Geotechnical Evaluation

Date Palm Drive Bridge and Roadway Widening

Cathedral City, California

DRAFT

KOA Corporation

3190 C Shelby Street | Ontario, California 91764

January 27, 2021 | Project No. 108980001

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

CELEBRATING 35 YEARS 1986 2021 NINYO & MOORE

Geotechnical Evaluation
Date Palm Drive Bridge and Roadway Widening
Cathedral City, California

DRAFT

Ms. Ming Guan, PE
KOA Corporation
3190 C Shelby Street | Ontario, California 91764

January 27, 2021 | Project No. 108980001

Draft
Christine M. Kuhns, PE
Project Engineer

Draft
Christina A. Tretinjak, PG, CEG
Senior Project Geologist

Draft
William R. Morrison, PE, GE
Senior Engineer

Draft
Kenneth H. Mansir Jr., PE, GE
Principal Engineer

CMK/CAT/WRM/KHM/gg

CONTENTS

TABLES

FIGURES

APPENDICES

1 INTRODUCTION

In accordance with your request and authorization, Ninyo & Moore has performed a geotechnical evaluation for the proposed Date Palm Drive bridge and roadway widening project located on Date Palm Drive between East Palm Canyon Drive (Highway 111) and Perez Road in Cathedral City, California (Figure 1). This report presents data from our background review, field exploration, and laboratory testing, provides conclusions regarding the geotechnical conditions at the project site, and provides recommendations regarding design and construction of the proposed improvements.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Reviewing readily available published and in-house geotechnical literature, a previous geotechnical evaluation report (Earth Mechanics, 2014; 2017), groundwater data, topographic maps, geologic maps, geologic hazard maps, fault maps, utility maps, project plans, and aerial photographs.
- Performance of a geotechnical site reconnaissance by a representative from Ninyo & Moore to observe and document the existing surface conditions at the project site. During our site reconnaissance we marked our boring locations for utility clearance by Underground Service Alert (USA).
- Obtaining an encroachment permit from the Cathedral City Engineering Department.
- Coordinating with a traffic control subcontractor to prepare traffic control plans and provide traffic control services during our subsurface evaluation.
- Performing a subsurface exploration consisting of the drilling, logging, and sampling of four small-diameter exploratory borings. The soil borings were excavated to depths ranging from approximately 5 feet to 51 feet below ground surface using hand augering equipment and a truck-mounted drill rig equipped with hollow stem augers. Logging of the borings was performed by a representative from Ninyo & Moore.
- Performing geotechnical laboratory testing on representative samples to evaluate pertinent soil parameters for design and classification purposes.
- Performing engineering analyses of the site geotechnical conditions based on data obtained from our background review, field exploration, and laboratory testing.
- Preparing this geotechnical evaluation report describing the findings and conclusions of our study and providing recommendations for design and construction of the proposed improvements.

3 SITE AND PROJECT DESCRIPTION

The project alignment is located along Date Palm Drive between East Palm Canyon Drive (Highway 111) and Perez Road in Cathedral City, California (Figure 2). This generally north-south trending alignment is approximately 1,300 feet in length and is adjacent to the existing asphalt concrete (AC) pavements of Date Palm Drive that extend through a commercial area of Cathedral City. The existing pavement currently consists of 3 inches of AC over 4 inches of aggregate base (Riverside County Flood Control, 1972). Elevations along the alignment generally vary between approximately 289 to 298 feet above mean sea level (MSL) near the northern and southern ends of the alignment, respectively. The lowest elevation along the alignment is approximately 286 above MSL near the existing Date Palm Drive bridge that extends over the North Cathedral Canyon Flood Control Channel. The channel bottom elevation in the vicinity of the bridge is approximately 275 feet above MSL.

Based on our discussions with you and review of the 50% plans (KOA, 2020), the project consists of a roadway and bridge widening on the east side of Date Palm Drive from East Palm Canyon Drive (Highway 111) to Perez Road. The existing Date Palm Drive bridge over the North Cathedral Canyon Flood Control Channel will be widened on the east side by approximately 11 feet. The bridge widening is anticipated to consist of a two-span reinforced concrete (RC) slab bridge supported by one wall pier (bent) structure in the middle of the channel and two abutments (CNS, 2020a; 2020b). The bridge extension will have two 22-foot long spans with a total bridge length of about 48½ feet (CNS, 2020a; 2020b).

Based on our review of the 50% plans (KOA, 2020), the project alignment will include two segments, the first from East Palm Canyon Drive (Highway 111) to Buddy Rogers Avenue and second from Buddy Rogers Avenue to Perez Road. The first segment will include three travel lanes in each direction separated by a striped/raised median with sidewalks and bicycle lanes in both directions. Street parking will be available along the west side of Date Palm Drive in the first segment. The second segment will include three travel lanes in each direction separated by a striped median with sidewalks in both directions, and a bicycle lane in the northbound direction.

In preparation of this report, we reviewed the as-built plans (Riverside County Flood Control, 1972), 50% plans (KOA, 2020), the general and foundation plans (CNS, 2020a; 2020b), and the Foundation Report and Log of Test Borings (LOTB) (Earth Mechanics, 2014; 2017) for the Date Palm Drive Bridge over the Whitewater Stormwater Channel, located approximately 200 feet north of Perez Road. The LOTB (Earth Mechanics, 2014; 2017) are included in Appendix A.

4 AS-BUILT FOUNDATION DATA

Based on the available as-built plans of the existing Date Palm Drive bridge over the Cathedral Canyon North Channel, the existing bridge was constructed in approximately 1972 (Riverside County Flood Control, 1972). The bridge is approximately 47 feet long. The bridge was constructed with cast-in-place (CIP), RC "T" beam girders. The bridge consists of two-spans that are each 22 feet in length and supported on a RC center pier (bent) structure and two, RC seated abutments at either end. The existing abutments and pier (bent) are indicated on the as-built plans as being supported on shallow foundations with design bearing capacities of 2,000 pounds per square foot (psf). The footings supporting the abutments are shown to be bearing at an elevation of approximately 275 feet above MSL, while the foundation supporting the center pier (bent) structure is indicated to be bearing at an approximate elevation of and 272 feet above MSL.

The bridge embankments at the abutments were constructed with approximately 1.5:1 (horizontal to vertical) fill slopes. The top of bridge deck elevation is approximately 284 feet above MSL. The concrete lined channel bottom elevation underneath the bridge is approximately 275 feet above MSL (Riverside County Flood Control, 1972).

The bridge as-built summary provided in this section is based on our review of the available as-built plans and is for informational purposes only. The contractor should review as-built plans and perform additional evaluation, including field verification, as necessary.

5 SUBSURFACE EVALUATION

Our subsurface exploration was performed on December 3 and 4, 2020 and consisted of the drilling, logging, and sampling of four, small-diameter borings (B-1 through B-4). Prior to commencing the subsurface exploration, USA was notified to mark out the existing utilities in the vicinity of our borings. The purpose of the borings was to evaluate subsurface conditions and to collect samples for geotechnical laboratory testing.

Borings B-1 and B-2 were performed on the north and south sides of the Cathedral Canyon North Channel and were drilled to depths of approximately 51 feet below ground surface using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Borings B-3 and B-4 were performed between East Palm Canyon Drive (Highway 111) and Buddy Rogers Avenue, and were excavated to depths of approximately 5 feet below ground surface using manual equipment (hand auger). During the drilling operations, the borings were logged and sampled by personnel from Ninyo & Moore. Representative bulk and in-place soil samples were obtained from the borings. The samples were then transported to our in-house geotechnical laboratory for testing. The approximate locations of the exploratory borings are shown on Figure 2. Logs of the borings are included in Appendix B.

6 LABORATORY TESTING

Geotechnical laboratory testing of representative soil samples included tests to evaluate in-situ moisture content and dry density, sieve (gradation) analysis, consolidation, shear strength, soil corrosivity, and R-value. The results of the moisture content and dry density testing are presented on the boring logs in Appendix B. Descriptions of the geotechnical laboratory test methods and the results of the other geotechnical laboratory tests performed are presented in Appendix C.

7 GEOLOGIC AND SUBSURFACE CONDITIONS

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

7.1. Regional Geologic Setting

The project area is situated in the Colorado Desert Geomorphic Province. This province is comprised of a depressed block between active branches of the San Andreas Fault. Ancient lake deposits and relict beach lines associated with the former Lake Cahuilla characterize much of the Colorado Desert geomorphic province. An outstanding geomorphic feature in this province is the northwest trending Salton Trough extending south from San Gorgonio Pass to the Gulf of California (Norris and Webb, 1990; Harden 2004). The Coachella Valley is located in the northern portion of the Salton Trough. The portion of the province in Riverside County that includes the project area consists generally of Quaternary alluvium and aeolian wind deposits (Figure 3).

7.2. Site Geology

The results of our geologic reconnaissance and subsurface evaluation indicate that the site is generally underlain by fill and alluvium (Dibblee and Mich, 2008). Generalized descriptions of the materials encountered during our subsurface exploration are presented below. Additional descriptions of the materials encountered in our exploratory borings are shown on the logs presented in Appendix B. The geology of the site is shown on Figure 3.

7.2.1. Fill

Fill materials were encountered at the ground surface in borings B-3 and B-4 and extended to the total depths explored of approximately 5 feet. As encountered, these materials generally consisted of light brown, dry, medium dense, silty sand with gravel and well graded sand with gravel. Cobbles, construction debris (including brick fragments) were encountered in the fill materials. Documentation of the placement and compaction of existing fill was not available for our review.

7.2.2. Alluvium

Quaternary-age alluvial sand and gravel of major creeks and stream washes and alluvial sand and gravel of valley areas are mapped at the project site (Dibblee and Mich, 2008) and were encountered at the surface in borings B-1 and B-2 and extended to the total depths explored of approximately 51 feet. As encountered, the alluvium generally consisted of various shades of brown and gray, dry to wet, medium dense to very dense, silty sand, well graded sand, poorly graded sand, sandy gravel, very stiff, silty clay. Gravel and cobbles were encountered in the alluvium.

7.3. Groundwater

Groundwater was not encountered during our subsurface evaluation or in nearby borings previously performed by Earth Mechanics (2014; 2017). However, perched water was encountered at the contact between granular and clayey materials at a depth of approximately 25 feet in boring B-2. Based on our review of the available well data, groundwater has been measured at a depth of approximately 200 feet below the ground surface (bgs) at a site south of the alignment (Delta, 2010). However, as the site includes the Cathedral Canyon North Channel, the area could be flooded by seasonal stormwater. The elevation of the Cathedral Canyon North Channel is approximately 275 feet above MSL. Fluctuations in the level of groundwater may occur due to variations in ground surface topography, subsurface stratification, rainfall, irrigation practices, groundwater pumping, and other factors.

8 GEOLOGIC HAZARDS

In general, hazards associated with seismic activity include strong ground motion, ground surface rupture, and liquefaction. These considerations and other geologic hazards, such as tsunamis and landsliding, are discussed in the following sections.

8.1. Faulting and Seismicity

Based on our review of the referenced geologic maps and aerial photographs, as well as our geologic field mapping, the subject site is not underlain by known active or potentially active faults (i.e., faults that exhibit evidence of ground displacement in the last 11,000 years and 2,000,000 years, respectively). The subject site is not located within a State of California Earthquake Fault Zone (EFZ) (formerly known as an Alquist-Priolo Special Studies Zone) (Hart and Bryant, 1997). However, like the majority of Southern California, the site is located in a seismically active area and the potential for strong ground motion is considered significant during the design life of the proposed improvements. The nearest known active fault is the San Andreas Fault, located approximately 5 miles east of the project site (Figure 4).

8.1.1. Surface Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project site. The active San Andreas Fault Zone is located approximately 5 miles east of the 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.

8.1.2. Ground Motion

The 2019 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCE_R) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. Per the 2019 CBC, a site-specific ground motion hazard analysis shall be performed for structures on Site Class D with a mapped MCE_R, 5 percent damped, spectral response acceleration parameter at a period of 1 second (S_1) greater than or equal to 0.2g in accordance with Sections 21.2 and 21.3 of the American Society of Civil Engineers (ASCE) Publication 7-16 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 0.653g using the 2019 Structural Engineers Association of California [SEAOC]/Office of Statewide Health Planning and Development [OSHPD] seismic design tool (web-based); therefore, a site-specific ground motion hazard analysis was performed for the project site.

The site-specific ground motion hazard analysis consisted of the review of available seismologic information for nearby faults and performance of probabilistic seismic hazard analysis (PSHA) and deterministic seismic hazard analysis (DSHA) to develop acceleration response spectrum (ARS) curves corresponding to the MCE_R for 5 percent damping. Prior to the site-specific ground motion hazard analysis, we obtained the mapped seismic ground motion values and developed the general MCE_R response spectrum for 5 percent damping in accordance with Section 11.4 of ASCE 7-16. The average shear wave velocity (V_S) for the upper 100 feet of soil (V_{S100}) is mapped to be 1,161 feet per second (fps) (354 meters per second) (Wills and Clahan, 2006) and the depths to V_S = 1,000 m/s and V_S = 2,500 m/s are assumed to be 60 meters and 270 meters, respectively (Southern California Earthquake Center [SCEC] Community Velocity Model Version 11.9.0 Basin Depth). These values were evaluated using the Open Seismic Hazard Analysis software developed by United States Geologic Survey (USGS) and (SCEC) (2020).

The 2014 new generation attenuation (NGA) West-2 relationships were used to evaluate the site-specific ground motions. The NGA relationships that we used for developing the probabilistic and deterministic response spectra are by Chiou and Youngs (2014), Campbell and Bozorgnia (2014), Boore, Stewart, Seyhan, and Atkinson (2014), and Abrahamson, Silva, and Kamai (2014). The Open Seismic Hazard Analysis software developed by USGS and SCEC (2019) was used for performing the PSHA. The Calculation of Weighted Average 2014 NGA Models spreadsheet by the Pacific Earthquake Engineering Research Center (PEER) was used for performing the DSHA (Seyhan, 2014).

The PSHA was performed for earthquake hazards having a 2 percent chance of being exceeded in 50 years multiplied by the risk coefficients per ASCE 7-16. The maximum rotated components of ground motions were considered in PSHA with 5 percent damping. For the DSHA, we analyzed accelerations from characteristic earthquakes on active faults within the region using the California Department of Transportation (Caltrans) ARS (Caltrans, 2020) seismic design tool (web-based) and the hazard curves and deaggregation plots at the site using the USGS Unified Hazard Tool application (USGS and SCEC, 2020). A magnitude 7.5 seismic event on the San Andreas fault zone with a rupture distance of 9.2 kilometers from the site was evaluated to be the controlling earthquake. Hence, the deterministic seismic hazard analysis was performed for the site using this event and corrections were made to the spectral accelerations for the 84th percentile of the maximum rotated component of ground motion with 5 percent damping.

The site-specific MCE_R response spectrum was taken as the lesser of the spectral response acceleration at any period from the PSHA and DSHA, and the site-specific general response spectrum was determined by taking two-thirds of the MCE_R response spectrum with some conditions in accordance with Section 21.3 of ASCE 7-16. 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 10.2 for the evaluation of seismic loads on the proposed structures.

The site-specific Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration (PGA_M) was calculated as 0.741g.

8.1.3. Liquefaction and Seismically Induced Settlement

Liquefaction is the phenomenon in which loosely deposited granular soils (with silt and clay contents of less than approximately 35 percent) and non-plastic silts located below the water table undergo rapid loss of shear strength when subjected to strong earthquake-induced ground shaking. Ground shaking of sufficient duration results in the loss of grain-to-grain contact due to a rapid rise in pore water pressure, and causes the soil to behave as a fluid for a short period of time. Liquefaction is known generally to occur in saturated or near-saturated cohesionless soils at depths shallower than 75 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.

As noted earlier, the bridge is underlain predominantly by thick layers of medium dense to very dense granular materials. Groundwater was not encountered during our evaluation or in the nearby geotechnical evaluations (Earth Mechanics, 2014; 2017) and based on review of groundwater monitoring well data in the site vicinity, groundwater is anticipated to be at a depth of 200 feet or greater (Geotracker, 2020). Note, that seepage was encountered at a depth of approximately 25 feet at the contact with a silty clay layer. While the bridge crosses a regulatory floodway, shallow static groundwater levels are not anticipated. Therefore, we do not consider liquefaction to be a design consideration.

Although we do not consider liquefaction to be a design consideration, the granular soils at the site could be subject to dry sand settlement during a design seismic event. Therefore, we evaluated the potential for dry sand settlement during a seismic event. The dry sand settlement evaluation was performed using a maximum moment magnitude of 7.5 associated with the San Andreas fault and MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) of 0.74g as discussed in previous sections. In order to estimate the amount of post-earthquake settlement, the methods proposed by Tokimatsu and Seed (1987) were used for the evaluation of dynamic settlement for dry sand settlement. 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. Post-earthquake total settlements on the order of approximately 1½-inch was calculated for the site using the computer program LiquefyPro (CivilTech Software, 2017). Differential settlements of approximately ¾-inch over a horizontal span of 50 feet should be expected. Our dry sand settlement analysis results are presented in Appendix D. These dynamic settlements are considered additional to the static settlement discussed in following sections.

8.1.4. Hydrocollapse

Conditions in arid and semi-arid climates favor the potential for hydrocollapse. Collapsible soils are susceptible to large volumetric strains when they become saturated. Based on our review of our laboratory testing results and our experience with similar soils in the general vicinity of the site, the onsite soils possess a low hazard for hydrocollapse.

8.2. Tsunamis and Seiches

Tsunamis are long wavelength seismic sea waves (long compared to the ocean depth) generated by sudden movements of the ocean bottom during submarine earthquakes, landslides, or volcanic activity. Seiches are similar oscillating waves on inland or enclosed bodies of water. Based on the inland location and elevation of the site, and the absence of nearby lakes or reservoirs, the potential for a tsunami or seiche to affect the site is not a design consideration.

8.3. Flood Hazards

Based on review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM, 2008), the majority of the project site is within the limits of Zone X: area of minimal flood hazard, with the exception of Date Palm Drive between Buddy Rogers Avenue and Perez Road (Figure 6). This area is within the Zone AE: special flood hazard area with base flood elevation and a regulatory floodway (Figure 6). Accordingly, the potential for flooding to impact the site is considered high at the site.

8.4. Landslides

Landslides, slope failures, and mudflows of earth materials generally occur where slopes are steep and/or the earth materials are too weak to support themselves. No landslides or indications of deep-seated landslides were noted underlying the project site based on our review of available geologic literature and topographic maps. Based on the relatively level topography at the site, the potential for landslides or mudflows to affect the project site is considered low.

9 CONCLUSIONS

Based on our geotechnical evaluation, it is our opinion that construction of the proposed improvements is feasible from a geotechnical standpoint, provided the following recommendations are incorporated into the design and construction of the project. Geotechnical considerations and conclusions include the following:

- The site is generally underlain by fill soils and alluvium. Fill soils were encountered in our borings that were drilled in the southern portion of the road alignment to depths up to approximately 5 feet below the ground surface.
- Groundwater was not encountered during our evaluation or in nearby previous evaluations performed by Earth Mechanics (2014; 2017). However, perched water was encountered as a zone of seepage at a depth of approximately 25 feet in our boring B-2. As previously mentioned, the alignment crosses a regulatory floodway. Fluctuations in the depth to groundwater will occur due to seasonal precipitation, flood events, variations in ground elevations, subsurface stratification, irrigation, groundwater pumping, storm water infiltration, and other factors.
- The onsite fill soils and alluvium can be generally excavated using heavy duty earthmoving equipment in good working condition. Difficult excavation should be anticipated in zones where cobbles are encountered.
- The onsite soils are considered to possess a low hazard for hydrocollapse.
- Due to the general cohesionless nature of the soils, caving should be anticipated by the contractor and excavations may require shoring, particularly if loose zones are encountered. Caving should also be anticipated if seepage, perched water conditions, and groundwater are encountered. Temporary casing may be needed for drilled hole excavations.
- There are no known active faults crossing at the site, and the potential for surface ground rupture is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible. Additionally, the San Andreas Fault is mapped approximately 5 miles east of the site.
- Based on a comparison of our laboratory testing data presented in Appendix B and the California amended (Caltrans, 2019) AASHTO (2017) corrosion criteria, the onsite soils are not considered corrosive.

10 RECOMMENDATIONS

Based on the results of our subsurface evaluation and our understanding of the project, we present the following geotechnical recommendations relative to the design and construction of the proposed street improvements. The proposed improvements should be constructed in accordance with the following recommendations and the requirements of the applicable governing agencies.

10.1. Earthwork

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

10.1.1. Pre-Construction Conference

We recommend that a pre-construction conference be held. The owner and/or their representative, the governing agencies' representatives, the civil engineer, Ninyo & Moore, and the contractor should be in attendance to discuss the work plan and project schedule and earthwork recommendations.

10.1.2. Site Preparation

Site preparations should begin with the removal of existing site improvements, vegetation, utility lines, asphalt, concrete, and other deleterious debris from areas to be graded. Tree stumps and roots should be removed to such a depth that organic material is generally not present. Clearing and grubbing should extend to the outside of the proposed excavation and fill areas. The debris and unsuitable material generated during clearing and grubbing should be removed from areas to be graded and disposed of at a legal dumpsite away from the project area.

10.1.3. Excavation Characteristics

The result of our field exploration program indicates that the project site is underlain by fill materials and alluvium. Excavation of the subsurface materials should be feasible with heavy-duty excavation equipment in good working condition. Difficult excavation should be anticipated if cobbles are encountered. Due to the cohesionless nature of the soils, caving should be anticipated by the contractor and excavations may require shoring, particularly where loose zones are encountered. Caving should also be anticipated if seepage, perched water conditions, and groundwater are encountered. Temporary casing may be needed for drilled hole excavations.

10.1.4. Temporary Excavations

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

Fill and Alluvium	*Type C*

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

10.1.5. Remedial Grading

In order to provide suitable support for the proposed new bridge structure and site retaining walls, we recommend that the existing alluvium be over-excavated and replaced with compacted, engineered fill. We recommend that the existing near-surface materials within the foundation footprint be removed to a depth of 3 feet below the bottom of footings. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed.

Subsequent to removal, the resulting surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompacted to a relative compaction of 90 percent as evaluated by the ASTM International (ASTM) Test Method D 1557 prior to placing new fill. Once the resulting removal surface has been recompacted, the overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index [EI] less than 50). These materials are anticipated to consist of the soils derived from on-site excavations that have been processed to meet the soils characteristics recommended in the "Materials for Fill" section of this report.

10.1.6. Materials for Fill

Materials for fill may be obtained from on-site excavations or may be import materials. Fill soils should possess an organic content of less than approximately 3 percent by volume (or 1 percent by weight). In general, fill material should not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ¾ inch. Large chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite.

Imported fill material, if needed, should generally be granular soils with a very low to low expansion potential (i.e., an EI of 50 or less). Import material should also be non-corrosive in accordance with the California amended AASHTO (2017) corrosion criteria (Caltrans, 2019), which is defined as a soil with an electrical resistivity value greater than 1,100 ohm-centimeters (ohm-cm), a chloride content of less than 500 parts per million (ppm), a soluble sulfate content of less than 1,500 ppm, and a pH greater than 5.5. The contractor should be responsible for the uniformity of import material brought to the site. We recommend that materials proposed for use as import fill be evaluated from a contractor's stockpile rather than in-place materials. Materials for use as fill should be evaluated by the project geotechnical consultant's representative prior to filling or importing. Do not import soils that exhibit a known risk to human health, the environment, or both.

10.1.7. Fill Placement and Compaction

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve a moisture content generally above the laboratory optimum, mixed, and then compacted by mechanical methods, to a relative compaction of 90 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved. The upper 12 inches of the subgrade materials underneath the pavements should be placed to a relative compaction of 95 percent as evaluated by ASTM D 1557. Additionally, aggregate base materials underneath vehicular pavements should be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557.

10.1.8. Pipe Bedding and Modulus of Soil Reaction (E')

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

The modulus of soil reaction (E') is used to characterize the stiffness of soil backfill placed at the sides of buried flexible pipes for the purpose of evaluating deflection caused by the weight of the backfill over the pipe (Hartley and Duncan, 1987). A soil reaction modulus of 1,200 pounds per square inch (psi) may be used for an excavation depth of up to approximately 5 feet when backfilled with granular soil compacted to a relative compaction of 90 percent as evaluated by the ASTM D 1557. A soil reaction modulus of 1,800 psi may be used for trenches deeper than 5 feet.

10.1.9. Utility Pipe Zone Backfill

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

It has been our experience that the voids within a crushed rock material are sufficiently large to allow fines to migrate into the voids, thereby creating the potential for sinkholes and depressions to develop at the ground surface. If open-graded gravel is utilized as pipe zone backfill, this material should be separated from the adjacent trench sidewalls and overlying trench backfill with a geosynthetic filter fabric

10.1.10. Utility Trench Zone Backfill

Utility trench zone backfill material should be generally free of trash, debris, roots, vegetation, or deleterious materials. Trench zone backfill should generally be free of rocks or hard lumps of material in excess of 3 inches in diameter. Rocks or hard lumps larger than about 3 inches in diameter should be broken into smaller pieces or should be removed from the site. Oversize materials should be separated from material to be used as trench backfill. Moisture conditioning (including drying and/or mixing) of existing on-site materials is anticipated if reused as trench backfill.

10.1.11. Lateral Pressures for Thrust Blocks

Thrust restraint for buried pipelines may be achieved by transferring the thrust force to the soil outside the pipe through a thrust block. Thrust blocks may be designed using the lateral passive earth pressures presented on Figure 7. Thrust blocks should be backfilled with granular backfill material and compacted in accordance with recommendations presented in this report.

10.1.12. Drainage

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained and drainage patterns should be established to divert and remove water from the site to appropriate outlets, in accordance with the recommendations of the project civil engineer.

10.2. Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 1 presents the site-specific spectral response acceleration parameters in accordance with the CBC (2019) guidelines.

Table 1 – 2019 California Building Code Seismic Design Criteria

Seismic Design Factors	Value
Site Class	D
Mapped Spectral Acceleration at 0.2-second Period, S_s	1.588g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.653g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.759g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	1.306g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	1.173g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.871g
Site-Specific Maximum Considered Earthquake Geometric Mean (MCE_G) Peak Ground Acceleration (PGA_M)	0.741g

10.3. Foundations

We anticipate that the proposed improvements will consist of a reinforced concrete slab bridge that is supported on conventional shallow foundations. 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.

10.3.1. Shallow Footings

Shallow, spread or continuous footings supported on compacted fill may be designed using an allowable bearing capacity of 2,500 psf. These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. We recommend that shallow foundations for the new improvements be founded 18 inches below the lowest adjacent grade, or below anticipated scour depth (whichever is deeper). Continuous footings should have a width of 18 inches and spread footings should be 24 inches in width. The footings should be reinforced in accordance with the recommendations of the project structural engineer.

Attention should be given to designing the foundations for the bridge extension adjacent to the existing portion of the bridge. It is advisable to place the foundations for the bridge extension at the same level as the foundations for the existing bridge so that the new footings will not undercut the soil beneath the existing footings. In spite of these precautions, small differential movements between the adjacent structures may be experienced.

To provide consistent bearing conditions for the foundations, we recommend that no utilities, piping, or duct banks be constructed within 1 foot of the zone of influence of the bottom of each foundation. The zone of influence is defined by a 1:1 (horizontal to vertical) downward projection that extends outward from the bottom, outside edge of the foundation.

10.3.2. Lateral Resistance

For resistance of footings to lateral loads bearing on compacted fill, we recommend an allowable passive pressure of 300 psf per foot of depth be used with a value of up to 3,000 psf. This value assumes that the ground is horizontal for a distance of 10 feet, or three multiplied by the height generating the passive pressure, whichever is more. We recommend that the upper 1 foot of soil not protected by pavement or a concrete slab be neglected when calculating passive resistance.

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.35 be used between soil and concrete. These values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

10.3.3. Static Settlement

We estimate that the proposed structures, designed and constructed as recommended herein, and founded in compacted fill will undergo total settlement on the order of 1 inch. Differential settlement on the order of ½ inch over a horizontal span of 40 feet should be expected.

10.4. Site Retaining Walls

Site retaining walls associated with the bridge and roadway widening may be supported on continuous footings bearing entirely on compacted fill. The continuous footings should have a width of 24 inches or more and be embedded a depth of 18 inches or more (or below the anticipated scour depth, whichever is deeper). An allowable bearing capacity of 2,500 psf may be used for the design of site retaining wall foundations. The allowable bearing capacity may be increased by one-third when considering loads of short duration, such as wind or seismic forces.

For the design of a site yielding retaining wall that is not restrained against movement by rigid corners or structural connections, lateral pressures are presented on Figure 8. For the design of a site rigid retaining wall that is restrained against movement by rigid corners or structural connections, lateral pressures are presented on Figure 9. These pressures assume granular backfill materials are used and free draining conditions. Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. A drain should be provided behind the retaining wall as shown on Figure 10. The drain should be connected to an appropriate outlet.

10.5. Preliminary Flexible Pavement Design

Our laboratory testing of near surface soil samples at the project site indicated R-values of 70 and 72. Due to the potential for variability of soils, we used an R-value of 50 for preliminary design of the pavement. KOA has provided us a Traffic Index (TI) of 10 for design of the pavement sections. The R-value of 50 and the design TI of 10 has been the basis of our preliminary flexible pavement design. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement section is presented in Table 2.

Table 2 – Recommended Preliminary Flexible Pavement Section

Traffic Index	Design R Value	Asphalt Concrete Thickness (inches)	Class 2 Aggregate Base Thickness (inches)
10	50	6.0	8.0

As indicated, the pavement section is based on a TI of 10. If traffic loads are different from those assumed herein, the pavement design should be re-evaluated. In addition, we recommend that the upper 12 inches of the subgrade and aggregate base materials be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557.

10.6. Corrosion

Laboratory testing was performed on representative samples of near-surface soil to evaluate soil pH, electrical resistivity, water-soluble chloride content, and water-soluble sulfate content. The soil pH and electrical resistivity tests were performed in general accordance with California Test Method (CT) 643. Chloride content tests were performed in general accordance with CT 422. Sulfate testing was performed in general accordance with CT 417.

The pH of the tested samples were measured at approximately 7.3 and 7.7, electrical resistivities were measured at approximately 6,500 and 7,900 ohm-cm, chloride contents were measured at approximately 10 and 55 ppm, and sulfate contents were measured at approximately 0.001 percent (i.e., 10 ppm). Based on a comparison with the California amended AASHTO (2017) corrosion criteria (Caltrans, 2019), the onsite soils would not be classified as corrosive. Corrosive soils are defined by the California amended AASHTO (2017) corrosion criteria (Caltrans, 2019) as soils with an electrical resistivity less than 1,100 ohm-cm, a chloride content more than 500 ppm, more than 0.15 percent sulfates (1,500 ppm), and/or a pH less than 5.5.

10.7. Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates that can be subject to premature chemical and/or physical deterioration. As noted, the soil sample tested in this evaluation indicated a water-soluble sulfate content of 0.001 percent by weight (i.e., 10 ppm). Based on the American Concrete Institute (ACI) 318 criteria, the site soils would correspond to exposure class S0. For this exposure class, ACI 318 recommends that normal weight concrete in contact with soil possess a compressive strength of 2,500 psi or more. However, due to the potential for variability of site soils, we also recommend that normal weight concrete in contact with soil use Type II, II/V, or V cement.

10.8. Sidewalks, Curbs, and Gutters

In general, sidewalks, curbs, and gutters may be supported on granular soils with a very low to low expansion potential (i.e., an EI of 50 or less) compacted to a relative compaction of 90 percent in accordance with the current edition of ASTM D 1557. The subgrade soil should be maintained at a moisture content slightly above the laboratory optimum until the concrete is poured. Sidewalks, curbs, and gutters should be designed and constructed in accordance with the Cathedral City or the governing agency standards.

10.9. Drainage

Proper surface drainage is imperative for satisfactory site performance. Positive drainage should be provided and maintained to direct surface water away from the new sidewalk and retaining wall improvements. Positive drainage is defined as a slope of 2 percent or more over a distance of 5 feet away from the foundations and tops of slopes. Runoff should then be directed by the use of swales or pipes into a collective drainage system. Surface waters should not be allowed to pond adjacent to footings or pavements.

11 PLAN REVIEW AND CONSTRUCTION OBSERVATION

The conclusions and recommendations provided in this report are based on our understanding of the proposed project and on our evaluation of the data collected based on subsurface conditions disclosed by widely spaced exploratory borings. If conditions are found to vary from those described in this report, Ninyo & Moore should be notified, and additional recommendations will be provided upon request. Ninyo & Moore should review the final project drawings and specifications prior to the commencement of construction. Ninyo & Moore should perform the needed observation and testing services during construction operations.

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client with a letter (with a copy to Ninyo & Moore) indicating that they fully understand Ninyo & Moore's recommendations, and that they are in full agreement with the design parameters and recommendations contained in this report. Construction of proposed improvements should be performed by qualified subcontractors utilizing appropriate techniques and construction materials.

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

- Observing excavation bottoms and the placement and compaction of fill, including retaining wall backfill.
- Evaluating imported materials prior to their use as fill, if used.
- Performing field tests to evaluate fill compaction.
- Observing foundation excavations for bearing materials and cleaning prior to placement of reinforcing steel or concrete.

12 LIMITATIONS

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

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

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

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

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

13 REFERENCES

Abrahamson, N.A. and Silva, W.J., 1996, Empirical Ground Motion Models, report prepared for Brookhaven National Laboratory, New York, NY, 144 p.

Abrahamson, N.A., Silva, W.J., and Kamai, R., 2014, Summary of the ASK 14 ground motion relation for active crustal regions, Earthquake Spectra Volume 30, pp. 1025-1055.

American Concrete Institute, 2014, ACI 318 Building Code Requirements for Structural Concrete (ACI 318) and Commentary (ACI 318R).

American Society of Civil Engineers (ASCE), 2017, Minimum Design Loads for Buildings and Other Structures, ASCE 7-16.

Blake, T.F., Hollingsworth, R.A. and Stewart, J.P., 2002, Recommended Procedures for Implementation of DMG Special Publication 117-Guidelines for Analyzing and Mitigating Landslide Hazards in California: Committee Organized Through the ASCE, Los Angeles Section Geotechnical Group, Document Published by the Southern California Earthquake Center, 101p.

Bray, J.D., Rathje E.M., Auguello, A.J. and Merry, S.M., 1998, Simplified Seismic Design Procedure for Geosynthetic-Lined Solid Waste Landfills: Geosynthetics International, V.5, No. 1-2, pp. 203-235.

Boore, D.M., Stewart, J.P., Seyhan, E., and Atkinson, G.M., 2014, NGA-West2 Equations for Predicting PGA, PGV, and 5% Damped PSA for Shallow Crustal Earthquakes, Earthquake Spectra, Vol. 30, No. 3, pp. 1057-1085: dated August.

Building News, 2018, "Greenbook," Standard Specifications for Public Works Construction: BNI Publications.

Building Seismic Safety Council, 2009, National Earthquake Hazards Reduction Program (NEHRP) Recommended Seismic Provisions for New Buildings and Other Structures (FEMA P-750).

California Building Standards Commission, 2019, California Building Code (CBC), Title 24, Part 2, Volumes 1 and 2.

California Department of Transportation (Caltrans), 2018, Corrosion Guidelines (Version 3.0), Division of Engineering and Testing Services, Corrosion Technology Branch: dated March.

California Department of Transportation (Caltrans), 2019, California Amendments to the AASHTO LRFD Bridge Design Specifications (2017 Eighth Edition): dated April.

California Department of Conservation Website, 2020, Interactive Fault Activity Map of California https://maps.conservation.ca.gov/cgs/fam/: accessed in October.

California Geological Survey (CGS), 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A.

California Geological Survey (CGS), 2015, Earthquake Fault Zones, Indio Quadrangle, Revised Official Map, Released December 4.

Campbell, K.W., and Bozorgnia, Y., 2014, NGA-West2 Ground Motion Model for the Average Horizontal Components of PGA, PGV, and 5% Damped Linear Acceleration Response Spectra, Earthquake Spectra, Vol. 30, No. 3, pp. 1087-1115: dated August.

Chiou, B. S.-J., and Youngs, R.R., 2014, Update of the Chiou and Youngs NGA Model for the Average Horizontal Component of Peak Ground Motion and Response Spectra, Earthquake Spectra, August 2014, Vol. 30, No. 3: dated August.

CivilTech Software, 2017, LiquefyPro (Version 5.9c), A Computer Program for Liquefaction and Settlement Analysis.

CNS Engineers, 2020a, Foundation Plan, Date Palm Drive Bridge Widening, Cathedral City, California: dated October 12.

CNS Engineers, 2020b, General Plan, Date Palm Drive Bridge Widening, Cathedral City, California: dated October 12.

County of Riverside, 2015, General Plan – Safety Element: dated December.

Department of Toxic Substances Control, 2001, Information Advisory - Clean Imported Fill Material: dated October.

Delta, 2010, First Quarter 2010 Remediation Status Update Report, Former Shell Service Station, 69010 Hwy. 111, Cathedral City, California, Site #200420469: dated April 12.

Dibblee, T.W., and Mich, J.A., 2008, Geologic Map of the Thousand Palms & Lost Horse Mountain 15 minute Quadrangles, Riverside County, California, Scale 1:62,500.

Earth Mechanics, Inc., 2014, Foundation Report, Date Palm Drive Bridge (Widen) Over the Whitewater Stormwater Channel, City of Cathedral City, California: dated April 30.

Earth Mechanics, Inc., 2017, Log of Test Borings, Date Palm Drive Bridge, City of Cathedral City, California: dated January 27.

GeoTracker, 2020, http://geotracker.waterboards.ca.gov/; accessed in October.

Google Earth, 2020, https://www.google.com/earth/: accessed in October.

Harden, D.R., 2004, California Geology: Prentice Hall, Inc.

Hart, E.W., and Bryant, W.A., 1997, Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps: California Geological Survey, Special Publication 42, with Supplements 1 and 2 Added in 1999.

Hartley, J.D., and Duncan, J.M., 1987, E’ and Its Variation with Depth: American Society of Civil Engineers (ASCE), Journal of Transportation Engineering, Vol. 113, No. 5: dated September.

Historic Aerials, 2020, https://www.historicaerials.com/viewer; accessed in October.

Ishihara, K. and Yoshimine, M, 1992, Evaluation of Settlements in Sand Deposits following Liquefaction during Earthquakes, Soils and Foundations, JSSMFE, Vol. 32, No. 1: dated March.

KOA, 2020, 50% Submittal Plans, Date Palm Drive, City of Cathedral City: undated.

Ninyo & Moore, In-house Proprietary Data.

Ninyo & Moore, 2020, Proposal for Geotechnical Evaluation, Date Palm Drive Bridge and Roadway Easterly Widening, Cathedral City, Riverside County, California, Project No. 108980001: dated September 24.

Norris, R.M. and Webb, R.W., 1990, Geology of California: John Wiley & Sons.

Riverside County Flood Control and Water Conservation District, 1972, Cathedral Canyon Plan & Profile Sta. 20+00 to Sta. 5+00 As-Built Plans: dated November 21.

Riverside County Mapping Portal Website, 2020, https://gisopendata-countyofriverside.opendata.arcgis.com/: accessed in October.

Seyhan, E, 2014, Weighted Average 2014 NGA West-2 GMPE, Pacific Earthquake Engineering Research Center.

Southern California Earthquake Data Center, 2005, SCEC Community Velocity Model, Version 4.

Structural Engineering Association of California (SEAOC), Office of Statewide Health Planning and Development (OSHPD), 2020, U.S. Seismic Design Maps website, https://seismicmaps.org/: Accessed in October.

Tokimatsu, K. and Seed, H.B., 1987, Evaluation of Settlements in Sands due to Earthquake Shaking, Journal of Geotechnical Engineering, 113(8), 861-878.

United States Federal Emergency Management Agency (FEMA), 2008, Flood Insurance Rate Map (FIRM), No. 06065C1577G: dated August 28.

United Stated Geological Survey, 2018, Thousand Palms 7.5 Minute Quadrangle, Riverside County, California, Scale 1:24,000.

United States Geological Survey, 2020a, Unified Hazard Tool; https://earthquake.usgs.gov/hazards/interactive/.

United States Geological Survey, 2020b, Slope Based Vs30 Map Viewer; https://usgs.maps.arcgis.com/apps/webappviewer/index.html?id=8ac19bc334f747e486550f32837578e1.

United States Geological Survey and Southern California Earthquake Center, 2020, Open Seismic Hazard Analysis, http://www.opensha.org/.

University of California, 2020, CalME: Caltrans Mechanistic-Empirical Tool (Version 3.DD001.3): dated August.

Wills, C.J., and Clahan, L.B., 2006, Developing a Map of Geologically Defined Site-Condition Categories for California, Bulletin of the Seismological Society of America, v. 96, no. 4A, p. 1483–1501.

Youd, T.L., and Idriss, I.M. (Editors), 1997, Proceedings of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, Salt Lake City, Utah, January 5 through 6, 1996, NCEER Technical Report NCEER-97-0022, Buffalo, New York.

Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J.P., Liao, S.S.C., Marcusson, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R.B., and Stokoe, K.H., II., 2001, Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils, Journal of Geotechnical and Geoenvironmental Engineering: American Society of Civil Engineering 124(10), pp. 817-833.

Youd, T.L., Hansen, C.M., and Bartlett, S.F., 2002, Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement: Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 12, p. 1007-1017.

DRAFT

FIGURES

1_108980001_SL.mxd 1/14/2021 AOB

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2020

FIGURE 1

SITE LOCATION

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108980001 | 1/21

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2020

2_108980001_EL.mxd 1/22/2021 AOB

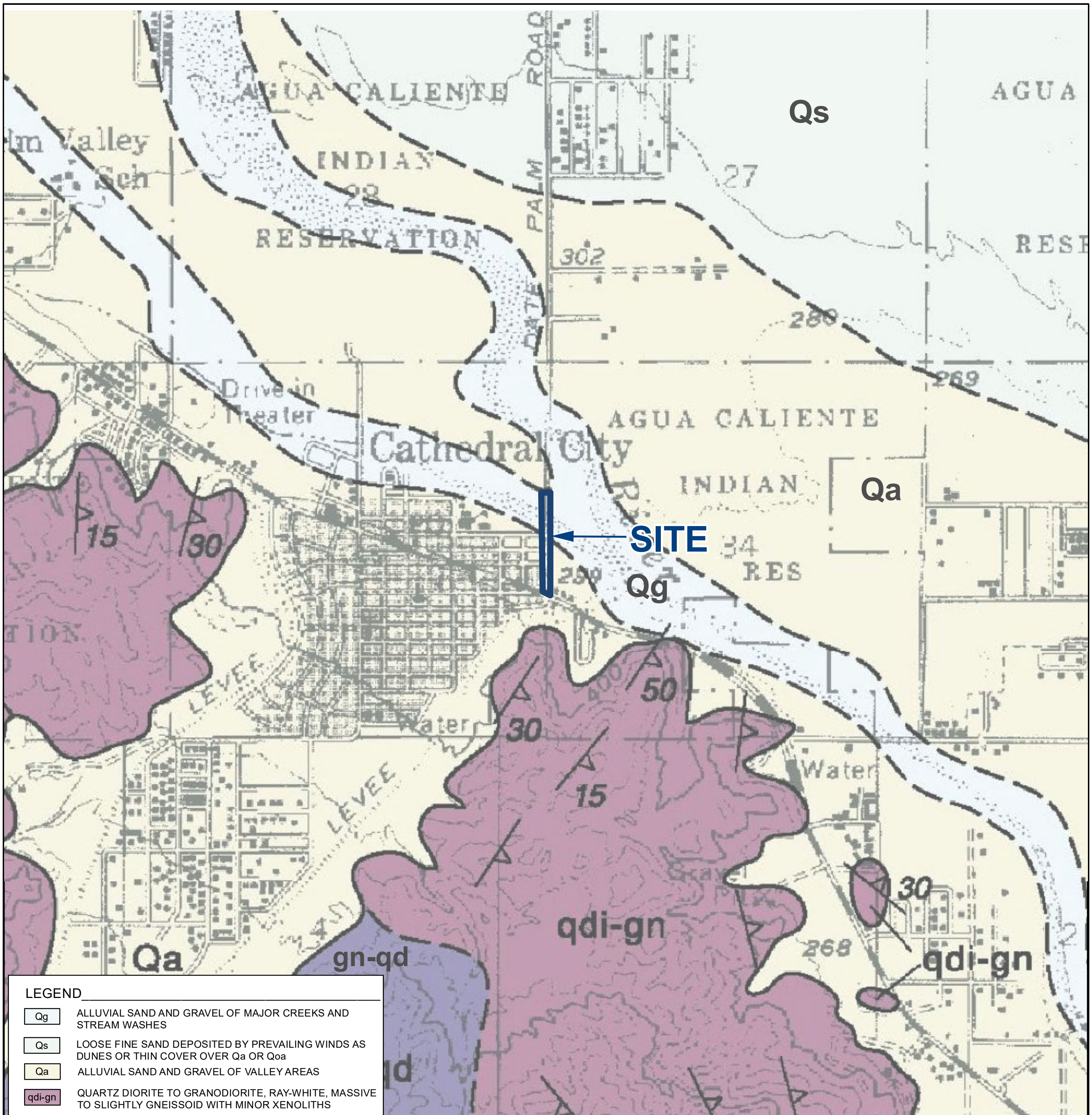
FIGURE 2

EXPLORATION LOCATIONS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108980001 | 1/21



LEGEND

Qg	ALLUVIAL SAND AND GRAVEL OF MAJOR CREEKS AND STREAM WASHES
Qs	LOOSE FINE SAND DEPOSITED BY PREVAILING WINDS AS DUNES OR THIN COVER OVER Qa OR Qoa
Qa	ALLUVIAL SAND AND GRAVEL OF VALLEY AREAS
qdi-gn	QUARTZ DIORITE TO GRANODIORITE, RAY-WHITE, MASSIVE TO SLIGHTLY GNEISSOID WITH MINOR XENOLITHS
gn-qd	GNEISS-SCHIST-QUARTZ DIORITE COMPLEX

70 U D — Fault - Solid where accurately located; dashed where approximately located; dotted where concealed. U = upthrown block, D = downthrown block. Arrow and number indicate direction and angle of dip of fault plane.

Strike and dip of primary igneous foliation

50 — Inclined

Vertical

REFERENCE: DIBBLEE, T.W., 2008, GEOLOGIC MAP OF THE THOUSAND PALMS & LOST HORSE MOUNTAIN 15-MINUTE QUADRANGLES, RIVERSIDE CALIFORNIA.

N

FEET

0 2,000 4,000

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

3_108980001_G.mxd 1/14/2021 AOB

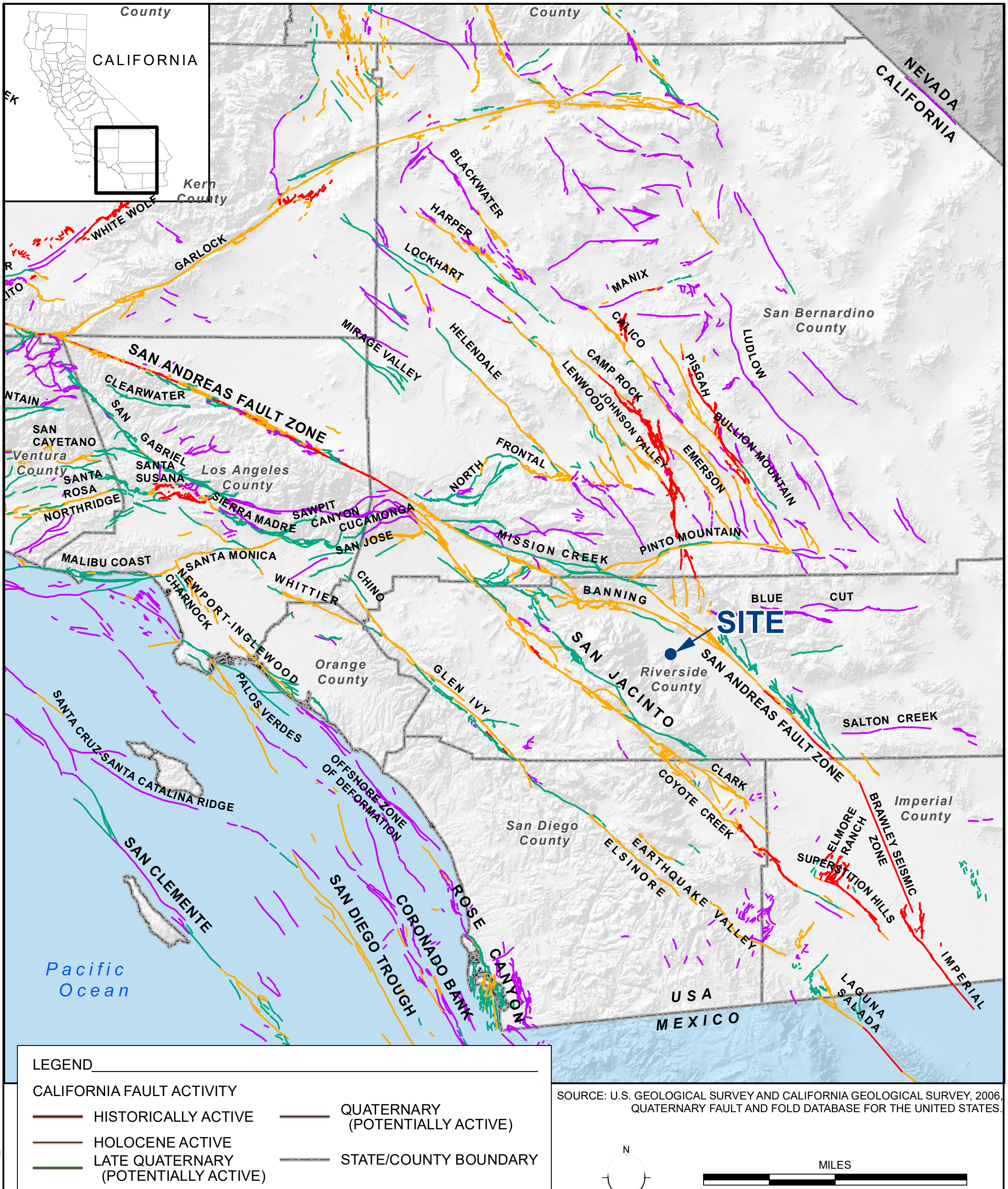
FIGURE 3

GEOLOGY

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108980001 | 1/21



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

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

4_108980001_FL.mxd 1/14/2021 AOB

FIGURE 4

FAULT LOCATIONS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

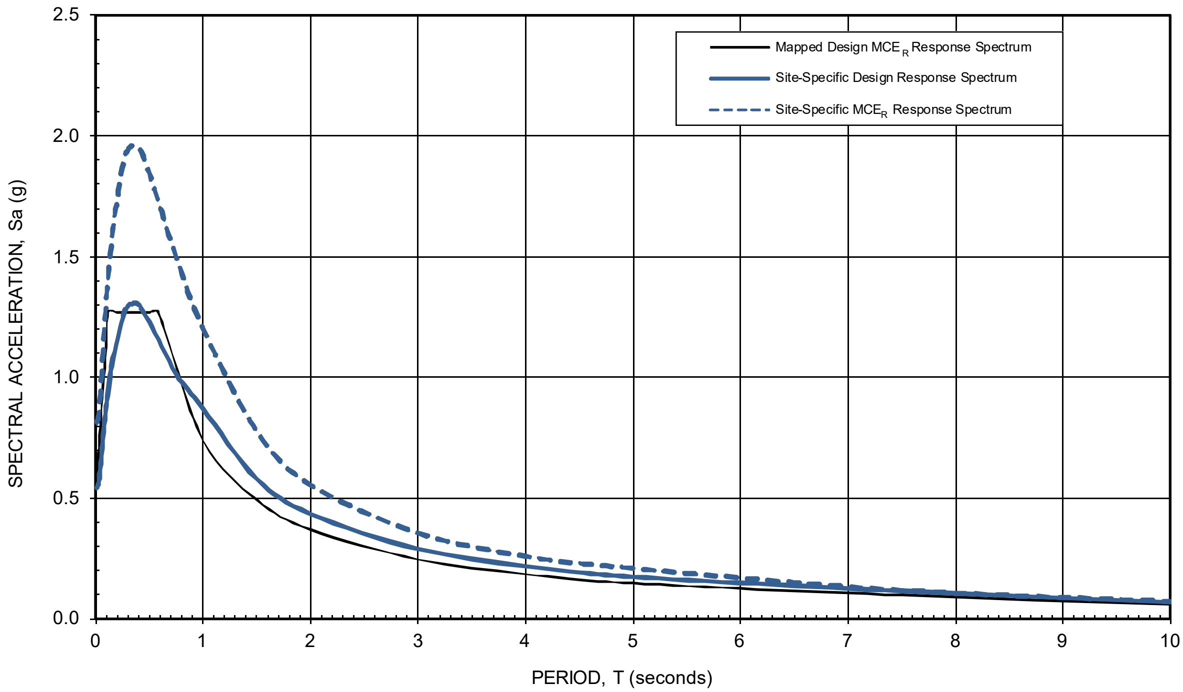
108980001 | 1/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM S_a (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM S_a (g)
0.010	0.815	0.543
0.020	0.823	0.548
0.030	0.852	0.568
0.050	0.966	0.644
0.075	1.157	0.772
0.100	1.323	0.882
0.150	1.570	1.046
0.200	1.737	1.158
0.250	1.870	1.246
0.300	1.945	1.296
0.400	1.954	1.303

PERIOD (seconds)	SITE-SPECIFIC MCE_R RESPONSE SPECTRUM S_a (g)	SITE-SPECIFIC DESIGN RESPONSE SPECTRUM S_a (g)
0.500	1.845	1.230
0.750	1.500	1.016
1.000	1.203	0.871
1.500	0.778	0.580
2.000	0.555	0.435
3.000	0.356	0.290
4.000	0.260	0.218
5.000	0.209	0.174
7.500	0.119	0.116
10.000	0.073	0.070

S_{DS} = 1.173 g	S_{D1} = 0.871 g	S_{MS} = 1.759 g	S_{M1} = 1.306 g	PGA_M = 0.741 g



NOTES:

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

2 The deterministic ground motion spectral response accelerations are for the 84th percentile of the geometric mean values in the maximum direction using the Chiou & Youngs (2014), Campbell & Bozorgnia (2014), Boore et al. (2014), and Abrahamson et al. (2014) attenuation relationships for deep soil sites considering a Mw 7.5 event on the San Andreas Fault fault zone located 9.20 kilometers from the site. It conforms with the lower bound limit per ASCE 7-16 Section 21.2.2.

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

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

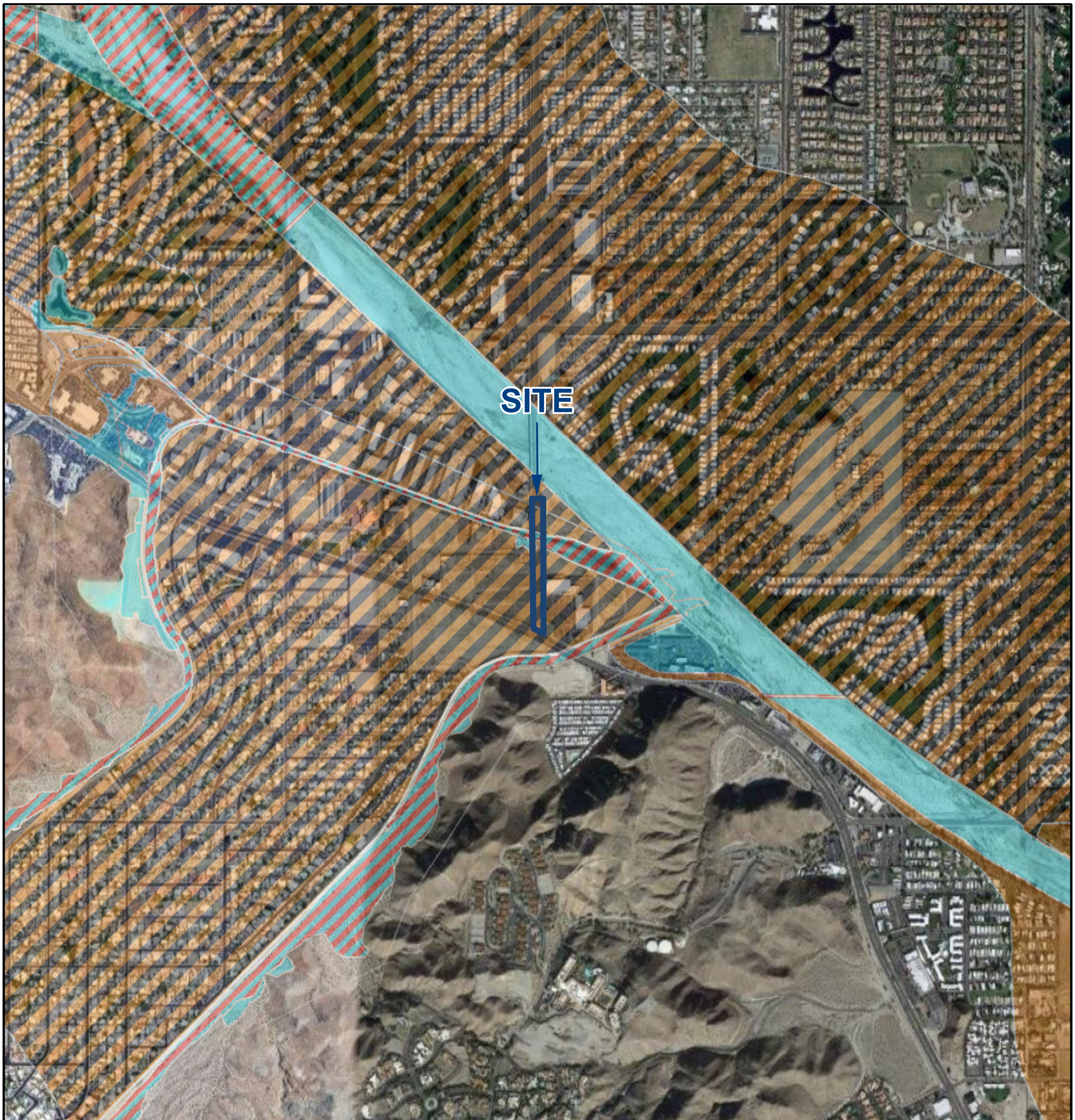
FIGURE 5

ACCELERATION RESPONSE SPECTRA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21



LEGEND

FLOOD HAZARD ZONES

ZONE TYPE

- 1% ANNUAL CHANCE FLOOD HAZARD
- REGULATORY FLOODWAY
- 0.2% ANNUAL CHANCE FLOOD HAZARD
- AREA WITH REDUCED FLOOD RISK DUE TO LEVEE

SOURCE: FLOOD INSURANCE RATE MAPS (FIRMS), FEDERAL EMERGENCY MANAGEMENT AGENCY (FEMA), DATED 6/23/13.

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FEET

0 1,500 3,000

6_108980001_FH.mxd 1/22/2021 AOB

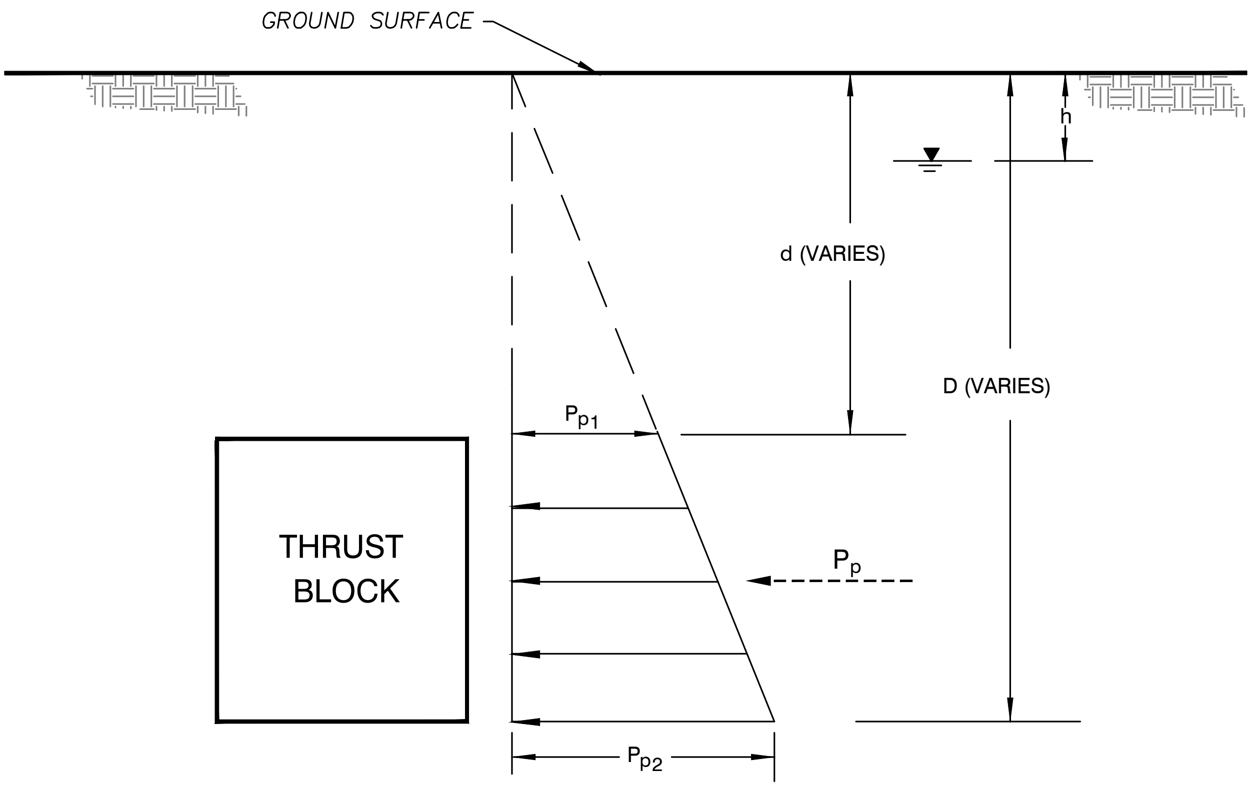
FIGURE 6

FLOOD HAZARDS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTES:

1. GROUNDWATER BELOW BLOCK
   $P_p = 175\,(D^2 - d^2)$ lb/ft
2. GROUNDWATER ABOVE BLOCK
   $P_p = 1.5\,(D - d)[\,124.8h + 58\,(D+d)]$ lb/ft
3. ASSUMES BACKFILL IS GRANULAR MATERIAL
4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
5. D, d AND h ARE IN FEET
6. ▼ GROUNDWATER TABLE

108980001 DETAIL FIGS.DWG AOB

NOT TO SCALE

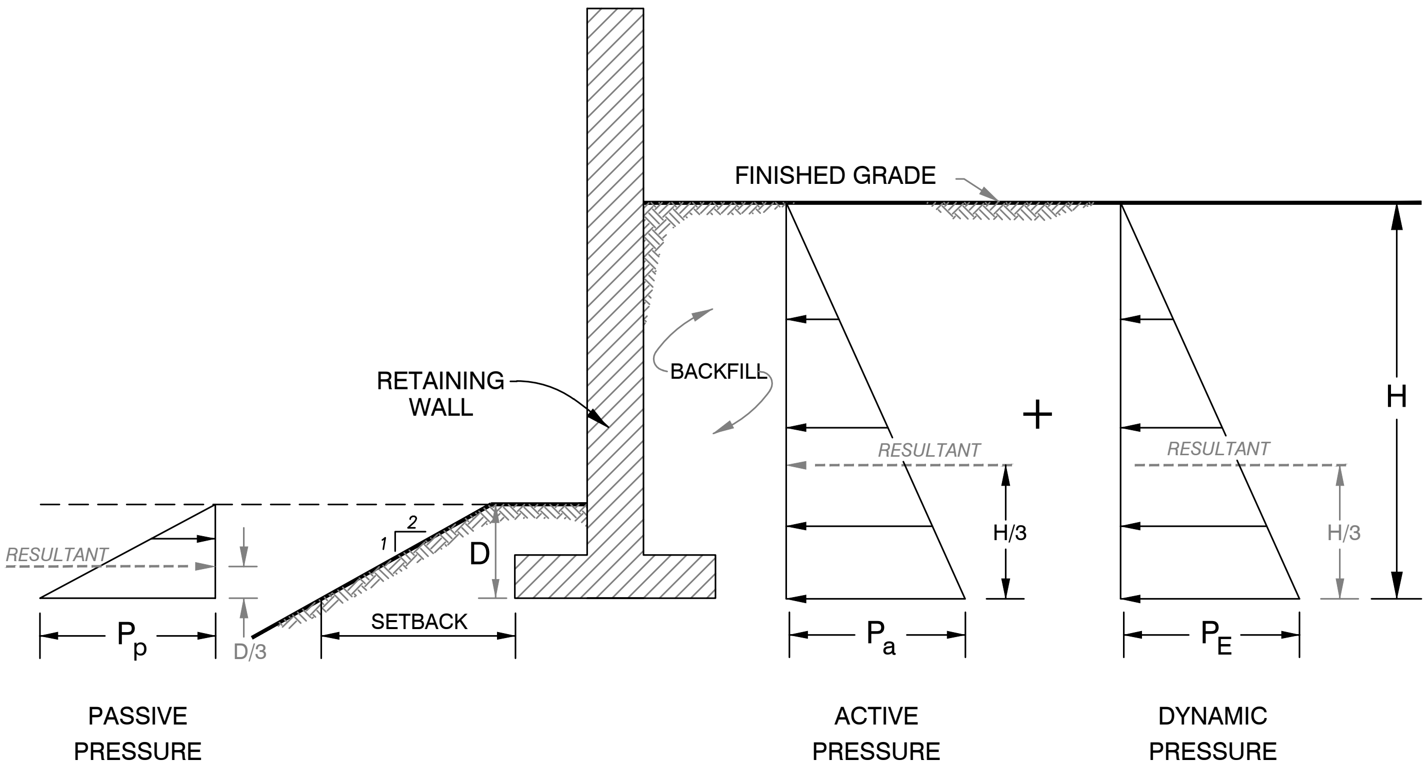
FIGURE 7

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

108980001 I 1/21



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. CALTRANS STRUCTURE BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A MAPPED DESIGN PEAK GROUND ACCELERATION OF 0.741g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET
8. SETBACK SHOULD BE IN ACCORDANCE WITH THE CBC (2019)

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)(1)	
P_a	Level Backfill with Granular Soils (2)	2H:1V Sloping Backfill with Granular Soils (2)
	35H	55H
P_E	34H	
P_p	Level Ground (Submerged)	2H:1V Descending Ground
	300D (140D)	130D

NOT TO SCALE

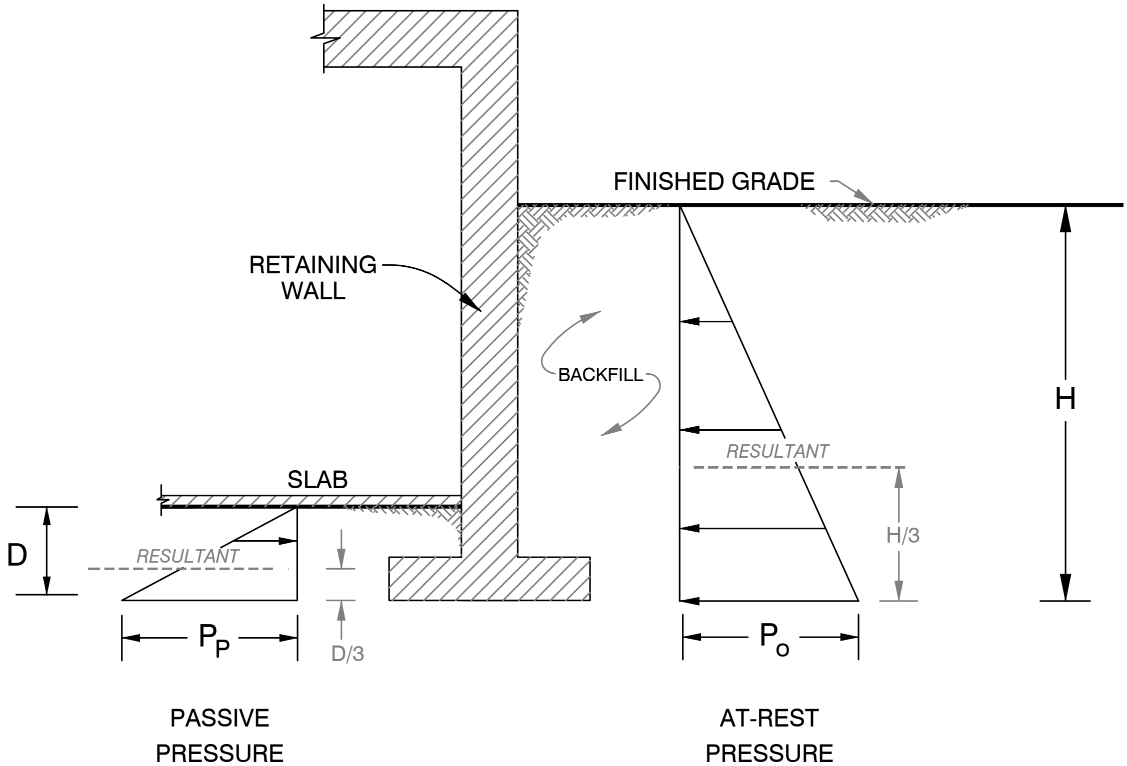
108980001 DETAIL FIGS.DWG AOB

FIGURE 8

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. CALTRANS STRUCTURE BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A MAPPED DESIGN PEAK GROUND ACCELERATION OF 0.741g
5. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure ($lb/ft^2/ft$)[1]	
P_O	Level Backfill with Granular Soils [2]	2H:1V Sloping Backfill with Granular Soils [2]
	55H	75H
P_E	34H	
P_P	Level Ground (Submerged)	2H:1V Descending Ground
	300D (140D)	130D

NOT TO SCALE

108980001 DETAIL FIGS.DWG AOB

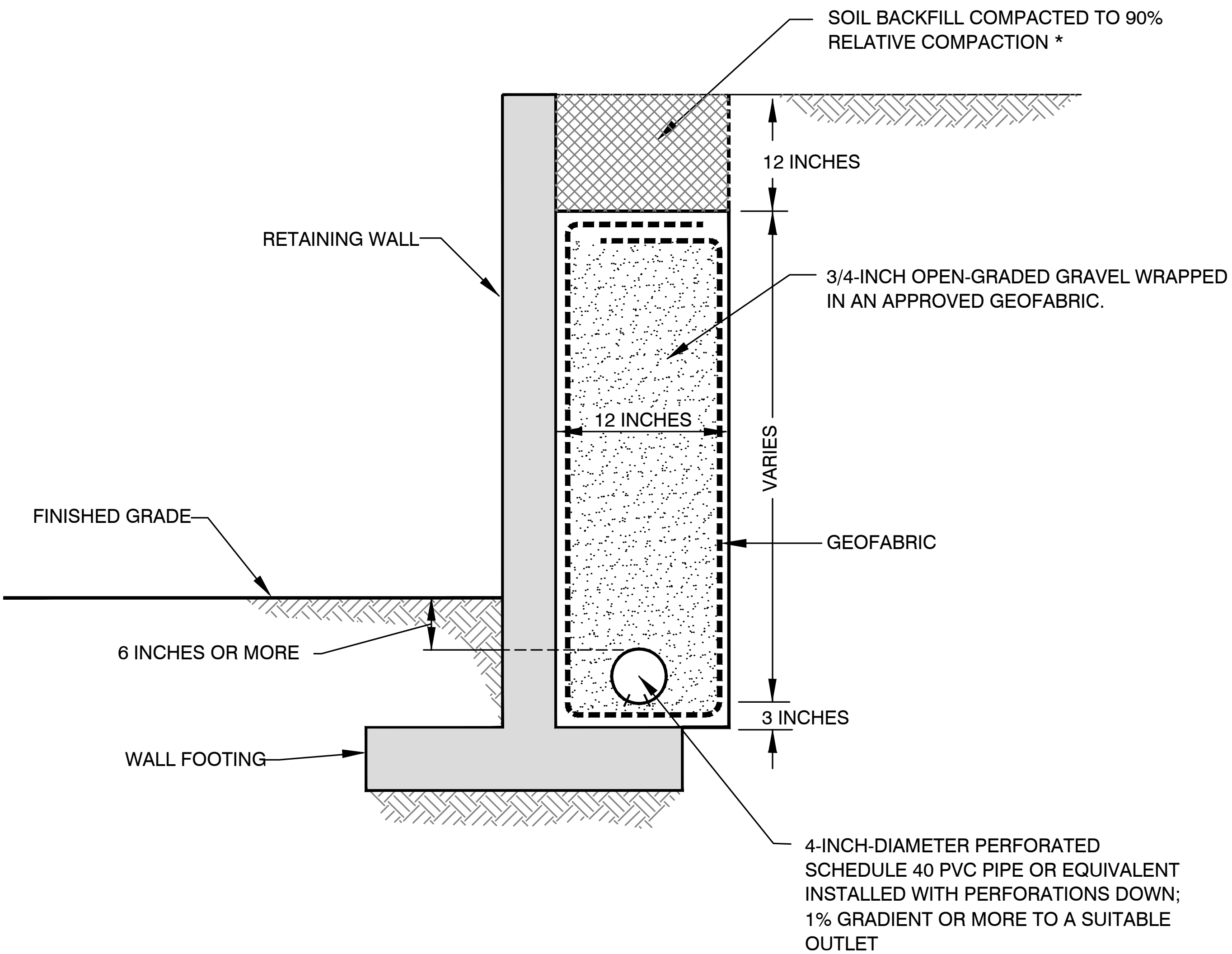
FIGURE 9

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108980001 I 1/21



*BASED ON ASTM D1557

NOT TO SCALE

FIGURE 10

RETAINING WALL DRAINAGE DETAIL

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
DATE PALM DRIVE, CATHEDRAL CITY, CALIFORNIA

108980001 I 1/21

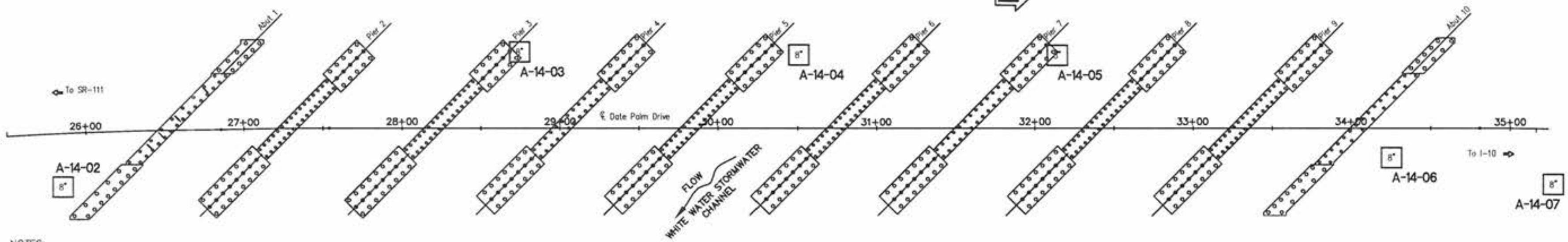
APPENDIX A

Log of Test Borings (Earth Mechanics, 2014 and 2017)

LOG OF BORING NO. A-14-01

Approx. Grade Elevation:	+288 ft				
Boring Depth:	11.5 ft	Driller:	2R Drilling Inc.		SHEET 1 OF 1
Borehole Diameter:	8 in	Type of Rig:	CME 55	Comments:	
Date Drilled:	1-21-14	Drive Wt. (lb):	140		
Logged By:	AT	Drop (in):	30		

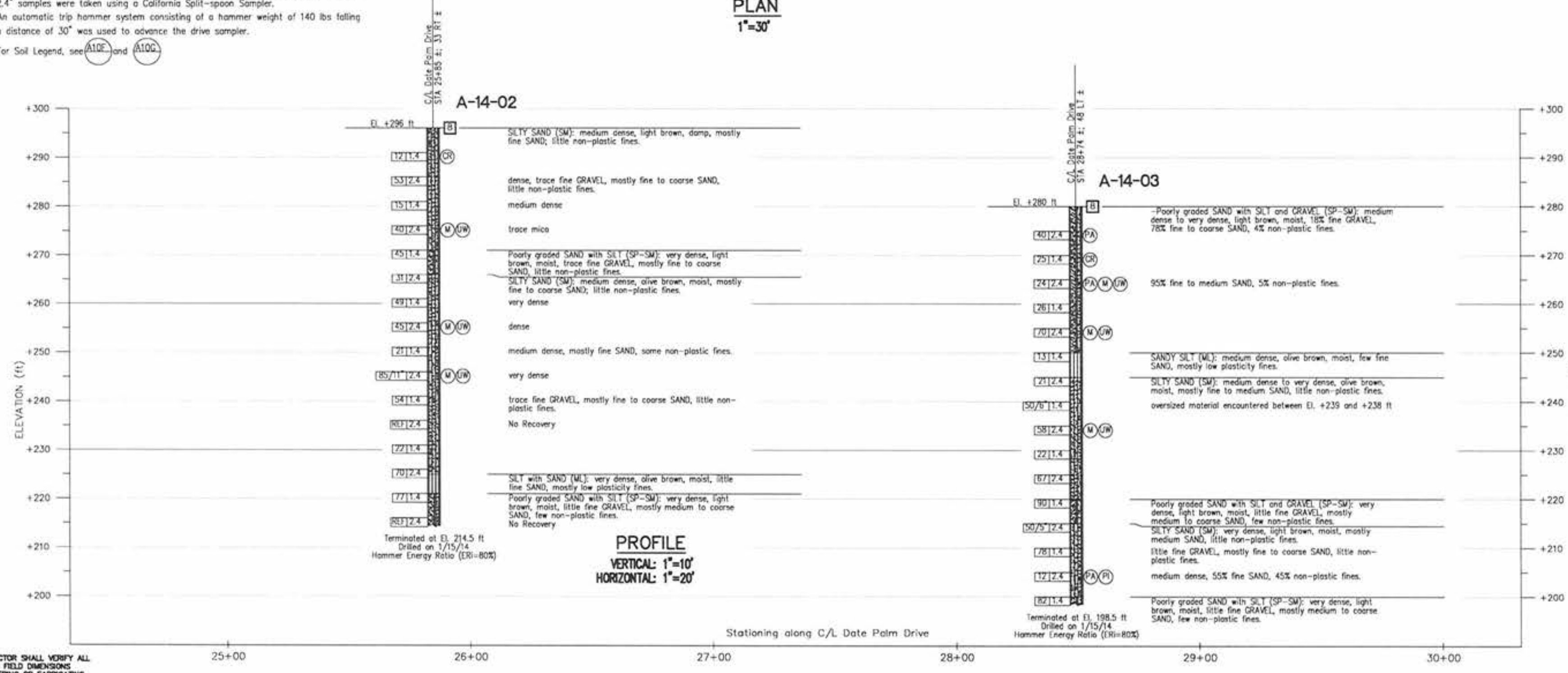
Depth (ft)	Sampler Type	Sample No.	Blows / ft	Graphic Log	GEOTECHNICAL DESCRIPTION	Moisture (%)	Total Unit Wt. (lb/ft^3)	Tests/ Results
0	B	0			AC (6") / AB (12") SAND with SILT (SP-SM): loose, brown, ,moist, mostly fine to coarse SAND, few non-plastic fines			R
5	S	1	7					
10	D	2	9					
					Boring Terminated at 11.5 ft Depth Groundwater was not encounted during drilling			
15								
20								
25								
30								
35								



PLAN
1"=30'

NOTES:

(1) This LOTB sheet (Boring Record) was prepared in accordance with the Caltrans Soil and Rock Classification Logging, Classification and Presentation Manual (June 2010).
(2) 2.4" samples were taken using a California Split-spoon Sampler.
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance the drive sampler.
(4) For Soil Legend, see A10F and A10G



PROFILE
VERTICAL: 1"=10'
HORIZONTAL: 1"=20'

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DIGALERT
Section 4216/4217 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert TOLL FREE
1-800-227-2600
Two working days before you dig
Underground Service Alert

NO.	BY	DATE	REVISION	DATE	APPR.

BENCH MARK: CATHEDRAL CITY BM 123 ELEV= 302.94
DESCRIPTION: CATHEDRAL CITY BRONZE DISK STAMPED "123 1999" IN CONCRETE BRIDGE ABUTMENT AT THE WESTERLY CORNER OF THE BRIDGE OVER WHITEWATER RIVER ON THE WEST SIDE OF DATE PALM DRIVE
PLAN DATUM NGVD 88
BASIS OF BEARING:

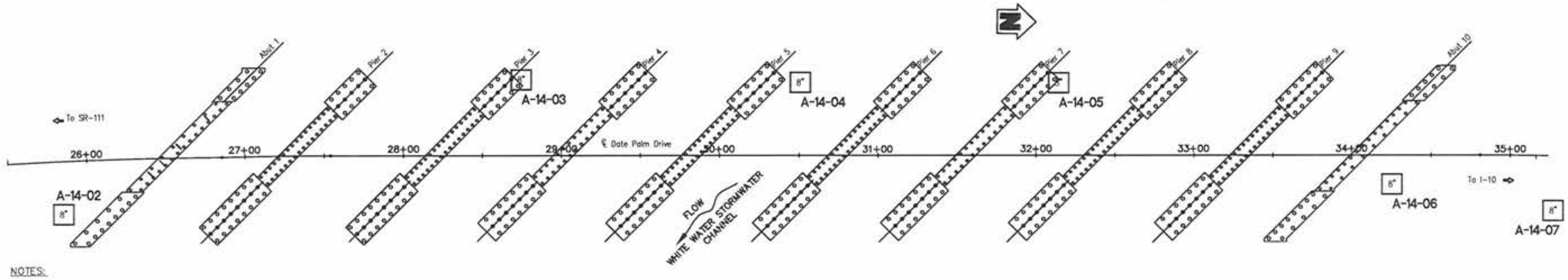
SCALE:	PLOT DATE:	DESIGNED BY:	DRAWN BY:	CHECKED BY:
AS SHOWN	04/25/14	E. BROWN	A. THURAIRAJAH	E. BROWN

APPROVED BY:
WILLIAM SIMONS, PE
CITY ENGINEER
RCE NO. — EXP.
DATE
PREPARED UNDER THE DIRECTION OF:
ERIC BROWN
RCE NO. 80246 — EXP. 6/30/2014
DATE

Cathedral City
68-700 Avenida Lalo Guerrero
Cathedral City, CA 92234
Ph. (760) 770-0349
FAX (760) 202-1460
Engineering@CathedralCity.gov

CITY OF CATHEDRAL CITY, CALIFORNIA
Street Improvement Plans
DATE PALM DRIVE BRIDGE (WIDEN) OVER THE WHITE WATER STORMWATER CHANNEL
FOR:

SHEET 32 OF 35 SHEETS



NOTES:

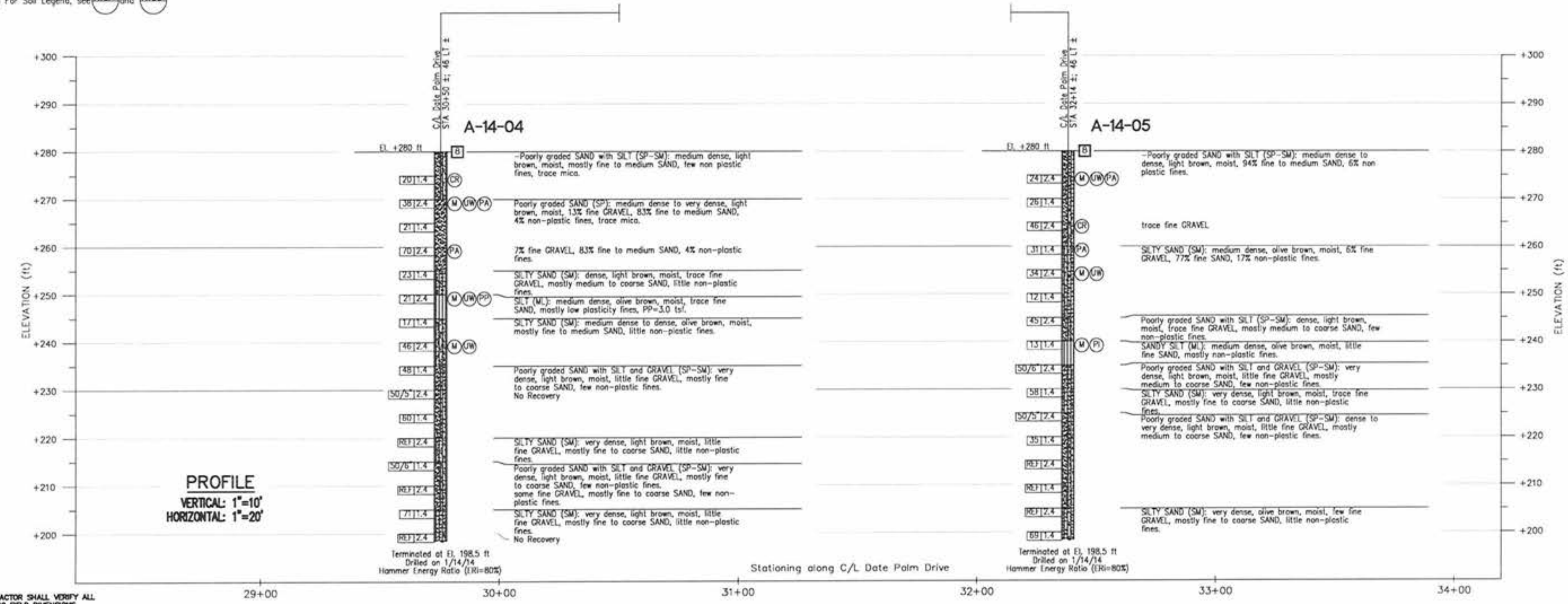
(1) This LOTB sheet (Boring Record) was prepared in accordance with the Caltrans Soil and Rock Classification Logging, Classification and Presentation Manual (June 2010).

(2) 2.4" samples were taken using a California Split-spoon Sampler.

(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance the drive sampler.

(4) For Soil Legend, see A10F and A10C.

PLAN
1"=30'



PROFILE
VERTICAL: 1"=10'
HORIZONTAL: 1"=20'

NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DIG ALERT
Underground Service Alert
Section 4216/4217 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert TOLL FREE
1-800-227-2600
Two working days before you dig

NO.	BY	DATE	REVISION	DATE	APPR.

BENCH MARK: CATHEDRAL CITY BM 123
DESCRIPTION: CATHEDRAL CITY BRONZE DISK STAMPED "123 1986" IN CONCRETE BRIDGE ABUTMENT AT THE WESTERLY CORNER OF THE BRIDGE OVER WHITEWATER RIVER ON THE WEST SIDE OF DATE PALM DRIVE
ELEV= 302.94
PLAN DATUM NGVD 88
BASIS OF BEARING:

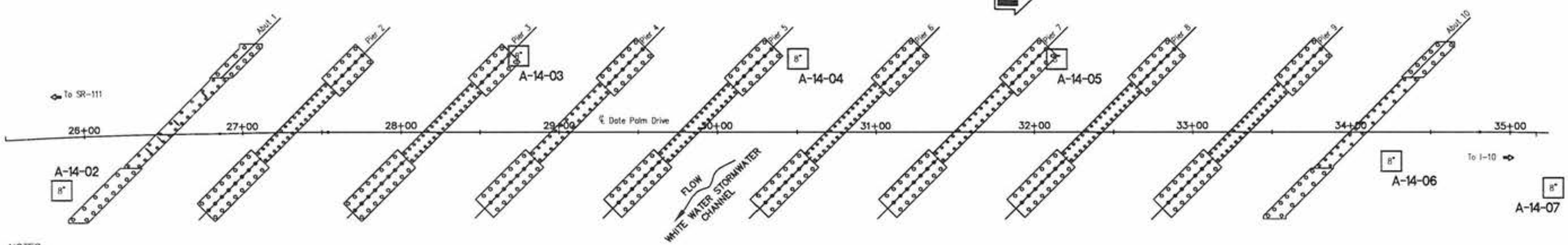
SCALE:	PLOT DATE:	DESIGNED BY:	DRAWN BY:	CHECKED BY:
AS SHOWN	04/25/14	E. BROWN	A. THURAIRAJAH	E. BROWN

APPROVED BY:
WILLIAM SIMONS, PE — DATE
CITY ENGINEER
RCE NO. — EXP.
PREPARED UNDER THE DIRECTION OF:
ERIC BROWN — DATE
RCE NO. 60346 — EXP. 6/30/2014

Cathedral City
68-700 Avenida Lalo Guerrero
Cathedral City, CA 92234
Ph. (760) 770-0349
FAX (760) 202-1460
Engineering@CathedralCity.gov

CITY OF CATHEDRAL CITY, CALIFORNIA
Street Improvement Plans
DATE PALM DRIVE BRIDGE (WIDEN) OVER THE WHITE WATER STORMWATER CHANNEL
FOR:

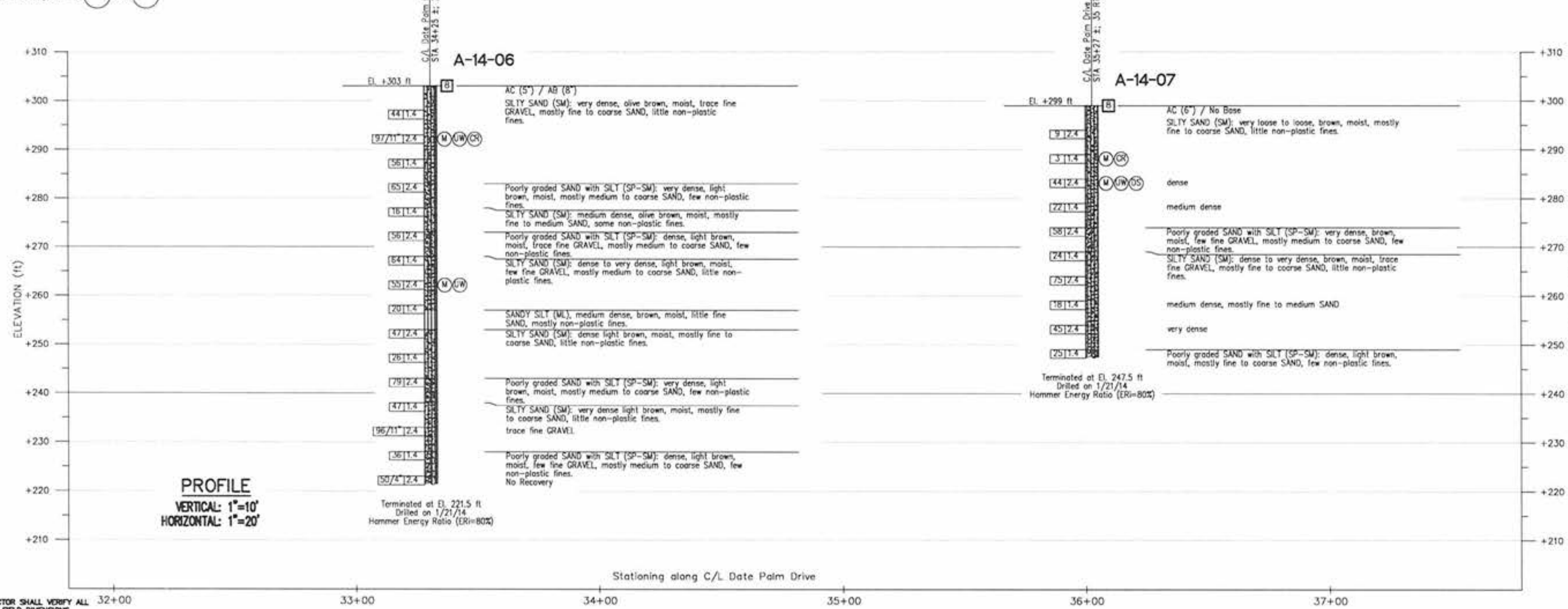
SHEET 33 OF 35 SHEETS



NOTES:

(1) This LOTB sheet (Boring Record) was prepared in accordance with the Caltrans Soil and Rock Classification Logging, Classification and Presentation Manual (June 2010).
(2) 2.4" samples were taken using a California Split-spoon Sampler.
(3) An automatic trip hammer system consisting of a hammer weight of 140 lbs falling a distance of 30" was used to advance the drive sampler.
(4) For Soil Legend, see A10F and A10G

PLAN
1"=30'



NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DIGALERT
Underground Service Alert
Section 4216/4217 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert TOLL FREE 1-800-227-2600
Two working days before you dig

NO.	BY	DATE	REVISION	DATE	APPR.

BENCH MARK: CATHEDRAL CITY BM 123
DESCRIPTION: CATHEDRAL CITY BRONZE DISK, STAMPED "123 1996" IN CONCRETE BRIDGE ABUTMENT AT THE WESTERLY CORNER OF THE BRIDGE OVER WHITEWATER RIVER ON THE WEST SIDE OF DATE PALM DRIVE. PLAN DATUM NGVD 88
ELEV= 302.94
BASIS OF BEARING:

SCALE:	PLOT DATE:	DESIGNED BY:	DRAWN BY:	CHECKED BY:
AS SHOWN	04/25/14	E. BROWN	A. THURAIRAJAH	E. BROWN

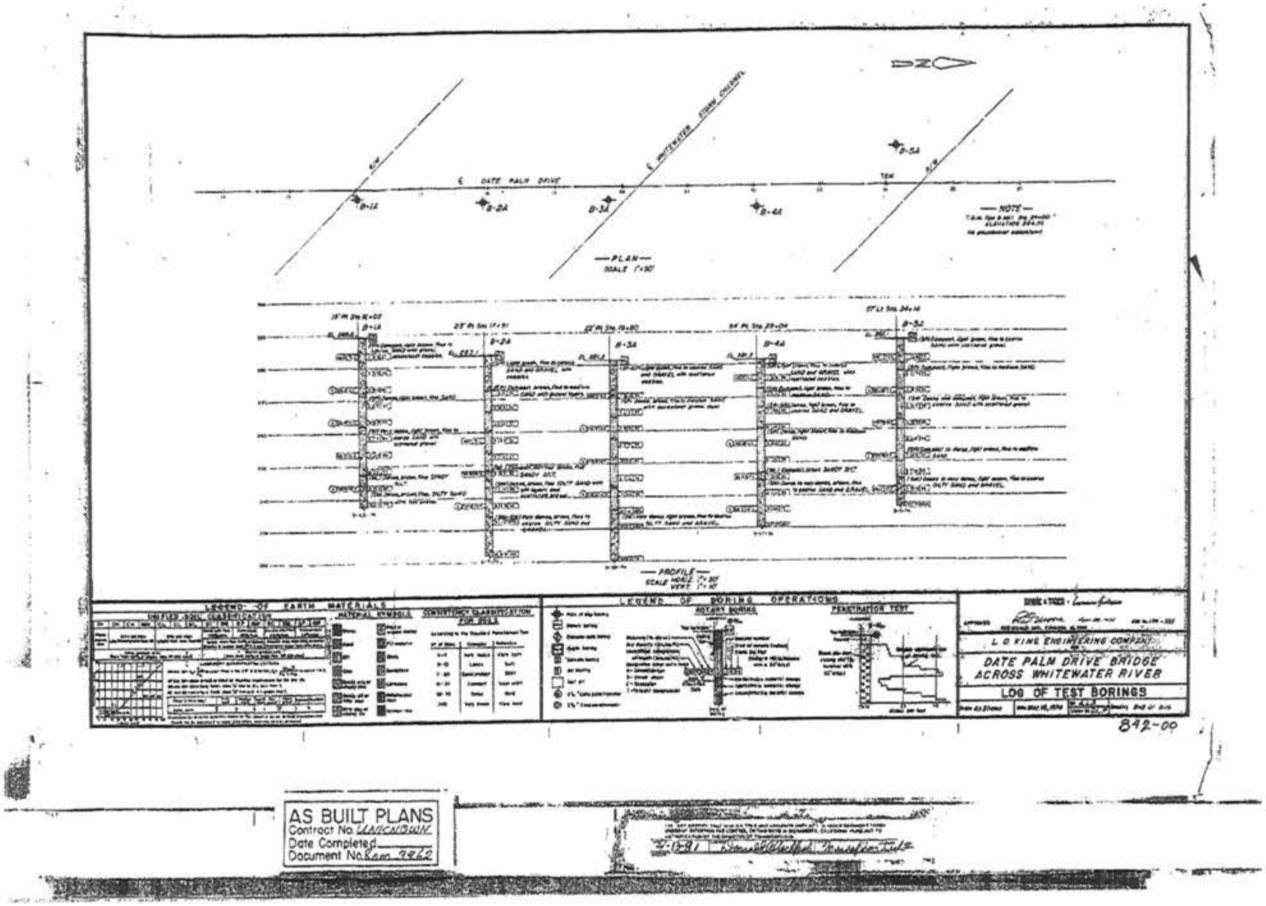
APPROVED BY:
WILLIAM SIMONS, PE
CITY ENGINEER
RCE NO. - EXP.
DATE

PREPARED UNDER THE DIRECTION OF:
ERIC BROWN
RCE NO. 60246 - EXP. 6/30/2014
DATE

Cathedral City
68-700 Avenida Lalo Guerrero
Cathedral City, CA 92234
Ph. (760) 770-0349
FAX (760) 202-1460
Engineering@CathedralCity.gov

CITY OF CATHEDRAL CITY, CALIFORNIA
Street Improvement Plans
DATE PALM DRIVE BRIDGE (WIDEN) OVER THE WHITE WATER STORMWATER CHANNEL
FOR:

SHEET 34 OF 35 SHEETS



NOTE:
THE CONTRACTOR SHALL VERIFY ALL CONTROLLING FIELD DIMENSIONS BEFORE ORDERING OR FABRICATING ANY MATERIAL.

DIGALERT

Section 4216/4217 of the Government Code requires a DIG ALERT Identification Number be issued before a "Permit to Excavate" will be valid. For your Dig Alert I.D. Number Call Underground Service Alert TOLL FREE
1-800-227-2600
Two working days before you dig

Underground Service Alert

NO.	BY	DATE	REVISION	DATE	APPR.

BENCH MARK: CATHEDRAL CITY BM 123
DESCRIPTION: CATHEDRAL CITY BRONZE DISK STAMPED "123 1986" IN CONCRETE BRIDGE ABUTMENT AT THE WESTERLY CORNER OF THE BRIDGE OVER WHITEWATER RIVER ON THE WEST SIDE OF DATE PALM DRIVE. PLAN DATUM NGVD 88
ELEV= 302.94

BASIS OF BEARING:

SCALE:	PLOT DATE:	DESIGNED BY:	DRAWN BY:	CHECKED BY:
AS SHOWN	04/25/14	E. BROWN	A. THURARAJAH	E. BROWN

APPROVED BY:
WILLIAM SIMONS, PE
CITY ENGINEER
RCE NO. — EXP. — DATE
PREPARED UNDER THE DIRECTION OF:
ERIC BROWN
RCE NO. 80248 — EXP. 6/30/2014 DATE

Cathedral City
68-700 Avenida Lalo Guerrero
Cathedral City, CA 92234
Ph. (760) 770-0349
FAX (760) 202-1460
Engineering@CathedralCity.gov

CITY OF CATHEDRAL CITY, CALIFORNIA
Street Improvement Plans
DATE PALM DRIVE BRIDGE (WIDEN) OVER THE WHITE WATER STORMWATER CHANNEL

FOR:

SHEET 35 OF 35 SHEETS

APPENDIX B

Boring Logs

APPENDIX B

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 12 to 18 inches with a 140-pound hammer free-falling 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 sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550-01. 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.

APPENDIX B

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 12 to 18 inches with a 140-pound hammer free-falling 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 sample barrel was driven into the ground with the weight of a hammer in general accordance with ASTM D 3550-01. 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.

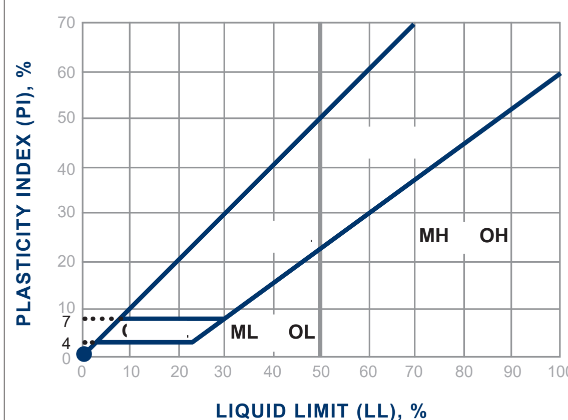
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions: Group Symbol	Secondary Divisions: Group Name
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	**GRAVEL** more than 50% of coarse fraction retained on No. 4 sieve	CLEAN GRAVEL less than 5% fines	GW	well-graded GRAVEL
			GP	poorly graded GRAVEL
		GRAVEL with DUAL CLASSIFICATIONS 5% to 12% fines	GW-GM	well-graded GRAVEL with silt
			GP-GM	poorly graded GRAVEL with silt
			GW-GC	well-graded GRAVEL with clay
			GP-GC	poorly graded GRAVEL with
		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

Consis tency	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	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.
							CL	Dashed line denotes material change.
15								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
								The total depth line is a solid line that is drawn at the bottom of the boring.
20								

DATE DRILLED	12/03/20	BORING NO.	B-1
GROUND ELEVATION	285' ± (MSL)	SHEET	1 OF 2
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY	SJQ	LOGGED BY SJQ	REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	ALLUVIUM: Light brown, dry, medium dense to dense, fine to coarse silty SAND; scattered gravel and cobbles.
			87/11"	1.4	110.2			Decrease in gravel.
10								
			50/4"	1.4	116.9			Increase in coarse sand and gravel.
			56	1.0				
20			62	0.8	120.8			Increase in moisture; coarse sand; gravel; occasional sub-angular gravel up to 1-inch diameter.
			61	1.2	103.7			Decrease in coarse sand; little to no gravel; mostly medium sand; dry.
30			82/11"	1.2	107.0			Very dense. Increase in coarse sand; increase in gravel; up to 1.5-inch diameter.
			86/9"	1.1				Increase in gravel size; scattered cobbles; 2-inch diameter cobble fragment in sampler.
40								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.

DATE DRILLED 12/03/20 BORING NO. B-1

GROUND ELEVATION 285' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

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

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY CAT

DESCRIPTION/INTERPRETATION

DEPTH (feet)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	U.S.C.S.	DESCRIPTION/INTERPRETATION
40	77	1.4	115.4	SM	ALLUVIUM: (Continued) Light brown, dry, very dense, fine to coarse silty SAND; scattered gravel and cobbles; fine to mostly coarse sand; increase in gravel up to 1-inch diameter.
	78/9"	1.6	111.4		
				GW	Light brown to light brownish gray, dry, very dense, well-graded sandy subangular GRAVEL; up to 2-inch diameter.
50	50/6"	2.3	111.3		
					Total Depth = 51.0 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 12/03/20. 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.
60					
70					
80					

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
								DATE DRILLED 12/04/20 BORING NO. B-2
								GROUND ELEVATION 285' ± (MSL) SHEET 1 OF 2
								METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)
								DRIVE WEIGHT 140 lbs. (Auto-Trip) DROP 30"
								SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY CAT
0							SP	ALLUVIUM: Light brown, dry, medium dense, poorly graded fine to coarse SAND with scattered gravel and occasional cobble.
			55	1.6	113.2		GW	Light brown, dry, dense, well-graded, fine to coarse sandy GRAVEL; angular to sub-angular, up to 1-inch diameter.
							SW	Light brown, dry, very dense, fine to predominantly medium well-graded SAND with scattered coarse sand; little to no gravel.
10			74/12"	1.6	109.9			
			68	2.0	121.5			Increase in moisture; increase in coarse sand and gravel; occasional 0.5-inch diameter gravel.
20			68/12"	6.2	104.2			Fine to coarse sand; with scattered fine gravel.
							CL-ML	Brown with gray laminae, wet, thinly laminated, very stiff, silty CLAY.
			23	29.9	90.1			
							SW	Light brown to light grayish brown, moist, very dense, well graded fine to coarse predominantly medium SAND with scattered gravel.
30			74	5.3	109.3			
							GW	Light grayish brown, moist, very dense, well-graded, fine to coarse sandy GRAVEL; occasional coarse sub-angular gravel up to 2-inch diameter; occasional cobbles.
			79	3.3	110.7			
							SW	Light brown to light grayish brown, moist, very dense, well graded fine to coarse gravelly
40								

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA
108980001 | 1/21

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			50/6"	2.6			SW	SAND; predominantly medium and coarse sand.
								ALLUVIUM: (Continued) Light brown to light grayish brown, moist, very dense, well-graded, fine to coarse gravelly SAND; predominantly medium and coarse sand; large 2.5-inch cobble fragments in sampler.
			50/3"				GW	Light grayish brown, moist to damp, dense to very dense, well-graded, fine to coarse sandy GRAVEL; predominantly medium and coarse sandy gravel; sub-rounded cobble; 2-inch diameter in sampler.
50			50/3" 50/6"					Increase in coarse gravel; increase in fine sand.
								Total Depth = 50.8 feet. Groundwater not encountered during drilling. Seepage encountered at approximately 25 feet during drilling. Backfilled shortly after drilling on 12/04/20.
								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.
60								
70								
80								

DATE DRILLED 12/04/20 BORING NO. B-2

GROUND ELEVATION 285' ± (MSL) SHEET 2 OF 2

METHOD OF DRILLING 8" Diameter Hollow Stem Auger (CME-75) (Baja Exploration)

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

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY CAT

DATE DRILLED 12/03/20 BORING NO. B-3

GROUND ELEVATION 285' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" Diameter Hand Auger

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY CAT

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SW	FILL: Light brown, dry, medium dense, well-graded, fine to coarse SAND with scattered gravel and occasional cobbles. Construction debris; brick fragment.
								Total Depth = 5 feet. Groundwater not encountered. Backfilled on 12/03/20. Notes: Groundwater, though not encountered at the time of excavation, 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								

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							SM	FILL: Light brown, dry, medium dense, well-graded, fine to coarse silty SAND with scattered gravel and occasional cobbles. Construction debris; brick fragment.
								Total Depth = 5 feet. Groundwater not encountered during drilling. Backfilled on 12/03/20. Notes: Groundwater, though not encountered at the time of excavation, 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 12/03/20 BORING NO. B-4

GROUND ELEVATION 295' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 4" Diameter Hand Auger

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY SJQ LOGGED BY SJQ REVIEWED BY CAT

FIGURE A- 8

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

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 2488-00. Soil classifications are indicated on the logs of the exploratory borings in Appendix B.

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 2937-04. The test results are presented on the logs of the exploratory borings in Appendix B.

Gradation Analysis
Gradation analysis tests were performed on selected representative soil samples in general accordance with ASTM D 422. The grain-size distribution curves are shown on Figures B-1 through B-3. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

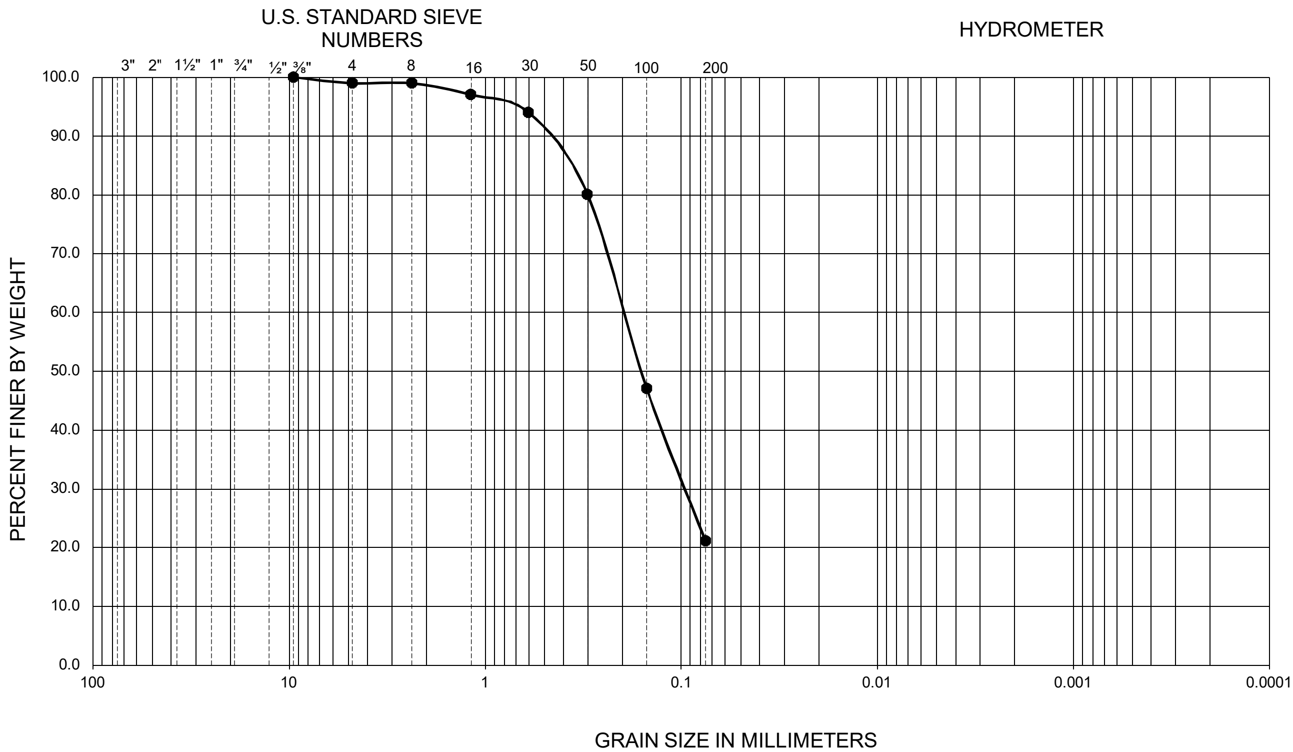
Consolidation
A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount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

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 B-5 and B-6.

Soil Corrosivity Tests
Soil pH, and minimum resistivity tests were performed on representative samples in general accordance with CT 643. The sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and 422, respectively. The test results are presented on Figure B-7.

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

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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	5.0-6.4	--	--	--	--	--	--	--	--	21	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

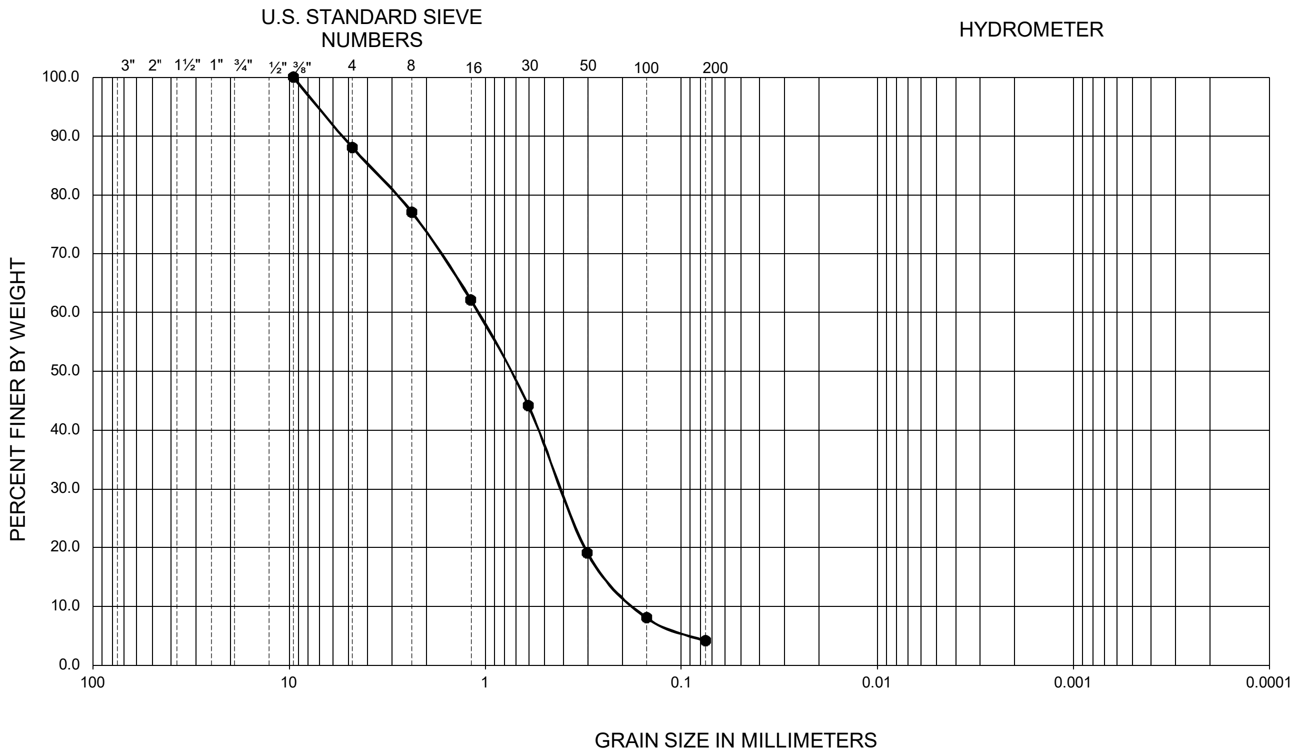
GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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-2	5.0-6.5	--	--	--	0.18	0.41	1.08	6.2	0.9	4	SP

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

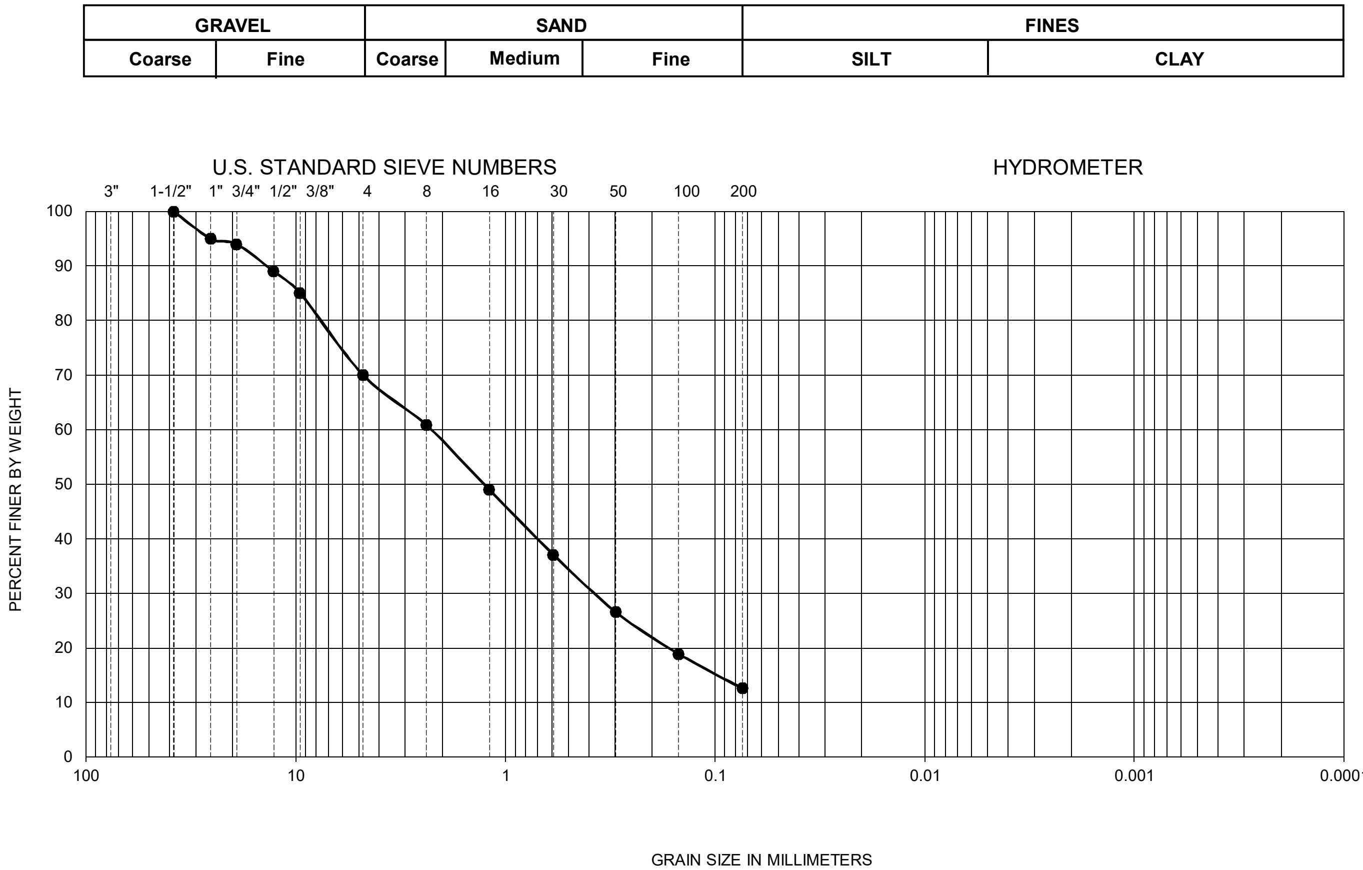
FIGURE B-2

GRADATION TEST RESULTS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

108980001 | 1/21



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-4	0.0-5.0	--	--	--	--	--	--	--	--	13	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

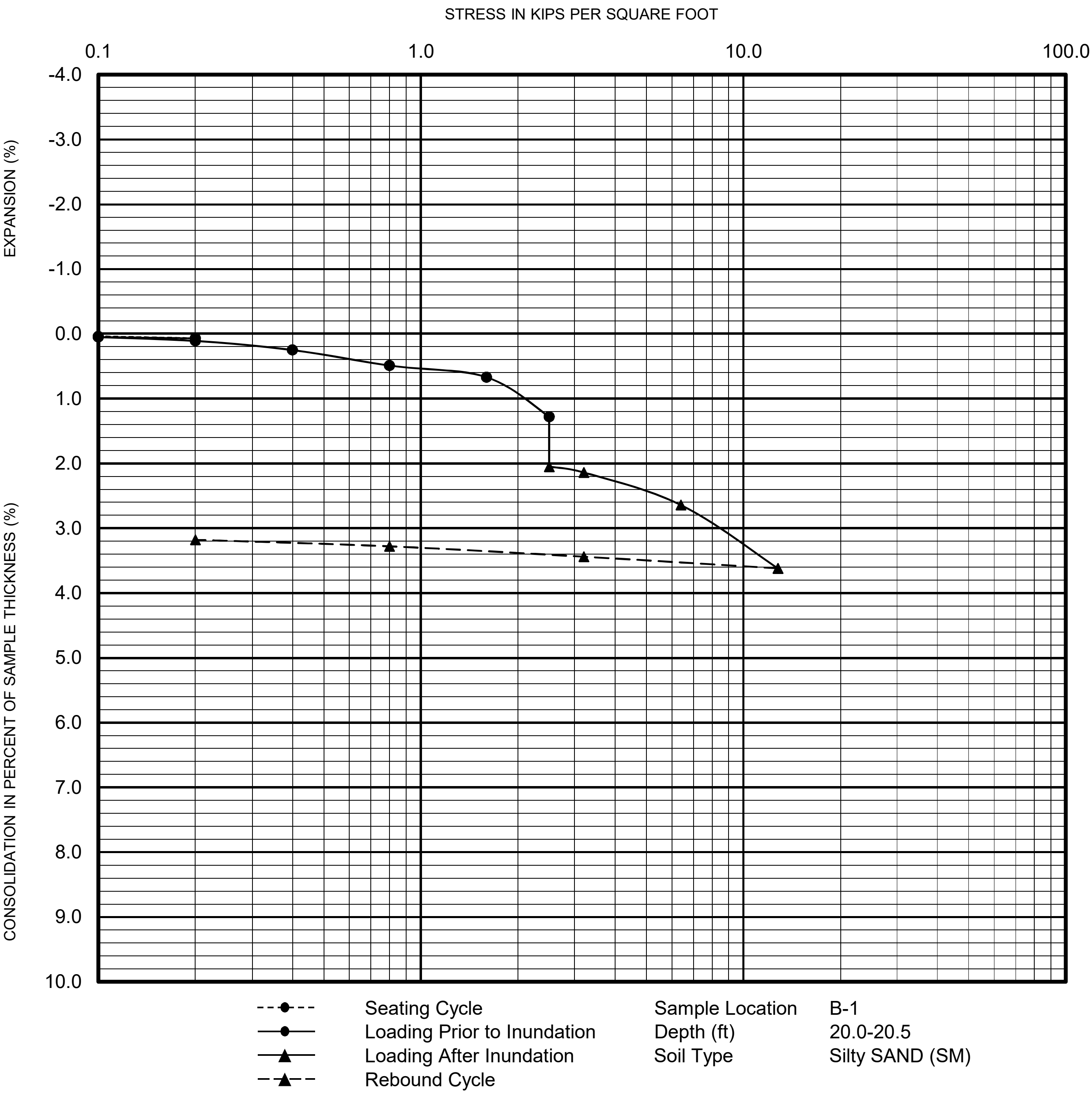
FIGURE B-3

GRADATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

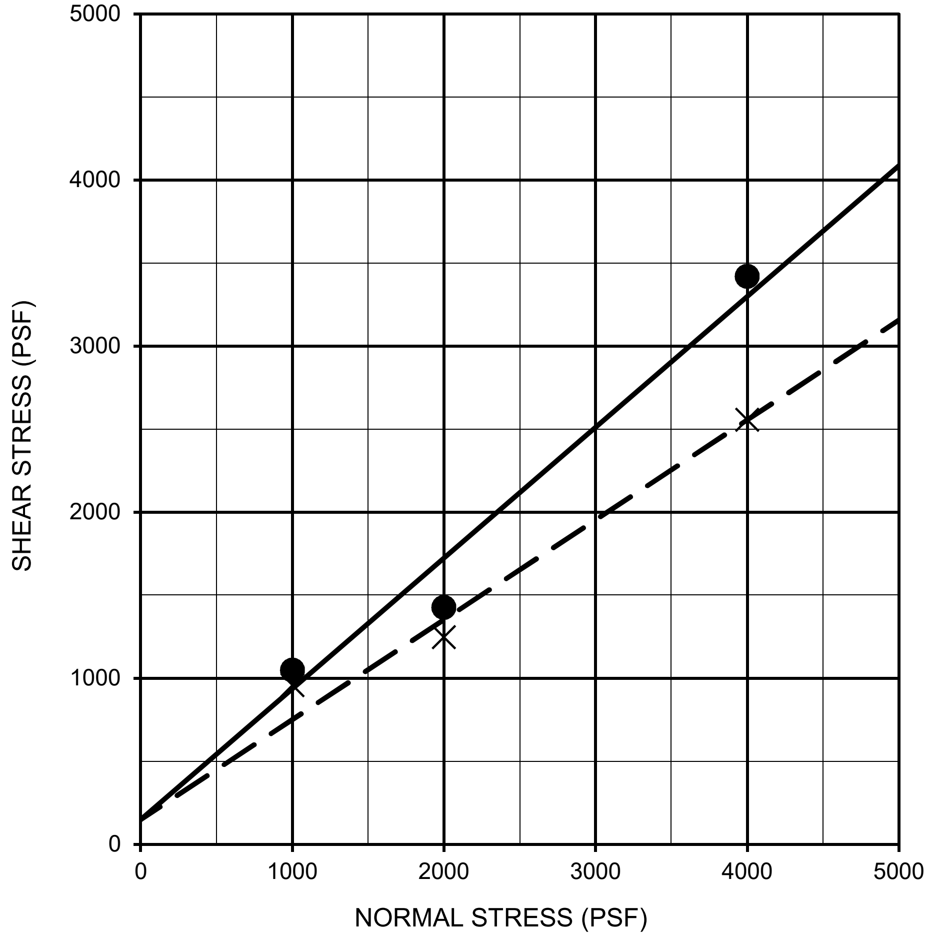
FIGURE B-4

CONSOLIDATION TEST RESULTS

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Silty SAND	—●—	B-1	12.5-13.3	Peak	150	38	SM
Silty SAND	- - X - -	B-1	12.5-13.3	Ultimate	150	31	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

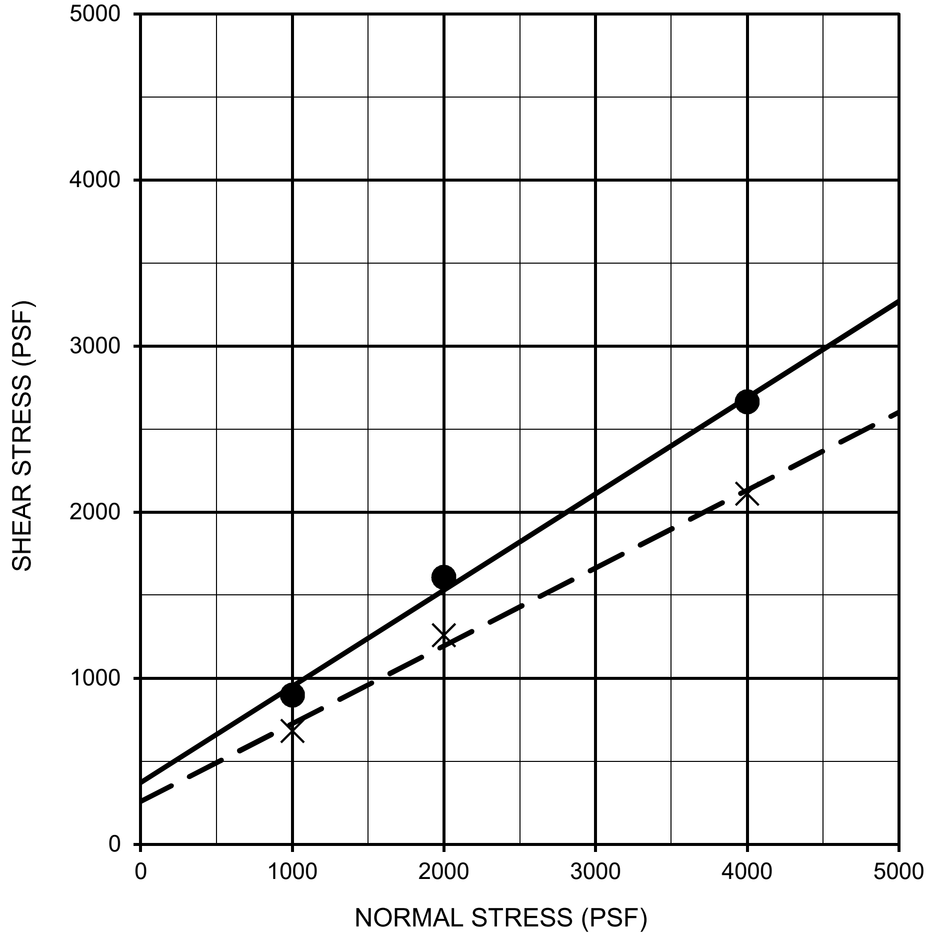
FIGURE B-5

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DIRECT SHEAR TEST RESULTS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Silty SAND	—●—	B-2	10.0-11.5	Peak	370	30	SM
Silty SAND	– – X – –	B-2	10.0-11.5	Ultimate	260	25	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-6

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

DIRECT SHEAR TEST RESULTS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm cm)	SULFATE CONTENT [2] (ppm)	SULFATE CONTENT [2] (%)	CHLORIDE CONTENT [3] (ppm)
B-2	0.0-5.0	7.3	6,500	10	0.001	10
B-3	0.0-5.0	7.7	7,900	10	0.001	55

[1] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 643

[2] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 417

[3] PERFORMED IN ACCORDANCE WITH CALIFORNIA TEST METHOD 422

FIGURE B-7

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

CORROSIVITY TEST RESULTS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R VALUE
B-1	0.0-5.0	Silty SAND (SM)	70
B-4	0.0-5.0	Silty SAND (SM)	72

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

FIGURE B-8

Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

R-VALUE TEST RESULTS

DATE PALM DRIVE BRIDGE AND ROADWAY WIDENING
CATHEDRAL CITY, CALIFORNIA

108980001 | 1/21

DRAFT

APPENDIX D

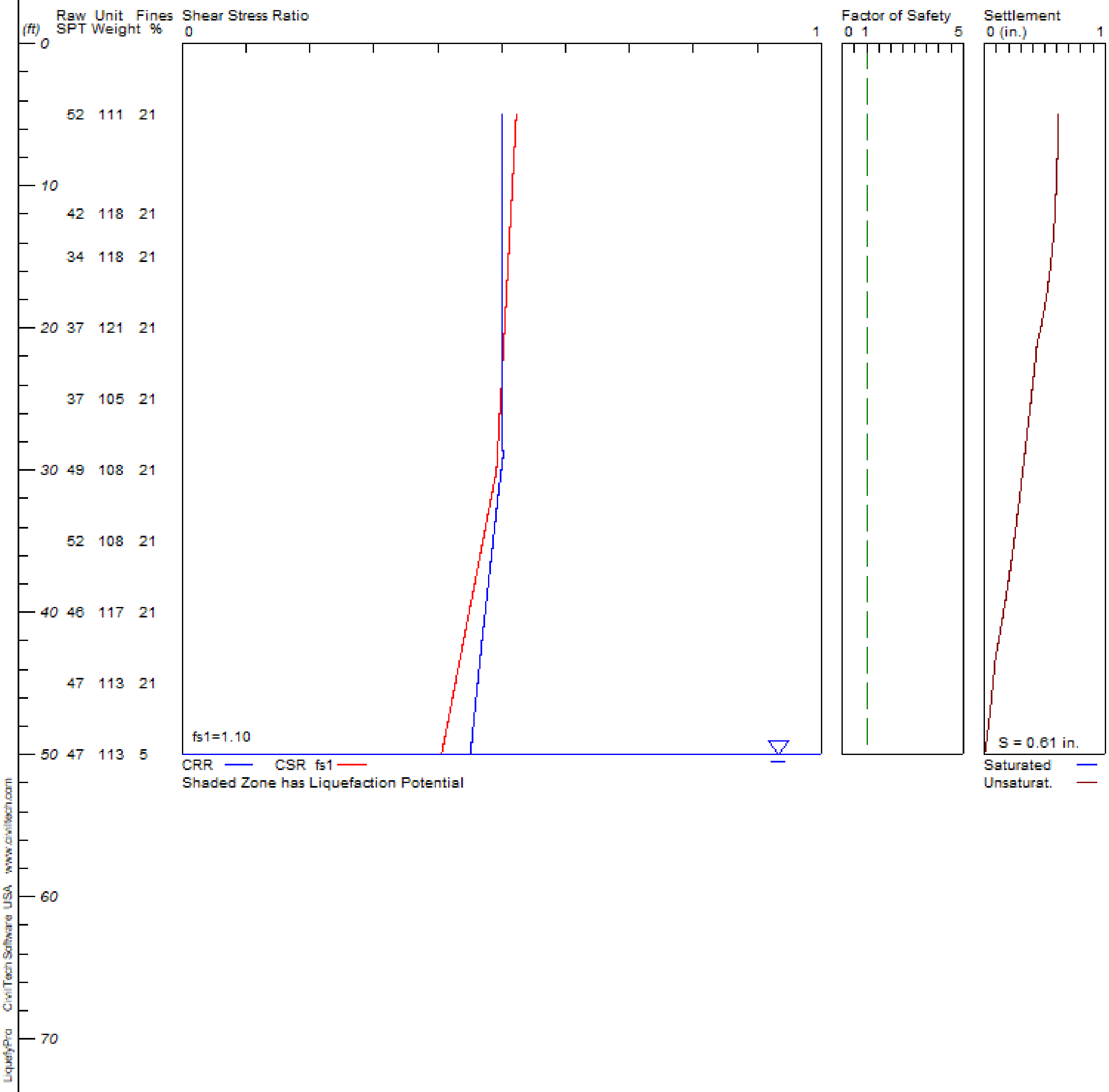
Seismic Settlement Evaluation

LIQUEFACTION ANALYSIS

Date Palm Drive Bridge and Roadway Widening

Hole No.=B-1   Water Depth=50 ft   Surface Elev.=285

Magnitude=7.5
Acceleration=0.74g

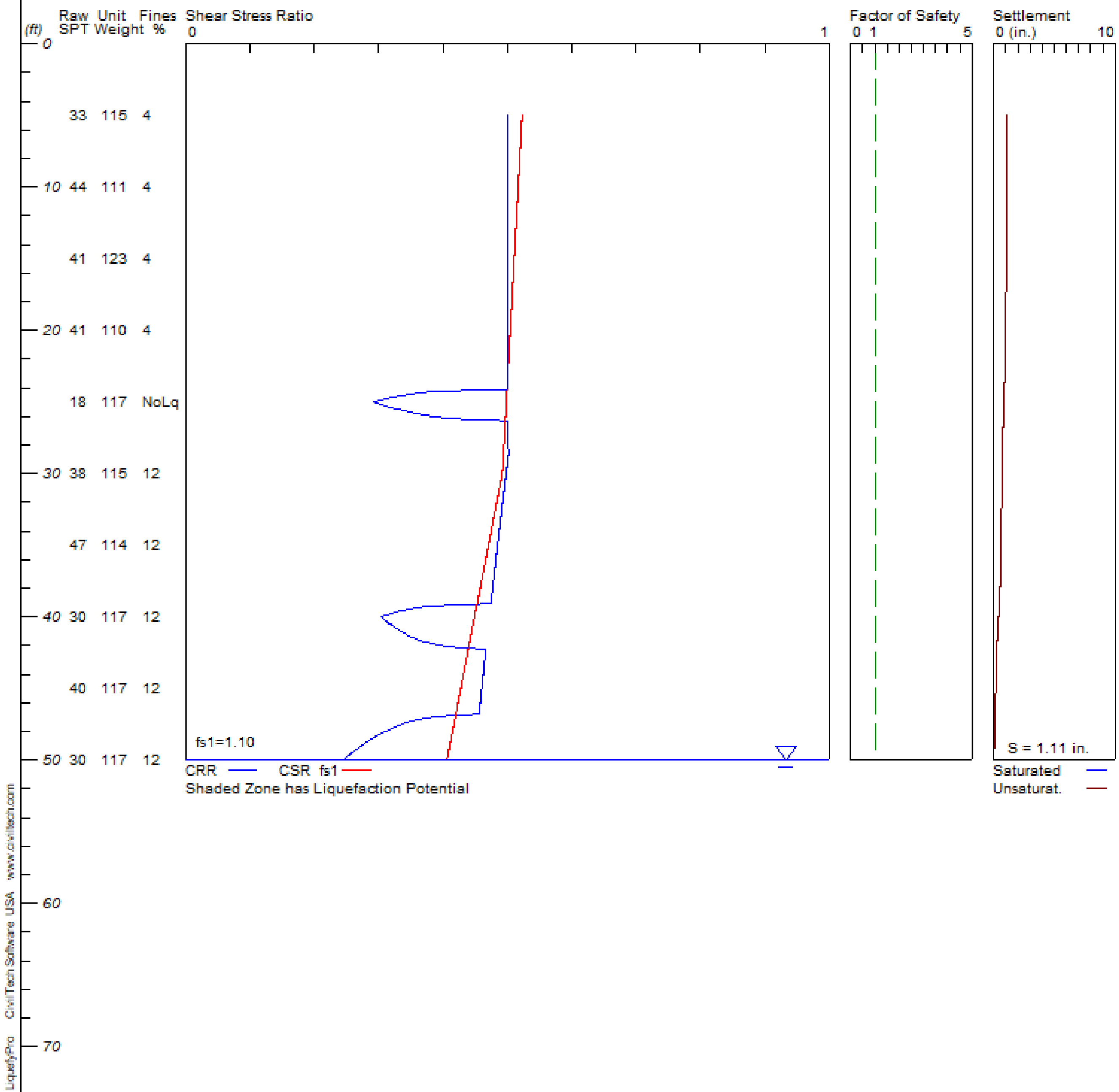


LIQUEFACTION ANALYSIS

Date Palm Drive Bridge and Roadway Widening

Hole No.=B-2   Water Depth=50 ft   Surface Elev.=285

Magnitude=7.5
Acceleration=0.74g



5710 Ruffin Road | San Diego, California 92123 | p. 858.576.1000

ARIZONA | CALIFORNIA | COLORADO | NEVADA | TEXAS | UTAH

ninyoandmoore.com

Ninyo & Moore
Geotechnical & Environmental Sciences Consultants