Revised Geotechnical Evaluation

Los Alamitos High School New Gymnasium

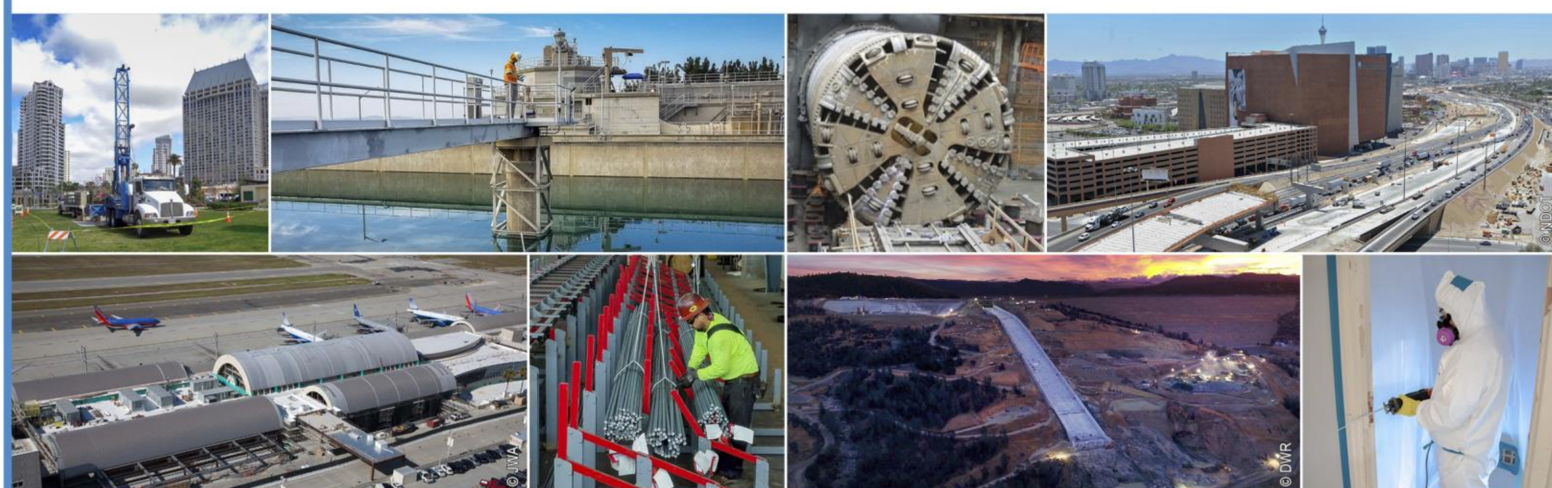
3591 West Cerritos Avenue

Los Alamitos, California

Los Alamitos Unified School District

10293 Bloomfield Street | Los Alamitos, California 90720

September 30, 2022 | Project No. 211897001



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1 INTRODUCTION

In accordance with your request, we have performed a geotechnical evaluation for the Los Alamitos High School New Gymnasium Project located at 3591 West Cerritos Avenue in Los Alamitos, California (Figure 1). The purpose of this study was to perform a subsurface evaluation and to provide geotechnical design recommendations for the construction of the new gymnasium in general accordance with the 2019 California Building Code (CBC) and California Geological Survey (CGS) Note 48 (2019). Upon the completion and issuance of our report on May 27, 2022, we received information from Mr. Roy Frey with Westgroup Designs regarding the conflict between the existing electrical conduit and the footings on the west side of the proposed gymnasium. Accordingly, we have revised our recommendations to mitigate the conflict.

2 SCOPE OF SERVICES

Our scope of services for this project included the following:

- Project coordination, planning, and scheduling for the subsurface exploration.
- Review of readily available background information, including in-house published geotechnical literature and geologic maps, fault and seismic hazard maps, topographic maps, and stereoscopic aerial photographs.
- Geotechnical site reconnaissance to observe the general site conditions, mark the boring and cone penetration test sounding (CPT) locations, and coordinate with Underground Service Alert for utility clearance.
- Performance a geophysical survey to clear the boring and CPT locations of underground utilities and obstructions prior to drilling.
- Acquisition of boring permits from Orange County Health Care Agency Environmental Health Division for drilling into groundwater.
- Subsurface exploration consisting of the drilling, logging, and sampling of five hollow-stem auger borings with a truck-mounted drill rig. The borings were excavated to depths ranging from approximately 31½ to 101½ feet below the ground surface. The borings were logged by a representative from our firm, and bulk and relatively undisturbed soil samples were collected at selected depth intervals for laboratory testing.
- Subsurface exploration consisting of two CPT soundings to depths of approximately 100 feet below the ground surface.
- Laboratory testing on selected soil samples, including evaluation of in-situ moisture and dry density, percentage of particles finer than the No. 200 sieve, Atterberg limits, consolidation, direct shear strength, and soil corrosivity.
- Compilation and geotechnical analyses of the information obtained from our background review, subsurface evaluation and laboratory testing.
- Preparation of this geotechnical report presenting our findings, conclusions, and recommendations for the design and construction of the proposed improvements.

3 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

Los Alamitos High School is located at 3591 West Cerritos Avenue in the city of Los Alamitos, California (Figure 1). The site of the proposed gymnasium building is located on the north central portion of the campus, bounded by the football field to the east, a parking lot to the south, an existing building and pool to the west, and softball and soccer fields to the north. The site latitude and longitude are approximately 33.813297 degrees north and -118.068715 degrees west, respectively (Google Earth, 2022). Topographically, the site is relatively flat with an elevation of approximately 28 feet above mean sea level (United States Geological Survey [USGS], 2021). The concrete lined Coyote Creek, a tributary of the San Gabriel River, is located approximately 500 feet north of the project site.

The project site is currently occupied by portable classrooms and shipping containers supported on asphalt concrete pavement. We understand that the existing structures at the project site will be removed and a new, approximately 38,000 square-foot gymnasium building will be constructed. The new gymnasium building will include basketball courts, restrooms, concession areas, offices, team rooms, storage, and a weight room. Based on our discussions and review of document provided by the design group, we understand that a 6-foot-wide electrical conduit exists along the west side of the proposed Gymnasium and is inside of the building footprint with a distance of approximately 14 inches between the proposed footing and the conduit.

4 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface evaluation was conducted on April 6 to 8, 2022 and consisted of the drilling, logging, and sampling of five small-diameter borings to depths ranging from approximately 31½ to 101½ feet and advancing two CPT soundings to depths of approximately 100 feet. The borings were drilled using a truck-mounted drill rig with 8-inch-diameter hollow-stem augers. The borings were drilled to evaluate the subsurface conditions and were logged by a representative from our firm. Bulk and relatively undisturbed soil samples were obtained at selected depths from the borings for laboratory testing. The CPT soundings were performed using a 30-ton truck-mounted CPT rig. Continuous soil profiles, including cone tip resistance and sleeve friction, were recorded during the soundings. Pore pressure dissipation tests were performed in both CPTs at selected depths. In addition, shear wave velocity measurement of the on-site soil was performed using a seismic cone in CPT-1. The borings and CPTs were backfilled with cement-bentonite grout in accordance with the requirements of the boring permit. The approximate locations of the borings and CPTs are presented on Figure 2. The boring and CPT sounding logs are presented in Appendices A and B, respectively.

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

5 GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Regional Geology Setting

The subject site is located in the Los Angeles Basin at the southern end of the Transverse Ranges geomorphic provinces of southern California (Norris and Webb, 1990). The Los Angeles Basin has been divided into four structural blocks, which are generally bounded by prominent northwest-trending and west-trending fault systems: the northwestern, southwestern, central, and northeastern blocks. The site is located on the central block that 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 block is bounded on the west by the onshore segment of the Newport-Inglewood fault zone and on the north by the Santa Monica fault zone that is located near the base of the Santa Monica Mountains. The eastern boundary is the Whittier fault zone. The Whittier fault zone becomes uncertain from north of the city of Whittier to the Santa Monica fault zone. Near the city of Corona, the Whittier fault zone merges with the Elsinore fault zone.

Regional geologic mapping indicates that the site is underlain by young alluvial flood-plain deposits (Saucedo, 2016). The alluvial deposits are described as consisting of poorly consolidated, poorly sorted, soft clay, silt and loose to moderately dense sand and silty sand. A regional geologic map is shown on Figure 3.

5.2 Subsurface Conditions

5.2.1 Existing Pavement

Structural pavement consisting of asphalt concrete (AC) underlain by aggregate base (AB) was encountered in all five borings. The AC ranged from approximately 2 to 3 inches thick and the AB ranged from 2 to 3½ inches thick. The AB generally consisted of moist, medium dense, well-graded gravel with sand.

5.2.2 Fill

Fill soils were encountered beneath the pavement sections to depths ranging from approximately 8 to 9 feet. The fill generally consisted of light brown and yellowish brown, moist, loose to medium dense, silty sand, poorly graded sand and sandy silt.

5.2.3 Alluvium

Alluvium was encountered beneath the fill to the total depths explored of up to approximately 101½ feet. The alluvial materials generally consisted light brown, brown and gray, moist to wet, loose to very dense, silty sand and sandy silt, and firm to hard, sandy lean clay.

5.3 Groundwater

Groundwater was encountered in our exploratory borings during drilling at depths ranging from approximately 9½ to 16½ feet below the ground surface. The groundwater depth encountered during drilling is not considered a stabilized water level. Fluctuations in groundwater levels may occur due to variations in precipitation, ground surface topography, subsurface stratification, irrigation, groundwater pumping, and other factors that may not have been evident at the time of our field evaluation.

Regional maps indicate that the historic high groundwater at the site is mapped as being approximately 10 feet below the ground surface (California Division of Mines and Geology [CDMG], 1998). Review of groundwater well data from a site located on the northeast corner of Norwalk Boulevard and West Cerritos Avenue (approximately 1,200 feet southwest of the site) indicates the depth to groundwater as approximately 11 feet below the ground surface (GeoTracker, 2022).

6 FLOOD HAZARDS

Based on our review of flood insurance rate maps for the project area (Federal Emergency Management Agency [FEMA], 2009), the project site is not located in the 100-year Flood Hazard Zone. The maps indicate that the site is located within a "Zone X" area with a reduced flood risk due to a levee. Zone X is defined as an area considered to have a 0.2 percent annual chance of flood; to have a 1 percent annual chance of flood with average depth of less than 1 foot or with drainage areas less than 1 square mile; or to be in an area protected by levees from 1 percent annual chance of flood (FEMA, 2009).

7 FAULTING, SEISMICITY AND GEOLOGIC HAZARDS

The site is located in a seismically active area, as is the majority of southern California. The numerous faults in southern California include active, potentially active, and inactive faults. As defined by the California Geological Survey (CGS), active faults are faults that have ruptured within Holocene time (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 site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone). Based on our review of referenced geologic literature, geologic maps, stereoscopic aerial photographs, and our geologic field reconnaissance, no active faults are known to cross the subject site. The active Newport-Inglewood fault is mapped approximately 4.2 miles (USGS, 2008) southwest of the site. The approximate locations of major active faults in the region and their geographic relationship to the site are shown on Figure 4.

An inferred buried trace of a strand of the potentially active Los Alamitos Fault has been mapped as approximately crossing the location of Los Alamitos High School (Figure 3) (Saucedo, 2016). This fault is not located on other fault maps (Figure 4) or on the State of California Seismic Hazard Zone maps (Figure 6) and is not considered to be active. Therefore, this mapped fault is not considered a hazard or constraint to the project.

Historical earthquakes, greater than magnitude 6.5 or that caused significant loss of life and property, within approximately 62 miles (100 kilometers) of the subject site are presented in Table 1. The nearest historical earthquake is the Long Beach earthquake, which occurred on March 11, 1933.

Table 1 – Historical Earthquakes

Date	Name, Location, or Region Affected	Approximate Fault to Site Distance in miles (km)	Magnitude
March 11, 1933	Long Beach	8.8 (14.1)	6.4
October 1, 1987	Whittier Narrows	17.7 (28.6)	6.0
January 17, 1994	Northridge	38.5 (61.9)	6.7
December 8, 1812	Wrightwood	45.3 (72.9)	7.3
February 9, 1971	San Fernando	45.4 (73.0)	6.6
July 22, 1899	Wrightwood	46.8 (75.3)	6.4
December 25, 1899	San Jacinto and Hemet	61.4 (98.8)	6.7
April 21, 1918	San Jacinto	61.5 (99.0)	6.8

Note:
CGS, 2022.

The principal seismic hazards that may impact the site are surface fault rupture, ground motion, liquefaction, dynamic settlement, lateral spreading, liquefaction-induced loss of bearing capacity, landsliding, and tsunamis and seiches. A brief description of these hazards and the potential for their occurrences on site are discussed in the following sections.

7.1 Surface Fault Rupture

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

7.2 Site Specific Ground Motion

Considering the proximity of the site to active faults capable of producing a maximum moment magnitude of 6.0 or more, the project area has a high potential for experiencing strong ground motion. The 2019 California Building Code (CBC) specifies that the risk-targeted maximum considered earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Based on the shear wave velocity measurement performed at CPT-1, the site shear wave velocity (Vs30) is approximately 217 meters per second (m/s). Accordingly, the site is classified as Site Class D. Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of the American Society of Civil Engineers (ASCE) Publication 7-16 (2016) for the Minimum Design Loads and Associated Criteria for Building and Other Structures. We calculated that the S_1 for the site is equal to 0.532g using the 2022 Applied Technology Council (ATC) seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project area.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum (ARS) curves corresponding to the MCE_R for 5 percent damping. 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 United States Geological Survey (USGS, 2021b) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center (PEER) was used for performing the DSHA (Seyhan, 2014).

PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the hazard curves and deaggregation plots at the site obtained from the USGS Unified Hazard Tool application (USGS, 2022b). A magnitude 7.3 event on the Compton fault with a rupture distance of 8.8 kilometers (5.5 miles) from the site was evaluated to be the controlling earthquake. Hence, the DSHA was performed for the site using this event and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. Figure 5 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The general mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 5 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2019 CBC, are provided in Section 9.2 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 0.685g.

7.3 Liquefaction Potential

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.

The State of California Seismic Hazard Zones Map (Figure 6) indicates the project area is located within an area mapped as subject to seismically induced liquefaction hazards (CDMG, 1999). The historic high depth to groundwater is mapped in the vicinity of the site as approximately 10 feet below the existing ground surface (CDMG, 1998). Groundwater was encountered during drilling at depths ranging from approximately 9½ to 16½ feet below the ground surface. Our review of the exploration results indicated a good agreement in correlation with borings and CPT soundings. However, due to the fact that CPT soundings provide nearly continuous subsurface soil strata data, liquefaction potential of subsurface soils was evaluated using the CPT soundings. The liquefaction analysis was based on the National Center for Earthquake Engineering Research (NCEER) procedure (Youd, et al., 2001) developed from the methods originally recommended by Seed and Idriss (1982) using the computer program LiquefyPro (CivilTech Software, 2019). A groundwater depth of 10 feet, a PGA_m of 0.685g, and a design earthquake magnitude of 7.3 were used in our analyses. Our liquefaction analysis indicates that the granular soil layers below the historic high depth to groundwater level and between depths of approximately 30 to 75 feet below the ground surface are susceptible to liquefaction during the design seismic event.

7.4 Liquefaction-Induced Settlement of Saturated Soils

As a result of seismically-induced liquefaction, the proposed gymnasium may be subject to 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

Under the current conditions and when using the data collected for CPT-1 and CPT-2, post-earthquake liquefaction-induced settlements of approximately 3.5 and 2.6 inches are calculated for the site, respectively. CPT-1 and CPT-2 are located at opposite ends of the proposed building, approximately 190 feet apart. Based on these test results and the guidelines presented in CGS Special Publication 117A (2008) and assuming relatively uniform subsurface stratigraphy across the site, we estimate a differential dynamic settlement on the order of 0.4 inch over a horizontal distance of 30 feet. Results of our liquefaction analysis are presented in Appendix E

With the existence of a non-liquefiable soil crust overlying liquefiable soil, the performance of a low-rise building with shallow foundations founded on the non-liquefiable soil crust during a soil liquefaction event has been observed to be generally satisfactory depending on the thickness of the non-liquefiable soil crust (Ishihara, 1995). In addition, Bouckovalas and Dakoulas (2007) have developed a design procedure to estimate the bearing capacity, the degraded post-shaking factor of safety against bearing capacity failure, and the dynamic settlement of a shallow foundation during an earthquake. Based on this procedure, our analysis indicated that the estimated dynamic settlement under a 3-foot-wide foundation footing is less than 0.1 inch for a 30-feet-thick non-liquefiable soil crust between the bottom of the footing and the underlying liquefiable soil layer at the proposed building site. Results of our dynamic settlement analysis are presented in Appendix F.

7.5 Dynamic Settlement of Dry Soils

Relatively dry soils (e.g., soils above the groundwater table) with low density or softer consistency tend to undergo dynamic settlement during a seismic event. Earthquake shaking often induces significant cyclic shear strain in a soil mass, which responds to the vibration by undergoing volumetric changes. Volumetric changes in dry soils take place primarily through changes in the void ratio (usually contraction in loose or normally consolidated, soft soils and dilation in dense or over consolidated, stiff soils) and secondarily through particle reorientation. Such volumetric changes are generally non-recoverable.

Based on our evaluation, the relatively loose soils in the upper approximately 10 feet could be susceptible to dynamic compaction of dry soils during a design earthquake. Our analysis indicated that up to approximately 1 inch of dynamic settlement of dry sand may occur during the design seismic event. However, with the remedial recommendation for overexcavation of approximately 8 feet of soil below the existing ground surface provided in Section 9.1.5 of this report, the dynamic settlement of dry sand during the design seismic event is not a design consideration.

7.6 Lateral Spread

Lateral spread of the 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, etc.) but has also been observed to a lesser extent on ground surfaces with gentle slopes. An empirical model developed by Youd, et al. (2002) is typically used to predict the amount of horizontal ground displacement within a site. For sites located in proximity to a free-face, the amount of lateral ground displacement is correlated with the distance of the site from the free-face as well as the depth of liquefiable strata which contribute to the lateral spreading. The depth of liquefiable strata below the ground surface is approximately twice of the height of the free face (Chu et al., 2006).

Other factors such as earthquake magnitude, distance from the causative fault, thickness of the liquefiable layers, and the fines content (FC) and particle sizes of the liquefiable layers also influence the amount of lateral ground displacement.

The concrete-line Coyote creek is located approximately 500 feet north of the proposed gymnasium. We estimated the height of the Coyote Creek is approximately 15 feet. Accordingly, the depth of the liquefiable soil layer contributing to lateral spreading on-site is approximately 30 feet below the existing ground surface. Due to the fine-grained nature of the soils in the upper 30 feet as well as the lacking of soil layers having corrected sampler blow counts less than 15 within the upper 30 feet, the site is not considered susceptible to seismically induced lateral spread.

7.7 Liquefaction-Induced Loss of Bearing Capacity

Our analysis also included using the residual shear strength of the liquefiable soil as recommended in the monograph by Earthquake Engineering Research Institute (EERI, 2008) to evaluate the potential for bearing capacity failure under the proposed footings. Due to the existence of a non-liquefiable soil crust (approximately 30 feet in thickness) overlying the liquefiable soils, our analysis indicated that the potential for bearing capacity failure during a seismic-induced soil liquefaction condition is low.

7.8 Landsliding

The site is located in an area of relatively flat terrain. There are no mapped landslides on site or in the vicinity. Landsliding is not considered to be a potential hazard at the site.

7.9 Tsunamis and Seiches

Tsunamis are long wavelength, seismic, sea waves (long compared to ocean depth) generated by the sudden movements 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 mapped within an area considered susceptible to tsunamis or seiches inundation. Therefore, damage due to tsunamis or seiches is not a design consideration.

8 DISCUSSIONS AND CONCLUSIONS

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

- Based on our exploratory borings, the site is underlain by fill overlying alluvial deposits. Fill was encountered to depths ranging from approximately 8 to 9 feet below the ground surface. The fill generally consisted of moist, loose to medium dense, silty sand, poorly graded sand

and sandy silt. The alluvial materials generally consisted of moist to wet, loose to very dense, silty sand and sandy silt, and firm to hard, sandy lean clay.

- Excavations into the underlying fill and alluvial deposits should be feasible with grading equipment in good working order. We anticipate that the on-site sandy soils should be generally suitable for use as compacted fill following moisture-conditioning, provided they are free of trash, debris, roots, vegetation, deleterious materials, and cobbles or hard lumps of materials in excess of 4 inches in diameter.

- Granular soils encountered at the site are anticipated to have little cohesion and may be subject to caving. These soils should be considered Type C soils in accordance with Occupational Safety and Health Administration (OSHA) soil classifications.

- Groundwater was encountered in our borings during drilling at depths ranging from approximately 9½ to 16½ feet below the ground surface. The historic high groundwater level is reported to be at approximately 10 feet below the ground surface. Fluctuations in the groundwater level may occur as a result of variations in seasonal precipitation, irrigation practices, groundwater pumping and other factors. Seepage and wet soil conditions should be anticipated during construction. Seepage should be anticipated by the contractor.

- The site is mapped within a State of California Seismic Hazards Zone as being potentially liquefiable (CDMG, 1999). Our liquefaction analysis indicated that liquefaction-induced dynamic settlement up to 3½ inches may occur during the design seismic event. Differential settlement on the order of 0.4 inch over a horizontal distance of 30 feet may be anticipated.

- The site-specific PGA_M was estimated to be 0.685g for the site.

- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). The probability of surface fault rupture is considered low at the site.

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

- The site is located within an area with a reduced flood risk due to a potential levee failure (FEMA, 2009).

- Based on our laboratory corrosion testing, the on-site soil can be classified as non-corrosive based on the Caltrans Corrosion Guidelines (Caltrans, 2021).

9 RECOMMENDATIONS

The following sections include our geotechnical recommendations for construction of the proposed improvements. Grading and building foundations plans were not available for review at the time of this report. It is important that Ninyo & Moore be notified and given an opportunity to reevaluate our recommendations once this information becomes available and prior to bidding the project for construction.

9.1 Earthwork

Earthwork at the site is anticipated to consist of remedial grading of the near-surface soils, fill placement, foundation excavations, trenching and backfilling for new utilities, pavement construction, and finish grading for establishment of site drainage. Earthwork should be performed in accordance with the requirements of applicable governing agencies and the recommendations presented in the following sections.

9.1.1 Construction Plan Review and Pre-Construction Conference

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

9.1.2 Site Clearing and Preparation

Prior to excavation and fill placement, the site should be cleared of existing site improvements, pavements, surface obstructions and other deleterious materials, and abandoned utilities and stripped of rubble, debris, and vegetation, as well as surface soils containing organic materials. Existing utilities to remain in place (if any) should be located and protected from damage by construction activities. Obstructions that extend below the finish grade, if any, 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 within the existing fill and alluvium materials at the site may be accomplished with earthmoving equipment in good working condition. The near surface fill soils encountered in the exploratory borings are comprised of moist, loose to medium dense, silty sand, poorly graded sand and sandy silt. The alluvial materials generally consisted of moist to wet, loose to very dense, silty sand and sandy silt, and firm to hard, sandy lean clay. In the event that oversize material (larger than 4 inches in longest diameter), including cobbles, is encountered during excavation operations, the oversized material is not 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

Temporary excavations above groundwater up to approximately 10 feet in depth should be stable at inclinations of up to approximately 1½:1 (horizontal to vertical). Excavations which expose friable, cohesionless sands, may be subject to caving. Some surficial sloughing may occur, and temporary excavations should be evaluated in the field in accordance with Occupational Safety and Health Administration (OSHA) guidelines. The surficial soils should be considered as OSHA Soil Type C, and temporary excavations should conform with OSHA regulations.

Temporary slope surfaces should be kept moist to retard raveling and sloughing. Water should not be allowed to flow over the top of excavations in an uncontrolled manner. Stockpiled material and/or equipment should be kept back from the top of excavations a distance equivalent to the depth of the excavation or more. Temporary excavations should be observed by the geotechnical consultant so that appropriate additional recommendations necessitated by actual field conditions may be provided. Temporary excavations are time sensitive, and failures are possible.

9.1.5 Treatment of Near Surface Soils

Based on our subsurface evaluation, it is our opinion that suitable foundation support for the proposed at-grade structure and associated improvements may be provided by remedial grading consisting of the overexcavation and recompaction of the near-surface fill soils. For the proposed construction, we recommend that the near-surface soils be overexcavated and recompacted to a depth of approximately eight (8) feet below the existing ground surface or the depth of the undocumented fill, whichever is deeper. The limits of overexcavation should extend laterally beyond the building footprint to a distance of five (5) or more feet. The actual depths and limits of overexcavation should be evaluated by our representative based on the materials exposed at the time of construction.

Due to the existence of an active electrical conduit inside the west side of the proposed building footprint, we recommend that the overexcavation in the areas where the proposed footings are parallel and adjacent to the electrical conduit as well as the 6-foot-wide electrical conduit area be excluded from the recommendations provided above. Instead, we recommend the proposed footings parallel to the electrical conduit be extended to the bottom of the electrical conduit to avoid surcharging the electrical conduit. Where the footings cross over the electrical conduit, the top of the electrical conduit should be encased with concrete designed by the project structural engineer.

Additional overexcavation of loose, soft, and/or wet areas may be appropriate, depending on our observations during construction. The subgrade at the bottom of the overexcavation should be scarified to a depth of 8 inches, moisture conditioned to slightly above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. The overexcavated area should be backfilled to the finished grade with on-site soils compacted to a relative compaction of 90 percent.

Exterior flatwork may be supported on compacted, low-expansion potential soil. Subgrade for exterior flatwork areas should be prepared by overexcavation and recompaction to a depth of approximately two (2) feet below the existing ground surface. At the bottom of the excavation, the upper approximately 8 inches of exposed subgrade should be scarified, moisture conditioned to slightly over optimum moisture content and compacted to 90 percent relative compaction as evaluated by ASTM D 1557.

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

9.1.6 Excavation Bottom Stability

Excavations close to or below the groundwater will encounter wet and loose or soft ground conditions. Excavations that expose loose/soft soils or encounter seepage or groundwater, or that become disturbed during excavation, may be unstable and subject to pumping under heavy equipment loads. In general, unstable bottom conditions may be mitigated by over-excavating to a depth of approximately 1 to 2 feet below the proposed subgrade and replacing the excavated soil with crushed aggregate base or gravel wrapped in geofabric. If aggregate base is used, it should consist of either Caltrans Class II aggregate base or crushed miscellaneous base. Caltrans Class II aggregate base should conform to the State of California Standard Specifications, Section 26 1.02A. Crushed miscellaneous base should conform to the Standard Specifications for Public Works Construction, Section 200 2.4. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by a Ninyo & Moore representative at the time of construction.

9.1.7 Fill Material

In general, the on-site sandy soils should be suitable re-use as structural fill and trench backfill provided that they are free of trash, debris, roots, vegetation, or other deleterious materials. Non-granular clay materials may be used as general fill, but should not be used as structure or trench backfill. Fill should generally be free of rocks or lumps of material in excess of 4 inches in diameter. Rocks or hard lumps larger than approximately 4 inches in diameter should be broken into smaller pieces or should be removed from the site. Structure backfill should be comprised of granular, non-expansive soil that conforms to the latest edition of "Greenbook" Standard Specifications for Public Works Construction for structural backfill. "Non-expansive" can be defined as soil having an expansion index (EI) of 20 or less in accordance with ASTM D 4829. The on-site materials will involve moisture-conditioning to achieve appropriate moisture content for compaction.

Imported materials, if used, should consist of clean, non-expansive, granular material, which conforms to the "Greenbook" for structure backfill. The imported materials should also meet the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a minimum resistivity greater than 1,500 ohm-cm, a chloride concentration less than 500 parts per million [ppm], a sulfate concentration of less than 0.15 percent (1,500 ppm), and a pH value greater than 5.5). Import materials for use as fill should be evaluated by the geotechnical consultant prior to importing. The contractor should be responsible for the uniformity of import material brought to the site.

9.1.8 Fill Placement and Compaction

Fill soils placed should be compacted in horizontal lifts to a relative compaction of 90 percent as evaluated by ASTM D 1557. The lift thickness for fill soils will vary depending on the type of compaction equipment used but should generally be placed in horizontal lifts not exceeding 8 inches in loose thickness. Fill soils should be placed at generally slightly above the optimum moisture content as evaluated by ASTM D 1557. Special care should be taken to avoid damage to utility lines when compacting fill and subgrade materials.

9.2 Site-Specific Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 2 presents the site-specific spectral response acceleration parameters in accordance with the CBC (2019) guidelines.

Table 2 – 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	1.491g
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.532g
Site-Specific Spectral Response Acceleration at 0.2-second Period, S_{MS}	1.637g
Site-Specific Spectral Response Acceleration at 1.0-second Period, S_{M1}	2.036g
Site-Specific Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.091g
Site-Specific Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.357g
Site-Specific Mapped Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration, PGA_M	0.685g

9.3 Foundations

The proposed gymnasium may be supported on shallow foundations including spread and continuous footings bearing on engineered fill material compacted in accordance with the recommendations presented in the Earthwork section of this report. 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

Spread footings for the proposed gymnasium should extend 24 inches or more below the adjacent finished grade. Continuous and isolated pad footings should have a width of 36 inches or more. Continuous footings should be reinforced with four No. 4 steel reinforcing bars, two placed near the top and two placed near the bottom of the footings, and further detailed in accordance with the recommendations of the structural engineer.

Footings, as described above, may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf). The bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Total and differential settlements for footings designed and constructed in accordance with the above recommendations are estimated to be less than approximately 1 and ½ inch over a horizontal span of 30 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 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 spread footings. If trenches are to be excavated near a continuous footing, the bottom of the trench should be located above a 1:1 (horizontal to vertical) plane projected downward from the bottom of the footing. Utility lines that cross beneath footings should be encased in concrete below the footing. In addition, footings constructed near existing underground utility lines should be deepened such that the utility line is located above a 1:1 (horizontal to vertical) plane projected downward from the base of the footing.

9.3.2 Slabs-On-Grade

Buildings supported on shallow footings should have floor slabs designed by the project structural engineer based on the anticipated loading conditions. Building floor slabs should be underlain by compacted fill prepared in accordance with the recommendations presented in this report. As a minimum we recommend that slabs have a thickness of 5 inches or more, and be reinforced with No. 4 steel reinforcing bars placed 18 inches on-center (each way) in the middle one-third of the slab height. Exterior slabs-on-grade may be 4 inches thick. The proper placement of the reinforcement in the slab is vital for satisfactory performance. The floor slab and foundations should be tied together by extending the slab reinforcement into the footings. The slab should be underlain by a polyethylene vapor retarder, 10-mil or thicker. The vapor retarder should further be underlain by a 4-inch-thick layer of sand or gravel with a particle size of approximately $^3/_4$ inch or smaller. The vapor retarder is recommended in areas where moisture sensitive floor coverings are anticipated. Soils underlying the slabs should be moisture conditioned and compacted in accordance with the recommendations contained in this report prior to concrete placement. Joints should be constructed at intervals designed by the structural engineer to help reduce random cracking of the slab.

9.4 Underground Utilities

We anticipate that utility pipelines will be supported on compacted fill or alluvial deposits. The depths of the pipelines are not known; however, we anticipate that the pipe invert depths will not exceed 5 feet. Trenches should not be excavated parallel to building footings. If needed, trenches can be excavated adjacent to a continuous footing, provided that the bottom of the trench is located above a 1:1 (horizontal to vertical) plane projected downward from a point 6 inches above the bottom of the adjacent footing. Utility lines that cross beneath footings should be encased in concrete below the footing. To reduce the potential for pipe to building differential settlement due to liquefaction which could cause pipe shearing; we recommend that a pipe joint be located close to the exterior of the building. The type of joint should be such that relative movement can be accommodated without distress. The pipe connections should be sufficiently flexible to withstand differential settlement on the order of 1½ inches.

9.4.1 Pipe Bedding

We recommend that 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 gravel be used for bedding material. It has been our experience that the voids within gravel 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.

Special care should be taken not to allow voids beneath and around the pipe. Compaction of the bedding material and backfill should proceed along both sides of the pipe concurrently. Trench backfill, including bedding material, should be placed in accordance with the recommendations presented in the Earthwork section of this report.

9.4.2 Trench Backfill

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

9.4.3 Modulus of Soil Reaction

The modulus of soil reaction is used to characterize the stiffness of soil backfill placed on the sides of buried flexible pipelines for the purpose of evaluating lateral deflection caused by the weight of the backfill above the pipe. We recommend that a modulus of soil reaction of

1,000 pounds per square inch be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

9.5 Sidewalks and Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a thickness of 4 inches and be reinforced with No. 3 steel reinforcing bars placed 24 inches on-center (each way) near the mid-height of the slab. The hardscape should be underlain by 4 inches of clean sand and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, joints between the new hardscape and adjacent curbs, existing hardscape, building walls, and/or other structures, and between sections of new hardscape, should be doweled.

9.6 Corrosivity

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

The soil pH was measured at approximately 7.9 and the electrical resistivity was measured to be approximately 4,485 ohm-centimeters. The chloride content of the sample was measured to be approximately 30 ppm. The sulfate content of the tested sample was approximately 0.001 percent (10 ppm). Based on the laboratory test results and Caltrans (2021) criteria, the project site can be classified as a non-corrosive site, which is defined as having earth materials with less than 500 ppm chlorides, less than 0.15 percent sulfates (i.e., 1,500 ppm), a pH of 5.5 or more, or an electrical resistivity of more than 1,500 ohm-centimeters. If corrosion susceptible improvements are planned on site, we recommend that a corrosion engineer be consulted for further evaluation and recommendations.

9.7 Concrete Placement

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. Based on the CBC (2019), the potential for sulfate attack is negligible for water-soluble sulfate contents in soil ranging from 0.00 to 0.10 percent by weight, moderate for water-soluble sulfate contents ranging from 0.10 to

0.20 percent by weight, severe for water-soluble sulfate contents ranging from 0.20 to 2.00 percent by weight, and very severe for water-soluble sulfate contents over 2.00 percent by weight. The soil sample tested for this evaluation, using Caltrans Test Method 417, indicates a water-soluble sulfate content of approximately 0.001 percent by weight (i.e., 10 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. However, due to the potential variability of the 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 structures be placed with a slump of 4 inches based on ASTM C 143. The slump should be checked periodically at the site prior to concrete placement. We further recommend that concrete cover over reinforcing steel for foundations be provided in accordance with CBC (2019). The structural engineer should be consulted for additional concrete specifications.

9.8 Drainage

Good surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to channel surface water away from foundations and off-site. Positive drainage is defined as a slope of two percent or more for a distance of 5 feet or more away from foundations and tops of slopes. Runoff should then be transported by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to foundations or on pavements. Concentrated runoff should not be allowed to flow over asphalt pavement as this can result in early deterioration of the pavement. We recommend that structures have roof drains and downspouts installed to collect runoff. 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 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 clearing, grubbing, and removals.
- Observing excavation bottoms and the placement and compaction of fill, including trench backfill.
- 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.

The recommendations provided in this report assume that Ninyo & Moore will be retained as the geotechnical consultant during the construction phase of this project. 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 environmental concerns or the presence of hazardous materials.

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

This report is intended for design purposes only. It does not provide sufficient data to prepare an accurate bid by contractors. It is suggested that the bidders and their geotechnical consultant perform an independent evaluation of the subsurface conditions in the project areas. The independent evaluations may include, but not be limited to, review of other geotechnical reports prepared for the adjacent areas, site reconnaissance, and additional exploration and laboratory testing.

Our conclusions, recommendations, and opinions are based on an analysis of the observed site conditions. If geotechnical conditions different from those described in this report are encountered, our office should be notified, and additional recommendations, if warranted, will be provided upon request. It should be understood that the conditions of a site could change with time as a result of natural processes or the activities of man at the subject site or nearby sites. In addition, changes to the applicable laws, regulations, codes, and standards of practice may occur due to government action or the broadening of knowledge. The findings of this report may, therefore, be invalidated over time, in part or in whole, by changes over which Ninyo & Moore has no control.

This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

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FIGURES

APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

Field Procedure for the Collection of Disturbed Samples

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

Bulk Samples

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

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of $1^{3}/_{8}$ inches. The sampler was driven into the ground 12 to 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM International (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 sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

Shelby Tube

The Shelby tube is a seamless, thin-walled, steel tube having an external diameter of 2.4 or 3.0 inches and a length of 8 to 30 inches. The tube was connected to the drill rod or a hand tool and pushed into an undisturbed soil mass to obtain a relatively undisturbed sample of soft, cohesive soil in general accordance with ASTM D 1587. When the tube was almost full (to avoid overpenetration), it was withdrawn from the excavation, removed from the drill rod or hand tool, sealed at both ends, and transported to the laboratory for testing.

APPENDIX B

Cone Penetration Test Data

APPENDIX C

Laboratory Testing

APPENDIX C

LABORATORY TESTING

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

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

200 Wash
An evaluation of the percentage of minus-200 sieve material in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure C-1.

Atterberg Limits
Tests were performed on selected representative fine-grained soil samples to evaluate the liquid limit, plastic limit, and plasticity index in general accordance with ASTM D 4318. The test results were utilized to evaluate the soil classification in accordance with the Unified Soil Classification System (USCS). The test results and classifications are shown on Figures C-2 and C-3.

Consolidation Tests
Consolidation tests were performed on selected relatively undisturbed soil samples in general accordance with ASTM D 2435. The samples were inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the tests are summarized on Figure C-4 and C-5.

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 selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figures C-6 and C-7.

Soil Corrosivity Tests
Soil pH and minimum resistivity tests were performed on a representative soil sample in general accordance with California Test (CT) 643. The chloride content of the selected sample was evaluated in general accordance with CT 422. The sulfate content of the selected sample was evaluated in general accordance with CT 417. The test results are presented on Figure C-8.

APPENDIX D

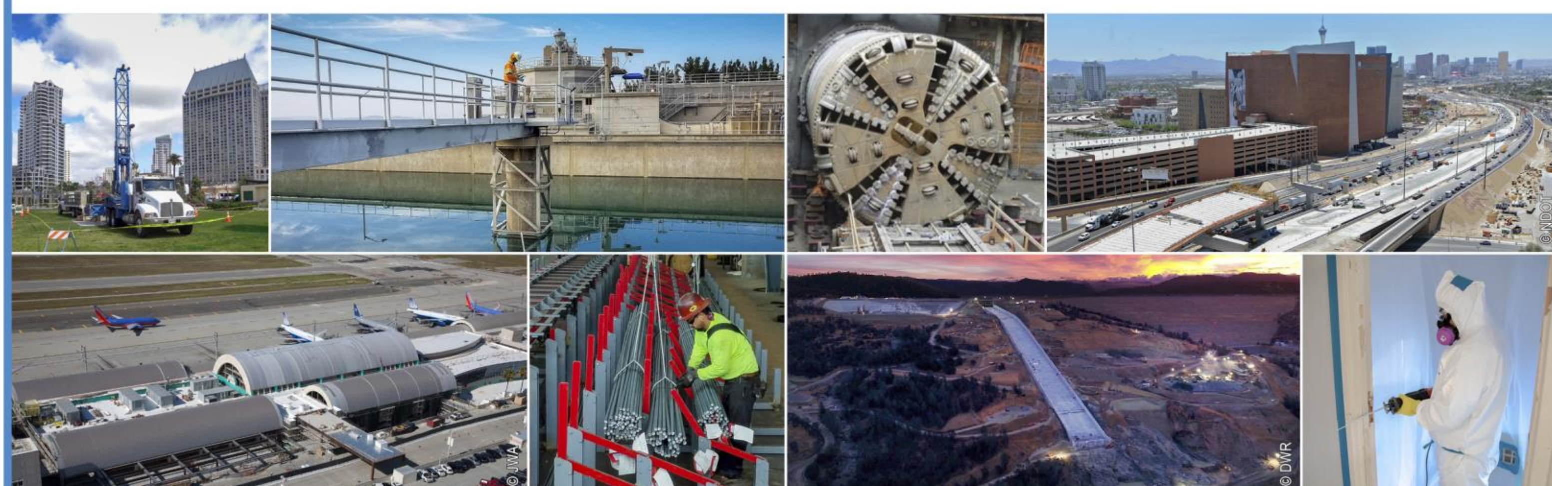
Site Specific Ground Motion Analysis

APPENDIX E

Liquefaction Analysis

APPENDIX F

Dynamic Settlement of Shallow Foundations Analysis



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