5'	50	23	26		CH
○							

N/P = Non-Plastic



GEOCON

ATTERBERG LIMITS

ASTM D-4318

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Project No.:

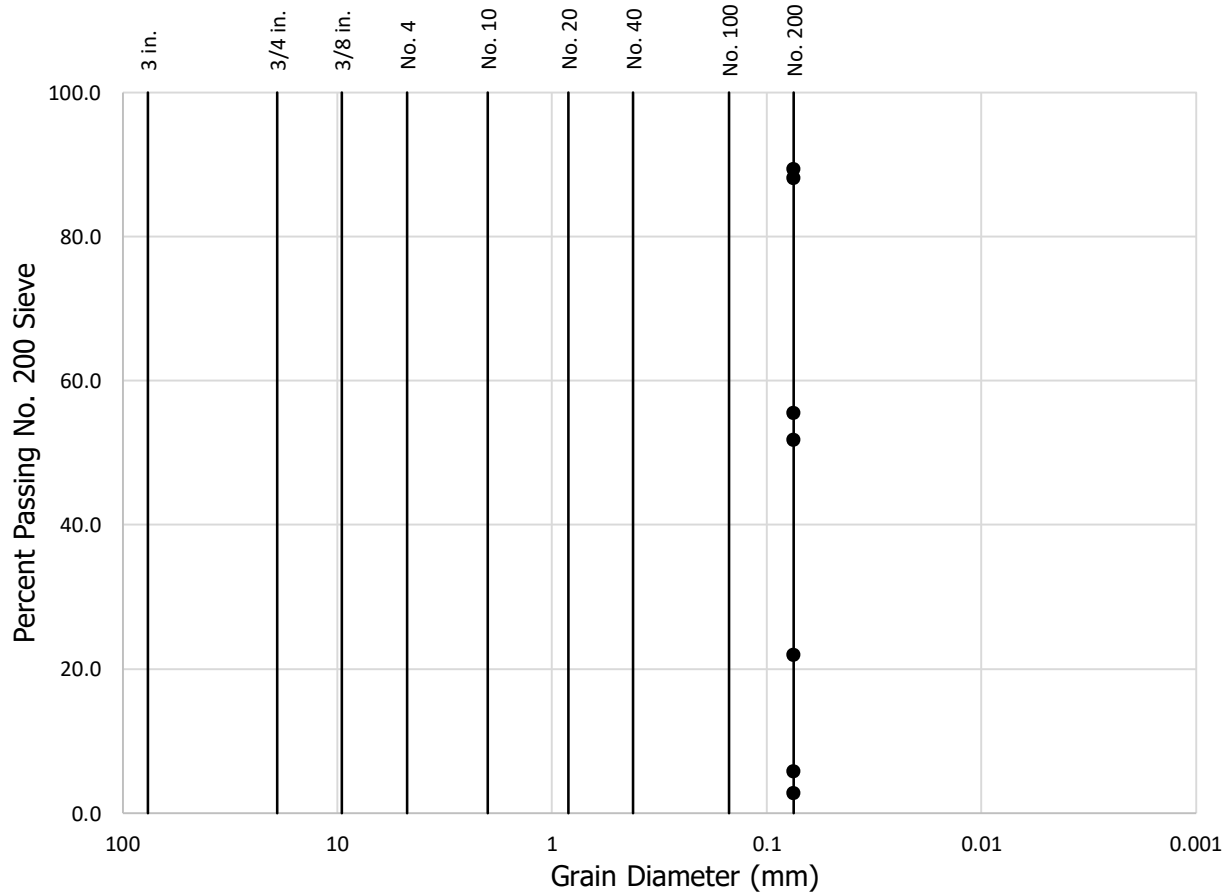
W1974-06-01

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Figure B5

GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	



Sample No.	Percent Passing No. 200 Sieve
B5 @ 17.5'	88.1
B5 @ 22.5'	55.5
B5 @ 27.5'	51.8
B5 @ 32.5'	21.9
B5 @ 37.5'	2.8
B5 @ 42.5'	5.8
B5 @ 47.5'	89.3



GEOCON

GRAIN SIZE ANALYSIS

ASTM D-1140

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Project No.:

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4900 CLARK AVENUE
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Figure B6

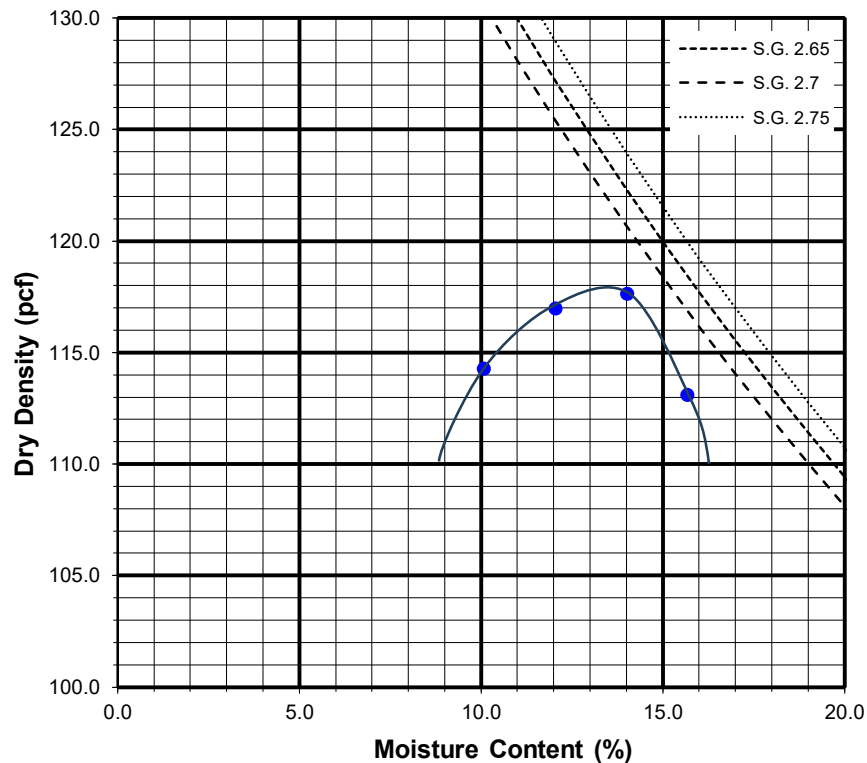
Sample No:

B1+B5 MIX	Sandy Silt (ML)
------------------	-----------------

TEST NO.		1	2	3	4	5	6
Wt. Compacted Soil + Mold	(g)	6308	6258	6262	6182		
Weight of Mold	(g)	4282	4282	4282	4282		
Net Weight of Soil	(g)	2026	1976	1980	1900		
Wet Weight of Soil + Cont.	(g)	2393.4	2389.3	2357.8	2291.0		
Dry Weight of Soil + Cont.	(g)	2145.4	2120.9	2144.7	2116.0		
Weight of Container	(g)	378.0	408.5	377.3	378.7		
Moisture Content	(%)	14.0	15.7	12.1	10.1		
Wet Density	(pcf)	134.1	130.8	131.1	125.8		
Dry Density	(pcf)	117.6	113.1	117.0	114.3		

Maximum Dry Density (pcf)	118.0
Bulk Specific Gravity (dry)	2.50
Corrected Maximum Dry Density (pcf)	120.0

Optimum Moisture Content (%)	13.0
Oversized Fraction (%)	7.0
Corrected Moisture Content (%)	12.1



Preparation Method: A



**COMPACTION CHARACTERISTICS USING
MODIFIED EFFORT TEST RESULTS**

ASTM D-1557

Checked by: ACS

Project No.: W1974-06-01

4900 CLARK AVENUE
LAKEWOOD, CALIFORNIA

Sep 24

Figure B7

B1+B5 MIX

MOLDED SPECIMEN		BEFORE TEST	AFTER TEST
Specimen Diameter	(in.)	4.0	4.0
Specimen Height	(in.)	1.0	1.0
Wt. Comp. Soil + Mold	(gm)	585.0	611.6
Wt. of Mold	(gm)	195.2	195.2
Specific Gravity	(Assumed)	2.7	2.7
Wet Wt. of Soil + Cont.	(gm)	306.8	611.6
Dry Wt. of Soil + Cont.	(gm)	276.6	350.5
Wt. of Container	(gm)	6.8	195.2
Moisture Content	(%)	11.2	18.8
Wet Density	(pcf)	117.6	125.4
Dry Density	(pcf)	105.7	105.6
Void Ratio		0.6	0.6
Total Porosity		0.4	0.4
Pore Volume	(cc)	77.2	82.8
Degree of Saturation	(%) [S_{meas}]	51.3	79.5

Date	Time	Pressure (psi)	Elapsed Time (min)	Dial Readings (in.)
8/8/2024	10:00	1.0	0	0.3824
8/8/2024	10:10	1.0	10	0.3821
Add Distilled Water to the Specimen				
8/9/2024	10:00	1.0	1430	0.4094
8/9/2024	11:00	1.0	1490	0.4094

Expansion Index (EI meas) =	27.3
Expansion Index (Report) =	27

Expansion Index, EI_{50}	CBC CLASSIFICATION *	UBC CLASSIFICATION **
0-20	Non-Expansive	Very Low
21-50	Expansive	Low
51-90	Expansive	Medium
91-130	Expansive	High
>130	Expansive	Very High

* Reference: 2022 California Building Code, Section 1803.5.3

** Reference: 1997 Uniform Building Code, Table 18-I-B.

 GEOCON	EXPANSION INDEX TEST RESULTS ASTM D-4829	Project No.:	W1974-06-01
		4900 CLARK AVENUE LAKEWOOD, CALIFORNIA	
	Checked by: ACS	September 2024	Figure B8

SUMMARY OF LABORATORY
POTENTIAL OF HYDROGEN (pH) AND RESISTIVITY TEST RESULTS
AASHTO T289 ASTM D4972 and AASHTO T288 ASTM G187

Sample No.	pH	Resistivity (ohm centimeters)
B1+B5 MIX	8.1	480 (Severely Corrosive)

SUMMARY OF LABORATORY CHLORIDE CONTENT TEST RESULTS
AASHTO T291 ASTM C1218

Sample No.	Chloride Ion Content (%)
B1+B5 MIX	0.017

SUMMARY OF LABORATORY WATER SOLUBLE SULFATE TEST RESULTS
AASHTO T290 ASTM C1580

Sample No.	Water Soluble Sulfate (% SO ₄)	Sulfate Exposure
B1+B5 MIX	0.060	S0



GEOCON

CORROSIVITY TEST RESULTS

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LAKEWOOD, CALIFORNIA

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Figure B9

SUMMARY OF LABORATORY R-VALUE TEST RESULTS

Sample No.	Soil Description	R-Value
B1 + B5 MIX	Sandy Silt (ML)	25



GEOCON

R-VALUE TEST RESULTS

ASTM D-2844

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Project No.: W1974-06-01

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LAKEWOOD, CALIFORNIA

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Figure B10