Geotechnical Evaluation

Mission Basin Groundwater Purification Facility

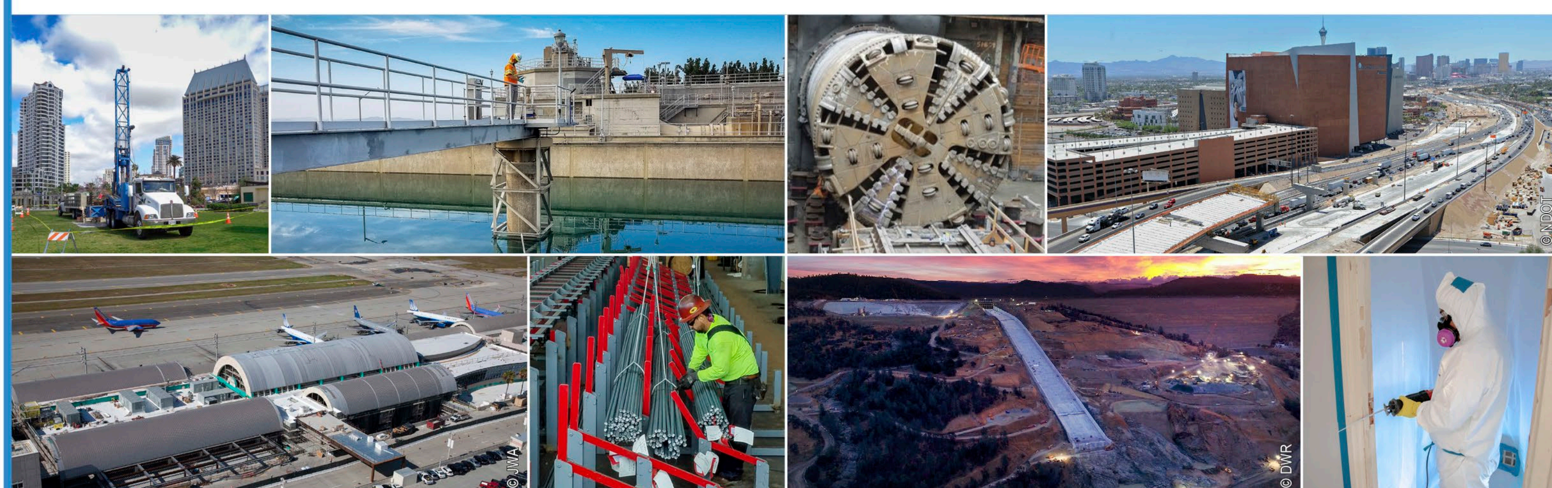
Well Expansion and Brine Minimization

Oceanside, California

GHD, Inc.

320 Goddard | Irvine, California 92618

August 30, 2022 | Project No. 109182001



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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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1. INTRODUCTION

In accordance with your authorization, Ninyo & Moore has performed a geotechnical evaluation for the City of Oceanside's Mission Basin Groundwater Purification Facility (MBGPF) Well Expansion and Brine Minimization project in Oceanside, California (Figure 1). The purpose of this geotechnical evaluation was to evaluate the general geologic conditions at the site and to develop conclusions and recommendations regarding the geotechnical aspects relating to the project. This report presents a summary of our findings and conclusions regarding the geotechnical conditions at the project site and our recommendations for the design and construction of the proposed improvements.

2. SCOPE OF SERVICES

Ninyo & Moore's scope of services for this evaluation included the following:

- Reviewing readily available published geotechnical literature, topographic and geologic maps, fault maps, and aerial photographs.
- Performing geotechnical site reconnaissance by a representative from Ninyo & Moore to observe and document the existing surface conditions at the project site. During our site reconnaissance visits, we marked our boring and cone penetrometer test (CPT) locations for utility clearance by Underground Service Alert (USA).
- Performing geotechnical logging and sampling of the upper 20 feet of soil encountered during drilling of two pilot holes (referred to herein as EX-1 and EX-2) at proposed well sites (drilling of pilot holes performed by others).
- Drilling, logging, and sampling of three small diameter exploratory borings (referred to herein as B-1 through B-3). The soil borings were drilled to depths of up to approximately 51.5 feet.
- Performing four CPT soundings (referred to herein as CPT-1 through CPT-4) using a truck-mounted CPT rig. The CPT borings were advanced to depths of up to 60.5 feet.
- Performing geotechnical laboratory testing on representative samples to evaluate their pertinent soil parameters for design and classification purposes.
- Performing limited environmental analytical testing on soil samples representative of the project areas. Generally, samples were collected from boring B-3 and CPT-4 at depths of 2 feet, 5 feet, and 10 feet.
- Performing engineering analyses of the site geotechnical conditions based on the data obtained from our background review, subsurface exploration, and laboratory testing.
- Preparing this geotechnical evaluation report describing the findings and conclusions of our study and providing recommendations for the design and construction of the proposed improvements.

3. SITE AND PROJECT DESCRIPTION

The property that includes the City of Oceanside's Mission Basin Groundwater Purification Facility (MBGPF) is located north of Mission Avenue and east of Foussat Road in Oceanside, California (Figure 1). The site consists of an irregularly-shaped parcel with the MBGPF facility located in the northern portion of the property. The southern portion of the property generally consists of vacant land. A San Diego Gas & Electric (SDGE) easement with electrical transmission lines bounds the site to the west. A residential development bounds the property to the east, and a concrete-lined drainage channel and both Fireside Park and vacant land are located to the north of the facility. An administrative building and associated parking area are located within the eastern portion of the facility, and the groundwater purification equipment is located within the western portion of the site. The MBGPF consists of a drinking water treatment facility that uses reverse osmosis technology and filters to treat brackish water generated from wells within Mission Basin. The ground surface elevation at the MBGPF site ranges from approximately 45 feet above mean sea level (MSL) in the north portion of the site to approximately 40 feet above MSL in the southern portion.

A secondary project site is located approximately 2,600 feet to the southwest of the MBGPF (Figure 1). The secondary site consists of a triangular-shaped parcel of undeveloped land partially vegetated with landscaped materials. The parcel is bounded by Mission Avenue to the south, Highway 76 to the north, the intersection of Highway 76 and Mission Avenue to the east, and the City of Oceanside Fire Department Station 7 to the west. The ground surface elevation of the secondary site is approximately 35 feet above MSL.

Based on our review of the Title XVI Feasibility for the project (Woodward & Curran, 2018), we understand that the City of Oceanside wishes to increase the capacity of the MBGPF and to increase the supply of water generated in the vicinity of the site by expanding the number of wells in Mission Basin and to add additional treatment systems at the site. As such, the City is considering installing wells in the southern portion of the property and on the secondary project site adjacent to the City of Oceanside Fire Department Station 7, and constructing additional reverse osmosis and associated treatment facilities, as well as piping and other improvements. We understand that the new structures will be constructed at grades that are similar to those that currently exist.

4. SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface exploration was conducted in multiple phases and mobilizations. The logging and sampling of the upper 20 feet of the soil profile encountered in borings EX-1 and EX-2 was performed on February 10 and 15, 2021. Borings EX-1 and EX-2 were pilot holes for planned production wells that were drilled using sonic drilling methods (under the direction of others). The drilling, logging, and sampling of two small-diameter borings (B-1 and B-2) at the MBGPF was performed on March 8, 2021. The drilling, logging, and sampling of a third small-diameter boring (B-3) was performed at the secondary project site on July 29, 2022 and four CPT soundings (CPT-1 through CPT-4) were advanced at the MBGPF on July 22, 2022. Prior to the subsurface exploration, the boring and CPT locations were cleared of underground utility conflicts by participating members of USA, by a private utility locator, and by MBGPF and/or City personnel. The purpose of the borings was to evaluate subsurface conditions and to collect soil samples for geotechnical laboratory and limited environmental analytical testing while the purpose of the CPT soundings was for the evaluation of subsurface conditions for the analysis of the liquefaction potential at the site.

Borings B-1 through B-3 were drilled to depths up to approximately 51.5 feet using a truck-mounted drill rig equipped with hollow-stem augers. Borings EX-1 and EX-2 were drilled to depths of 260 feet using a truck-mounted drill rig equipped with sonic technology. However, logging and sampling of these two borings by Ninyo & Moore was limited to the upper 20 feet. The four CPT soundings (CPT-1 through CPT-4) were advanced to depths up to approximately 60.5 feet using a truck-mounted CPT rig. During the drilling operations, Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and ASTM International (ASTM) Test Method D 2488 by observing cuttings and drive samples. Representative bulk and in-place soil samples were obtained from the borings. The samples were then transported to our in-house geotechnical laboratory for testing purposes. The approximate locations of the exploratory borings and CPT soundings are shown on Figure 2. Logs of the borings and CPT soundings are included in Appendix A.

4.1. Geotechnical Laboratory Testing

Geotechnical laboratory testing of representative soil samples included the performance of tests to evaluate in-situ moisture content and dry density, gradation, Atterberg limits, consolidation, shear strength, expansion index, modified Proctor density, soil corrosivity, and R-value. The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. Descriptions of the geotechnical laboratory test methods and the results of the other geotechnical laboratory tests performed are presented in Appendix B.

4.2. Limited Environmental Analytical Testing

Representative soil samples collected from boring B-3 at the secondary site and CPT-4 at the MGBPF site for limited environmental analytical laboratory testing were submitted to Eurofins Calscience, a State-certified environmental testing laboratory. The samples were analyzed for Total Petroleum Hydrocarbons (TPH), Volatile Organic Compounds (VOC), Pesticides, Polychlorinated Biphenyls (PCB), and Title 22 Metals. The environmental analytical laboratory results are included as Appendix C.

5. GEOLOGY

Our findings regarding regional and site geology at the project location are provided in the following sections.

5.1. Regional Geology

The project site is situated in the western portion of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 2004). The province varies in width from approximately 30 to 100 miles and generally consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The portion of the province in western San Diego County that includes the project site generally consists of uplifted and dissected coastal plain underlain by Tertiary- and Quaternary-age sedimentary rocks.

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are considered to be active. The Elsinore, San Jacinto and San Andreas faults are active fault systems located northeast of the project site and the Rose Canyon, Coronado Bank, San Diego Trough, and San Clemente faults are active faults located west of the project site (Figure 3). Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. The offshore portion of the Newport-Inglewood Fault Zone, the nearest active fault system, has been mapped approximately 7 miles west of the project site.

5.2. Site Geology

Geologic units mapped at the site and encountered during our subsurface exploration included fill soils and young alluvium (Kennedy et al., 2007). Generalized descriptions of the earth units encountered during our field reconnaissance and subsurface exploration are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the boring logs in Appendix A. The regional geologic map of the site is shown on Figure 4.

5.2.1. Fill

Fill soils were encountered in each of our exploratory borings (EX-1, EX-2, and B-1 through B-3) and our CPTs (CPT-1 through CPT-4) from the ground surface and extending to depths of up to approximately 9 feet. As encountered, the fill soils generally consisted of various shades of brown, dry to moist, loose to medium dense, silty sand, poorly graded sand, and poorly graded sand with silt. Scattered gravel, roots, and wood debris were encountered in the upper portions of the fill. Documentation regarding the placement of existing fills was not available for our review.

5.2.2. Young Alluvium

Young alluvium was encountered in our borings and CPTs below the fill soils and extending to the depths explored of approximately 60.5 feet. As encountered, the young alluvium generally consisted of various shades of brown and gray, moist to wet, loose to very dense, silt, sandy silt, silty sand, poorly graded sand, poorly graded sand with silt, well graded sand with silt, and stiff clay. Scattered gravel was observed within the young alluvium.

5.3. Groundwater

Groundwater was encountered in borings EX-1, B-1, and B-2 at approximate depths ranging between 17 and 21 feet. Seepage was encountered in boring B-3 at an approximate depth of 18.5 feet. Fluctuations in the depth to groundwater will occur due to flood events, seasonal precipitation, variations in ground elevations, subsurface stratification, irrigation, groundwater pumping, storm water infiltration, tidal and river influences, and other factors. Additionally, perched water conditions may be encountered at the site due to the presence of trench backfill and underground utilities, as these areas tend to act as a conduit for subsurface water.

5.4. Faulting and Seismicity

The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey, active faults are faults that have ruptured within

Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years), but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active and potentially active faults in the vicinity of the site and their geographic relationship to the site are shown on Figure 3.

The site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion is considered significant during the design life of the proposed structure. Based on our review of the referenced geologic maps, as well as on our site reconnaissance, no faults are mapped underlying the project site. The site is not within of a State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 2007). The nearest known active fault is the offshore portion of the Newport-Inglewood Fault, located approximately 7 miles west of the site.

Based on this information, we consider the seismic parameters associated with the closest known active fault, the Newport-Inglewood Fault, to be appropriate for design purposes. In general, hazards associated with seismic activity include strong ground motion, ground rupture, liquefaction, seismically induced settlement, and tsunamis. These hazards, along with landsliding, are discussed in the following sections.

5.4.1. Site-Specific Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2019 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures as updated by ASCE 7-16 Supplement 1 (ASCE, 2018). We calculated that the S_1 for the site is equal to 0.353g using the 2022 Structural Engineers Association of California [SEAOC]/Office of Statewide Health Planning and Development [OSHPD] seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project site.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum (ARS) curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the general MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16. The average shear wave velocity (V_S) for the upper 30 meters of soil (V_{S30}) is mapped to be 198 meters per second (m/s) (Wald and Allen, 2008) and the depths to V_S = 1,000 m/s and V_S = 2,500 m/s are assumed to be 50 meters and 230 meters, respectively (Southern California Earthquake Center [SCEC] Community Velocity Model Version 11.9.0 Basin Depth). These values were evaluated using the Open Seismic Hazard Analysis software developed by USGS and SCEC (2022c). These values were utilized in our site-specific analysis.

The 2014 new generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships that we used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The Open Seismic Hazard Analysis software developed by USGS (USGS, 2022c) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center (PEER) was used for performing the DSHA (Seyhan, 2014).

The PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the Building Seismic Safety Council 2014 Event Set (USGS, 2022b). A characteristic magnitude 7.0 event on the Newport-Inglewood (offshore) fault with a depth to top of rupture of 6.5 kilometers (km) and Joyner-Boore distance of 11.4 kilometers from the site was evaluated to be the controlling earthquake. Hence, the deterministic seismic hazard analysis was performed for the site using this event and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16 as updated by ASCE 7-16 Supplement 1 (ASCE, 2018). Figure 5 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The general mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 5 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2019 CBC, are provided in Section 7.2 for the evaluation of seismic loads on buildings and other structures. The site-specific maximum considered earthquake geometric mean (MCE_G) peak ground acceleration, PGA_M, was calculated as 0.495g.

5.4.2. Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project vicinity. Therefore, the potential for ground rupture due to faulting at the site is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

5.4.3. Liquefaction and Seismically Induced Settlement

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

According to the County of San Diego Seismic Department of Planning and Land Use (2007), the project vicinity is mapped as an area that is prone to liquefaction. As noted in the previous sections, the MBGPF and secondary sites are underlain by loose and medium dense fill soils and alluvium, and groundwater was encountered as shallow as approximately 17 feet bgs in our borings. Based on the proposed improvements, we evaluated the liquefaction potential at the MBGPF site using the data obtained from the CPT soundings. Liquefaction evaluation was performed using a characteristic magnitude of 7.0 associated with the Newport-Inglewood (offshore) fault and MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) of 0.495g as discussed in previous sections. A groundwater depth of 15 feet was used in our analysis based on our subsurface exploration. The liquefaction analysis was performed using the computer program LiquefyPro (CivilTech Software, 2007). The analysis was based on the National Center for Earthquake Engineering Research (NCEER) procedure (Youd, et al., 2001) using Modified Robertson Method (1997).

5.4.3.1. Dynamic Settlement of Saturated Soils

As a result of liquefaction, the proposed improvements may be subject to several hazards, including liquefaction-induced settlement. In order to estimate the amount of post-earthquake settlement, the method proposed by Tokimatsu and Seed (1987) was used in which the seismically induced cyclic stress ratios and corrected N-values are related to the volumetric strain of the soil. The amount of soil settlement during a strong seismic event depends on the thickness of the liquefiable layers and the density and/or consistency of the soils. Based on our knowledge and experience with the site soils, the occasional clay layers that were encountered are not sensitive enough to undergo cyclic strain softening.

We evaluated dynamic settlement at the site using the LiquefyPro software (CivilTech Software, 2007). Based on our evaluation, which assumes a peak ground acceleration of 0.495g and a characteristic magnitude of 7.0, a post-earthquake total settlement of up to approximately 5 inches is estimated for the MBGPF site (Appendix D). Based on the guidelines presented in CGS Special Publication 117A (2008) and assuming relatively uniform subsurface stratigraphy across the site, we estimate differential settlement to be approximately one-half of the total settlement over a horizontal distance of approximately 40 feet (i.e., on the order of 2.5 inches over a horizontal distance of approximately 40 feet). Some of the dynamic settlement can be reduced by remedial grading as described in the Recommendations section of this report.

Based on the proximity of the secondary site to the MBGPF site, as well as the similar subsurface conditions, we anticipate dynamic settlements at the secondary site to also occur. However, given the proposed improvements at the secondary site (i.e., well), design considerations for such settlements are not expected to be necessary. If site-specific dynamic settlement analysis is requested by the designer, additional subsurface exploration and/or evaluation can be performed.

5.4.3.2. Lateral Spread

Lateral spreading of ground surface during an earthquake usually takes place along weak shear zones that have formed within a liquefiable soil layer. Lateral spread has generally been observed to take place in the direction of a free-face (i.e., retaining wall, slope, channel) but has also been observed to a lesser extent on ground surfaces with very gentle slopes. An empirical model developed by Bartlett and Youd (1995, revised 1999) is typically used to predict the amount of horizontal ground displacement within a site. For a site located in proximity to a free-face, the amount of lateral ground displacement is strongly correlated with the distance of the site from the free-face. Other factors such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also affect the amount of lateral ground displacement.

The project site is relatively flat and the nearest channel free face is located approximately 1,200 feet west of the site. Accordingly, lateral displacement is not a design consideration for the MBGPF facility.

Based on the proximity of the secondary site to the MBGPF site, as well as the similar subsurface conditions, we also anticipate lateral displacement at the secondary site to not be a design consideration.

5.4.3.3. Surface Manifestation of Liquefaction

Based on the design curves developed by Ishihara (1985) the potential for surface manifestation of liquefaction (i.e., ground subsidence, sand boils, and/or seismically induced bearing failure) at the MBGPF and secondary sites is considered low.

5.4.4. Tsunamis

Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. According to the tsunami inundation map for the Oceanside and San Luis Rey Quadrangles (California Emergency Management Agency, 2009), the project site is mapped as lying outside of tsunami inundation areas. Based on our review of the tsunami inundation map, along with the site's distance from the Pacific Ocean, the potential for damage due to tsunamis is considered unlikely.

5.4.5. Landsliding

Per Tan and Giffen (1995), the site is mapped as “marginally susceptible” to landsliding. Based on our review of referenced geologic maps, literature, and topographic maps, and on our site reconnaissance and subsurface exploration, landslides or indications of deep-seated landsliding do not underlie the project site. In our opinion, the potential for significant large-scale slope instability at the site is not a design consideration.

5.5. Flood Hazards

Based on review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM), the site is located within a mapped flood zone (FEMA, 2012). The site is in an area mapped as Zone A99, which represents areas “subject to inundation by the 1-percent-annual-chance flood event, but which will ultimately be protected upon completion of an under-construction Federal flood protection system.”

6. CONCLUSIONS

Based on the results of our geotechnical evaluation, the following conclusions are provided for the proposed project:

- The site is generally underlain by fill soils and young alluvium.
- Groundwater was encountered in our borings at the site at approximate depths ranging between 17 and 21 feet.
- The onsite fill and young alluvial soils at the site are anticipated to be generally excavatable using heavy duty earthmoving equipment in good working condition.
- There are no known active faults crossing the project site and the potential for surface ground rupture is considered low. The offshore portion of the Newport-Inglewood Fault is mapped approximately 7 miles west of the project site. The potential for relatively strong seismic ground motions should be considered in the project design.
- The site is susceptible to liquefaction with total seismic settlements estimated to be up to approximately 5 inches.
- Results of our laboratory testing indicate that the upper soils at the MBGPF site possess a very low expansion potential.
- Based on the results of our limited geotechnical laboratory testing when compared to the Caltrans (2021) corrosion guidelines, the onsite soils are not considered corrosive.

7. RECOMMENDATIONS

Based on the results of our subsurface evaluation and our understanding of the proposed construction, we present the following general geotechnical recommendations relative to the design and construction of the proposed pressure reducing stations and ancillary improvements. Based on our understanding of the project, the following recommendations are provided for the design and construction of the proposed project.

7.1. Earthwork

In general, earthwork should be performed in accordance with the recommendations presented in this report. Ninyo & Moore should be contacted for questions regarding the recommendations or guidelines presented herein.

7.1.1. Site Preparation

Prior to performing site excavations, the project alignment should be cleared of vegetation, surface obstructions, rubble and debris, abandoned utilities and foundations, 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 of at a legal dumpsite.

7.1.2. Remedial Earthwork

Our subsurface exploration at the MBGPF site indicates that existing fills having thicknesses of up to 9 feet are present. Documentation regarding the placement of the existing fills was not available for our review. Consequently, we consider the existing fill to be potentially compressible in its current condition. In addition, the project site is underlain by alluvial soils that are susceptible to liquefaction during a seismic event. Our evaluation indicates that total liquefaction-induced seismic settlement on the order of 5 inches, with differential settlement on the order of 2.5 inches over a horizontal distance 40 feet, could occur. In order to mitigate the compressible nature of the existing fill materials along with some of the adverse effects associated with liquefaction (i.e., loss of bearing, excessive differential settlement, sand boils, etc.), we recommend that the existing fill that underlies proposed buildings and other settlement-sensitive structures within the MBGPF site be removed down to the alluvial materials and be replaced with compacted fill. Based on our subsurface exploration, we anticipate that the existing fill extends to depths of 9 feet or more in the area of the MBGPF where structures are planned.

In addition, we recommend that settlement-sensitive structures be underlain by 5 feet or more of fill that is reinforced with geosynthetic material (geogrid) to help mitigate against the adverse effects of liquefaction at the site. To accomplish this, the fill soils, and upper portions of the alluvium where existing fills are less than 5 feet in thickness, should be removed. As such, the removals should extend to depths where the existing fill is removed, or 5 feet below the bottom of the proposed building foundation (whichever is deeper). For the purpose of this report, structural building areas are defined as the areas underlying the buildings and extending a horizontal distance of 10 feet beyond the footprints of the structures. The resultant removal surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM International (ASTM) Test Method D 1557.

Subsequent to the scarification and recompaction of the bottom of the overexcavation, compacted fill should be placed within the overexcavation to finish grade. These materials should be compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Materials for use as fill should be evaluated by Ninyo & Moore's representative prior to filling or importing. A layer of Tensar TriAx TX-7 geogrid (or equivalent) should be placed at a depth of 5 feet below finish grade within the soils placed as backfill. As placement of the fill materials within the overexcavated area proceeds, 2 additional layers of Tensar TriAx TX-7 geogrid (or equivalent) should be placed within the compacted fill. The additional layers of geogrid should be placed with a vertical spacing of approximately 1 foot within the fill (i.e. geogrid placed at approximate depths of 5 feet, 4 feet, and 3 feet below finish grade) and should extend across the width and length of the overexcavation. The penetrations of pipelines and other conduits through the geogrid reinforcement (where necessary) should be made in accordance with manufacturer guidelines.

7.1.3. Excavation Characteristics

The results of our field exploration program indicates that the site is underlain by fill soils and young alluvium. Excavation of the fill soils and young alluvium should be feasible with heavy-duty excavation equipment in good working condition. Due to the variable nature of the existing fill materials along with some portions of the alluvium, the contractor should anticipate caving and/or sloughing conditions when performing excavations due to the presence of loose soil.

7.1.4. Temporary Excavations and Shoring

For temporary excavations, we recommend that the following Occupational Safety and Health Administration (OSHA) soil classifications be used:

Fill and Young Alluvium	*Type C*

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1.5:1 (horizontal to vertical) in fill soils and young alluvium. Temporary excavations that encounter seepage may be shored or stabilized by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. Onsite safety of personnel is the responsibility of the contractor.

In areas with limited space for construction (where temporary excavations may not be laid back at the recommended slope inclination), a temporary shoring system may be utilized to support the excavation sidewalls during construction. The shoring system should be designed using the magnitudes and distributions of the lateral earth pressures presented on Figure 6 for braced shoring and Figure 7 for cantilevered shoring. The recommended design earth pressures are based on the assumptions that (a) the shoring system is constructed without raising the ground surface elevation behind the shoring, (b) that there are no surcharge loads, such as soil stockpiles, construction materials, or vehicular traffic, and (c) that no loads act above a 1:1 plane extending up and back from the base of the shoring system. For shoring subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on lateral earth pressures acting on the shoring wall.

Settlement of the ground surface may occur behind the shoring wall during excavation. The amount of settlement depends on the type of shoring system, the quality of contractor's workmanship, and soil conditions. Settlement may cause distress to adjacent structures, if present. To reduce the potential for distress to adjacent structures, we recommend that the shoring system be designed to limit the ground settlement behind the shoring to ½ inch or less. Possible causes of settlement that should be addressed include vibration during installation of the sheet piling, excavation for construction, construction vibrations, dewatering, and removal of the support system. We recommend that the potential settlement distress be evaluated carefully by the contractor prior to construction.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are for preliminary design purposes and the contractor should evaluate the adequacy of these parameters and make 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. We further recommend that the construction methods provided herein be carefully evaluated by a qualified specialty contractor prior to commencement of the construction.

7.1.5. Excavation Bottom Stability

If unstable excavation bottom conditions are exposed (particularly during remedial earthwork, or during backfilling of utility trenches), they may be mitigated by over excavating the excavation bottom to suitable depths and replacing with a layer of ¾- to 1½-inch crushed gravel encased in a woven geotextile (e.g., Mirafi® 600X geotextile or an approved equivalent). Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

7.1.6. Materials for Fill

Materials for fill (including materials utilized as backfill for excavations) may be processed from on-site excavations, or may consist of import materials. On-site soils with an organic content of less than approximately 3 percent by volume (or 1 percent by weight) are suitable for reuse as general fill material. Fill soils should be free of trash, debris, roots, vegetation, organics, or other deleterious materials. Fill and utility trench backfill materials should not contain rocks or lumps over 3 inches, and not more than 30 percent larger than ¾ inch. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite.

Imported fill material, if needed, should meet the criteria described above and be granular soil possessing a low or very low expansion potential (i.e., an EI of 50 or less as evaluated by ASTM D 4829). Imported materials should also be non-corrosive in accordance with the Caltrans (2021) corrosion guidelines. Materials for use as fill should be evaluated by the geotechnical consultant's representative prior to filling or importing.

7.1.7. Compacted Fill

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 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.

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. The upper 12 inches of the subgrade materials beneath vehicular pavements should be compacted to a relative compaction of 95 percent relative density as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

7.1.8. Modulus of Soil Reaction

We anticipate that trenching operations will be used on this project. The modulus of soil reaction *(E')* is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipelines for the purpose of evaluating deflection caused by the weight of the backfill above the pipe. For pipelines constructed in granular fill soils, we recommend that a modulus of soil reaction of 1,200 pounds per square inch (psi) be used for design for 0 to 5 feet deep excavations and 1,800 psi for excavations exceeding 5 feet depth, provided that granular bedding material is placed adjacent to the pipe, as recommended in the following section of this report.

7.1.9. Pipe Bedding

We recommend that new pipelines, where constructed in an open excavation, be supported on 6 or more inches of granular bedding material. Granular pipe bedding should be provided to distribute vertical loads around the pipe. Bedding material and compaction requirements should be in accordance with this report. Pipe bedding typically consists of graded aggregate with a coefficient of uniformity of three or greater.

7.1.10. Pipe Zone Backfill

The pipe zone backfill extends from the top of the pipe bedding material and continues to extend to 1 foot or more above the top of the pipe in accordance with the recent edition of the Standard Specifications for the Public Works Construction ("Greenbook"). Pipe zone backfill should have a Sand Equivalent (SE) of 30 or greater, and be placed around the sides and top of the pipe. Special care should be taken not to allow voids beneath and around the pipe. Compaction of the pipe zone backfill should proceed up both sides of the pipe.

It has been our experience that the voids within a crushed rock material 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. If open-graded gravel is utilized as pipe zone backfill, this material should be separated from the adjacent trench sidewalls and overlying trench backfill with a geosynthetic filter fabric.

7.1.11. Trench Zone Backfill

Trench zone backfill should consist of granular soil that conforms to the most recent edition of the Standard Specifications for the Public Works Construction ("Greenbook"). In general, the material should be comprised of low-expansion-potential granular soil and should be free of trash, debris, roots, vegetation, or deleterious materials. Fill should generally be free of rocks or hard lumps of material in excess of 3 inches in diameter. Rocks or hard lumps larger than about 3 inches in diameter should be broken into smaller pieces or should be removed from the site. Wet materials generated from on-site excavations should be aerated to a moisture content near the laboratory optimum to allow compaction.

Imported materials should consist of clean, granular materials with a low expansion potential, corresponding to an expansion index of 50 or less as evaluated in accordance with ASTM D 4829. The corrosion potential of proposed imported soils should also be evaluated if structures will be in contact with the imported soils. Import material should be submitted to the geotechnical consultant for review prior to importing to the site. The contractor should be responsible for the uniformity of import material brought to the site.

7.1.12. Lateral Pressures for Thrust Blocks

Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the lateral passive earth pressures presented on Figure 8. Thrust blocks should be backfilled with granular backfill material and compacted in accordance with recommendations presented in this report.

7.1.13. Drainage

Roof, pad, and slope drainage should be directed such that runoff water is diverted away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.). Positive drainage adjacent to structures should be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside building perimeters, and further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained over the pad area and drainage patterns should be established to divert and remove water from the site to appropriate outlets.

Care should be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of final grading should be maintained for the life of the project. The property owner and the maintenance personnel should be made aware that altering drainage patterns might be detrimental to foundation performance.

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

Table 1 – 2019 California Building Code Seismic Design Criteria	
Site Coefficients and Spectral Response Acceleration Parameters	**Values**
Site Class	D
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	0.956g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.353g
Site-Specific Spectral Response Acceleration at 0.2-second Period, S_{MS}	1.137g
Site-Specific Spectral Response Acceleration at 1.0-second Period, S_{M1}	0.740g
Site-Specific Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.758g
Site-Specific Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.493g
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.495g

7.3. Foundations – Buildings and Settlement-Sensitive Structures

As described earlier, the site is underlain by alluvial soils that are susceptible to liquefaction during a seismic event. Our analyses indicate that the site may undergo up to approximately 5 inches of total seismically-induced settlement, with a differential settlement on the order of 2.5 inches over a horizontal distance of 40 feet, during the design seismic event. The intent of the recommendations herein is to mitigate the effects of liquefaction such that buildings and settlement-sensitive structures at the MBGPF site would not be susceptible to collapse, however it may be susceptible to damage during the design earthquake and would potentially be in need of repair.

Accordingly, we are providing recommendations to support proposed buildings and settlement-sensitive improvements building on a mat foundation underlain by reinforced fill (recommended in Section 7.1.2. Design of foundations should also be in accordance with structural considerations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of structures. In the event the structural are categorized such that they are to withstand the liquefaction effects during the design seismic event, additional recommendation can be provided for the use of deep foundation systems or the implementation of ground improvement techniques.

7.3.1. Mat Foundations

As noted above, a reinforced concrete mat foundation supported on a reinforced fill is recommended. For the design of a mat foundation bearing on a 5-foot-thick reinforced fill mat above groundwater, a net allowable bearing pressure of 2,000 pounds per square foot (psf) may be used. This allowable bearing capacity may be increased by one-third when considering loads of a short duration such as wind or seismic forces. We recommend that mats be constructed with an embedment of 18 inches or more. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of the project structural engineer.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils directly underlying the mat. A design modulus of subgrade reaction (K) of 200 pounds per cubic inch (pci) should be used for the reinforced fill mat above groundwater when evaluating such deflections. This value is based on a unit square foot area and should be adjusted for large mats. Adjusted values of the modulus of subgrade reaction, Kv, can be obtained from the following equation for mats of various widths:

$$\mathbf{K_v = K[(B+1)/2B]^2 \quad (pci)}$$

The B in the above equation represents the width (i.e., the lesser dimension of the width and length) of the mat in feet.

For frictional resistance to lateral loads on mat, we recommend a coefficient of friction of 0.3 at the concrete-soil interface. For a mat with an embedment depth shallower than 18 inches, passive earth pressure should be ignored while evaluating lateral resistance; only frictional resistance should be considered. For mats with embedment depths greater than 18 inches, passive earth pressure equal to an equivalent fluid weight of 300 pounds per cubic foot (pcf) may be combined with frictional resistance to evaluate the total lateral resistance. The passive earth pressure should be considered to be applied at depths greater than 18 inches. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces. These recommendations assume no moisture-sensitive floor covering are planned for the proposed buildings at the site.

7.3.2. Static Settlement

We estimate that the proposed structures, designed and constructed as recommended herein, will undergo total static settlement of less than approximately 1 inch and differential static settlement of approximately ½ inch over a horizontal distance of 40 feet. Note, this does not include the seismically-induced settlements resulting from liquefaction during the design seismic event.

7.4. Underground Structures

Underground structures may be designed for lateral pressures represented by the pressure diagram on Figure 9. For preliminary design purposes, we recommend that the groundwater level be assumed to be at a depth of 15 feet or more below the ground surface for evaluation of lateral pressures and calculating the factor of safety against uplift. It is recommended that the exterior of underground walls and horizontal and vertical construction joints be waterproofed, as indicated by the project civil engineer and/or architect. For pipe wall penetrations into the lift station, vaults, and other structures, standard "water-tight" penetration design should be utilized. To reduce the potential for relative pipe to wall differential settlement, which could cause pipe shearing, we recommend that a pipe joint be located close to the exterior of the wall. The type of joint should be such that relative movement across the joint can be accommodated without distress.

7.5. Exterior Concrete Flatwork

Exterior concrete flatwork should be 5 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on-center both ways. A vapor retarder is not needed for exterior flatwork. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer. Before placement of concrete, the subgrade soils should be scarified to a depth of 8 inches, moisture conditioned to generally above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557.

7.6. Preliminary Flexible Pavement Design

As part of the new construction, we anticipate that new pavements will be constructed to provide access to the buildings and site. Our laboratory testing of a near surface soil sample at the project site indicated a R-value of 48. For planning purposes, our preliminary pavement design has utilized a design R-Value of 40. This R-value, along with assumed design Traffic Indices (TI) of 5, 6, 6.5, and 7 has been the basis of our preliminary flexible pavement design. The assumed TIs should be evaluated by the Civil Engineer based on anticipated traffic loading at the site. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement sections are presented in Table 2.

Table 2 – Recommended Preliminary Flexible Pavement Sections

Traffic Index (Pavement Usage)	Design R-Value	Asphalt Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
5 (Parking Areas)	40	3	6
6 (Driveways)	40	3.5	6
7 (Fire Lanes)	40	4	7

As indicated, the pavement structural sections recommended above assume TIs of 7.0 or less for site pavements. If traffic loads are different from those assumed herein, the pavement design should be re-evaluated. We recommend that the upper 12 inches of the subgrade be compacted to a relative compaction of 95 percent of the modified Proctor density as evaluated by the current version of ASTM D 1557. Additionally, the aggregate base materials should be compacted to a relative compaction of 95 percent of the modified Proctor density as evaluated by the current version of ASTM D 1557. The AC should be compacted to a relative compaction of 95 percent of Hveem unit weight.

Where rigid pavement sections are proposed, we recommend 7 inches of Portland cement concrete underlain by 4 inches of compacted aggregate base. We recommend that the Portland cement concrete have a 600 pounds per square inch (psi) flexural strength and that it be reinforced with No. 3 bars that are placed 18 inches on center (both ways). The rigid pavement and aggregate base should be placed on compacted subgrade that is prepared in accordance with the recommendations presented above.

7.7. Corrosivity

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

The results of the corrosivity testing of the two samples indicated electrical resistivities of 1,900 ohm-centimeters (ohm-cm) and 11,400 ohm-cm, soil pH of 7.0 and 7.1, chloride contents of 20 parts per million (ppm) and 55 ppm, and sulfate contents of 0.001 percent and 0.011 percent (i.e., 10 ppm and 110 ppm). Based on a comparison with the Caltrans corrosion (2021) criteria, the onsite soils would not be classified as corrosive. Corrosive soils are defined as soil with an electrical resistivity less than 1,100 ohm-cm, a chloride content more than 500 ppm, more than 0.15 percent sulfates (1,500 ppm), and/or a pH less than 5.5.

7.8. 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. The soil samples tested in this evaluation indicated water-soluble sulfate contents of 0.001 percent and 0.011 percent by weight (i.e., about 10 ppm and 110 ppm). Based on the American Concrete Institute (ACI) 318 criteria, the site soils would correspond to exposure class S0. For this exposure class, ACI 318 recommends that normal weight concrete in contact with soil possess a compressive strength of 2,500 pounds per square inch (psi) or more. Due to the potential for variability of site soils, we recommend that normal weight concrete in contact with soil use Type II, II/V, or V cement.

7.9. Pre-Construction Conference

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the plans, the project, and the proposed construction schedule.

7.10. Plan Review and Construction Observation

The conclusions and 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 one exploratory boring. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

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 it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client 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. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

8. 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.

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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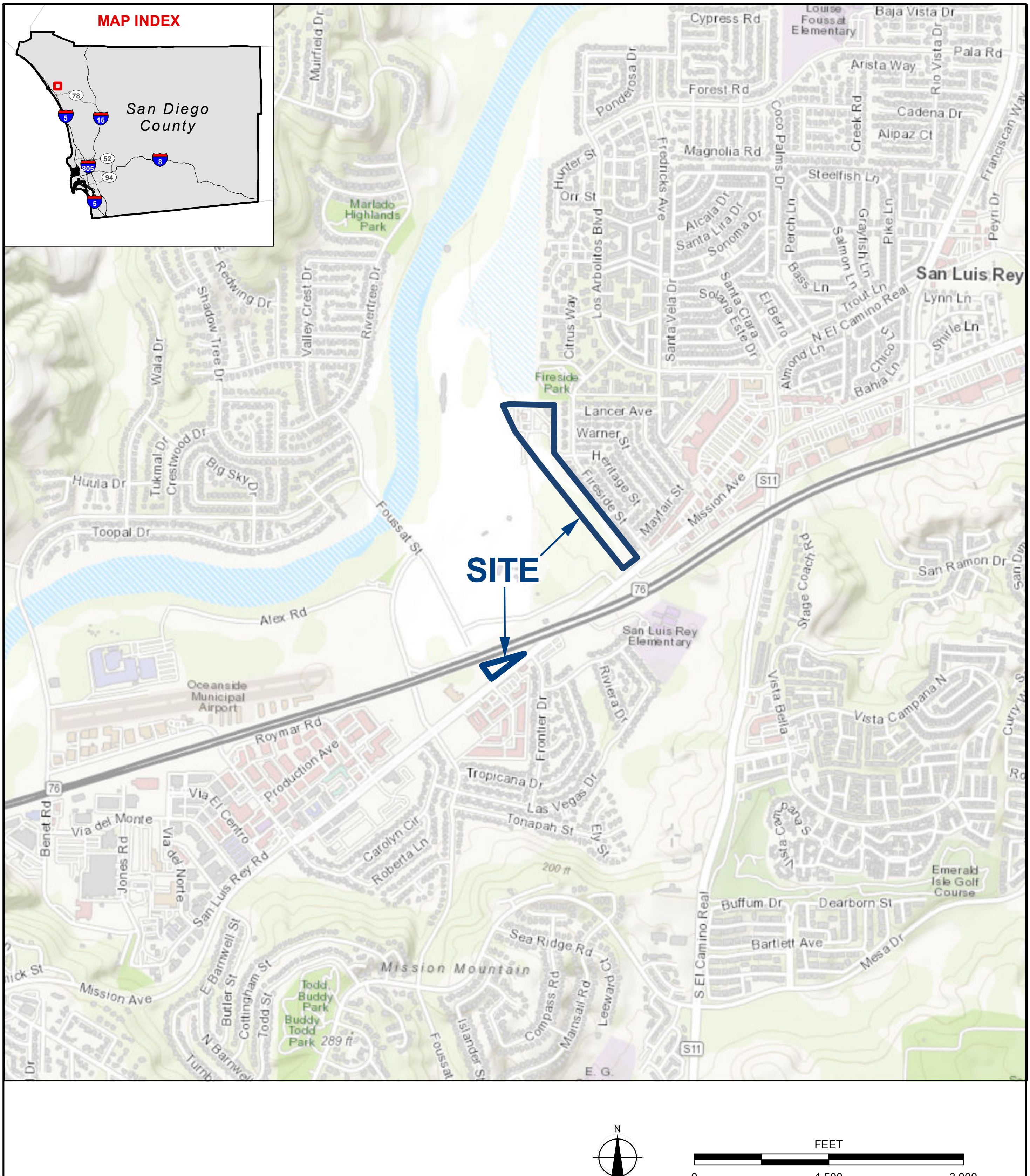
FIGURES



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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2021

FIGURE 1

SITE LOCATION

MBGPF WELL EXPANSION AND BRINE MINIMIZATION
OCEANSIDE, CALIFORNIA

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LEGEND

SITE BOUNDARY

B-3 TD=20.0 BORING
TD=TOTAL DEPTH IN FEET

CPT-4 TD=60.0 CONE PENETRATION TEST
TD=TOTAL DEPTH IN FEET

EX-2 TD=20.0 BOREHOLE (LOGGED AND SAMPLED BY NINYO & MOORE)
TD=TOTAL DEPTH IN FEET

N

FEET 0 500 1,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2022

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FIGURE 2

EXPLORATION LOCATIONS

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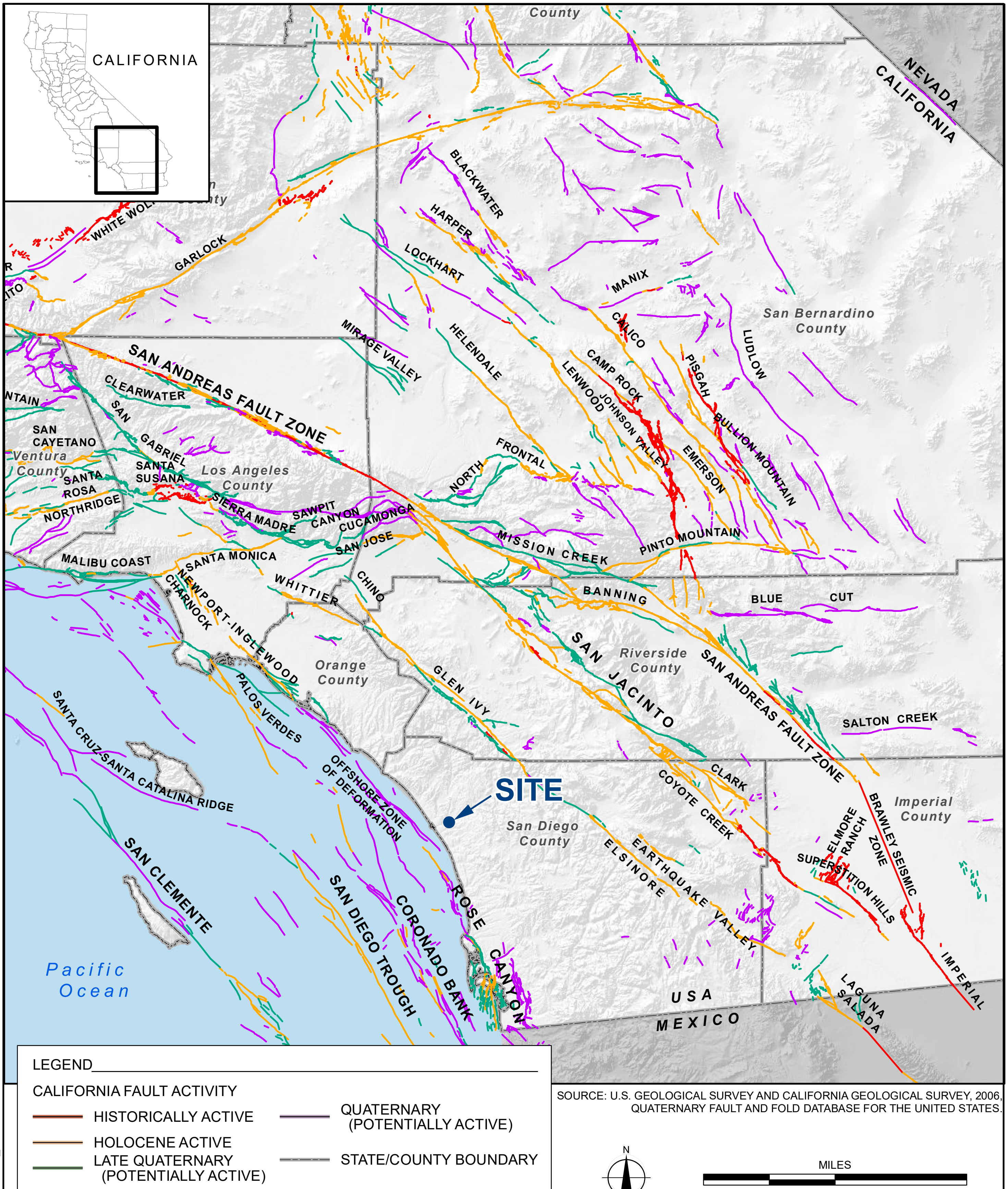

SOURCE: U.S. GEOLOGICAL SURVEY AND CALIFORNIA GEOLOGICAL SURVEY, 2006, QUATERNARY FAULT AND FOLD DATABASE FOR THE UNITED STATES.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 3

FAULT LOCATIONS

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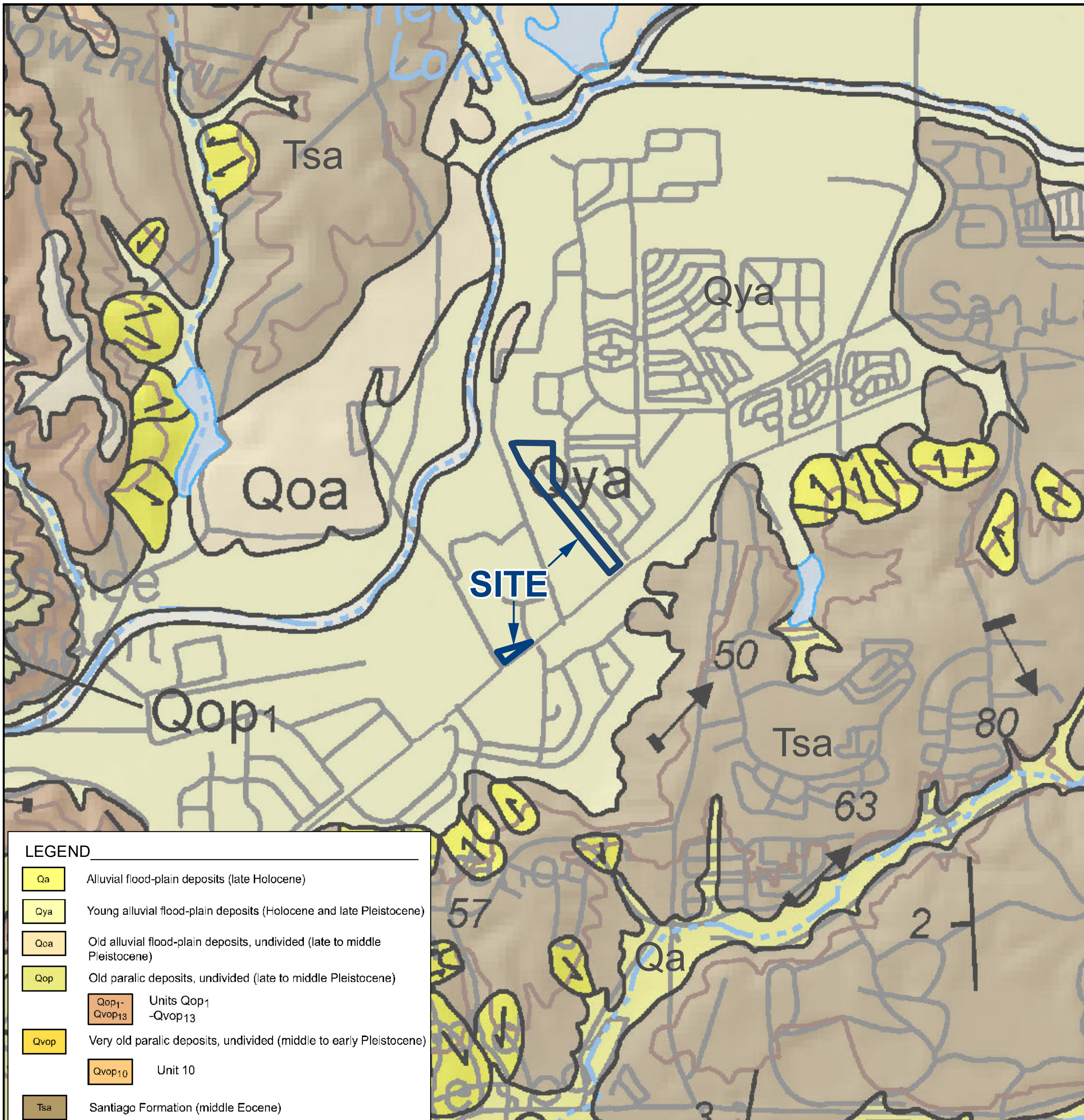

LEGEND

Qa	Alluvial flood-plain deposits (late Holocene)
Qya	Young alluvial flood-plain deposits (Holocene and late Pleistocene)
Qoa	Old alluvial flood-plain deposits, undivided (late to middle Pleistocene)
Qop	Old paralic deposits, undivided (late to middle Pleistocene)
Qop_1-$Qvop_{13}$	Units Qop_1 -$Qvop_{13}$
Qvop	Very old paralic deposits, undivided (middle to early Pleistocene)
$Qvop_{10}$	Unit 10
Tsa	Santiago Formation (middle Eocene)
Qls?	Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.
70 U D	Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.
70	Strike and dip of beds

REFERENCE: KENNEDY, M.P., TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30 X 60-MINUTE QUADRANGLE, CALIFORNIA

N

FEET

0 2,000 4,000

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 4

GEOLOGY

MBGPF WELL EXPANSION AND BRINE MINIMIZATION
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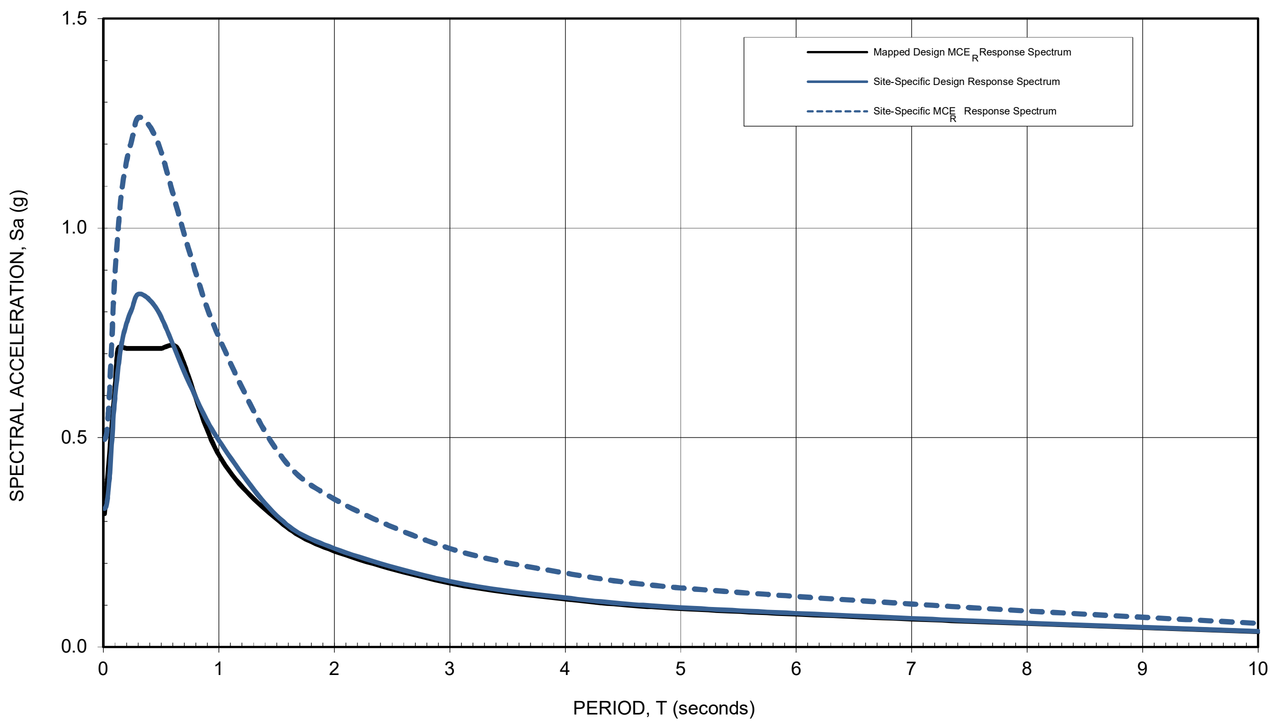
PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.496	0.330
0.020	0.500	0.333
0.030	0.513	0.342
0.050	0.589	0.393
0.075	0.745	0.496
0.100	0.889	0.593
0.150	1.067	0.711
0.200	1.158	0.772
0.250	1.214	0.809
0.300	1.263	0.842
0.400	1.243	0.829

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	1.182	0.788
0.750	0.939	0.626
1.000	0.740	0.493
1.500	0.471	0.314
2.000	0.353	0.235
3.000	0.235	0.157
4.000	0.177	0.118
5.000	0.141	0.094
7.500	0.094	0.063
10.000	0.056	0.038

S_{DS} = 0.758 g	S_{D1} = 0.493 g	S_{MS} = 1.137 g	S_{M1} = 0.740 g	PGA_M = 0.495 g



NOTES:

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

2 The deterministic ground motion spectral response accelerations are for the 84th percentile of the geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.0 event on the Newport-Inglewood (offshore) fault zone located 13.10 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

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

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

FIGURE 5

ACCELERATION RESPONSE SPECTRA

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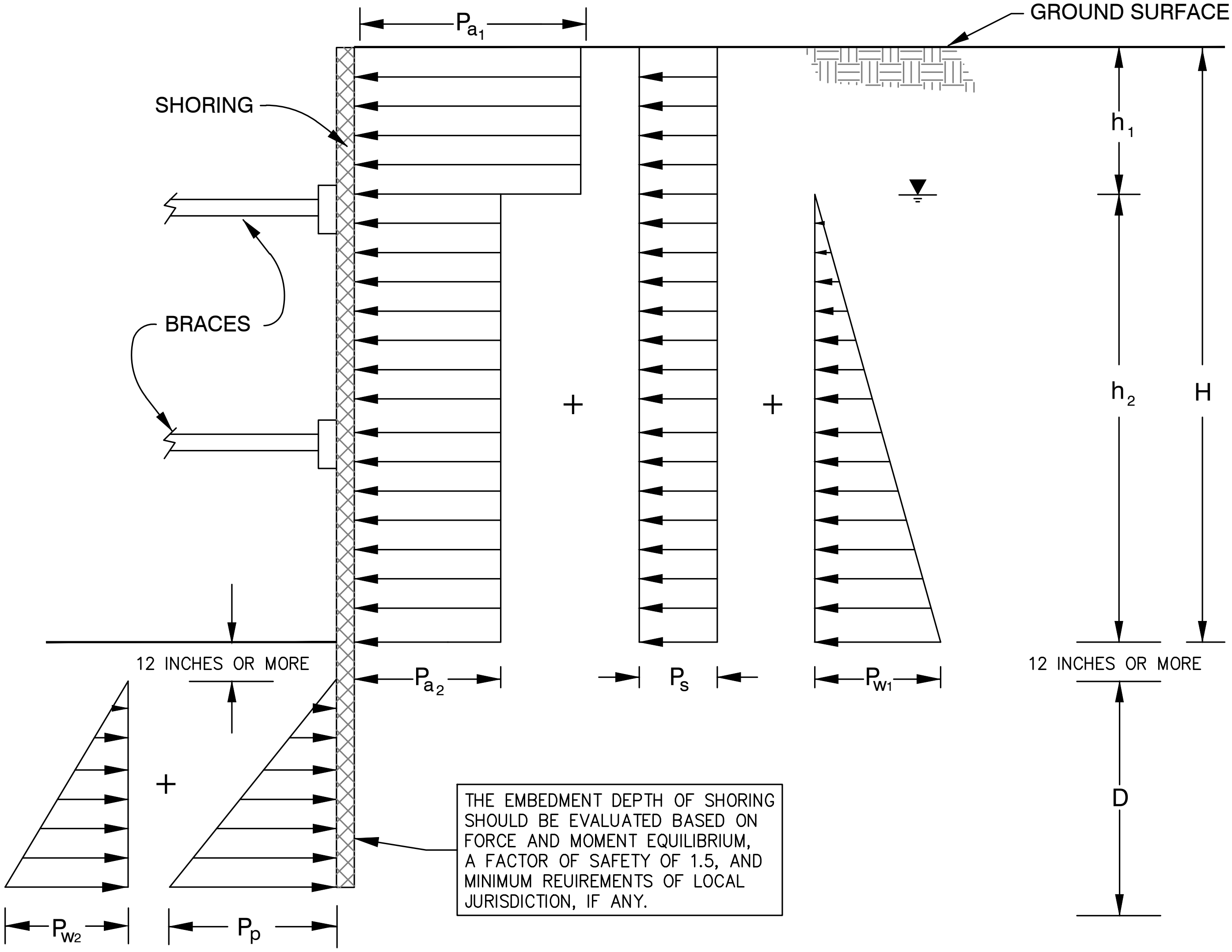

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{a1} AND P_{a2}
   $P_{a1} = 29h_1$ psf
   $P_{a2} = 14h_2$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   $P_s = 120$ psf
3. HYDROSTATIC PRESSURE, P_{w1} & P_{w2}
   $P_{w1} = 62.4(H - h)$ psf
   $P_{w2} = 62.4D$ psf
4. PASSIVE PRESSURE, P_p
   $P_p = 160D$ psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H, h_1, h_2 AND D ARE IN FEET
7. ▽ GROUNDWATER TABLE
8. P_{w2} IS APPLICABLE WHEN DEWATERING IS TO BE PERFORMED FROM INSIDE OR OUTSIDE OF THE EXCAVATION

NOT TO SCALE

109182001 DETAIL FIGS 22 08.DWG AOB

FIGURE 6

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION

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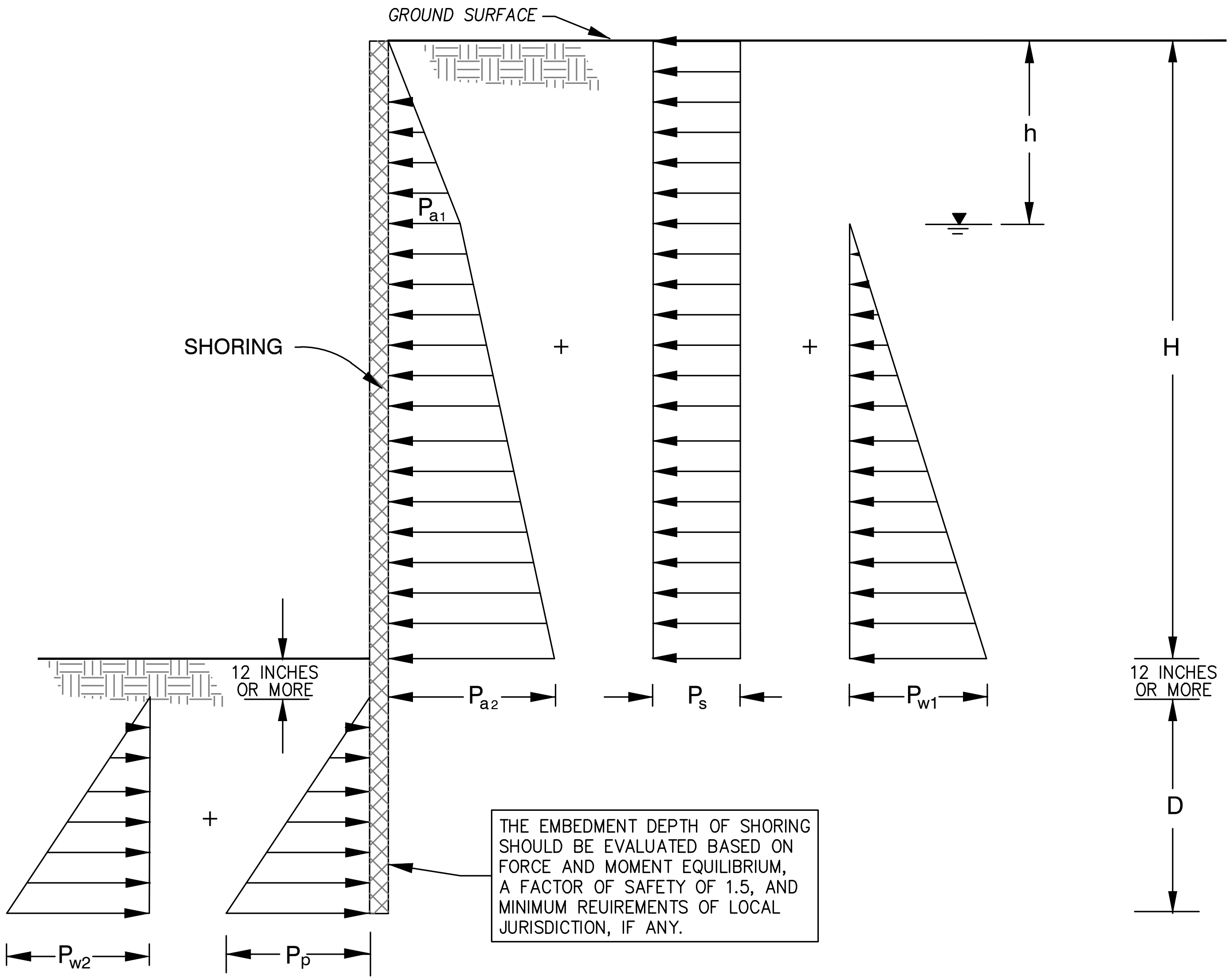

NOTES:

1. ACTIVE LATERAL EARTH PRESSURE, P_a
   P_{a1} = 44h psf; $P_{a2} = P_{a1} + 21(H - h)$ psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. HYDROSTATIC PRESSURE, P_{w1} & P_{w2}
   $P_{w1} = 62.4(H - h)$ psf
   P_{w2} = 62.4D psf
4. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 160D psf
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H, h AND D ARE IN FEET
7. GROUNDWATER TABLE
8. P_{w2} IS APPLICABLE WHEN DEWATERING IS TO BE PERFORMED FROM INSIDE OR OUTSIDE OF THE EXCAVATION

NOT TO SCALE

FIGURE 7

LATERAL EARTH PRESSURES FOR TEMPORARY CANTILEVERED SHORING

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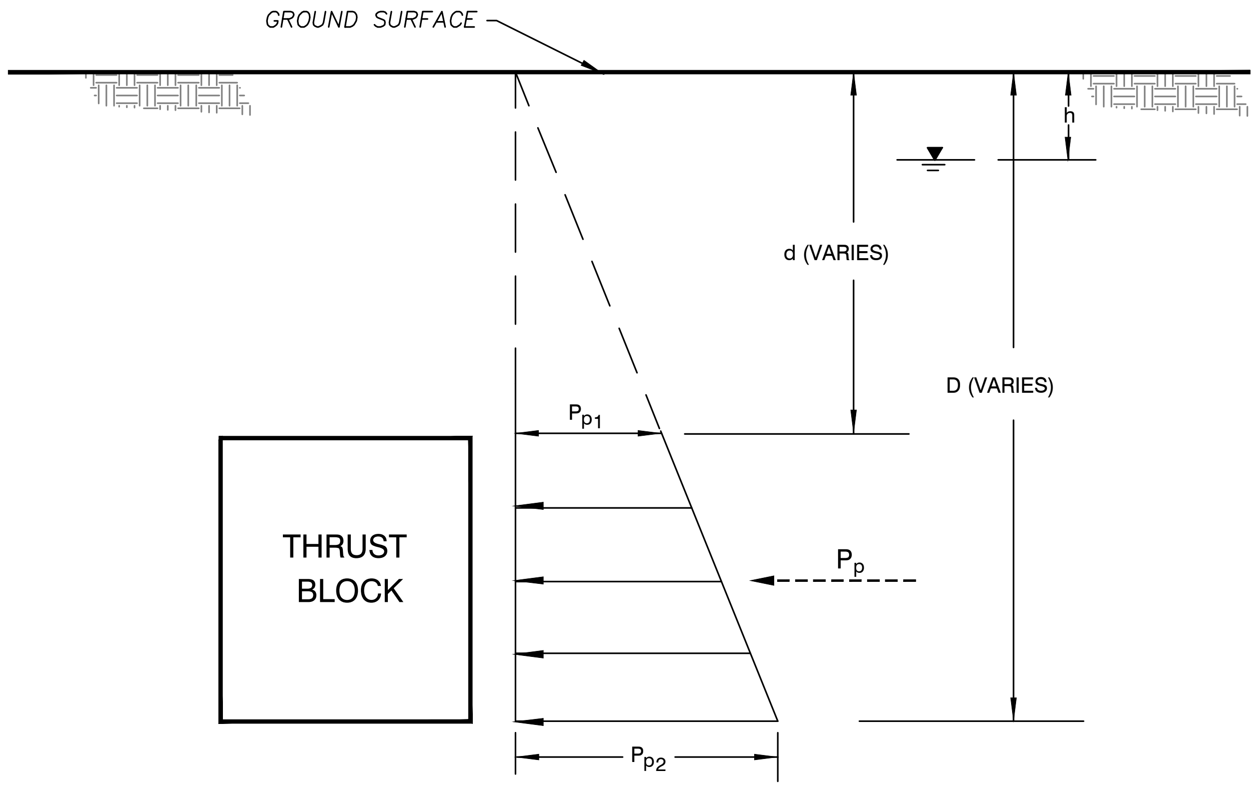

NOTES:

1. GROUNDWATER BELOW BLOCK
   $P_p = 150\,(D^2\text{-}d^2)$ lb/ft
2. GROUNDWATER ABOVE BLOCK
   $P_p = 1.4(D - d)[124.8h + 58(D+d)]$ lb/ft
3. ASSUMES BACKFILL IS GRANULAR MATERIAL
4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
5. D, d AND h ARE IN FEET
6. GROUNDWATER TABLE

NOT TO SCALE

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FIGURE 8

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

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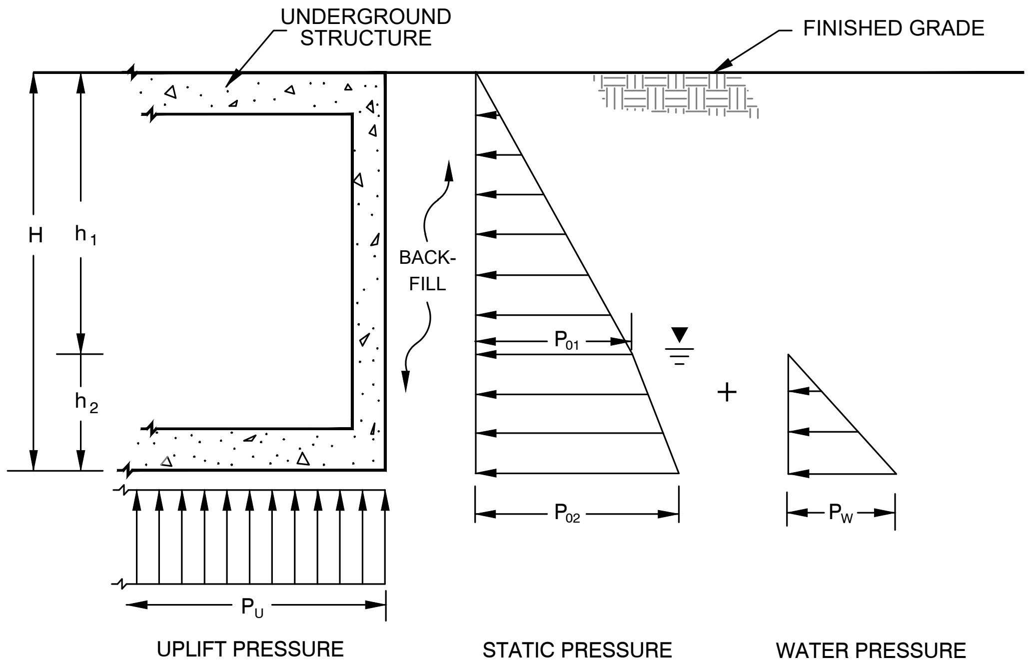

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{01} AND P_{02}
   $P_{01} = 64h_1$ psf
   $P_{02} = 64h_1 + 31h_2$ psf
2. HYDROSTATIC PRESSURE, P_w
   $P_w = 62.4h_2$ psf
3. UPLIFT PRESSURE, P_u
   $P_u = 62.4h_2$ psf
4. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
5. H, h_1 AND h_2 ARE IN FEET
6. GROUNDWATER TABLE

NOT TO SCALE

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FIGURE 9

LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

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