Geotechnical Evaluation

Devil's Punchbowl Nature Center Replacement Project

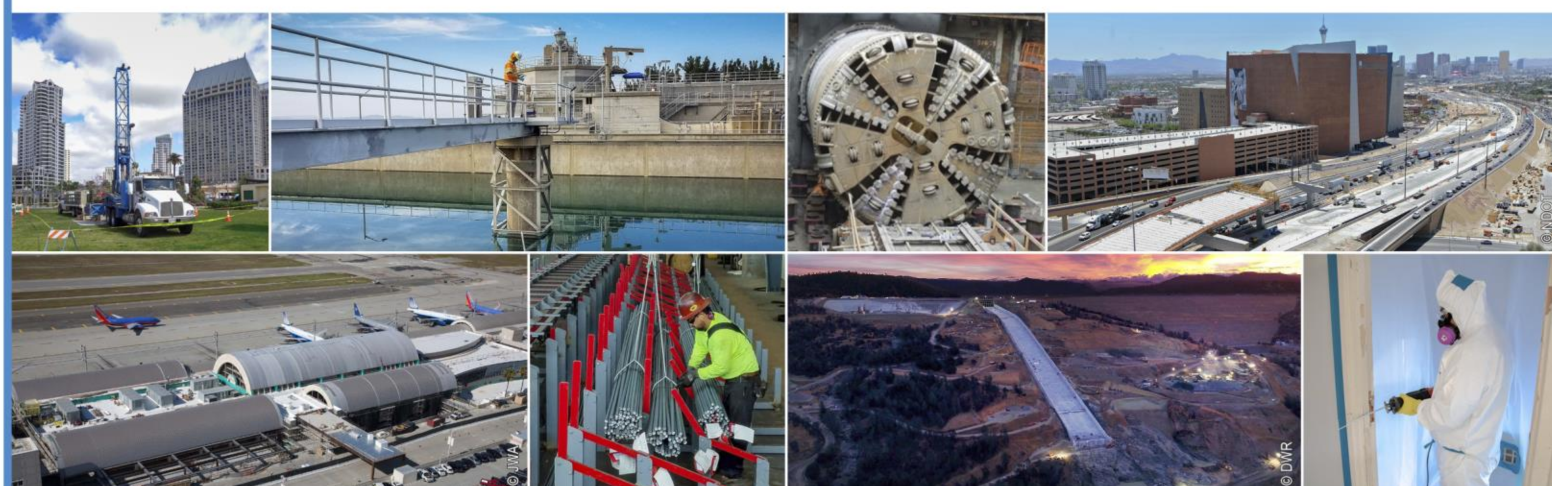
28000 Devil's Punchbowl Road

Pearblossom, California

ECORP Consulting, Inc.

215 N. 5th Street | Redlands, California 92374

December 9, 2022 | Project No. 212036002



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Devil's Punchbowl Nature Center Replacement Project
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Pearblossom, California

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1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed Devil's Punchbowl Nature Center Replacement Project located at 28000 Devil's Punchbowl Road in Pearblossom, 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 new nature center. This report presents our geotechnical findings, conclusions, and recommendations regarding the project improvements.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration.
- Review of readily available background material, including published geologic maps, fault and seismic hazards maps, groundwater data, topographic maps, and stereoscopic aerial photographs.
- Geotechnical site reconnaissance to observe the general site conditions, mark out the proposed boring locations, and coordinate with Underground Service Alert for utility clearance.
- Subsurface exploration consisting of the drilling, logging, and sampling of three small-diameter borings to depths of approximately 26 feet below the ground surface. The borings were logged by a representative of our firm and bulk and relatively undisturbed soil and formational samples were collected at selected intervals for laboratory testing.
- Laboratory testing on selected soil and formational samples, including evaluation of in-situ moisture content and dry density, percentage of soil particles finer than the No. 200 sieve, direct shear strength, and corrosivity.
- Data compilation and engineering analyses of the information obtained from our background review, subsurface evaluation, 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 AND PROPOSED CONSTRUCTION

The project site is located at the Devil's Punchbowl Natural Area at 28000 Devil's Punchbowl Road in the Pearblossom community of Los Angeles County, California (Figure 1). The project site is currently occupied by a classroom/administrative building, bathrooms, trailheads, and an asphalt-concrete parking lot. The previous Devil's Punchbowl Nature Center was destroyed in the 2020 Bobcat fire. Remnants of the destroyed nature center remain on the northern portion of the site. The project site has ground surface elevations ranging from approximately 4,745 to 4,760 feet above the mean sea level (Chris Nelson & Associates, Inc., 2022). On the east side of the project site, the exposed Punchbowl Formation slopes downward to the east, at a slope ratio of approximately 1.4 to 1 (horizontal to vertical).

We understand that the project will include replacement of the nature center and improvements to the surrounding areas. The new nature center will consist of a 3,000 square feet reinforced masonry structure that will include the nature center, administrative offices, and a gift shop. The building will have a green roof that will also act as a lookout platform for visitors. Additional site improvements will include picnic areas, shade structures, ADA access to buildings and trails, and improvements to the trailheads (Figure 2).

4 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration was performed on October 19, 2022 and consisted of drilling, logging, and sampling of three small-diameter borings using a truck-mounted drill rig with 8-inch-diameter hollow-stem augers to depths of approximately 26 feet. The borings were logged by a representative from our firm and bulk and relatively undisturbed soil and formational samples were obtained at selected depths for laboratory testing. The approximate locations of the borings are presented on Figure 2. The boring logs are presented in Appendix A.

Laboratory testing was performed to evaluate in-situ moisture content and dry density, percentage of soil particles finer than the No. 200 sieve, direct shear strength, and corrosivity. The results of our in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The results of the remaining laboratory tests are presented in Appendix B.

5 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geology

The project site is located in the San Gabriel Mountains, between the Los Angeles Basin and the Mojave Desert, within the Transverse Ranges geomorphic province of southern California (Norris and Webb, 1990). The province is characterized by a region of east to west trending mountain ranges and valleys that contrast with the typical northwest-southeast structural trend of California. The contrasting trend is considered the result of a restraining bend along the San Andreas fault (i.e., the "Big Bend"), which resulted in rotation of the province as well as tectonic uplift, folding, and faulting of thick sequences of marine and non-marine sedimentary rock.

Regional geologic maps (Figure 3) indicate that the project site is underlain by older alluvium and the Miocene-age Punchbowl Formation. The older alluvium consists of gravel and sand and the

Punchbowl Formation consists of medium to coarse-grained sandstone and conglomerate with interbeds of claystone (Dibblee, 2002).

5.2 Site Geology

Materials encountered during our subsurface exploration consisted of alluvium underlain by formational materials of the Miocene-age Punchbowl Formation to the total depth explored of approximately 26 feet. Alluvium was encountered at the ground surface and extended to depths ranging from approximately 9 to 13 feet below the ground surface. The alluvium generally consisted of dry to moist, medium dense to very dense, silty sand with gravel and poorly graded gravel with silt and sand. Cobbles and boulders were encountered in the alluvium. The Punchbowl Formation was encountered beneath the alluvium and generally consisted of moist, moderately cemented, sandstone with rounded to subrounded gravel. More detailed descriptions of the subsurface materials 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 26 feet. Regional maps indicate that the historic high groundwater at the site is deeper than 30 feet below the ground surface (California Geological Survey [CGS], 2003). 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.

6 FAULTING AND SEISMICITY

The project site is located in a seismically active area, as is the majority of southern and central California. The numerous faults in this area of California include active, potentially active, and inactive faults. As defined by the California Geological Survey, 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 the 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. The nearest active fault is the San Andreas fault, located approximately 2 miles northeast of the site (United States Geological Survey [USGS], 2008).

Based on our review of the 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 site is located approximately 2 miles away from the active San Andreas Fault Zone. The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, liquefaction, and landsliding. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

6.1 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.

6.2 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 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. Based on our review of CGS's shear wave velocity map, the average shear wave velocity in the upper 100 feet (i.e., 30 meters) of the subsurface profile (V_{S30}) is estimated to be approximately 1,270 feet per second (i.e., 387 meters 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. Additionally, the shallow depth to Punchbowl Formation materials encountered in our subsurface evaluation supports a Site Class C.

In accordance with ASCE 7-16, the mapped MCE_R ground motion response accelerations were determined using the 2022 Applied Technology Council (ATC) 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 2019 CBC, are provided in Section 8.3 for the evaluation of seismic loads on buildings and other structures.

6.3 Liquefaction Evaluation

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.

According to the State of California Seismic Hazards Zones map (CGS, 2003), the site is not located in an area mapped as a potential liquefaction hazard zone. Additionally, our subsurface exploration indicates that the site is underlain by relatively dense soils and shallow sandstone formational materials. 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.

6.4 Landslides

The site of the proposed nature center is not located in an area mapped by the State of California as an area susceptible to earthquake-induced landslides on the Seismic Hazards Zones Map (CGS, 2003) (Figure 5). Although the descending slope just east of the site (Devil's Punchbowl) is mapped as an area considered susceptible to earthquake-induced landslides, it is our opinion that the site of the proposed nature center is relatively level and not likely to be subject to landsliding or slope instability.

7 CONCLUSIONS

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

- The project area is generally underlain by alluvium and formational materials of the Punchbowl Formation. The alluvium generally consisted of dry to moist, medium dense to very dense, silty sand with gravel and poorly graded gravel with silt and sand. Cobbles and boulders were encountered in the alluvium. The Punchbowl Formation was encountered beneath the alluvium at depths ranging from approximately 9 to 13 feet and generally consisted of moist, moderately cemented, sandstone with rounded to subrounded gravel.

- In general, excavation of the on-site material should be achievable with heavy earthmoving equipment in good working condition. However, contractors should anticipate difficulty excavating in very dense gravel and cobbles.
- We anticipate that the on-site excavated materials should be generally suitable for use as compacted fill following moisture-conditioning, but cobble size material (greater than 4 inches in diameter) should be removed before being used as fill. Fill material should be free of trash, debris, roots, vegetation, deleterious materials, and cobbles or hard lumps of materials in excess of 4 inches in diameter.
- The granular soils encountered at the site have little cohesion and may 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 during drilling to the total depth explored of approximately 26 feet. Fluctuations in the groundwater level may occur as a result of variations in seasonal precipitation, irrigation practices, groundwater pumping and other factors.
- Based on the relatively shallow depth of formational materials, it is our opinion that the project site is not susceptible to earthquake-induced liquefaction.
- The site is not located within a State of California Earthquake Fault Zone. Based on our review of the 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 to be low.
- Although the descending slope just east of the site is mapped as an area considered susceptible to earthquake-induced landslides, it is our opinion that the site of the proposed nature center is relatively level and not likely to be subject to landsliding or slope instability.
- 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.

8 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.

8.1 Earthwork

Earthwork at the site is anticipated to consist of site clearing, shallow cuts and fills associated with subgrade preparation for foundations, trenching for underground utilities, new exterior hardscape, and finish grading for establishment of site drainage. Earthwork operations should be performed in accordance with the requirements of the applicable governing agencies and recommendations presented in the following sections of this report.

8.1.1 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, Ninyo & Moore, and the contractor should attend to discuss the work plan, project schedule, and earthwork requirements.

8.1.2 Clearing and Site Preparation

Prior to excavation and fill placement, the site should be cleared of existing site improvements, pavements, surface obstructions and other deleterious materials, and abandoned utilities. Existing utilities to remain in place (if any) should be located and protected from damage by construction activities. Obstructions such as existing foundations that extend below the finished grade, if any, should generally be removed and the resulting holes filled with compacted soil. The materials generated from the clearing operations should be removed from the site and disposed of at a legal dump site.

8.1.3 Excavation Characteristics

Based on the subsurface exploration data, we anticipate that excavations should be feasible with heavy earthmoving equipment in good working order. The on-site alluvial deposits are comprised predominantly of medium dense to very dense, silty sand with gravel and poorly graded gravel with silt and sand. Cobbles and boulders were encountered in the alluvium and in the Punchbowl Formation. Difficult excavating should be anticipated in material containing dense to very dense cobbles and boulders. Weathered formational materials were encountered at depths ranging from approximately 9 to 13 feet. If excavations extend into the Punchbowl Formation, additional excavating effort should be anticipated by the contractor. Trench excavations may be particularly difficult where large boulders are encountered and may involve over-excavation or chipping with breaker bars or other specialized excavating equipment. Excavations for foundations may result in disturbed bottoms due to removal of large cobbles and boulders. Loose disturbed material should be removed from foundation excavation bottoms. Holes resulting from removal of boulders may be filled with compacted fill or concrete. Our representative should check foundation excavations prior to placement of reinforcing steel and concrete.

8.1.4 Temporary Excavations

Soils at the project site include medium dense sand and gravel with little cohesion that are considered to be prone to caving. In particular, bedding materials for existing pipelines, if encountered, may be prone to caving. Temporary slopes in the site soils should be stable at inclinations of approximately 1:1 (horizontal to vertical) up to a depth of about 4 feet below the existing grade and stable at inclinations of approximately 1½:1 (horizontal to vertical) for excavations deeper than 4 feet but 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.

8.1.5 Subgrade Preparation

Based on our subsurface evaluation, it is our opinion that suitable foundation support for the proposed building may be provided by remedial grading consisting of the over-excavation and recompaction of near-surface site soils. We recommend that over-excavation and recompaction extend to a depth that will provide 2 feet or more of compacted fill below the bottom of the proposed footings. The horizontal limits of remedial over-excavation should extend approximately 2 feet beyond the structure 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) over-excavating to the depths discussed above, 2) scarifying, moisture-conditioning, and compacting the exposed subgrade soils to a depth of 8 inches or more, 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 90 percent as evaluated by ASTM International (ASTM) test method D 1557.

In areas of concrete walkways and other hardscape improvements, we recommend that the top 8 inches of subgrade soils be scarified, moisture-conditioned to slightly over the optimum moisture content, and compacted to 90 percent relative compaction as evaluated by ASTM D 1557.

8.1.6 Fill Material

Oversize cobbles and boulders are not considered suitable to use as fill and should be screened out of material for use as fill. After removal of oversize material, we anticipate that the on-site soil will be suitable for re-use as fill and trench backfill. Fill should generally be free of rocks or lumps of material in excess of 4 inches in diameter. Fill should be free of trash, debris, roots, vegetation, or other deleterious materials. 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 earth materials will involve moisture-conditioning to bring those to near the optimum moisture content prior to placing and compacting those as fill.

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.

8.1.7 Fill Placement and Compaction

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve moisture contents generally at or slightly above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 90 percent as evaluated in accordance with the ASTM D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by the governing agencies. It is the contractor's responsibility to notify this office and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill materials should be moisture-conditioned to generally at or slightly above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture-conditioning of fill soils should be generally consistent within the soil mass. Wet soils, or soil with a relatively high moisture content, if encountered during excavation, should be allowed to dry to near the laboratory optimum moisture content prior to their placement as backfill.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture-conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally at or slightly above the laboratory optimum, mixed, and then compacted by mechanical methods to a relative compaction of 90 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a similar manner until the desired finished grades are achieved. Special care should be taken to avoid pipe damage when compacting trench backfill above the pipes.

8.2 Underground Utilities

We anticipate that utility lines will be supported on compacted fill or alluvial deposits. The depths of the utility pipelines are not known; however, we anticipate that the pipe invert depths will not exceed 10 feet. 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 the bottom outer edges of the adjacent footing. Utility lines that cross beneath footings should be encased in concrete below the footing.

8.2.1 Pipe Bedding

We recommend that utility pipelines be supported on 6 inches or more of granular bedding material. Bedding material should be placed around pipe zones to 1 foot or more above the top of the pipe. The bedding material should be classified as sand, be free of organic material, and have a sand equivalent (SE) of 30 or more. We do not recommend that gravel be used for bedding material because of the nature of the subsurface material. It has been our experience that the voids within gravel are sufficiently large 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. Compaction of the bedding material and backfill should proceed uniformly up both sides of the pipe. Trench backfill, including bedding material, should be placed in accordance with the recommendations presented in the Earthwork section of this report.

8.2.2 Trench Backfill

Based on our subsurface evaluation, the on-site sandy soils should generally be suitable for re-use as trench backfill provided they are free of organic material, clay lumps, debris, and rocks more than approximately 4 inches in diameter. We recommend that trench backfilling be in general conformance with the Standard Specifications for Public Works Construction ("Greenbook"). Fill should be moisture-conditioned to at or slightly above the laboratory optimum. Wet soils should be allowed to dry to a moisture content near the optimum prior to their placement as trench backfill. Trench backfill should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

8.2.3 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 the granular bedding material is placed adjacent to the pipe, as recommended in this report.

8.3 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the site in accordance with the CBC (2019) guidelines.

Table 1 – 2019 California Building Code Seismic Design Criteria

Spectral Response Acceleration Parameters	Values
Site Class	C
Mapped Spectral Response Acceleration at 0.2-second Period, S_S	2.184g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.924g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	2.621g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.293g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.747g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.862g
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	1.129g

8.4 Foundations

The proposed building may be supported on shallow spread footings and slab-on-grade foundation systems bearing on compacted fill prepared in accordance with the recommendations presented in the Earthwork section of this report. Shade structures are anticipated to be supported on drilled piers. 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 structures.

8.4.1 Spread Footings

Spread footings for building structures should extend 24 inches or more below the lowest adjacent finished grade and bear on compacted fill soils. Continuous footings should have a width of 24 inches or more. Isolated pad footings should have a width of 36 inches 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.

Footings, as described above and bearing on compacted fill soils with very low to low expansion potential, may be designed using a net allowable bearing capacity of 3,000 pounds per square foot (psf). The net allowable bearing capacity may be increased by 250 and 500 psf for every additional foot of width and depth, respectively, up to a value of 4,000 psf. Total and differential settlements for the footing designed 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.

The footings bearing on compacted granular fill may be designed using a coefficient of friction of 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. The footings may be designed using a passive resistance value of 350 psf per foot of depth up to a value of 3,500 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 net allowable bearing capacity and passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

8.4.2 Slabs-On-Grade

Building floor slabs should be designed by the project structural engineer based on the anticipated loading conditions. Building floor slabs should be underlain by compacted fill prepared in accordance with the recommendations presented in the Earthwork section of this report. We recommend that slabs be 5 inches thick and reinforced with No. 4 steel reinforcing bars placed 18 inches on-center (each way) near the mid-height of the slab. The placement of the reinforcement in the slab is vital for satisfactory performance. The slab should be underlain by a vapor retarder and capillary break system consisting of a polyethylene vapor retarder (with a thickness of 10 mil or more) membrane placed over 4 inches of medium to coarse, clean sand or pea gravel. As an alternative, the slab underlayment may consist of a 15-mil Stego Wrap vapor barrier (or equivalent) placed over 4 inches of crushed gravel. The steel reinforcements for the floor slab shall be placed on the vapor retarder using chairs, as appropriate. The vapor retarder is recommended in areas where moisture-sensitive floor coverings are anticipated. Soils underlying the slabs 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.

8.4.3 Pole Foundations

We anticipate that shade structures will be supported on drilled pier foundations. Details of the shade structures were not available at the time of this report. The actual depth of the drilled piers should be evaluated when details of the shade structures are available.

Drilled pier foundations for the shade structures should have a diameter of 18 inches or more and may be designed using an allowable side friction value of 100 psf under static loading conditions starting at a depth of 1 foot below the ground surface. End bearing should be ignored for these drilled piers. In addition, an allowable resistance of 60 psf for uplift can also be used for design. The lateral capacity of drilled piers may be evaluated using a passive resistance of 300 psf per foot of depth, up to a value of 3,000 psf, acting against the width of the pier. The passive resistance can be increased by a factor of 2 to account for the soil arching effect on individual piers. Passive resistance should be ignored in the upper 1 foot of soil. If the ground surface at and around the piers is finished with hardscape or asphalt concrete, passive resistance may be used in the upper 1 foot of depth. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces. These calculations assume that the piers have a minimum spacing of three times the pier diameter. The project structural engineer should evaluate the design depth of the piers based on the recommendations provided above.

8.5 Retaining Walls

Retaining walls may be supported by foundations designed in accordance with the recommendations presented in the previous section of this report. Lateral earth pressures recommended for the design of restrained retaining walls are provided on Figure 6. Passive pressures may be increased by one-third when considering loads of short duration, including wind and seismic loads. Further, for sliding resistance, a friction coefficient of 0.35 may be used for the concrete and soil interface. The allowable resistance may be taken as the sum of the frictional and passive resistance, provided that the passive portion does not exceed one-half of the total allowable resistance.

Retaining walls should be backfilled with free-draining, granular, imported soil with non-expansive material (CBC Expansion Index of 20 or less). Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. Drainage design should include free-draining backfill materials and subsurface drainage provisions as shown on Figure 7.

8.6 Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a minimum thickness of 4 inches. The hardscape should be underlain by 4 inches of granular material such as Crushed Aggregate Base (CAB) or Crushed Miscellaneous Base (CMB) 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, consideration may be given to doweling the joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape.

8.7 Corrosivity

Laboratory testing was performed on a representative soil sample 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 soil pH was measured at approximately 6.5 and the electrical resistivity was measured to be approximately 6,941 ohm-cm. The chloride content of the sample was measured to be approximately 80 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 corrosion criteria (2021), the project site can be classified as a non-corrosive site, which is defined as having earth materials with less than 500 ppm chlorides, less than 0.15 percent sulfates, a pH of more than 5.5, and an electrical resistivity of more than 1,500 ohm-cm.

8.8 Concrete

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 American Concrete Institute criteria (2019), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight 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, indicates a water-soluble sulfate content of 0.001 percent by weight (i.e., 10 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.

8.9 Drainage

Positive surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to transport surface water away from foundations and other site improvements. Positive drainage is defined as a slope of 5 percent or more (2 percent or more if paved) for a distance of 10 feet or more away from the foundations. Surface water should not be allowed to pond adjacent to the footings. Concentrated runoff should not be allowed to flow over asphalt pavement as this can result in early deterioration of the pavement. Area drains for landscaped and paved areas are recommended.

9 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.
- Evaluating on-site soil for suitability of its use as engineered fill/structural backfill prior to placement.

- Evaluating imported materials prior to their use as fill, if any.
- 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.

10 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.

11 REFERENCES

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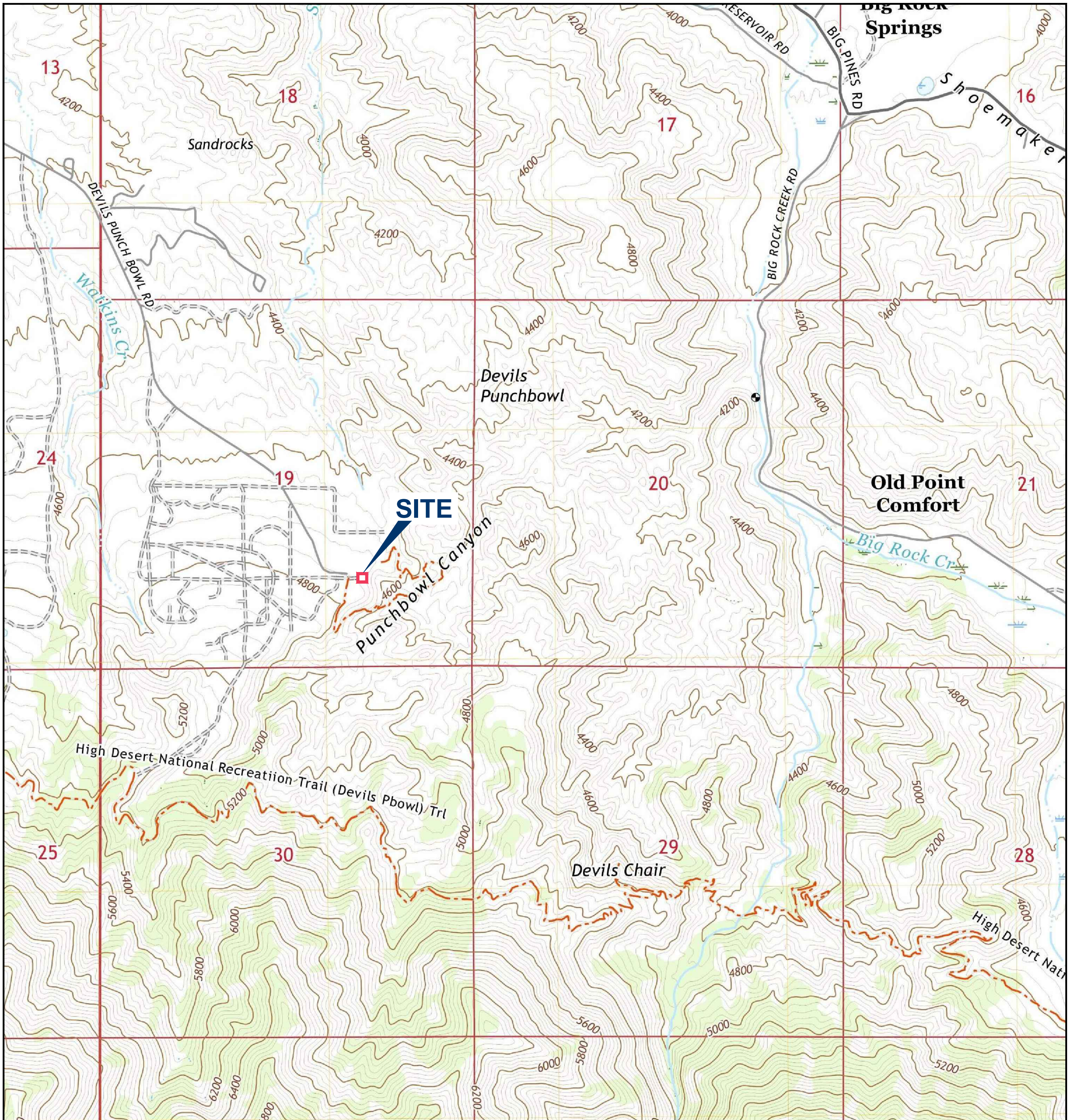
FIGURES



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

N

FEET

0 2,000 4,000

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FIGURE 1

SITE LOCATION

DEVIL'S PUNCHBOWL NATURE CENTER REPLACEMENT PROJECT
PEARBLOSSOM, CALIFORNIA

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LEGEND

B-3 TD=26.4 — BORING; TD=TOTAL DEPTH IN FEET

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: WITHERS & SANDGREN, 2022.

N

FEET 0 60 120

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FIGURE 2

SITE PLAN AND BORING LOCATIONS

DEVIL'S PUNCHBOWL NATURE CENTER REPLACEMENT PROJECT
PEARBLOSSOM, CALIFORNIA

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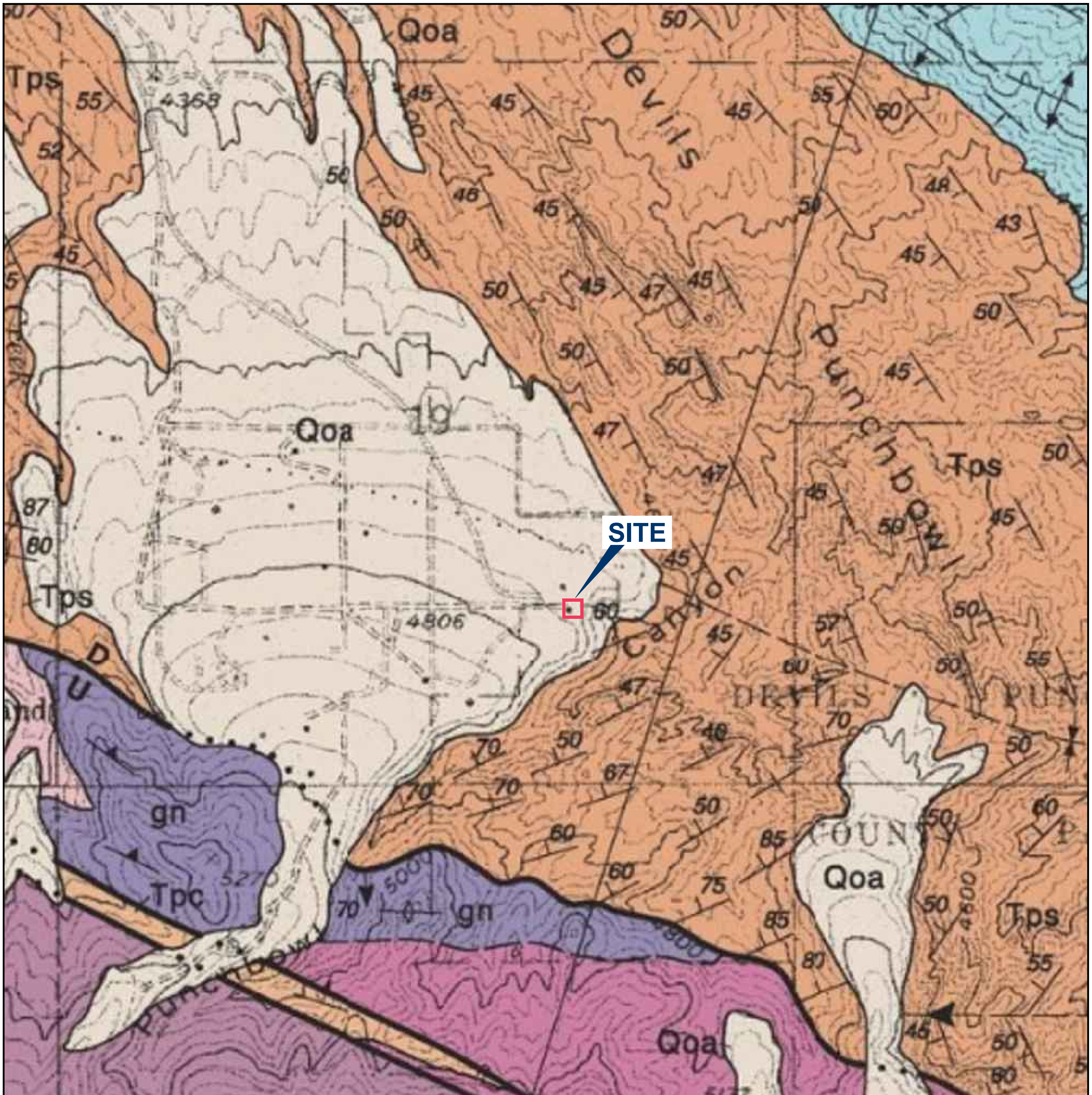

LEGEND

Qoa	OLDER ALLUVIUM		GEOLOGIC CONTACT
Tps	PUNCHBOWL FORMATION		FAULT

N

FEET

0 1,000 2,000

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: THOMAS W. DIBBLEE, JR., 2002.

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FIGURE 3

REGIONAL GEOLOGY

DEVIL'S PUNCHBOWL NATURE CENTER REPLACEMENT PROJECT
PEARBLOSSOM, CALIFORNIA

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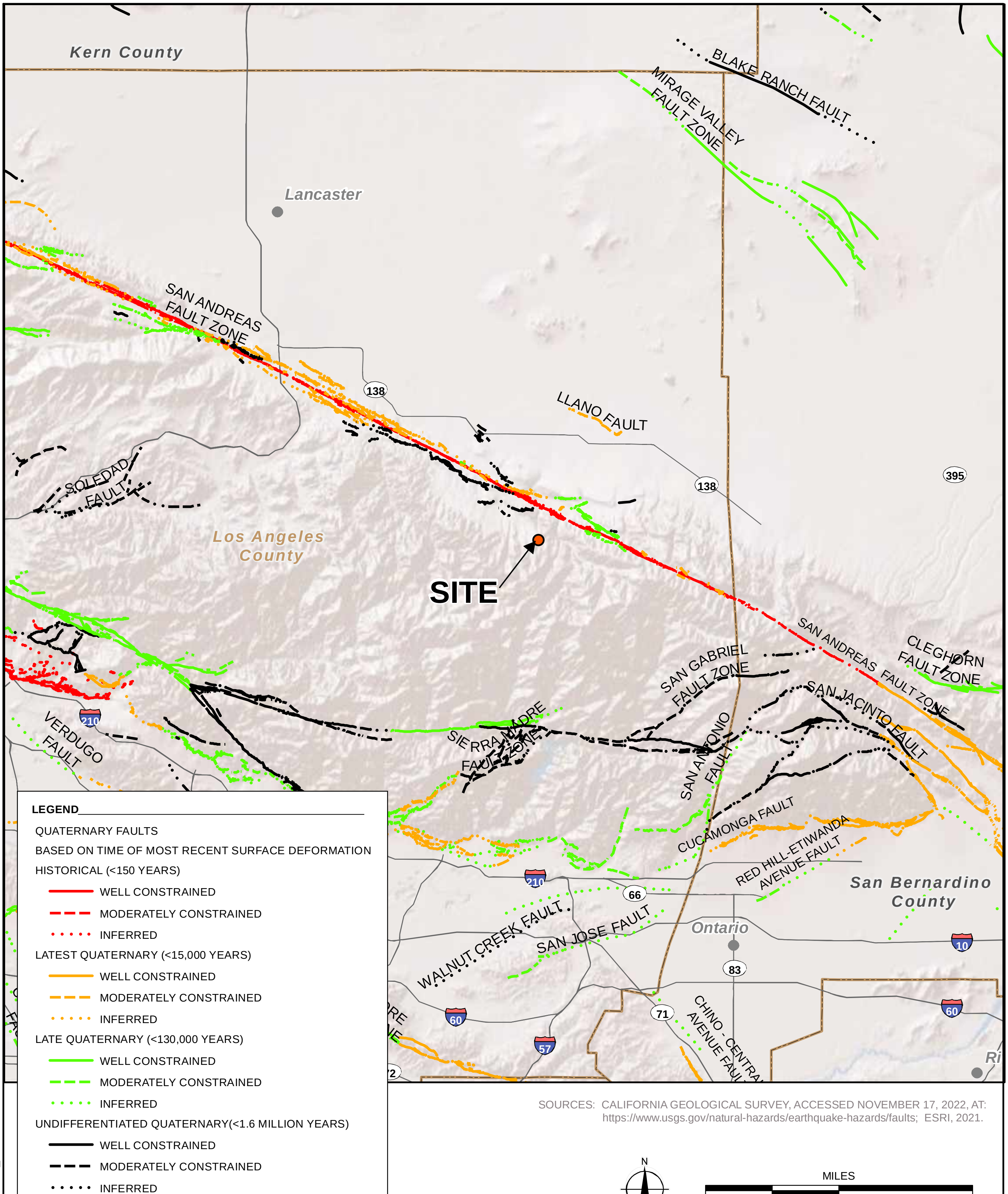

SOURCES: CALIFORNIA GEOLOGICAL SURVEY, ACCESSED NOVEMBER 17, 2022, AT: https://www.usgs.gov/natural-hazards/earthquake-hazards/faults; ESRI, 2021.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 4

FAULT LOCATIONS

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PEARBLOSSOM, CALIFORNIA

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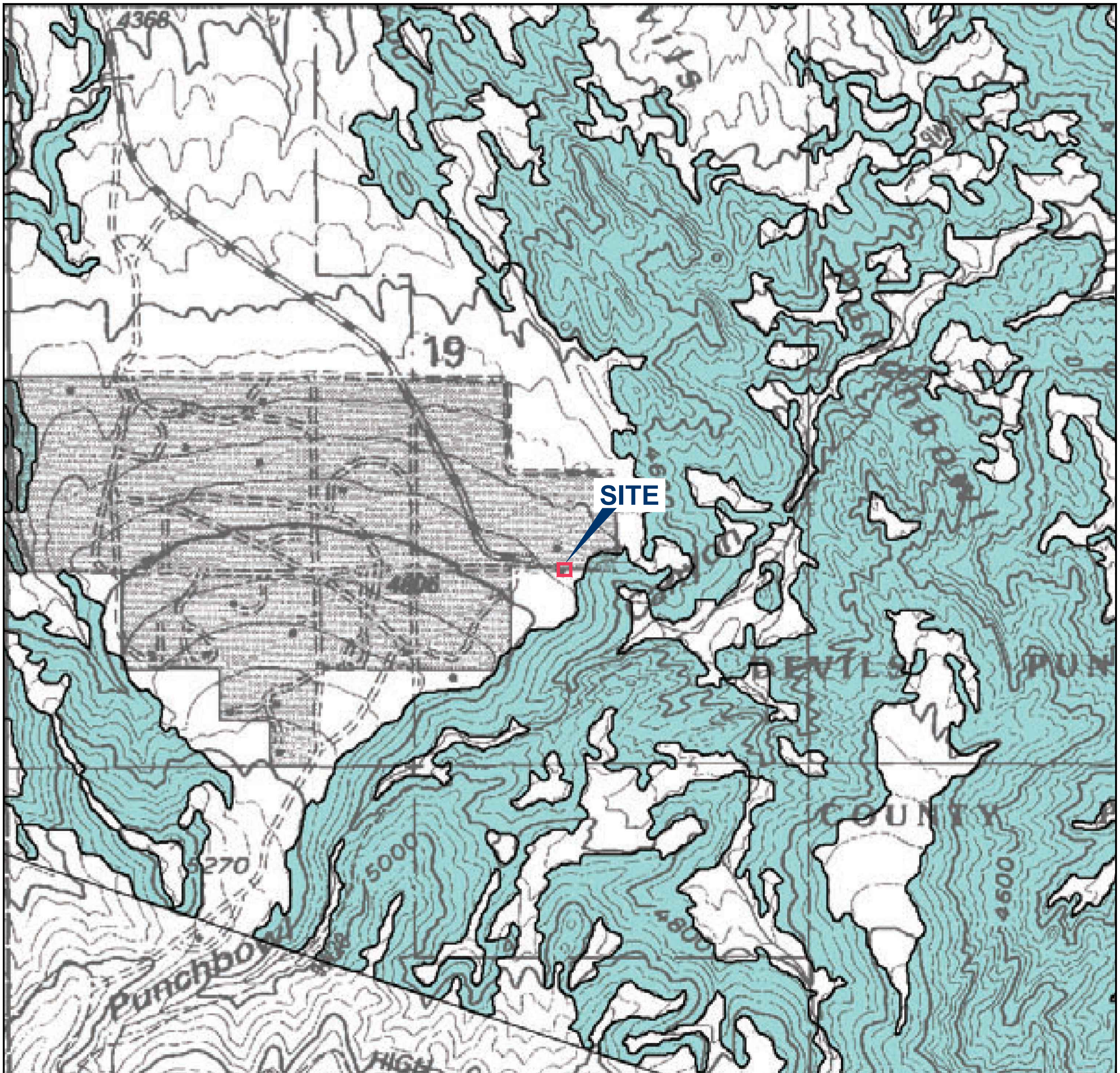

LEGEND

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.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. I REFERENCE: CGS, 2002.

N

FEET

0 1,500 3,000

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FIGURE 5

SEISMIC HAZARD ZONES

DEVIL'S PUNCHBOWL NATURE CENTER REPLACEMENT PROJECT
PEARBLOSSOM, CALIFORNIA

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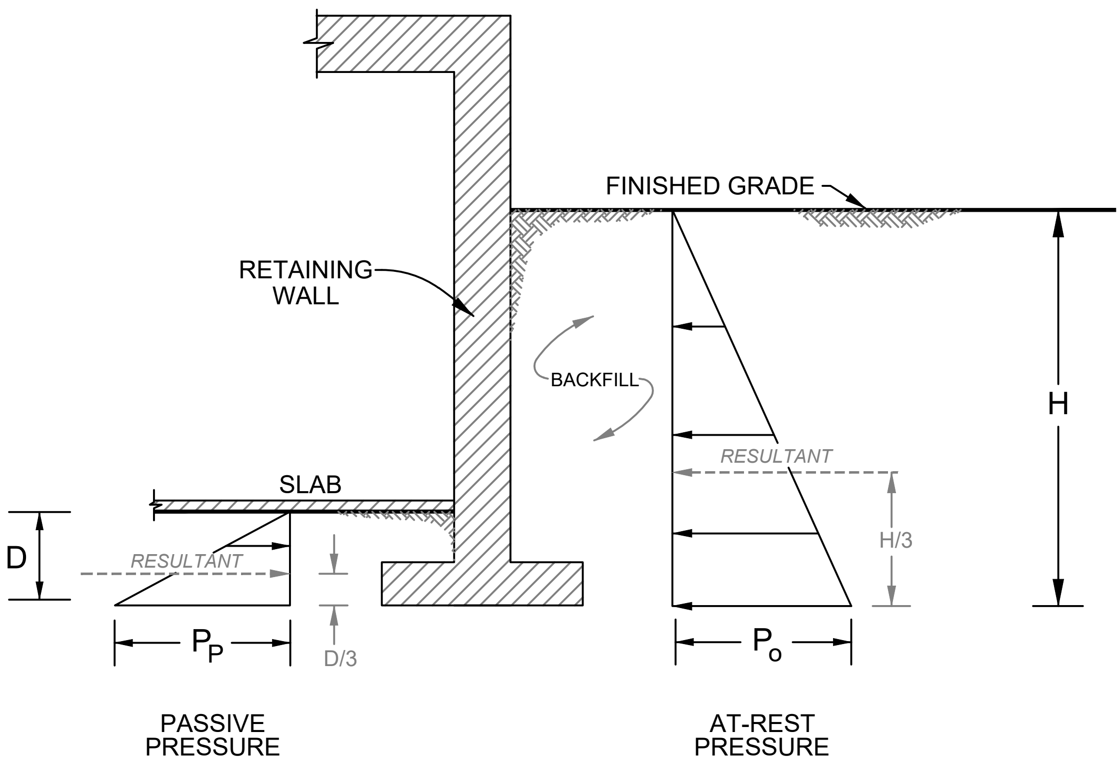

NOTES:

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 IGNORED AND DEEMED INAPPLICABLE DUE TO THE AT-REST CONDITION OF THE RETAINING WALL
5. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
6. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_O	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	56H	82H
P_P	Level Ground	2H:1V Descending Ground
	350D	140D

NOT TO SCALE

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FIGURE 6

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

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DEVIL'S PUNCHBOWL NATURE CENTER REPLACEMENT PROJECT
PEARBLOSSOM, CALIFORNIA

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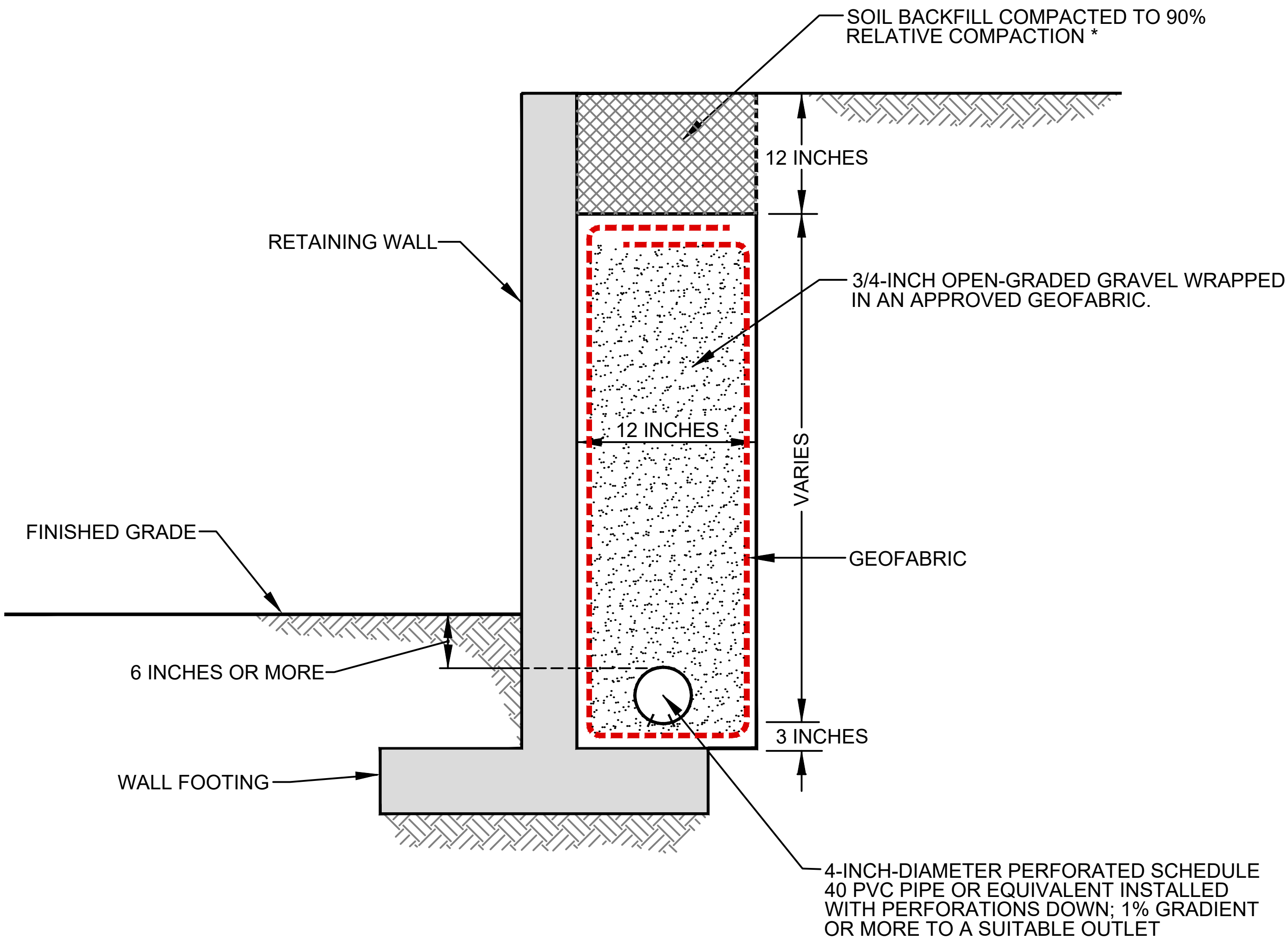

*BASED ON ASTM D1557

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NOT TO SCALE

FIGURE 7

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RETAINING WALL DRAINAGE DETAIL

DEVIL'S PUNCHBOWL NATURE CENTER REPLACEMENT PROJECT
PEARBLOSSOM, CALIFORNIA

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

Disturbed soil samples were obtained in the field using the following method.

Bulk Samples

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

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of $1^3/_8$ inches. The sampler was driven into the ground 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.

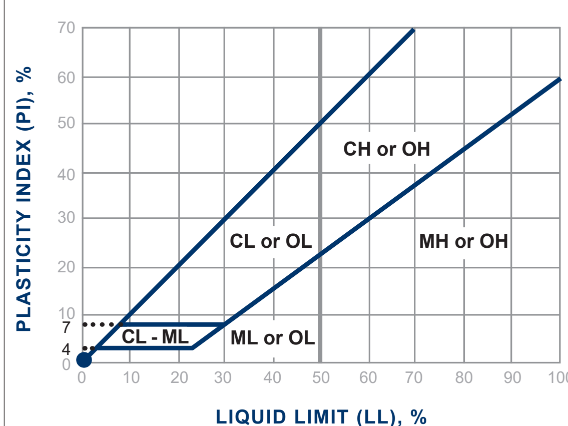
Soil Classification Chart Per ASTM D 2488

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
	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

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

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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.
								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								

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BORING LOG

DATE DRILLED 10/19/22 BORING NO. B-1

GROUND ELEVATION 4,749' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-stem Auger (2R Drilling)

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						SM	ALLUVIUM: Yellowish brown, moist, medium dense, silty SAND with gravel; trace cobbles.
		X	93	1.6	124.2			Dry; very dense.
							GP-GM	Yellowish brown, moist, very dense, poorly graded GRAVEL with silt and sand.
10		X	50/6"					
								PUNCHBOWL FORMATION: Dark yellowish brown, moist, moderately cemented, SANDSTONE; with rounded to subrounded gravel.
		X	64					
20		X	50/5"					
		X	50/4"					
								Total Depth = 25.8 feet. Groundwater not encountered during drilling. Backfilled with cement-bentonite grout on 10/19/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

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DEVIL'S PUNCHBOWL NATURE CENTER REPLACEMENT PROJECT
PEARBLOSSOM, CALIFORNIA

212036002 | 12/22

DATE DRILLED 10/19/22	BORING NO. B-2	
GROUND ELEVATION 4,751' ± (MSL)	SHEET 1	OF 1
METHOD OF DRILLING 8" Hollow-stem Auger (2R Drilling)		
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	■						SM	ALLUVIUM: Yellowish brown, moist, medium dense, silty SAND with gravel; few cobbles; trace boulders.
		■	50/6"				GP-GM	Yellowish brown, moist, very dense, poorly graded GRAVEL with silt and sand.
10		■	62	4.1	115.8			PUNCHBOWL FORMATION: Dark yellowish brown, moist, moderately cemented, SANDSTONE; with rounded to subrounded gravel.
		■	50/4"					Trace rootlets.
20		■	50/5"					
		■	50/5"					
								Total Depth = 25.9 feet. Groundwater not encountered during drilling. Backfilled with cement-bentonite grout on 10/19/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Yellowish brown, moist, medium dense, silty SAND with gravel; few cobbles.
			87					Very dense.
10			60	4.3	119.4			PUNCHBOWL FORMATION: Dark yellowish brown, moist, moderately cemented, SANDSTONE; with rounded to subrounded gravel.
			73					
20			50/5"					
			50/5" 50/6					
30								Total Depth = 26.4 feet. Groundwater not encountered during drilling. Backfilled with cement-bentonite grout on 10/19/22. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
40								

DATE DRILLED 10/19/22 BORING NO. B-3

GROUND ELEVATION 4,746' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-stem Auger (2R Drilling)

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 logs of the exploratory borings in Appendix A.

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-1.

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-2.

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 were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-3.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	10.0-11.5	POORLY GRADED GRAVEL WITH SILT AND SAND	41	7	GP-GM
B-2	5.0-6.5	POORLY GRADED GRAVEL WITH SILT AND SAND	33	8	GP-GM
B-3	5.0-6.5	SILTY SAND WITH GRAVEL	62	18	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-1

NO. 200 SIEVE ANALYSIS TEST RESULTS

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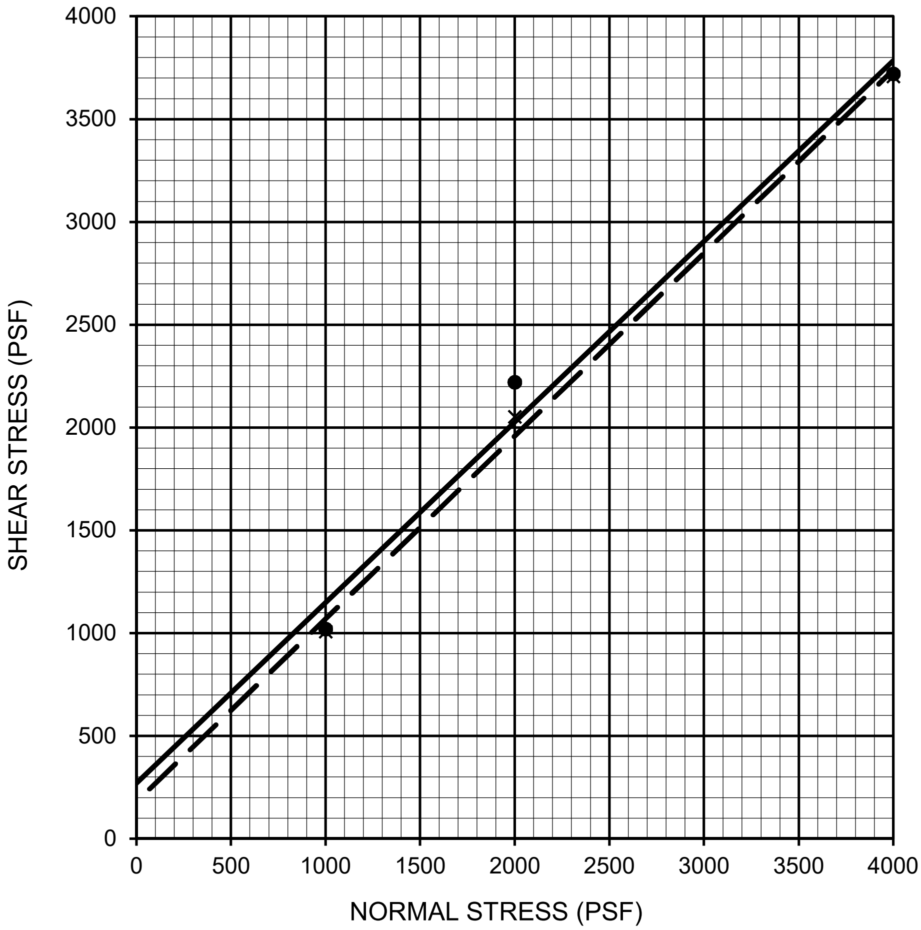

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
SILTY SAND WITH GRAVEL	———●———	B-1	5.0-6.5	Peak	270	41	SM
SILTY SAND WITH GRAVEL	— — X — —	B-1	5.0-6.5	Ultimate	180	42	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-2

DIRECT SHEAR TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2] (ppm)	SULFATE CONTENT [2] (%)	CHLORIDE CONTENT [3] (ppm)
B-1	0.0-5.0	6.5	6,941	10	0.001	80

[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-3

CORROSIVITY TEST RESULTS

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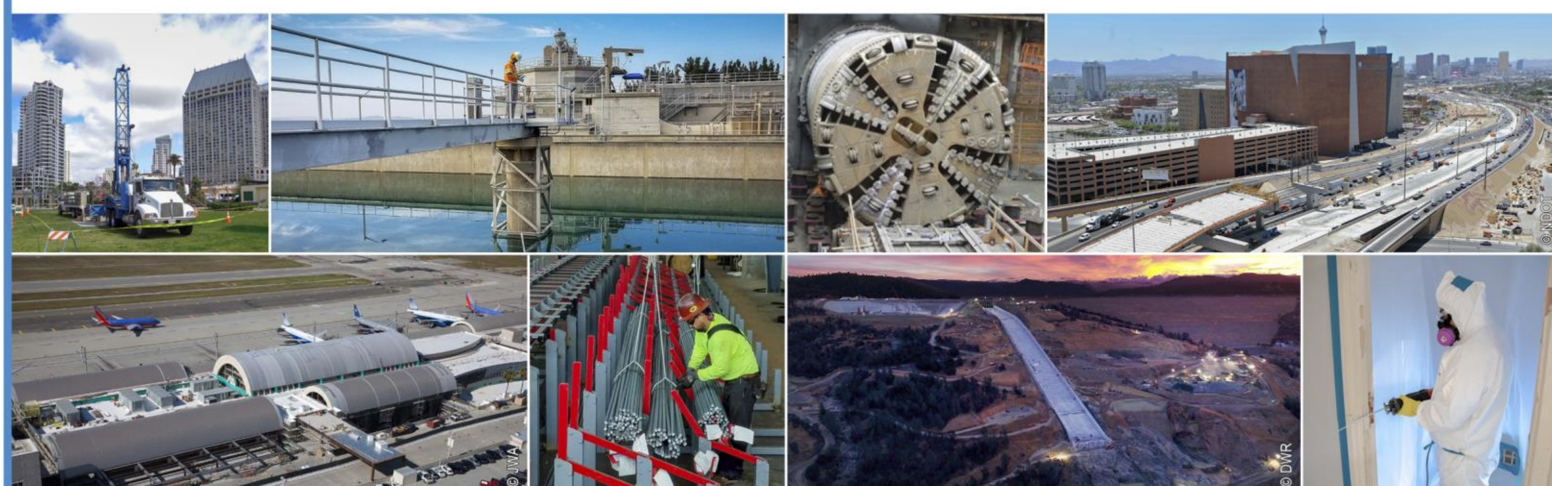

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