APPENDIX E

SANDHILL BASIN IMPROVEMENT PROJECT
PROJECT NO.: SW 23-01

SANDHILL BASIN
GEOTECHNICAL EVALUATION

Geotechnical Evaluation

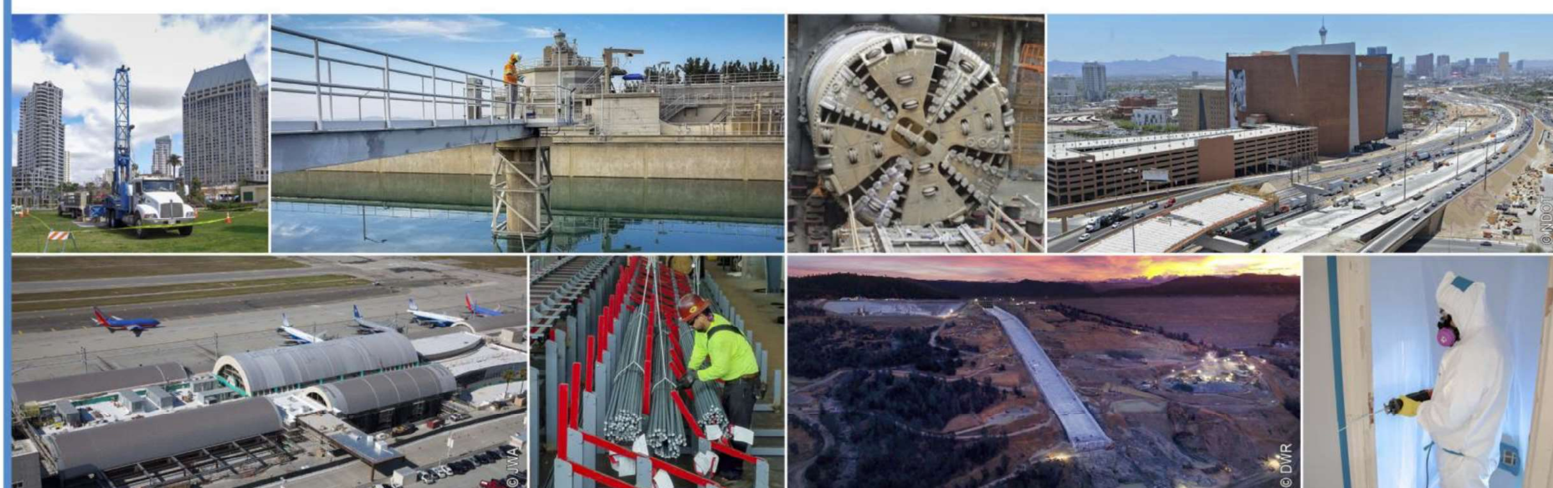
Sandhill Basin Improvement Project

El Segundo, California

City of El Segundo

350 Main Street | El Segundo, California 90245

September 8, 2023 | Project No. 211922002



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

Geotechnical Evaluation

Sandhill Basin Improvement Project

El Segundo, California

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350 Main Street | El Segundo, California 90245

September 8, 2023 | Project No. 211922002

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1 INTRODUCTION

In accordance with your request and authorization, we have prepared this geotechnical evaluation for the proposed Sandhill Basin Improvement Project at the City of El Segundo's Sandhill Basin in El Segundo, California, (Figure 1). The purpose of this study was to evaluate the soil and geologic conditions at the site in order to develop geotechnical design and construction recommendations for the improvements of the basin. This report presents our findings, conclusions, and recommendations for the project.

2 SCOPE OF SERVICES

The scope of our geotechnical services included the following:

- Project coordination, planning, and scheduling for the subsurface exploration.
- Review of readily available background information, including in-house published geotechnical literature and geologic maps, fault and seismic hazard maps, topographic maps, and stereoscopic aerial photographs.
- Acquisition of a boring permit from the County of Los Angeles, Department of Environmental Health permit for drilling to depths greater than 10 feet below the ground surface.
- Geotechnical site reconnaissance to observe the general site conditions, mark the boring locations, and coordinate with Underground Service Alert for utility clearance.
- Subsurface exploration consisting of drilling, logging, and sampling of three small-diameter, hollow-stem auger borings to depths of up to approximately 21.5 feet below the existing ground surface. The borings were logged by a representative of our firm and relatively undisturbed and bulk soil samples were collected at selected intervals for laboratory testing.
- Laboratory testing of selected, representative soil samples to evaluate in-situ moisture content and dry density, gradation, percentage of soil particles finer than the No. 200 sieve, consolidation/collapse potential, direct shear strength, Proctor density, and corrosivity.
- Compilation and geotechnical analyses of the information obtained from our background review, subsurface evaluation, and laboratory data.
- Preparation of this geotechnical report presenting our findings, conclusions, and geotechnical recommendations pertaining to this project.

3 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The Sandhill Basin is an earthen basin located approximately 450 feet west of Hillcrest Street, between West Maple Avenue and West Oak Avenue, in El Segundo, California (Figure 1). The basin is bordered on the east by single-family residential properties, on the south by an undeveloped area, on the west by an asphalt-concrete lined access road, and on the north by a gravel access road and West Maple Avenue. The access roads and areas to the south, west, and north of the basin are part of the City of Los Angeles Department of Water and Power easement. The basin's embankment slopes vary in height from approximately 10 to 15 feet and vary in slope ratio from approximately 4:1 to 2:1 (horizontal to vertical). An asphalt concrete maintenance road located on the north side of the basin provides access to the bottom of the basin. Portions of the basin are covered with dense vegetation, and the basin is surrounded by a chain link fence. A deteriorated asphalt concrete spillway is located on the southeast side of the basin.

Based on our discussions with you, we understand the basin undergoes significant erosion during rain events and the existing asphalt concrete spillway at the southeast corner of the basin is severely damaged and in need of repair. We understand the city plans to demolish the existing damaged spillway and replace it with a new concrete spillway.

4 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface exploration consisted of the drilling, logging, and sampling of three small-diameter borings to depths of up to approximately 21.5 feet below the ground surface. The borings were drilled using a limited-access track-mounted drill rig utilizing hollow-stem augers. The borings were drilled to evaluate the subsurface conditions at the site and a representative from Ninyo & Moore logged the borings and obtained bulk and relatively undisturbed soil samples at selected depths. The approximate locations of the borings are presented on Figure 2. The boring logs are presented in Appendix A. The borings were backfilled with cement-bentonite grout in accordance with our boring permit.

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

5 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geology

The subject site is located in the Los Angeles Basin, which is situated at the northwest end of the Peninsular Ranges geomorphic provinces of southern California (Norris and Webb, 1990). The Los Angeles Basin has been divided into four structural blocks, which are generally bounded by prominent northwest-trending fault systems: the northwestern, southwestern, central, and northeastern blocks. The site is located in the southwestern block, which is bounded by the Newport-Inglewood fault to the northeast, the Palos Verdes Hills fault to the southwest, and the

Santa Monica-Hollywood-Raymond fault system to the northwest. The block is underlain by up to approximately 20,500 feet of Miocene to Pleistocene-age marine sedimentary rock over basement rock consisting of the Mesozoic age Catalina Schist. Variable thicknesses of late Pleistocene to Holocene-age alluvial deposits associated with the ancestral Los Angeles and San Gabriel Rivers generally overlie the sedimentary rock.

Regional geologic maps indicate that the project site is underlain by late to middle-Pleistocene age older eolian deposits generally consisting of dense to very dense, poorly graded fine to coarse grained sand and silty sand (Saucedo et. al., 2016).

5.2 Site Geology

Eolian (aerially deposited sand dune) deposits were encountered during our subsurface exploration to the total depth explored of approximately 21.5 feet. The eolian deposits generally consisted of very friable, moist, medium dense to very dense, poorly graded sand, poorly graded sand with silt, and silty sand. More detailed descriptions of the subsurface materials encountered during our exploration are presented on the boring logs in Appendix A.

5.3 Groundwater

Groundwater was not encountered in our exploratory borings during drilling to the total depth explored of approximately 21.5 feet. Groundwater monitoring well data from the State of California Department of Water Resources Geotracker (2023) website indicates groundwater was measured at depths ranging from approximately 80 to 100 feet below the ground surface in groundwater monitoring wells located approximately 0.9 mile north of the site. It should be noted that fluctuations in the level of groundwater at the project site may occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors which may not have been evident at the time of our evaluation.

6 FLOOD HAZARDS

Based on our review of flood insurance rate maps for the project area (Federal Emergency Management Agency [FEMA], 2021), the project site is not located in the 100-year Flood Hazard Zone. The site is located within Zone X, an area of minimal flood hazard.

7 FAULTING AND SEISMICITY

The subject site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone) (California Geological Survey [CGS], 2018). However, the project site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered significant during the design life of the project. The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the CGS, active faults are faults that have ruptured within the 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 faults in the site vicinity and their geographic relationship to the site are shown on Figure 4. The nearest active fault is the Palos Verdes fault located approximately 3.8 miles southwest of the site (USGS, 2008).

The principal seismic hazards that may affect the project improvements include surface fault rupture, ground motion, and liquefaction. A brief description of these hazards and the potential for their occurrences are discussed below.

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 ground 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 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 2022 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. Based on our review of CGS's shear wave velocity map, the average shear wave velocity in the upper 30 meters (100 feet) of the subsurface profile (V_{S30}) is estimated to be approximately 387 meters per second (1,268 feet per second) (CGS, 2015). In accordance with Chapter 20 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures, the site classification is Site Class C.

In accordance with ASCE 7-16, the mapped MCE_R ground motion response accelerations were determined using the 2023 Applied Technology Council seismic design tool (web-based). The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits. Spectral response acceleration parameters, consistent with the 2022 CBC, are provided in Section 9.2 for the evaluation of seismic loads on buildings and other structures.

7.3 Liquefaction

Liquefaction is the phenomenon in which loosely deposited granular soils and cohesionless fine-grained soils located below the water table undergo rapid loss of shear strength due to excess pore pressure generation when subjected to strong earthquake-induced ground shaking. Sufficient ground shaking duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure. This 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 ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

Based on our review of the State of California Seismic Hazard Zones map, the site is not located in an area mapped as being potentially susceptible to liquefaction (California Department of Conservation, Division of Mines and Geology, 1999). Groundwater was not encountered in our exploratory borings during drilling to the total depth explored of approximately 21.5 feet. Nearby monitoring well data indicates groundwater was measured at depths ranging from approximately 80 to 100 feet below the ground surface. Based on the depth to groundwater and the relatively dense materials encountered in our borings, 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.

8 CONCLUSIONS

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

- Based on our exploratory borings, the site is generally underlain by eolian deposits to the total depth explored of approximately 21.5 feet. The eolian deposits generally consisted of moist, medium dense to very dense, poorly graded sand, poorly graded sand with silt, and silty sand.
- In general, excavations in the existing eolian deposits should be feasible with earthmoving equipment in good working condition.

- We anticipate that the on-site soils should be suitable for use as compacted fill following moisture-conditioning, provided they are free of trash, debris, roots, vegetation, deleterious materials, and cobbles or hard lumps of materials in excess of 4 inches in diameter.
- The eolian deposits at the site have little to no cohesion and will be subject to caving. These soils should be considered Type C soils in accordance with Occupational Safety and Health Administration (OSHA) soil classifications.
- Groundwater was not encountered in our exploratory borings during drilling to the total depth explored of approximately 21.5 feet. Fluctuations in the groundwater level may occur as a result of variations in seasonal precipitation, irrigation practices, groundwater pumping and other factors.
- The project area is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps and aerial photographs, there are no known active faults that underlie the site. The potential for surface fault rupture at the site is considered low.
- The site is not located within a mapped Seismic Hazards Zone considered susceptible to liquefaction. Based on the depth to groundwater and the relatively dense materials encountered, it is our opinion that liquefaction is not a design consideration.
- Our limited laboratory corrosivity testing indicates that the on-site earth materials can be classified as non-corrosive based on the California Department of Transportation (Caltrans, 2021) 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 project understanding. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and requirements of the applicable governing agencies.

9.1 Earthwork

Based on our understanding of the project, earthwork at the site is anticipated to consist of demolition of the existing spillway and excavation and re-compaction of near-surface soil below the proposed spillway. Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections.

9.1.1 Pre-Construction Conference

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

9.1.2 Site Clearing

Prior to performing site excavations, the site should be cleared of surface obstructions, pavements, abandoned utilities, and other deleterious materials. Existing utilities within the project limits, if any, should be re-routed or protected from damage by construction activities. Obstructions that extend below finish grade, if any, should be removed and the resulting holes filled with compacted soils. Materials generated from the clearing operations should be removed from the project site and disposed at a legal dumpsite.

9.1.3 Excavation Characteristics

Based on our subsurface exploration, we anticipate that excavations within the eolian deposits may be accomplished with conventional earthmoving equipment in good working order. We anticipate that the materials excavated at the site will consist of moist, medium dense to very dense, poorly graded sand, poorly graded sand with silt, and silty sand. These soils have little to no cohesion and will be prone to caving. Rubber-tired grading equipment may have difficulty traveling across the site where loose sand is present. Contractors should make their own independent evaluation of the site conditions and excavatability of the on-site materials prior to submitting their bids.

9.1.4 Temporary Excavations and Shoring

Soils at the project site have little to no cohesion and are considered to be prone to caving. Bedding materials for existing pipelines, if encountered, will also be prone to caving. Temporary slopes in the site soils that are not subjected to surcharge should be stable at inclinations of approximately 1½:1 (horizontal to vertical) not exceeding the depth of 20 feet below the existing grade. Temporary excavations should be evaluated in the field and constructed in accordance with applicable OSHA guidelines. The site soils should be considered as OSHA Soil Type C. Onsite safety of personnel is the responsibility of the contractor.

Excavations should be planned in a manner so as not to impair the bearing capacity or cause settlement or undermining of the existing structures or utilities to be protected in-place. As a general guideline, excavations adjacent to and parallel to existing foundations should not extend below an imaginary 1:1 (horizontal to vertical) plane extending outward and downward from the bottom outer edges of the foundations.

Temporary shoring may be needed if there are boundary constraints with existing streets and buildings. Shoring systems, if used, should be designed for the anticipated soil conditions using the lateral earth pressure values shown on Figures 5 and 6 for braced and cantilevered excavations, respectively. The recommended design pressures are based on the assumption that the shoring system is constructed without raising the ground surface elevation behind the shored sidewalls of the excavation, that there are no surcharge loads, such as soil stockpiles and construction materials, and that no loads act above a 1:1 (horizontal to vertical) plane ascending from the base of the shoring system. For a shoring system subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures acting on the shored walls. Spoils should not be placed near the edge of the open cut excavation. For open cut excavations, the spoil pile should be placed at a distance more than the depth of excavation from the top of the excavation. OSHA and other applicable agency requirements pertaining to worker safety should be met during the excavation activities.

We anticipate that settlement of the ground surface will occur behind shored excavations. 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 preliminary in nature, 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.

9.1.5 Treatment of Near-Surface Soils

Based on our subsurface evaluation, it is our opinion that suitable support for the proposed spillway may be provided by remedial grading consisting of the overexcavation and recompaction of the near-surface soils. The base of the concrete spillway should extend 2 feet or more below the potential scour depth of the spillway. Additionally, we recommend that over-excavation and recompaction extend to a depth that will provide 2 feet of compacted fill below the bottom of the proposed spillway. The horizontal limits of remedial over-excavation should extend approximately 4 feet beyond the spillway footprint. The over-excavation should remove existing loose surficial soils and expose relatively dense alluvial deposits. The removal and recompaction work should consist of 1) removing existing on-site soil to a depth of approximately 2 feet below the potential scour depth to establish the base of the spillway, 2) removing existing on-site soil to a depth of approximately 2 feet below the bottom of the spillway, and 3) replacing the excavated materials with recompacted fill. The fill soils should be moisture-conditioned to generally above the optimum moisture content and should be compacted to a relative compaction of 95 percent as evaluated by ASTM International (ASTM) D 1557. We anticipate that the scour depth will be determined by the designer based on a hydraulic/hydrologic study. Additional overexcavation may be appropriate depending on the materials exposed during construction.

9.1.6 Fill Material

In general, the on-site soil materials should be suitable for reuse as fill material, provided they are free of over-size rocks, 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. Structure backfill should be comprised of granular, non-expansive soil that conforms to the latest edition of “Greenbook” Standard Specifications for Public Works Construction for structural backfill. “Non-expansive” can be defined as soil having an expansion index (EI) of 20 or less in accordance with ASTM D 4829. The on-site materials will involve moisture-conditioning to bring the materials near optimum content prior to placement and compaction.

Imported materials, if used, should consist of clean, non-expansive, granular material, which conforms to the “Greenbook” for structure backfill. The imported materials should also meet the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a minimum resistivity greater than 1,500 ohm-centimeters (ohm-cm), a chloride concentration less than 500 parts per million (ppm), a sulfate concentration of less than 0.15 percent (1,500 ppm), and a pH value greater than 5.5). Import 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.

9.1.7 Fill Placement and Compaction

Fill soils placed should be compacted in horizontal lifts to a relative compaction of 95 percent as evaluated by ASTM D 1557. The lift thickness for fill soils will vary depending on the type of compaction equipment used but should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Fill soils should be placed at slightly above the optimum moisture content as evaluated by ASTM D 1557. Special care should be taken to avoid damage to utility lines when compacting fill and subgrade materials. Placement and compaction of the fill soils should be in general accordance with applicable grading ordinances and good construction practice.

9.2 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 seismic design parameters for the site in accordance with the 2022 CBC guidelines.

Table 1 – 2022 California Building Code Seismic Design Criteria

Spectral Response Acceleration Parameters	Values
Site Classification	C
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_s	1.874g
Mapped MCE_R Spectral Response Acceleration at 1.0-Second Period, S_1	0.663g
MCE_R Spectral Response Acceleration at Short Periods, Adjusted for Site Class, S_{MS}	2.249g
MCE_R Spectral Response Acceleration at 1.0-Second Period, Adjusted for Site Class, S_{M1}	0.928g
Design Spectral Response Acceleration at Short Periods, S_{DS}	1.499g
Design Spectral Response Acceleration at 1.0-Second Period, S_{D1}	0.619g
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.970g

9.3 Concrete Spillway

The concrete spillway should be designed by the project structural engineer based on the anticipated loading conditions. The base of the concrete spillway should extend 2 feet or more below the potential scour depth of the spillway and be supported on compacted fill soils as recommended in the Earthwork section of this report. We anticipate that the scour depth will be determined by the designer based on a hydraulic/hydrologic study. We recommend that the concrete spillway have a thickness of 6 inches or more and be reinforced appropriately per the recommendation of the structural engineer. The spillway should be underlain by 4 inches of granular material such as crushed aggregate base or crushed miscellaneous base and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking.

The spillway may be designed using a net allowable bearing capacity of 2,500 pounds per square foot. Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be on the order of 1 inch and ½ inch over a horizontal span of 40 feet, respectively.

9.4 Scour Protection

We recommend scour protection measures be implemented to mitigate scour and erosion around the new concrete spillway. The depth of scour along the spillway should be evaluated by the project designer. Scour protection measures may include articulated concrete block (ACB) mats or other erosion control mats, rip-rap, geosynthetics, and shallow and deep-rooted plant varieties. The type of scour protection should be evaluated and selected by the project designer. We understand that new slopes or erosion control of existing basin embankment slopes are not planned as part of the current scope for the project.

9.5 Corrosivity

Laboratory testing was performed on a representative sample of near-surface soil to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with CT 643. The 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 soil pH was measured at approximately 7.5 and the electrical resistivity was measured to be approximately 36,523 ohm-cm. The chloride content of the sample was measured to be approximately 15 ppm. The sulfate content of the tested sample was approximately 0.001 percent (i.e., 10 ppm). Based on the laboratory test results and Caltrans (2021) corrosion criteria, the project site would be classified as a non-corrosive site, which is defined as having earth materials less than 500 ppm chlorides, less than 0.15 percent sulfates (i.e., 1,500 ppm), a pH of more than 5.5, and an electrical resistivity of more than 1,500 ohm-cm.

9.6 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of soluble sulfates can be subject to chemical 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 sample tested for this evaluation, using Caltrans Test Method 417, indicate a water-soluble sulfate content of approximately 0.001 percent by weight. Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of the on-site soils, consideration should be given to using Type II/V cement for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structure 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 (2022). The structural engineer should be consulted for additional concrete specifications.

10 CONSTRUCTION OBSERVATION

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

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

- Observe clearing, grubbing, and removals.
- Observing excavation bottoms and the placement and compaction of fill.
- Evaluating imported materials prior to their use as fill (if used).
- Performing field tests to evaluate fill compaction.
- Performing material testing services including concrete compressive strength 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.

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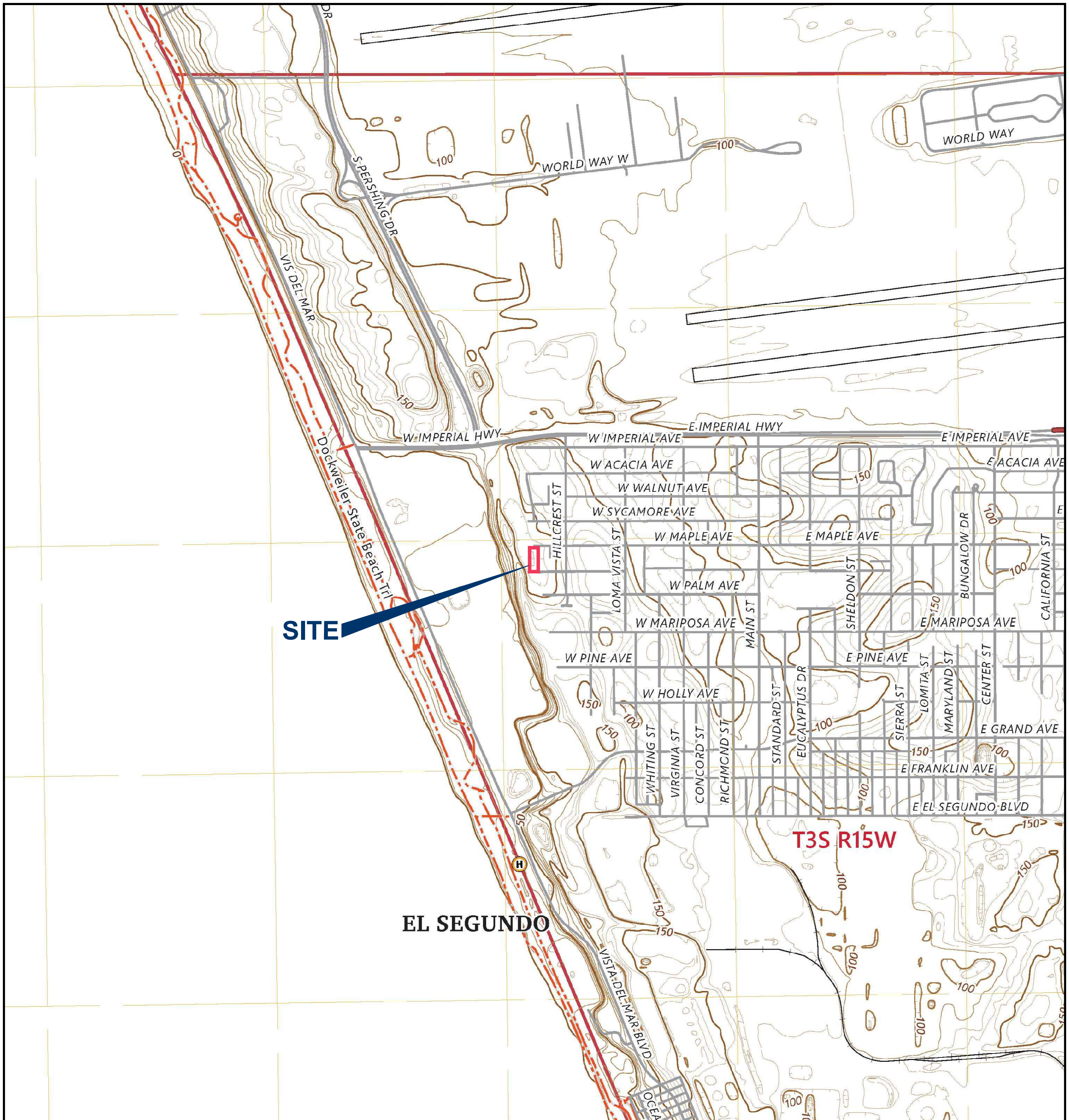
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: USGS, 2023.

N

FEET

0 2,000 4,000

FIGURE 1

SITE LOCATION

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EL SEGUNDO, CALIFORNIA

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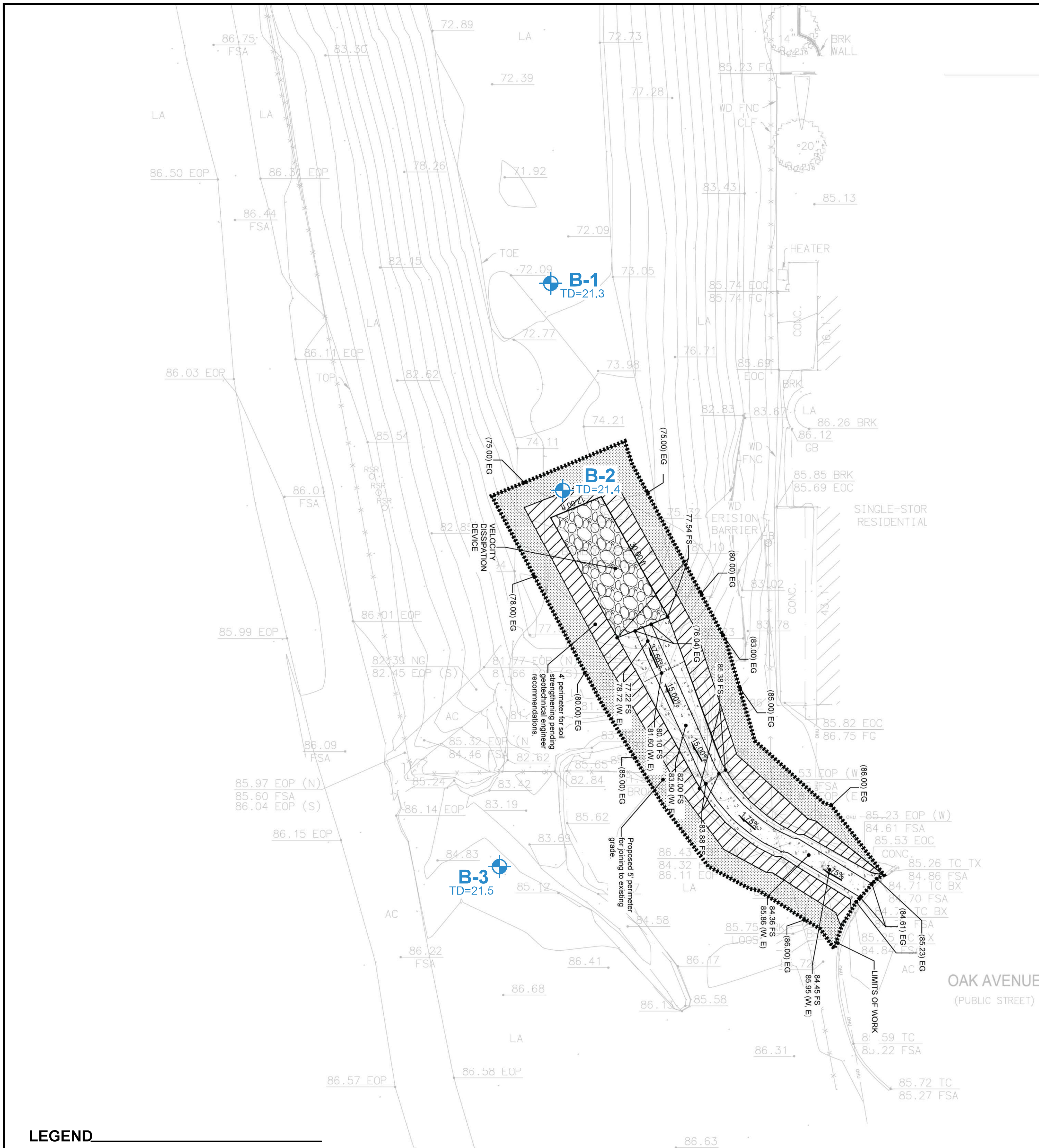

LEGEND

B-3 TD=21.5 — BORING; TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: KPFF, 2023.

N

FEET 0 30 60

FIGURE 2

BORING LOCATIONS

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LEGEND

Qoa	OLD ALLUVIUM	Qe	EOLIAN DEPOSITS
Qoe	OLD EOLIAN DEPOSITS	Qps	PLEISTOCENE SEDIMENTARY DEPOSITS, UNDIVIDED
Qom	OLD SHALLOW MARINE DEPOSITS ON WAVE-CUT SURFACE	(solid line)	GEOLOGIC CONTACT
Qye	YOUNG EOLIAN DEPOSITS	••••••?•	FAULT; DOTTED WHERE CONCEALED, QUERIED WHERE UNCERTAIN

N

FEET

0 3,000 6,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: SAUCEDO, ET AL., 2016.

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FIGURE 3

REGIONAL GEOLOGY

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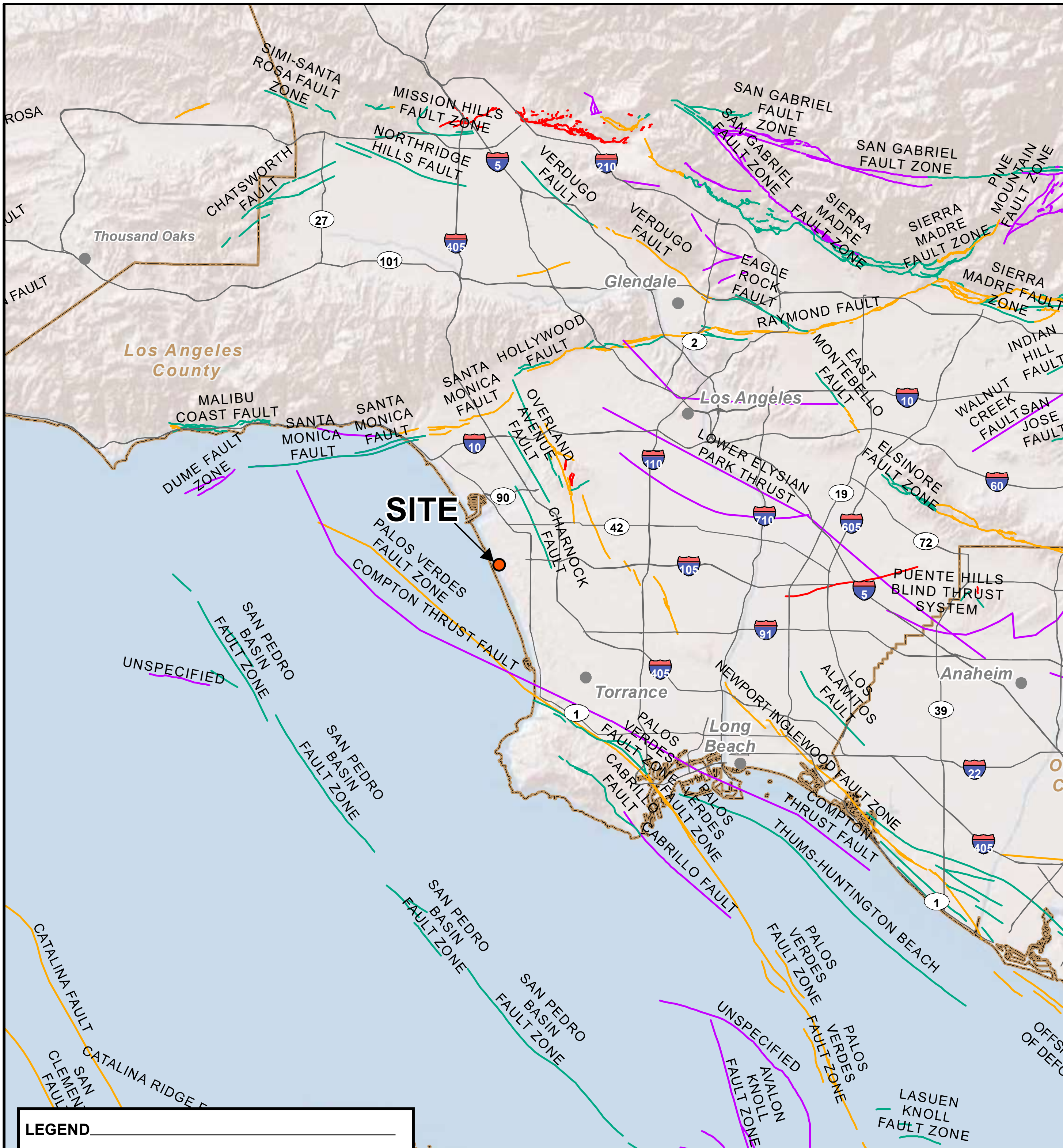

LEGEND

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

SOURCES: QUATERNARY FAULTS DATABASE - U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES, ACCESSED JULY 12, 2023, AT: https://www.usgs.gov/programs/earthquake-hazards/faults, ESRI, 2023.

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FIGURE 4

FAULT LOCATIONS

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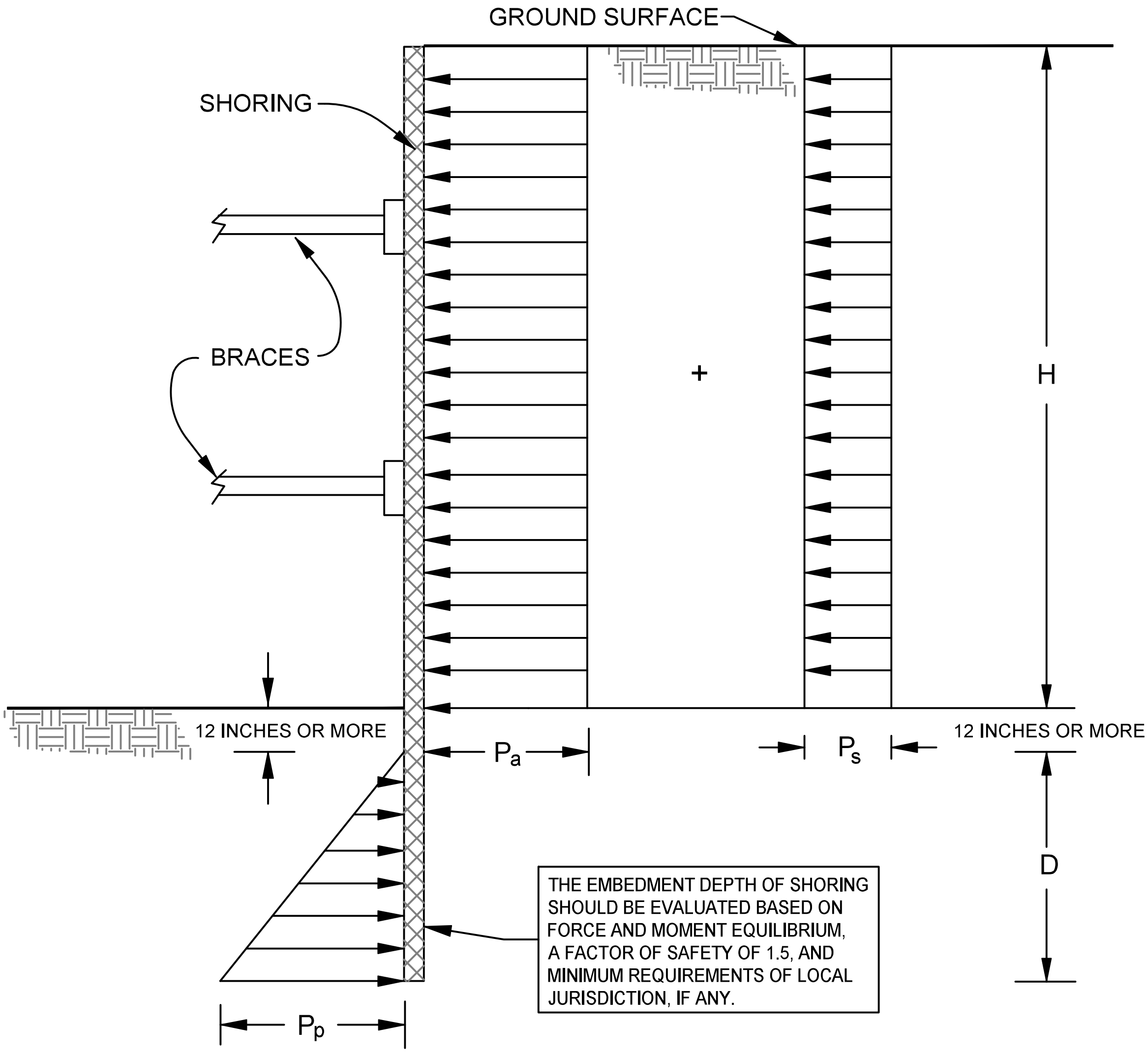

NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 21H 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

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FIGURE 5

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION (GRANULAR SOIL)

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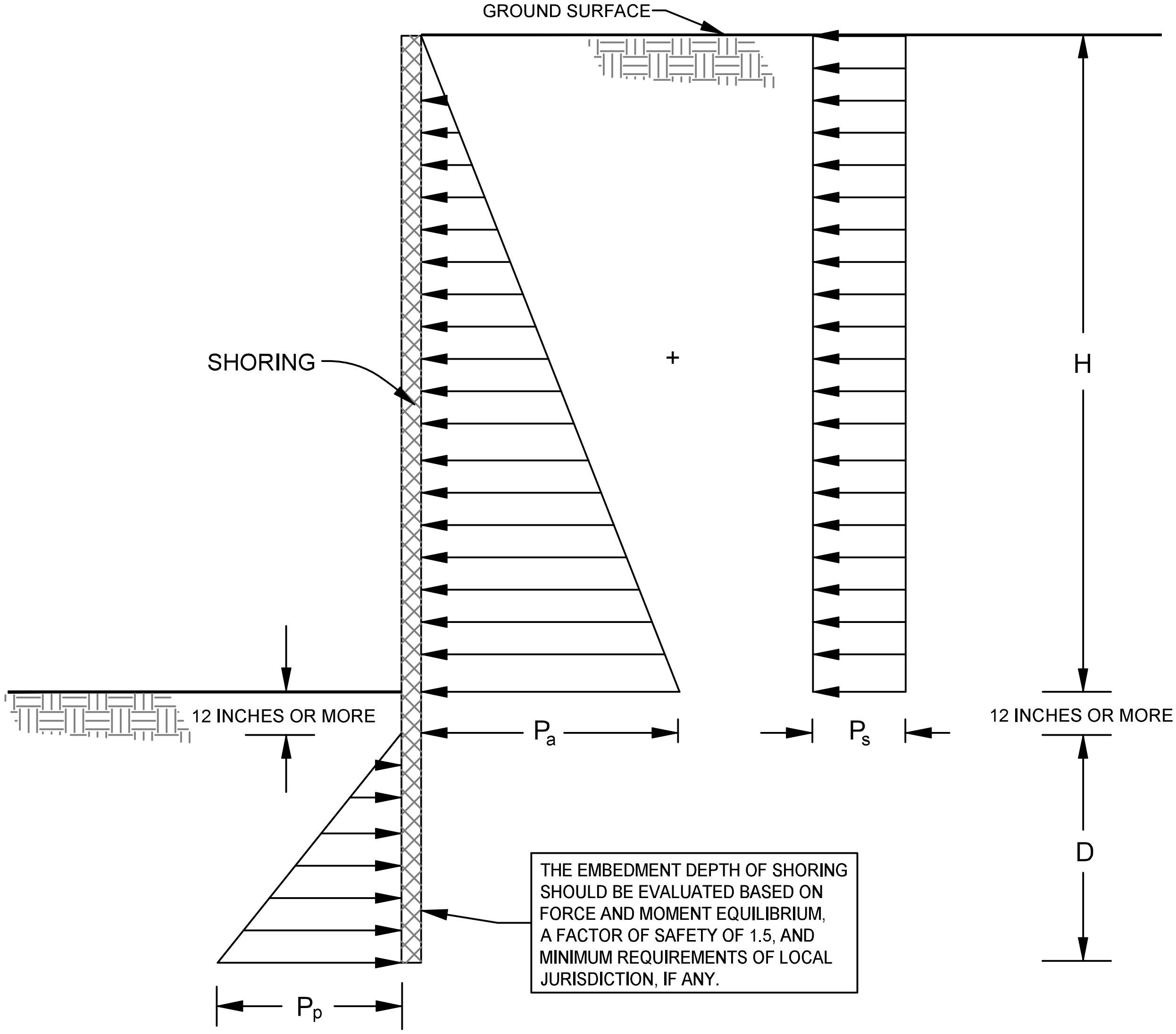

NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_a = 32H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 72 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

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FIGURE 6

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING

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SANDHILL BASIN IMPROVEMENT PROJECT
EL SEGUNDO, CALIFORNIA

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

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

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

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

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

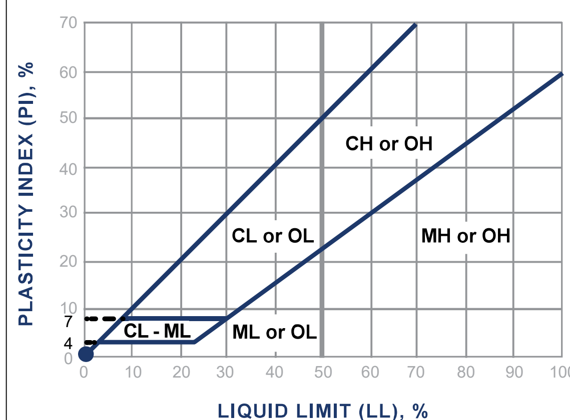
Soil Classification Chart Per ASTM D 2488

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
	Fine	#4 - 3/4”	0.19 - 0.75”	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19”	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079”	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017”	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029”	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

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

Consistency - Fine-Grained Soil

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

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USCS METHOD OF SOIL CLASSIFICATION

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.
15								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
20								The total depth line is a solid line that is drawn at the bottom of the boring.

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BORING LOG

DATE DRILLED 6/28/23 BORING NO. B-1

GROUND ELEVATION 72' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (Baja Exploration)

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

SAMPLED BY GM LOGGED BY GM REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■						SP	EOLIAN DEPOSITS: Light yellowish brown, moist, medium dense, poorly graded SAND.
	■						SP-SM	Gray, moist, medium dense, poorly graded SAND with silt; trace gravel.
		■	21	6.0	103.9			Reddish brown.
							SP	Yellowish brown, moist, medium dense, poorly graded SAND.
10		■	27					
		■	48					Dense.
20		■	87/9"					Very dense.
								Total Depth = 21.3 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 6/28/23. 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.
30								
40								

DATE DRILLED 6/28/23	BORING NO. B-2	
GROUND ELEVATION 75' ± (MSL)	SHEET 1	OF 1
METHOD OF DRILLING 8" Hollow-Stem Auger (Baja Exploration)		
DRIVE WEIGHT 140 lbs (Auto. Trip Hammer)	DROP 30"	
SAMPLED BY GM	LOGGED BY GM	REVIEWED BY RDH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	X						SP-SM	EOLIAN DEPOSITS: Gray, moist, medium dense, poorly graded SAND with silt; trace gravel.
		X	25	5.8	105.7			Dark reddish gray.
10		X	32					Decrease in silt content.
							SP	Yellowish brown, moist, dense, poorly graded SAND.
		X	48					
20		X	87/11"					Very dense.
								Total Depth = 21.4 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 6/28/23. 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.
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SP	EOLIAN DEPOSITS: Yellowish brown, moist, medium dense, poorly graded SAND.
			18	2.9	94.8			
10			20	3.4	97.3			
							SM	Reddish gray, moist, medium dense, silty SAND; trace gravel.
			21					
20			42					
								Total Depth = 21.5 feet. Groundwater was not encountered during drilling. Backfilled with cement-bentonite grout on 6/28/23. 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.
30								
40								

DATE DRILLED 6/28/23 BORING NO. B-3

GROUND ELEVATION 85' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (Baja Exploration)

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

SAMPLED BY GM LOGGED BY GM REVIEWED BY RDH

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 in Appendix A.

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

Gradation Analysis
A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-1. The test results were utilized in evaluating the soil classifications in accordance with the USCS.

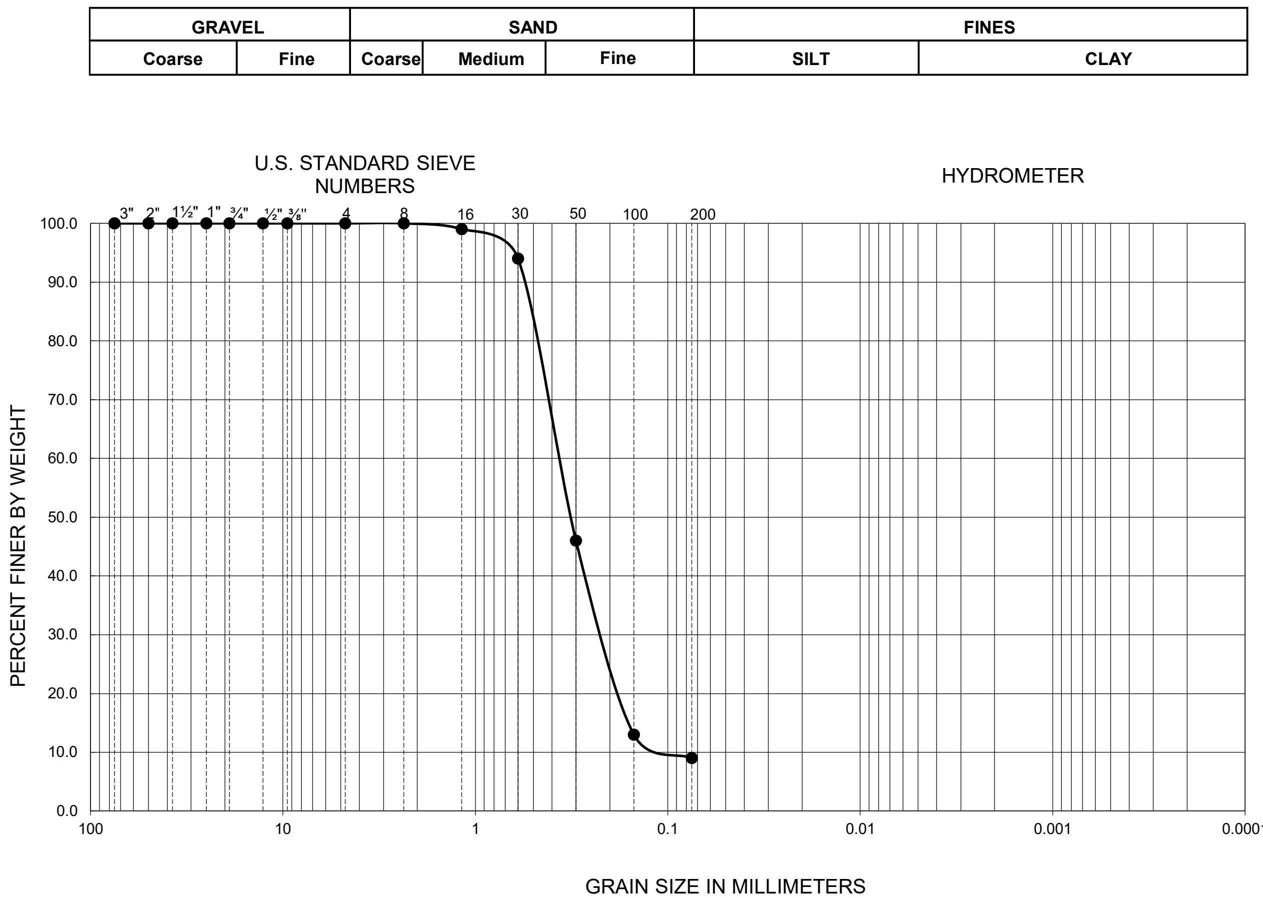
200 Wash
An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-2.

Consolidation Test
A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test are summarized on Figure B-3.

Direct Shear Test
A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-4.

Proctor Density Test
The maximum dry density and optimum moisture content of a selected representative soil sample was evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of this test are summarized on Figure B-5.

Soil Corrosivity Tests
Soil pH and resistivity tests were performed on a representative sample in general accordance with California Test (CT) 643. The soluble sulfate and chloride content of the selected sample was evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-6.



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	0.0-5.0	--	--	--	0.12	0.23	0.38	4.1	1.4	9	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 6913

FIGURE B-1

GRADATION TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	5.0-6.5	POORLY GRADED SAND WITH SILT	100	7	SP-SM
B-3	0.0-5.0	POORLY GRADED SAND	100	1	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-2

NO. 200 SIEVE ANALYSIS TEST RESULTS

SANDHILL BASIN IMPROVEMENT PROJECT
EL SEGUNDO, CALIFORNIA

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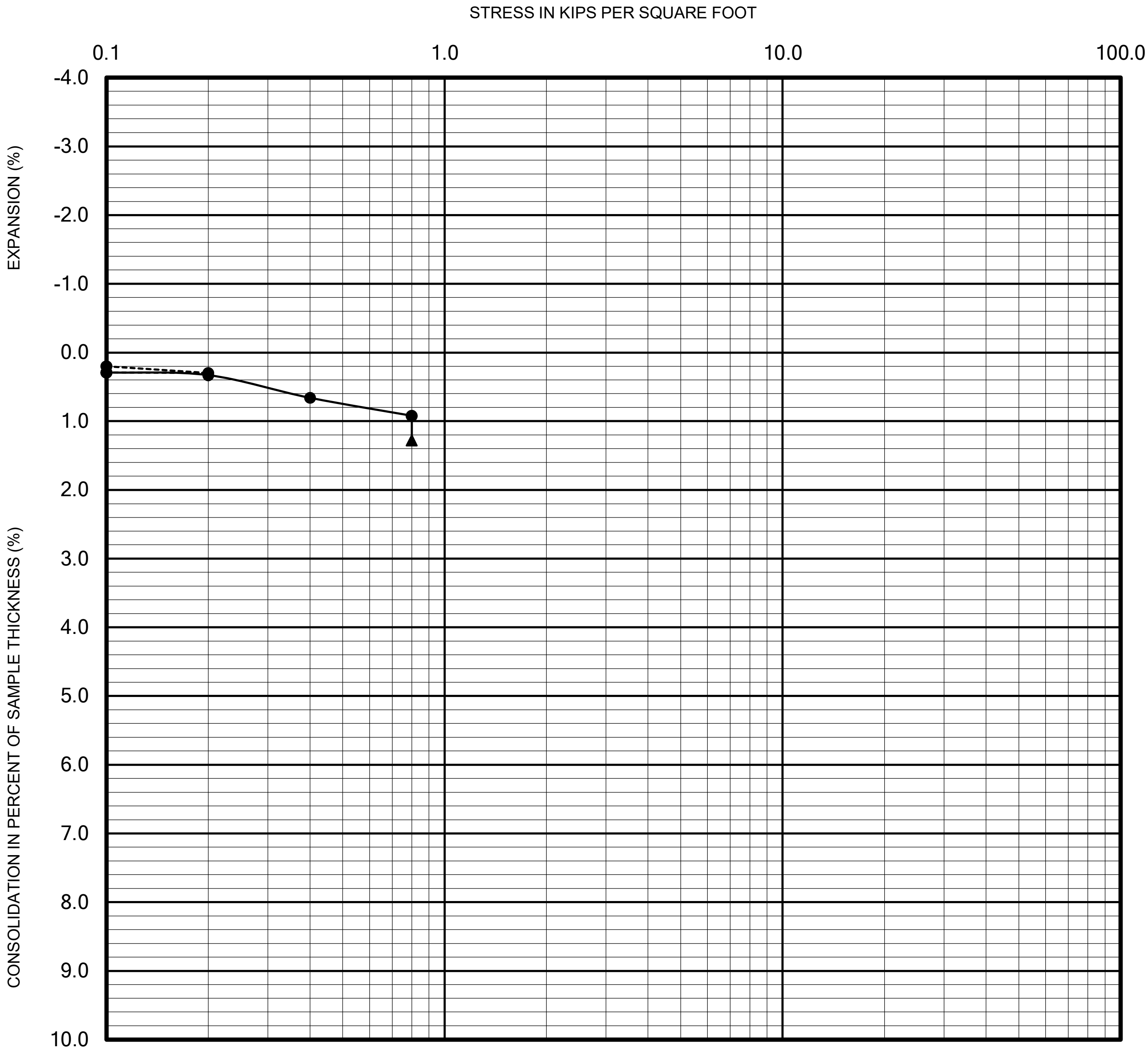

Seating Cycle
Loading Prior to Inundation
Loading After Inundation
Rebound Cycle

Sample Location	B-3
Depth (ft)	5.0-6.5
Soil Type	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-3

CONSOLIDATION TEST RESULTS

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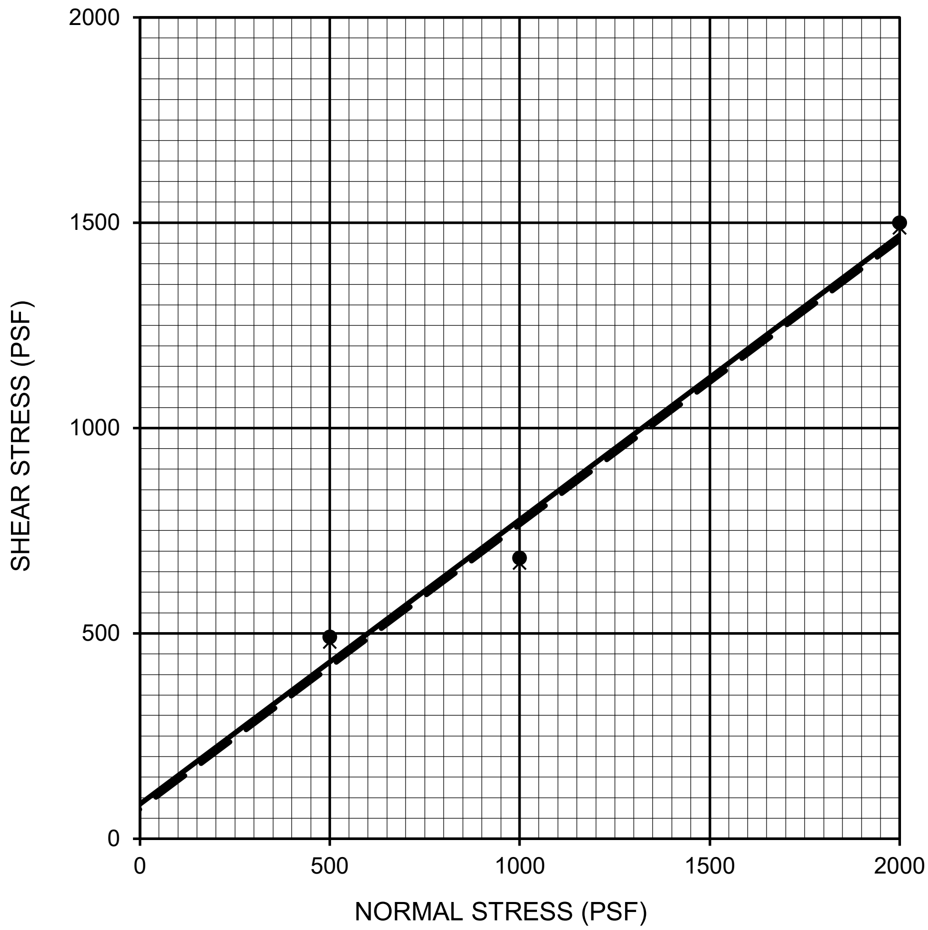

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
POORLY GRADED SAND WITH SILT	——●——	B-2	0.0-5.0	Peak	84	35	SP-SM
POORLY GRADED SAND WITH SILT	– – X – –	B-2	0.0-5.0	Ultimate	72	35	SP-SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080 ON A SAMPLE REMOLDED TO 90% RELATIVE COMPACTION

FIGURE B-4

DIRECT SHEAR TEST RESULTS

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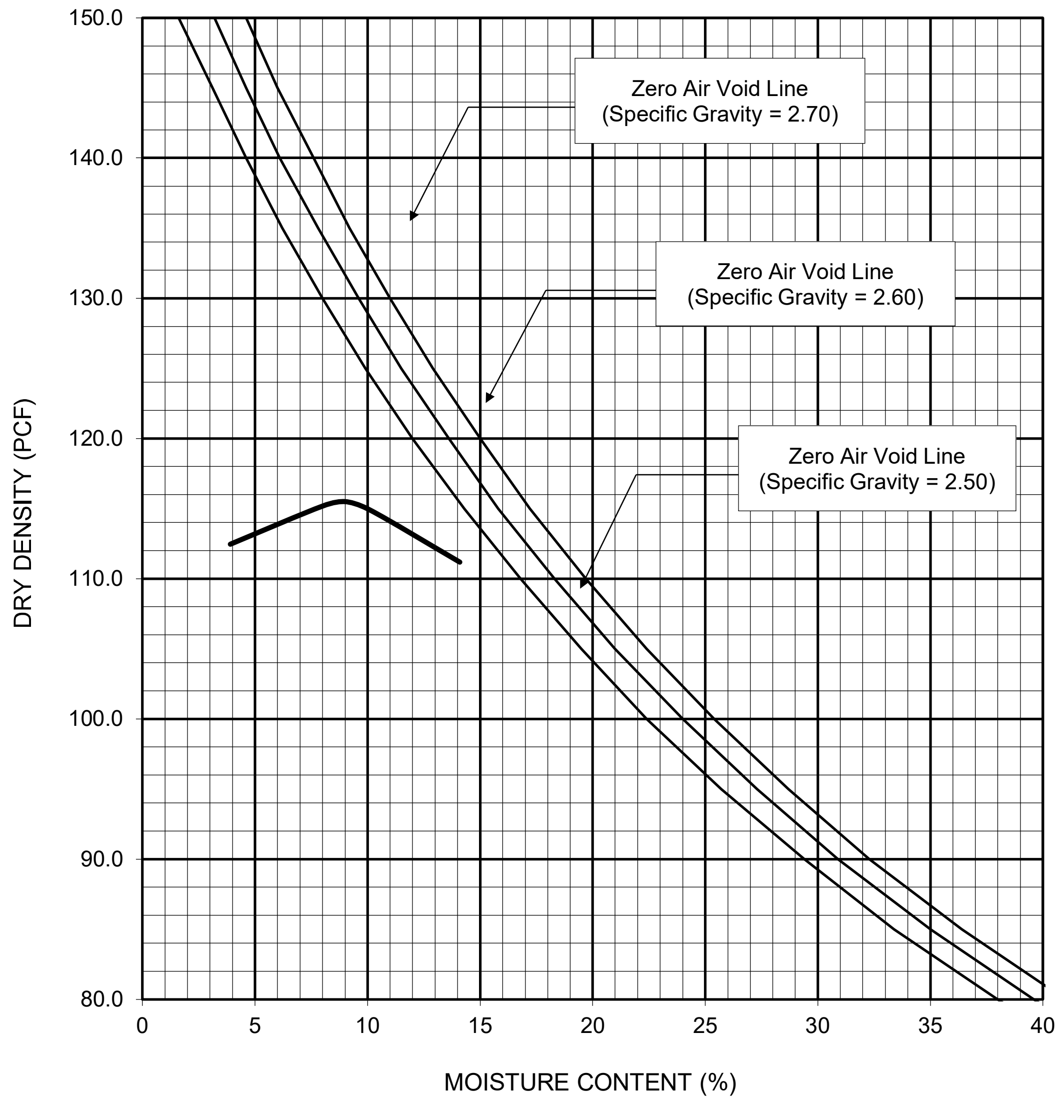

Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-2	0.0-5.0	Gray Poorly Graded Sand with Silt	115.5	9.0
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718)			N/A	N/A

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

FIGURE B-5

PROCTOR DENSITY TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-3	0.0-5.0	7.5	36,523	10	0.001	15

[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-6

CORROSIVITY TEST RESULTS

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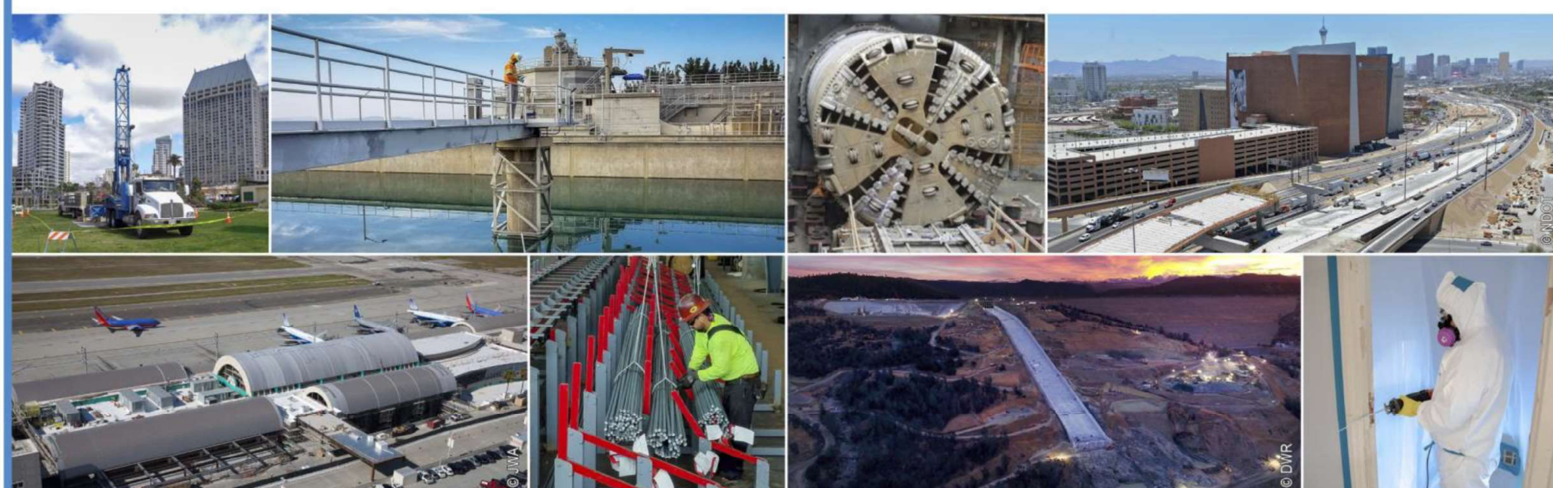

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