Geotechnical Evaluation

Syd Kronenthal Park Stormwater Capture Feasibility Study

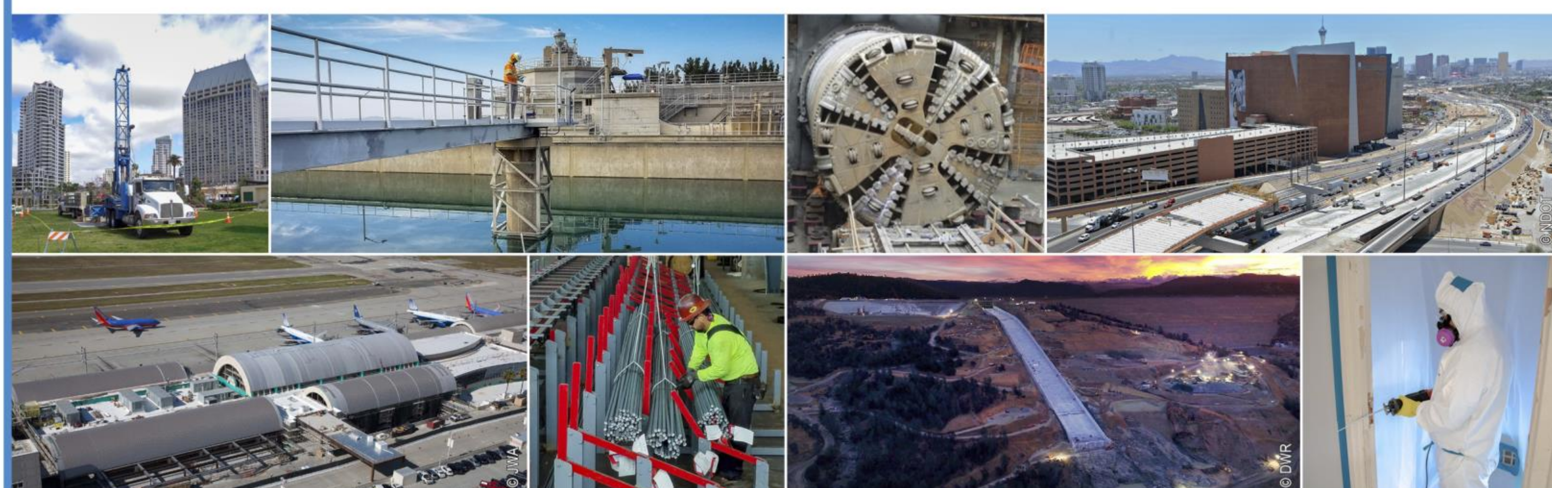
3459 McManus Avenue

Culver City, California

Paradigm Environmental

9320 Chesapeake Drive, Suite 100 | San Diego, California 92123

December 9, 2022 | Project No. 212034001



Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the Syd Kronenthal Park Stormwater Capture Feasibility Study located at 3459 McManus Avenue in Culver City, California (Figure 1). The purpose of our study was to evaluate the soil and groundwater conditions at the site, develop geotechnical recommendations for construction of the underground storage structure and associated improvements, and to evaluate the feasibility of infiltrating the captured stormwater at the park. Our evaluation was performed in general accordance with our referenced proposal dated January 11, 2022 (Ninyo & Moore, 2022). This report presents our findings, conclusions, and recommendations for the project.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.
- Review of readily available background materials, including published topographic maps, geologic maps, fault and seismic hazard maps, groundwater data, stereoscopic aerial photographs, and in-house geotechnical information.
- Acquisition of a Los Angeles County Department of Public Health Environmental Health Division (LACEHD) well permit for performing borings deeper than 10 feet.
- Field reconnaissance to observe the site conditions, mark-out the boring and percolation test locations for underground utility clearance by Underground Service Alert, and meet with personnel from the City of Culver City.
- Subsurface exploration consisting of the drilling, sampling, and logging of three hollow-stem auger borings to depths ranging from approximately 20.9 to 71$^1/_2$ feet below the ground surface. The borings were logged in the field by our representative and relatively undisturbed and bulk samples were collected and returned to our laboratory for evaluation and testing. In accordance with the LACEHD requirements, the borings were backfilled with cement-bentonite grout.
- Field percolation testing was performed in two of the borings in general accordance with the methods presented in the Los Angeles County Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration (County of Los Angeles, 2021).
- Geotechnical laboratory testing of representative soil samples to evaluate in-situ moisture content and dry density, gradation, percentage of particles finer than the No. 200 sieve, Atterberg limits, collapse/consolidation potential, direct shear strength, and soil corrosivity.
- Data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, percolation testing, and laboratory testing.
- Preparation of this geotechnical report presenting our findings, conclusions, and recommendations pertaining to the design and construction of the proposed improvements.

3 SITE DESCRIPTION

Syd Kronenthal Park is a City of Culver City park located at 3459 McManus Avenue. The park is an irregular-shaped lot bounded by National Boulevard to the south, Ballona Creek to the east, and residential properties to the north and west (Figure 2). Improvements in the southern portion of the park consist of a large grass-covered and tree-lined area with two softball fields. The northern portion of the park consists of a single-story recreation building, parking lots, tennis/pickleball courts, a basketball court, covered seating areas, hardscape, and a playground area. Topographically, the park is relatively flat and the project area has an elevation of approximately 74 feet above mean sea level (Michael Baker International, 2022).

Review of aerial photographs dating back to 1948 indicate that the park was previously developed with structures (Historic Aerials, 2022). A relatively large structure was present in the northeastern portion of the grass-covered field in 1948. The structure was demolished and the area was filled sometime between 1952 and 1964. The approximate location of the structure in 1948 is presented on Figure 2. Additionally, park maintenance staff reported that northeastern portion of the grass-covered field is uneven in some places and that subsidence may have occurred.

4 PROJECT DESCRIPTION

Based on our review of the project drawings (Michael Baker International, 2022) and the Request for Proposal (City of Culver City, 2022), it is our understanding that the project intent is to collect stormwater runoff from existing storm drains and Ballona Creek/Adams Channel into an underground storage chamber and a shallow reservoir used for a passive irrigation system. The proposed footprint of the underground storage structure is presented on Figure 2. Infiltration of the water captured in the underground storage structure is being considered; however, if the soil and/or groundwater conditions indicate that infiltration is not feasible from a geotechnical perspective, other options such as diversion to the sanitary sewer system, irrigation, and/or treatment and discharge will be utilized. Two storm diversion structures are proposed at the northeast and southern end of the storage structure. The invert depth of the storage structure is approximately 16 feet below the ground surface. We anticipate that the underground structures (storage structure and storm diversion structures) will be supported on mat foundations. Other improvements associated with the project consist of the construction of a bike bridge across Ballona Creek, a rubber dam in Adams Channel, a pump station, a force main, and a storm drain along Roberts Avenue; however, our scope was limited to evaluating the subsurface conditions in the grass-covered area of the park in order to provide geotechnical input on the design and construction of the underground storage structure and the feasibility to infiltrate the captured stormwater.

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface evaluation was conducted on October 10, 2022, and consisted of the drilling, logging, and sampling of three hollow-stem auger borings (B-1, P-1, and P-2). The borings were drilled using a truck-mounted drill rig with 8-inch diameter augers. Boring B-1 was drilled to a depth of approximately 71½ feet below the ground surface and borings P-1 and P-2 were drilled to depths of approximately 20.9 feet and 31½ feet below the ground surface, respectively. The proposed depths of borings P-1 and P-2 ranged from approximately 15 to 20 feet based on the planned design depths of infiltration; however, boring P-2 was increased to a depth of approximately 31½ feet in an attempt that to find a granular soil layer that would potentially result in higher percolation rates. Boring B-1 was drilled to a depth of approximately 71½ to evaluate for the presence of groundwater.

The borings were logged in the field by a representative of Ninyo & Moore and representative bulk and relatively undisturbed soil samples were collected from the borings at selected depths for laboratory testing. Percolation testing was performed in borings P-1 and P-2 as further discussed in Section 8 of this report. Logs of the exploratory borings are provided in Appendix A. The approximate locations of the boring and percolation tests are presented on Figure 2. The borings were backfilled with cement-bentonite grout upon completion of the drilling and percolation testing in general accordance with the requirements of LACEHD.

Geotechnical laboratory testing of representative soil samples included tests to evaluate in-situ moisture content and dry density, gradation, percentage of particles finer than the No. 200 sieve Atterberg limits, consolidation, direct shear strength, and soil corrosivity. Moisture and density test results are presented on the boring logs in Appendix A. The remaining test results are presented in Appendix B.

6 GEOLOGIC AND SUBSURFACE CONDITIONS

The project site is located in the Transverse Ranges geomorphic province in southern California. The geomorphic province encompasses an area that extends approximately 320 miles from Point Arguello and San Miguel Island on the west to the mountains bordering Joshua Tree National Monument on the east (Norris and Webb, 1990). The Transverse Ranges province varies in width from approximately 40 to 60 miles and is characterized by the east-west trending mountain ranges with the San Andreas fault system forming the northern boundary of the province. The site is located within the central block of the Los Angeles Basin bounded by the Whittier fault on the east, the Santa Monica fault to the north, and the Newport-Inglewood fault zone on the west. The central block is comprised of lowland areas of the Los Angeles coastal plain surrounded by various hills including the Coyote Hills uplift and the San Joaquin Hills.

Review of regional geologic maps indicate that the site is underlain by Holocene- to late Pleistocene-age alluvium generally consisting of unconsolidated, friable, stream-deposited silt, sand, and gravel on flood plains (Campbell et al., 2014) (Figure 3). However, the park is located adjacent to Ballona Creek; therefore, wash deposits associated with deposition from streamflow within the Ballona Creek before it was channelized may also be present

Materials encountered during our subsurface exploration generally consisted of undocumented fill underlain by alluvium. Undocumented fill was encountered in each of the borings to depths ranging from approximately 2 to 7 feet below the ground surface. The undocumented fill generally consisted of moist, firm, lean clay and loose to medium dense, clayey sand, silty sand, and sandy silt. Variable amounts of gravel, cobbles, concrete debris, and asphalt debris were encountered in the undocumented fill. Documentation regarding the limits of fill or the placement and compaction of the fill soils was not available for our review. Alluvium was encountered beneath the undocumented fill to the total depth explored of up to approximately 71½ feet. The alluvium generally consisted of moist to west, stiff to hard, lean clay and loose to very dense, clayey sand, silty sand, poorly graded sand with silt, well-graded sand with silt, and poorly graded sand. Variable amounts of gravel were encountered in the alluvium. The more granular soils encountered in boring B-1 and near the bottom of percolation test hole P-2 may be related to wash deposits from active deposition along Ballona Creek while the clayey deposits encountered in percolation test hole P-2 may be associated with floodplain deposition. More detailed descriptions of the subsurface materials are presented on the boring logs in Appendix A.

7 GROUNDWATER

Groundwater was observed at the time of drilling in exploratory boring B-1 at a depth of approximately 67 feet below the ground surface. The groundwater depth observed at the time of drilling is not considered a stabilized groundwater condition and may vary from the recorded level. Regional maps indicate that the historic high depth to groundwater at the project site is approximately 15 feet below the ground surface (California Division of Mines and Geology [CDMG], 1998). Groundwater monitoring well data from the State of California Department of Water Resources website (2022) indicates that the depth to groundwater at two monitoring wells located within a $^{1}/_{4}$-mile radius from the site, ranged from approximately 11 to 73 feet below the ground surface. Groundwater levels are subject to variation due to seasonal rainfall, irrigation, groundwater pumping, subsurface stratigraphy, topography, and other factors which may not have been evident at the time of our evaluation.

8 FIELD PERCOLATION TESTING

Percolation testing was performed in borings P-1 and P-2 in general accordance with the County of Los Angeles Guidelines for Geotechnical Investigation and Reporting Low Impact Development Stormwater Infiltration (County of Los Angeles, 2021). The testing was performed to evaluate the infiltration rate of the on-site soils for use in design of the BMPs. The approximate locations of the percolation test borings are shown on Figure 2.

Borings P-1 and P-2 were drilled to depths of approximately 20.9 feet and 31½ feet, respectively. As stated above, both percolation tests were originally planned for depths ranging from approximately 15 to 20 feet based on the proposed depths of the BMPs; however, boring P-2 was increased to a depth of approximately 31½ feet in an attempt to find a granular soil layer that would potentially result in higher percolation rates. Clayey soils were encountered from a depth of 20 to 31½ feet in boring P-2 and granular soils were not encountered; therefore, we performed the infiltration test in boring P-2 in the originally planned depth interval.

Preparation of each boring for percolation testing included the installation of a 2-inch-diameter slotted polyvinyl chloride (PVC) pipe in the boring and backfilling the annular space between the borehole wall and pipe with clean gravel. The infiltration zones were pre-soaked with water for at least one hour prior to performing percolation testing. After the borings were pre-soaked, constant-head percolation testing was performed in boring P-1 and falling-head percolation testing was performed in boring P-2.

The constant-head test method involved placing and maintaining a constant head of clean water into the PVC pipe and measuring the flow rate in gallons per minute required to keep the water level constant inside the borehole. A flow meter was used to record the volumetric flow rate of water entering the test boring. Once a stabilized head was established in the boring, the constant-head test was initiated and the flow was maintained for a period of approximately three hours. The field percolation rate was calculated by dividing the average stabilized volumetric rate by the total surface area of infiltration within the boring. The measured field percolation rate is presented in Table 1.

For the falling-head percolation test, clean water was placed in the PVC pipe to establish a head of water and the rate at which the water level dropped in the pipe at consecutive time intervals (approximately 30 minutes) was measured. The test readings were repeated for three hours and until a stabilized rate was obtained. The field percolation rate was calculated by measuring the total volume of water infiltrated during the time intervals and dividing by the surface area of the tested zone of the boring based on the average of the last three consecutive readings. The measured field percolation rate is presented in Table 1.

The County of Los Angeles guidelines indicate that the measured field percolation rates should be reduced to account for the long-term performance of the proposed improvements by dividing the rates by the "Total Reduction Factor (RF)." They define the RF as the sum of the "test-specific" reduction factor (RF_t), the "site variability" reduction factor (RF_v), and the "long-term siltation, plugging, and maintenance" reduction factor (RF_s) (i.e., $RF = RF_t + RF_v + RF_s$). The guidelines indicate that the RF_t should be applied to account for variations in the direction of flow during the test and the reliability of the different test methods. The guidelines provide RF_t values to be used in the equation that vary based on the test method performed. A value of 2 and 3 is specified for the falling-head and constant-head (high flow rate) percolation tests, respectively, and was applied to the RF equation accordingly. The RF_v value is applied to account for site variability, number of tests, and thoroughness of the subsurface investigation and ranges from 1 to 3. Based on the limited number of percolation tests performed and the variability of the on-site soils, we recommend using an RF_v value of 3. This value may be adjusted during the final design phase if additional percolation testing is performed. The long-term siltation, plugging, and maintenance value (RF_s) also ranges from 1 to 3 and will generally vary on the level of pre-treatment performed prior to infiltration and the level of future maintenance of the system. For the purposes of this evaluation, we have assumed an RF_s value of 1; however, the RF_s value should be provided by the BMP designer. The RF_t, RF_v, RF_s, and resulting RF values used in our analysis are presented in Table 1. The adjusted preliminary percolation rates based on these values are also presented in Table 1.

Table 1 – Percolation Test Results

Test Boring	Test Type	Approximate Depth of Tested Zone (feet)	Field Percolation Rate (inches/hour)	Reduction Factor				Adjusted Percolation Rate (inches/hour)
				RF_t	RF_v	RF_s	RF	
P-1	Constant Head	16 – 20½	27.8	3	3	1	7	4.0
P-2	Falling Head	14 – 20½	0.17	2	3	1	6	0.03

Notes:
RF_t – Test Specific Reduction Factor
RF_v – Site Variability Reduction Factor
RF_s – Long-Term Siltation, Plugging, and Maintenance Reduction Factor (To be adjusted by the BMP designer as needed)
RF – Total Reduction Factor

9 FLOOD HAZARDS

Based on our review of flood insurance rate maps for the project area (Federal Emergency Management Agency [FEMA], 2018), the project site is not located in the 100-year Flood Hazard Zone, A99. Zone A99 includes areas to be protected from a 100-year flood by the Federal Flood Protection System under construction at the time of publication of the FEMA map; no base flood elevations are given. The site is located within Zone X, which includes areas with a 0.2 percent annual chance of flood hazard (areas with one percent annual chance of flood with an average depth less than one-foot or with drainage areas less than one square mile).

10 FAULTING AND SEISMICITY

The site is in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the proposed project. Figure 4 shows the approximate site location relative to the major faults in the region. The site is located within a State of California EFZ (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 2018). The nearest mapped active fault to the site is the Newport-Inglewood fault located less than 0.10 mile southwest of the site (California Geological Survey [CGS], 2022).

The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, and liquefaction. These potential hazards are discussed in the following sections.

10.1 Surface Fault Rupture

Surface fault rupture is the offset or rupturing of the ground surface by relative displacement across a fault during an earthquake. Based on our review of referenced geologic and fault hazard data, the mapped trace of the Newport-Inglewood fault is approximately 600 feet southwest of the site and the majority of the field area of the park is located within an Earthquake Fault Zone (Figure 6). Since the proposed project does not include structures for human occupancy, evaluation of the surface fault rupture hazard at the site was not needed for this project. Without a site-specific fault study to evaluate for the presence of active faults, the potential for surface rupture cannot be ruled out. Lurching or cracking of the ground surface as a result of nearby seismic events is also possible at the site.

10.2 Site-Specific Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2019 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures Using the measured standard penetration test (SPT) blow counts from boring B-1, we calculated that the average shear wave velocity in the upper 100 feet (30 meters) of the subsurface profile (V_{S30}) is approximately 919 feet per second (280 meters per second) based on empirical correlations (Brandenberg et al., 2010). In accordance with Chapter 20 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures, the site classification is Site Class D.

Per the 2022 CBC, a site-specific ground motion hazard analysis shall be performed in accordance with Section 21.2 of ASCE 7-16 for structures on Site Class D with a mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g. We calculated that the S_1 for the site is equal to 0.71g using the 2022 Applied Technology Council (ATC) seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project area.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum (ARS) curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the mapped MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 using the 2022 ATC seismic design tool. The depths to V_S = 3,281 ft/s (1,000 m/s) and V_S = 8,202 ft/s (2,500 m/s) are assumed to be 1,969 feet (600 meters) and 14,272 feet (4,350 meters), respectively (Southern California Earthquake Center, 2014). These values were evaluated using the Open Seismic Hazard Analysis (OpenSHA) software developed by United States Geological Survey (USGS, 2021).

The 2014 new generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships that we used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The OpenSHA software (USGS, 2021) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center was used for performing the DSHA (Seyhan, 2014).

PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per Section 21.2.1.1 of ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the hazard curves and deaggregation plots at the site obtained from the USGS Unified Hazard Tool application (USGS, 2022). A magnitude 7.5 event on the Compton fault with a rupture distance of 12 kilometers from the site was evaluated to be the controlling earthquake. Even though the Newport-Inglewood fault is closer to the site, the Compton fault resulted in higher accelerations due to the nature of the fault type. Hence, the DSHA was performed for the site using this event and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 5 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 5 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2022 CBC, are provided in Section 12.2 for the evaluation of seismic loads on buildings and other structures.

ASCE 7-16 specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the maximum considered earthquake geometric mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). The PGA_M is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The site-specific PGA_M was calculated as 0.915g.

10.3 Liquefaction

Liquefaction is the phenomenon in which loosely deposited granular soils with silt and clay contents of less than approximately 35 percent and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near saturated cohesionless soils at depths shallower than 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

The State of California Hazard Zones map (CGS, 2018) indicates that the subject site is located within a mapped area that is considered susceptible to seismically induced liquefaction (Figure 6). Accordingly, the liquefaction potential of the subsurface soils was evaluated using the boring data recorded during our subsurface exploration and our laboratory test results of representative soil samples. The liquefaction analysis was based on the National Center for Earthquake Engineering Research procedure (Youd, et al., 2001) using the computer program LiquefyPro (CivilTech, 2019). A groundwater depth of 15 feet was used in our analysis based on the historic high depth to groundwater. A PGA_M of 0.915g was used in our analysis for a design earthquake magnitude of 7.5. Due to the presence of clayey soils and dense sands below a depth of 15 feet, it is our opinion that liquefaction is not a design consideration for the project.

11 CONCLUSIONS

Based on the results of our evaluation and infiltration testing, the on-site soils and infiltration rates are highly variable. The variability in the test results is due to the variability of the soil conditions. The varying soil conditions may be related to the depositional environments of the coarser sands associated with the active Ballona Creek prior to channelization as comparted to finer floodplain deposits that would be deposited along the banks of Ballona Creek.

The test results from our two percolation tests P-1 and P-2 indicate that the infiltration rates of the on-site soils are 4.0 and 0.03 inches per hour, respectively. An infiltration rate of 0.03 inch per hour will not meet the County of Los Angeles minimum rate for infiltration (0.3 inch per hour). The soils in the upper 20 feet of our borings generally consisted of lean clay and clayey sand. Due to presence of clayey soils and the low infiltration rate of percolation test P-2, large-scale infiltration at the site is generally not considered to be feasible; however, the underground storage structure may still be utilized to store stormwater runoff.

Performing additional infiltration testing at different locations and depths at the subject site in a future design phase will be appropriate to evaluate the overall infiltration rate of the on-site soils and the feasibility of infiltration, or if smaller-scale infiltration in selected areas is feasible. Performing additional subsurface exploration in selected project areas prior to mobilization for constructing the percolation test holes may also be considered to evaluate for the presence of more coarse-grained soils so that the percolation test holes can target those layers for testing.

It is our opinion that construction of the underground storage structure for the project is feasible from a geotechnical standpoint, provided the recommendations presented in this report are incorporated into the design and construction of the project. Geologic mapping of the underground storage structure excavation bottom may reveal the boundary between alluvial wash and floodplain deposits and may further guide where coarser soils may be present that could be suitable for infiltration. In general, the following conclusions were made:

- The subject site is underlain by undocumented fill overlying alluvial materials. The thickness of the undocumented fill encountered in our borings ranged from approximately 2 to 7 feet below the ground surface. The undocumented fill generally consisted of moist, firm, lean clay and loose to medium dense, clayey sand, silty sand, and sandy silt. Concrete and asphalt debris were encountered in the undocumented fill. The alluvium generally consisted of moist to west, stiff to hard, lean clay and loose to very dense, clayey sand, silty sand, poorly graded sand with silt, well-graded sand with silt, and poorly graded sand. Variable amounts of gravel were encountered in the alluvium.

- Our two percolation tests performed in borings P-1 and P-2 indicate that the on-site soils tested at depths ranging from approximately 15 to 20 feet have adjusted percolation rates ranging from approximately 0.03 to 4.0 inches per hour. An infiltration rate of 0.03 inch per hour will not meet the County of Los Angeles minimum rate for infiltration.

- Based upon our review of historical aerial photographs, a relatively large structure in the northeastern portion of the grass-covered field was demolished and filled in between 1952 and 1964. Documentation regarding fill placement was not available at the time of our evaluation. Due to the variable thickness and material types that might comprise the fill soils, there is a potential for settlement in this location. Remedial grading consisting of the overexcavation and recompaction of the existing fill should be performed prior to construction of the BMPs.

- In general, excavations in the existing fill soil and alluvium should be feasible with earthmoving equipment in good working condition. Some of the granular soils that will be encountered near the subgrade elevation of the underground storage structure are very dense and may involve additional excavation effort. Oversized materials and deleterious materials in the undocumented fill should be anticipated by the contractor.

- We anticipate that the on-site excavated materials should be suitable for re-use as engineered fill and trench backfill provided that they are free of trash, debris, roots, contamination, deleterious materials, and cobbles or hard lumps of material in excess of 4 inches in diameter. Processing of the materials to bring them near the laboratory optimum moisture content (i.e., drying and/or wetting) prior to use as fill should be planned by the contractor.

- On-site soils should be considered as Type C soils in accordance with Occupational Safety and Health Administration (OSHA) soil classifications. The on-site soils will be subject to caving. Where excavations cannot be laid back, temporary shoring is anticipated. Shoring should be designed by the contractor to support the excavation sidewalls and to reduce the potential for settlement of adjacent structures, roadways, and other site improvements. Shoring should be designed in accordance with OSHA regulations.

- Groundwater was encountered in boring B-1 at a depth of approximately 67 feet below the ground surface. The historic high depth to groundwater is mapped as being approximately 15 feet at the site (CDMG, 1998). Based on our review of groundwater monitoring well data (State of California, 2022), groundwater was encountered at depths ranging from approximately 11 to 73 feet below the ground surface in two monitoring wells located within a $^{1}/_{4}$-mile radius from the site. Fluctuations in the level of groundwater will occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors that were not evident at the time of our field evaluation.

- The site is located within an Earthquake Fault Zone with the potential for fault rupture as defined by the Alquist-Priolo Earthquake Fault Zoning Act (Hart and Bryant, 2018). Strong ground shaking should be anticipated and potential fault rupture may occur. Without a site-specific fault evaluation to check that a fault does not transect the site, the potential for surface rupture cannot be ruled out.
- The site is located within a mapped Seismic Hazards Zone considered susceptible to liquefaction (CDMG, 1998). Due to the presence of clayey soils and dense sands below a depth of 15 feet, it is our opinion that liquefaction is not a design consideration for the project.
- The site is not located within a designated flood inundation zone from failure of a dam or the 100-year and 500-year flood events (FEMA, 2018).
- Based on our laboratory corrosion testing, the on-site soils should be classified as corrosive based on the California Department of Transportation (Caltrans) Corrosion Guidelines (2021).

12 RECOMMENDATIONS

The following sections present our geotechnical recommendations for design and construction of the project. Our subsurface exploration and percolation testing indicate highly variable soil conditions and that large-scale infiltration may not be feasible. We recommend performing additional infiltration testing at different locations and depths within the project area during a subsequent design phase to better characterize the infiltration rates of the on-site soils if stormwater infiltration will be incorporated into the project. Our recommendations are presented below with the understanding that infiltration at the site will not be performed as part of this project.

This project is in the preliminary design phase and some aspects of the design will be subject to change. Accordingly, the following recommendations should be considered preliminary. Ninyo & Moore should review the final plans and develop additional geotechnical recommendations as appropriate. These recommendations are based on our evaluation of the site geotechnical conditions, our understanding of the planned construction, and experience in the vicinity of the project. The work should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards.

12.1 Earthwork

We anticipate that earthwork at the site will consist of cuts and fills associated with excavations to install the proposed BMP improvements, including the underground storage structure and storm diversion structures. Based on the preliminary plans (Michael Baker International, 2022), excavations for the proposed BMP improvements will be on the order of 20 feet in depth. Earthwork will also include trenching and backfilling for new utilities and finish grading for establishment of site drainage. Earthwork operations should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections.

12.1.1 Pre-Construction Conference

We recommend that grading and foundation plans be submitted to Ninyo & Moore for review to check for conformance to the recommendations provided in this report. We further recommend that a pre-construction conference be held to discuss the grading recommendations presented in this report. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan, project schedule, and earthwork requirements.

12.1.2 Clearing and Site Preparation

Prior to excavating or other earthwork, the proposed area of improvements should be cleared of surface obstructions, debris, pavement, abandoned utilities, and other deleterious materials. Obstructions that extend below finish grade, if any, should be removed and the resulting holes filled with compacted soils. Existing utilities should be re-routed or protected from damage by equipment. Materials generated from the clearing operations should be removed from the project site and disposed at a legal dump site.

12.1.3 Excavation Characteristics

We anticipate that excavations in the undocumented fill and alluvium should be feasible with earthmoving equipment in good working order. Some of the granular soils that will be encountered near the subgrade elevation of the underground storage structure are very dense and may involve additional excavation effort. The fill and alluvial materials generally consisted of moist to west, stiff to hard, lean clay and loose to very dense, clayey sand, silty sand, sandy silt, poorly graded sand with silt, well-graded sand with silt, and poorly graded sand. Variable amounts of gravel, cobbles, concrete debris, and asphalt debris were encountered in the undocumented fill and should be anticipated in the excavations. Processing of the materials to bring them near the laboratory optimum moisture content (i.e., drying and/or wetting) prior to use as fill should be planned by the contractor.

12.1.4 Subgrade Preparation for Buried Structures

Based on our exploratory borings, alluvium is anticipated at the bottom of the planned buried improvements, including the new storage structure and storm diversion structures. However, deeper undocumented fill may be present within the footprint of the structure that was previously abandoned. In order to provide suitable support for proposed buried structures, we recommend that the existing undocumented fill and upper loose alluvial deposits be removed from beneath the structures. The excavation bottom for the underground storage structure, and possibly the diversion structures, may expose very dense granular soils and very stiff clay. In order to provide more consistent soil conditions beneath the structures, the structure foundation footprint areas should be overexcavated 2 feet or more and the material blended and placed as newly compacted fill material. The overexcavation should remove undocumented fill and expose relatively dense/stiff alluvial deposits. Additional overexcavation of loose, soft, and/or wet areas may be appropriate. The excavation bottom should be evaluated by our representative during the excavation work and additional recommendations, if needed, be based on field observations. The limits of removal should extend approximately 2 feet beyond the footprint of the foundations. If drainage rock is placed beneath the foundations, this can be considered part of the 2-foot thick layer of compacted fill beneath the foundations. Prior to placing compacted fill and/or drainage rock, the upper approximately 8 inches of the exposed bottom should be scarified, moisture-conditioned to near optimum moisture content, and recompacted to a relative compaction of 90 percent as evaluated by ASTM International (ASTM) D 1557.

12.1.5 Temporary Excavations and Shoring

Temporary near-vertical excavations not exceeding a depth of approximately 4 feet should be feasible. Excavations that are unstable or deeper than 4 feet should be laid back to slope inclinations of approximately 1½:1 (horizontal to vertical) or flatter. For deeper excavations or where temporary slopes are not possible, shoring will be involved. Excavations should be performed in accordance with OSHA regulations. On-site soils should be considered as Type C soils in accordance with OSHA guidelines.

Shoring systems should be designed for the anticipated soil conditions using the lateral earth pressure values shown on Figures 7 and 8 for braced and cantilevered excavations, respectively. The recommended design pressures are based on the assumption that the shoring system is constructed without raising the ground surface elevation behind the shored sidewalls of the excavation, that there are no surcharge loads, such as soil stockpiles and construction materials, and that no loads act above a 1:1 (horizontal to vertical) plane ascending from the base of the shoring system. For a shoring system subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures acting on the shored walls.

We anticipate that settlement of the ground surface will occur behind the shored excavation. The amount of settlement depends heavily on the type of shoring system, the contractor's workmanship, and soil conditions. To reduce the potential for distress to adjacent improvements, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to ½ inch or less. Possible causes of settlement that should be addressed include settlement during installation of the shoring elements, excavation for structure construction, construction vibrations, and removal of the support system. We recommend that shoring installation be evaluated carefully by the contractor prior to construction and that ground vibration and settlement monitoring be performed during construction.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are minimum requirements, and the contractor should evaluate the adequacy of these parameters and make the appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed.

12.1.6 Fill Material

In general, the on-site soils should be suitable for reuse as fill materials, provided they are free of trash, debris, oversize material, or other deleterious materials. Fill should generally be free of rocks or lumps of material in excess of 4 inches in diameter. Rocks or hard lumps larger than approximately 4 inches in diameter should be broken into smaller pieces or should be removed from the site.

Imported fill material, if used, should also consist of clean, granular material with a very low expansion potential, corresponding to an expansion index of 20 or less. The soil should also be tested for corrosive properties prior to importing. We recommend that the imported materials satisfy the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of less than 500 parts per million [ppm], a soluble sulfate content of less than approximately 0.15 percent (1,500 ppm), a pH value of more than 5.5, or an electrical resistivity of more than 1,500 ohm-centimeters). Materials for use as fill should be evaluated by Ninyo & Moore prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

12.1.7 Fill Placement and Compaction

Fill material, including trench backfill, should be moisture conditioned and compacted in horizontal lifts to a relative compaction of 90 percent or more as evaluated by ASTM D 1557.

Fill material should be moisture-conditioned to slightly above the laboratory optimum moisture content. The lift thickness for fill soils will depend on the type of compaction equipment used but generally should not exceed 8 inches in loose thickness. Special care should be exercised to avoid damaging pipes during compaction of trench backfill. Placement and compaction of the fill soils should be in general accordance with local grading ordinances and good construction practice.

12.1.8 Pipe Bedding

We recommend that pipes be supported on 6 inches or more of granular bedding material, such as sand, with a sand equivalent value of 30 or more. Bedding material should be placed around the pipe and 12 inches or more above the top of the pipe in accordance with the current “Greenbook” Standard Specifications for Public Works. We do not recommend the use of crushed rock as bedding material. It has been our experience that the voids within crushed rock are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the surfaces.

Special care should be taken not to allow voids beneath the pipe. Compaction of the bedding material and backfill should proceed along both sides of the pipe concurrently. Trench backfill, including bedding material, should be placed and compacted with mechanical equipment in accordance with the recommendations presented in the Earthwork section of this report.

12.1.9 Modulus of Soil Reaction for Pipe Design

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed along the sides of buried flexible pipelines for the purpose of evaluating deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 400 pounds per square inch be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in the previous section.

12.2 Site-Specific Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 2 presents the site-specific spectral response acceleration parameters in accordance with the 2019 CBC guidelines.

Table 2 – 2019 California Building Code Seismic Design Criteria	
Site-Specific Spectral Response Acceleration Parameters	**Values**
Site Classification	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_s	2.004g
Mapped MCE_R Spectral Response Acceleration at 1.0-Second Period, S_1	0.710g
MCE_R Spectral Response Acceleration at Short Periods Adjusted for Site Class, S_{MS}	2.128g
MCE_R Spectral Response Acceleration at 1.0-Second Period Adjusted for Site Class, S_{M1}	1.921g
Design Spectral Response Acceleration at Short Periods, S_{DS}	1.419g
Design Spectral Response Acceleration at 1.0-Second Period, S_{D1}	1.280g
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.915g

12.3 Mat Foundations

It is our opinion that the proposed underground structures (storage structure and storm diversion structures) may be supported by mat foundations. Mat foundations should be founded approximately 12 inches below the adjacent finish grade and supported by compacted fill. Mat foundations may be designed using a net allowable bearing capacity of 3,000 pounds per square foot (psf). The total and differential settlement corresponding to this allowable bearing load are estimated to be less than approximately 1 inch and ½ inch over a horizontal span of 40 feet, respectively. Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils underlying the mat. A design modulus of subgrade reaction of 50 tons per cubic foot may be used for the compacted subgrade soils in evaluating such deflections.

Foundations bearing on compacted fill may be designed using a coefficient of friction of 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. Foundations may be designed using a passive resistance of 300 psf per foot of depth for level ground condition up to a value of 3,000 psf. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

12.4 Lateral Earth Pressures for Thrust Blocks

Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the passive lateral earth pressures presented on Figure 9. Excavations for construction of thrust blocks should be backfilled with granular backfill material and compacted following the recommendations presented in this report.

12.5 Lateral Earth Pressures

Walls for below-grade structures when constructed as recommended above may be designed for lateral pressures represented by the pressure diagram on Figure 10. To reduce the potential for pipe-to-wall differential settlement, which could cause pipe shearing, we recommend that a flexible pipe joint be located close to the exterior of the wall. The type of joint should be such that minor relative movement can be accommodated without distress. The pipe connections should be sufficiently flexible to withstand differential settlement of approximately ¾ inch.

12.6 Exterior Flatwork

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a thickness of 4 inches and be reinforced with No. 3 steel reinforcing bars placed 24 inches on-center (each way) near the mid-height of the slab. The hardscape should be underlain by 4 inches of clean sand and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape, should be doweled.

12.7 Corrosivity

Laboratory testing was performed on a representative sample of near-surface soil to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. Chloride content testing was performed in general accordance with CT 422. Sulfate content testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix B.

The soil pH of the sample tested was measured to be 6.9 and the electrical resistivity was measured to be 745 ohm-centimeters. The chloride content of the sample was measured to be 190 parts per million (ppm). The sulfate content of the sample was measured to be 0.006 percent by weight (i.e., 60 ppm). Based on the laboratory test results and Caltrans criteria (2021), the project site should be classified as a corrosive site, which is defined as having earth materials with a pH of less than 5.5, an electrical resistivity of less than 1,500 ohm-centimeters, chloride concentrations of more than 500 ppm, and more than 0.15 percent sulfates (i.e., 1,500 ppm). A corrosion engineer should be consulted if corrosion susceptible improvements are planned.

12.8 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. Based on the CBC (2019), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight, moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight, severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight, and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil sample tested for this evaluation, using CT 417, indicate a water-soluble sulfate content of approximately 0.006 percent by weight (i.e., 60 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of the on-site soils, consideration should be given to using Type II/V cement for the project.

To reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed improvements be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with CBC (2019). The structural engineer should be consulted for additional concrete specifications.

12.9 Drainage

Proper surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from existing foundations. Positive drainage is defined as a slope of 2 percent or more for a distance of 5 feet or more away from foundations and tops of slopes. Surface waters should not be allowed to pond adjacent to foundations. We recommend that above-ground structures, if constructed, have roof drains and downspouts installed to collect runoff.

12.10 Landscaping

Project landscaping should consist of drought tolerant plants. Landscape irrigation should be kept to a level just sufficient to maintain plant vigor. Overwatering should not be permitted

13 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the geotechnical consultant checks the interpolated subsurface conditions during construction. We recommend that Ninyo & Moore review the project plans and specifications prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report may be revised or modified.

During construction we recommend that the duties of the geotechnical consultant include, but not be limited to:

- Observing site clearing, grubbing, and removals.
- Observing excavation bottoms, and the placement and compaction of fill, including trench backfill.
- Evaluating imported materials prior to their use as fill (if used).
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Performing material testing services including concrete compressive strength and steel tensile strength tests and inspections.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of this project. If another geotechnical consultant is selected, we request that the selected consultant indicate to the owner and to our firm in writing that our recommendations are understood and that they are in full agreement with our recommendations.

14 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analysis presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request. Please also note that our evaluation was limited to assessment of the geotechnical aspects of the project, and did not include evaluation of structural issues, environmental concerns, or the presence of hazardous materials.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Ninyo & Moore should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document.

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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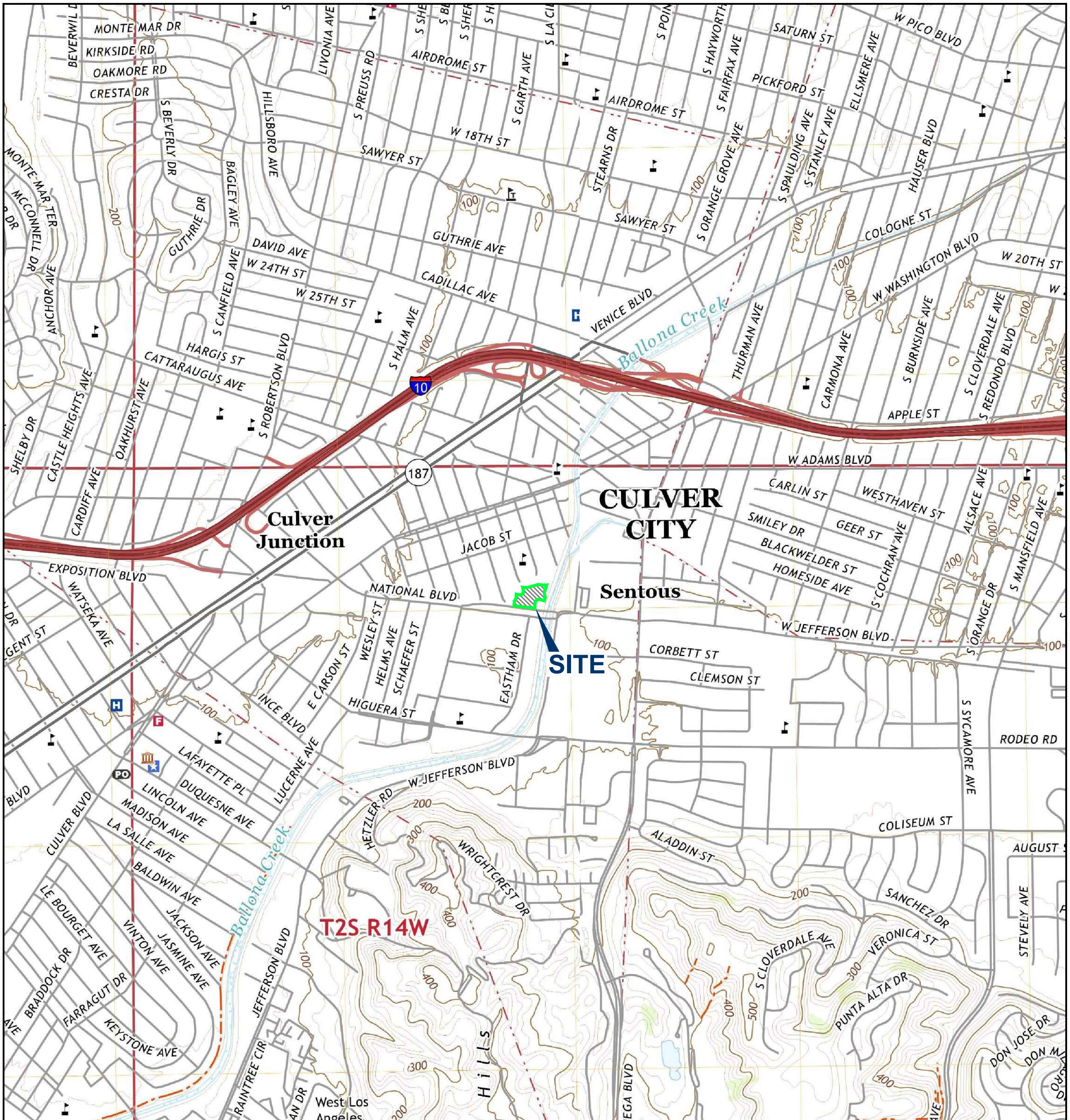
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2022.

N

FEET

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FIGURE 1

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

SITE LOCATION

SYD KRONENTHAL PARK STORMWATER CAPTURE FEASIBILITY STUDY
3459 MCMANUS AVENUE
CULVER CITY, CALIFORNIA

212034001 I 12/22

LEGEND

SITE BOUNDARY

B-1 TD=71.5 BORING; TD=TOTAL DEPTH IN FEET

P-2 TD=31.5 PERCOLATION TEST; TD=TOTAL DEPTH IN FEET

PROPOSED UNDERGROUND STORAGE STRUCTURE

APPROXIMATE LOCATION OF BUILDING IN 1948

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH, 2022.

N

FEET 0 200 400

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FIGURE 2

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SITE PLAN AND BORING AND PERCOLATION TEST LOCATIONS

SYD KRONENTHAL PARK STORMWATER CAPTURE FEASIBILITY STUDY
3459 MCMANUS AVENUE
CULVER CITY, CALIFORNIA

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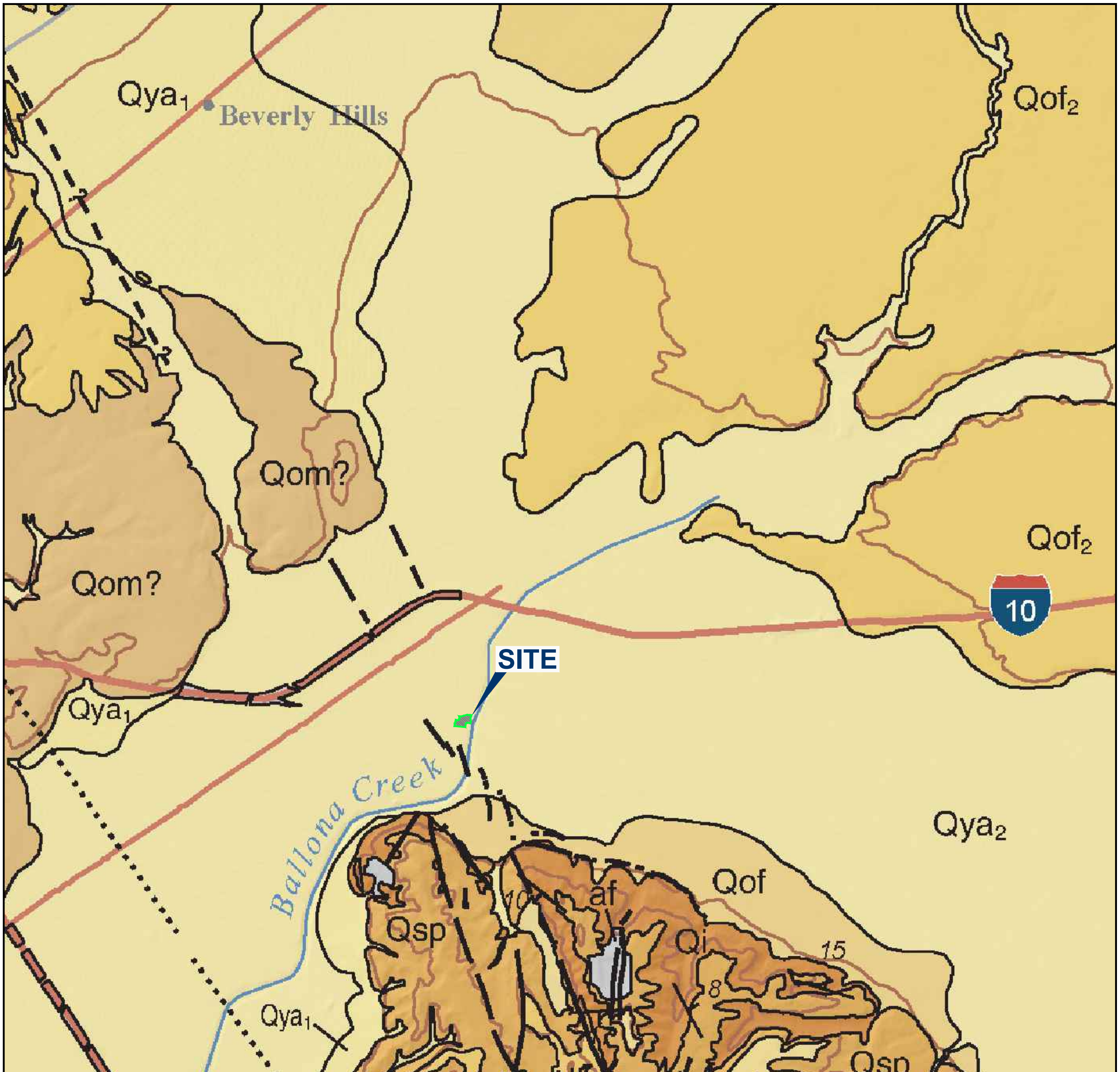

LEGEND

Qya	YOUNG ALLUVIUM	Qi	INGLEWOOD FORMATION
Qof	OLD ALLUVIAL FAN DEPOSITS		GEOLOGIC CONTACT
Qom	OLD SHALLOW MARINE DEPOSITS		FAULT; DASHED WHERE INFERRED, DOTTED WHERE CONCEALED
Qsp	SAN PEDRO FORMATION		

N

FEET

0 4,000 8,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: R.H. CAMPBELL, ET. AL., 2014.

212034001_RG.dwg 12/09/2022 GK, JDP

FIGURE 3

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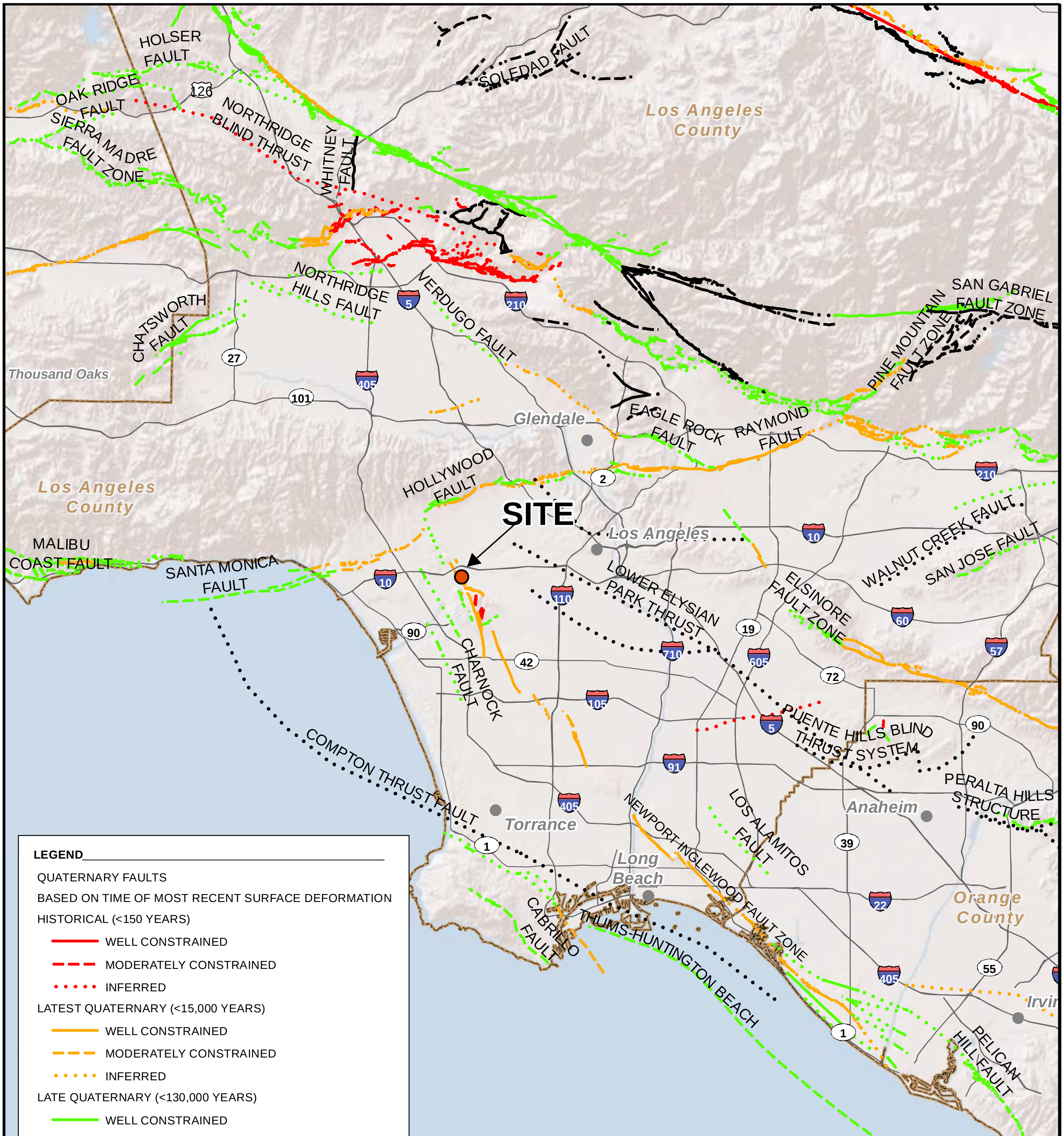
REGIONAL GEOLOGY

SYD KRONENTHAL PARK STORMWATER CAPTURE FEASIBILITY STUDY
3459 MCMANUS AVENUE
CULVER CITY, CALIFORNIA

212034001 I 12/22



LEGEND

QUATERNARY FAULTS

BASED ON TIME OF MOST RECENT SURFACE DEFORMATION

HISTORICAL (<150 YEARS)
- WELL CONSTRAINED
- MODERATELY CONSTRAINED
- INFERRED

LATEST QUATERNARY (<15,000 YEARS)
- WELL CONSTRAINED
- MODERATELY CONSTRAINED
- INFERRED

LATE QUATERNARY (<130,000 YEARS)
- WELL CONSTRAINED
- MODERATELY CONSTRAINED
- INFERRED

UNDIFFERENTIATED QUATERNARY(<1.6 MILLION YEARS)
- WELL CONSTRAINED
- MODERATELY CONSTRAINED
- INFERRED

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, ACCESSED OCTOBER 26, 2022, AT: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults; ESRI, 2021.

N

MILES

0 8 16

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FIGURE 4

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FAULT LOCATIONS

SYD KRONENTHAL PARK STORMWATER CAPTURE FEASIBILITY

3459 MCMANUS AVENUE

CULVER CITY, CALIFORNIA

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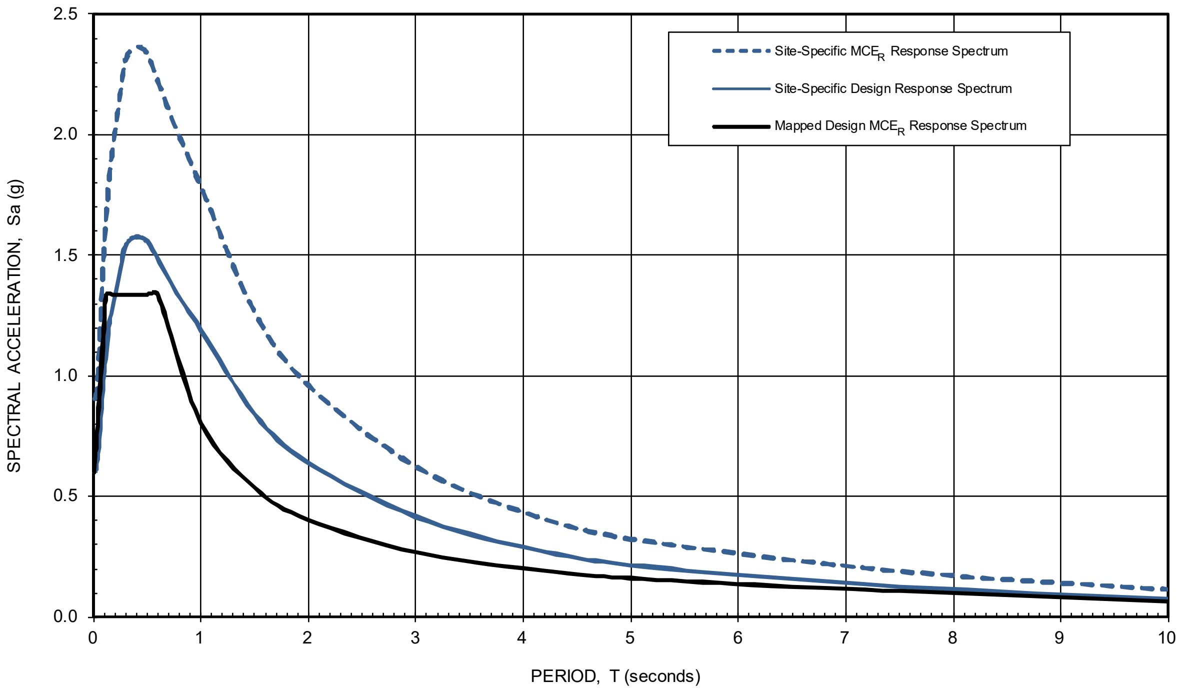
PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.906	0.604
0.020	0.913	0.609
0.030	0.935	0.624
0.050	1.053	0.702
0.075	1.299	0.866
0.100	1.532	1.021
0.150	1.832	1.221
0.200	2.007	1.338
0.250	2.168	1.445
0.300	2.313	1.542
0.400	2.365	1.576

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	2.337	1.558
0.750	2.045	1.364
1.000	1.787	1.191
1.500	1.265	0.843
2.000	0.960	0.640
3.000	0.624	0.416
4.000	0.435	0.290
5.000	0.324	0.216
7.500	0.189	0.126
10.000	0.114	0.076

S_{MS} = 2.128 g	S_{M1} = 1.921 g	S_{DS} = 1.419 g	S_{D1} = 1.280 g	PGA_M = 0.915 g



NOTES:

1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% probability of exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships and the risk coefficients per ASCE 7-16 Section 21.2.1.1.

2 The deterministic ground motion spectral response accelerations are the 84th percentile geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.5 event on the Compton fault zone located 12.0 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

3 The Site-Specific MCE_R Response Spectrum is the lesser of the spectral ordinates of the deterministic and probabilistic accelerations at each period per ASCE 7-16 Section 21.2.3. The Site-Specific Design Response Spectrum conforms with the lower bound limit per ASCE 7-16 Section 21.3.

4 The Mapped Design MCE_R Response Spectrum is computed from the mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11.4. It is presented for the sake of comparison.

FIGURE 5

ACCELERATION RESPONSE SPECTRA

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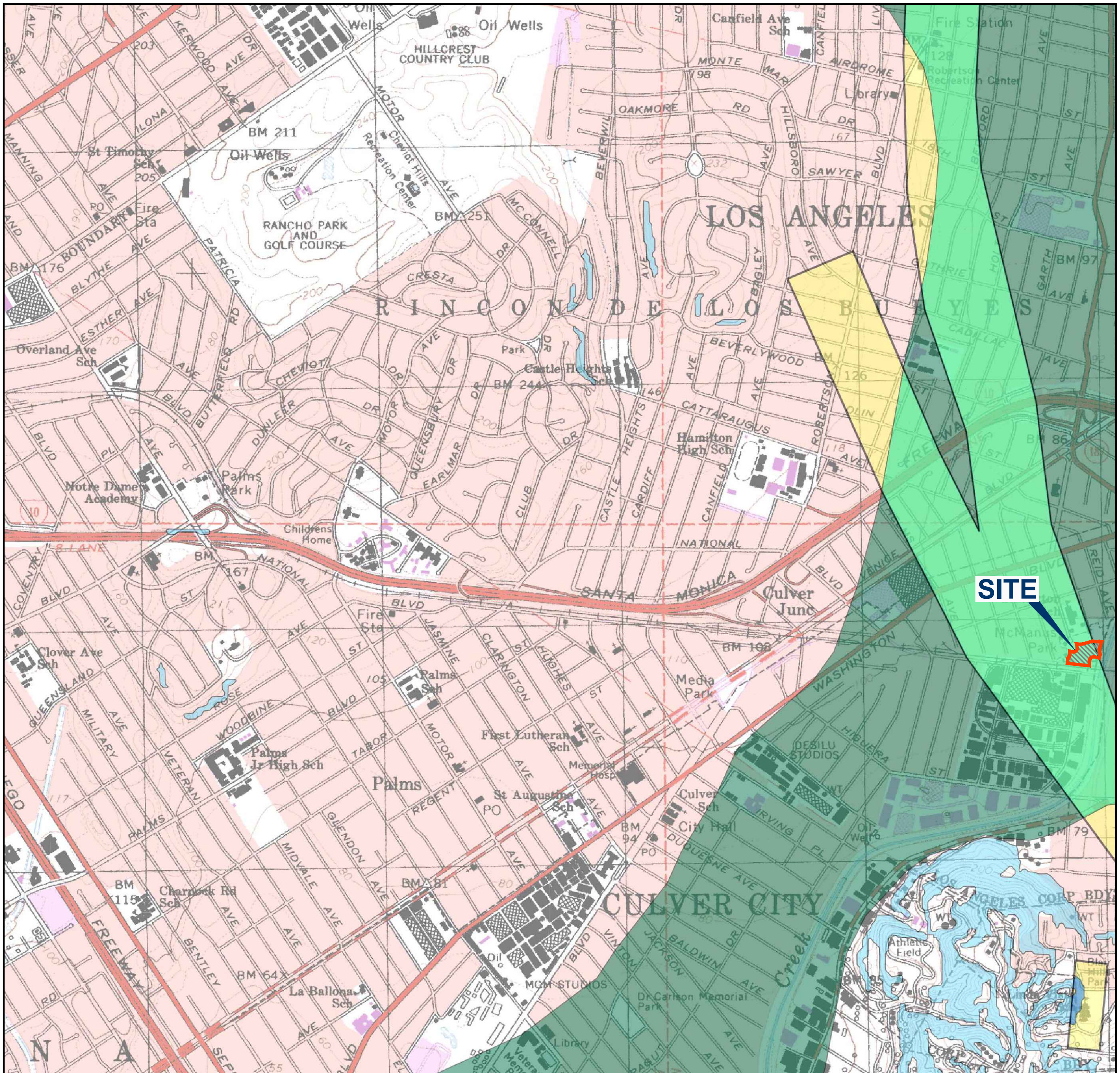
3459 MCMANUS AVENUE
CULVER CITY, CALIFORNIA



LEGEND

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

FEET
0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: CGS, 2018.

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FIGURE 6

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SEISMIC HAZARD ZONES

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CULVER CITY, CALIFORNIA

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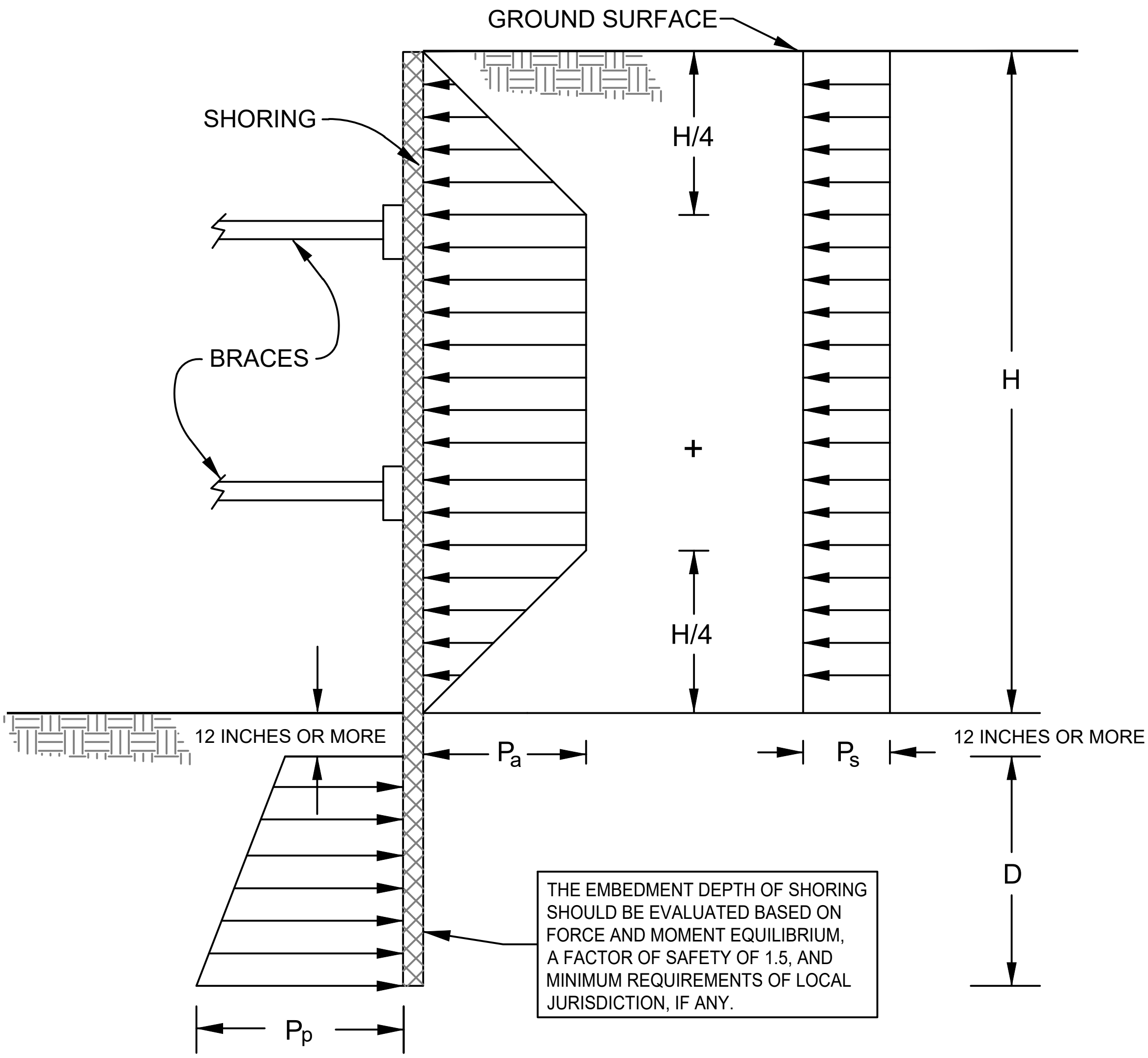

NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 48H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 120D + 1,500 psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

212034001_LEP-BE(SC).dwg 12/09/2022 JDP

FIGURE 7

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION

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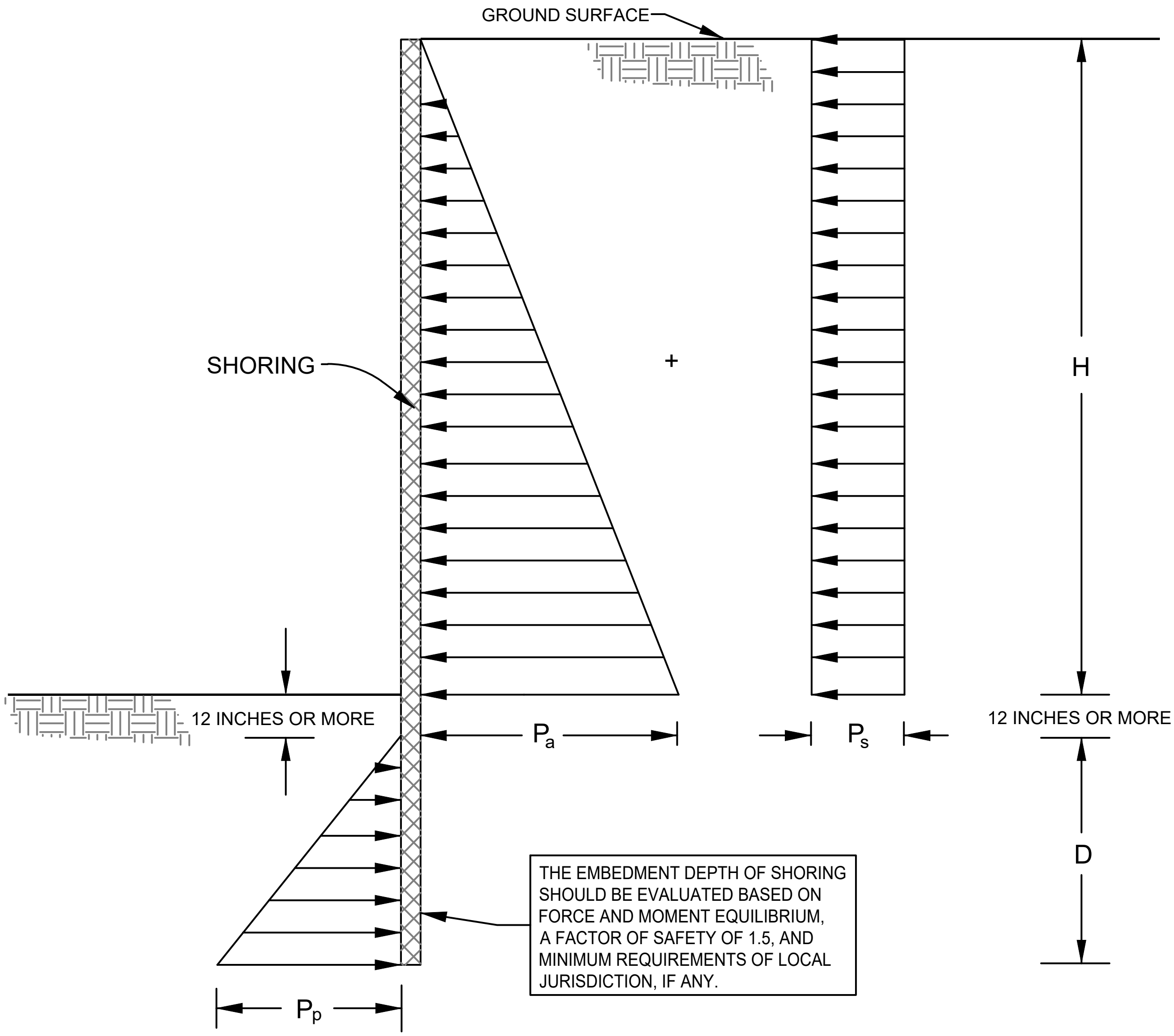

NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_a = 42H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 345D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

FIGURE 8

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING

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3459 MCMANUS AVENUE
CULVER CITY, CALIFORNIA

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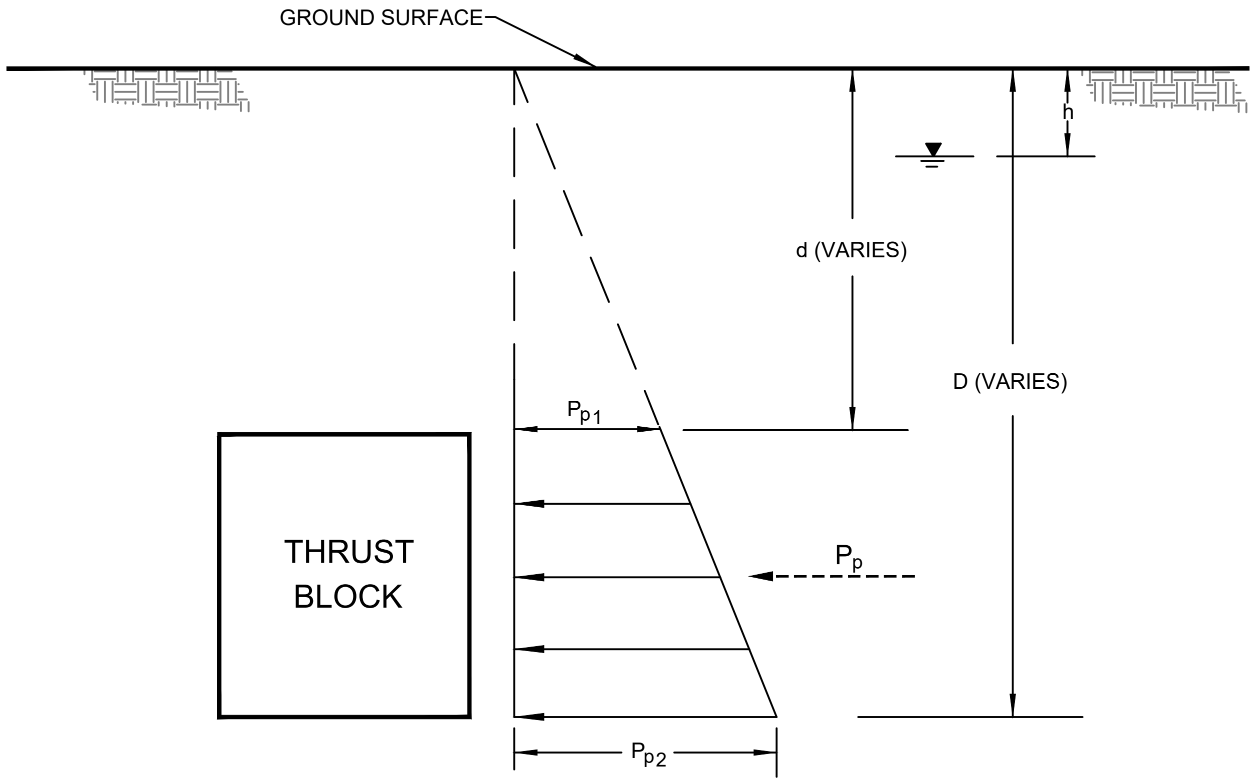

NOTES:

1. GROUNDWATER BELOW BLOCK
   $P_p = 172(D^2 - d^2)$ lb/ft
2. GROUNDWATER ABOVE BLOCK
   $P_p = 1.44(D - d)[124.8h + 57.6(D+d)]$ lb/ft
3. ASSUMES BACKFILL IS GRANULAR MATERIAL
4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
5. D, d AND h ARE IN FEET
6. ▽ GROUNDWATER TABLE

NOT TO SCALE

212034001_LEP-TB.dwg 12/09/2022 JDP

FIGURE 9

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THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM
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3459 MCMANUS AVENUE
CULVER CITY, CALIFORNIA
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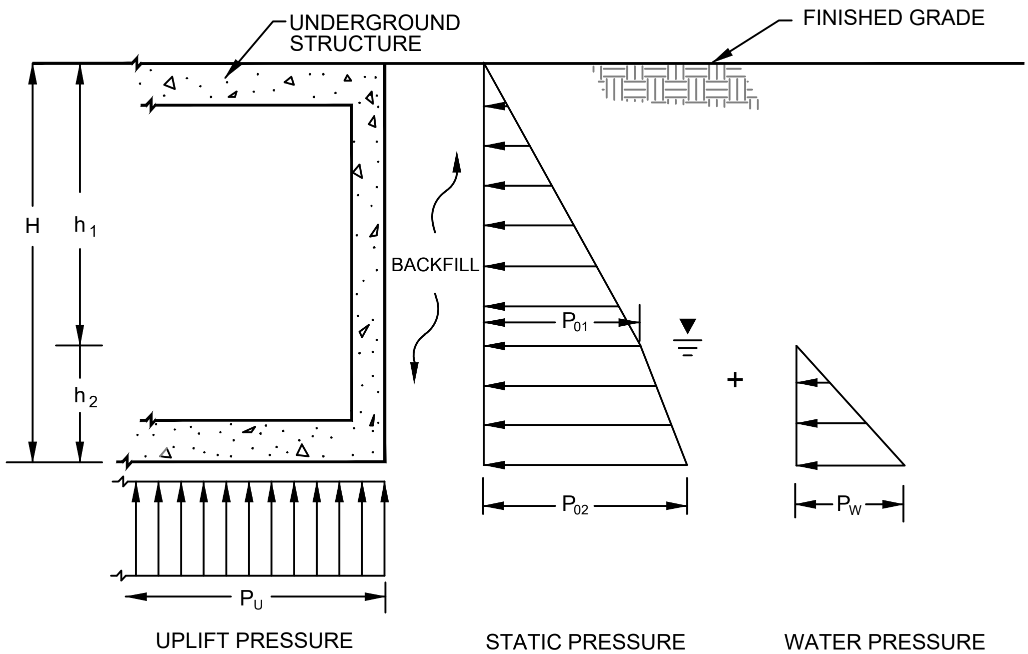

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{01} AND P_{02}
   $P_{01} = 62h_1$ psf
   $P_{02} = 62h_1 + 30h_2$ psf
2. HYDROSTATIC PRESSURE, P_w
   $P_w = 62.4h_2$ psf
3. UPLIFT PRESSURE, P_u
   $P_u = 62.4h_2$ psf
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H, h_1 AND h_2 ARE IN FEET
6. ▼ GROUNDWATER TABLE

NOT TO SCALE

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FIGURE 10

LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples
Disturbed soil samples were obtained in the field using the following method.

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory borings. The samples were bagged and transported to the laboratory for testing.

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer free-falling from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

Field Procedure for the Collection of Relatively Undisturbed Samples
Relatively undisturbed soil samples were obtained in the field using the following method.

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
								Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

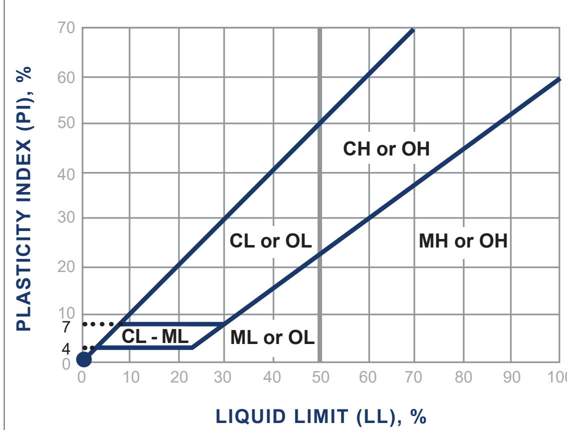
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVEL more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	SILT and CLAY liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

Apparent Density	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Loose	≤ 4	≤ 8	≤ 3	≤ 5
Loose	5 - 10	9 - 21	4 - 7	6 - 14
Medium Dense	11 - 30	22 - 63	8 - 20	15 - 42
Dense	31 - 50	64 - 105	21 - 33	43 - 70
Very Dense	> 50	> 105	> 33	> 70

Consistency - Fine-Grained Soil

Consis-tency	Spooling Cable or Cathead: SPT (blows/foot)	Spooling Cable or Cathead: Modified Split Barrel (blows/foot)	Automatic Trip Hammer: SPT (blows/foot)	Automatic Trip Hammer: Modified Split Barrel (blows/foot)
Very Soft	< 2	< 3	< 1	< 2
Soft	2 - 4	3 - 5	1 - 3	2 - 3
Firm	5 - 8	6 - 10	4 - 5	4 - 6
Stiff	9 - 15	11 - 20	6 - 10	7 - 13
Very Stiff	16 - 30	21 - 39	11 - 20	14 - 26
Hard	> 30	> 39	> 20	> 26

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USCS METHOD OF SOIL CLASSIFICATION

DATE DRILLED 10/10/22 BORING NO. B-1

GROUND ELEVATION 74' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY SCM/MLP

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							CL	FILL: Black and brown, moist, firm, lean CLAY with sand; few gravel; few organics.
							CL	ALLUVIUM: Black, moist, stiff, lean CLAY with sand; few organics.
			13	22.4	84.1			
							SC	Black, moist, medium dense, clayey SAND; few gravel.
10			9					
								Grayish brown, clayey sand with gravel; oxidation staining.
			33	9.0	118.4			
20			49					Very dense.
							SP	Yellowish red, moist, very dense, poorly graded SAND; few gravel.
			92	3.2	112.5			
30			50					Light brown.
							SP-SM	Light brown, moist, very dense, poorly graded SAND with silt; few gravel.
			100/8"					
40								

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3459 MCMANUS AVENUE, CULVER CITY, CALIFORNIA

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DATE DRILLED	10/10/22	BORING NO.	B-1
GROUND ELEVATION	74' ± (MSL)	SHEET 2 OF 2	
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	VAM	LOGGED BY VAM	REVIEWED BY SCM/MLP

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			60				SP-SM	ALLUVIUM: (Continued) Light brown, moist, very dense, poorly graded SAND with silt; few gravel.
			87/10"	7.9	102.5			Gray.
50			57					
			87/9"	7.4	102.7			
60			65					
			95/11"					@ 67': Groundwater encountered during drilling.
							SM	Gray, wet, very dense, silty SAND.
70			56					
80								Total Depth = 71.5 feet. Groundwater was encountered during drilling at approximately 67 feet. Groundwater was measured at approximately 67 feet after 20 minutes. Backfilled with cement-bentonite grout on 10/10/22. Notes: Groundwater may rise to a level higher than that measured in borehole due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED	10/10/22	BORING NO.	P-1
GROUND ELEVATION	74' ± (MSL)	SHEET	1 OF 1
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	VAM	LOGGED BY	VAM
REVIEWED BY	SCM/MLP		

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0					SC	FILL: Brown, moist, medium dense, clayey SAND; concrete debris.
					SM	Light brown, moist, loose, silty SAND.
	Driven	29			ML	Grayish brown, moist, medium dense, sandy SILT; few cobbles.
					CL	ALLUVIUM: Black, moist, very stiff, lean CLAY; caliche veins.
10	Driven	20	20.9	99.9		
	Driven	11			SP-SM	Light brown, moist, medium dense, poorly graded SAND with silt.
					CL	Black, moist, very stiff, lean CLAY.
					SW-SM	Light brown, moist, very dense, well-graded SAND with silt and gravel.
20	Driven	50/5"	4.7	116.6		
30						
40						

Total Depth = 20.9 feet.
Groundwater was not encountered during drilling.
Boring converted into 2-inch diameter percolation test on 10/10/22.
In-situ percolation test performed on 10/11/22.
Backfilled with gravel and cement-bentonite grout on 10/11/22.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED 10/10/22 BORING NO. P-2

GROUND ELEVATION 74' ± (MSL) SHEET 1 OF 2

METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling)

DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30"

SAMPLED BY VAM LOGGED BY VAM REVIEWED BY SCM/MLP

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SC	FILL: Brown and dark brown, moist, medium dense, clayey SAND; few gravel; asphalt debris.
							SC	ALLUVIUM: Grayish brown, moist, medium dense, clayey SAND; few gravel.
							CL	Black, moist, stiff, lean CLAY; few organics.
			9					
10			22	26.3	96.5			Dark brown; very stiff.
			7					Stiff.
							SC	Grayish brown, loose, clayey SAND.
							CL	Olive gray, moist, very stiff, lean CLAY with sand.
20			18	30.7	90.6			
			23					Hard; interbedded thin, poorly graded sand layers.
30			16					Very stiff.
								Total Depth = 31.5 feet. Groundwater was not encountered during drilling. Boring converted into 2-inch diameter percolation test on 10/10/22. In-situ percolation test performed on 10/11/22. Backfilled with gravel and cement-bentonite grout on 10/11/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 10/10/22 BORING NO. P-2 GROUND ELEVATION 74' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling) DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30" SAMPLED BY VAM LOGGED BY VAM REVIEWED BY SCM/MLP DESCRIPTION/INTERPRETATION
40								not sufficiently accurate for preparing construction bids and design documents.
50								
60								
70								
80								

APPENDIX B

Laboratory Testing

APPENDIX B

GEOTECHNICAL LABORATORY TESTING

Classification

Soils were visually and texturally classified in accordance with the Unified Soil Classification System (USCS) in general accordance with ASTM D 2488. Soil classifications are indicated on the logs of the exploratory borings in Appendix A.

In-Place Moisture and Density Tests

The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

Gradation Analysis

Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-1 and B-2. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

200 Wash

An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-3.

Atterberg Limits

Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classifications in accordance with the USCS. The test results and classifications are shown on Figure B-4.

Consolidation Test

A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are presented on Figure B-5.

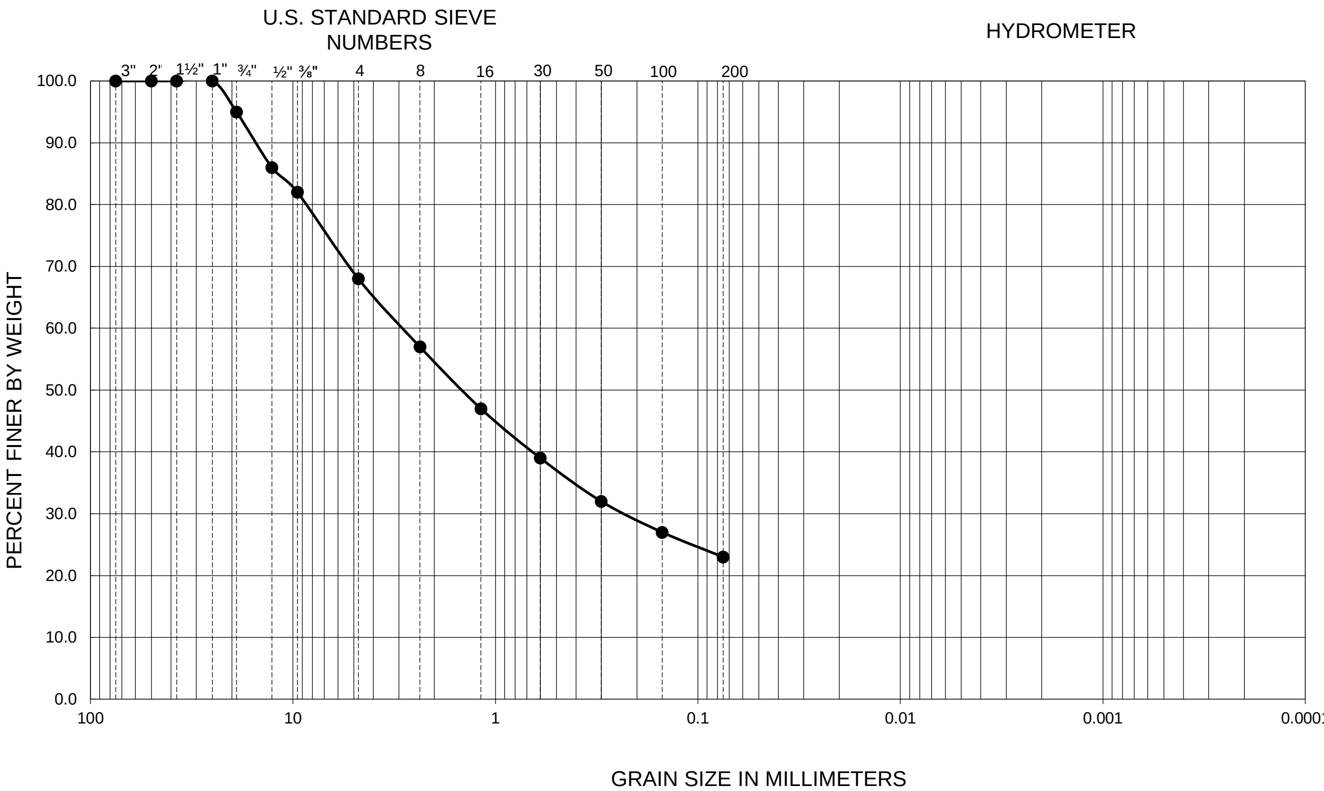
Direct Shear Tests

Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-6 and B-7.

Soil Corrosivity Tests

Soil pH and resistivity tests were performed on a representative sample in general accordance with CT 643. The soluble sulfate content and chloride content of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-8.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	13.0-17.0	--	--	--	--	--	--	--	--	23	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-1

GRADATION TEST RESULTS

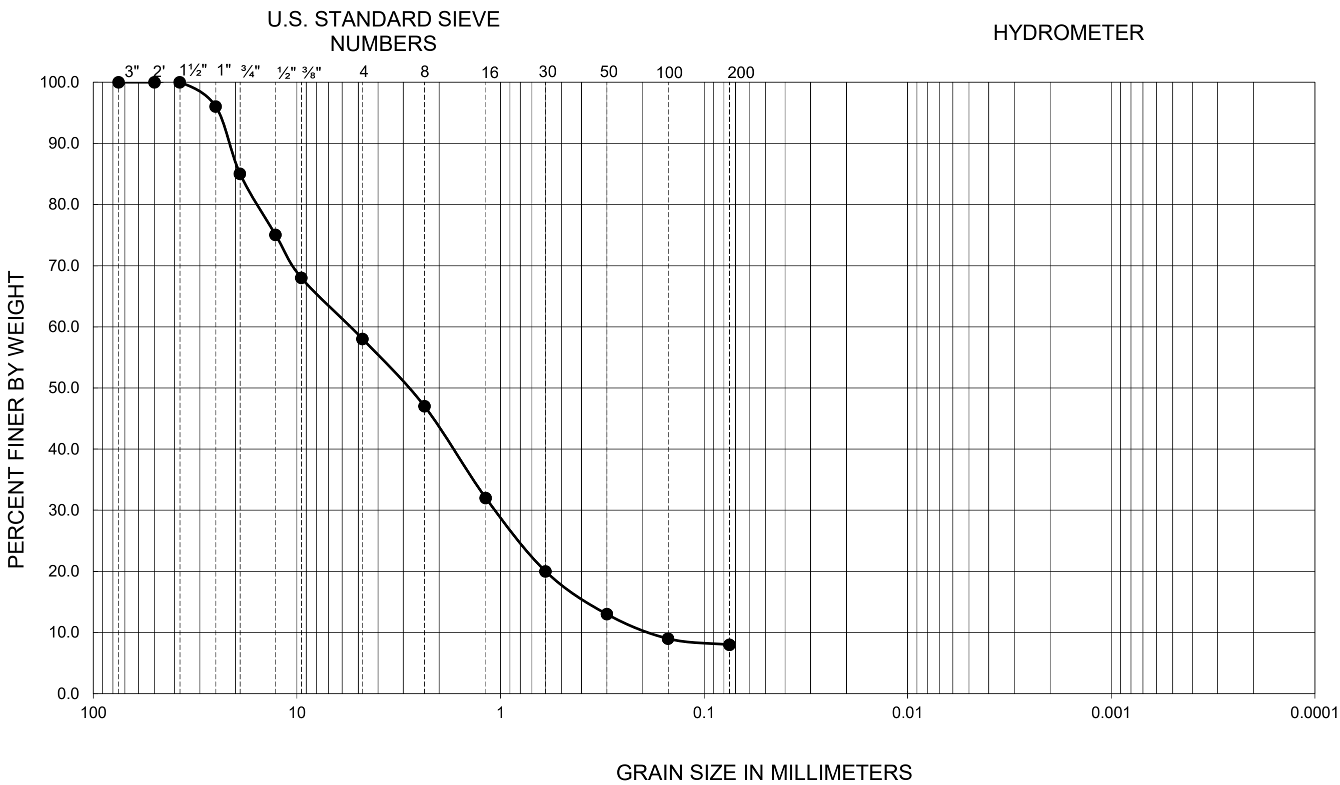
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SYD KRONENTHAL PARK STORMWATER CAPTURE FEASIBILITY STUDY
3459 MCMANUS AVENUE, CULVER CITY, CALIFORNIA

212034001 | 12/22

212034001 Fig B-1_SIEVE w No 8 @ B-1 13.0-17.0

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	P-1	20.0-20.9	--	--	--	0.19	1.08	5.40	28.8	1.2	8	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-2

GRADATION TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	10.0-11.5	CLAYEY SAND	90	33	SC
B-1	25.0-26.5	POORLY GRADED SAND	91	3	SP
B-1	45.0-46.3	POORLY GRADED SAND WITH SILT	100	6	SP-SM
P-2	20.0-21.5	LEAN CLAY WITH SAND	100	81	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-3

NO. 200 SIEVE ANALYSIS TEST RESULTS

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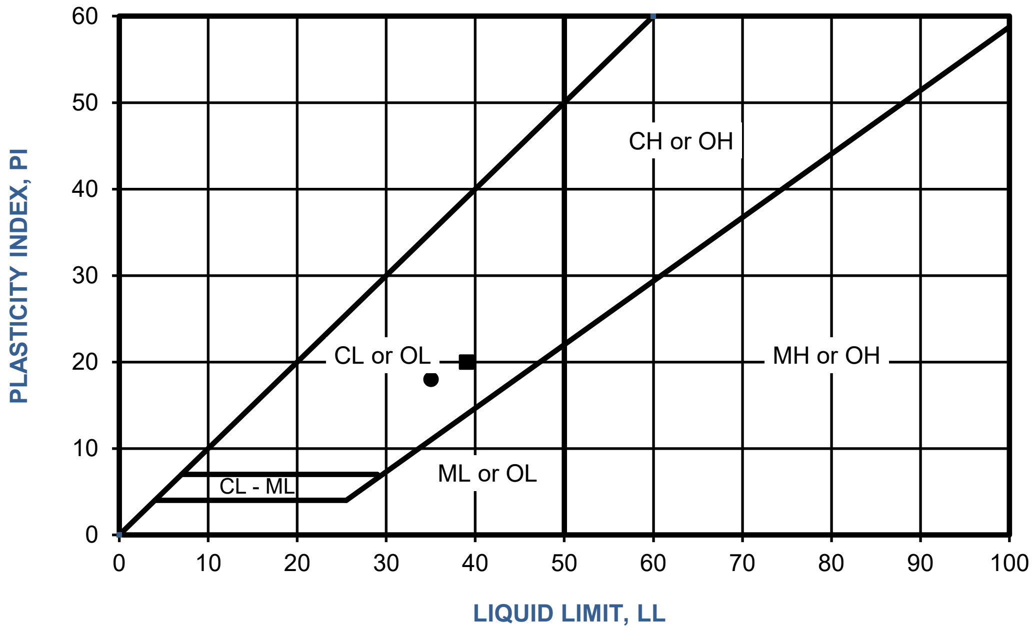
SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-1	0.0-5.0	35	17	18	CL	CL
■	P-2	20.0-21.5	39	19	20	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-4

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ATTERBERG LIMITS TEST RESULTS

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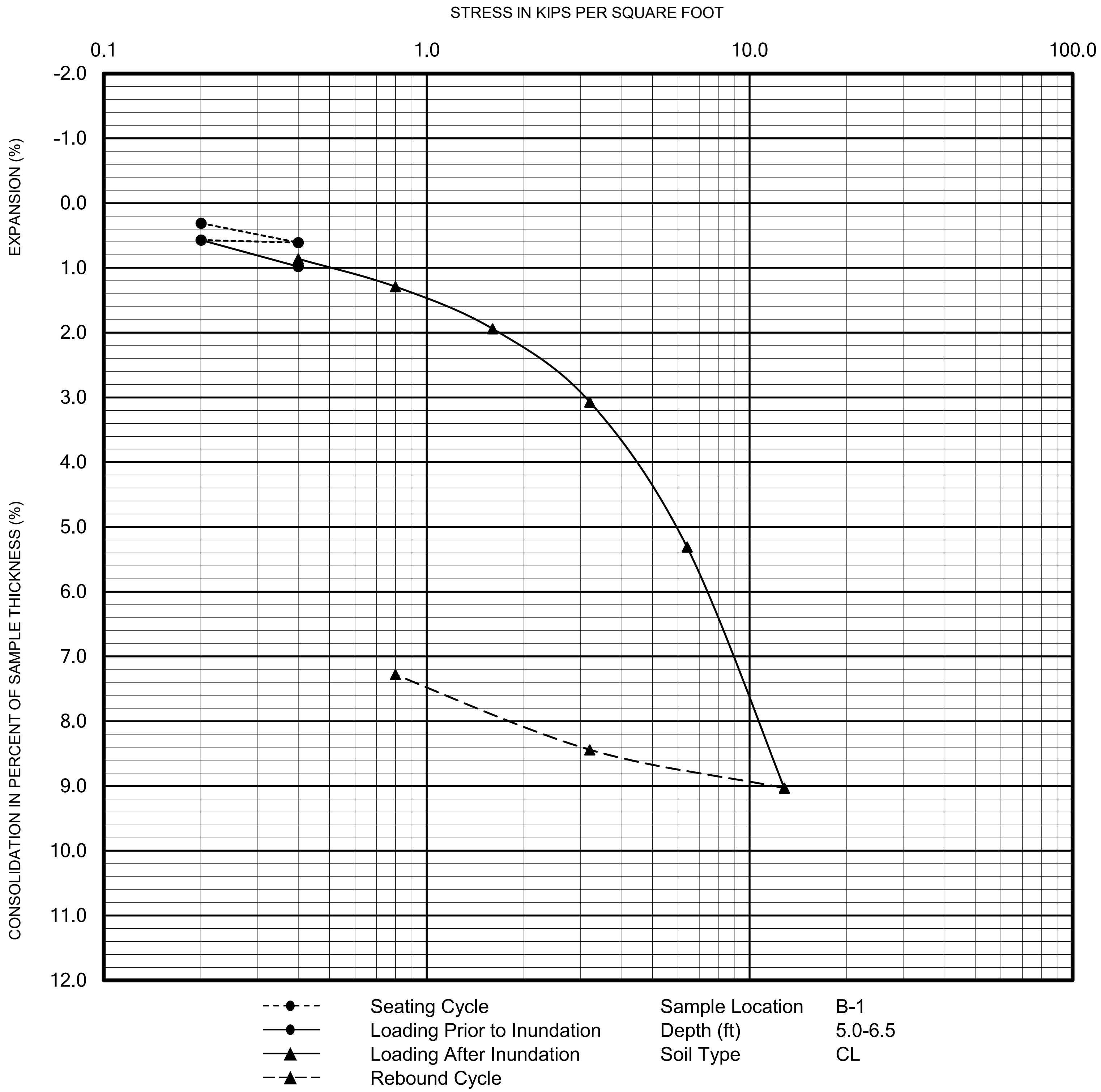

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-5

CONSOLIDATION TEST RESULTS

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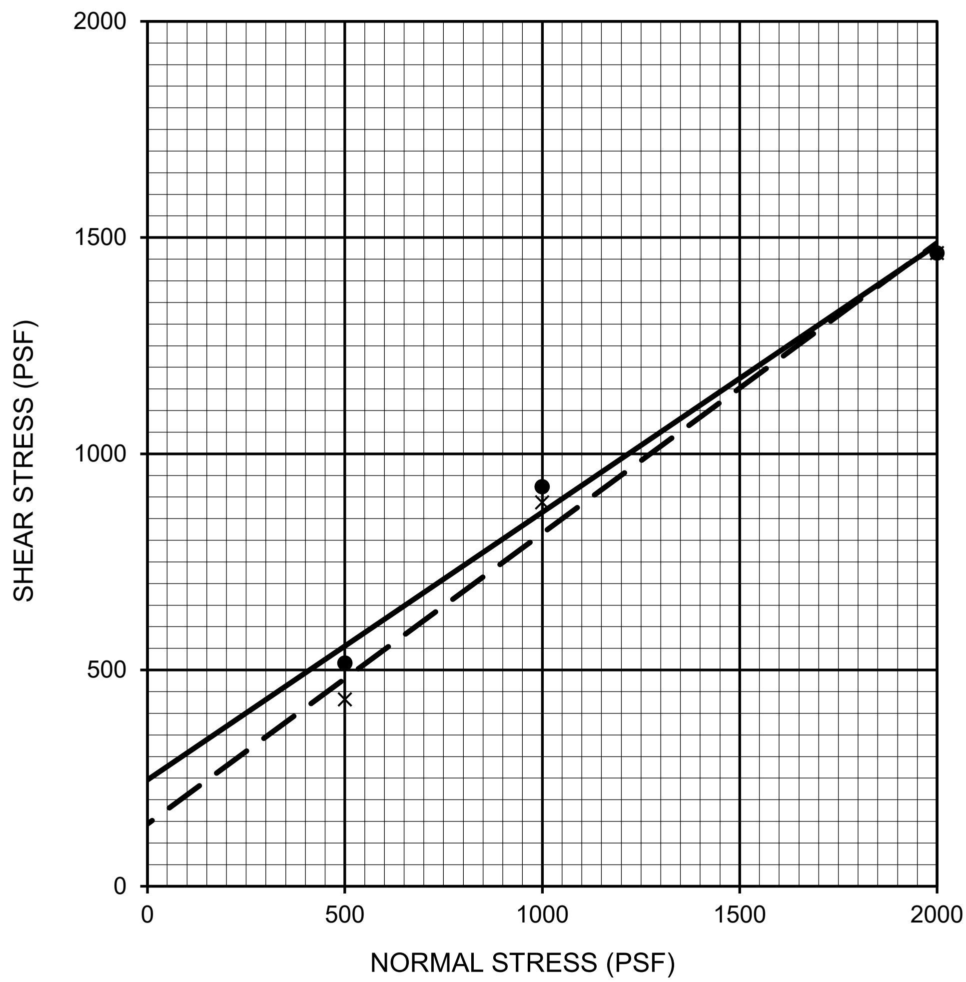

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Lean CLAY	—●—	B-1	5.0-6.5	Peak	245	32	CL
Lean CLAY	– – X – –	B-1	5.0-6.5	Ultimate	145	34	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-6

DIRECT SHEAR TEST RESULTS

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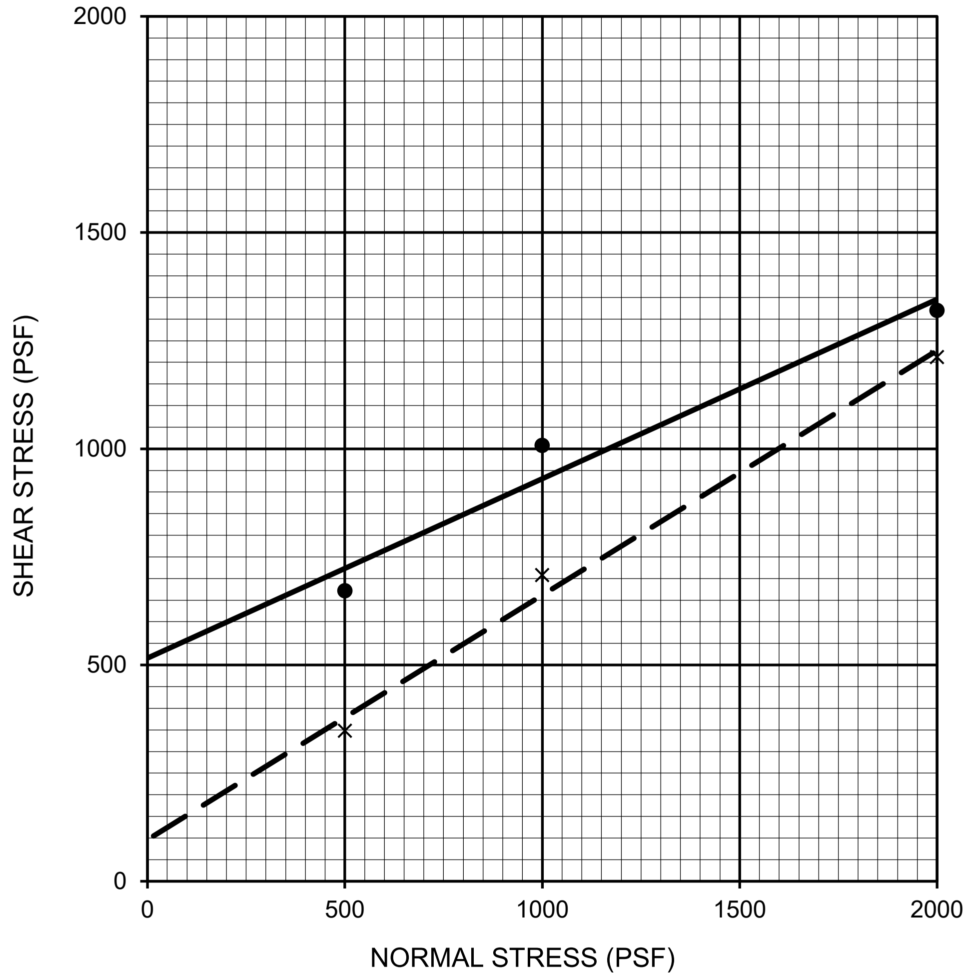

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Lean CLAY	—●—	P-2	10.0-11.5	Peak	515	23	CL
Lean CLAY	– – X – –	P-2	10.0-11.5	Ultimate	95	29	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-7

DIRECT SHEAR TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	0.0-5.0	6.9	745	60	0.006	190

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-8

CORROSIVITY TEST RESULTS

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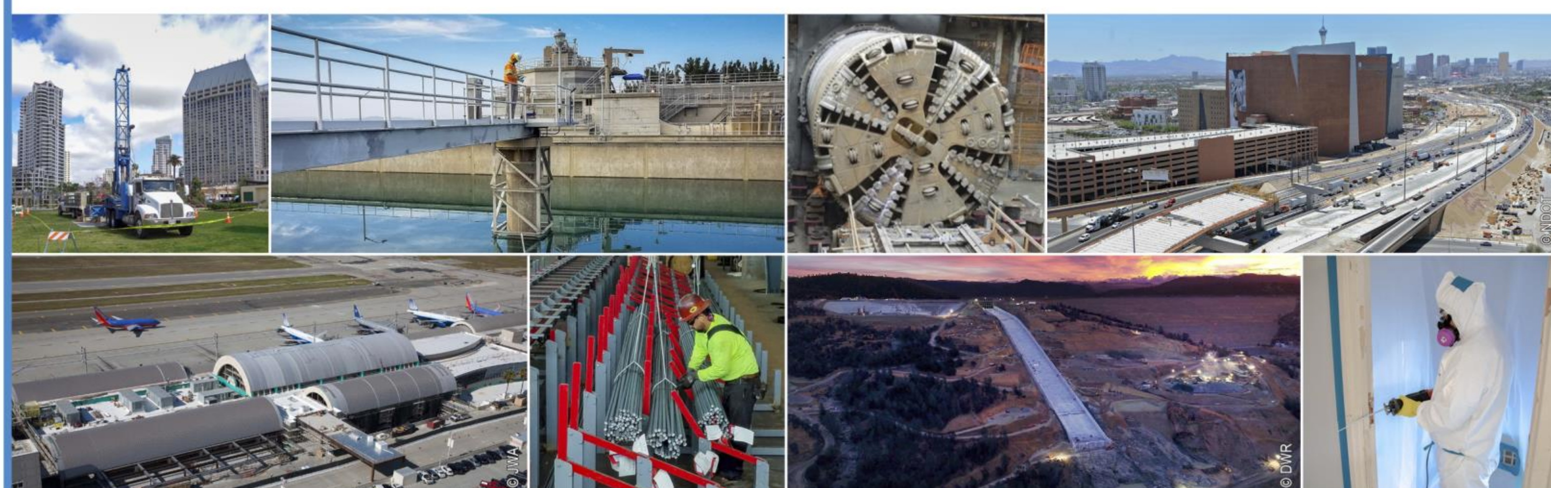

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