Geotechnical Evaluation

Sunset Reservoir Replacement and Groundwater Treatment Facility Project

Pasadena Water and Power

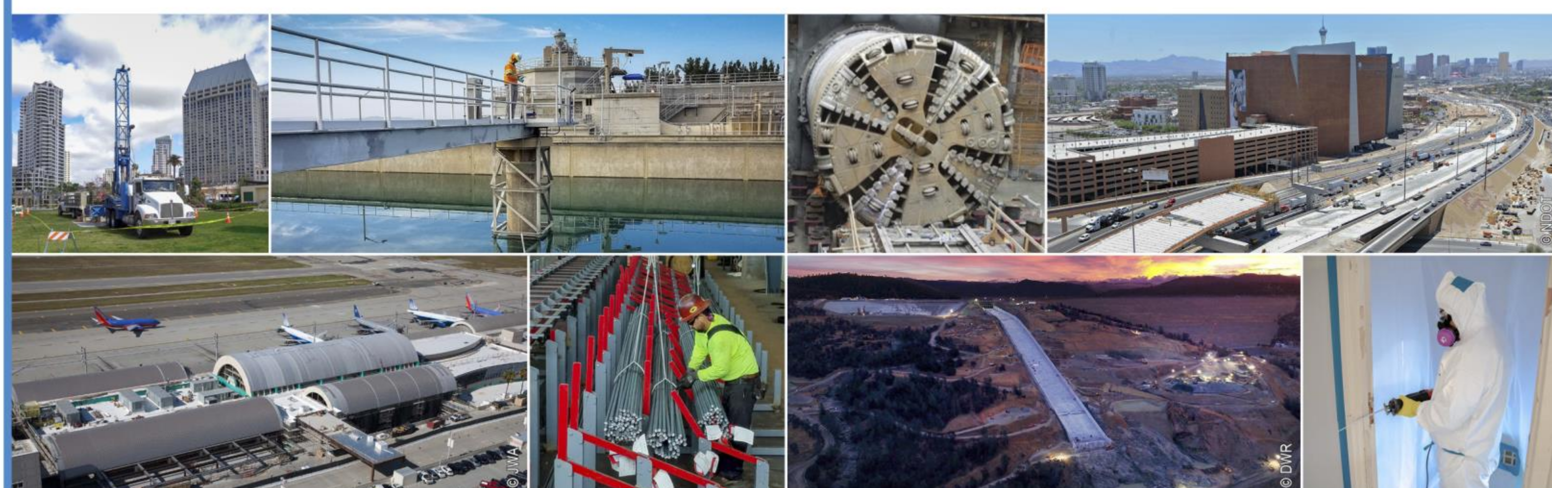
201 West Mountain Street

Pasadena, California

Kennedy Jenks Consultants

300 N. Lake Avenue, Suite 1020 | Pasadena, California 91101

April 2, 2021 | Project No. 211621001



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

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

Geotechnical Evaluation

Sunset Reservoir Replacement and Groundwater Treatment Facility Project Pasadena Water and Power 201 West Mountain Street

Pasadena, California

Mr. Paul Chau
Kennedy Jenks Consultants
300 N. Lake Avenue, Suite 1020 | Pasadena, California 91101

April 2, 2021 | Project No. 211621001

Jennifer R. Schmidt, PG, CEG
Senior Geologist

Soumitra Guha, PhD, PE, GE
Principal Engineer

KMB/JRS/SG/sc

cc: Mr. Steve Montagna, Kennedy Jenks

CONTENTS

TABLES

PLATES

FIGURES

APPENDICES

1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed Sunset Reservoir Replacement and Groundwater Treatment Facility Project located at 201 West Mountain Street in Pasadena, California (Figure 1). The purpose of our study was to evaluate the soil and geologic conditions at the site and provide geotechnical recommendations for the design and construction of the proposed improvements. This report presents our geotechnical findings, conclusions, and recommendations regarding the project.

Several relatively recent geotechnical and seismic studies have been performed at the Sunset Reservoir site by other consultants, including seismic vulnerability assessments by William Lettis & Associates (2005) and G&E Engineering Systems, Inc. (G&E, 2006), a geotechnical investigation for the Sunset Reservoir perchlorate treatment facility by Diaz Yourman & Associates (2009), a seismic evaluation for the southern-most existing reservoir at the site by Carollo Engineers (2015), a draft preliminary design report for the Sunset Reservoir Replacement Project by Carollo Engineers (2018), and a preliminary design report for a future buildout of the groundwater treatment facility by Kennedy Jenks in 2019 (Kennedy Jenks, 2021). More detailed discussions regarding the conclusions and recommendations of these evaluations are discussed in Kennedy Jenks' Basis of Design Report, dated February 9, 2021.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.
- Preparation of a Site-Specific Health and Safety Plan (SSHSP).
- Review of readily available background material, including published geologic and seismic hazards maps, previous reports, published literature, in-house information, stereoscopic aerial photographs, and reports and plans provided by the client.
- Acquisition of a well permit and an encroachment permit from the City of Pasadena Water and Power Department and the City of Pasadena Department of Public Works, respectively.
- Geotechnical site reconnaissance to observe the general site conditions, mark the proposed boring locations, and coordinate with Underground Service Alert for utility clearance.
- Geophysical utility clearance at our boring locations.
- Subsurface exploration consisting of the drilling, logging, and sampling of seven small-diameter exploratory borings to depths ranging from approximately 31.5 to 53.0 feet below the ground surface. The borings were logged by a representative of our firm and bulk and relatively undisturbed soil samples were collected at selected representative intervals for laboratory testing. The borings were backfilled with bentonite-cement grout and borings in

paved areas were patched with rapid-set concrete. Soil cuttings were drummed and stored on-site for disposal by the City of Pasadena Water and Power Department.

- Laboratory testing on selected soil samples, including evaluation of in-situ moisture content and dry density, gradation analysis, percent of particles finer than the No. 200 sieve, Atterberg limits, consolidation, direct shear strength, Proctor densities, R-value, and soil corrosivity.
- Geotechnical engineering analysis of data from our background review, subsurface exploration, and laboratory testing.
- Preparation of this report presenting our findings, conclusions, and recommendations pertaining to the geotechnical aspects of the design and construction of the proposed improvements.

3 SITE DESCRIPTION

The project site is located at the City of Pasadena's Department of Water and Power Sunset Reservoir complex located at 201 West Mountain Street in Pasadena, California (Figure 1). The site is bounded by residential properties to the north, the City of Pasadena's Public Works Department property to the west, West Mountain Street to the south, and Sunset Avenue and a residential neighborhood to the east. The existing reservoir complex currently includes two relatively large, partially buried water reservoirs with a combined storage capacity of approximately 15.5 million gallons (MG) (Kennedy Jenks, 2021a). We understand that these existing reservoirs were originally constructed in the late 1800s and early 1900s as open-earth embankment structures (Kennedy Jenks, 2021a). The existing oval-shaped reservoir on the south side of the site is Sunset Reservoir 1 (SR1), and is bifurcated into two sub-basins (Figure 2) (Kennedy Jenks, 2021a). Based on conversations with a Department of Water and Power representative, we understand that the bottom of the southern sub-basin is at an elevation of approximately 930.6 feet above mean sea level (MSL) and the bottom of the northern sub-basin is at an elevation of approximately 934.1 feet above MSL. Sunset Reservoir 2 (SR2) is the irregular-shaped polygonal structure to the north of SR1 with a bottom elevation of approximately 930.6 feet above MSL (Figure 2). Wood-framed roofs and corrugated steel decks were added to the open-earth embankment structures of SR1 and SR2 in the 1890s and 1900s, respectively. In the 1920s and 1930s repairs and additions to both reservoirs generally consisted of replacement of the original roofs with galvanized, corrugated metal sheets, replacement of the original linings with reinforced wire-mesh gunite linings, and the addition of approximately 4-foot-high concrete walls around the reservoir perimeters to increase storage capacities (G&E, 2006c).

Additional site equipment and facilities located in the reservoir complex include two well facilities (Bangham Well and Sunset Well No. 20), a disinfection facility, and the Glorieta Pump Station. The Bangham Well site is located in the northeast corner of the complex and is enclosed by chain-link fencing (Figure 2). The enclosure houses a transformer, switchgear control room, and a rectangular reinforced concrete well building. The Glorieta Booster Pump Station is located on the east side of the site, adjacent to Sunset Avenue, and between the two existing reservoirs (Figure 2). The Glorieta pump station is a reinforced concrete structure that was built to transport water from the Sunset Reservoir to the Calveras Reservoir (G&E, 2006c). It houses a control center, two motors, a flow meter, and a vertical turbine pump. We understand that the Bangham Well and the Glorieta Pump Station will be protected in place during construction of the proposed project improvements and will continued to be used as part of the new reservoir complex.

Several existing underground utilities are present within the Sunset Reservoir site as well as on Sunset Avenue and Mountain Street. Based on review of the referenced plans and previous reports for the site, Sunset Avenue, Mountain Street, and the area of the intersection of the two streets to the south of the site are relatively heavily congested with underground utilities, including gas, sewer, electrical, storm drains, and 12-, 20-, 24-, and 36-inch-diameter water pipelines. Several abandoned water pipelines ranging in size from 8- to 26-inches in diameter are also located in Sunset Avenue. We anticipate that several of the utilities on the site and in the adjacent streets are buried to depths of up to around 20 feet below the ground surface.

Based on review of historic topographic maps (Historical Aerials, 2021), the site was once a gently sloping alluvial fan that was graded with cuts and fills to construct the existing reservoir complex. We anticipate fill thicknesses gradually increase toward the western side of the site, where fills were placed to create a larger, flatter parcel when the reservoirs on the east side of the site were excavated. Ground surface elevations at the site range from approximately 935 to 948 feet above MSL and generally slope from north to the south. Ground surface elevations at the site are generally a few feet to around approximately 10 feet higher than the streets on the east and south sides of the site, and up to around 20 feet higher than the Public Works Department property on the west side of the site. Retaining walls ranging in height from approximately 2 to 10 feet high border much of the site perimeter. The north and west sides of the site are parking areas paved in asphalt concrete (AC). The site is enclosed by a chain-link fence with access gates located on Mountain Street and Sunset Avenue.

4 PROPOSED CONSTRUCTION

The Sunset Reservoir Replacement and Groundwater Treatment Facility Project is a City of Pasadena Department of Water and Power project that involves the design and construction of two new partially-buried, pre-stressed concrete water storage tanks, a pre-stressed concrete clearwell tank, a new drain vault, and a new groundwater treatment facility (Plate 1). The proposed new Reservoir 1 will be located in the southern portion of the project site and will have a water storage capacity of approximately 4.9 MG. The new Reservoir 1 will be approximately 25 feet high and approximately 210 feet in diameter. The proposed new Reservoir 2 will be located to the north of the new Reservoir 1 and will have a water storage capacity of approximately 5.5 MG. The new Reservoir 2 will be approximately 25 feet high and approximately 224 feet in diameter. The proposed new clearwell tank will be located to the west of the two new storage tanks and will have a storage capacity of approximately 0.6 MG. The new clearwell tank will be approximately 29 feet high and approximately 80 feet in diameter (Kennedy Jenks, 2021a). A new subsurface drain vault will be located to the west of the new Reservoir 2 and is anticipated to be up to approximately 15 to 20 feet deep. The proposed new groundwater treatment facility will be located to the north of the new reservoirs and will consist of two at-grade groundwater treatment areas and a disinfection facility. The northern-most groundwater treatment area will have a footprint of approximately 40 by 40 feet and the southern-most groundwater treatment area will have a footprint of approximately 50 by 85 feet. The groundwater treatment areas will include slab-on-grade concrete pads for well pumps, ion exchange pre-filters, ion exchange vessels, granular activated carbon vessels, and backwash tanks. The new disinfection facility will have a footprint of approximately 32 by 40 feet. Room for future expansions will be included in the design and construction of the groundwater treatment and disinfection facility areas. Two emergency generators with footprints of approximately 10 by 20 feet will be located near the groundwater treatment facility and the Glorieta Pump Station (Plate 1).

Other improvements will include the construction of a new 36-inch-diameter transmission main pipeline in Sunset Avenue that will connect the two new reservoirs to an existing 36-inch-diameter water main in the intersection of Sunset Avenue and Mountain Street. New underground pipelines and electrical duct banks ranging from 6- to 24-inches in diameter will be constructed across the site to connect the new reservoirs, groundwater treatment facility, drain vault, clearwell, Bangham Well, Glorieta Pump Station, and other related equipment. The proposed new underground utilities will be constructed using open cut-and-cover construction methods.

Grading for the project will generally include relatively shallow remedial grading and some deeper vertical cuts where the proposed new tanks are located in close proximity to other existing improvements and/or property lines and cannot be sloped back. Shoring is anticipated along Sunset Avenue and Mountain Street and around the Glorieta Pump Station to facilitate construction and grading at the site. Shoring may also be used to construct the proposed new drain vault. We anticipate that new fills up to approximately 20 feet deep will be placed at the proposed new groundwater treatment facility to raise the finished grade after SR2 is demolished. New fills at the site will partially bury the proposed new reservoirs and finished grading will slope gently to the south and southwest. The site will be paved in AC pavements with parking for 159 vehicles. The site perimeter will be fenced and the gates on Sunset Avenue and Mountain Street will remain at the existing locations for site access. The existing site conditions and anticipated grading are shown on Cross Sections A-A’ through D-D’ on Plate 2.

5 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration was performed on February 23 through 25, 2021, and consisted of drilling, logging, and sampling of seven small-diameter borings using a truck-mounted drill rig with 8-inch-diameter hollow-stem augers to depths ranging from approximately 31.5 to 53.0 feet below the ground surface. The approximate locations of the exploratory borings are shown on Plate 1 and Figure 2. The borings were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths for laboratory testing. The logs of the exploratory borings are presented in Appendix A.

Laboratory testing was performed on representative samples to evaluate in-situ moisture content and dry density, gradation analysis, percent of particles finer than the No. 200 sieve, Atterberg limits, consolidation, direct shear strength, Proctor density, R-value, and corrosivity. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory test results are presented in Appendix B.

6 GEOLOGY AND SUBSURFACE CONDITIONS

6.1 Regional Geology

The site is located within the northwestern block of the Los Angeles Basin within the Transverse Ranges geomorphic province of southern California (Norris and Webb, 1990). Geologically, the Los Angeles Basin and vicinity is a region divided into four blocks that include uplifted portions and synclinal depressions. The northwestern block is bordered by the Raymond and Santa Monica faults to the south, the San Gabriel Mountains to the northeast, and the mountain ranges of the Ventura Basin to the northwest and west.

The subject site is situated on a gently sloping alluvial fan in the San Gabriel Valley, northeast of Los Angeles. Geologic mapping by Dibblee (1989) indicates that the site is underlain by late Pleistocene-age alluvial fan deposits generally consisting of sand and gravel (Figure 3). Our interpretations of the subsurface conditions are shown on our Cross Sections presented on Plate 2.

6.2 Site Geology

Materials encountered during our subsurface exploration consisted of AC pavement sections underlain by fill and alluvium. Pavement sections were encountered at borings B-1 through B-3 and B-5 through B-7. Boring B-4 was located in an unpaved landscaping planter. The AC ranged in thickness from approximately 4 to 7 inches. At borings B-5 through B-7, the AC was underlain by aggregate base. The aggregate base was approximately 8 to 12 inches thick and generally consisted of brown, moist, medium dense to dense, poorly graded gravel.

Fill was encountered below the pavement sections in borings B-1 through B-3 and B-5 through B-7, and at the ground surface in boring B-4, to depths of approximately 5 to 15 feet below the ground surface. The fill material generally consisted of reddish yellow and light brown to dark brown, moist, loose to dense, silty sand, silty sand with gravel, poorly graded sand with gravel, and poorly graded gravel with silt and scattered cobbles.

The fill soils were underlain by alluvium to the total depths explored. The alluvium generally consisted of light brown, brown, and reddish yellow, moist, medium dense to very dense, silty sand, silty sand with gravel, poorly graded sand, poorly graded sand with silt and gravel, well graded sand with silt, and poorly graded gravel with silt and sand. The alluvium materials also contained scattered cobbles. More detailed descriptions of the subsurface materials encountered during our subsurface exploration are presented on the boring logs in Appendix A.

6.3 Groundwater

Groundwater was not encountered in our borings at the time of drilling. Additionally, temporary piezometers consisting of screened polyvinyl chloride pipe was installed in two of the deeper borings (B-4 and B-6) so they could be left open overnight for further groundwater observation. Groundwater was not observed in either boring the following day. Regional maps indicate that the historic high groundwater at the site is more than 140 feet below the ground surface (California Department of Conservation, Division of Mines and Geology [CDMG], 1998). Groundwater monitoring well data from the State of California Water Resources Control Board's GeoTracker website (2021) indicates that the depth to groundwater at a monitoring well located approximately 1.8 miles south of the site was measured at approximately 150 feet below the ground surface. Fluctuations in groundwater levels will occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors that may not have been evident at the time of our field evaluation.

7 FAULTING AND SEISMICITY

The project site is located in a seismically active area, as is the majority of southern California. The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey (CGS), active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active faults in the region and their geographic relationship to the project sites are shown on Figure 4.

Based on our review of seismic hazard maps, geologic literature, and geologic maps, the site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone), and no active faults are known to cross the subject site. The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, liquefaction, and dynamic compaction of dry soils. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

7.1 Surface Fault Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. Therefore, the probability of damage from surface fault rupture is considered to be low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

7.2 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. Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed 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 in accordance with Sections 21.2 and 21.3 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. We calculated that the S_1 for the site is equal to 0.75g using the 2021 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 general MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 (ATC, 2021). The average shear wave velocity (V_S) for the upper 30 meters of soil (V_{S30}) is assumed to be 365 meters per second (m/s) (CGS, 2016) and the depths to V_S = 1,000 m/s and V_S = 2,500 m/s are assumed to be 100 meters and 650 meters, respectively (Southern California Earthquake Center [SCEC], 2005). These values were evaluated using the Open Seismic Hazard Analysis software developed by USGS (USGS, 2020).

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 Open Seismic Hazard Analysis software developed by USGS (USGS, 2020) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center (PEER) 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 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, 2021). A magnitude 7.0 event on the Elysian Park fault with a rupture distance of 10.4 kilometers from the site was evaluated to be the controlling earthquake. 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 general 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 2019 CBC, are provided in Section 9.5 for the evaluation of seismic loads on buildings and other structures. The site-specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration, PGA_M, was calculated as 0.896g.

7.3 Liquefaction Potential

Liquefaction is the phenomenon in which loosely deposited granular soils 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. Liquefaction is also known to occur in relatively fine-grained soils (i.e., sandy silt and clayey silt) with a plasticity index (PI) of less than 12 and an in-place moisture content more than 85 percent of the liquid limit (LL) and sensitive silts and clays with a PI more than 18. 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.

Review of the State of California Seismic Hazards Zones map (CDMG, 1999) indicates that the site is not located in an area mapped as a potential liquefaction hazard zone (Figure 6). Additionally, the historic high groundwater at the site is more than approximately 140 feet below the ground surface. Accordingly, it is our opinion that liquefaction and liquefaction-related seismic hazards (e.g., dynamic settlement, ground subsidence, and/or lateral spreading) are not design considerations for the project.

7.4 Dynamic Compaction of Dry Soils

Relatively dry soils (e.g., soils above the groundwater table) with low density or softer consistency tend to undergo a degree of compaction during a seismic event. Earthquake shaking often induces significant cyclic shear strain in a soil mass, which responds to the vibration by undergoing volumetric changes. Volumetric changes in dry soils take place primarily through changes in the void ratio (usually contraction in loose or normally consolidated soft soils, and dilation in dense or overconsolidated stiff soils) and secondarily through particle reorientation. Such volumetric changes are generally non-recoverable. Based on our subsurface exploration, the alluvium at the site generally consists of medium dense to very dense materials. Accordingly, it is our opinion that dynamic compaction of dry soils is not a design consideration for the project.

8 CONCLUSIONS

Based on the results of our evaluation, it is our opinion that construction of the proposed project improvements is feasible from a geotechnical perspective, provided that the following recommendations are incorporated into the design and construction of the project. In general, the following conclusions were made:

- The site is underlain by fill and alluvial deposits generally consisting of moist, loose to very dense, silty sand, silty sand with gravel, poorly graded sand with gravel, poorly graded sand with silt and gravel, well graded sand with silt, poorly graded gravel with silt, and poorly graded gravel with silt and sand with scattered cobbles. Excavations in the existing fill and alluvium should be feasible with earthmoving equipment in good working condition.

- Earthwork at the site is anticipated to expose granular soils with little cohesion. Accordingly, steep excavations will be subject to caving. Temporary construction slopes should be excavated at a slope ratio of approximately 1½:1 (horizontal to vertical). Alternatively, steeper excavations should be shored. Excavations and shoring should conform to the Occupational Safety and Health Administration (OSHA) standards for Type C soil.

- The on-site sandy soils should be suitable for re-use as backfill once moisture-conditioned to near the optimum moisture content. Oversize materials with a diameter of 4 inches or more should be anticipated and should be removed before use as fill. The contractor should anticipate handling oversize materials during grading and construction.

- Groundwater was not encountered during our subsurface exploration. Historical high groundwater levels at the site are approximately 140 feet below the ground surface. Accordingly, groundwater and dewatering is not expected during construction. However, some groundwater seepage may be encountered during the construction and should be anticipated by the contractor.

- The site soils are not subject to dynamic settlement due to earthquake-induced liquefaction or to dynamic compaction of dry soils.

- The site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps, there are no known active faults underlying the site. Therefore, the potential for surface fault rupture at the site is considered to be low.

- Our limited laboratory corrosivity testing indicates that the on-site materials can be classified as non-corrosive based on the California Department of Transportation (Caltrans, 2018) corrosion guidelines.

9 RECOMMENDATIONS

The recommendations presented in the following sections provide geotechnical criteria regarding the design and construction of the proposed site improvements. The recommendations are based on the results of our subsurface evaluation, geotechnical analysis, and our project understanding. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards. A summary of our recommended geotechnical parameters are presented on a summary table created by the tank manufacturer in Appendix C.

9.1 Earthwork

Based on our understanding of the project, earthwork at the site is anticipated to consist of site clearing, remedial grading associated with the preparation of the new tank pads, remedial grading and new fill placement for the groundwater treatment facility areas, an excavation up to approximately 25 feet deep and retaining wall backfills for construction of the drain vault, trenching and backfilling associated with underground utility installation, and finished grading for establishment of site drainage. Earthwork operations at the site should be performed in accordance with the recommendations provided in the following sections of this report and applicable governing agencies.

9.1.1 Pre-Construction Conference

We recommend that a pre-construction conference be held. The owner and/or their representative, the governing agencies' representatives, the civil engineer, the geotechnical engineer, and the contractor should attend to discuss the work plan, project schedule, and earthwork requirements.

9.1.2 Demolition, Clearing, and Grubbing

Prior to performing excavations or other earthwork, the area should be cleared of existing structures, reservoir improvements, AC pavements, rubble and debris, abandoned utilities, surface obstructions, and other deleterious materials. Existing utilities within the project limits should be re-routed or protected from damage by construction activities. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dumpsite.

9.1.3 Excavation Characteristics

Based on our field exploration, we anticipate that excavations at the site may be accomplished with conventional earthmoving equipment in good working condition. We anticipate that existing undocumented fill and alluvial materials encountered during construction will be generally comprised of interbedded layers of silty sand, sand, and gravel. Cobbles and possible boulders should also be anticipated during construction. Oversized material is not suitable for backfill and should be broken into smaller pieces or disposed off-site. Contractors should make their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

9.1.4 Temporary Excavations

Temporary excavations up to approximately 4 feet in depth should be generally stable at a slope inclination no steeper than 1:1 (horizontal to vertical). Excavations that expose friable, relatively dry sands and gravels, however, may be subject to caving. Excavations that are unstable or deeper than 4 feet may need to be laid back at a slope inclination of approximately 1.5:1 (horizontal to vertical) or flatter. Where temporary slopes are not possible, shoring will be involved.

We recommend that trenches and excavations be designed and constructed in accordance with OSHA regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Trenches and excavations over 20 feet deep should be designed by the contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that the materials on site be considered as OSHA soil Type C. If seepage is encountered in excavations, they may need shoring or the seepage may be mitigated by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor.

9.1.5 Shoring

We anticipate that construction of some project improvements will involve shored vertical excavations. Shoring systems should be designed for the anticipated soil conditions using the lateral earth pressure values shown on Figures 7 and 8. The recommended design pressures are based on the assumptions 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.

9.1.6 Subgrade Preparation

After the site has been cleared of surface improvements, vegetation, and subsurface obstructions, remedial grading operations can be performed to support the construction of the proposed improvements. Due to the site history, it is possible that prior improvements including, but not limited to, concrete, underground vaults, pipelines, utilities, and voids will be encountered during grading activities. In order to provide suitable support and reduce the potential settlements of the proposed improvements, we recommend that soils beneath the proposed structure footprints be overexcavated and replaced with newly compacted fill. The recommended depths and lateral limits of the new fill placed beneath the improvements are presented below.

9.1.6.1 New Reservoir and Clearwell Tanks

In order to provide suitable support and reduce the potential settlement of the proposed new reservoir and clearwell tanks, we recommend that the soil beneath the planned foundation areas be overexcavated and recompacted to a depth that provides 2 or more feet of newly compacted fill beneath the proposed tank foundations. The overexcavation should expose relatively dense fill or native alluvial deposits. The limits of the excavation should extend laterally so that the bottom of the excavation is approximately 5 feet beyond the outside edge of the tank’s footprint, or a distance corresponding to the depth of the overexcavation, whichever is farther. The excavation bottom should be evaluated by our representative during the excavation work. Additional overexcavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. Prior to placing newly compacted fill, the exposed bottom should be scarified, moisture-conditioned, and recompacted to a depth of approximately 8 inches.

Care should be taken by the contractor to avoid undermining adjacent existing foundations and improvements. New excavations should not extend within the “zone of influence” of existing foundations, which is defined as a 1:1 (horizontal to vertical) plane projecting out from the bottom outside edge of the foundations. In the event that excavations will extend within the “zone of influence” of existing foundations, our office should be notified and appropriate recommendations provided, such as temporary underpinning of impacted foundations and/or temporary shoring.

9.1.6.2 New Equipment Pads and Structures with Shallow Foundations

In order to provide suitable support and reduce the potential settlement of proposed new structures with shallow foundations (i.e., groundwater treatment facility equipment, drain vault, emergency generators, etc.), we recommend that the structure footprint be overexcavated and recompacted to a depth that provides 2 or more feet of newly compacted fill beneath the proposed foundations. The overexcavation should expose relatively dense fill or native alluvial deposits. The limits of the excavation should extend laterally so that the bottom of the excavation is approximately 5 feet beyond the outside edge of the structure’s footprint, or a distance corresponding to the depth of the overexcavation, whichever is farther. The excavation bottom should be evaluated by our representative during the excavation work. Additional overexcavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. Prior to placing new compacted fill in areas that are overexcavated and/or in areas where the existing subgrade will be raised with new fill, the exposed bottom should be scarified, moisture-conditioned, and recompacted to a depth of approximately 8 inches.

Care should be taken by the contractor to avoid undermining adjacent existing foundations and improvements. New excavations should not extend within the “zone of influence” of existing foundations, which is defined as a 1:1 (horizontal to vertical) plane projecting out from the bottom outside edge of the foundations. In the event that excavations will extend within the “zone of influence” of existing foundations, our office should be notified and appropriate recommendations provided, such as temporary underpinning of impacted foundations and/or temporary shoring.

9.1.6.3 Exterior Hardscape

In order to provide suitable support and reduce the potential settlement of hardscape, we recommend that the subgrade materials beneath the proposed hardscape areas be scarified to a depth of approximately 8 inches, moisture-conditioned to near optimum moisture content, and recompacted. The limits of the scarification should extend laterally 2 feet beyond the outside edge of hardscape. The exposed subgrade should be evaluated by our representative during the excavation work. Loose, soft, and/or wet areas may need to be overexcavated, depending on our observations during construction. Prior to placing new compacted fill in areas that are overexcavated and/or in areas where the existing subgrade will be raised with new fill, the exposed bottom should be scarified, moisture-conditioned, and recompacted to a depth of approximately 8 inches.

9.1.7 Fill Material

In general, the on-site soils should be suitable for re-use as fill, structural fill, and trench backfill, provided they are free of trash, debris, roots, vegetation, 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. On-site soils used as fill will involve moisture conditioning to achieve appropriate moisture content for compaction.

Fill used as backfill behind retaining walls should consist of free-draining, granular, non-expansive soil that conforms with the latest edition of “Greenbook” Standard Specifications for Public Works Construction for structure backfill. “Non-expansive” can be defined as soil having an expansion index (EI) of 20 or less in accordance with ASTM International (ASTM) D 4829 (CBC, 2019).

Imported materials should consist of clean, non-expansive, granular material, which conforms to the latest edition of “Greenbook” Standard Specifications for Public Works Construction for structure backfill in accordance with ASTM D 4829 (CBC, 2019). Soil should also be tested for corrosive properties prior to importing. We recommend that the imported materials comply with the Caltrans (2018) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million (ppm) or less, a soluble sulfate content of approximately 0.15 percent (1,500 ppm) or less, a pH value of 5.5 or higher, and a resistivity of 1,100 ohm-centimeters (ohm-cm) or more). Materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

9.1.8 Fill Placement and Compaction

Fill placed for support of the new reservoir and clearwell tanks should be compacted in horizontal lifts to a relative compaction of 95 percent or more as evaluated by ASTM D 1557.

Fill placed outside the tank areas, including beneath other site improvements such as the groundwater treatment facility, drain vault, and trench backfill should be compacted in horizontal lifts to a relative compaction of 90 percent or more as evaluated by ASTM D 1557. Fill soils should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. The optimum lift thickness of fill will depend on the type of compaction equipment used but generally should not exceed 8 inches in loose thickness. Placement and compaction of the fill soils should be in general accordance with appropriate governing agency grading ordinances and good construction practice.

9.2 Underground Utilities

We anticipate that new underground utility pipelines will be supported on fill or alluvial deposits. Utility trenches should not be excavated parallel to building footings. If needed, trenches can be excavated adjacent to a continuous footing, provided that the bottom of the trench is located above a 1:1 (horizontal to vertical) plane projected downward from a point 6 inches above the bottom of the adjacent footing. Utility lines that cross beneath footings should be encased in concrete below the footing.

9.2.1 Pipe Bedding

We recommend that pipelines be supported on 6 inches or more of granular bedding material such as sand with a sand equivalent (SE) value of 30 or more. Bedding material should be placed and compacted 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 for bedding material. It has been our experience that the voids within a crushed rock material are sufficiently large enough to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface.

Special care should be taken not to allow voids beneath and around the pipe. Bedding material and compaction requirements should be in accordance with the recommendations of this report, the project specifications, and applicable requirements of the appropriate agencies. Compaction of the bedding material and backfill should proceed along both sides of the pipe concurrently and be compacted to 90 percent or more relative compaction as evaluated by ASTM D 1557.

9.2.2 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed on the sides of buried flexible pipelines for the purpose of evaluating lateral deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of 1,000 pounds per square inch (psi) be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

9.2.3 Lateral 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.

9.3 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 1 presents the site-specific spectral response acceleration parameters in accordance with the CBC (2019) guidelines.

Table 1 – 2019 California Building Code Seismic Design Criteria

Site Coefficients and Spectral Response Acceleration Parameters	Values
Site Class	D
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	2.041g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.750g
Site-Specific Spectral Response Acceleration at 0.2-second Period, S_{MS}	2.073g
Site-Specific Spectral Response Acceleration at 1.0-second Period, S_{M1}	1.500g
Site-Specific Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.382g
Site-Specific Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.000g
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.896g

9.4 Foundations

The proposed new reservoir and clearwell tanks may be supported on a ring wall foundation or a mat foundation bearing on compacted fill material prepared in accordance with the recommendations presented in the Earthwork section of this report. Other site improvements may be supported on shallow spread footings. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

9.4.1 Ring Wall Foundations

The footing design recommendations provided below are based on the assumption that the footing for the new reservoir and clearwell tanks will extend 2 feet or more below the lowest adjacent finished grade with a width of 5 feet or more. These recommendations are also based on the assumption that the footings will bear on engineered, low-expansion, granular fill soils compacted to 95 percent relative compaction or more. Spread footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top and two placed near the bottom of the footings, and further detailed in accordance with the recommendations of the structural engineer. In addition, requirements of the governing jurisdictions and applicable building codes should be considered in the design.

Footings, as described above and bearing on compacted fill soils with very low to low expansion potential, may be designed using an allowable bearing capacity of 5,000 pounds per square foot (psf). Total and differential settlements for the new reservoir footings designed and constructed in accordance with the above recommendations are estimated to be on the order of ½ inch and ¼ inch over a horizontal span of approximately 40 feet, respectively. Considering the granular nature of the foundation subgrade, settlement is anticipated to occur relatively quickly.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.40, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 400 psf per foot of depth for level ground condition up to a value of 4,000 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.

To reduce the potential for pipe-to-tank differential settlement, which could cause pipe shearing, we recommend that a flexible pipe joint be located close to the exterior of the tank. 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 up to approximately 1 inch.

9.4.2 Mat Foundations

Mat foundations for at-grade equipment bearing on compacted fill as outlined in the preceding sections of this report may be designed using a net allowable bearing capacity of 3,000 psf. The total and differential settlements corresponding to these allowable bearing loads are estimated to be less than approximately ½ inch and ¼ inch over a horizontal span of 40 feet, respectively. The allowable bearing capacity may be increased by one third when considering loads of short duration, such as wind or seismic forces. 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 150 tons per cubic foot (tcf) may be used for the subgrade soils in evaluating such deflections. This value is based on a unit square foot area and should be adjusted for large mats. Adjusted values of the modulus of subgrade reaction, K, can be obtained from the following equations for mats of various widths:

$K = 150[(B+1)/2B]^2$ (tcf); where B is the width of mat measured in feet

For frictional resistance to lateral loads on mat foundations, we recommend a coefficient of friction of 0.40 for compacted granular subgrade soil. For a mat with an embedment depth shallower than 2 feet, an allowable passive earth pressure of 400 psf per foot should be ignored while evaluating lateral resistance; only frictional resistance should be considered. For mats with embedment depths more than 2 feet, passive earth pressure may be combined with frictional resistance to evaluate the total lateral resistance. In such cases, the 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 resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.4.3 Spread Footings

The drain vault walls, retaining walls, groundwater treatment facility, and other miscellaneous at-grade equipment pads, and retaining walls may be supported on shallow spread footings bearing on compacted fill prepared in accordance with the earthwork recommendations of this report. Footings should extend 24 inches or more below the lowest adjacent finished grade. Continuous footings should have a width of 24 inches or more. Isolated pad footings should have a width of 24 inches or more. Spread footings should be reinforced with a minimum of two No. 4 steel reinforcing bars, one placed near the top and one placed near the bottom of the footings, and further detailed in accordance with the recommendations of the structural engineer.

Footings, as described above and bearing on compacted fill soils with low expansion potential, may be designed using a net allowable bearing capacity of 4,000 psf. The allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces.

Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be less than approximately 1 inch and ½ inch over a horizontal span of 40 feet, respectively.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.40, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 400 psf per foot of depth for level ground condition up to a value of 4,000 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. In the event that the passive resistance is greater than one-half of the total allowable resistance, the passive resistance should be reduced to be the same value as the frictional resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.5 Lateral Earth Pressures for Retaining Walls and Drain Vault

Lateral earth pressures recommended for design of yielding retaining walls are provided on Figure 10. Passive pressures may be increased by one-third when considering loads of short duration, including wind and seismic loads. Retaining walls should be backfilled with free-draining, granular soil with a low-expansion potential. Measures should be taken to reduce the potential for build-up of hydrostatic pressure behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as depicted on Figure 11. Solid outlet pipes should be connected to the perforated drains and then routed to a suitable area for discharge. If weep holes or other outlets along the drain lines are deemed necessary, we recommend that they extend to the front and near the base of the wall, and be spaced at approximately 20 feet.

Below-grade walls of the drain vault structure may be considered to be restrained from lateral displacement under static loading conditions. Lateral earth pressures for precast vaults are typically provided with the precast structure specifications. In the event that a cast-in-place vault is used for the project, vault walls subjected to lateral earth pressures should be designed using the parameters presented on Figure 12.

9.6 Tank Slabs-On-Grade

Floor slabs subjected to dead and live loads should be designed by the project structural engineer based on the anticipated loading conditions. Floor slabs should be underlain by compacted soil prepared with the recommendations presented in this report. We recommend that slabs be 6 inches thick and reinforced with No. 4 steel reinforcing bars placed 24 inches on-center (each way) placed near the mid-height of the slab. The placement of the reinforcement in the slab is vital for satisfactory performance. The floor slab and foundations should be tied together by extending the slab reinforcement into the foundations.

The slab should be underlain by a 4-inch-thick, or more, layer of sand or gravel with a particle size of approximately 3/8 inch or smaller. Soils underlying the slab should be moisture-conditioned and compacted in accordance with the recommendations presented in this report prior to concrete placement. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab.

9.7 Pavement Construction

Paved access roads and parking will be part of the proposed improvements for this project. Accordingly, laboratory testing was performed on a representative subgrade soil sample and indicated an R-value ranging of approximately 76. Considering the potential variability of the on-site soils, an R-value of 50 was used for the pavement design in accordance with Caltrans guidelines. We evaluated the structural pavement sections assuming traffic indices (TI) of 6, 7, and 8, which generally cover for the range of traffic loading conditions typically associated with infrequent heavy truck traffic and emergency fire lane traffic.

Our AC pavement analysis was performed using the methodology outlined by the Highway Design Manual (Caltrans, 2019b). For the design of Portland Cement Concrete (PCC) pavements, we used the methodology presented in the Navy Pavement Design Manual (1979). The analysis assumes an approximate 20-year design life for the new pavements. Our preliminary pavement sections are presented in Table 2.

Table 2 – Preliminary Flexible and Rigid Pavement Structural Sections

Traffic Index	Full Depth AC (inches)	AC over CAB or CMB (inches)	PCC (inches)
6.0	6.0	3.5/4.5	6.5
7.0	7.0	3.5/5.5	8.5
8.0	8.0	5.0/5.5	9.5

Notes:
AC – Asphalt Concrete
CAB – Crushed Aggregate Base
CMB – Crushed Miscellaneous Base
PCC – Portland Cement Concrete, with a 28-day compressive strength of 2,500 psi

Prior to placement of new pavement sections, the upper approximately 8 inches of the subgrade beneath new pavements should be scarified, moisture-conditioned, and re-compacted to a relative compaction of 90 percent or more as evaluated by ASTM test method D1557. Base material should be placed at a relative compaction of 95 percent or more as evaluated by the latest edition of ASTM D 1557. If a full depth AC pavement is selected, the subgrade soil should be compacted to 95 percent relative compaction. Grinding and recycling existing AC and existing base material may be considered as a potential source of CMB provided they meet the requirements of the Standard Specifications.

Aggregate base material should conform to the latest specifications in Section 200 2.2 for crushed aggregate base or Section 200 2.4 for crushed miscellaneous base of the Greenbook and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1557. AC should conform to Section 203.6 of the Greenbook and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1557.

9.8 Exterior Flatwork

New exterior concrete sidewalks and flatwork (hardscape) should 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.

9.9 Corrosivity

Laboratory testing was performed on two representative soil samples to evaluate 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 test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated soil pH of approximately 6.4 and 6.7. The measured electrical resistivity values were approximately 1,970 and 2,645 ohm-cm. The measured chloride content values of the two samples were approximately 140 and 175 ppm. The sulfate content of both samples was approximately 0.006 percent (i.e., 60 ppm). Based on the laboratory test results and Caltrans (2018) criteria, the soils at the project site can be classified as non-corrosive, which is defined as having earth materials with less than 500 ppm chlorides, less than 1,500 ppm sulfates, a pH of 5.5 or more, and an electrical resistivity of more than 1,100 ohm-cm.

9.10 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 criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight. The potential for sulfate attack is 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 samples tested for this evaluation, using Caltrans Test Method 417, indicates a water-soluble sulfate content for both samples of 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 soils on site, consideration should be given to using Type II/V cement for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structures 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.

9.11 Drainage

Positive surface drainage is imperative for performance of site improvements. Positive drainage should be provided and maintained to transport surface water away from foundations and other site improvements. Positive drainage incorporates a slope of 2 percent or more over a distance of 5 feet or more away from structures, pavements, and top of slopes. Surface water should not be allowed to flow over slope faces or pond adjacent to footings.

9.12 Instrumentation and Documentation of Conditions of Adjacent Properties

We recommend that consideration be given to implementing an instrumentation program for evaluating design assumptions, monitoring vibrations at adjacent structures, monitoring deformations of the excavations, and monitoring ground surface settlements. The monitoring program should include seismographs and an array of surface control points. The data obtained should be distributed to appropriate parties during the course of construction.

9.12.1 Documentation of Existing Conditions

We recommend that consideration be given to performing pre-construction condition surveys on structures within approximately 50 feet of the proposed excavations prior to construction. This survey should include locating existing cracks and measuring widths of cracks, in combination with videotape documentation of existing conditions.

9.12.2 Construction Vibrations

People can perceive vibrations from construction activities at significantly lower levels than might cause cosmetic damage to structures. Jones & Stokes (2004) indicate that transient vibrations from construction activities may be noticeable. The vibrations may be disturbing and result in complaints and/or damage claims at peak particle velocities as low as 0.2 to 0.6 inch per second (ips). However, these vibration levels are well below the level considered to cause cosmetic damage to residential construction.

There is also the possibility of settlement of the sand, sand with silt, silty sand, and sandy silt underlying structures during construction activities. This settlement may result in damage to the structures. If the construction vibrations can be maintained below a peak particle velocity of 0.2 ips, the settlement should be limited to acceptable levels based on past projects in similar conditions.

For the above stated reasons, we recommend that consideration be given to using seismographs in the early stages of construction. Seismographs should be located near the residential structures near the construction sites. Additional seismographs should be located at various structures farther from the construction activities to monitor vibrations as a function of distance from the site. After review of the data obtained, the number of seismographs may be reduced at the discretion of the client and the geotechnical consultant.

9.12.3 Ground Surface Settlement

We recommend that consideration be given to installing arrays of ground surface settlement points around shored excavations. The settlement points should be installed near the excavations at approximately 20-foot spacing. We recommend that the contractor be responsible for maintaining total settlement at any survey point to less than ½ inch. If the settlements reach this limit, we recommend that a further review of construction methodologies be performed and appropriate changes be made.

It is also advisable to install and monitor survey points on nearby residences. In this way, a record of the performance of the structure will be maintained. This information, in combination with pre-construction surveys, is helpful in reducing the potential for damage claims regarding pre-existing cracks and to facilitate settlement of legitimate damage claims.

10 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 observed in our exploratory borings. It is imperative that the geotechnical consultant checks the subsurface conditions during construction.

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

- Observing clearing, grubbing, and removals.
- Observing excavation, placement, and compaction of fill, including trench backfill.
- Evaluating on-site soil for suitability as use as engineered fill/structural backfill prior to placement.
- 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 are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that the services of Ninyo & Moore are not utilized during construction, we request that the selected consultant provide the owner with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report.

11 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses 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.

12 REFERENCES

Abrahamson, N.A., Silva, W.J. and Kamai, R., 2014, Summary of the ASK14 Ground Motion Relation for Active Crustal Regions, Earthquake Spectra: Vol. 30, No. 3, pp. 1025-1055, dated August.

American Concrete Institute, 2015, Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary (ACI 318-14).

American Concrete Institute (ACI), 2016, ACI Manual of Concrete Practice.

American Society of Civil Engineers (ASCE), 2016, Minimum Design Loads and Associated Criteria for Building and Other Structures, ASCE Standard 7-16.

Applied Technology Council (ATC), 2021, Hazards by Location, https://hazards.atcouncil.org/#/.

ASTM International (ASTM), 2016, Annual Book of ASTM Standards, West Conshohocken, Pennsylvania.

Atik, Linda L. and Sitar, N., 2010, Seismic Earth Pressures on Cantilever Retaining Structures, ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 136, No. 10, October 1.

Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M., 2014, NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes, Earthquake Spectra, Vol. 30, No. 3, pp. 1057-1085, dated August.

Bowles, J.E., 1996, Foundation Analysis and Design, Fifth Edition, The McGraw-Hill Companies, Inc.

Bozorgnia, Y. and Campbell, K.W., 2004, The Vertical-to-Horizontal Response Spectral Ratio and Tentative Procedures for Developing Simplified V/H and Vertical Design Spectra, Journal of Earthquake Engineering, Vol. 8, No. 2, pp. 175-207.

Brandenberg, S.J., Bellana, N., and Shantz, T., 2009, Shear Wave Velocity as Function of SPT Penetration Resistance and Vertical Effective Stress at California Bridge Sites, Soil Dynamics and Earthquake Engineering, In Press, Corrected Proof, Available online 26 May 2010.

Building Seismic Safety Council, 2009, National Earthquake Hazards Reduction Program (NEHRP) Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750).

California Building Standards Commission, 2019, California Building Code: California Code of Regulations, Title 24, Part 2, Volumes 1 and 2, based on the 2018 International Building Code.

California Department of Conservation, Division of Mines and Geology, State of California, 1998, Seismic Hazard Evaluation of the Pasadena 7.5-Minute Quadrangle, Los Angeles County, California: Open-File Report 98-05.

California Department of Conservation, Division of Mines and Geology, State of California, 1999, Seismic Hazard Zones Official Map, Pasadena Quadrangle, 7.5-Minute Series: Scale 1:24,000, Open-File Report 98-05, dated March 25.

California Department of Transportation (Caltrans), 2019a, CalME, Version 3.0.0, dated June.

California Department of Transportation (Caltrans), 2019b, Highway Design Manual.

California Department of Transportation (Caltrans), 2021, ARS Online Web Tool, version 3.0.2, https://arsonline.dot.ca.gov/.

California Geological Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A, dated September 11.

California Geological Survey (CGS), 2016, CGS Map Sheet 48: Shear-wave Velocity in Upper 30m of Surficial Geology (Vs30); https://gis.conservation.ca.gov/server/rest/services/CGS/MS48_Vs30/MapServer, dated September 30.

California Geological Survey (CGS), 2018, Earthquake Fault Zones, A Guide for Government Agencies, Property Owners/Developers, and Geoscience Practitioners for Assessing Fault Rupture Hazards in California: Special Publication 42.

Campbell, K.W., and Bozorgnia, Y., 2014, NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra, Earthquake Spectra, Vol. 30, No. 3, pp. 1087-1115, dated August.

Carollo Engineers, 2015, Seismic Evaluation (Draft), Sunset Reservoir No. 1, Pasadena Water & Power, dated June.

Carollo Engineers, 2018, Preliminary Design Report (Draft), City of Pasadena, Pasadena Water and Power, Sunset Reservoir Replacement Project, dated March.

Chiou, B. S.-J., and Youngs, R.R., 2014, Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra, Earthquake Spectra, August 2014, Vol. 30, No. 3, dated August.

City of Pasadena Engineering Street Department, 1960, Project No. 188 Mountain Street Drain As-Built, City of Pasadena, Sheets 1 through 7, dated July 13.

City of Pasadena, 2017a, DS 314A, City of Pasadena Water District Map, printed November 6.

City of Pasadena, 2017b, DS 414, City of Pasadena Water District Map, printed November 6.

County of Los Angeles, Public Works, 2021, General Wells, https://dpw.lacounty.gov/general/wells/.

Diaz Yourman & Associates (DYA), 2009, Geotechnical Investigation, Sunset Reservoir Perchlorate Treatment Facility, Pasadena, California, Project No. 2005-017, dated January 15.

Dibblee, T.W., Jr., 1989, Geologic Map of the Pasadena Quadrangle, Los Angeles County, California: Dibblee Foundation, DF-23, Scale 1:24,000.

DTN Engineers, Inc., 2009, Pasadena Water and Power, W.O. 02549, Glorietta Booster Station Electrical Upgrades, Water Division, Pasadena Water and Power, City of Pasadena, File Number E-1663, Sheets 1 through 18, dated January.

Federal Emergency Management Agency, 2008, Flood Insurance Rate Map, Los Angeles County, California and Incorporated Areas, Panel 1375, Map Number 06037C1375F, dated September 26.

G&E Engineering Systems, Inc., 2006a, Seismic Criteria Document, City of Pasadena, Water Department, Report 81.01.06, Revision 0, dated June 23.

G&E Engineering Systems, Inc., 2006b, Seismic Vulnerability Assessment, City of Pasadena, Water System, Report 81.01.08, Revision 1, dated December 10.

G&E Engineering Systems, Inc., 2006c, Seismic Vulnerability Assessment, Supplementary Topics: Storage, Water Quality, Benefit Cost, Service Goals, Emergency Planning, Hydraulics, SCADA, Report 81.01.07, Revision 0, dated June 26.

Google Earth, 2021, Website: http://earth.google.com.

Historical Aerials, 2021, www.historicaerials.com.

Jennings, C.W., and Bryant, W.A., 2010, Fault Activity Map: California Geological Survey, California Geologic Data Map Series, Map No. 6, Scale 1:750,000.

Jones and Stokes, 2004, Transportation and Construction Induced Vibration Guidance Manual, California Department of Transportation, Environmental Program, Environmental Engineering, Noise, Vibrations, and Hazardous Waste Management Office, dated June.

Kennedy Jenks, 2021a, Sunset Complex – Draft Basis of Design Report, KJ Project No. 2044525*00, dated February 9.

Kennedy Jenks, 2021b, Sunset Complex, Draft Basis of Design Report As Built, Pasadena Water & Power, Pasadena, CA, Sheets C-3, C-4, and C-5, dated March 4.

Mononobe, N. and Matsuo, H., 1929, On the Determination of Earth Pressure during Earthquakes, Proceedings of the World Engineering Conference, No. 9.

Naval Facilities Engineering Command (NAVFAC), 1979, Civil Engineering, Pavements, Design Manual 5.4, dated October.

Naval Facilities Engineering Command (NAVFAC), 1982, Foundations and Earth Structures Design Manuals, dated May.

Ninyo & Moore, 2021, Proposal for Geotechnical Evaluation, Sunset Reservoir Storage Tanks Project, Pasadena, California, Proposal No. 04-03099, dated January 25.

Norris, R.M., and Webb, R.W., 1990, Geology of California, Second Edition: John Wiley & Sons.

Pasadena Department of Public Works, 2021, Pasadena Sewer Map Viewer, https://pasgis.maps.arcgis.com/apps/webappviewer/index.html?id=0f6f9010fe0340f0b5f832e805d3acd9

Pasadena Water Department, 1934a, 36” C.I. Main In Sunset Ave., From Sunset Res. No. 2 To A Point 138FT. North of Mountain St., File Number E-126, dated May 17.

Pasadena Water Department, 1934b, Sunset Reservoir No. 1, Plan, Construction of 4 Ft. Concrete Wall, File Number E-122, Sheets 1 through 7, dated March 15.

Pasadena Water Department, 1934c, Sunset Reservoir No. 2 Plan, Construction of 4 Ft. Concrete Wall & Roof, File Number P-1054, Sheets 1 through 6, dated July 24.

Pasadena Water Department, 1940, 24” By Pass Connection, Sunset Ave. & Mountain St., Work Order 3535, File Number C-272, dated December 20.

Pasadena Water Department, 1964, Glorieta Booster Station As-Built, Pasadena Water Department, File Number P-5016 Sheets 1 through 6, dated January 9.

Public Works Standard, Inc., 2018, The “Greenbook”: Standard Specifications for Public Works Construction, 2018 Edition, with Errata No. 1 dated 2019.

S.D.K., 1994, New Sunset Well As-Built, Water & Power Department, City of Pasadena, Sheets 1 through 15, dated August 3.

Seyhan, E, 2014, Weighted Average 2014 NGA West-2 GMPE, Pacific Earthquake Engineering Research Center.

Southern California Earthquake Center, 2005, SCEC Community Velocity Model, Version 4.

State of California, State Water Resources Control Board (SCWRCB), 2021, GeoTracker Database System, http://geotracker.swrcb.ca.gov/.

Stetson Engineers, Inc., 2013, Pasadena Water and Power, Sunset Reservoir (201 W. Mountain St.), Water Treatment Project, Phase 1 (Disinfection Facility) As-Built, D.S>314-A, Sheets 1 through 14, dated January 3.

United States Geological Survey (USGS), 2018, USGS US Topo 7.5-Minute Map for Pasadena, CA: USGS - National Geospatial Technical Operations Center (NGTOC), dated September 24.

United States Geological Survey (USGS), 2021a, Unified Hazard Tool; https://earthquake.usgs.gov/hazards/interactive/.

United States Geological Survey (USGS), 2021b, Quaternary Fault and Fold Database of the United States, https://www.usgs.gov/natural-hazards/earthquake-hazards/hazards, accessed December 18.

United States Geological Survey and Southern California Earthquake Center, 2020, Open Seismic Hazard Analysis, version 2.0, http://www.opensha.org/.

University of California Santa Barbara Library (UCSBL), 2021, Historic Aerial Photographs.

USDA, Aerial Photograph Date 11-19-53, Flight AXJ-14K, Number 10 and 11, Scale 1:20,000.

Water & Power Department, 2010, As-Built, Mountain Street, Water & Power Department, City of Pasadena, Sheets 1 through 3, File Number E-1409, dated October 22.

Water Division, City of Pasadena, 2017, Sunset Reservoir Replacement Project, City of Pasadena, Water District Map, Figure 2-2, dated November.

William Lettis & Associates (WLA), Inc., 2005, Pasadena Water and Power, Geological/Geotechnical Seismic Vulnerability, Assessment Summary Report, dated January.

Wills, C.J., and Clahan, L.B., 2006, Developing a Map of Geologically Defined Site-Condition Categories for California, Bulletin of the Seismological Society of America, v. 96, no. 4A, p. 1483–1501.

Wills, C.J., C.I. Gutierrez, F.G. Perez, and D.M. Branum (2015). A next-generation Vs30 map for California based on geology and topography, Bulletin of the Seismological Society of America, 105, no 6, p. 3083-3091.

Yerkes, R.F., and Campbell, R.H., 2005, Preliminary Geologic map of the Los Angeles 30' x 60' Quadrangle, Southern California, Version 1.0: United States Geological Survey Open-file Report 05-1019: Scale 1:100,000.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcuson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., II., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 124(10), 817-833.

PLATES

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: KENNEDY JENKS, 2021.

PLATE 1

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SITE PLAN

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET 0 40 80

PLATE 2

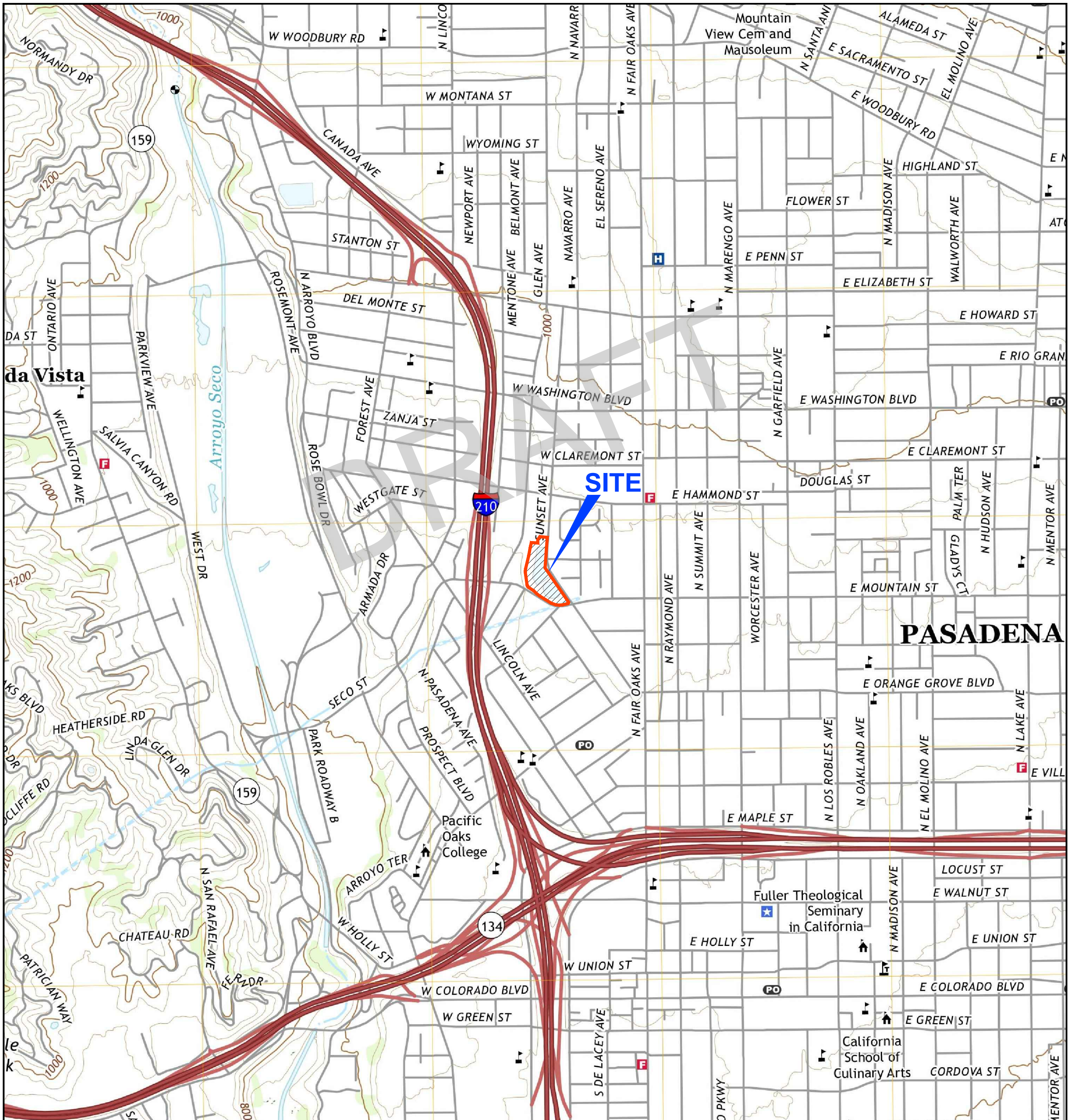
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CROSS SECTIONS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

FIGURES



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

N

FEET

0 2,000 4,000

211621001_SL.dwg 04/02/2021 JDP

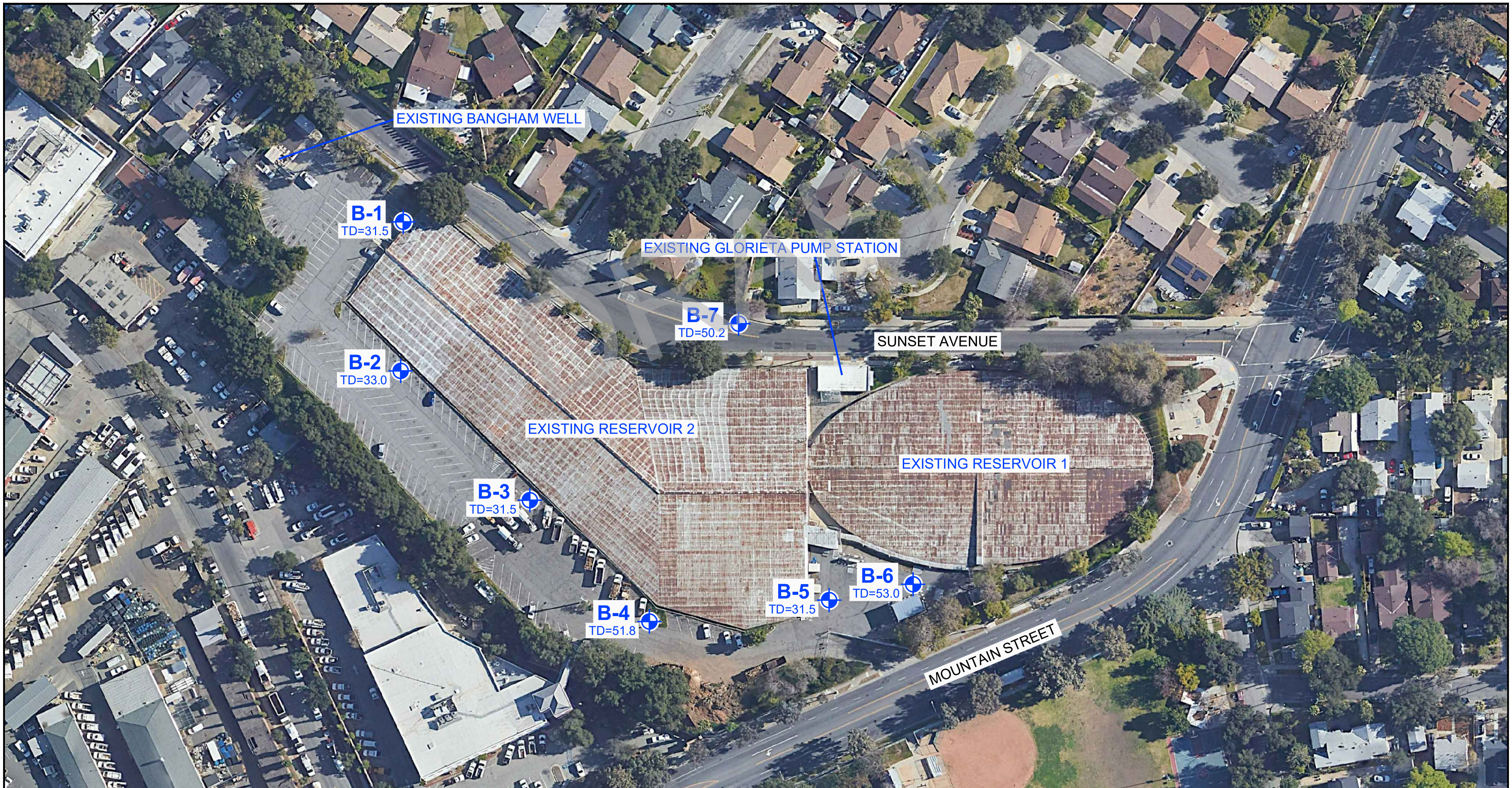
FIGURE 1

SITE LOCATION

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 I 4/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



LEGEND

B-7 TD=50.2 — BORING; TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: GOOGLE EARTH, 2021.

FEET
0 150 300

FIGURE 2

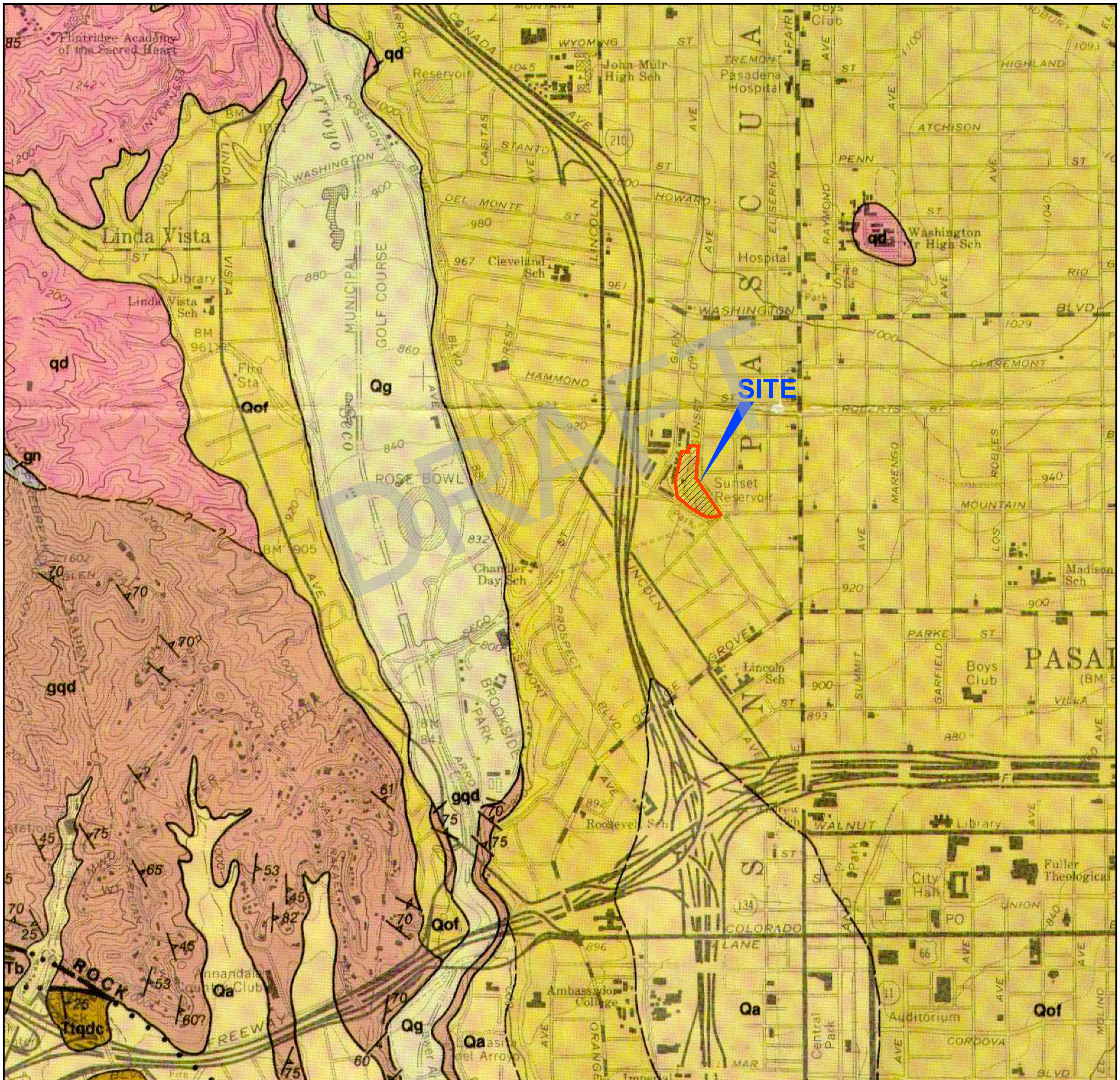
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

BORING LOCATIONS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 I 4/21

211621001_BL.dwg 04/02/2021 JDP



LEGEND

Qg	STREAM CHANNEL DEPOSITS	qd	QUARTZ DIORITE		GEOLOGIC CONTACT
Qa	ALLUVIAL DEPOSITS	gqd	GNEISSOID QUARTZ DIORITE		EAGLE ROCK FAULT; DOTTED WHERE CONCEALED
Qof	ALLUVIAL FAN DEPOSITS				

N

FEET

0 2,000 4,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: DIBBLEE, 1989.

211621001_RG.dwg 04/02/2021 JDP

FIGURE 3

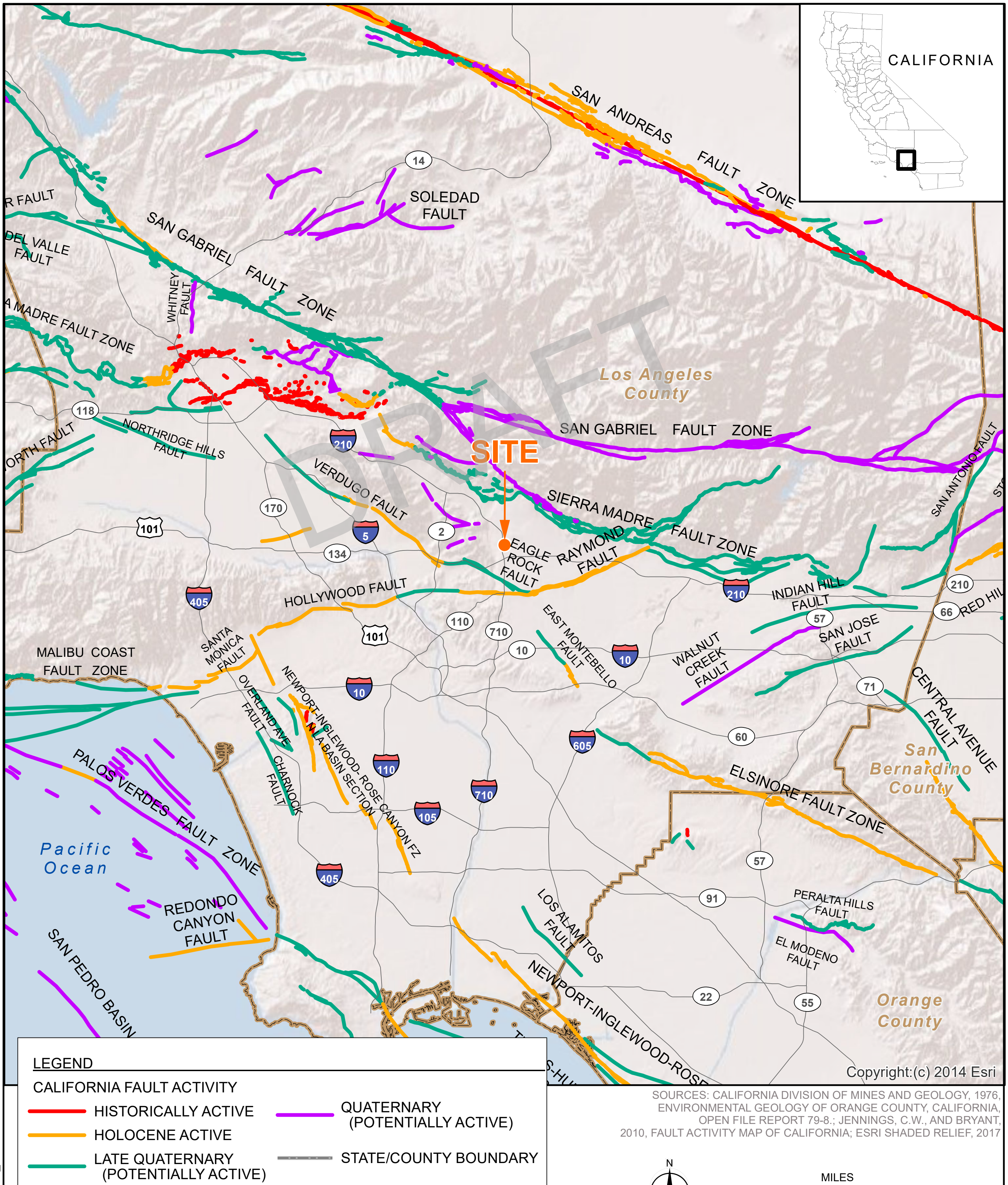
Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 I 4/21



SOURCES: CALIFORNIA DIVISION OF MINES AND GEOLOGY, 1976, ENVIRONMENTAL GEOLOGY OF ORANGE COUNTY, CALIFORNIA, OPEN FILE REPORT 79-8.; JENNINGS, C.W., AND BRYANT, 2010, FAULT ACTIVITY MAP OF CALIFORNIA; ESRI SHADED RELIEF, 2017

N

MILES

0 8 16

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

211621001_FL.mxd 3/1/2021 JDL

FIGURE 4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

FAULT LOCATIONS

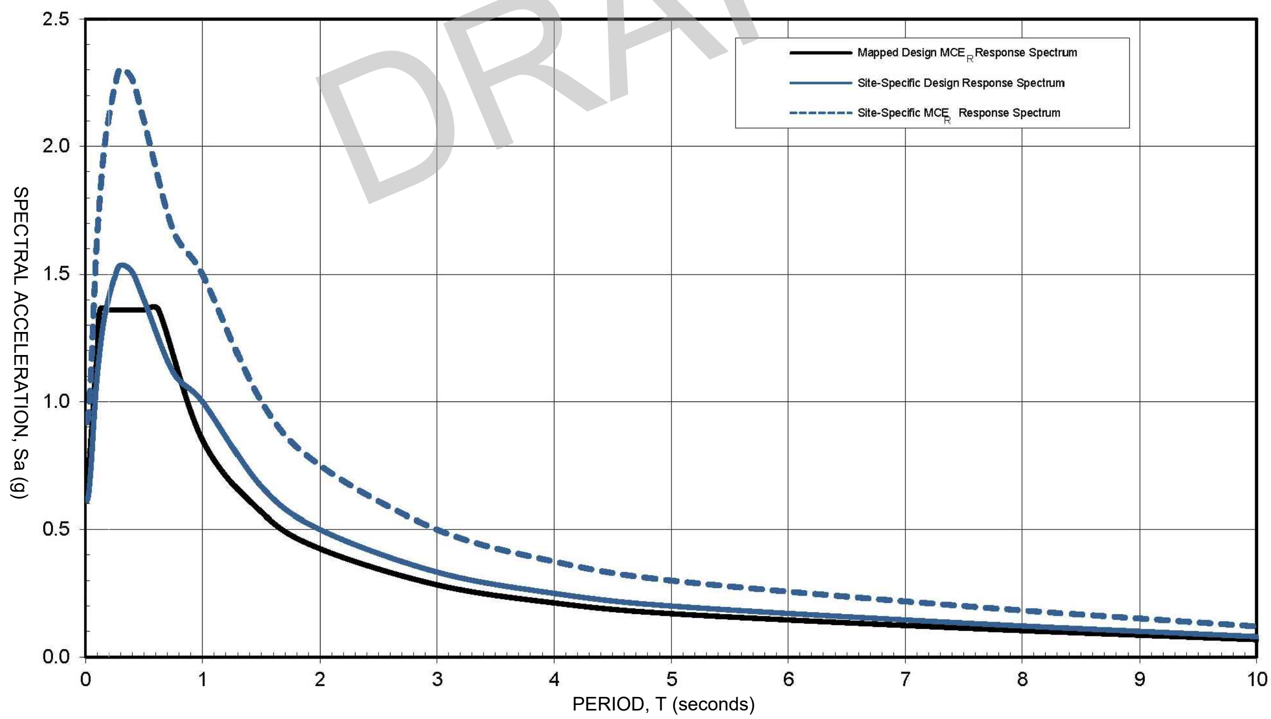
SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

PERIOD (seconds)	SITE-SPECIFIC MCER RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.918	0.612
0.020	0.928	0.619
0.030	0.972	0.648
0.050	1.140	0.760
0.075	1.416	0.944
0.100	1.647	1.098
0.150	1.944	1.296
0.200	2.113	1.408
0.250	2.232	1.488
0.300	2.303	1.536
0.400	2.263	1.509

PERIOD (seconds)	SITE-SPECIFIC MCER RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	2.101	1.400
0.750	1.673	1.116
1.000	1.500	1.000
1.500	1.000	0.667
2.000	0.750	0.500
3.000	0.500	0.333
4.000	0.375	0.250
5.000	0.300	0.200
7.500	0.200	0.133
10.000	0.120	0.080

S_{DS} = 1.382 g	S_{D1} = 1.000 g	S_{MS} = 2.073 g	S_{M1} = 1.500 g	PGA_M = 0.896 g



NOTES:

1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCER) 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.

2 The deterministic ground motion spectral response accelerations are for the 84th percentile of the 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 7.0 event on the Elysian Park fault zone located 10.4 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

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

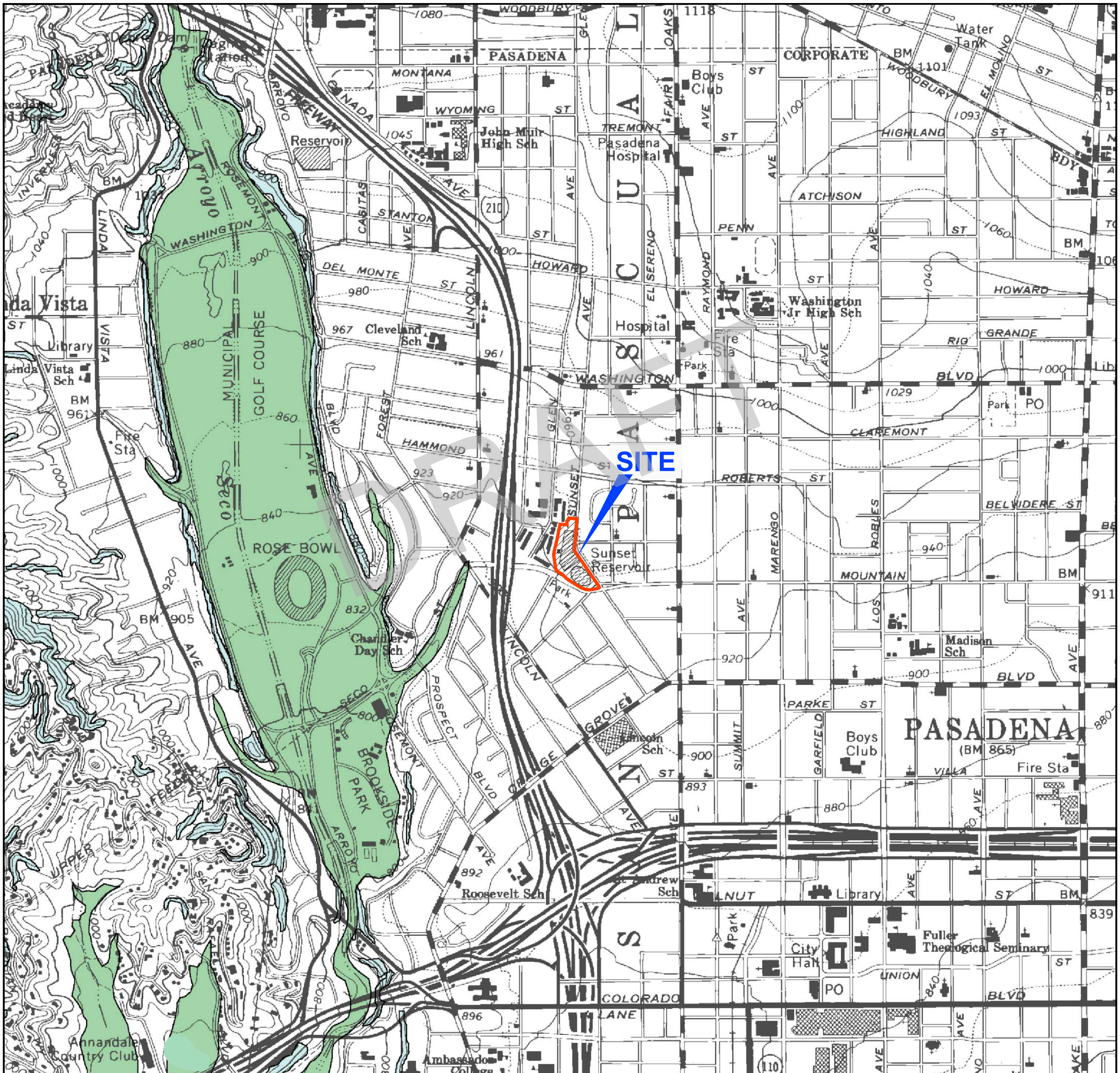
4 The Mapped Design MCE$_R$ Response Spectrum is computed from mapped spectral ordinates modified for Site Class D (stiff soil profile) per ASCE 7-16 Section 11.4. It is presented for the sake of comparison.

FIGURE 5

ACCELERATION RESPONSE SPECTRA

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



LEGEND

EARTHQUAKE-INDUCED LANDSLIDES
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.

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

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: CDMG, 1999.

N

FEET
0 2,000 4,000

211621001_SHZ.dwg 04/02/2021 JDP

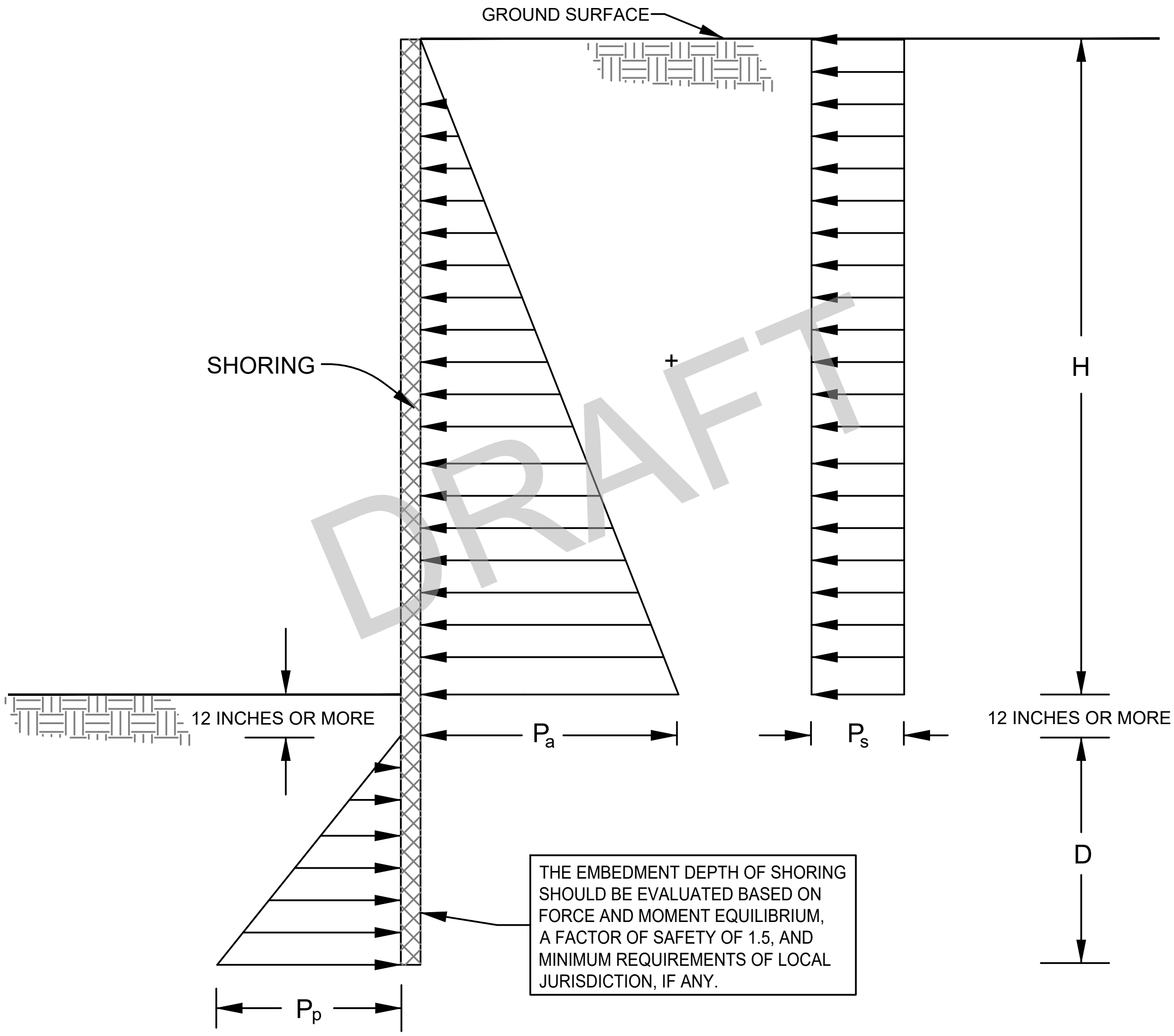
FIGURE 6

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SEISMIC HAZARD ZONES

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 I 4/21



NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_a = 35H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 400D psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. H AND D ARE IN FEET

NOT TO SCALE

211621001_LEP-CS1.dwg 04/02/2021 JDP

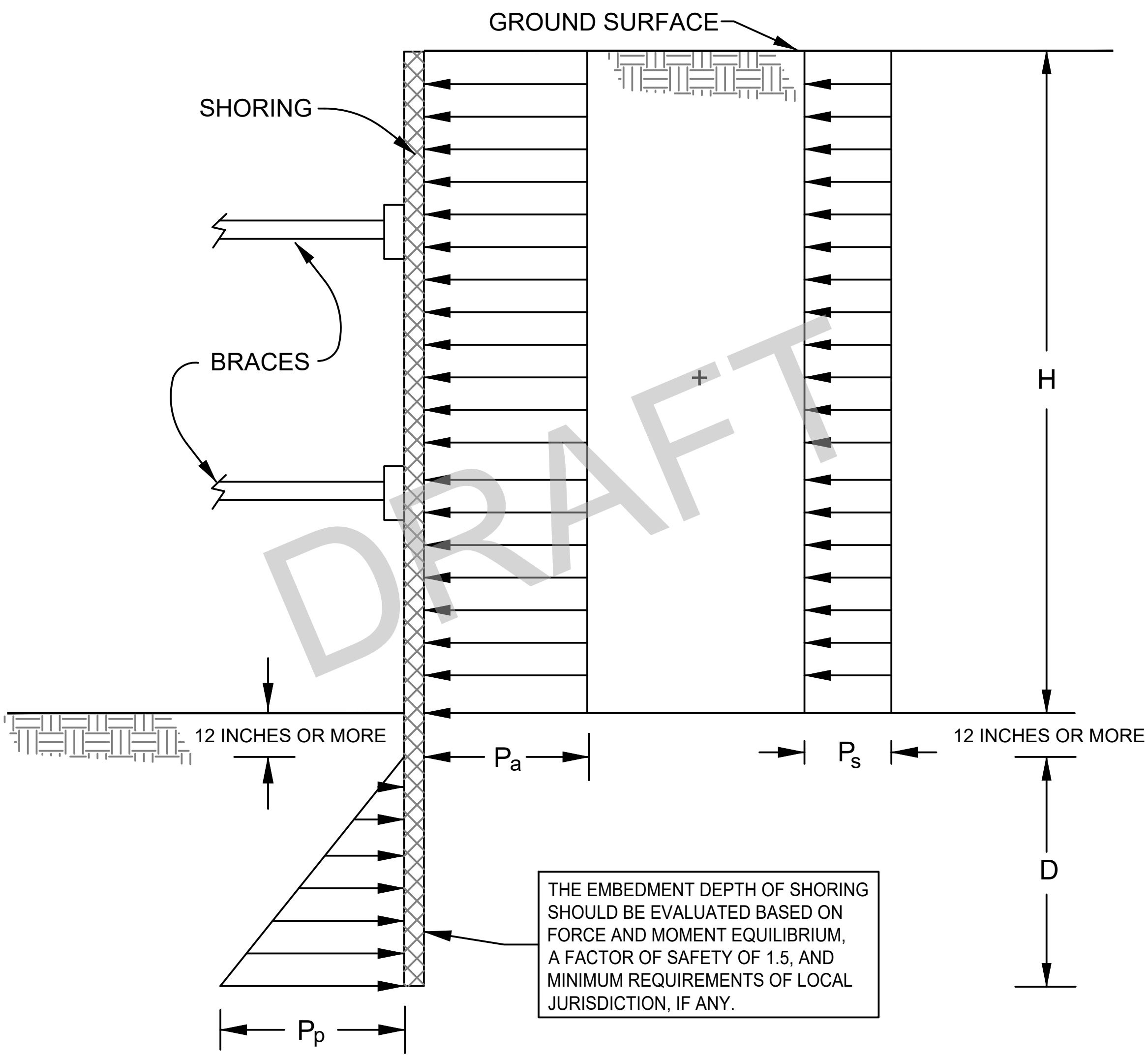
FIGURE 7

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

211621001 | 4/21



NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 22H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 400D 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

211621001_LEP-BE(GS).dwg 04/02/2021 JDP

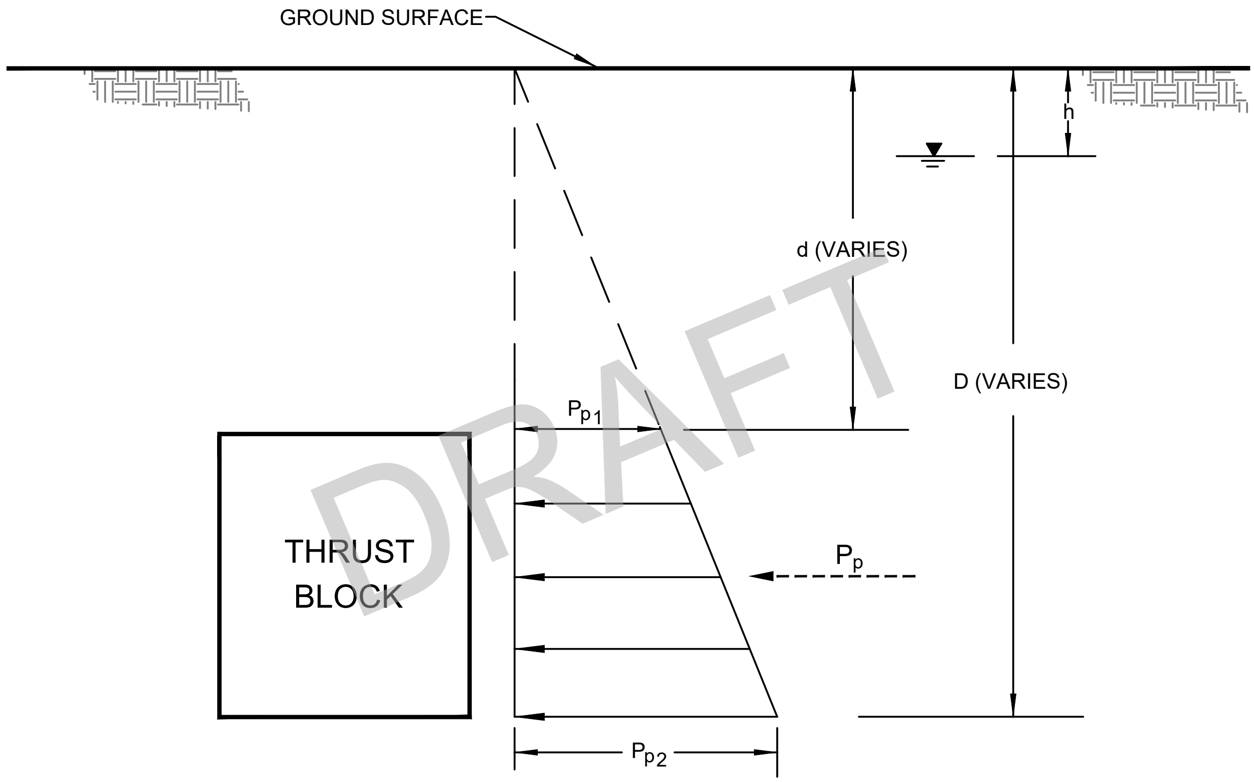
FIGURE 8

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

211621001 | 4/21



NOTES:

1. GROUNDWATER BELOW BLOCK
   $P_p = 200(D^2 - d^2)$ lb/ft
2. GROUNDWATER ABOVE BLOCK
   $P_p = 1.7(D - d)[124.8h + 57.6(D+d)]$ 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

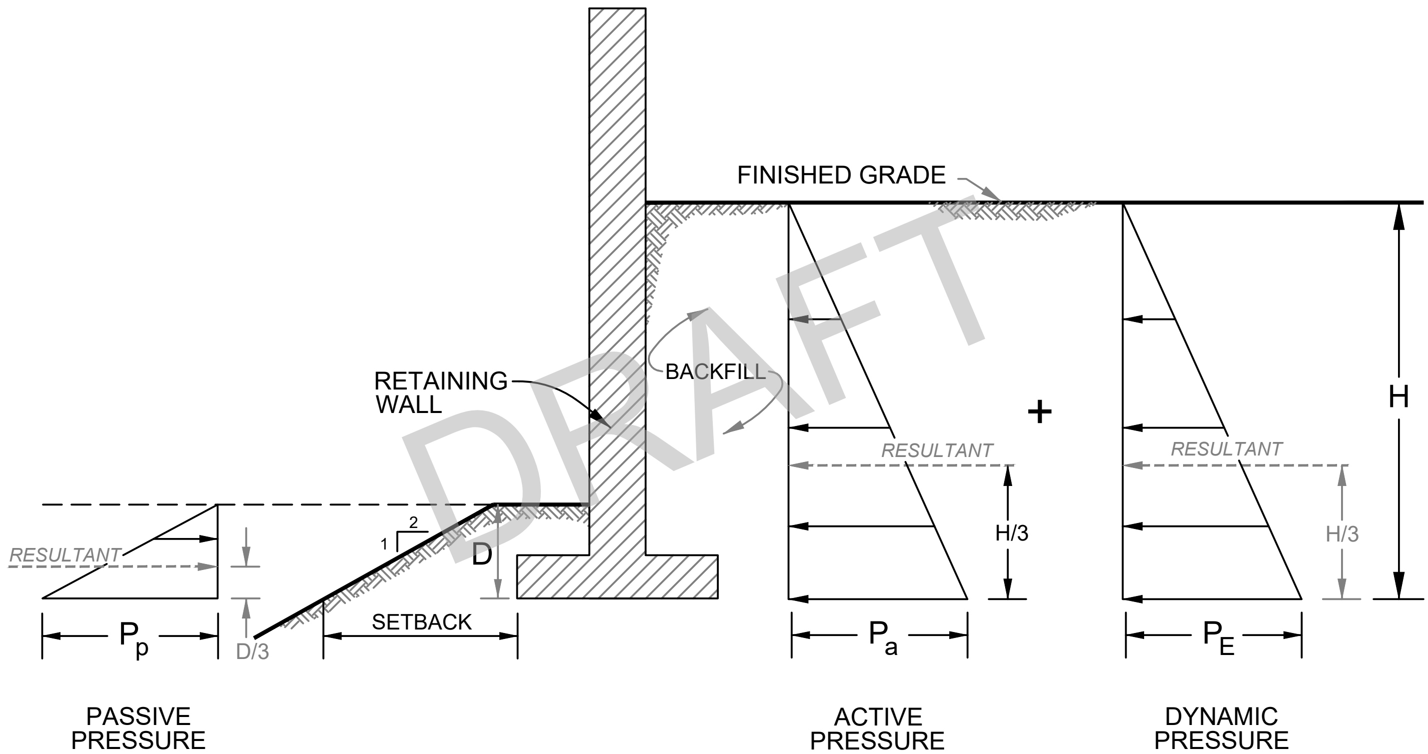
211621001_LEP-TB.dwg 04/02/2021 JDP

FIGURE 9

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. STRUCTURAL, GRANULAR BACKFILL MATERIALS AS SPECIFIED IN GREENBOOK SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A MAPPED DESIGN PEAK GROUND ACCELERATION OF 0.55 g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET
8. SETBACK SHOULD BE IN ACCORDANCE WITH FIGURE 1808.7.1 OF THE CBC (2016)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_a	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	35H	50H
P_E	20H	27H
P_p	Level Ground	2H:1V Descending Ground
	400D	150D

NOT TO SCALE

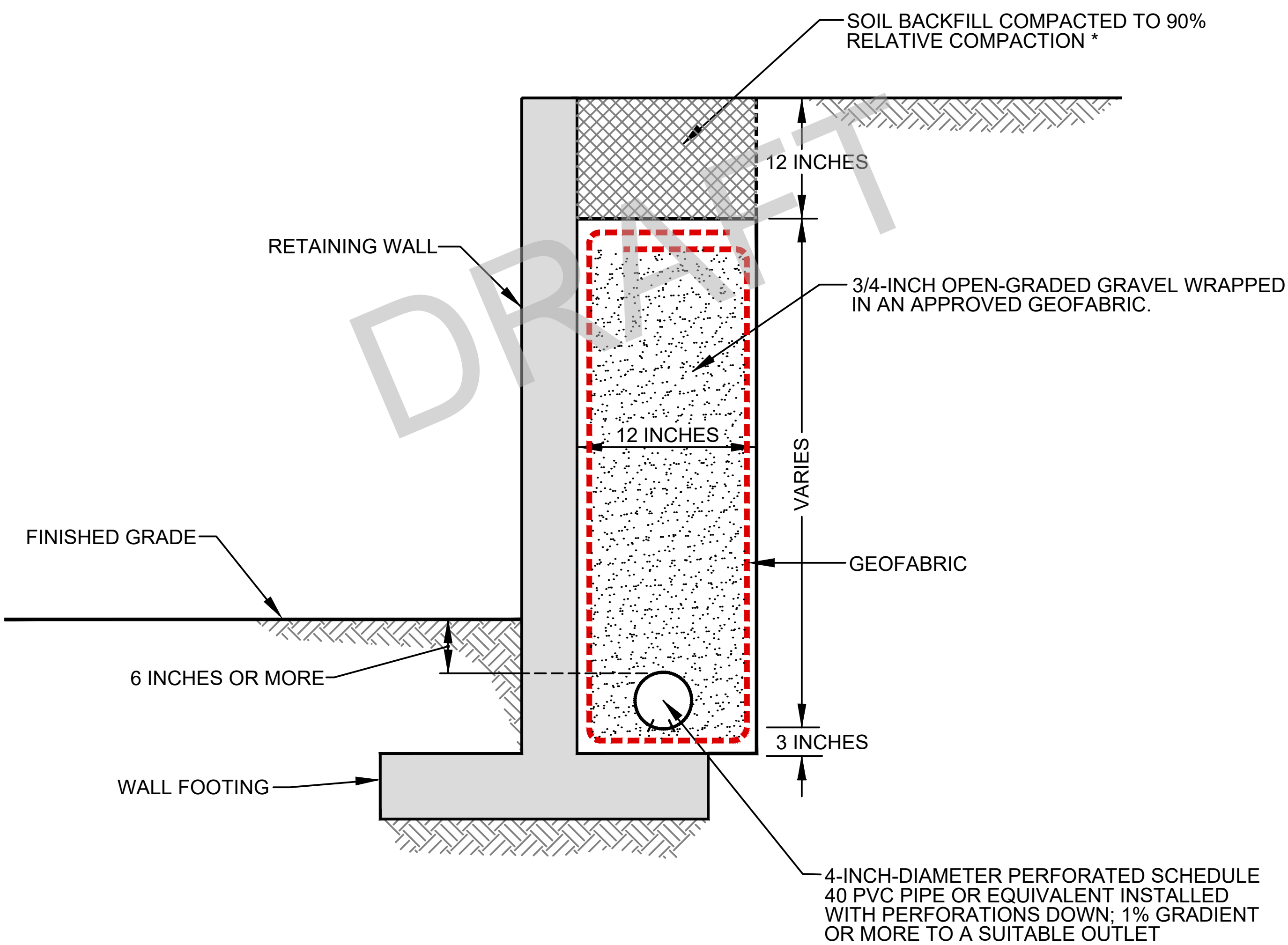
211621001_YRW1.dwg 04/02/2021 JDP

FIGURE 10

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



*BASED ON ASTM D1557

NOT TO SCALE

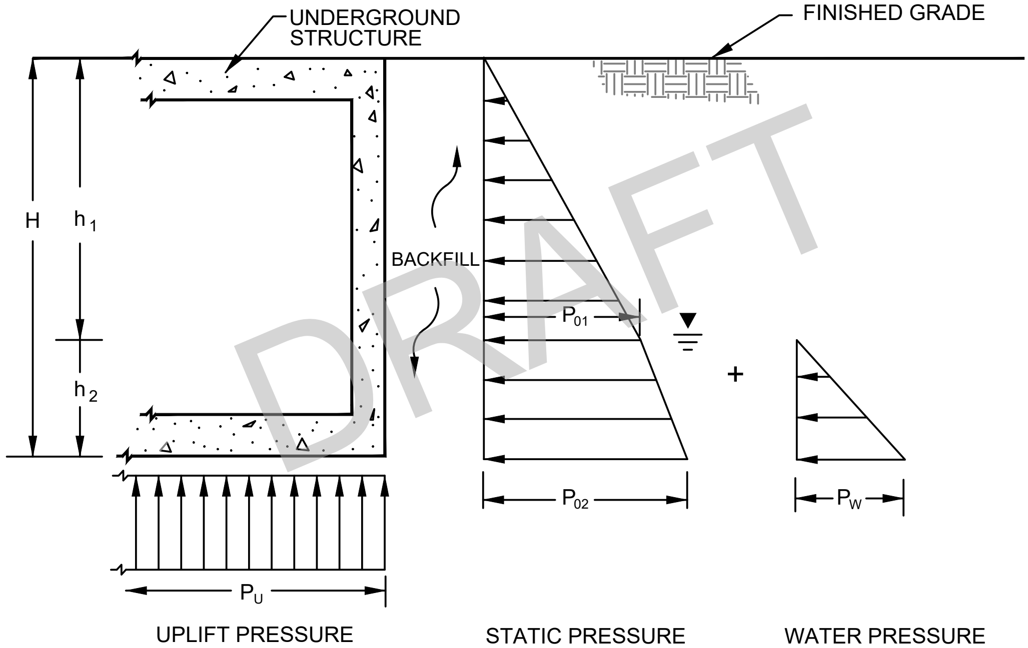
FIGURE 11

RETAINING WALL DRAINAGE DETAIL

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 I 4/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{01} AND P_{02}
   $P_{01} = 55h_1$ psf
   $P_{02} = 55h_1 + 25h_2$ 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. $\underline{\blacktriangledown}$ GROUNDWATER TABLE

NOT TO SCALE

211621001_LEP-US.dwg 04/02/2021 JDP

FIGURE 12

LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

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 methods.

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 falling freely 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 3.0 inches, was lined with 1-inch long, thin brass rings with inside diameters of approximately 2.4 inches. The sampler 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 sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

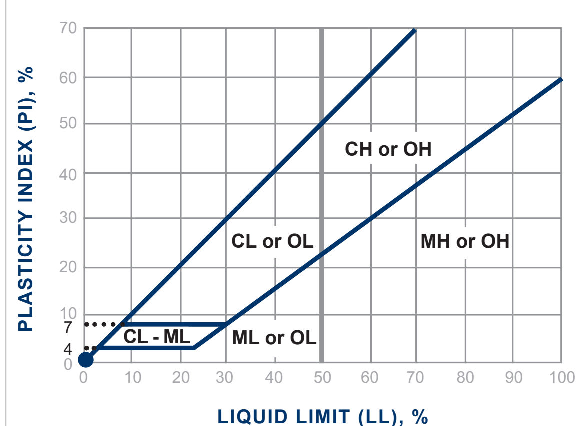
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: 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
Gravel	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
Sand	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
Sand	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

Consistency	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

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

USCS METHOD OF SOIL CLASSIFICATION

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								

DATE DRILLED	2/24/21	BORING NO.	B-1	
GROUND ELEVATION	947' ± (MSL)	SHEET	1	OF 1
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling Co)			
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"	
SAMPLED BY	KMB	LOGGED BY	KMB	REVIEWED BY JRS

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	Bulk						ASPHALT CONCRETE: Approximately 4 inches thick.
	Bulk					SM	FILL: Brown, moist, medium dense, silty SAND with gravel. Light brown; dense.
	Bulk / Driven	52	3.0	114.1		SP-SM	ALLUVIUM: Light brown to brown, moist, dense, poorly graded SAND with silt and gravel. Rig chatter on gravel and possible cobbles between approximately 6 and 8 feet.
	Bulk / Driven	32					
10	Bulk / Driven	43					Very dense.
	Driven	18	11.2	98.0		SM	Brown, moist, medium dense, silty SAND.
	Driven	10					Thin interbedded layers of sandy clay.
20	Driven	66				SP-SM	Light brown to brown, moist, very dense, poorly graded SAND with gravel and silt. Rig chatter on gravel between approximately 24 and 25 feet.
	Driven	73					
	Driven	16				SM	Brown, moist, medium dense, silty SAND.
30	Driven	35				SP-SM	Light brown to brown, moist, very dense, poorly graded SAND with silt; few to little gravel.
						SM	Brown, moist, very dense, silty SAND.
							Total Depth = 31.5 feet. Groundwater not encountered during drilling. Backfilled with bentonite-cement grout and patched with rapid-set concrete on 2/24/21. 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.
40							

DATE DRILLED	2/24/21	BORING NO.	B-2
GROUND ELEVATION	947' ± (MSL)	SHEET	1 OF 2
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling Co)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	KMB	LOGGED BY	KMB
REVIEWED BY	JRS		

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SP-SM	ASPHALT CONCRETE: Approximately 4 inches thick.
							FILL: Brown to dark brown, moist, dense, poorly graded SAND with gravel and silt.
		11				GP-GM	Brown, moist, medium dense, poorly graded GRAVEL with silt and sand. Rig chatter on gravel and possible cobbles between approximately 5 and 7 feet.
10		52	3.8	111.2			
		19				SM	Brown to reddish yellow, moist, medium dense, silty SAND with gravel.
		17				SP-SM	ALLUVIUM: Light brown, moist, medium dense, poorly graded SAND with silt.
20		59	10.8	116.9			Light brown to brown; dense.
		35					Very dense; thin interbedded layers of silty sand. Rig grinding on gravel between approximately 22 and 23 feet.
		22					Medium dense; thin interbedded layers of clayey sand.
30		31				SM	Brown, moist, medium dense, silty SAND; trace gravel.
		32					Very dense.
						SP-SM	Light brown to brown, moist, very dense, poorly graded SAND with silt.
							Total Depth = 33 feet. Groundwater not encountered during drilling. Backfilled with bentonite-cement grout and patched with rapid-set concrete on 2/24/21.
							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.
40							The ground elevation shown above is an estimation only. It is based on our interpretations

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/24/21 BORING NO. B-2 GROUND ELEVATION 947' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling Co) DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30" SAMPLED BY KMB LOGGED BY KMB REVIEWED BY JRS **DESCRIPTION/INTERPRETATION**
40								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.
50								
60								
70								
80								

DATE DRILLED	2/23/21	BORING NO.	B-3
GROUND ELEVATION	946' ± (MSL)	SHEET	1 OF 1
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling Co)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	KMB	LOGGED BY	KMB
REVIEWED BY	JRS		

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						GP-GM	ASPHALT CONCRETE: Approximately 5 inches thick.
							FILL: Brown, moist, medium dense, poorly graded GRAVEL with sand and silt; trace cobbles.
		10	9.6	96.9		SM	Brown, moist, loose, silty SAND with gravel.
		4					
10		36				GP-GM	Brown, moist, very dense, poorly graded GRAVEL with sand and silt. Rig grinding and chattering on gravel and possible cobbles between approximately 10 and 12 feet.
						SP-SM	ALLUVIUM: Light brown to reddish yellow, moist, medium dense, poorly graded SAND with silt; few to little gravel; oxidation staining.
		20	8.0	103.3			
		10					
20		18					Few gravel.
		85					Very dense; trace cobbles.
		42					
30		43					
							Total Depth = 31.5 feet. Groundwater not encountered during drilling. Backfilled with bentonite-cement grout on 2/23/21 and patched with rapid- set concrete on 2/24/21.
							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.
40							

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA
211621001 | 4/21

DATE DRILLED 2/23/21 & 2/24/21 BORING NO. B-4

GROUND ELEVATION 945' ± (MSL) SHEET 1 OF 2

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

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

SAMPLED BY KMB LOGGED BY KMB REVIEWED BY JRS

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SP	FILL: Dark brown, moist, loose, poorly graded SAND with gravel; trace cobbles; trace brick fragments; top 6 inches consist of planter top soil and mulch.
		20					
10		20	8.6	105.9			Rig chatter on gravel and possible cobbles between approximately 10 and 11.5 feet.
						SM	Dark brown, moist, loose, silty SAND with gravel.
		11					
		12				SP-SM	ALLUVIUM: Light brown, moist, medium dense, poorly graded SAND with silt; trace gravel.
						SM	Light brown, moist, medium dense, silty SAND with gravel.
20		25	4.5	109.1			
		13					
						SP-SM	Light brown, moist, dense, poorly graded SAND with silt and gravel.
		32					
						SW-SM	Light brown, moist, dense, well graded SAND with silt; few gravel.
30		46	5.8	114.4			
		31					
						SP-SM	Light brown, moist, dense, poorly graded SAND with silt.
		56					
		41					Very dense.
40							

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION

DATE DRILLED 2/23/21 & 2/24/21 BORING NO. B-4

GROUND ELEVATION 945' ± (MSL) SHEET 2 OF 2

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

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

SAMPLED BY KMB LOGGED BY KMB REVIEWED BY JRS

DEPTH (feet)	BLOWS/FOOT	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40	84	SP-SM	ALLUVIUM: (Continued) Light brown to brown, moist, very dense, poorly graded SAND with silt; trace gravel.
	74		
	64		Increase in gravel.
50	50/5"		
	93/9"		Trace coarse gravel.
			Total Depth = 51.8 feet. Groundwater not encountered during drilling. After drilling, a slotted PVC pipe was placed in the boring to avoid caving and the boring was left open until 2/25/21. Groundwater was not observed in the boring on 2/25/21. The PVC pipe was removed and the boring was backfilled with bentonite-cement grout and patched with rapid- set concrete on 2/25/21. 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.
60			
70			
80			

DATE DRILLED	2/25/21
BORING NO.	B-5
GROUND ELEVATION	943' ± (MSL)
SHEET	1 OF 1
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling Co)
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)
DROP	30"
SAMPLED BY	KMB
LOGGED BY	KMB
REVIEWED BY	JRS

DEPTH (feet)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0					ASPHALT CONCRETE: Approximately 7 inches thick.
				GP	AGGREGATE BASE: Brown, moist, medium dense, poorly graded GRAVEL; approximately 8 inches thick.
				GP-GM	FILL: Brown, moist, medium dense, poorly graded GRAVEL with silt.
	26	4.4	125.5		
	13			SM	Brown, moist, medium dense, silty SAND with gravel. Auger bouncing and grinding on gravel between approximately 5.5 and 7.5 feet.
10	34			SP-SM	ALLUVIUM: Light brown, moist, very dense, poorly graded SAND with silt and gravel. Rig chatter on gravel between approximately 10 and 12 feet.
					Rig grinding on gravel and possible cobbles between approximately 13.5 and 15 feet.
	35	5.7	108.9		Medium dense; interbedded with very thin layers of silty sand.
	13				
20	16			SM	Light brown to reddish yellow, moist, medium dense, silty SAND.
				SP-SM	Light brown, moist, dense, poorly graded SAND with silt; few gravel.
	70	6.4	124.7		
	40			SM	Light brown to reddish yellow, moist, very dense, silty SAND with gravel.
30	27				Dense; trace gravel.
					Total Depth = 31.5 feet. Groundwater not encountered during drilling. Backfilled with bentonite-cement grout and patched with rapid-set concrete on 2/25/21. 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.
40					

FIGURE A- 7

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

DATE DRILLED	2/23/21	BORING NO.	B-6
GROUND ELEVATION	943' ± (MSL)	SHEET	1 OF 2
METHOD OF DRILLING	8" Hollow-Stem Auger (Martini Drilling Co)		
DRIVE WEIGHT	140 lbs. (Auto. Trip Hammer)	DROP	30"
SAMPLED BY	KMB	LOGGED BY	KMB
REVIEWED BY	JRS		

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						ASPHALT CONCRETE: Approximately 6 inches thick.
	Bulk				GP	AGGREGATE BASE: Brown, moist, medium dense, poorly graded GRAVEL with sand; approximately 12 inches thick.
	Bulk				SM	FILL: Brown, moist, medium dense, silty SAND; trace gravel and cobbles.
	Bulk, Driven	8				
10	Bulk, Driven	32	3.4	115.5	GP-GM	Brown, moist, medium dense, poorly graded GRAVEL with silt and sand; broken rootlet. Rig chatter on gravel between approximately 10 and 11 feet.
	Bulk				SP-SM	ALLUVIUM: Brown, moist, very dense, poorly graded SAND with silt and gravel. Rig chatter on gravel and possible cobbles between approximately 13 and 14 feet.
	Driven	38				
						Rig grinding on gravel and possible cobbles between approximately 17 and 17.5 feet.
					SM	Light brown to brown, moist, medium dense, silty SAND; few to little gravel.
20	Driven	26	12.9	111.1		
	Driven	30				Dense. Rig chatter on gravel and possible cobbles between approximately 22 and 24 feet.
	Driven	45				Very dense; increase in gravel.
30	Driven	78	9.7	123.3		
	Driven	58				
					SM	Brown, moist, medium dense, silty SAND.
	Driven	16				
					SP-SM	Brown, moist, medium dense, poorly graded SAND with silt.
40						

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40		■	86				SP-SM	ALLUVIUM: (Continued) Brown, moist, very dense, poorly graded SAND with silt.
		◣	51					
		◣	74/11"					
50		■	95/10"					Trace gravel.
		◣	80					
								Total Depth = 53 feet. Groundwater not encountered during drilling. After drilling, slotted PVC pipe was placed in the boring to avoid caving and the boring was left open until 2/24/21. Groundwater was not observed in the boring on 2/24/21. The PVC pipe was removed and the boring was backfilled with bentonite-cement grout and patched with rapid set concrete on 2/24/21.
								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.
60								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.
70								
80								

DATE DRILLED 2/23/21 BORING NO. B-6

GROUND ELEVATION 943' ± (MSL) SHEET 2 OF 2

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

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

SAMPLED BY KMB LOGGED BY KMB REVIEWED BY JRS

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/25/21 BORING NO. B-7 GROUND ELEVATION 940' ± (MSL) SHEET 1 OF 2 METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling Co) DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30" SAMPLED BY KMB LOGGED BY KMB REVIEWED BY JRS **DESCRIPTION/INTERPRETATION**
0							GP SM	ASPHALT CONCRETE: Approximately 4.5 inches thick. AGGREGATE BASE: Brown, moist, dense, poorly graded GRAVEL; approximately 8 inches thick. FILL: Brown, moist, dense, silty SAND with gravel; trace cobbles in upper approximately 5 feet.
			29					
							GP-GM	ALLUVIUM: Brown, moist, dense, poorly graded GRAVEL with silt and sand. Auger grinding and bouncing on gravel between approximately 7 and 9 feet.
10			70	3.1	123.4			Light brown to brown.
			14				SM	Light brown to reddish yellow, moist, medium dense, silty SAND with gravel.
			18					
20							SP-SM	Light brown to brown, moist, dense, poorly graded SAND with silt; few to little gravel.
			63	5.4	117.3			
			46					Very dense.
			65					
30			56	5.9	115.7			Dense.
			63					Very dense.
			63					
40								

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA
211621001 | 4/21

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 2/25/21 BORING NO. B-7 GROUND ELEVATION 940' ± (MSL) SHEET 2 OF 2 METHOD OF DRILLING 8" Hollow-Stem Auger (Martini Drilling Co) DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30" SAMPLED BY KMB LOGGED BY KMB REVIEWED BY JRS **DESCRIPTION/INTERPRETATION**
40			50/4"				SP-SM	ALLUVIUM: (Continued) Brown, moist, very dense, poorly graded SAND with silt; trace gravel.
			62					
			50/6"					Few gravel. Rig chatter on gravel between approximately 46 and 48 feet.
50			50/3"					Total Depth = 50.2 feet. Groundwater not encountered during drilling. Backfilled with bentonite-cement grout and patched with rapid-set concrete on 2/25/21. 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.
60								
70								
80								

APPENDIX B

Laboratory Testing

APPENDIX B

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/excavations in Appendix A.

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

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 6913. The grain-size distribution curves are shown on Figures B-1 through B-4. 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-5.

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 classification in accordance with the USCS. The test results and classifications are shown on Figure B-6.

Consolidation Tests
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were 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 summarized on Figures B-7 and B-8.

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 the selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures B-9 through B-11.

Proctor Density Tests
The maximum dry density and optimum moisture content of selected representative soil samples were evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of these tests are summarized on Figures B-12 and B-13.

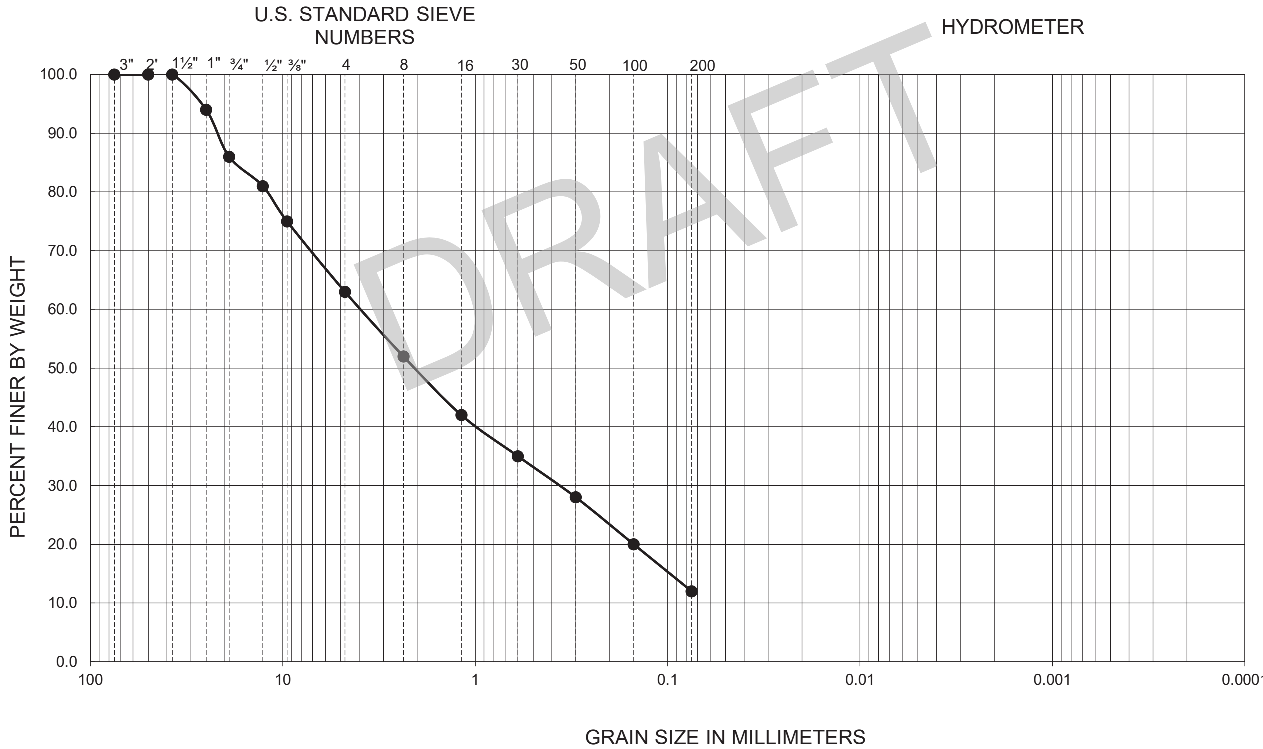
R-Value
The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and

expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-14.

Soil Corrosivity Tests

Soil pH and resistivity tests were performed on representative samples in general accordance with California Test (CT) 643**.** The soluble sulfate and chloride content of selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-15.

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	6.5-8.0	--	--	--	--	0.36	3.90	--	--	12	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-1

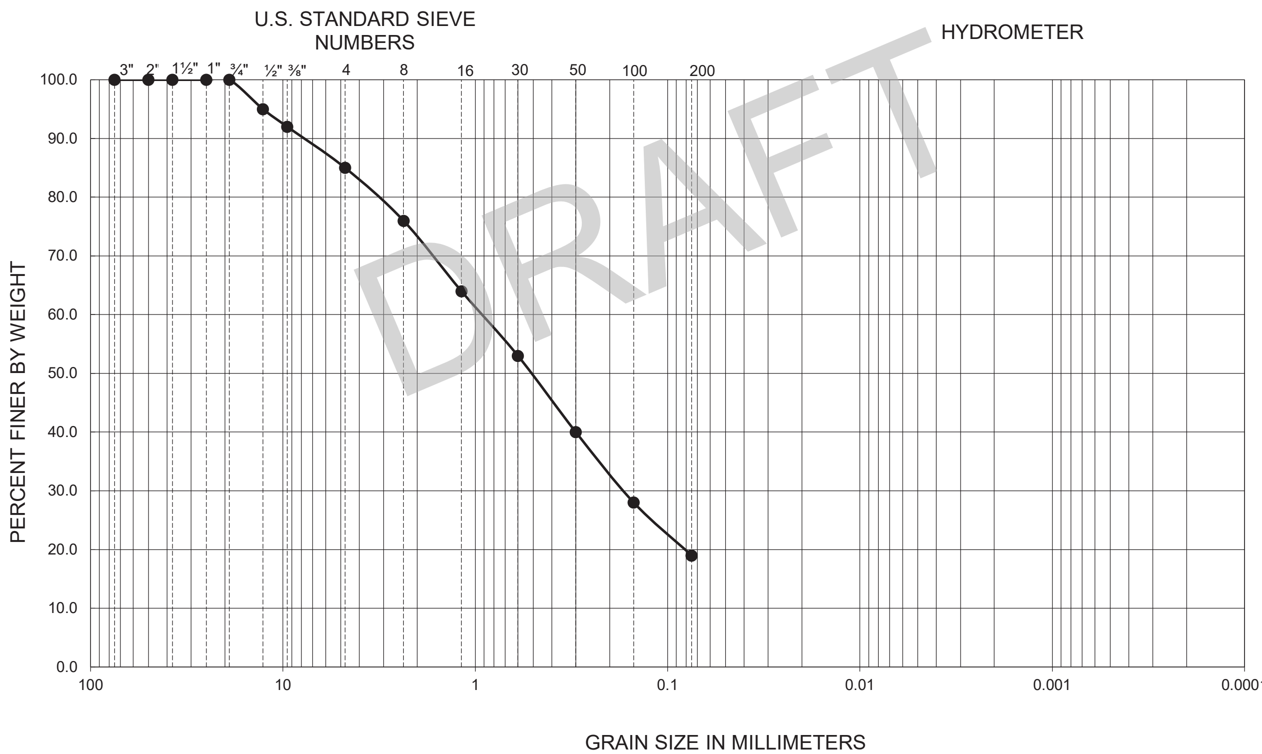
GRADATION TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

211621001 | 4/21

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-2	11.5-13.0	--	--	--	--	0.17	0.92	--	--	19	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

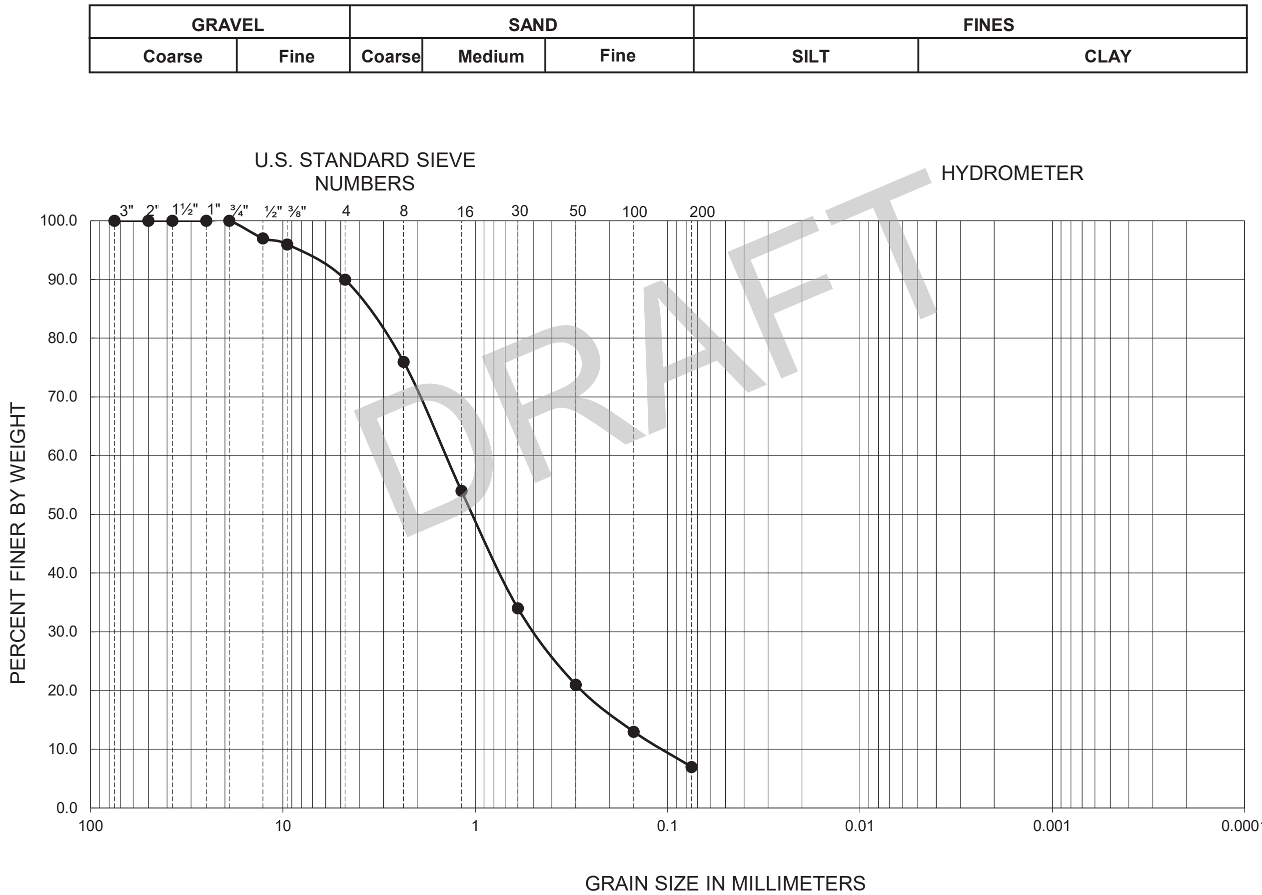
FIGURE B-2

GRADATION TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

211621001 | 4/21



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-4	30.0-31.5	--	--	--	0.104	0.50	1.45	13.9	1.7	7	SW-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-3

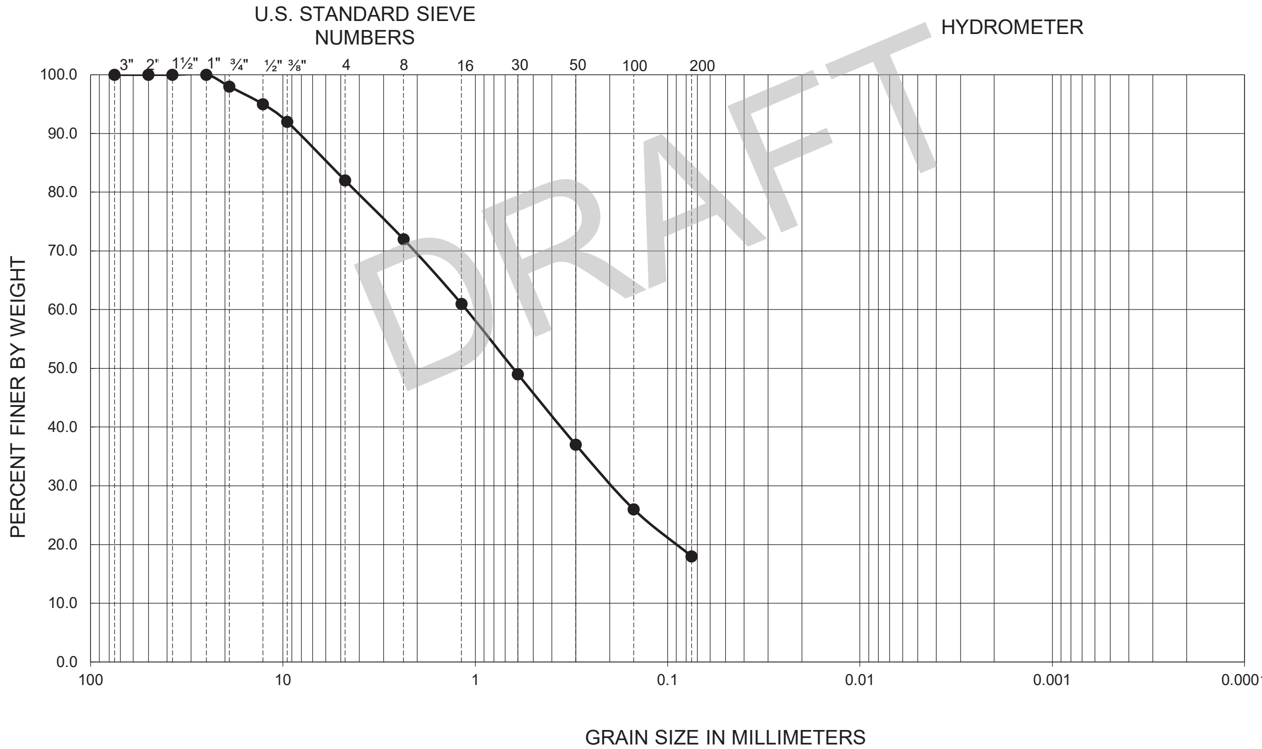
GRADATION TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

211621001 | 4/21

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-5	26.5-28.0	--	--	--	--	0.19	1.10	--	--	18	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-4

GRADATION TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

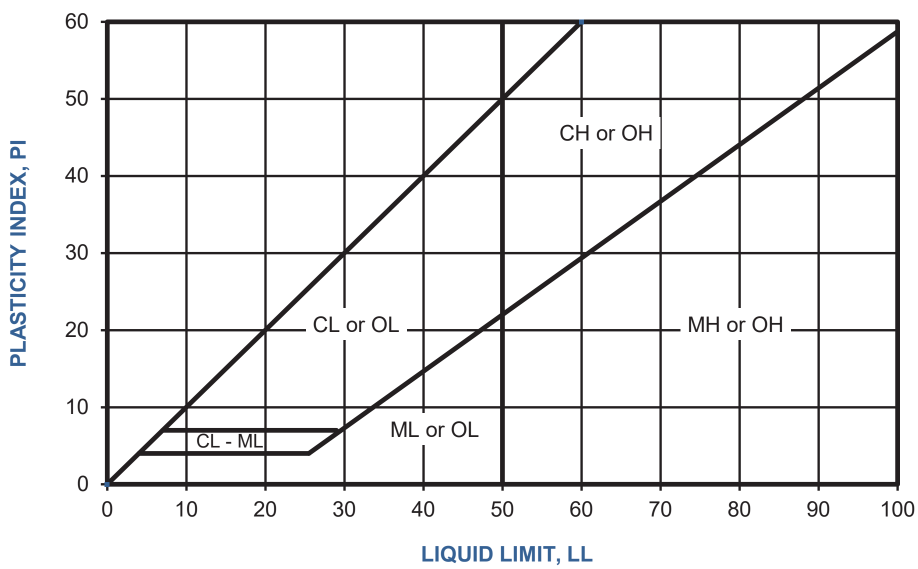
Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	0.0-5.0	SILTY SAND WITH GRAVEL	67	13	SM
B-2	5.0-6.5	POORLY GRADED GRAVEL WITH SILT AND SAND	52	7	GP-GM
B-3	6.5-8.0	SILTY SAND WITH GRAVEL	78	22	SM
B-3	15.0-16.5	POORLY GRADED SAND WITH SILT	89	11	SP-SM
B-4	11.5-13.0	SILTY SAND WITH GRAVEL	61	16	SM
B-4	20.0-21.5	SILTY SAND WITH GRAVEL	82	13	SM
B-5	6.5-8.0	SILTY SAND WITH GRAVEL	78	15	SM
B-5	15.0-16.5	POORLY GRADED SAND WITH SILT AND GRAVEL	80	7	SP-SM
B-6	10.0-11.5	POORLY GRADED GRAVEL WITH SILT AND SAND	50	9	GP-GM
B-6	20.0-21.5	SILTY SAND	86	20	SM
B-6	25.0-26.5	SILTY SAND WITH GRAVEL	82	19	SM
B-7	1.0-5.0	SILTY SAND WITH GRAVEL	67	21	SM
B-7	10.0-11.5	POORLY GRADED GRAVEL WITH SILT AND SAND	49	5	GP-GM
B-7	15.0-16.5	SILTY SAND WITH GRAVEL	84	21	SM
B-7	25.0-26.5	POORLY GRADED SAND WITH SILT	86	12	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-5	6.5-8.0	-	-	NP	ML	SM
■	B-6	10.0-11.5	-	-	NP	ML	GP-GM

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

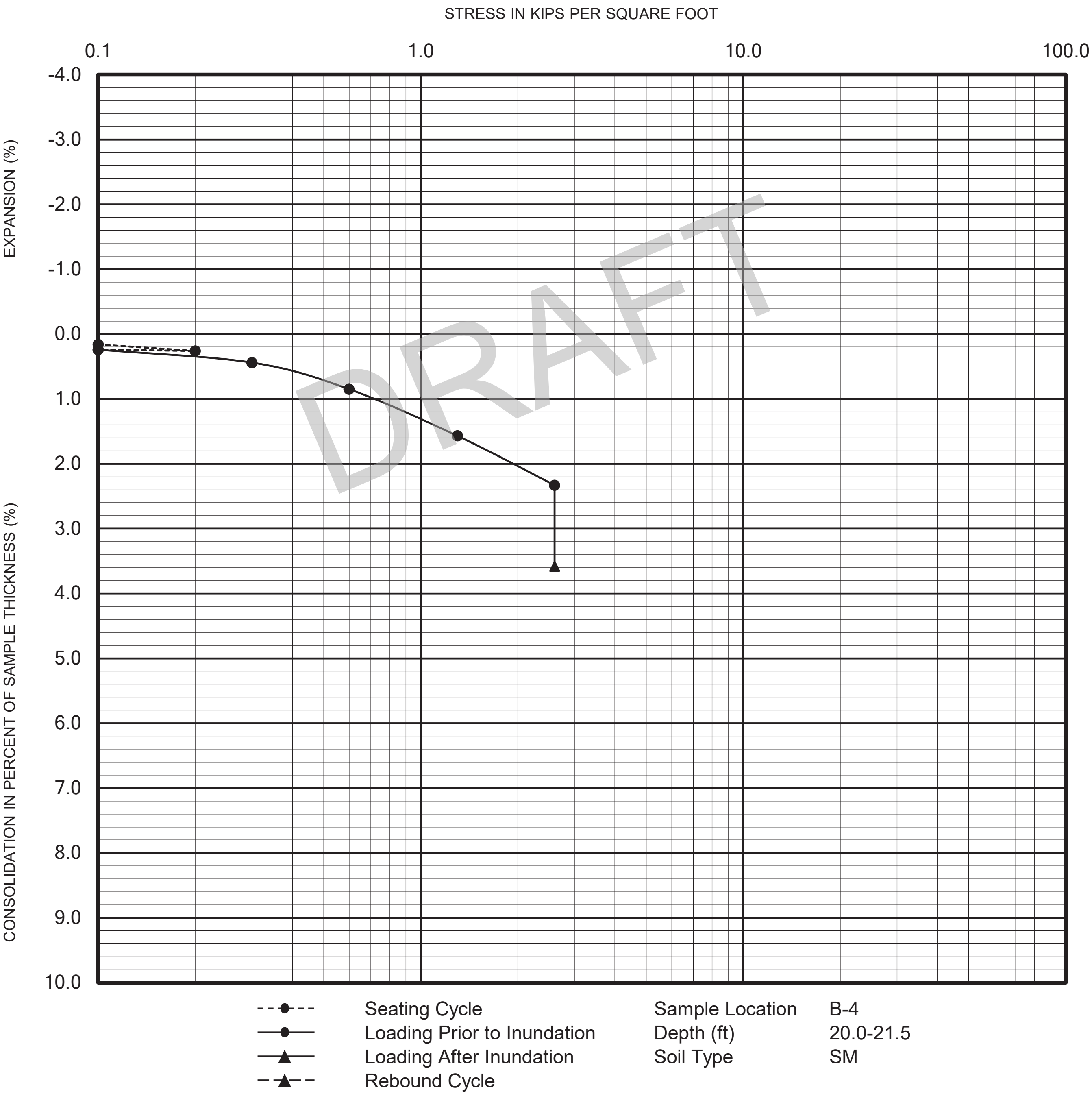
FIGURE B-6

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

ATTERBERG LIMITS TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21



Sample Location	B-4
Depth (ft)	20.0-21.5
Soil Type	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

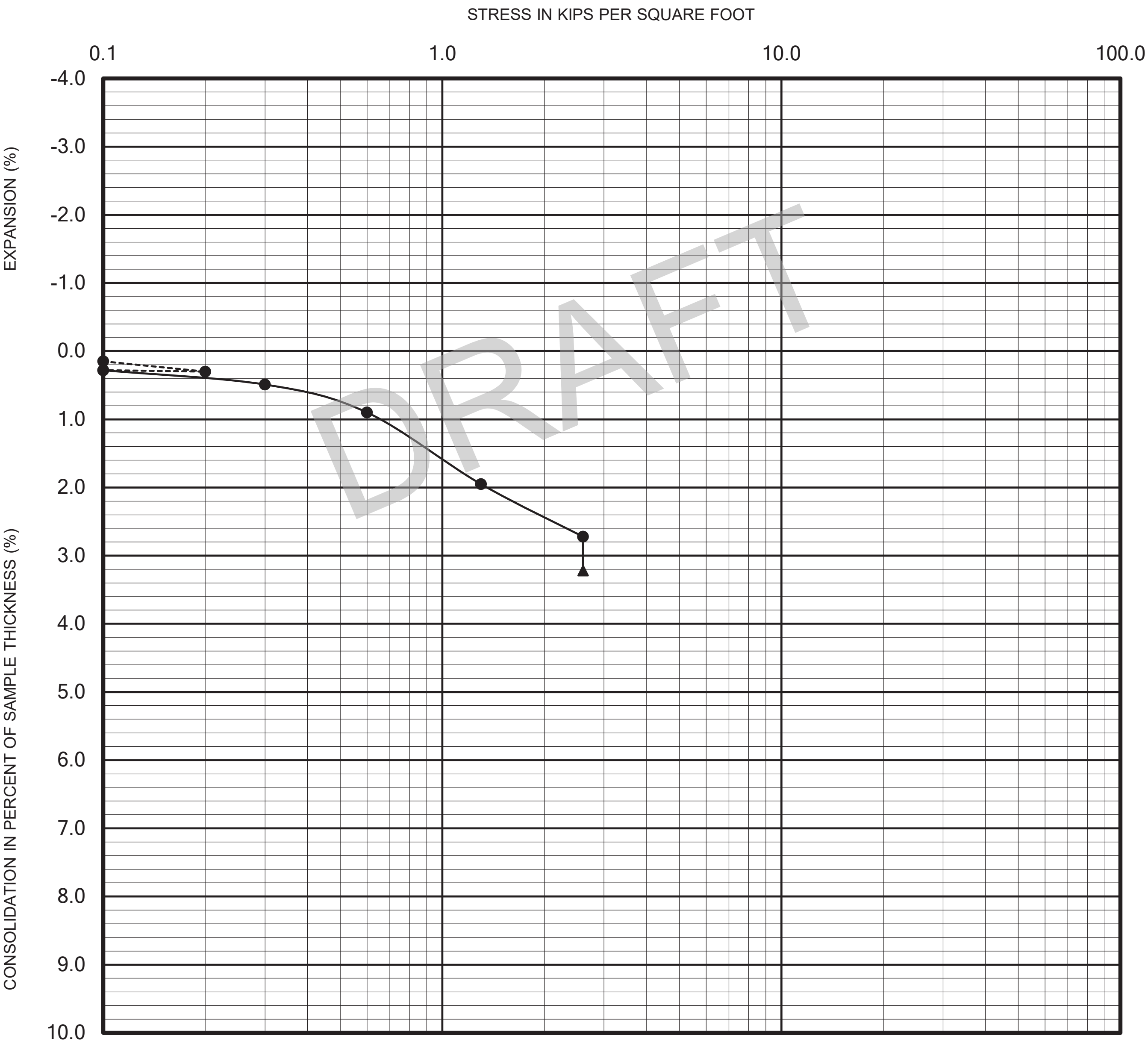
FIGURE B-7

CONSOLIDATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21



Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Sample Location	B-6
Depth (ft)	20.0-21.5
Soil Type	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

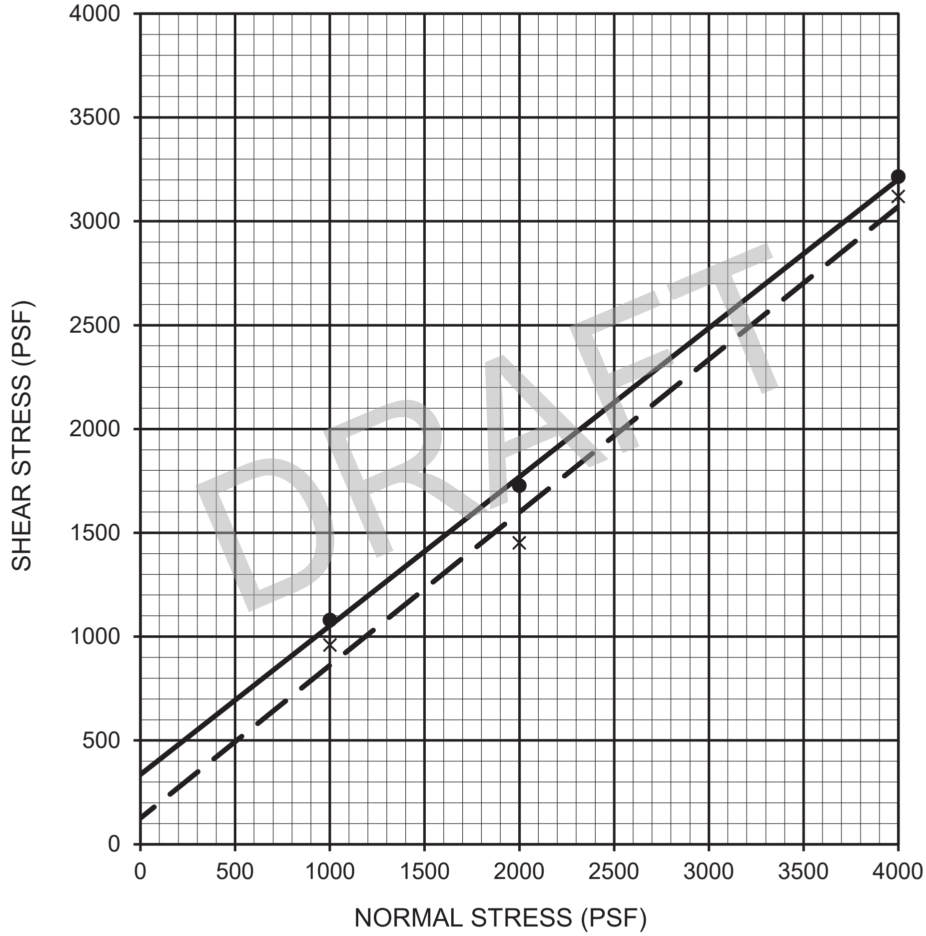
FIGURE B-8

CONSOLIDATION TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SILTY SAND WITH GRAVEL	—●—	B-4	20.0-21.5	Peak	336	36	SM
SILTY SAND WITH GRAVEL	– – X – –	B-4	20.0-21.5	Ultimate	126	36	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

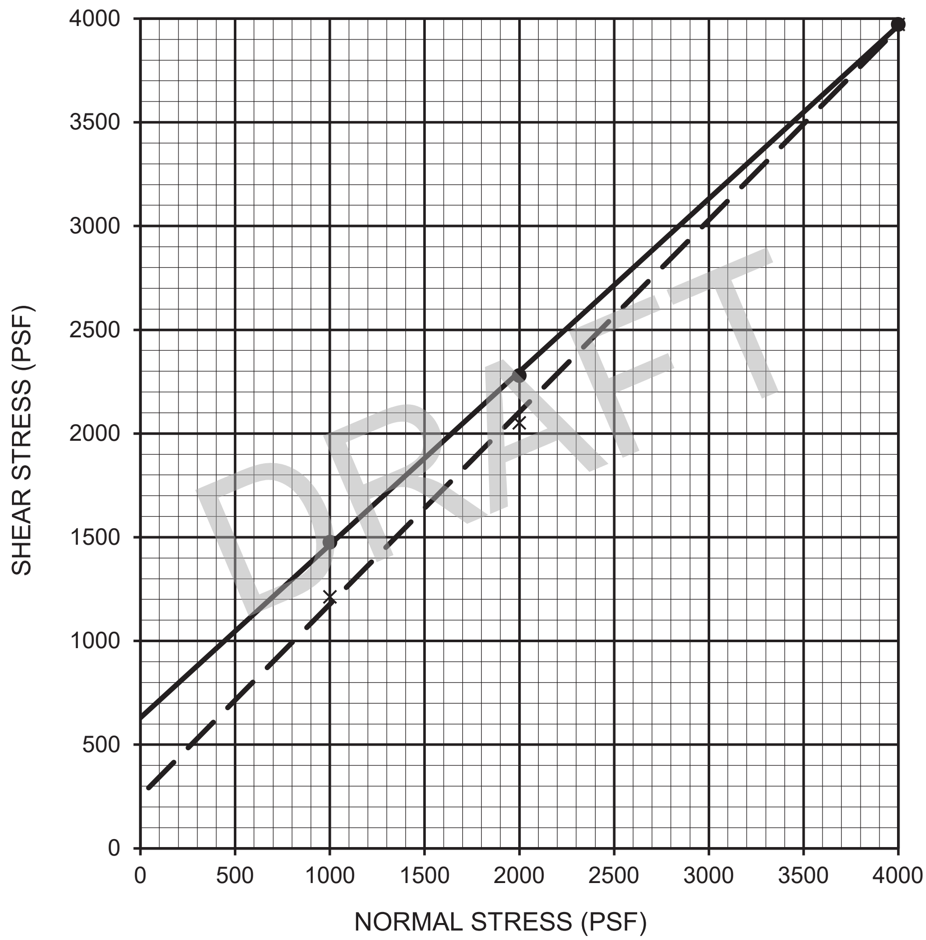
FIGURE B-9

DIRECT SHEAR TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

211621001 | 4/21



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SILTY SAND	——●——	B-6	20.0-21.5	Peak	630	40	SM
SILTY SAND	– – X – –	B-6	20.0-21.5	Ultimate	252	43	SM

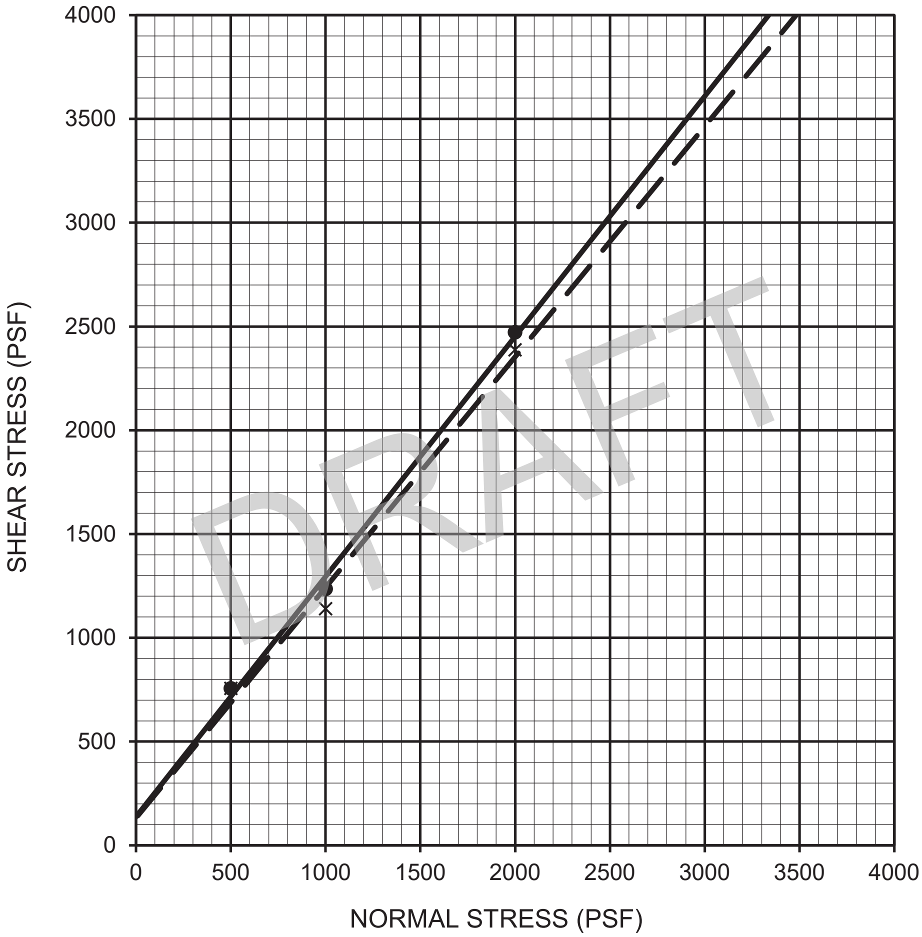
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-10

DIRECT SHEAR TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
POORLY GRADED GRAVEL WITH SILT AND SAND	—●—	B-7	10.0-11.5	Peak	138	49	GP-GM
POORLY GRADED GRAVEL WITH SILT AND SAND	- - x - -	B-7	10.0-11.5	Ultimate	132	48	GP-GM

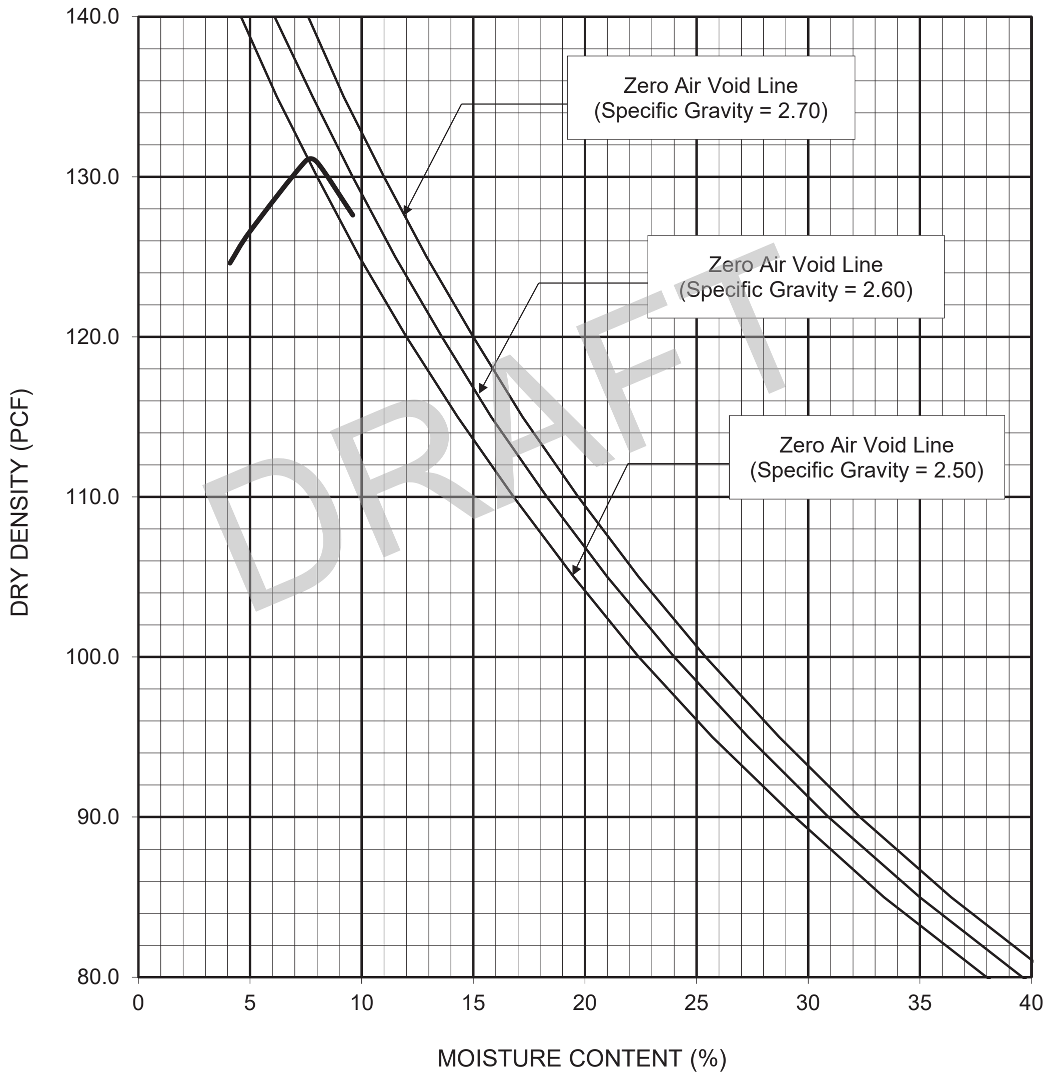
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-11

DIRECT SHEAR TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-4	5.0-10.0	DARK BROWN SILTY SAND WITH GRAVEL	131.0	7.5
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718)			133.0	7

PERFORMED IN GENERAL ACCORDANCE WITH ☑ ASTM D 1557 ☐ ASTM D 698 METHOD ☐ A ☐ B ☑ C

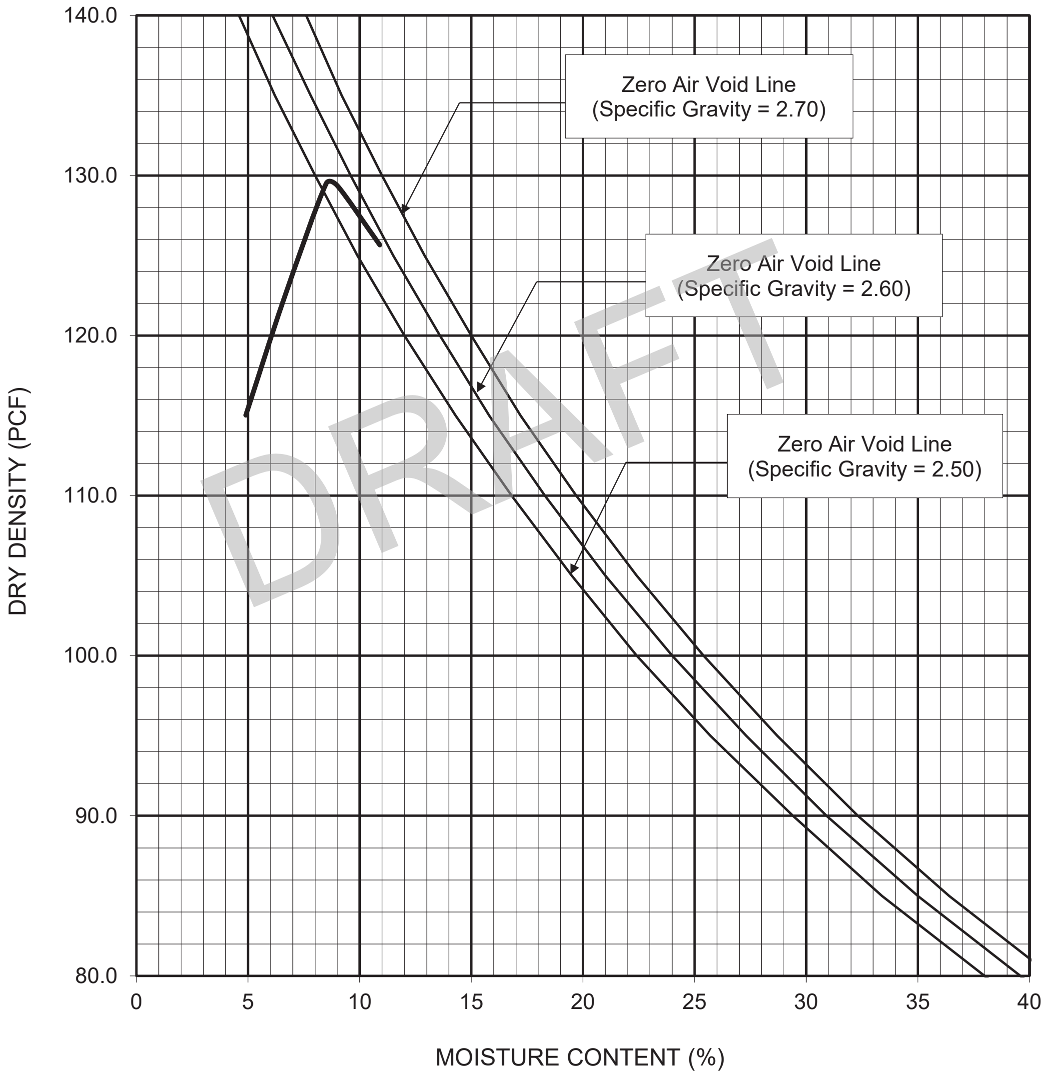
FIGURE B-12

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PROCTOR DENSITY TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21



Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-6	5.0-10.0	DARK BROWN SILTY SAND WITH GRAVEL	129.5	8.5
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718)			N/A	N/A

PERFORMED IN GENERAL ACCORDANCE WITH ☑ ASTM D 1557 ☐ ASTM D 698 METHOD ☐ A ☐ B ☑ C

FIGURE B-13

PROCTOR DENSITY TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B.2	0.0 - 5.0	SM	76

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-14

R-VALUE TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-4	15.0-16.5	6.7	1,970	60	0.006	175
B-6	5.0-6.5	6.4	2,645	60	0.006	140

[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-15

CORROSIVITY TEST RESULTS

SUNSET RESERVOIR REPLACEMENT PROJECT
PASADENA, CALIFORNIA

211621001 | 4/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

APPENDIX C

Summary of Geotechnical Parameters

The following geotechnical parameters are requested for the tank design:

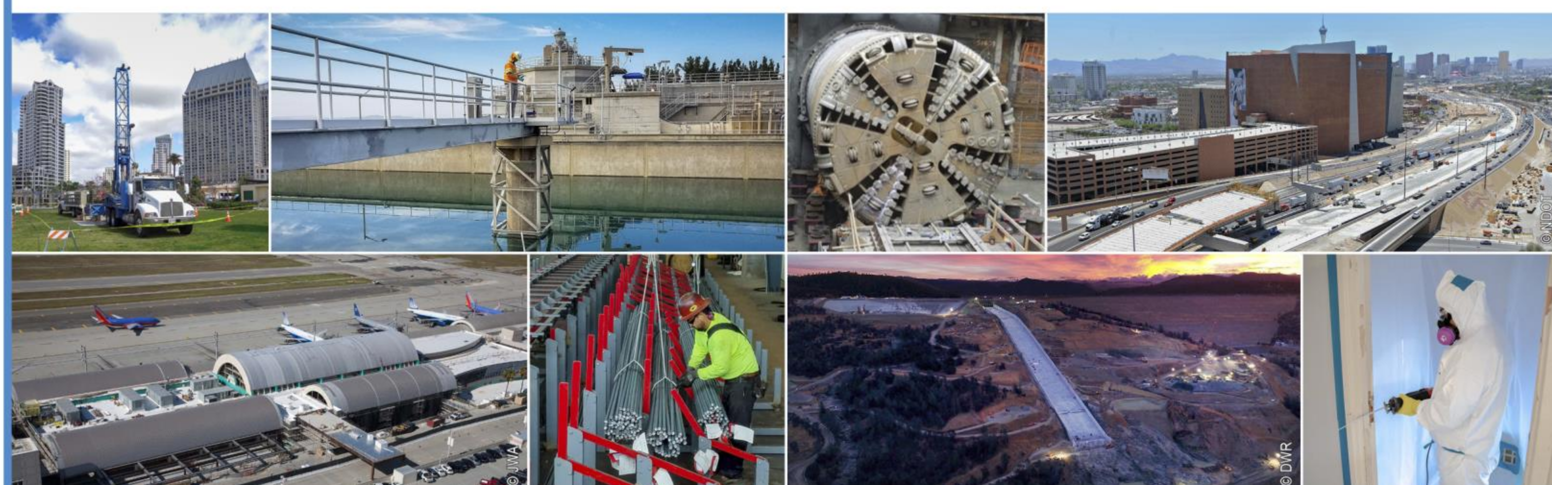
Backfill Soil Information (If tank is to be backfilled)	
Equivalent Liquid At-Rest Pressure (PCF)	55
Backfill Pressure Increase on Wall Under Seismic Excitation (PCF)	20
Equivalent Liquid Active Earth Pressure (PCF)	35
Equivalent Liquid Passive Earth Pressure (PCF)	400
Backfill Soil Density (PCF)	120
Downward Drag Coefficient of Backfill on Wall	N/A
Vehicle Load on Backfill	-

NOTE: Backfill Shall NOT Contain Sulfides or Expansive Material.

Soils Information	
Gross Soil Bearing Capacity, Including Backfill Soil and Liquid Loads (PSF)	5,000
Anticipated Total Settlement of Tank (Inches)	0.5
Anticipated Differential Settlement Across Tank Radius (Inches)	0.25
Maximum Groundwater Elevation from Surface (FT)	approx. 200 ft below ground surface
Coefficient of Friction Between Soil and Concrete Slab	0.40
Potential Vertical Rise, if plastic clays are present (Inches)	N/A

NOTE: Subgrade Shall NOT Contain Sulfides or Expansive Material.

Seismic Design Information	
Seismic design shall be based on the applicable sections of AWWA D110-13 and IBC-2012.	
AWWA D110-13: 1% Probability of Exceedance in 50 Years	
Importance Factor, I	Provided by Structural Engineer
Impulsive Structural Coefficient, R_i	Provided by Structural Engineer
Convective Structural Coefficient, R_c	Provided by Structural Engineer
IBC – 2012: 1% Probability of Exceedance in 50 Years (2019 CBC)	
Seismic Design Category	Provided by Structural Engineer
Mapped MCE_R, 5% Damped, Spectral Response Acceleration Parameter at Short Periods(S_S)(% g)	2.041
Mapped MCE_R, 5% Damped, Spectral Response Acceleration Parameter at a Period of 1 sec (S_1)(% g)	0.750
Design, 5% Damped, Spectral Response Acceleration Parameter at Short Period (S_{DS})(% g)	1.382
Design, 5% Damped, Spectral Response Acceleration Parameter at a Period of 1 sec (S_{D1})(% g)	1.000
Soil Site Class	D
Importance Factor, I_e	Provided by Structural Engineer
Response Modification Factor, R_i	Provided by Structural Engineer
Long Period Transition Period, T_L (Sec)	8.0



475 Goddard, Suite 200 | Irvine, California 92618 | p. 949.753.7070

ARIZONA | CALIFORNIA | COLORADO | NEVADA | TEXAS | UTAH

ninyoandmoore.com

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants