California
Department of Conservation
California Geological Survey

CALIFORNIA GEOLOGICAL SURVEY
DEPARTMENT OF CONSERVATION

APPLICATION
FOR ASSESSMENT OF GEOLOGIC HAZARD REPORTS

CGS Form 1A (8/2022)

For CGS use only
CGS project number ________________
Date received ________________

In order for CGS to review geologic hazard reports for a proposed school project, as described on Division of the State Architect (DSA) Interpretation of Regulations IR-4 (see https://www.dgs.ca.gov/DSA/Publications#IRs), the following material must be submitted to CGS.

1. Upload to Box (https://www.conservation.ca.gov/cgs/upload-school):
   - this form; and site plan; and site data report
   - Geologic Hazard Report(s) and Geotechnical Report(s) to be reviewed
2. Mail to CGS:
   - this form, which will help CGS and the DSA coordinate reviews;
   - TWO *WET-SIGNED COPIES* of the Work Order (below), signed by an authorized representative of the District;
   - a check for $4,800 to cover the time and materials needed for CGS review

Address: California Geological Survey
School Review Unit
715 P Street, MS 1901
Sacramento, CA 95814

Name of School:	Bennett/Kew Leadership Academy of Excellence		
School District or State Agency:	Inglewood Unified School District		
Mailing Address (street, city, zip):	401 S Inglewood Ave. Inglewood CA. 90301		
District Superintendent:	James Morris Ed.D.		
Telephone Number:	(310) 419-2705	E-mail Address:	james.morris@inglewoodusd.com
District Director of Facilities:	Jordan Miles		
Telephone Number:	(310) 680-4837	E-mail Address:	jordan.miles@inglewoodusd.com

Scope of Work: New building with six classrooms, one makerspace, gendered restrooms, a faculty restroom, and a storage room.
Applicable Building Code (year): 2022 Community College Project per: ☐ DSA-SS, or ☐ DSA-SS/CC amendments
This project includes a site-specific ground motion analysis in accordance with: ☐ none ☑ ASCE 7 ☐ ASCE 41
Project location (Street Address): 11710 South Cherry Avenue
City and Zip Code: Inglewood, 90303 County: LA
DSA Application Number (if assigned):

California
Department of Conservation
California Geological Survey

CALIFORNIA GEOLOGICAL SURVEY

DEPARTMENT OF CONSERVATION

WORK ORDER

FOR ASSESSMENT OF GEOLOGIC HAZARD REPORTS

CGS Form 1B (8/2022)

The parties to this Work Order are the State of California, Department of Conservation, California Geological Survey (CGS) and Inglewood Unified School District (District). The Parties agree to the following terms and conditions:

1. CGS agrees to conduct an independent assessment of District-provided geologic hazard report(s) associated with the District's proposed school construction project to determine whether the reports are technically adequate.

2. The State of California, Department of General Services, Division of the State Architect (DSA) will rely upon the CGS technical assessment in reviewing plans for construction of the District's proposed construction project and permitting the project. Information regarding CGS assessment of district geologic hazard reports and the DSA's instructions to K-12 and community college districts regarding the CGS assessment can be found in DSA Interpretation of Regulation (IR A-4) at https://www.dgs.ca.gov/DSA/Publications#IRs

3. The District shall list the specific reports to be reviewed by CGS in the Application (above). The District shall provide copies of the reports to CGS when submitting the signed Work Order and payment, as described below.

4. The District shall provide any additional information determined by CGS to be needed to complete its assessment.

5. The term of this Work Order shall begin upon full execution of the Work Order by both parties and shall end in 365 days or 12 months, whichever occurs first. "Full execution" as used herein means approval by authorized representatives of both Parties and payment to CGS of four thousand eight hundred dollars ($4,800) in consideration of the promise by CGS to perform the technical assessment. Payment in full shall accompany two copies of this Work Order, each containing an original signature of a District representative authorized to sign the Work Order. CGS will return a copy of the Work Order containing an original signature of its authorized representative upon execution of the Work Order.

6. Failure of the District to submit the necessary documents or the $4,800 payment will result in termination of this Work Order.

7. No amendment or variation of the terms of this Work Order shall be valid unless made in writing and signed by both Parties. No oral understanding not incorporated into this Work Order is binding on either Party.

8. Either Party, in writing, may terminate this Work Order at any time with 30 days written notice; however, should the District terminate this Work Order after work has been commenced by CGS, CGS will retain the $4,800 payment for any work completed by CGS prior to the notice of termination.

September 10th, 2024

Ms. Margaret Hyland
California Geological Survey
School Review Unit
801 K Street, MS 12-31
Sacramento, CA 95814-3531

Subject: Site Data Report
Inglewood Unified School District Bennett Kew New Classroom Building
Inglewood, CA
Project No. 2023-IU002-002

Dear Ms. Hyland: Per your request.

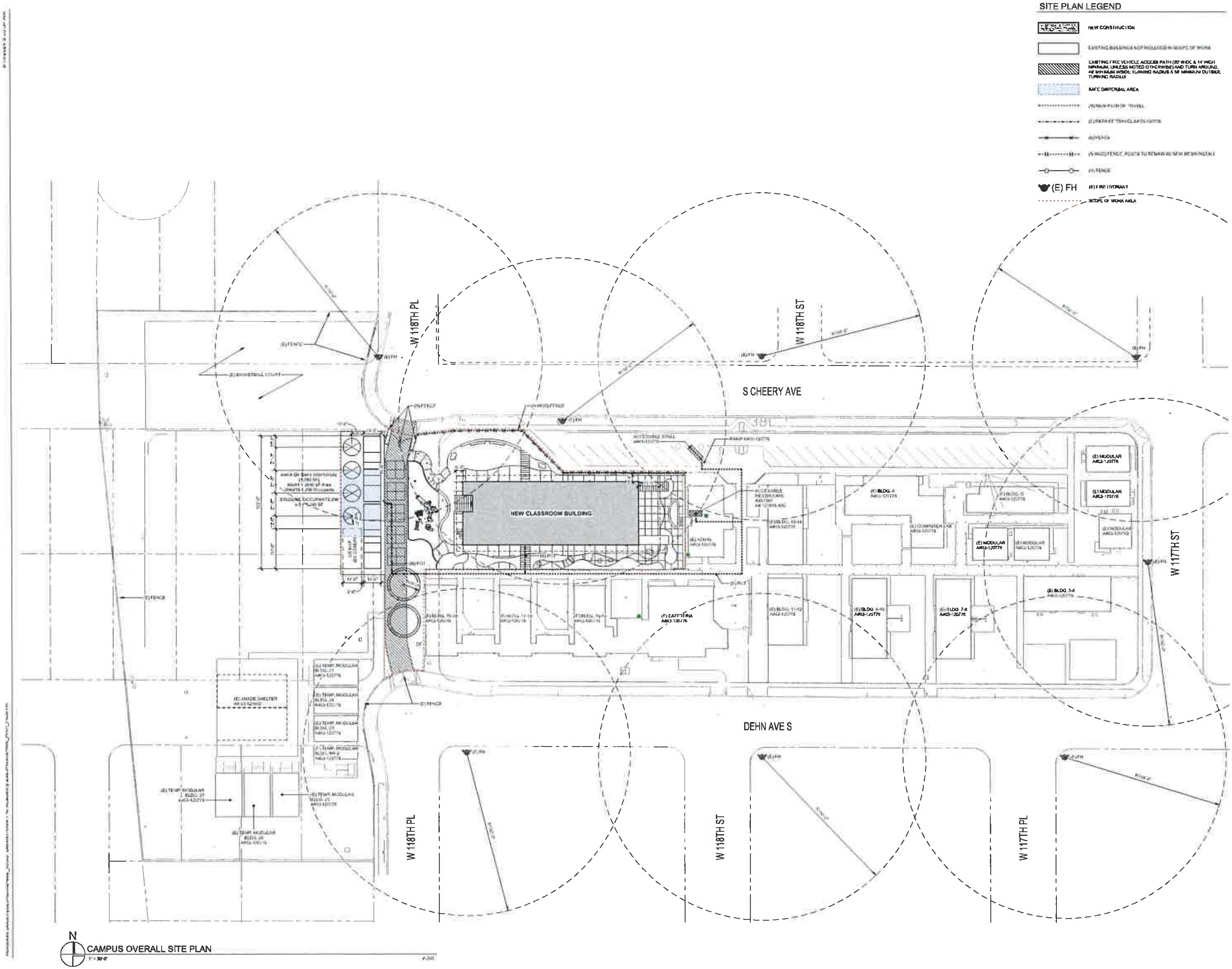
SITE DATA REPORT

1. Type of Service:
   K-12 Classrooms Building
2. Construction Materials:
   Type V-B wood framed construction
3. Type of Construction:
   New Classroom Building
4. Seismic Force Resisting System:
   Shear wall
5. Foundation System:
   Conventional shallow footings
6. Analysis Procedure Used for BOD:
   Modal Response Spectrum Analysis
7. Building Characteristics:
   - New 1-story Classroom Building
     - First floor 9,500 sf
8. Special Features:
   None

Respectfully submitted,

William McCarthy AIA, LEED AP
Associate | Architecture

213-542-4500
550 South Hope St.,
Suite 2500
Los Angeles, CA 90071



Inglewood Unifie
School District

IUSD Bennett-K
P-8 Academy

Δ	Date	Issued For
1		
2		Development
3		Construction Documents

NOT FOR CONSTRUCTION

FOR REFERENCE ONLY

HED

Site Plan

AS101

Updated Geotechnical Evaluation

Bennett/Kew Elementary School

11710 South Cherry Avenue

Inglewood, California

Cordoba Corporation

401 South Inglewood Avenue | Inglewood, California 90301

June 11, 2024 | Project No. 209822017

Geotechnical | Environmental | Construction Inspection & Testing | Forensic Engineering & Expert Witness

Geophysics | Engineering Geology | Laboratory Testing | Industrial Hygiene | Occupational Safety | Air Quality | GIS

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

35+ YEARS

Updated Geotechnical Evaluation
Bennett/Kew Elementary School
11710 South Cherry Avenue
Inglewood, California

Ms. Stephanie Pulcifer, Design and Construction Manager
Cordoba Corporation
401 South Inglewood Avenue | Inglewood, California 90301

June 11, 2024 | Project No. 209822017

Rosalie Chavez, EIT
Senior Staff Engineer

Ronald Hallum, PG, CEG
Principal Geologist

CERTIFIED ENGINEERING GEOLOGIST
RONALD D HALLUM
No. 1484
STATE OF CALIFORNIA

Soumitra Guha, PhD, PE, GE
Principal Engineer

REGISTERED PROFESSIONAL ENGINEER
SOUMITRA GUHA
NO. GE 2632
GEOTECHNICAL
STATE OF CALIFORNIA

RVC/RDH/SG/scs

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1 INTRODUCTION

In accordance with your request, we have performed an updated geotechnical evaluation for two new buildings at Bennett/Kew School located at 11710 South Cherry Avenue in Inglewood, California (Figure 1). We understand that the two new buildings will be comprised of one-story and two-story, at-grade structures. The purpose of our geotechnical services was to evaluate the soil, geologic and groundwater conditions at the project site and to provide conclusions and recommendations regarding the geotechnical aspects of the planned demolition and new construction.

We prepared a geotechnical evaluation report for the project in 2019 (Ninyo & Moore, 2019). However, the project was delayed for several years and we were recently requested to update our 2019 report in accordance with the 2022 California Building Code (CBC) requirements. The design and construction recommendations presented in this updated report supersede our previous report.

2 SCOPE OF SERVICES

Our original scope of services included the following:

- Project coordination, planning, and scheduling of the subsurface exploration
- Review of readily available background material, including published geologic maps, fault and seismic hazards maps, groundwater data, topographic maps, stereoscopic aerial photographs, and project-related plans provided by the client.
- Permit acquisition from the Los Angeles County Department of Public Health.
- Geotechnical reconnaissance to observe and document the existing site conditions and to mark proposed boring locations for utility clearance with Underground Service Alert.
- Subsurface exploration consisting of the drilling, logging and sampling of five hollow-stem auger borings using truck-mounted equipment. The borings were advanced to depths ranging from approximately 5 to 26.5 feet. The borings were logged by representatives from our firm, and bulk and relatively undisturbed soil samples collected at selected intervals for laboratory testing.
- Geotechnical laboratory testing of representative soil samples to evaluate in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, gradation, Atterberg limits, Proctor density, consolidation, direct shear strength, expansion potential, R-value, and soil corrosivity.
- Compilation and geotechnical analysis of field and laboratory data
- Preparation of a geotechnical report dated June 24, 2019, presenting our findings, conclusions, and recommendations regarding the proposed project.

Our additional scope of work included:

- Performance of a refraction microtremor (ReMi) geophysical field survey to evaluate the average shear wave velocity (V_{S30}) at the project site.
- Review and update the seismic analyses performed in 2019 and update it to provide appropriate 2022 CBC seismic design parameters.
- Preparation of this updated geotechnical report for this project presenting our original and updated findings, conclusions, and recommendations regarding the proposed project.

3 SITE DESCRIPTION AND PROJECT DESCRIPTION

The site is located within the southern portion of the Bennett/Kew Elementary School campus (Figure 2). The site is bounded by existing classroom buildings to the north and east, an existing grass field to the south, South Cherry Avenue and a parking area to the west. The site is currently an outdoor recreation area consisting of tetherball and wall ball courts paved with asphalt concrete (AC).

The project area is located at approximately latitude 33.9265 degrees north and longitude 118.3328 degrees west. Topography in the vicinity of the project area is relatively level with an approximate ground elevation of approximately 60 feet above the mean sea level (MSL) (United States Geological Survey [USGS], 2018).

We understand that the project will include the construction of a two-story, at-grade classroom building with a footprint area of approximately 8,400 square feet, and a one-story, at-grade, building with a footprint area of approximately 7,200 square feet (HED Design, 2019) (Figure 3). We anticipate that the earthwork will consist of shallow cuts and fills of up to about 4 feet in depth.

4 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface evaluation was conducted on May 4, 2019, and consisted of the drilling, logging, and sampling of five small-diameter exploratory borings to depths ranging from approximately 5 to 26.5 feet. The exploratory borings were drilled using truck-mounted drilling equipment fitted with hollow-stem augers. The purpose of the borings was to evaluate the subsurface conditions at the project site and to collect bulk and relatively undisturbed soil samples for laboratory testing. Logs of the exploratory borings are presented in Appendix A. The approximate locations of the borings are presented on Figures 2 and 3.

Laboratory testing was performed on the representative samples retrieved from our exploratory borings to evaluate in-situ moisture content and dry density, percentage of particles finer than the No. 200 sieve, gradation, Atterberg limits, Proctor density, consolidation, direct shear strength, expansion potential, R-value, and soil corrosivity (soil pH, electrical resistivity, water-soluble sulfate content, and chloride content). The results of the in-situ moisture content and dry density tests are presented on the boring logs in Appendix A. The remaining laboratory test results are presented in Appendix B.

5 FIELD PERCOLATION TESTING

In-situ percolation testing was performed on May 4, 2019, in boring P-1 to evaluate the infiltration rate of the on-site soils in general accordance with the County of Los Angeles Department of Public Works (CLADPW) guidelines (CLADPW, 2017). The testing included the placement of a 2-inch diameter slotted polyvinyl chloride (PVC) pipe to a depth of approximately 5 feet below the ground surface. Gravel was then placed around the annular space of the pipe. The borings were pre-saturated for an hour prior to testing. Percolation testing was conducted by measuring the volume of water drop from the initial water depth over eight consecutive readings. The results of our field percolation testing, including the depth interval tested, are presented in Table 1.

Table 1 – Percolation Test Result

Boring Location	Depth Interval (feet)	Soil Type at Test Interval (USCS Classification)	Field Measured Percolation Rate (inch/hour)
P-1	0.0-5.0	CL	1.0

Notes:
USCS – Unified Soils Classification System

The 2017 guidelines indicate that the design infiltration rate should be evaluated by the civil engineer or infiltration system designer by dividing the field measured percolation rate by a Total Reduction Factor (RF). The RF should be calculated using the field Reduction Factor (RF_t) multiplied by site-specific reduction factors for site variability (RF_v) and long-term siltation (RF_s). The RF_t is used to account for non-vertical flow from the sides of the boring. Per the CLADPW guidelines, a RF_t of 2 should be used. The RF_v is used to account for site variability and the number of tests performed. A RF_v of 2 is appropriate for this project. The RF_s is used to account for long-term siltation and plugging of the infiltration system. The RF_s should be provided by the infiltration system designer based on the type of storm water infiltration system and planned maintenance programs (CLADPW, 2017).

$$\text{Design Infiltration Rate}\left(\frac{\text{inches}}{\text{hour}}\right) = \frac{\text{Measured Percolation Rate}\left(\frac{\text{inches}}{\text{hour}}\right)}{\text{RF}}$$

where $RF = RF_t \times RF_v \times RF_s$

6 REFRACTION MICROTREMOR SURVEY

On May 22, 2024, Ninyo & Moore performed a seismic refraction survey at the project site using the ReMi method to measure the shear wave velocity of site soils from the ground surface to a depth of approximately 100 feet. The average shear wave velocity in the upper 100 feet or 30 meters (V_{s30}) was calculated and used to evaluate the seismic Site Classification in accordance with Chapter 20 of American Society of Civil Engineers (ASCE) publication 7-16. The approximate length and orientation of the ReMi survey line are depicted on Figure 2 and Figure 3. The survey results are presented in Appendix C.

A Geode 24–Channel Seismograph (Geometrics Inc.) was used for the ReMi survey with 4.5 Hertz (Hz) vertical component geophones spaced 10 feet apart for a total profile length of 230 feet. Approximately 30 records were collected, with a record length of 30 seconds (s) and a sample interval of 2 milliseconds (ms). The field data were digitally recorded in SEG-Y format, reviewed in the field for data quality, saved to a hard disk, and documented.

The ReMi seismic data were processed using SeisImager/SW Analysis of Surface Waves software. The dispersive characteristics of surface waves are used to evaluate the subsurface velocity at depth. Longer wavelength (i.e., longer period and lower frequency) surface waves travel deeper and thus contain more information about deeper velocity structure. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure. The dispersion is dependent on the material properties, such as surface wave velocity, relative material densities, and Poisson's ratio. An inversion is performed on the collected passive seismic shear wave records to produce a model of the variation in shear wave velocities with depth with a convergence of model that yielded less than 5 percent root mean square error.

Shear wave data resolution generally decreases with depth, due to the loss of sensitivity of the dispersion curve to changes in shear wave velocity as depth increases. The layered models indicate our interpretation of the approximate changes in shear wave velocity vertically with depth across the surveyed locations. The calculated shear wave velocities in the upper 100 feet (V_{S30}) at the location of the geophone arrays for the site resulted in an average value of approximately 1,100 feet per second (fps). Based on this information and the guidelines presented in Chapter 20 of ASCE publication 7-16, Site Class D may be assigned to this site for design purposes.

7 GEOLOGY AND SUBSURFACE CONDITIONS

The subject site is located in the Los Angeles Basin, which is situated at the northwest end of the Peninsular Ranges geomorphic provinces of southern California (Norris and Webb, 1990). The Los Angeles Basin has been divided into four structural blocks, which are generally bounded by prominent northwest-trending fault systems: the northwestern, southwestern, central, and northeastern blocks. The site is located in the southwestern block, which is bounded by the Newport-Inglewood fault to the northeast, the Palos Verdes Hills fault to the southwest, and the Santa Monica-Hollywood-Raymond fault system to the northwest. The block is underlain by up to approximately 20,500 feet of Miocene to Pleistocene-age marine sedimentary rock over basement rock consisting of the Mesozoic age Catalina Schist. Variable thicknesses of late Pleistocene to Holocene-age alluvial deposits associated with the ancestral Los Angeles and San Gabriel Rivers generally overlie the sedimentary rock (Norris and Webb, 1990). Based on our review, the subject site is underlain by moderately well consolidated and poorly sorted middle to late Pleistocene-age old alluvial deposits consisting of clay, silt, sand, and gravel (Saucedo, et al., 2016). A representative cross-section of the site in the north-south direction showing the general geologic profile is depicted on Figure 4, and a broader view of the regional geology is presented on Figure 5.

The materials encountered during our subsurface exploration at the site include asphalt concrete pavement underlain by native alluvium to the total depths of up to approximately 26.5 feet. Structural pavement consisting of AC underlain by an aggregate base (AB) layer was encountered in all our borings. The AC pavement encountered in our boring was approximately 1.5 to 2.5 inches thick and the AB thickness ranged from approximately 1 to 2 inches. The alluvial deposits generally consisted of moist, very stiff to hard, lean clay and medium dense to very dense, sandy silt, clayey sand, silty sand, and poorly graded sand with clay. Detailed descriptions of the subsurface conditions are presented on the boring logs in Appendix A.

8 GROUNDWATER

Groundwater was not encountered in our exploratory borings at the time of drilling. The historic high depth to groundwater is mapped in the vicinity of the site at approximately 30 feet below the existing ground surface (California Geological Survey [CGS], 1998). Review of groundwater monitoring data from a well located approximately 1.3 miles northwest of the site indicates a groundwater depth of approximately 37 feet below the ground surface (GeoTracker, 2019).

It should be noted that fluctuations in groundwater levels may occur due to variations in ground surface topography, subsurface stratification, precipitation, irrigation, groundwater pumping, and other factors which may not have been evident at the time of our field evaluation.

9 FLOOD HAZARDS

Based on review of the Flood Insurance Rate Map, the site is mapped in an area considered outside the 0.2 percent chance of flooding (Federal Emergency Management Agency, 2008). Based on this review, the potential for flooding at the project site is considered to be very low.

10 FAULTING AND SEISMICITY

The project site is not located within a State of California Earthquake Fault Zone (formerly known as an Alquist-Priolo Special Studies Zone [Hart and Bryant, 2007]). However, the site is located in a seismically active area, as is the majority of southern California, and the potential for strong ground motion in the project area is considered to be significant during the design life of the project. Figure 6 shows the approximate site location relative to the major faults in the region.

The principal seismic hazards evaluated at the subject site are surface fault rupture, ground motion, and liquefaction. A brief description of these hazards and the potential for their occurrences at the site are discussed below.

10.1 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.

10.2 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 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. Using the results of our seismic refraction survey at the site, we calculated that the average shear wave velocity in the upper 30 meters (i.e., 100 feet) of the subsurface profile (V_{S30}) is approximately 335 meters per second (i.e., 1,100 feet per second). 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 therefore D.

Per the 2022 CBC, a site-specific ground motion hazard analysis shall be performed in accordance with Section 21.2 of ASCE 7-16 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. We calculated that the S_1 for the site is equal to 0.656g using the 2024 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. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the mapped MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16 using the 2024 ATC seismic design tool. The depths to V_S = 1,000 m/s and V_S = 2,500 m/s are assumed to be 600 meters and 4,250 meters, respectively (Southern California Earthquake Center [SCEC], 2024). These values were evaluated using the Open Seismic Hazard Analysis (OpenSHA) software developed by USGS (2021).

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 OpenSHA software (USGS, 2021) 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 Section 21.2.1.1 of 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, 2024). A magnitude 7.2 event on the Newport-Inglewood fault with a rupture distance of 1.48 kilometers (0.9 miles) and a magnitude 7.5 event on the Compton fault with a rupture distance of 12.1 kilometers (7.5 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 8 presents the site-specific MCE_R response spectrum and the site-specific design response spectrum. The mapped design response spectrum calculated in accordance with Section 11.4 of ASCE 7-16 is also presented on Figure 8 for comparison. The site-specific spectral response acceleration parameters, consistent with the 2022 CBC, are provided in Section 12.2 for the evaluation of seismic loads on buildings and other structures.

ASCE 7-16 specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the maximum considered earthquake geometric mean (MCE_G) peak ground acceleration adjusted for site effects (PGA_M). The PGA_M is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The site-specific PGA_M was calculated as 0.898g.

10.3 Liquefaction

Liquefaction is the phenomenon in which loosely deposited granular soils and cohesionless fine-grained soils located below the water table undergo rapid loss of shear strength due to excess pore pressure generation when subjected to strong earthquake-induced ground shaking. Sufficient ground shaking duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure. This causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 50 feet below 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.

Based on our review of the State of California Seismic Hazard Zones map (CGS, 1999), the project site is not located in an area mapped as being potentially susceptible to liquefaction. Due to the fine-grained nature of on-site soil and depth to the historic high groundwater level (approximately 30 feet below the existing ground surface), it is our opinion that the site is not susceptible to significant soil liquefaction.

11 CONCLUSIONS

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

- The site is generally underlain by alluvial deposits consisting of very stiff to hard, lean clay and medium dense to very dense, sandy silt, clayey sand, silty sand, and poorly graded sand with clay.
- Site grading for new improvements should be feasible with heavy earthmoving equipment in good working condition.
- We anticipate that the materials generated during the excavation of the alluvial soils should be generally suitable for reuse as compacted fill provided that they are free of trash, debris, roots, vegetation, other deleterious materials, and contamination.
- On-site soils should be considered as Type C soils in accordance with the Occupational Safety and Health Administration (OSHA) soil classifications. Sandy soil may be prone to caving during earthwork operations.
- Based on limited laboratory test results, the near-surface site soils can be classified as non-corrosive based on the California Department of Transportation (Caltrans, 2021) corrosion guidelines.
- Laboratory testing indicates that the near-surface fine-grained soils have a very low potential for expansion.
- The subject site is not located within a State of California EFZ. The potential for surface fault rupture as defined by the Alquist-Priolo Earthquake Fault Zoning Act is considered to be relatively low.
- Groundwater was not encountered during the drilling of our exploratory borings. Groundwater levels are, however, subject to variation depending on rainfall, irrigation, groundwater pumping and other factors. Seepage should be anticipated during construction activities.
- Based on our review of the State of California Seismic Hazard Zones map (CGS, 1998), the subject site is not located in an area mapped as being potentially liquefiable. The potential for dynamic settlement due to liquefaction is not a design consideration for the project.
- Based on our evaluation and the guidelines presented in Chapter 20 of ASCE publication 7-16, Site Class D may be assigned to this site for design purposes.
- The PGA_M was calculated to be 0.898g for the site.

12 RECOMMENDATIONS

The following sections present our geotechnical recommendations for the proposed improvements at the project site. These recommendations are based on our evaluation of the site geotechnical conditions and our understanding of the planned project. The project design and construction should be performed in conformance with the recommendations presented in this report, project specifications, and appropriate agency standards.

12.1 Earthwork

Earthwork at the site is anticipated to consist of site clearing, removal and recompaction of the near-surface soil, trenching and backfilling for new utilities, and grading for the new hardscape areas. Earthwork should be performed in accordance with the requirements of the applicable governing agencies and the recommendations presented in the following sections.

12.1.1 Construction Plan Review and Pre-Construction Conference

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

12.1.2 Site Clearing and Preparation

Prior to excavation and fill placement, the site should be cleared of existing site improvements, 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 finished grade, if any, should be removed and the resulting holes filled with compacted soil. Materials generated from the clearing operations should be removed from the project site and disposed of at a legal dump site.

12.1.3 Excavation Characteristics

Based on our field exploration, we anticipate that excavations at the site may be accomplished with conventional earthmoving equipment in good working condition. We anticipate that existing alluvial deposits encountered during construction will be generally comprised of sandy silt, lean clay, clayey sand, silty sand, and poorly graded sand with clay. In the event that oversize materials (larger than 4 inches in longest diameter), including cobbles, are encountered during excavation operations, the oversized materials will need to 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.

12.1.4 Treatment of Near-Surface Soils

We recommend that the existing alluvial soils be excavated and recompacted in order to provide relatively uniform foundation bearing support to the new structures. The underlying soils should be excavated to a depth of approximately 24 inches below the planned bottom elevation of the structural footings. The limits of excavation should extend laterally so as to provide a 1 to 1 prism of compacted fill extending down and out from the outside edges of the footings. The actual depths and limits of excavation should be evaluated by our representative based on the materials exposed at the time of construction.

The subgrade at the bottom of the excavation should be scarified to a depth of 8 inches, moisture-conditioned generally to slightly above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by the ASTM International (ASTM) test method D 1557. The excavated areas should be backfilled to the finished grade with on-site soils compacted to a relative compaction of 90 percent. The exposed subgrade should be evaluated by our representative during the excavation work.

In order to provide suitable support and reduce the potential settlements of new and reconstructed hardscape (i.e., sidewalks, curbs and gutter, ribbon gutters, etc.), we recommend that the subgrade materials beneath the proposed hardscape areas be excavated to a depth of approximately 12 inches, moisture-conditioned to near the optimum moisture content, and recompacted. The limits of the excavation should extend laterally 12 inches beyond the outside edges of hardscape. The exposed subgrade should be evaluated by our representative during the excavation work. Loose, soft, and/or wet areas may need to be further excavated, depending on our observations during construction. Prior to placing new compacted fill in areas that are excavated and/or in areas where the existing subgrade will be raised with new fill, the exposed bottom should be scarified, moisture-conditioned, and recompacted to a depth of approximately 8 inches.

12.1.5 Fill Material

In general, the on-site soils should be suitable for use as compacted fill provided that they are free of construction/demolition debris, trash, roots, vegetation, deleterious materials, and contamination. 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. On-site soils used as fill will involve moisture-conditioning and/or aeration to achieve appropriate moisture content for compaction. The clayey on-site soils are not suitable for use as retaining wall backfill.

Import material should consist of clean, non-expansive, granular material, which conforms to the "Greenbook" for structure backfill. Soil should also be tested for corrosive properties prior to importing. We recommend that the imported materials conform with the Caltrans (2021) criteria for non-corrosive soils (i.e., soils having a chloride concentration of 500 parts per million [ppm] or less, a soluble sulfate content of approximately 0.15 percent [1,500 ppm] or less, a pH value of 5.5 or more and an electrical resistivity of 1,500 ohm-centimeter [ohm-cm] or more). Import material should be submitted to Ninyo & Moore for review prior to importing to the site. The contractor should be responsible for the uniformity of import material brought to the site.

12.1.6 Fill Placement and Compaction

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

12.1.7 Temporary Excavations

We recommend that trenches and excavations be designed and constructed in accordance with the OSHA regulations. These regulations provide trench sloping and shoring design parameters for trenches up to 20 feet deep based on the soil types encountered. Trenches/excavations over 20 feet deep should be designed by the contractor's engineer based on site-specific geotechnical analyses. For planning purposes, we recommend that the on-site soils be considered as OSHA soil Type C.

Temporary excavations should be constructed in accordance with the OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met by using appropriate shoring (including trench boxes) or by laying back the slopes no steeper than 1.5:1 (horizontal to vertical). Temporary excavations that encounter seepage may need shoring or may be mitigated by placing sandbags or gravel along the base of the seepage zone. Excavations encountering seepage should be evaluated on a case-by-case basis. On-site safety of personnel is the responsibility of the contractor. Recommendations for temporary shoring can be provided, if requested.

12.1.8 Excavation Bottom Stability

In general, we anticipate that the bottoms of excavations at depths of approximately 4 feet or less should be relatively stable and suitable for placement of backfill. However, stabilization may be needed if soft and/or loose alluvium is encountered, which may be unstable and subject to pumping under heavy equipment loads. Stabilization may involve excavation and replacement with compacted AB material or crushed rock to thicknesses of approximately 1 to 3 feet. If crushed rock is used, it should be wrapped in a suitable geotextile filter fabric to minimize infiltration of fine-grained soils and collapse of overlying fill material. Recommendations for stabilizing excavation bottoms should be based on evaluation in the field by the geotechnical consultant at the time of construction.

12.2 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of the governing jurisdictions and applicable building codes. Table 2 presents the site-specific spectral response acceleration parameters in accordance with the 2022 CBC guidelines.

Table 2 – 2022 California Building Code Seismic Design Criteria

Site-specific Spectral Response Acceleration Parameters	Values
Site Class	D
Site Coefficient, Fa	1.0
Site Coefficient, Fv	1.7
Mapped MCE_R Spectral Response Acceleration at 0.2-second Period, S_s	1.863 g
Mapped MCE_R Spectral Response Acceleration at 1.0-second Period, S_1	0.656 g
Site-Specific Spectral Response Acceleration at 0.2-second Period, S_{MS}	2.014 g
Site-Specific Spectral Response Acceleration at 1.0-second Period, S_{M1}	1.529 g
Site-Specific Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.343 g
Site-Specific Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	1.020 g
Maximum Considered Earthquake Geometric Mean (MCEG) Peak Ground Acceleration, PGA_M	0.898g

12.3 Foundations

The proposed building structures may be supported on shallow foundations including spread footings bearing on compacted fill in accordance with the recommendations presented in the Earthwork section of this report. Foundations should be designed in accordance with the 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.

12.3.1 Spread Footings

Spread footings for the building structures should extend 24 inches or more below the adjacent finished grade. Continuous footings should have a width of 24 inches or more.

Isolated pad footings are anticipated to have a width of 36 inches or more. In addition, the footings constructed near the 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. Spread footings should be reinforced and detailed in accordance with the recommendations of the structural engineer.

Footings, as described above, may be designed using a net allowable bearing capacity of 2,500 pounds per square foot (psf). The net allowable bearing capacity may be increased by 250 and 500 psf for each additional foot of width and depth, respectively, up to a value of 3,500 psf. These allowable bearing capacities 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 these recommendations are estimated to be less than approximately 1 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 0.35, where the total frictional resistance equals the coefficient of friction times the dead load. Footings may be designed using a passive resistance of 350 psf per foot of depth for level ground condition up to a value of 3,500 psf. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance, provided the passive resistance does not exceed one-half of the total allowable resistance. The passive resistance may be increased by one-third when considering loads of short duration such as wind or seismic forces.

12.4 Underground Utilities

We anticipate that utility pipelines will be supported on alluvial deposits. The depths of the pipelines are not known; however, we anticipate that the pipe invert depths will not exceed 10 feet. Trenches should not be excavated parallel to building footings. If needed, trenches can be excavated adjacent to a continuous footing, provided that the bottom of the trench is located above a 1:1 (horizontal to vertical) plane projected downward from the outside edge of the adjacent footing at a point 6 inches above the bottom of the footing. Utility lines that cross beneath footings should be encased in concrete below the footing.

12.4.1 Pipe Bedding

We recommend that pipelines be supported on 4 inches or more of granular bedding material. Bedding material should be placed around pipe zones to 1 foot or more above the top of the pipe. The bedding material should be classified as sand, be free of organic material, and have a sand equivalent (SE) of 30 or more. We do not recommend gravel be used for bedding material because of the nature of the subsurface 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 uniformly up both sides of the pipe. Trench backfill, including bedding material, should be placed in accordance with the recommendations presented in the preceding section.

12.4.2 Trench Backfill

Based on our subsurface evaluation, the on-site 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.

12.4.3 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 (psi) be used for design, provided that granular bedding material is placed adjacent to the pipe, as recommended in this report.

12.5 Preliminary Pavement Design

We understand that new pavement will be constructed for the new parking lot. New AC pavement sections were designed based on the subgrade soil conditions encountered. A design R-value of 13 was used based on our laboratory test results. Our flexible and rigid pavement analyses were performed using the methodology outlined in the Caltrans Highway Design Manual (Caltrans, 2023) and the Navy Pavement Design Manual (Naval Facilities Engineering Command [NAVFAC], 1979), respectively. The analysis assumes a 20-year design life for new pavements and a traffic index (TI) of 5, 6, and 7. Based on the design R-value and TIs, our preliminary pavement structural sections are provided below in Table 3.

Table 3 – Preliminary Structural Pavement Sections

Traffic Index	AC over CAB or AC over CMB (inches)	Full Depth PCC over CAB or CMB (inches)
≤5.0	3 over 8½	5½ over 4
6.0	3 over 12	6½ over 4
7.0	3½ over 14½	8½ over 4

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.

Prior to placement of the new structural pavement section, the upper approximately 8 inches of the subgrade beneath new pavements should be scarified, moisture-conditioned, and re-compacted to a relative compaction of 95 percent as evaluated by ASTM D1557. Base material should be placed at a relative compaction of 95 percent or more as evaluated by ASTM D 1557. Aggregate base material should conform to the latest specifications in Section 200-2.2 for CAB or Section 200-2.4 for Crushed Miscellaneous Base (CMB) of the Greenbook. AC should conform to the latest specifications in Section 203-1 of the Greenbook and should be compacted to a relative compaction of 95 percent per ASTM D1557 methods.

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 performed by an engineering company specialized in this type of work.

12.6 Hardscape

We recommend that new exterior concrete sidewalks and flatwork (hardscape) have a minimum thickness of 4 inches and be appropriately reinforced per the recommendation of the structural engineer. The hardscape should be underlain by 4 inches of granular material such as CAB or CMB and installed with crack-control joints at an appropriate spacing as designed by the structural engineer to reduce the potential for shrinkage cracking. Positive drainage should be established and maintained adjacent to flatwork. To reduce the potential for differential offset, 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.

12.7 Corrosivity

Laboratory testing was performed on a representative sample of near-surface soil collected from boring B-1 to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. The chloride content test was performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417. The laboratory test results are presented in Appendix B.

The soil pH was measured at approximately 7.7 and the electrical resistivity was measured to be approximately 3,332 ohm-cm. The chloride content of the sample was measured to be approximately 50 ppm. The sulfate content of the tested sample was approximately 0.011 percent (i.e., 110 ppm). Based on the laboratory test results and Caltrans (2021) corrosion criteria, the project site may 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, and an electrical resistivity of 1,500 ohm-cm or more.

12.8 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. The sample tested during this evaluation indicated a water-soluble sulfate content of approximately 0.011 percent by weight (i.e., 110 ppm). Based on the American Concrete Institute (ACI) 318-14 criteria 318-14 (ACI, 2016), the potential for sulfate attack is considered negligible for water-soluble sulfate contents in soils of less than 0.1 percent by weight (1,000 ppm), moderate for water-soluble sulfate contents in soils between 0.1 and 0.2 percent by weight (1,000 to 2,000 ppm), and severe for water-soluble sulfate contents in soils between 0.2 and 2 percent by weight (2,000 to 20,000 ppm). Accordingly, the on-site soils are considered to have a negligible potential for sulfate attack. Per the ACI 318-14 criteria (ACI, 2016), Type II cement is considered to be appropriate for the project. Due to the potential variability in soil conditions across the site, Type II/V cement with a water/cement ratio of 0.45 or less may be considered 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 also recommend that crack control joints be provided in sidewalks and exterior hardscape in accordance with the recommendations of the structural engineer to reduce the potential for distress due to minor soil movement and concrete shrinkage. The structural engineer should be consulted for additional concrete specifications.

12.9 Drainage

Good surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to channel surface water off the pavement, away from foundations and off-site. Positive drainage is defined as a slope of 2 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 and discharged to suitable facilities. Surface waters should not be allowed to pond adjacent to footings or on pavements. 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.

13 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. In addition, we should review the plans and specifications 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 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. If another geotechnical consultant is selected, we request that the selected consultant indicate to the school administrator and our firm in writing that our recommendations are understood and that they are in full agreement with our recommendations.

14 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. A hazardous materials assessment conducted for the subject properties by Ninyo & Moore is presented under separate cover.

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

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

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

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

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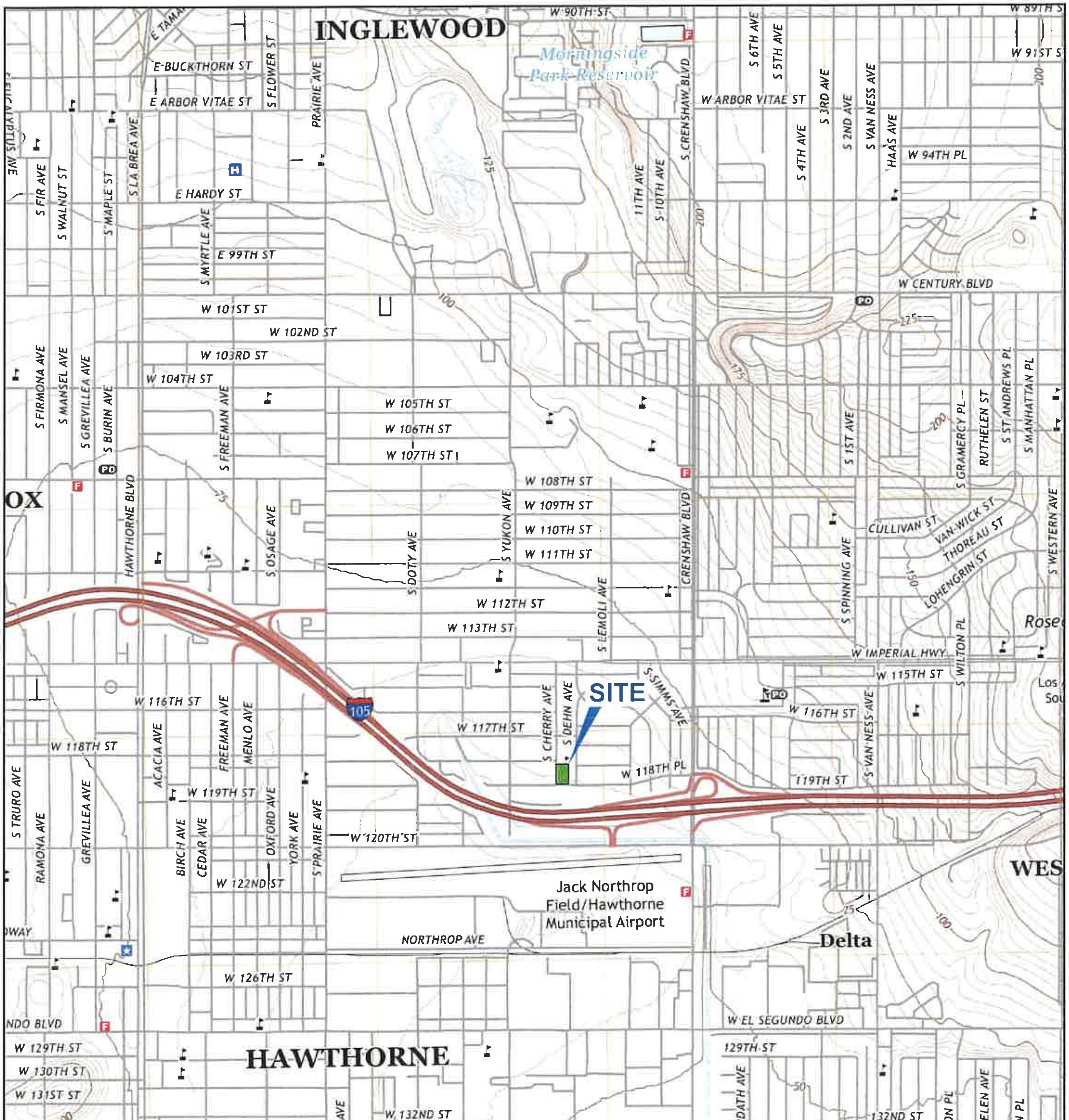
FIGURES



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: USGS, 2018.

N

FEET

0 2,000 4,000

FIGURE 1

SITE LOCATION

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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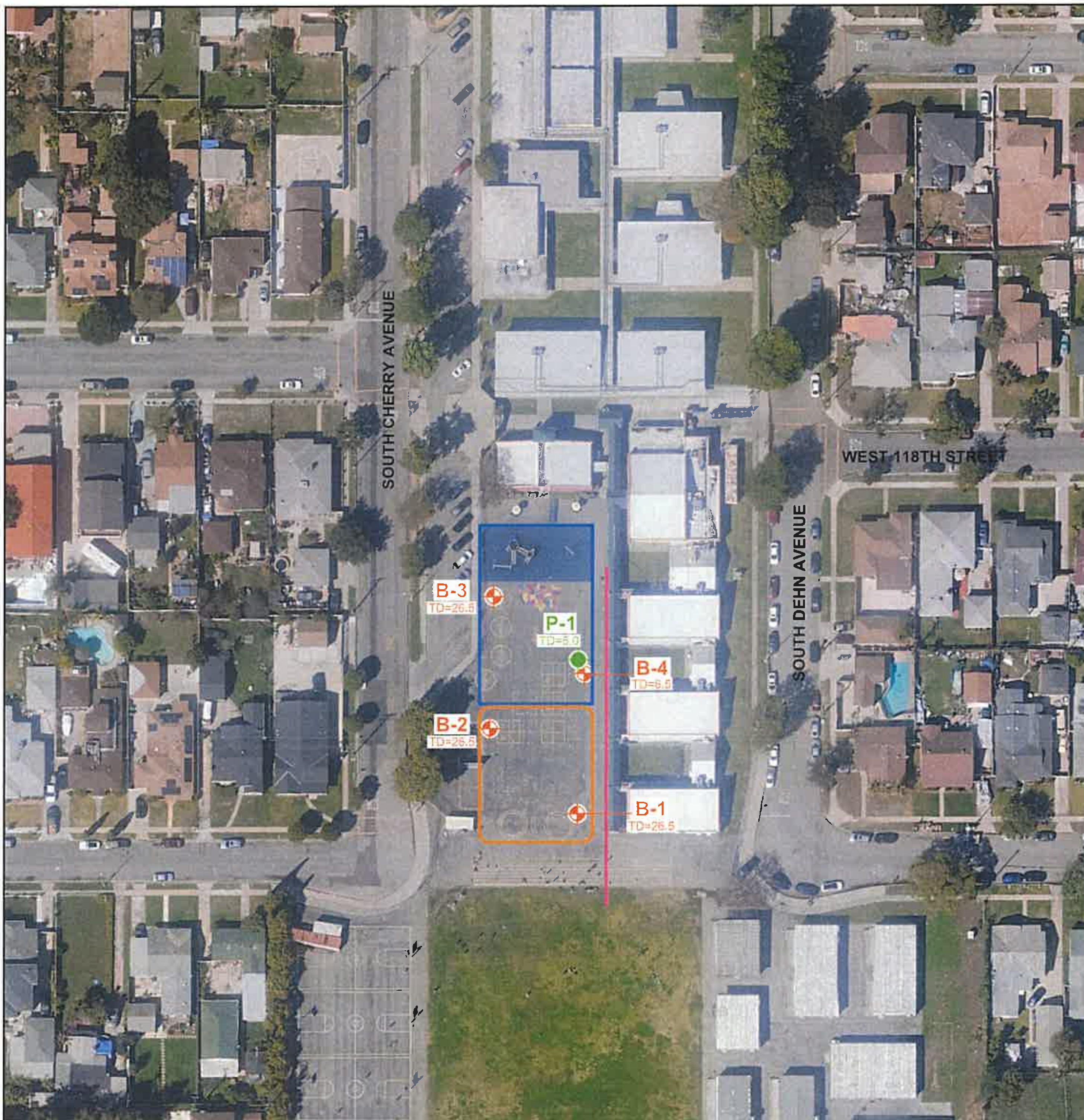
Geotechnical & Environmental Sciences Consultants

209822017 | 6/24



LEGEND

B-4 TD=6.5	BORING; TD=TOTAL DEPTH IN FEET	orange line	NEW TWO-STORY BUILDING
		blue line	NEW ONE-STORY BUILDING
P-1 TD=5.0	PERCOLATION TEST; TD=TOTAL DEPTH IN FEET	pink line	REMI SURVEY LINE

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: GOOGLE EARTH, 2019.

N

FEET
0 100 200

FIGURE 2

SITE AERIAL

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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LEGEND

B-4 TD=6.5	BORING; TD=TOTAL DEPTH IN FEET		NEW TWO-STORY BUILDING
			NEW ONE-STORY BUILDING
P-1 TD=5.0	PERCOLATION TEST; TD=TOTAL DEPTH IN FEET		REMI SURVEY LINE
		A A'	CROSS SECTION

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: HED DESIGN, 2019

FIGURE 3

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SITE PLAN AND BORING LOCATIONS

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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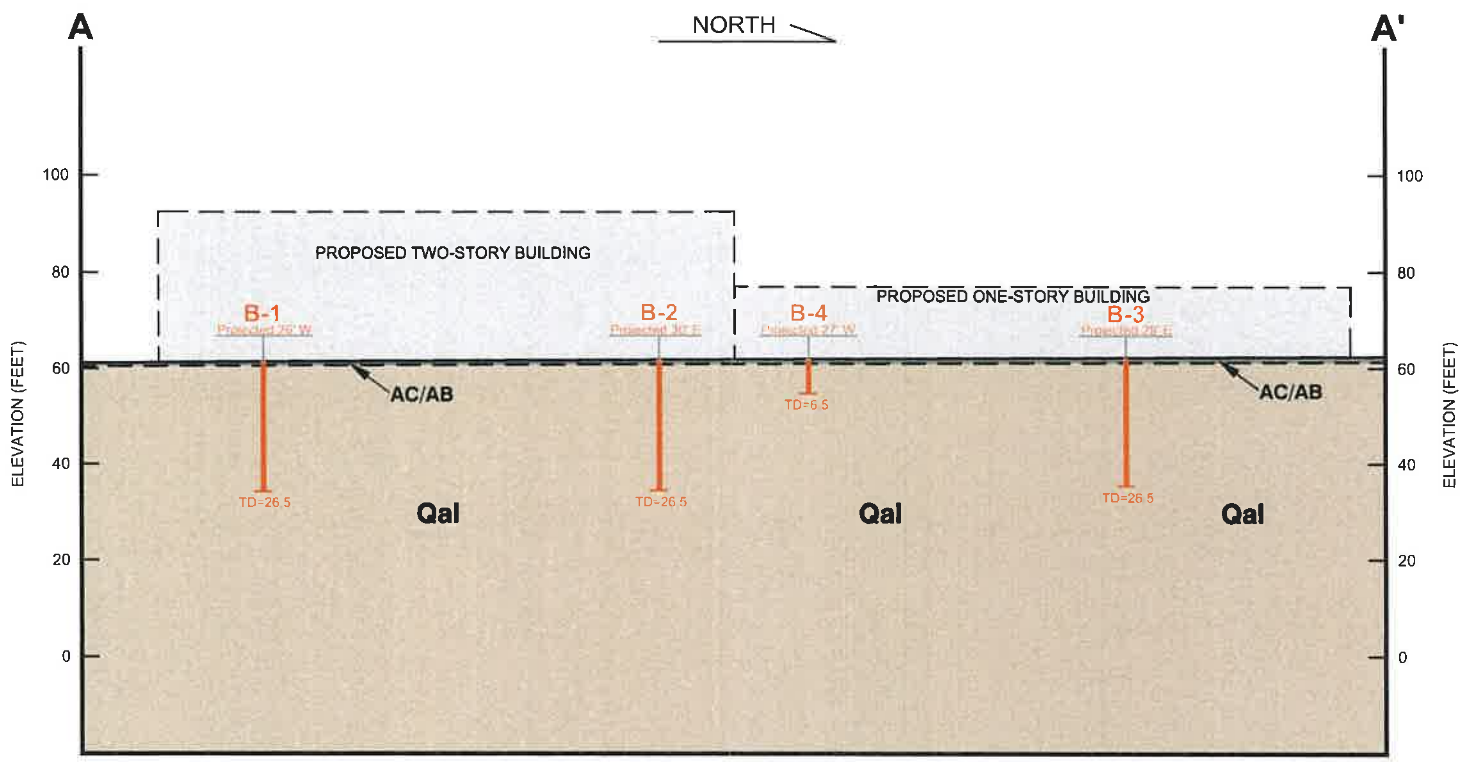

LEGEND

BORING;
TD=TOTAL DEPTH IN FEET

GEOLOGIC CONTACT

GROUND SURFACE

AC/AB EXISTING ASPHALT CONCRETE AND AGGREGATE BASE

Qal ALLUVIUM

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 4

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CROSS SECTION A-A'

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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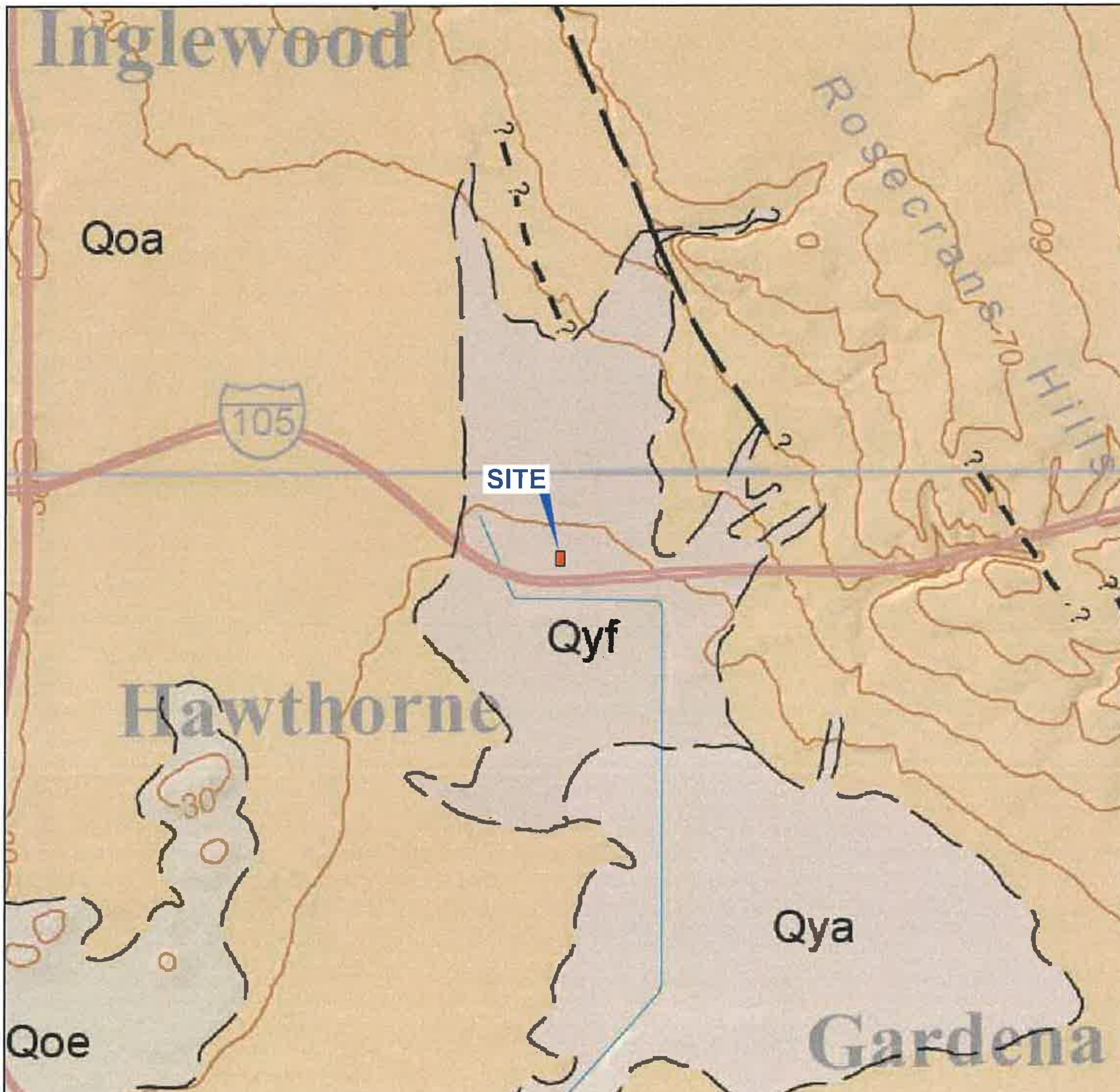

LEGEND

Qyf	YOUNG ALLUVIAL FAN DEPOSITS, UNDIVIDED	— —••••	FAULT
Qoa	OLDER ALLUVIUM, UNDIVIDED	⁓	GEOLOGIC CONTACT

N

FEET

0 3,000 6,000

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

FIGURE 5

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

REGIONAL GEOLOGY

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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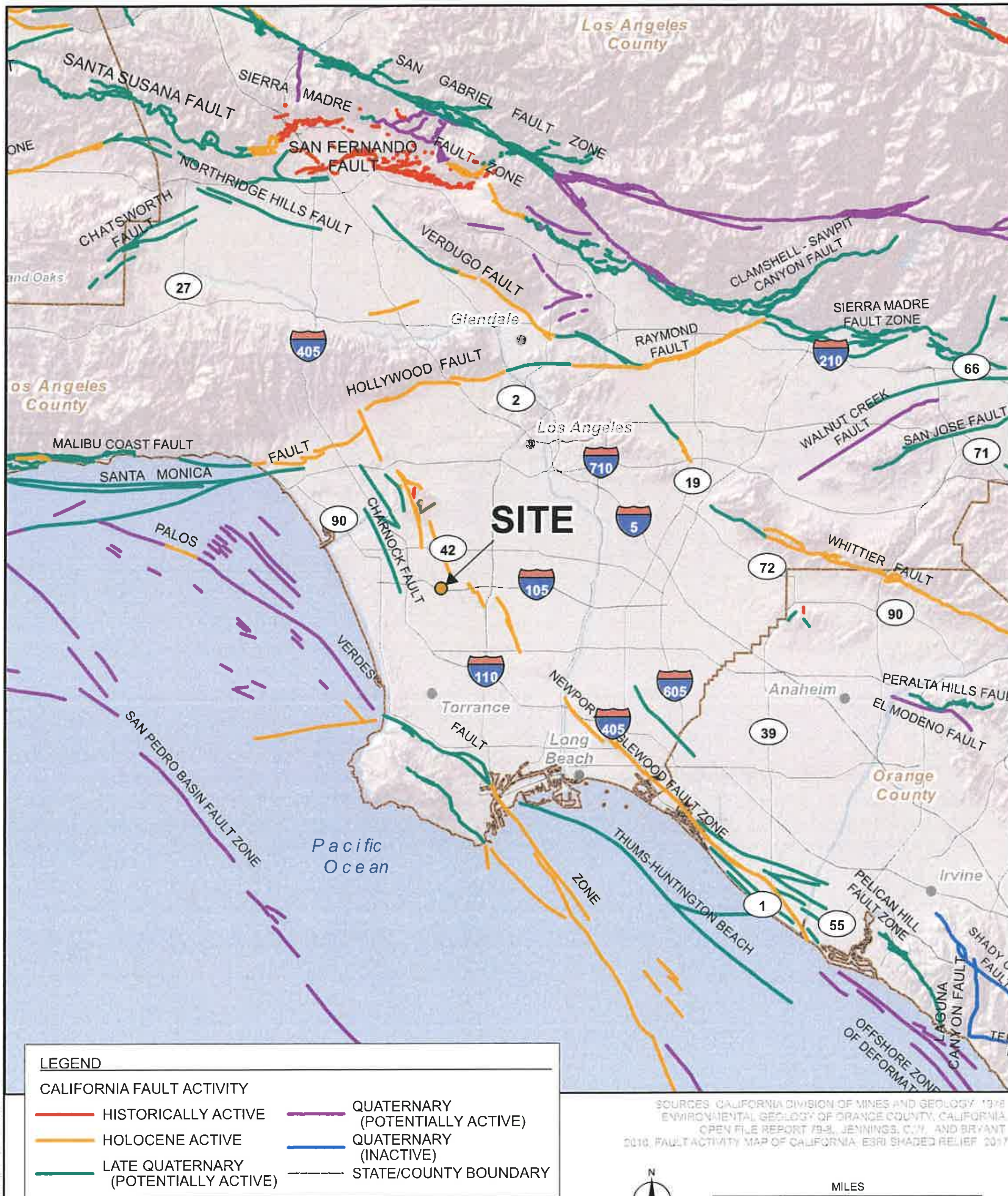


NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

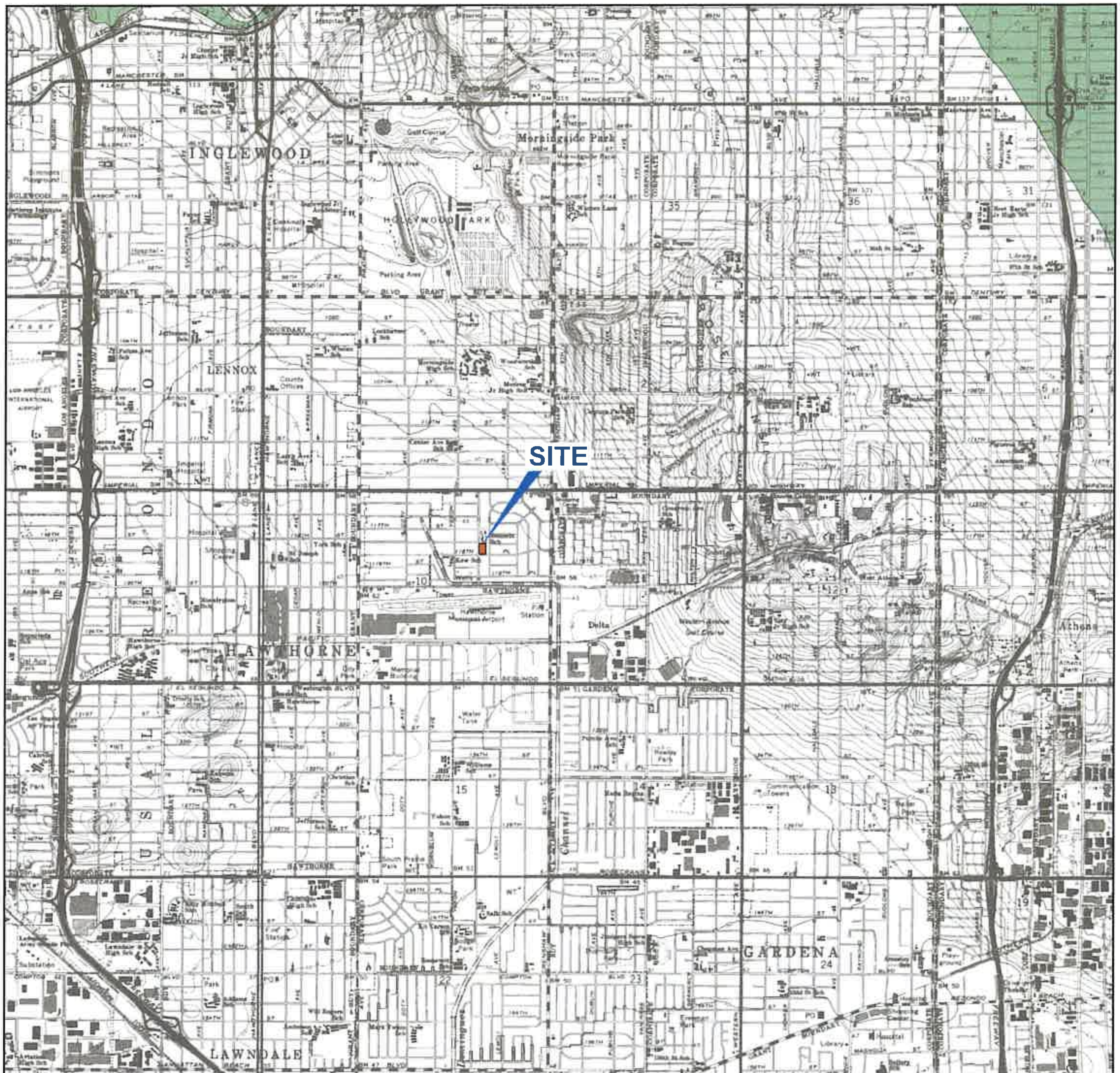
FIGURE 6

FAULT LOCATIONS

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

209822017 | 6/24



LEGEND

LIQUEFACTION

Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE. | REFERENCE: CDMG, 1999.

N

FEET

0 4,000 8,000

FIGURE 7

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SEISMIC HAZARD ZONES

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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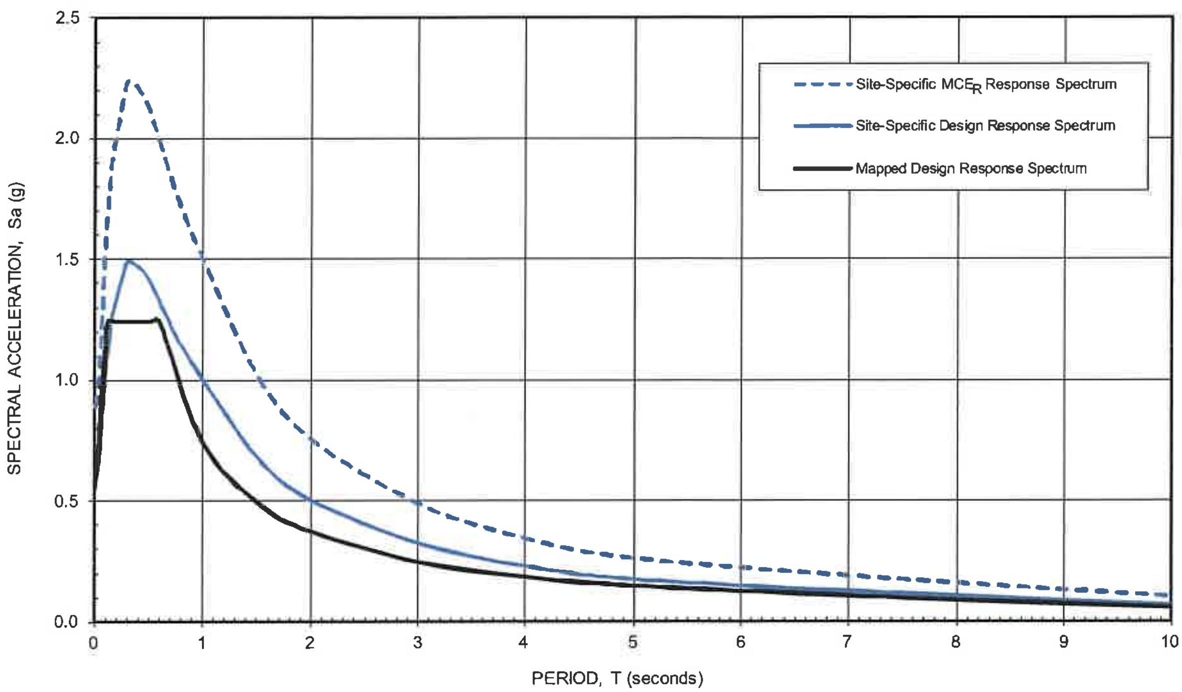
PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.010	0.892	0.595
0.020	0.901	0.600
0.030	0.939	0.626
0.050	1.087	0.725
0.075	1.352	0.901
0.100	1.588	1.059
0.150	1.880	1.253
0.200	2.029	1.353
0.250	2.148	1.432
0.300	2.238	1.492
0.400	2.202	1.468

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM Sa (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM Sa (g)
0.500	2.118	1.412
0.750	1.776	1.184
1.000	1.501	1.001
1.500	1.020	0.680
2.000	0.754	0.503
3.000	0.488	0.325
4.000	0.344	0.229
5.000	0.262	0.175
7.500	0.175	0.117
10.000	0.105	0.070

S_{MS} = 2.014 g	S_{M1} = 1.529 g	S_{DS} = 1.343 g	S_{D1} = 1.020 g	PGA_M = 0.898 g



NOTES:

1 The probabilistic ground motion spectral response accelerations are based on the risk-targeted Maximum Considered Earthquake (MCE_R) having a 2% probability of exceedance in 50 years in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships and the risk coefficients per ASCE 7-16 Section 21.2.1.1.

2 The deterministic ground motion spectral response accelerations are the 84th percentile geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.5 event on the Compton fault zone located 9.1 kilometers from the site and a Mw 7.2 event on the Newport Inglewood fault zone located 1.5 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

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

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

FIGURE 8

ACCELERATION RESPONSE SPECTRA

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

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

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

The Standard Penetration Test (SPT) Sampler
Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1-3/8 inches. The sampler was driven into the ground 18 inches with a 140-pound hammer falling freely from a height of 30 inches in general accordance with ASTM D 1586. The blow counts were recorded for every 6 inches of penetration; the blow counts reported on the logs are those for the last 12 inches of penetration. Soil samples were observed and removed from the sampler, bagged, sealed and transported to the laboratory for testing.

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

The Modified Split-Barrel Drive Sampler
The sampler, with an external diameter of 3 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.

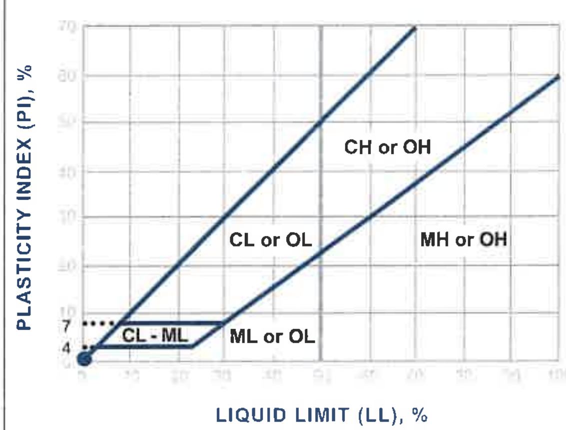
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Group Symbol	Group Name
			Secondary Divisions	
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with clay
		GRAVEL with FINES more than 12% fines	GM	silty GRAVEL
			GC	clayey GRAVEL
			GC-GM	silty, clayey GRAVEL
	SAND 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SW	well-graded SAND
			SP	poorly graded SAND
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SW-SM	well-graded SAND with silt
			SP-SM	poorly graded SAND with silt
			SW-SC	well-graded SAND with clay
			SP-SC	poorly graded SAND with clay
		SAND with FINES more than 12% fines	SM	silty SAND
			SC	clayey SAND
			SC-SM	silty, clayey SAND
FINE-GRAINED SOILS 50% or more passes No. 200 sieve	**SILT and CLAY** liquid limit less than 50%	INORGANIC	CL	lean CLAY
			ML	SILT
			CL-ML	silty CLAY
		ORGANIC	OL (PI > 4)	organic CLAY
			OL (PI < 4)	organic SILT
	SILT and CLAY liquid limit 50% or more	INORGANIC	CH	fat CLAY
			MH	elastic SILT
		ORGANIC	OH (plots on or above "A"-line)	organic CLAY
			OH (plots below "A"-line)	organic SILT
	Highly Organic Soils		PT	Peat

Grain Size

Description		Sieve Size	Grain Size	Approximate Size
Boulders		> 12"	> 12"	Larger than basketball-sized
Cobbles		3 - 12"	3 - 12"	Fist-sized to basketball-sized
Gravel	Coarse	3/4 - 3"	3/4 - 3"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	0.19 - 0.75"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	0.079 - 0.19"	Rock-salt-sized to pea-sized
	Medium	#40 - #10	0.017 - 0.079"	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	0.0029 - 0.017"	Flour-sized to sugar-sized
Fines		Passing #200	< 0.0029"	Flour-sized and smaller

Plasticity Chart



Apparent Density - Coarse-Grained Soil

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

Consistency - Fine-Grained Soil

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

BORING LOG EXPLANATION SHEET

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	
0								Bulk sample.
								Modified split-barrel drive sampler.
								No recovery with modified split-barrel drive sampler.
								Sample retained by others.
								Standard Penetration Test (SPT).
5								No recovery with a SPT.
			XX/XX					Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.
								No recovery with Shelby tube sampler.
								Continuous Push Sample.
								Seepage.
10								Groundwater encountered during drilling.
								Groundwater measured after drilling.
							SM	<u>MAJOR MATERIAL TYPE (SOIL)</u>: Solid line denotes unit change.
							CL	Dashed line denotes material change. Attitudes: Strike/Dip b: Bedding c: Contact j: Joint f: Fracture F: Fault cs: Clay Seam s: Shear bss: Basal Slide Surface sf: Shear Fracture sz: Shear Zone sbs: Shear Bedding Surface
15								
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

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BORING LOG

DATE DRILLED 5/4/19 BORING NO. B-1

GROUND ELEVATION 62' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (ABC Liovin)

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

SAMPLED BY ECH LOGGED BY ECH REVIEWED BY RDH/SG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ASPHALT CONCRETE: Approximately 2 inches thick.
							CL	AGGREGATE BASE: Brown, moist, medium dense, silty SAND with gravel; approximately 2 inches thick.
								ALLUVIUM: Brown, moist, hard, sandy lean CLAY.
			36	19.1	109.1			Trace gravel; trace iron oxide staining; trace pinhole-sized voids.
							SP-SC	Brown, moist, dense, poorly graded SAND with clay; trace gravel.
10			28					
							CL	Brown, moist, hard, sandy lean CLAY.
			41	15.3	115.1			Pinhole-sized voids.
							SM	Brown, moist, medium dense, silty SAND.
20			17					
							ML	Brown, moist, medium dense, sandy SILT; trace iron oxide staining; micaceous.
			36					
								Total Depth = 26.5 feet. Groundwater not encountered during drilling. Backfilled with cement-bentonite grout and capped with concrete on 5/4/19.
30								
40								

Total Depth = 26.5 feet.
Groundwater not encountered during drilling.
Backfilled with cement-bentonite grout and capped with concrete on 5/4/19.

Notes:
Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.

The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.

DATE DRILLED 5/4/19 BORING NO. B-2

GROUND ELEVATION 62' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (ABC Liovin)

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

SAMPLED BY ECH LOGGED BY ECH REVIEWED BY RDH/SG

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	ASPHALT CONCRETE: Approximately 2 inches thick.
						SM	AGGREGATE BASE: Brown, moist, medium dense, silty SAND with gravel; approximately 1 inch thick. ALLUVIUM: Brown, moist, dense, silty SAND; trace gravel.
		33				CL	Brown, moist, hard, lean CLAY; trace gravel.
						SM	Brown, moist, medium dense, silty SAND; trace gravel; trace pinhole-sized voids.
10		30	8.3	111.8			
							Very dense.
		37				CL	Brown, moist, hard, lean CLAY; trace iron oxide staining.
						SM	Light brown, moist, medium dense, silty SAND; trace pinhole-sized voids.
20		34	11.1	107.1			
						ML	Light brown, moist, dense, sandy SILT.
		21					
							Total Depth = 26.5 feet. Groundwater not encountered during drilling. Backfilled with cement bentonite grout and capped with concrete on 5/4/19. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30							
40							

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED 5/4/19 BORING NO. B-3 GROUND ELEVATION 62' ± (MSL) SHEET 1 OF 1 METHOD OF DRILLING 8" Hollow-Stem Auger (ABC Liovin) DRIVE WEIGHT 140 lbs. (Auto. Trip Hammer) DROP 30" SAMPLED BY ECH LOGGED BY ECH REVIEWED BY RDH/SG DESCRIPTION/INTERPRETATION
0							SM SC	ASPHALT CONCRETE: Approximately 1.5 inches thick. AGGREGATE BASE: Brown, moist, medium dense, silty SAND with gravel; approximately 1 inch thick. ALLUVIUM: Dark brown, moist, dense, clayey SAND; trace gravel.
			46	24.8	107.6		CL	Brown, moist, hard, lean CLAY with sand; trace gravel.
10			17					Very stiff.
			56					Hard; trace iron oxide staining.
							SM	Brown, moist, medium dense, silty SAND.
20			13					
			34					Light brown; trace iron oxide staining; very fine sand.
								Total Depth = 26.5 feet. Groundwater not encountered during drilling. Backfilled with cement bentonite grout and capped with concrete on 5/4/19. Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
30								
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ASPHALT CONCRETE: Approximately 2.5 inches thick.
								AGGREGATE BASE: Brown, moist, medium dense, silty SAND with gravel; approximately 1 inch thick.
							CL	ALLUVIUM: Dark brown, moist, hard, sandy lean CLAY; trace gravel.
			35					Brown.
								Total Depth = 6.5 feet. Groundwater not encountered during drilling. Backfilled with soil cuttings and capped with concrete on 5/4/19.
10								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20								
30								
40								

DATE DRILLED 5/4/19 BORING NO. B-4

GROUND ELEVATION 62' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (ABC Liovin)

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

SAMPLED BY ECH LOGGED BY ECH REVIEWED BY RDH/SG

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ASPHALT CONCRETE: Approximately 2.5 inches thick.
								AGGREGATE BASE: Brown, moist, medium dense, silty SAND with gravel; approximately 1 inch thick.
							CL	ALLUVIUM: Dark brown, moist, hard, lean CLAY.
								Total Depth = 5 feet. Groundwater not encountered during drilling. Backfilled with gravel and capped with concrete on 5/4/19.
								Notes: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
								The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
20								
30								
40								

DATE DRILLED 5/4/19 BORING NO. P-1

GROUND ELEVATION 62' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 8" Hollow-Stem Auger (ABC Liovin)

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

SAMPLED BY ECH LOGGED BY ECH REVIEWED BY RDH/SG

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 in Appendix A.

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

200 Wash
An evaluation of the percentage of particles finer than the No. 200 sieve in selected soil samples was performed in general accordance with ASTM D 1140. The results of the tests are presented on Figure B-1.

Gradation Analysis
Gradation analysis test was performed on a selected representative soil sample in general accordance with ASTM D 422. The grain-size distribution curve is shown on Figure B-2. The test result was utilized in evaluating the soil classification in accordance with the USCS.

Atterberg Limits
Testing was 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 result was utilized to evaluate the soil classification in accordance with the USCS. The test results and classifications are shown on Figure B-3.

Proctor Density Test
The maximum dry density and optimum moisture content of a selected representative soil sample was evaluated using the Modified Proctor method in general accordance with ASTM or D 1557. The results of this test are summarized on Figure B-4.

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

Direct Shear Test
Direct shear test was performed on a remolded sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the potential fill material derived from near-surface site soils compacted to 90 percent relative compaction. The sample was inundated during shearing to represent adverse field condition. The test results are presented on Figure B-6.

Expansion Index Test
The expansion index of a selected material was evaluated in general accordance with ASTM D 4829. Specimens were 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 specimens were 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 result of these tests is 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. A Sample was 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 result is shown on Figure B-8.

Soil Corrosivity Tests

Soil pH, and resistivity tests were performed on a representative sample in general accordance with 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-9.

SAMPLE LOCATION	SAMPLE DEPTH (ft)	DESCRIPTION	PERCENT PASSING NO. 4	PERCENT PASSING NO. 200	USCS (TOTAL SAMPLE)
B-1	10.0-11.5	POORLY GRADED SAND WITH CLAY	94	12	SP-SC
B-2	10.0-11.5	SILTY SAND	100	38	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 1140

FIGURE B-1

NO. 200 SIEVE ANALYSIS TEST RESULTS

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INGLEWOOD, CALIFORNIA

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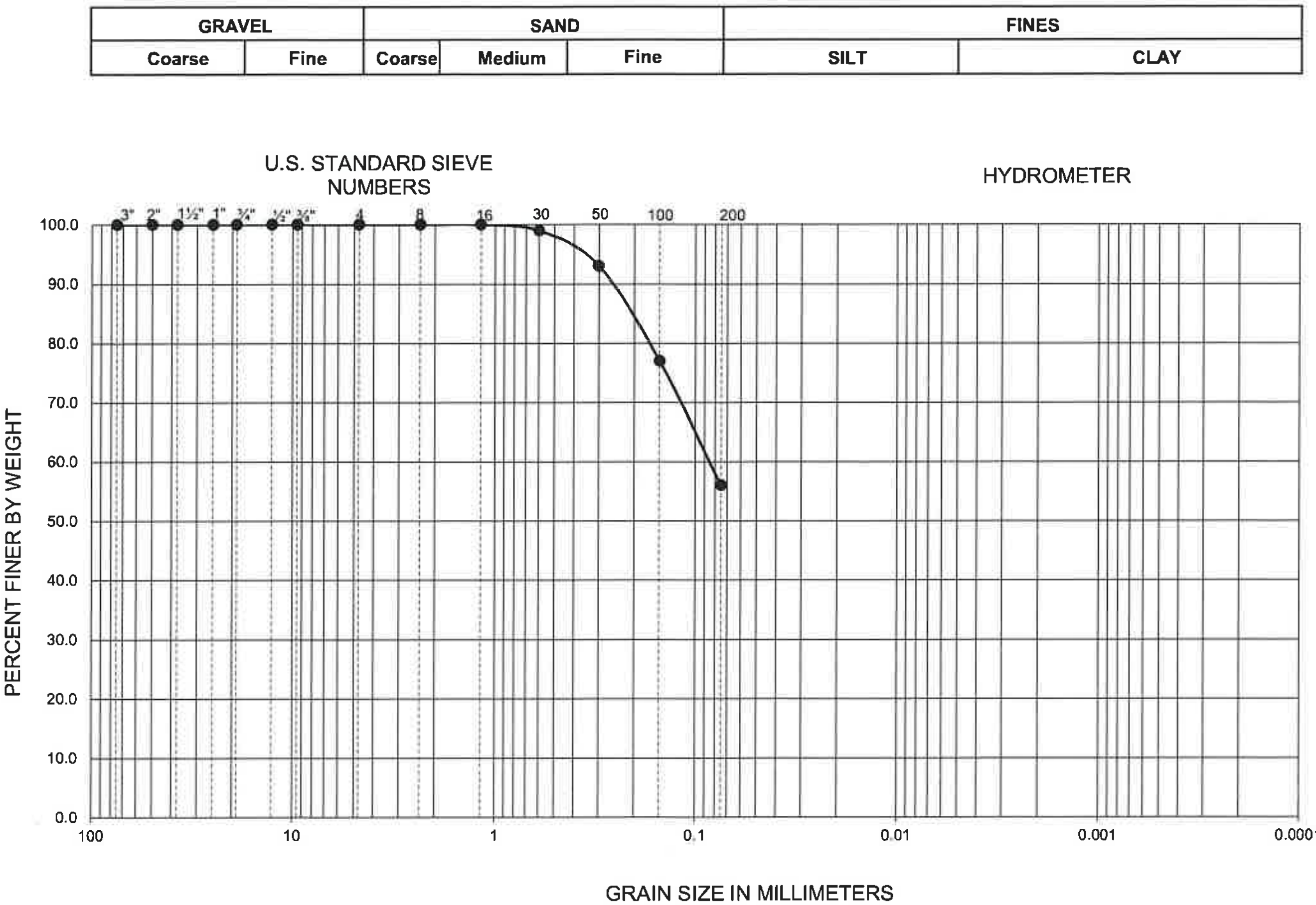
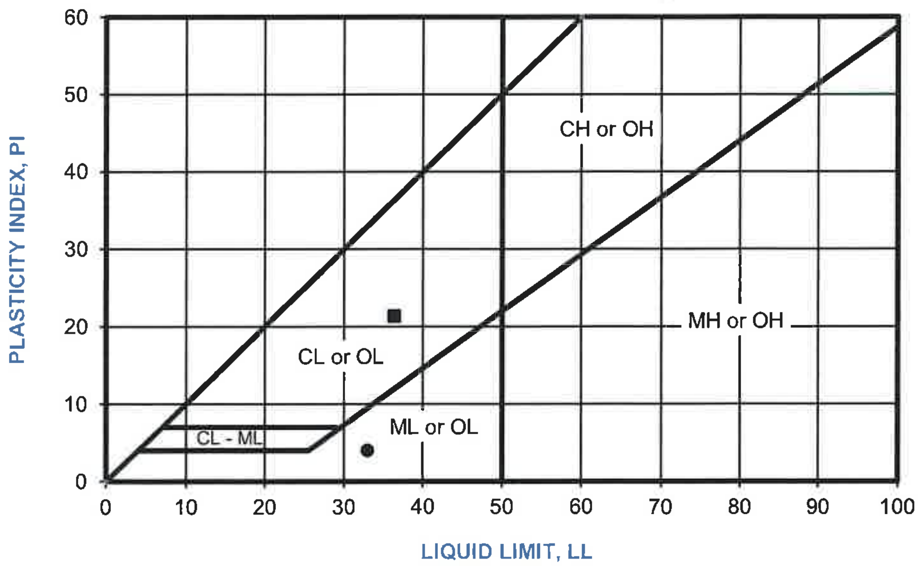


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D_{10}	D_{30}	D_{60}	C_u	C_c	Passing No. 200 (percent)	USCS
●	B-1	15.0-16.5	--	--	--	--	--	--	--	--	56	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

SYMBOL	LOCATION	DEPTH (ft)	LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS
●	B-2	25.0-26.5	33	29	4	ML	ML
■	B-3	5.0-6.5	36	15	21	CL	CL



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318

FIGURE B-3

ATTERBERG LIMITS TEST RESULTS

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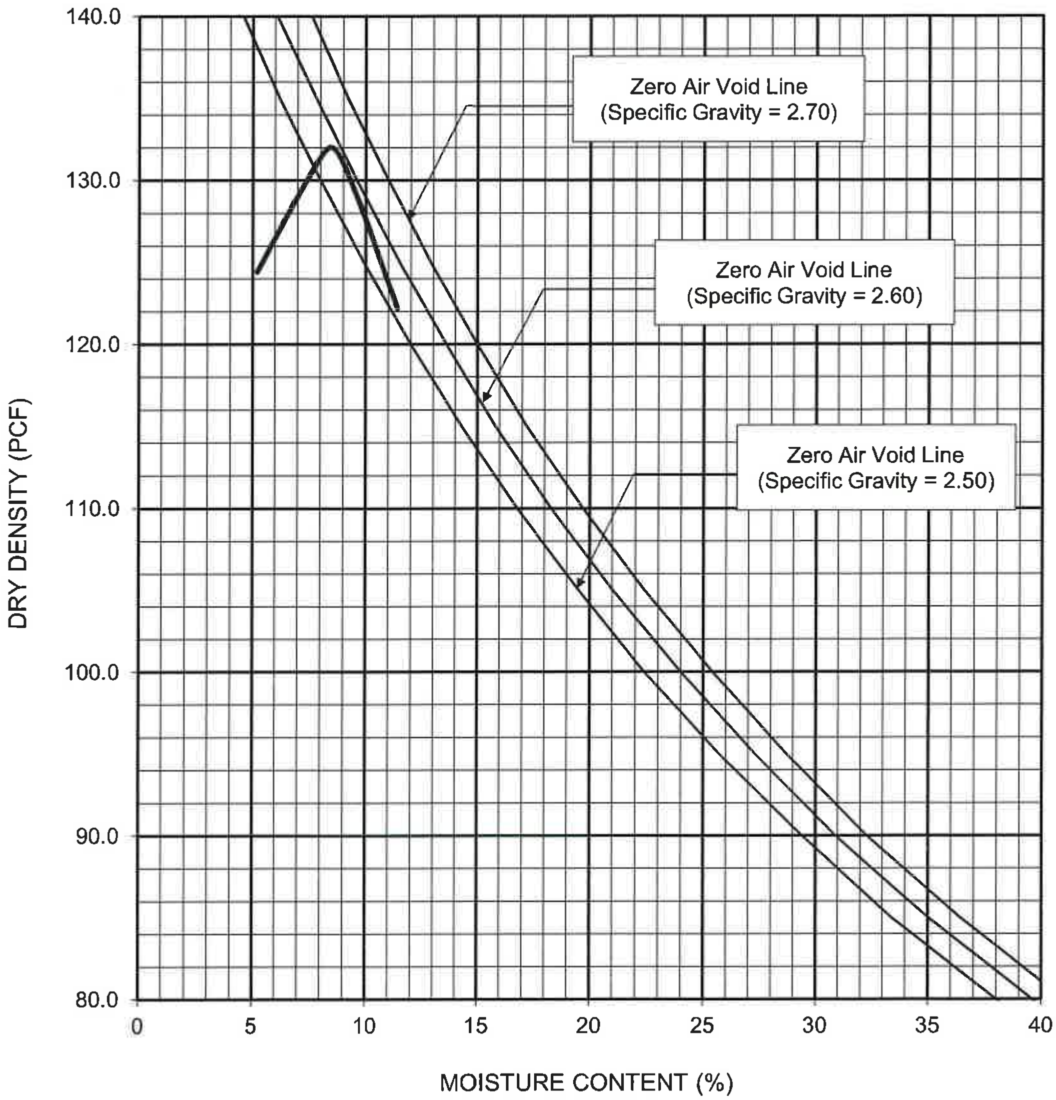

Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-3	0.0-5.0	BROWN CLAYEY SAND	132.0	8.5
Dry Density and Moisture Content Values Corrected for Oversize (ASTM D 4718)			N/A	N/A

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

FIGURE B-4

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PROCTOR DENSITY TEST RESULTS

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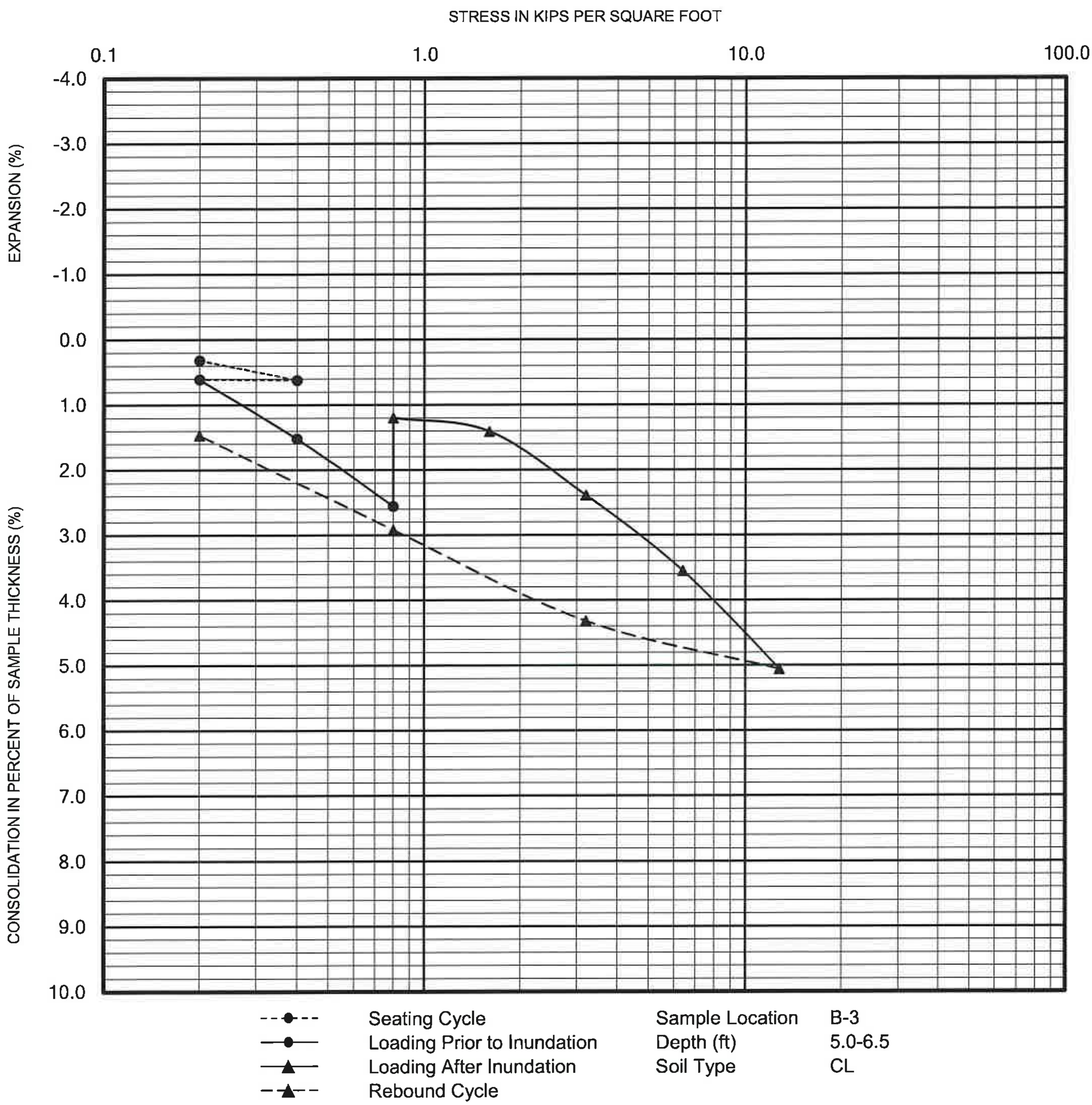

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

FIGURE B-5

CONSOLIDATION TEST RESULTS

BENNETT/KEW ELEMENTARY SCHOOL
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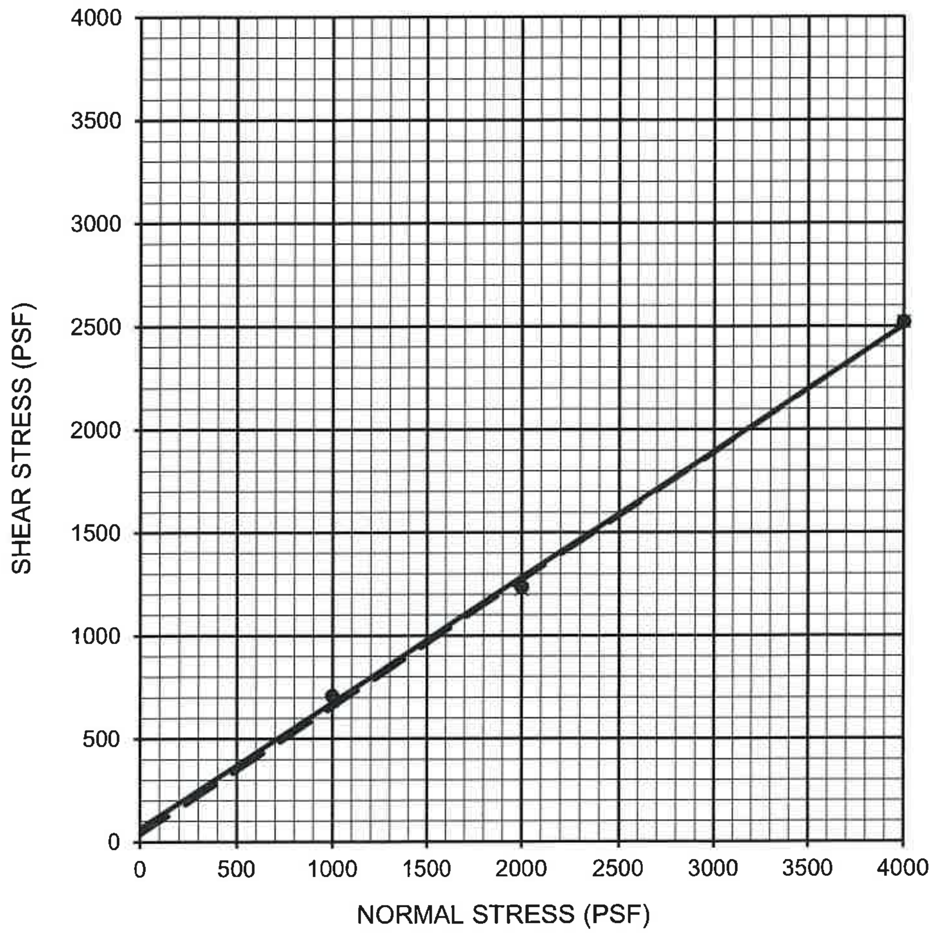

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
CLAYEY SAND	——●——	B-3	0.0-5.0	Peak	66	31	SC
CLAYEY SAND	– – X – –	B-3	0.0-5.0	Ultimate	36	32	SC

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

FIGURE B-6

DIRECT SHEAR TEST RESULTS

BENNETT/KEW ELEMENTARY SCHOOL
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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-3	0.0-5.0	8.8	115.7	14.2	0.003	3	Very Low

PERFORMED IN GENERAL ACCORDANCE WITH ☐ UBC STANDARD 18-2 ☑ ASTM D 4829

FIGURE B-7

EXPANSION INDEX TEST RESULTS

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BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
B-1	0.0-5.0	SANDY LEAN CLAY	13

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

R-VALUE TEST RESULTS

BENNETT/KEW ELEMENTARY SCHOOL
INGLEWOOD, CALIFORNIA

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	0.0-5.0	7.7	3,332	110	0.011	50

[1] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643
[2] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417
[3] PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-9

CORROSIVITY TEST RESULTS

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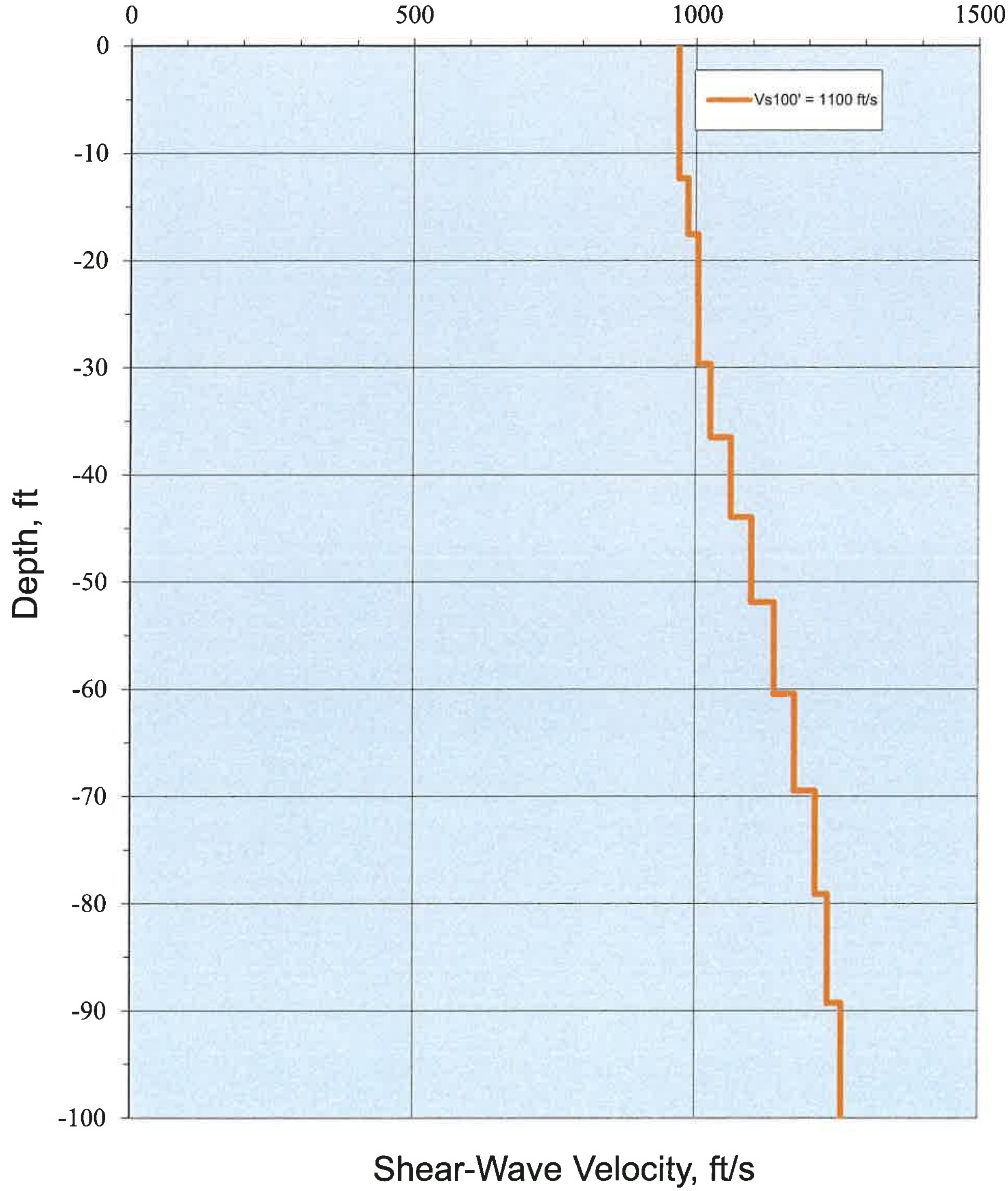
APPENDIX C

ReMi Survey Results

209822017 - Bennet Kew Elementary School
Vs Model, Line 1



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REMI SHEAR WAVE CALCULATIONS

209822017

Bennet Kew Elementary School
11710 S Cherry Ave, Inglewood, CA 90303

Line 1

d_i	v_{si}	d_i/v_{si}
3.6	969.1	0.00369
4.1	969.1	0.00425
4.7	969.7	0.00482
5.2	985.7	0.00530
5.8	1003.5	0.00575
6.3	1004.0	0.00629
6.9	1025.8	0.00670
7.4	1061.8	0.00699
8.0	1098.1	0.00726
8.5	1138.6	0.00748
9.1	1175.6	0.00771
9.6	1212.5	0.00793
10.2	1235.0	0.00823
10.7	1259.6	0.00851
	Average v_s	1100

$$\overline{v_s} = \frac{\sum d_i}{\sum \frac{d_i}{v_{si}}}$$

Per ASCE 7-22: Equation 20.4-1

d_i = thickness of any layer between 0 and 100 feet

v_{si} = shear wave velocity in feet per second

475 Goddard, Suite 200 | Irvine, California 92618 | p. 949.753.7070

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