Geotechnical Evaluation

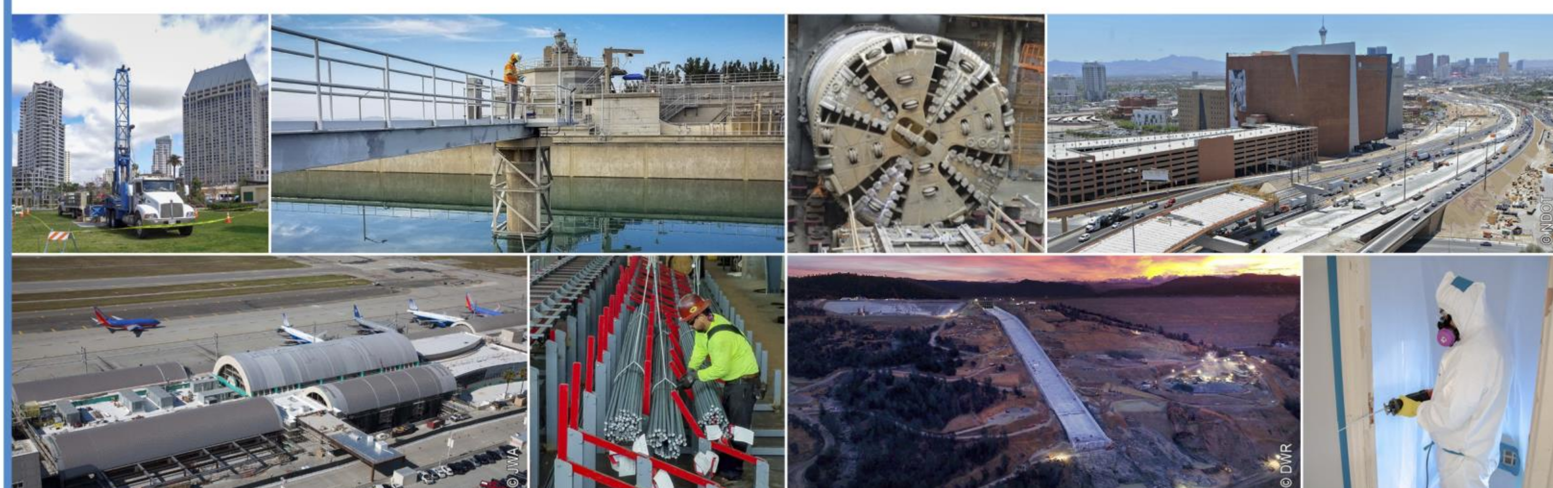
Yorba Linda Boulevard Improvements
Imperial Highway to Lakeview Avenue
City of Yorba Linda

Yorba Linda, California

BKF Engineers

4675 MacArthur Court, Suite 400 | Newport Beach, California 92660

July 26, 2024 | Project No. 212354001



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35+ YEARS

Geotechnical Evaluation

Yorba Linda Boulevard Improvements Imperial Highway to Lakeview Avenue City of Yorba Linda

Yorba Linda, California

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1 INTRODUCTION

In accordance with your request and authorization, Ninyo & Moore has performed a geotechnical evaluation for the City of Yorba Linda's Yorba Linda Boulevard Widening project located in Yorba Linda, California (Figure 1). The purpose of our study was to evaluate the soil and geologic conditions at the site in order to provide geotechnical recommendations for the proposed improvements. This report presents our geotechnical findings, conclusions, and recommendations regarding the proposed project improvements.

2 SCOPE OF SERVICES

The scope of our geotechnical services included the following:

- Project coordination, planning, and scheduling for the subsurface exploration.
- Review of readily available background information, including in-house published geotechnical literature and geologic maps, fault and seismic hazard maps, a previous geotechnical report in the site vicinity, topographic maps, and stereoscopic aerial photographs.
- Geotechnical site reconnaissance to observe the general site conditions, mark the boring and test pit locations, and coordinate with Underground Service Alert for utility clearance.
- Subsurface exploration consisting of drilling, logging, and sampling of four borings to depths ranging from approximately 5 to 41.5 feet below the ground surface. Two borings were drilled with a track mounted limited access rig and a tripod rig, and two borings were drilled using a hand auger. The borings were logged by a representative from our firm, and bulk and relatively undisturbed soil samples were collected at selected depths for laboratory testing. The borings were backfilled with drill cuttings.
- Hand-excavation and logging of two test pits to obtain information on the depth of the foundation for the existing parking structure. The test pits were excavated with hand equipment to depths ranging from approximately 4.6 to 6.5 feet below the ground surface. The test pits were backfilled with the excavated soil.
- Laboratory testing of selected, representative soil samples to evaluate in-situ moisture content and dry density, gradation, percentage of soil particles finer than the No. 200 sieve, Atterberg limits, direct shear strength, expansion index, corrosivity, and R-value.
- Data compilation and engineering analysis of the information obtained from our background review, subsurface evaluation, and laboratory testing.
- Preparation of this geotechnical report presenting our findings, conclusions, and geotechnical recommendations pertaining to this project.

3 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The proposed project will involve widening the eastbound side of Yorba Linda Boulevard from approximately 270 feet west of Imperial Highway to 300 feet east of Lakeview Avenue in Yorba Linda, California. Topographically, the alignment grades upwards from Imperial Highway to

Lakeview Avenue with elevations ranging from approximately 372 feet to 395 feet above mean sea level (MSL) (United States Geological Survey [USGS], 2022).

Based on conceptual plans, we understand that the proposed improvements will include a new dedicated right turn lane on the eastbound side of Yorba Linda Boulevard, additional left turn lanes onto Imperial Highway, new concrete sidewalks, curbs, and gutters, new turn signals, new raised medians, and new pavement (BKF Engineers, 2023).

The addition of a dedicated right turn lane on the eastbound side of Yorba Linda Boulevard will require the construction of a new retaining wall. Currently, the area where the new lane and retaining wall will be added consists of an approximately 12-foot-high, 2:1 (horizontal to vertical) slope that descends from the existing sidewalk to the lower level of an existing parking structure. The new retaining wall is proposed to be constructed in the lower portion of the existing slope, approximately 6.5 feet north of the existing parking structure wall. Based on discussions with the structural engineer, the wall is expected to have a retained height of approximately 7 to 10 feet. The new retaining wall may be supported on a spread footing; however, due to the height of the proposed retaining wall and the space constraints, supporting the retaining wall on cast-in-drilled-hole (CIDH) pile foundation may be a preferred alternative.

4 SUBSURFACE EXPLORATION AND LABORATORY TESTING

Our subsurface evaluation was performed on August 9, 2023, and May 21 to 22, 2024, and consisted of drilling, logging, and sampling of four small-diameter borings (B-1 through B-4) and the excavation of two test pits (TP-1 and TP-2).

The borings were drilled to depths ranging from approximately 5 to 41.5 feet below the ground surface. Boring B-1 was drilled using a tripod drill rig, boring B-2 was drilled using a track-mounted limited access rig, and borings B-3 and B-4 were manually drilled using a hand auger. A representative from Ninyo & Moore logged the borings and obtained bulk and relatively undisturbed soil samples at selected depths for laboratory testing.

Two test pits were hand-excavated to depths ranging from approximately 4.6 to 6.5 feet below the ground surface to evaluate the depth of the foundation for the existing parking structure. In TP-1, the bottom of the footing was exposed. In TP-2, we were not able to expose the bottom of the footing, and Concrete Gauge Thickness (CTG) equipment was used to measure the depth of the footing. After the top of the footing was exposed, the CTG equipment was placed on top of the footing, and readings were taken to measure the footing thickness. After the data were collected, the test pits were backfilled with the excavated soils. The approximate locations of the borings and test pits are presented on Figures 2 and 3. The boring and test pit logs are presented in Appendix A.

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

5 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geology

The project site is situated within the central block of the Los Angeles Basin in the Peninsular Ranges Geomorphic Province of California. The Los Angeles Basin has been divided into four structural blocks, which are generally bounded by prominent fault systems: The Northwestern Block, the Southwestern Block, the Central Block, and the Northeastern Block (Norris and Webb, 1990). The Central Block is bordered by the Whittier fault to the northeast, the Newport-Inglewood fault to the southwest, and the Santa Monica Mountains to the northwest. The central block is characterized by uplifted hills between low-lying plains resulting from anticlinal and synclinal structural features including Signal Hill, Huntington Beach Mesa, Central Plain, La Habra Valley, and Coyote Hills. The Central block's main portion is occupied by the Downey Plain, a broad synclinal sag about 12 to 14 miles wide with deep alluvial deposits. Geologic mapping by Morton and Miller (2006) indicates that the site is underlain by very old and young alluvial fan deposits. The very old alluvial fan deposits generally consist of moderately to well consolidated silt, sand, gravel, and conglomerate. The young alluvial fan deposits generally consist of unconsolidated to moderately consolidated silt, sand, pebbly cobbly sand, and bouldery deposits. A regional geologic map for the site vicinity is shown on Figure 4.

5.2 Site Geology

Materials encountered during our subsurface exploration consisted of fill and very old alluvial fan deposits. Fill was encountered from the ground surface to depths of up to approximately 13 feet. The fill generally consisted of moist, stiff to very stiff, sandy lean clay and moist, medium dense, clayey sand. Variable amounts of gravel and asphalt concrete (AC) fragments were encountered in the fill. Very old alluvial fan deposits were encountered beneath the fill to the explored depths of up to approximately 41.5 feet. The alluvial fan deposits generally consisted of moist, firm to hard, clayey silt and sandy lean clay, and moist, dense to very dense, clayey sand. More detailed descriptions of the subsurface materials encountered during our exploration are presented on the boring and test pit logs in Appendix A.

5.3 Groundwater

Groundwater was not encountered in our exploratory borings during drilling to the explored depths of up to approximately 41.5 feet. Regional maps indicate that the historic high depth to groundwater at the project site is more than 30 feet below the ground surface (California Geological Survey, 2005). Groundwater monitoring well data from the State of California Department of Water Resources Geotracker (2024) website indicates that groundwater was measured at a depth of approximately 65 feet below the ground surface in monitoring wells located at the intersection of Yorba Linda Boulevard and Lakeview Avenue. It should be noted that fluctuations in the level of groundwater at the project site may occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors which may not have been evident at the time of our evaluation.

6 FLOOD HAZARDS

Based on our review of flood insurance rate maps for the project area (Federal Emergency Management Agency, 2009), the project site is not located in the 100-year Flood Hazard Zone. The site is located within Zone X, an area of minimal flood hazard.

7 FAULTING AND SEISMICITY

The project site is located in a seismically active area, as is the majority of southern California. The numerous faults in California include active, potentially active, and inactive faults. As defined by the California Geological Survey (CGS), active faults are faults that have ruptured within Holocene time, or within approximately the last 11,000 years. Potentially active faults are those that show evidence of movement during Quaternary time (approximately the last 1.6 million years) but for which evidence of Holocene movement has not been established. Inactive faults have not ruptured in the last approximately 1.6 million years. The approximate locations of major active faults in the region and their geographic relationship to the project sites are shown on Figure 5. The nearest mapped active fault to the site is the Whittier fault, located approximately 1.7 miles northeast of the site (USGS, 2008).

Based on our review of seismic hazard maps, geologic literature, and geologic maps, the site is not located within a State of California Earthquake Fault Zone (formerly known as Alquist-Priolo Special Studies Zone) (CGS, 2018), and no active faults are known to cross the subject site. The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, liquefaction, landslides, and tsunami and seiche inundation. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

7.1 Surface Fault Rupture

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

7.2 Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2022 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Based on our review of CGS's shear wave velocity map, the average shear wave velocity in the upper 30 meters (100 feet) of the subsurface profile (V_{S30}) is estimated to be approximately 387 meters per second (1,270 feet per second) (CGS, 2015). In accordance with Chapter 20 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures, the site classification is Site Class C.

In accordance with ASCE 7-16, the mapped MCE_R ground motion response accelerations were determined using the 2024 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 2022 CBC, are provided in Section 9.2 for the evaluation of seismic loads on buildings and other structures.

7.3 Liquefaction Potential

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

The site is not located within a mapped area subject to seismically induced liquefaction hazards (CGS, 2005). Furthermore, based on the depth to groundwater at the site, and the relatively dense granular soils and hard fine-grained soils encountered at the site, 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 this project.

7.4 Landslides

The site is located in an area of relatively level terrain. We did not observe evidence of instability on the existing slope that descends from Yorba Linda Boulevard to the adjacent parking structure. The majority of the existing slope will be eliminated and replaced with a retaining wall as part of the road widening project. Based on our site reconnaissance and review of published seismic hazard maps, geologic maps, and stereoscopic aerial photographs, landslides or indications of deep-seated slope instability are not considered potential hazards at the site.

7.5 Tsunamis and Seiches

Tsunamis are long wavelength, seismic sea waves (long compared to ocean depth) generated by the sudden movement of the ocean floor during submarine earthquakes, landslides, or volcanic activity. Seiches are waves generated in a large, enclosed body of water. The project area is not located within an area considered susceptible to tsunamis or seiche inundation. Therefore, damage due to tsunamis or seiches is not a design consideration.

8 CONCLUSIONS

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

- Based on our exploratory borings, the site is generally underlain by fill and very old alluvial fan deposits. Fill was encountered to depths of up to approximately 13 feet and generally consisted of moist, stiff to very stiff, sandy lean clay and moist, medium dense, clayey sand. Variable amounts of gravel and AC fragments were encountered in the fill. The very old alluvial fan deposits generally consisted of moist, hard, sandy lean clay, and moist, dense to very dense, clayey sand.

- We anticipate that the on-site soils should be suitable for use as general fill following moisture-conditioning, provided they are free of trash, debris, roots, vegetation, deleterious materials, and cobbles or hard lumps of materials in excess of 4 inches in diameter. We anticipate that on-site soils will not be suitable for reuse as structural backfill fill due to the high fines content of the soils. We anticipate that import structure backfill will be required beneath spread footings for the new retaining wall and also as retaining wall backfill.

- On-site soils should be considered as Type C soils in accordance with Occupational Safety and Health Administration (OSHA) soil classifications. Temporary shoring should be provided in accordance with OSHA regulations.

- Groundwater was not encountered in our exploratory borings during drilling to the explored depths of up to approximately 41.5 feet. Groundwater monitoring well data indicates that groundwater was measured at a depth of approximately 65 feet below the ground surface in monitoring wells located at the intersection of Yorba Linda Boulevard and Lakeview Avenue (State of California, 2024). Fluctuations in the groundwater level may occur as a result of variations in seasonal precipitation, irrigation practices, groundwater pumping and other factors.

- The site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). Based on our review of published geologic maps and aerial photographs, no known active or potentially active faults transect the site. The potential for surface fault rupture at the site is considered low.

- The site is not located within a mapped Seismic Hazards Zone area considered susceptible to seismically induced soil liquefaction.

- The site is not located in an area considered susceptible to landslides, tsunamis, or seiches.

- Our limited laboratory corrosivity testing indicates that the on-site earth materials can be classified as corrosive due to relatively low electrical resistivity based on the California Department of Transportation (Caltrans, 2021) corrosion guidelines.

- In TP-1, the top of the footing for the existing parking structure was measured at an elevation of approximately 367.0 feet above MSL. The footing thickness was measured to be approximately 1.4 feet, which corresponds to an elevation of approximately 365.6 feet above MSL for the bottom of the footing. The footing extended outwards from the concrete masonry unit (CMU) wall a distance of approximately 3 feet.

- In TP-2, the top of the footing for the existing parking structure was measured at an elevation of approximately 366.3 feet above MSL. The bottom of the footing was not exposed in our test pit; however, the thickness of the footing was measured to be approximately 3.7 to 4 feet using the CTG equipment, which corresponds to an elevation of 362.6 to 362.3 feet above MSL for the bottom of the footing. The footing extended outwards from the CMU wall a distance of approximately 3.3 feet.

9 RECOMMENDATIONS

The recommendations presented in the following sections provide geotechnical criteria regarding the design and construction of the proposed site improvements. The recommendations are based on the results of our subsurface evaluation, geotechnical analysis, and project understanding. The proposed work should be performed in conformance with the recommendations presented in this report, project specifications, and requirements of the applicable governing agencies.

9.1 Earthwork

Earthwork at the site is anticipated to consist of cuts and fills associated with subgrade preparation for the retaining wall foundation, backfilling of the retaining wall, pavement and hardscape reconstruction, and finish grading for establishment of site drainage. If the proposed retaining wall is supported on drilled piers, earthwork will consist of drilling for the proposed retaining wall foundation. Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections.

9.1.1 Pre-Construction Conference

We recommend that 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.

9.1.2 Clearing and Site Preparation

Prior to excavation and fill placement, the site should be cleared of existing site improvements, pavements, surface obstructions, landscape vegetation and other deleterious materials, and abandoned utilities. Existing utilities to remain in place should be located and protected from damage by construction activities. Obstructions such as existing foundations that extend below the finished grade should 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.

9.1.3 Excavation Characteristics

Based on our field exploration, we anticipate that excavations should be feasible with conventional earthmoving equipment in good working order. The on-site fill and alluvial deposits are comprised predominantly of moist, stiff to hard, sandy lean clay and moist, medium dense to very dense, clayey sand. Variable amounts of gravel and AC fragments were encountered in the fill. Oversize materials (larger than 4 inches in the longer dimension), including cobbles, are not considered suitable for backfill and should be disposed of off-site. Contractors should make their own independent evaluation of the excavatability of the on-site materials prior to submitting their bids.

9.1.4 Temporary Excavations and Shoring

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 1.5:1 (horizontal to vertical) for excavations deeper than 4 feet but not exceeding the depth of 20 feet below the existing grade. Bedding materials for existing pipelines, if encountered, may be prone to caving. Temporary excavations should be evaluated in the field and constructed in accordance with applicable OSHA guidelines. The site soils should be considered as OSHA Soil Type C. Onsite safety of personnel is the responsibility of the contractor.

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

We anticipate temporary shoring may be needed if the proposed retaining wall is supported on a spread footing foundation, due to boundary constraints with existing streets and other improvements that will be kept in-place. Shoring systems, if used, should be designed for the anticipated soil conditions using the lateral earth pressure values shown on Figures 6 and 7 for braced and cantilevered excavations, respectively. The recommended design pressures are based on the assumption that the shoring system is constructed without raising the ground surface elevation behind the shored sidewalls of the excavation, that there are no surcharge loads, such as soil stockpiles and construction materials, and that no loads act above a 1:1 (horizontal to vertical) plane ascending from the base of the shoring system. For a shoring system subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on the lateral earth pressures acting on the shored walls. Spoils should not be placed near the edge of the open cut excavation. For open cut excavations, the spoil pile should be placed at a distance more than the depth of excavation from the top of the excavation. OSHA and other applicable agency requirements pertaining to worker safety should be met during the excavation activities.

We anticipate that settlement of the ground surface will occur behind shored excavations. The amount of settlement depends heavily on the type of shoring system, the contractor’s workmanship, and soil conditions. To reduce the potential for distress to adjacent improvements, we recommend that the shoring system be designed to limit the ground settlement behind the shoring system to 0.5 inch or less. Possible causes of settlement that should be addressed include settlement during installation of the shoring elements, excavation for structure construction, construction vibrations, and removal of the support system. We recommend that shoring installation be evaluated carefully by the contractor prior to construction. Ground vibration and settlement monitoring may be appropriate during construction depending on the depths of the shored excavations.

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

9.1.5 Treatment of Near-Surface Soils

Based on our subsurface evaluation, it is our opinion that suitable foundation bearing for the new retaining wall supported on shallow spread footings may be provided by remedial grading consisting of the excavation and recompaction of the near-surface soils. Undocumented fill was encountered in our borings and should be removed and recompacted as discussed below to provide suitable support for settlement-sensitive improvements. 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 wall footings, or to the depth of the undocumented fill, whichever is deeper. The horizontal limits of remedial over-excavation should extend approximately 2 feet beyond the footings, removing existing undocumented fill, and exposing relatively dense native soils. The removal and recompaction work should consist of 1) 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 compacted import fill. The fill soils should be moisture-conditioned generally above the laboratory optimum moisture content and should be compacted to a relative compaction of 90 percent as evaluated by ASTM International (ASTM) test method D 1557. The remedial grading recommendations detailed above would not be applicable if the new retaining wall is to be supported on drilled piers.

When the retaining wall has been constructed and is ready to be backfilled, any remaining landscape vegetation that is on the existing slope that was not removed during clearing and grubbing should be removed. Based on our borings, the existing slope is comprised of fill. During placement of new retaining wall backfill, the existing slope surface should be benched to remove any loose soil and roots from the landscape vegetation to expose competent fill.

Structural pavement sections and exterior flatwork may be supported on compacted, low-expansion potential soil. Subgrade for pavements should be prepared by excavating the upper approximately 12 inches of exposed subgrade, moisture-conditioning to slightly over the optimum moisture content and compacting to 95 percent relative compaction as evaluated by ASTM D 1557. However, we anticipate that a new driveway entrance will be constructed from Yorba Linda Boulevard to the upper deck of the existing parking structure. In order to reduce the potential for differential settlement between the new pavements and the upper deck of the parking structure, we recommend that the subgrade soils beneath the new pavement area are compacted to 95 percent relative compaction to a depth of 3 feet below the new pavement subgrade elevation. Alternatively, 2-sack sand/cement slurry may be used in the upper 3 feet of the subgrade soils in lieu of using compacted, low expansion potential soil. The limits of the 3-foot-deep 95 percent compacted fill zone, or 2-sack sand/cement slurry, should extend approximately 5 feet laterally from the connection point with the parking structure deck.

Subgrade for exterior flatwork areas, such as new sidewalks, should be prepared by scarifying the upper approximately 8 inches of exposed subgrade, moisture conditioning to slightly over optimum moisture content and compacting to 90 percent relative compaction as evaluated by ASTM D 1557. Additional excavation may be appropriate depending on the materials exposed during construction.

9.1.6 Fill Material

In general, the existing on-site soils should be suitable for use as general fill and trench backfill. However, we anticipate the existing on-site soils will not be suitable for reuse as engineered fill beneath the footings or as retaining wall backfill due to the relatively high fines content. We anticipate that import fill will be needed to support the spread footings of the retaining wall, if that option is chosen instead of drilled piers, and also as backfill for the retaining wall. If the retaining wall is to be supported on drilled piers, import fill will be needed for the wall backfill zone only. Structure backfill should be comprised of granular, non-expansive soil that conforms to the latest edition of “Greenbook” Standard Specifications for Public Works Construction for structural backfill. “Non-expansive” can be defined as soil having an expansion index (EI) of 20 or less in accordance with ASTM D 4829. The on-site materials will involve moisture-conditioning to bring the materials near optimum content prior to placement and compaction.

As indicated above, imported materials 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 chloride concentration of less than 500 parts per million [ppm], a soluble sulfate content of less than approximately 0.15 percent [1,500 ppm], a pH value of more than 5.5, and an electrical resistivity of more than 1,500 ohm-centimeters [ohm-cm]). Import materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

9.1.7 Fill Placement and Compaction

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 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, or to 95 percent in areas of new pavements and the new entrance to the upper deck of the existing parking structure as recommended in Section 9.1.5. Successive lifts should be treated in a similar manner until the desired finished grades are achieved. 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. Additionally, aggregate base (AB) materials underneath vehicular pavements should be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557.

9.1.8 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 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.

9.1.9 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 700 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.

9.2 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the seismic design parameters for the site in accordance with the 2022 CBC guidelines.

Table 1 – 2022 California Building Code Seismic Design Criteria

Spectral Response Acceleration Parameters	Values
Site Classification	C
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_s	1.816g
Mapped MCE_R Spectral Response Acceleration at 1.0-Second Period, S_1	0.639g
MCE_R Spectral Response Acceleration at Short Periods, Adjusted for Site Class, S_{MS}	2.179g
MCE_R Spectral Response Acceleration at 1.0-Second Period, Adjusted for Site Class, S_{M1}	0.895g
Design Spectral Response Acceleration at Short Periods, S_{DS}	1.453g
Design Spectral Response Acceleration at 1.0-Second Period, S_{D1}	0.597g
Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.936g

9.3 Foundations

The proposed new retaining wall may be supported on continuous footings bearing on compacted fill prepared in accordance with the recommendations presented in the Earthwork section of this report. Alternately, the retaining wall may 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 the structures.

9.3.1 Spread Footings

Continuous footings should have a width of 24 inches, and be founded at a depth of 24 inches or more below the lowest adjacent finished grade. Continuous footings should be reinforced and detailed in accordance with the recommendations of the structural engineer.

Footings, as described above and bearing on compacted fill soils, may be designed using a net allowable bearing capacity of 2,500 pounds per square foot (psf). The bearing capacity may be increased by 200 psf and 500 psf for every additional foot of increase in width and depth, respectively, up to a value of 3,000 psf. The allowable bearing capacity may be increased by one-third when considering loads of short duration, such as wind or seismic forces. Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be on the order of 1 inch and 0.5 inch over a horizontal span of 40 feet, respectively.

Footings bearing on compacted fill may be designed using a coefficient of friction of 0.30, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 300 psf per foot of depth for level ground condition up to a value of 3,000 psf. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance. The 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.

Trenches should not be excavated adjacent to footings. If trenches are to be excavated near a footing, the bottom of the trench should be located above a 1:1 (horizontal to vertical) plane projected downward from the bottom outer edges of the footing. Utility lines that cross beneath footings should be encased in concrete below the footing.

9.3.2 CIDH Piles (Drilled Piers)

As an alternate to shallow spread footings, the retaining wall may be supported on CIDH piles. Design loads were not available at the time of this report. The actual depth of the drilled piers should be evaluated when details of the piles are available. The axial capacity of 24 and 30-inch-diameter CIDH piles were analyzed using the computer program SHAFT version 8.12 (Ensoft, 2017). Soil strength parameters were estimated from our boring logs and laboratory testing for the site. The axial capacity was based on side friction resistance; end bearing was not included in our analysis. A minimum embedment depth of 20 feet was considered for the piles. A factor of safety of 2.0 and 3.0 were used for the allowable axial capacity in compression and uplift/tension, respectively. The results of our axial pile capacity evaluation are summarized below in Table 2. A plot of the depth versus ultimate axial capacity for the CIDH piles is presented in Appendix C.

If the measures outlined in this report are implemented effectively, the total settlement of the CIDH piles will be limited to 0.25-inch or less provided that the drilled pier bearing materials are not significantly disturbed during construction. This estimate is based on the soil conditions observed in our exploratory borings, anticipated conditions, and our experience with similar geologic materials. The axial capacity does not include the weight of the pile. The axial pile capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. To avoid the group action of the pile foundation, we recommend that piles be spaced no closer than three times the pile diameter.

Table 2 – Axial Load Capacity of Single CIDH Pile

Pile Type	Allowable Axial Capacity (kips)		Embedment Depth (feet)
	Compression	Tension	
24-inch-diameter CIDH	75	50	20
30-inch-diameter CIDH	90	60	20

The lateral pile capacities were evaluated for free-head conditions at lateral head deflections of 0.25- and 0.50- inch using the computer program LPILE Version 12.05 (Ensoft, 2022). Results of our lateral pile capacity analyses for 24 and 30-inch-diameter CIDH piles are summarized in Table 3. Lateral pile capacity plots are presented in Appendix C.

Table 3 – Lateral Load Capacity of Single CIDH Pile

Pile Design Parameters	24-inch CIDH Pile Free Head		30-inch CIDH Pile Free Head	
Lateral Deflection of Pile Head (inch)	0.25	0.50	0.25	0.50
Design Pile Length (feet)	20	20	20	20
Allowable Axial Capacity (kips)	75	75	90	90
Shear Force at top of pile (kips)	16.1	21.3	25.6	34.5
Maximum Shear Force (kips)	17.6	23.0	25.7	34.6
Depth to Maximum Shear Force (feet)	11.6	12.0	13.2	13.6
Maximum Positive Moment (kips-foot)	84.2	117.1	151.2	214.4
Maximum Negative Moment (kips-foot)	3.0	3.5	N/A	N/A
Depth to Maximum Positive Moment (feet)	7.4	7.8	8.4	8.8
Depth to Maximum Negative Moment (feet)	16.8	17.6	N/A	N/A
Depth to First Zero Deflection (feet)	11.7	12.1	13.1	13.7

9.3.2.1 Drilled Pier Construction Considerations

The CIDH pile construction should be observed by the project geotechnical consultant during excavation to evaluate if the piles have been extended to the recommended depths. The excavations should be cleaned of loose soil and gravel. The CIDH pile drilling contractor should mobilize equipment of sufficient size and operating capability to achieve the recommended embedment length. The excavation technique chosen by the contractor should not adversely affect the quality or strength of the pier side or end bearing materials. If refusal is encountered in these materials during actual installation, the geotechnical engineer should be retained to evaluate the subsurface condition to establish that true refusal has been met with adequate drilling equipment. It is the Contractor's responsibility to take the appropriate measures to provide for the integrity of the excavation and to see that the excavations are cleaned and straight and that sloughed loose soil is removed from the bottom of the excavation prior to the placement of concrete. Drilled CIDH piles should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pile is approximately 3 inches from the plan location. It is usually acceptable for a pile to be out of plumb by one percent of the depth of the pile. The minimum center-to-center spacing of piles should be no less than three times the nominal diameter of the pile.

Groundwater is not expected to be encountered during drilling; however, seepage and perched water conditions may be encountered. Landscape irrigation of the existing slope where new drilled piers will be constructed should be stopped a few weeks prior to construction to reduce the potential for seepage. However, the contractor should be prepared to advance the drilled piers under such conditions. We recommend that the contractor be prepared to take appropriate measures during construction to reduce the potential for caving of the drilled holes, including the use of casing and/or drilling mud. Casing may be needed in the pier holes to reduce water infiltration. While pouring concrete, the casing should be withdrawn gradually.

The concrete should be placed by tremie method so that the aggregate and cement do not segregate during concrete placement. Concrete utilized in the drilled piers should be a fluid mix with sufficient slump so that it will fill the void between the rebar cage and the drill hole wall. The contractor should take care to reduce enlargement of the excavation at the tops of drilled piers, which could result in mushrooming of the drilled pier top.

Drilled pier holes should be cleaned prior to placement of concrete. Care should be taken to check that the soils at the drilled pier bottom have not been disturbed. The successful advancement of drill holes for the construction of drilled shafts will depend largely on the suitability of the drilling equipment and the skill of the operator. The drilled foundation contractor should try to reduce the time during which the excavation remains open. The contractor should schedule the sequence of operations so that each excavation can be finished, the rebar cage placed, and the concrete poured within the same work day. Drilled pier excavations should not be left open overnight. In case of delay in placing concrete within the drill hole due to equipment breakdown or other unforeseen circumstances, casing may be used to protect the integrity of the hole. While pouring concrete, the casing should be withdrawn gradually.

The contractor should not place drilled piers adjacent to each other until the first one is set. The installation of drilled piers should be scheduled to allow the concrete in adjacent piers to set before drilling the next pier. Drilled piers spaced closer than about three pier diameters (clear spacing) should be placed on alternate days.

Consideration should be given to performing load testing on CIDH piles to evaluate the axial and lateral capacities, as appropriate. The most common pile load tests typically involve compression load testing, tensile load testing, lateral load testing, and high strain dynamic testing (Pile Driving Analyzer®) in accordance with ASTM test methods D 1143, D 3689, D 3966, and D 4945, respectively.

The drilled pier installation should be observed by the Geotechnical Engineer or a qualified representative to check that, among other things: 1) subsurface conditions are as anticipated from the boring, 2) the pier bottom surface is clean and competent, 3) the drilled piers are constructed to the specified size and penetration, 4) drilled piers are within allowable tolerances for plumbness, and 5) reinforcements are placed per project specifications. These items are fundamental to the installation and behavior of the drilled piers. Furthermore, we recommend the following for the installation of drilled piers:

- The clear spacing between the rebar cage and the drill hole surface should be three times the maximum size of the coarse aggregate used in the concrete.
- Centralizers should be installed to keep the rebar cage positioned per project specifications.
- If casing is used, a sufficient head of concrete that fills the casing should be placed before pulling the casing.

9.4 Lateral Earth Pressures for Retaining Walls

Lateral earth pressures recommended for design of yielding retaining walls are provided on Figure 8. Passive pressures may be increased by one-third when considering loads of short duration, including wind and seismic loads. Retaining walls should be backfilled with free-draining, granular soil with a low-expansion potential. Measures should be taken to reduce the potential for build-up of hydrostatic pressure behind the retaining walls. Drainage design should include free-draining backfill materials and perforated drains as shown on Figure 9. Solid outlet pipes should be connected to the perforated drains and then routed to a suitable area for discharge.

9.5 Pavement Recommendations

Pavement designs were performed for new pavement construction. The pavement designs were based our evaluation of the subgrade soil conditions and our laboratory testing. Laboratory testing was performed on representative subgrade soil samples and indicated an R-value ranging from approximately 12 to 22. Due to the variability of the on-site soils, an R-value of 12 was used for the pavement design.

Our AC pavement analysis utilized the methodology outlined by the Highway Design Manual (Caltrans, 2022a) and the computer software program CalME (Caltrans, 2022b). For the design of Portland cement concrete (PCC) pavements, we used the methodology presented in the Navy Pavement Design Manual (1979) assuming a 28-day compressive strength of concrete equal to 2,500 psi. We have evaluated pavement structural sections for Traffic Indices (TI) ranging from 7 to 10. The analysis assumes an approximate 20-year design life for new pavements. Based on the R-value and TIs, recommendations for new pavement sections are provided in Table 4.

Table 4 – Preliminary Structural Pavement Sections

Traffic Index	AC over CAB or AC over CMB (inches)	Full Depth AC (inches)	PCC (inches)
7.0	5 over 12	10	8½
8.0	5½ over 15	12	10
9.0	6 over 17½	13½	11½
10.0	7 over 20	15	13

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

We recommend that approximately 4 inches of crushed aggregate/miscellaneous base be placed under the PCC. Prior to placement of the new structural pavement section presented above, the subgrade soils should be prepared in accordance with the recommendations provided in the Earthwork section of this report.

Aggregate base material should conform to the latest specifications in Section 200-2.2 for crushed aggregate base or Section 200-2.4 for crushed miscellaneous base (CMB) of the Greenbook and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1557. Grinding and recycling existing AC and existing base material may be considered as a potential source of CMB material provided they meet the requirements in the "Greenbook." AC should conform to Section 203-6 of the Greenbook and should be compacted to a relative compaction of 95 percent in accordance with ASTM D 1560 or California Test Method (CT) 304.

Pavement sections should be selected based on actual anticipated traffic loading conditions and evaluation of the subgrade materials at the time of construction. We recommend that the paving operations be observed and tested by Ninyo & Moore. We further recommend that mix designs for the various pavements be made by an engineering company specialized in this type of work.

9.6 Hardscape

We recommend that new exterior concrete sidewalks and flatwork have a thickness of 4 inches. The hardscape should be underlain by 4 inches of clean sand and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, walls and/or other structures, and between sections of new hardscape, should be doweled.

9.7 Corrosivity

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

The soil pH was measured at approximately 8.8 and the electrical resistivity was measured to be approximately 656 ohm-cm. The chloride content of the sample was measured to be approximately 125 ppm. The sulfate content of the tested sample was approximately 0.008 percent (i.e., 80 ppm). Based on the laboratory test results and Caltrans (2021) corrosion criteria, the project site would be classified as a corrosive site due to relatively low electrical resistivity of the sample tested. A corrosive site is defined as having earth materials with a chloride concentration of 500 ppm or more, sulfate concentration of 1,500 ppm or more, a pH of 5.5 or less, or an electrical resistivity of 1,500 ohm-cm or less. We recommend that a corrosion engineer be consulted for further evaluation and recommendations during the project’s detailed design phase.

9.8 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. Based on the American Concrete Institute criteria (2022b), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight and moderate for water-soluble sulfate contents ranging from 0.10 to 0.20 percent by weight. The potential for sulfate attack is severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil sample tested for this evaluation, using Caltrans Test Method 417, indicate a water-soluble sulfate content of approximately 0.008 percent by weight. Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of the on-site soils, consideration should be given to using Type II/V cement for the project.

In order to reduce the potential for shrinkage cracks in the concrete during curing, we recommend that the concrete for the proposed structure be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with CBC (2022). The structural engineer should be consulted for additional concrete specifications.

9.9 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.

10 CONSTRUCTION OBSERVATION

The recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. It is imperative that the interpolated subsurface conditions be checked by our representative during construction. Observation of remedial grading bottoms beneath retaining wall spread footing foundations should be performed by Ninyo & Moore representatives. Observation and testing of compacted fill and backfill should also be performed by our representative during construction. We further recommend that the project plans and specifications be reviewed by this office prior to construction. It should be noted that, upon review of these documents, some recommendations presented in this report might be revised or modified.

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

- Observing site clearing, grubbing, and removals.
- Observing remedial grading excavation bottoms and the placement and compaction of fill.
- Evaluating imported materials prior to their use as fill.
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.
- Observing drill hole excavations and cleaning prior to placement of rebars and concrete in the drill hole.
- Performing material testing services including concrete compressive strength and inspections.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that the services of Ninyo & Moore are not utilized during construction, we request that the selected consultant provide the owner with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report.

11 LIMITATIONS

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

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

This report is intended for design purposes only. It may 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

APPENDIX A

Boring Logs

APPENDIX A

BORING AND TEST PIT 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 1.4 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 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 of the drill rig in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sampler barrel in the brass rings, sealed, and transported to the laboratory for testing.

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

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

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

Gradation Analysis
A gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 6913. The grain-size distribution curve is shown on Figure B-1. This test result was utilized in evaluating the soil classification in accordance with the USCS.

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

Atterberg Limits
Tests were performed on a selected representative fine-grained soil sample to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. These test results were utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-3.

Direct Shear Tests
Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent the adverse field conditions. The test results are shown on Figures B-4 and B-5.

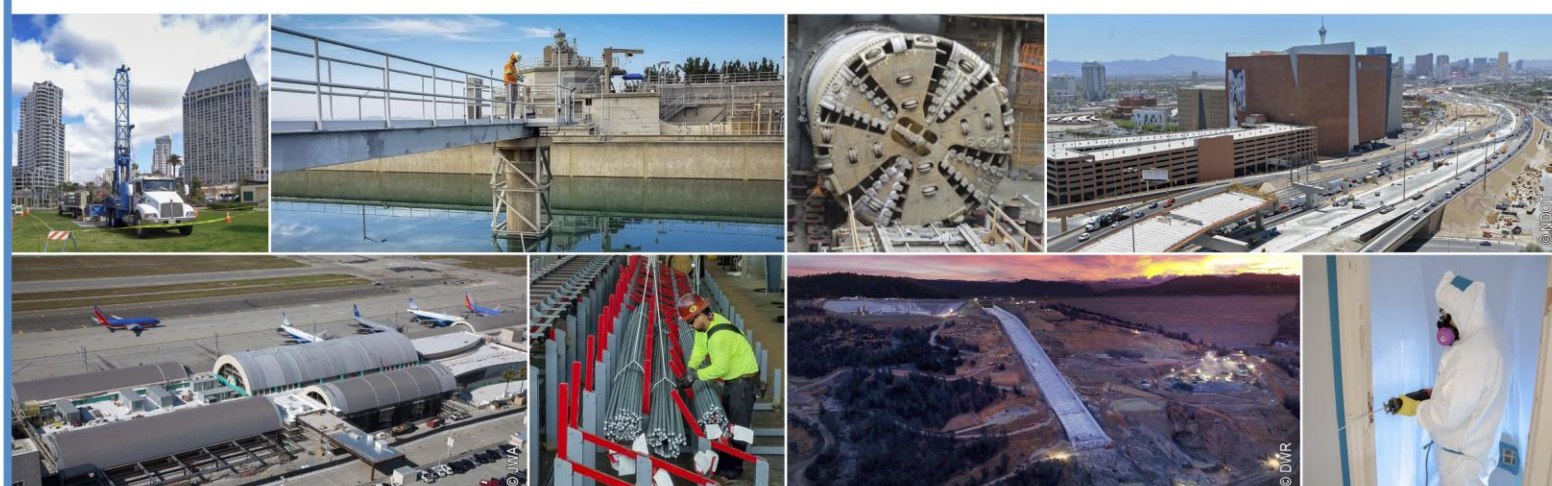
Expansion Index Test
The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. The specimen was molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimen was loaded with a surcharge of 144 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of this test are presented on Figure B-6.

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 contents of the selected sample were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-7.

R-Value
The resistance value, or R-value, for site soils was evaluated in general accordance with California Test (CT) 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-8.

APPENDIX C

CIDH Pile Design Plots



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