Geotechnical Evaluation

John F. Kennedy Neighborhood Park Improvements
4825 Ocean View Boulevard
San Diego, California

Estrada Land Planning

225 Broadway, Suite 1160 | San Diego, California 92101

August 3, 2018 | Project No. 108602001

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

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1 INTRODUCTION

In accordance with your request and authorization, we have performed a geotechnical evaluation for the proposed John F. Kennedy (JFK) Neighborhood Park Improvements project located 4825 Ocean View Boulevard in San Diego, California (Figure 1). The purpose of our geotechnical services was to evaluate the near-surface soil conditions at the site and provide recommendations for the design and construction of the proposed park development. This report presents our findings, conclusions, and recommendations based on our subsurface evaluation, laboratory testing, and geotechnical analyses.

2 SCOPE OF SERVICES

Our scope of services included the following:

- Reviewing background information including available geotechnical reports, geologic maps, groundwater data, and stereoscopic aerial photographs.
- Performing a site reconnaissance to observe geotechnical conditions and mark exploration locations for clearance by Underground Service Alert (USA).
- Performing a subsurface exploration consisting of the drilling, logging, and sampling of four small diameter exploratory borings. The soil borings were drilled to depths of up to approximately15.4 feet using a truck-mounted drill rig equipped with continuous-flight, hollow-stem augers and hand augering equipment. Logging of the borings was performed by a representative from Ninyo & Moore. Relatively undisturbed drive samples and bulk soil samples of the materials encountered were collected at selected intervals from the borings and transported to our in-house geotechnical laboratory for testing.
- Performing near-surface infiltration testing within two shallow borings.
- Performing geotechnical laboratory testing on selected samples to evaluate soil parameters for design purposes.
- Compiling and performing engineering analyses of the data obtained from our background review, subsurface exploration, and laboratory testing.
- Preparing this report presenting our findings, conclusions, and geotechnical recommendations relative to geologic hazards, seismic parameters, grading and earthwork, foundations, corrosive soils, and in-situ infiltration rates as they relate to the proposed development.

3 SITE AND PROJECT DESCRIPTION

The existing JFK Park is a rectangular-shaped parcel located at 4825 Ocean View Boulevard in San Diego, California (Figure 1). Existing improvements at the site include an open lawn area surrounded and transected by paved walkways, a restroom building, a playground, and picnic areas. The site is bounded to the north by Ocean View Boulevard, to the east by Gloria Street,

to the south by T Street, and to the west by the Porter Elementary School campus. The site coordinates are approximately 32.699974°N latitude and -117.091444°W longitude. Elevations at the site range from approximately 146 feet above mean sea level (MSL) on the northern edge of the site to approximately 122 feet MSL on the southern edge of the site. We understand that the proposed improvements at the site will include the demolition of the existing restroom building and construction of a new pre-engineered restroom building, new playground areas, BBQ units, tables, and fitness equipment.

4 SUBSURFACE EVALUATION AND LABORATORY TESTING

Our subsurface exploration was conducted on July 2, 2018. The subsurface exploration consisted of drilling, logging, and sampling of four small-diameter exploratory borings (B-1, B-2, B-3/IT-1, and B-4/IT-2). The borings were drilled to depths up to approximately 15.4 feet using a combination of a truck-mounted drill rig equipped with 8-inch diameter, hollow-stem augers and 6-inch diameter, hand augering equipment. Prior to commencing the subsurface exploration, USA was notified to mark out the existing utilities. The purpose of the borings was to evaluate subsurface conditions and to collect samples for geotechnical laboratory testing. The borings were logged and sampled by Ninyo & Moore personnel. Representative bulk and in-place soil samples were obtained from within 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 A.

4.1 Geotechnical Laboratory Testing

Geotechnical laboratory testing was performed on representative soil samples to evaluate the in-situ moisture content and dry density, sieve (gradation) analysis, shear strength, proctor density, and soil corrosivity. The results of the moisture content and dry density tests are presented on the exploration logs in Appendix A. The results of the other laboratory tests are presented in Appendix B.

5 INFILTRATION TESTING

On July 3, 2018, infiltration testing was performed at two locations designated IT-1 and IT-2 (Figure 2). Approximately 2 inches of gravel was placed on the bottom of each prepared test boring. A 2-inch diameter, perforated PVC pipe was installed in the test boring and the annulus was then backfilled with pea gravel. As part of the test procedure, a presoak was performed on July 2, 2018 to represent adverse conditions for infiltration. The presoak consisted of

maintaining approximately 2 foot of water in each test boring for approximately 4 hours. The water level was then allowed to drop overnight. Infiltration testing was then performed in the presoaked test borings. The test borings were filled with a minimum of 6 inches of water. Measurements of the water depth were generally recorded every 30 minutes for approximately 6 hours. The borings were refilled as needed to maintain the water level until the infiltration rate stabilized. For testing purposes, a stabilized rate was defined as two consecutive readings without a change of more than 0.01 foot in the depth of the water column.

The infiltration testing was conducted for landscape-related purposes only. Based on our field infiltration testing, percolation rates at the site were calculated to be 11.9 minutes per inch and 8.0 minutes per inch in infiltration tests IT-1 and IT-2, respectfully. These rates are not intended to be used in the design of permanent stormwater BMPs.

6 GEOLOGY

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

6.1 Regional Geologic Setting

The project area is situated in the western portion of the Peninsular Ranges Geomorphic Province. This geomorphic province encompasses an area that extends approximately 900 miles from the Transverse Ranges and the Los Angeles Basin south to the southern tip of Baja California (Norris and Webb, 1990; Harden, 2004). The province varies in width from approximately 30 to 100 miles and generally consists of rugged mountains underlain by Jurassic metavolcanic and metasedimentary rocks, and Cretaceous igneous rocks of the southern California batholith. The portion of the province in San Diego County that includes the project area consists generally of Pleistocene-age surficial deposits, underlain by Tertiary- and Cretaceous-age sedimentary rocks (Figure 3).

The Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults, which are shown on Figure 4, are considered active faults. The Elsinore, San Jacinto, and San Andreas faults are active fault systems located northeast of the project area and the Rose Canyon, Coronado Bank, San Diego Trough, and San Clemente faults are active faults located west of the project area. Major tectonic activity associated with these and other faults within this regional tectonic framework consists primarily of right-lateral, strike-slip movement. The Rose Canyon Fault, the nearest active fault, has been mapped approximately 4.2 miles west of the project site.

6.2 Site Geology

The site is mapped as being underlain by very old paralic deposits (Kennedy and Tan, 2008) as shown on Figure 3. Geologic units encountered during our subsurface exploration included fill soils underlain by very old paralic deposits. Generalized descriptions of the earth units encountered during our subsurface exploration and mapped at the site are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the boring logs in Appendix A. Geologic cross sections at the site are shown on Figure 5.

6.2.1 Fill

Fill soils were encountered at the surface in each of our borings. The fill generally extended to depths up to 6 feet. As encountered, the fill soils generally consisted of various shades of brown, moist, loose to medium dense, silty to clayey sand. Scattered gravel and cobbles were encountered within the fill materials. Landscape related grass and roots were encountered in the fill materials.

6.2.2 Very Old Paralic Deposits

Very old paralic deposits were encountered in borings B-1, B-2, and B-3/IT-1 beneath the fill materials and extended to the total depths explored. The very old paralic deposits were not encountered in boring B-4/IT-2. As encountered, the very old paralic deposits consisted of light brown and light reddish brown, moist, weakly to strongly cemented, silty sandstone. Scattered gravel and cobbles were encountered within the very old paralic deposits.

6.3 Groundwater

Groundwater was not encountered in our exploratory borings. Fluctuations in the groundwater level and perched conditions may occur due to variations in ground surface topography, subsurface geologic conditions and structure, rainfall, irrigation, and other factors. Additionally, due to relatively impermeable clay soil layers, perched water conditions may be encountered and should be anticipated at the site. Furthermore, the backfill and bedding materials tend to act as a conduit for water and perched water conditions may be present along the existing trench lines.

7 GEOLOGIC HAZARDS

In general, hazards associated with seismic activity include strong ground motion, ground surface rupture, and liquefaction. These considerations and other potential geologic hazards are addressed in the following sections.

7.1 Faulting and Seismicity

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, as well as on 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, 2007). 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 structures. Figure 4 shows the approximate site location relative to the major faults in the region. The nearest known active fault is the Rose Canyon fault, located approximately 4.2 miles west of the site.

Table 1 lists selected principal known active faults that may affect the subject site, the approximate fault to site distance, and the maximum moment magnitude (M_{max}) and the fault types provided by the United States Geological Survey (USGS) National Seismic Hazard Maps – Fault Parameters website (USGS, 2008). The locations and magnitudes of the faults were calculated from near the center of the project alignment at 32.699974°N latitude and -117.091444°W longitude.

Table 1 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (M_{max})
Rose Canyon	4.2 (6.8)	6.9
Coronado Bank	15.0 (24.2)	7.4
Newport-Inglewood (Offshore Segment)	37.1 (59.7)	7.0
Elsinore (Julian Segment)	39.6 (63.7)	7.4
Earthquake Valley	44.1 (71.0)	6.8
Elsinore (Temecula Segment)	44.4 (71.5)	7.1
Elsinore (Coyote Mountain Segment)	47.8 (76.8)	6.9
San Jacinto (Coyote Creek Segment)	60.6 (97.5)	7.0

7.1.1 Ground Motion

The 2016 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. The MCE_R ground motion response accelerations are based on the spectral response accelerations for 5 percent damping in the direction of maximum horizontal response and incorporate a target risk for structural collapse equivalent to 1 percent in 50 years with deterministic limits for near-source effects. The horizontal peak ground acceleration (PGA) that corresponds to the

MCE_R for the site was calculated as 0.442g using the United States Geological Survey (USGS, 2018) seismic design tool (web-based). Spectral response acceleration parameters, consistent with the 2016 CBC, are also provided in Section 9.2 for the evaluation of seismic loads on buildings and other structures.

The 2016 CBC specifies that the potential for liquefaction and soil strength loss be evaluated, where applicable, for the Maximum Considered Earthquake Geometric Mean (MCE_G) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCE_G peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCE_G peak ground acceleration with adjustment for site class effects (PGA_M) was calculated as 0.455g using the USGS (USGS, 2017) seismic design tool that yielded a mapped MCE_G peak ground acceleration of 0.423g for the site and a site coefficient (F_{PGA}) of 1.077 for Site Class D.

7.1.2 Surface Ground Rupture

As there are no known active faults crossing the subject site, the potential for ground rupture due to faulting is considered low. Surface ground cracking related to shaking from distant events is not considered a significant hazard, although it is a possibility.

7.1.3 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 50 feet below the ground surface. Factors known to influence liquefaction potential include composition and thickness of soil layers, grain size, relative density, groundwater level, degree of saturation, and both intensity and duration of ground shaking.

Based on the relatively dense nature of the very old paralic deposits and the absence of a shallow groundwater table, liquefaction and seismically induced settlement are not design considerations.

7.2 Landslides

Based on our background review and subsurface exploration, no landslides or related features underlie the project site. The site is in an area mapped as being generally susceptible to landsliding (Tan, 1995). The potential for significant large-scale slope instability at the site is not a design consideration.

7.3 Flood Hazards

Based on our review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM), the project site is located outside of the mapped 100-year flood zone. Therefore, flooding is not a design consideration for the project.

7.4 City of San Diego Seismic Safety Study

As part of our evaluation, we have reviewed the City of San Diego Seismic Safety Study (City of San Diego, 2008). The project site is located in Geologic Hazard Category 52, described as consisting of "Other level areas, gently sloping to steep terrain, favorable geologic structure, Low risk" (Figure 6).

8 CONCLUSIONS

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

- The site is underlain by fill soils and very old paralic deposits. Cobbles were encountered in the very old paralic deposits and will result in additional processing and handling prior to reuse. The on-site materials are generally considered suitable for reuse provided that they are processed in accordance with the recommendations herein. Processing of the on-site materials may include removal of oversize materials and moisture conditioning.

- Excavations are generally considered feasible with heavy-duty earthmoving equipment in good working order. However, very difficult drilling and/or excavating should be anticipated within the very old paralic deposits due to the scattered cobbles and potential for strongly cemented zones. The contractor should be prepared for these conditions.

- Groundwater was not encountered during our subsurface exploration. However, given the current site use, seepage and/or perched conditions should be anticipated. Additionally, the depth to groundwater varies due to seasonal precipitation, subsurface conditions, irrigation, groundwater pumping, and other factors.

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

- Field infiltration testing generally indicated percolation rates to be 11.9 minutes per inch and 8.0 minutes per inch in infiltration tests IT-1 and IT-2, respectfully. These rates are intended to be used for landscaping purposes and are not considered suitable for use in the design of permanent stormwater BMPs.

- Based on laboratory corrosion testing, the on-site soils are classified as corrosive in accordance with Caltrans (2018) guidelines and ACI 318.

9 RECOMMENDATIONS

The geotechnical recommendations relative to the design and construction of the proposed improvements are presented in the following paragraphs.

9.1 Earthwork

Earthwork at the site is anticipated to consist of minor grading. Earthwork should be performed in accordance with the requirements of applicable governing agencies, and the recommendations presented in the following sections.

9.1.1 Pre-Construction Conference

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

9.1.2 Site Preparation

Prior to performing site excavations, the surface areas should be cleared of existing improvements, vegetation, surface obstructions, and other deleterious materials. Construction materials, vegetation, and debris from the clearing operations should be disposed of off-site. Obstructions that extend below the finished grade, if any, should be removed and the resulting holes filled with compacted fill.

9.1.3 Excavation Characteristics

Based on our excavations, the on-site earth materials should be generally excavatable with heavy-duty earthmoving equipment in good working condition. However, the presence of cobbles and/or strongly cemented zones will result in the need for additional effort during excavation.

9.1.4 Remedial Grading

We understand that the existing restroom building will be demolished prior to construction of the new restroom building. Due to the undocumented nature of the existing near surface fill soils, and the potential for disturbance of the underlying soils during demolition, we recommend overexcavating the fill soils within the restroom pads. For the purposes of this report, the pad is defined as the structural footprint of the improvement (including foundations for attached overhangs, canopies, and other appurtenances) plus a horizontal distance of 5 feet, where feasible.

We recommend that the existing fill soils within the restroom pads be overexcavated to a depth of 2 feet below the bottom of the deepest proposed footing and replaced with fill material as described in this report. The low expansion index materials (EI <50) generated from the remedial grading may be reused onsite provided they meet the criteria for fill materials described in Section 9.1.5 of this report. The over excavation should extend to the horizontal limits of the improvement pads as previously defined. The extent and depths of removals and over excavations 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 6 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 over excavation should be backfilled with generally granular soils that possess a low-expansion potential (i.e., an expansion index [EI] of 50 or less).

9.1.5 Fill Material

In general, the existing on-site materials are considered suitable for reuse as fill, provided that the oversize materials (i.e., materials with dimensions in excess of those outlined herein) are removed from the soil mass prior to reuse. Fill material should be free of trash,

debris or other deleterious materials. Material for use as fill should not contain rocks or lumps greater than approximately 4 inches in size.

Utility trench backfill material should not contain rocks or lumps over approximately 3 inches in general. Soils classified as silts or clays should not be used for backfill in the pipe zone. Larger chunks, if generated during excavation, may be broken into acceptably sized pieces or disposed of offsite.

Imported fill material, if used, should generally be granular soils with a low expansion potential (i.e., an expansion index [EI] of 50 or less evaluated in accordance ASTM D 4829. Import material should also be non-corrosive in accordance with the Caltrans (2016) corrosion guidelines. Ninyo & Moore should evaluate materials for use as fill prior to filling or importing.

9.1.6 Fill Placement and Compaction

Prior to placement of compacted fill the contractor should request an evaluation of the exposed ground surface by Ninyo & Moore. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve generally consistent moisture contents above the laboratory optimum. The scarified materials should then be compacted to 90 percent relative compaction in accordance with ASTM D 1557. The evaluation of compaction by the geotechnical consultant should not be considered to preclude any requirements for observation or approval by governing agencies. It is the contractor's responsibility to notify the geotechnical consultant and the appropriate governing agency when project areas are ready for observation, and to provide reasonable time for that review.

Fill materials conforming to our recommendations for Materials for Fill, should be moisture conditioned to generally above the laboratory optimum moisture content prior to placement. The optimum moisture content will vary with material type and other factors. Moisture conditioning of fill soils should be generally consistent within the soil mass.

Prior to placement of additional compacted fill material following a delay in the grading operations, the exposed surface of previously compacted fill should be prepared to receive fill. Preparation may include scarification, moisture conditioning, and recompaction.

Compacted fill should be placed in horizontal lifts of approximately 8 inches in loose thickness. Prior to compaction, each lift should be watered or dried as needed to achieve generally above optimum moisture content, mixed, and then compacted by mechanical methods to a relative compaction of 90 percent as evaluated by ASTM D 1557. The upper 12 inches of subgrade for flatwork/sidewalks that will be subject to vehicle loading should be compacted by mechanical methods to 95 percent as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

9.2 Seismic Design Considerations

Design of the proposed improvements should be performed in accordance with the requirements of governing jurisdictions and applicable building codes. Table 2 presents the seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted MCE_R spectral response acceleration parameters (USGS, 2018).

Table 2 – 2016 California Building Code Seismic Design Criteria D

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.106
Site Coefficient, F_v	1.652
Mapped Spectral Acceleration at 0.2-second Period, S_s	0.984g
Mapped Spectral Acceleration at 1.0-second Period, S_1	0.374g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.089g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.618g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.726g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.412g

9.3 Foundations

The proposed structures may be supported on shallow, spread footings or cast-in-drilled-hole (CIDH) pile foundations in accordance with the recommendations presented in this report. Foundations should be designed in accordance with structural considerations and the following recommendations. In addition, requirements of the appropriate governing jurisdictions and applicable building codes should be considered in the design of the structures.

9.3.1 Shallow Foundations

The restroom building and playground equipment may be founded on shallow, spread or continuous footings bearing entirely on compacted fill. Shallow foundations may be designed using a net allowable bearing capacity of 3,000 pounds per square foot (psf). This allowable bearing capacity may be increased by one-third when considering loads of short duration such as wind or seismic forces. Shallow, spread or continuous footings should be founded 15 inches or more below the lowest adjacent grade and should have a width of 18 inches or more. The shallow, spread or continuous footings should be reinforced in accordance with the recommendations of the project structural engineer.

9.3.2 Lateral Resistance - Shallow Foundations

For resistance of footings to lateral loads, we recommend a 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 times the height generating the passive pressure, whichever is greater. 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 0.3 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance.

9.3.3 Light Pole Foundations

If light poles are planned as part of the park improvement, we recommend that the improvement foundations be supported on cast-in-drilled-holes (CIDH) pile foundations. Geotechnical recommendations for CIDH foundations are provided below. These types of structures typically impose relatively light axial loads on the foundations. We anticipate that the foundation dimensions will be generally controlled by the lateral load or uplift demand. The type, depth, and size of the foundations should be evaluated by the structural engineer based on the loading conditions and these geotechnical recommendations. We recommend that the foundation plans and design submittal be reviewed by this office for general conformance to these recommendations prior to finalizing.

Although we anticipate that pile dimensions will be generally controlled by the lateral load demand, we recommend that CIDH pile foundations have a diameter of 1.5 feet or more and extend into the formational soils.

For the analysis of the CIDH pile foundations using lateral resistance pressures, we recommend an allowable passive pressure of 300 pcf of depth be used with a value of up to 3,000 psf. For this case, an effective width of one times the diameter of the piles may be considered when evaluating lateral resistance. The recommended passive pressure assumes the piles are not adversely affected by approximately ½-inch of movement at the top of the pile. Furthermore, this value assumes that the ground is horizontal for a distance of 10 feet, or three times the height generating the passive pressure, whichever is greater. 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 0.2 be used between soil and concrete. The allowable lateral resistance can be taken as the sum of the frictional resistance and passive resistance provided the passive resistance does not exceed one-half of the total allowable resistance.

The lateral resistance values presented above may be increased by one-third when considering loads of short duration such as wind or seismic forces.

The drilled pile construction should be observed by the geotechnical consultant during construction to evaluate if the piles have been extended to the design depths. The drilled holes should be cleaned of loose soil and gravel. It is the contractor's responsibility to (a) take appropriate measures for maintaining the integrity of the drilled holes, (b) see that the holes are cleaned and straight, and (c) see that sloughed loose soil is removed from the bottom of the hole prior to the placement of concrete. Drilled piles should be checked for alignment and plumbness during installation. The amount of acceptable misalignment of a pile is approximately 3 inches from the plan location. It is usually acceptable for a pile to be out of plumb by 1 percent of the depth of the pile.

9.4 Static Settlement

We estimate that the proposed structures, designed and constructed with shallow foundations bearing on compacted fill soils as recommended herein, 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.

9.5 Concrete Flatwork

Conventional concrete flatwork should be 4 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 18 inches on-center both ways. To reduce the potential manifestation of distress to exterior concrete flatwork due to minor soil movement and concrete shrinkage, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer.

Concrete flatwork that will be subjected to vehicular loads should be 5½ inches in thickness and should be reinforced with No. 4 reinforcing bars placed at 18 inches on-center both ways. To reduce the potential manifestation of distress to exterior concrete flatwork due to minor soil movement and concrete shrinkage, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer.

9.6 Corrosivity

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

The results of the corrosivity testing indicated electrical resistivity of 410 ohm-cm, soil pH of 7.5, chloride content of 485 parts per million (ppm), and sulfate content of 0.019 percent (i.e., 190 ppm). Based on the laboratory test results and Caltrans (2018) corrosion criteria, the project site would be classified as corrosive. Based on the Caltrans (2018) corrosion criteria, a project site is classified as corrosive if one or more of the following conditions exist for the representative soil samples retrieved from the site: chloride concentration of 500 ppm or greater, soluble sulfate concentration of 1,500 ppm or greater, electrical resistivity of 1,100 ohm-centimeters or less, and a pH of 5.5 or less.

9.7 Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates can be subject to premature chemical and/or physical deterioration. As noted, the soil sample tested in this evaluation indicated a water-soluble sulfate content of 0.019 percent by weight (i.e., 190 ppm). Based on the ACI 318 criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soils ranging from about 0.00 to 0.10 percent by weight. Therefore, the site soils may be considered to have a negligible potential for sulfate attack.

However, due to the potential variability of site soils, potential use of reclaimed water and landscaping soil amendments and fertilizers, consideration should be given to using Type II/V cement for normal weight concrete in contact with soil.

9.8 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 improvements, pedestrian walkways and slopes. 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.

10 PLAN REVIEW AND CONSTRUCTION OBSERVATION

The conclusions and recommendations presented in this report are based on analysis of observed conditions in widely spaced exploratory excavations. 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.

11 LIMITATIONS

The field evaluation, laboratory testing, and geotechnical analyses presented in this geotechnical report have been conducted in general accordance with current practice and the standard of care exercised by geotechnical consultants performing similar tasks in the project area. No warranty, expressed or implied, is made regarding the conclusions, recommendations, and opinions presented in this report. There is no evaluation detailed enough to reveal every

subsurface condition. Variations may exist and conditions not observed or described in this report may be encountered during construction. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface evaluation will be performed upon request.

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

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

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

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

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FIGURES

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2018

N

FEET

0 1,500 3,000

1_108602001_SL.mxd 7/13/2018 JDL

FIGURE 1

SITE LOCATION

JFK NEIGHBORHOOD PARK IMPROVEMENTS, 4825 OCEAN VIEW BOULEVARD
SAN DIEGO, CALIFORNIA

108602001 | 8/18

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LEGEND

B-2 TD=16.5 — BORING TD=TOTAL DEPTH IN FEET	**Af** ARTIFICIAL FILL
B-4/IT-2 TD=3.0 — BORING/INFILTRATION TEST TD=TOTAL DEPTH IN FEET	**Qvop** VERY OLD PARALIC DEPOSITS
B B' CROSS SECTION	

N

FEET

0 100 200

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCES: AERIAL TOPO SURVEY - SAN-LO AERIAL SURVEYS, INC., dATED: 6-4-20108; GOOGLE EARTH, 2018

2_108602001_BL.mxd 8/2/2018 JDL

FIGURE 2

GEOTECHNICAL MAP

JFK NEIGHBORHOOD PARK IMPROVEMENTS, 4825 OCEAN VIEW BOULEVARD
SAN DIEGO, CALIFORNIA

108602001 | 8/18

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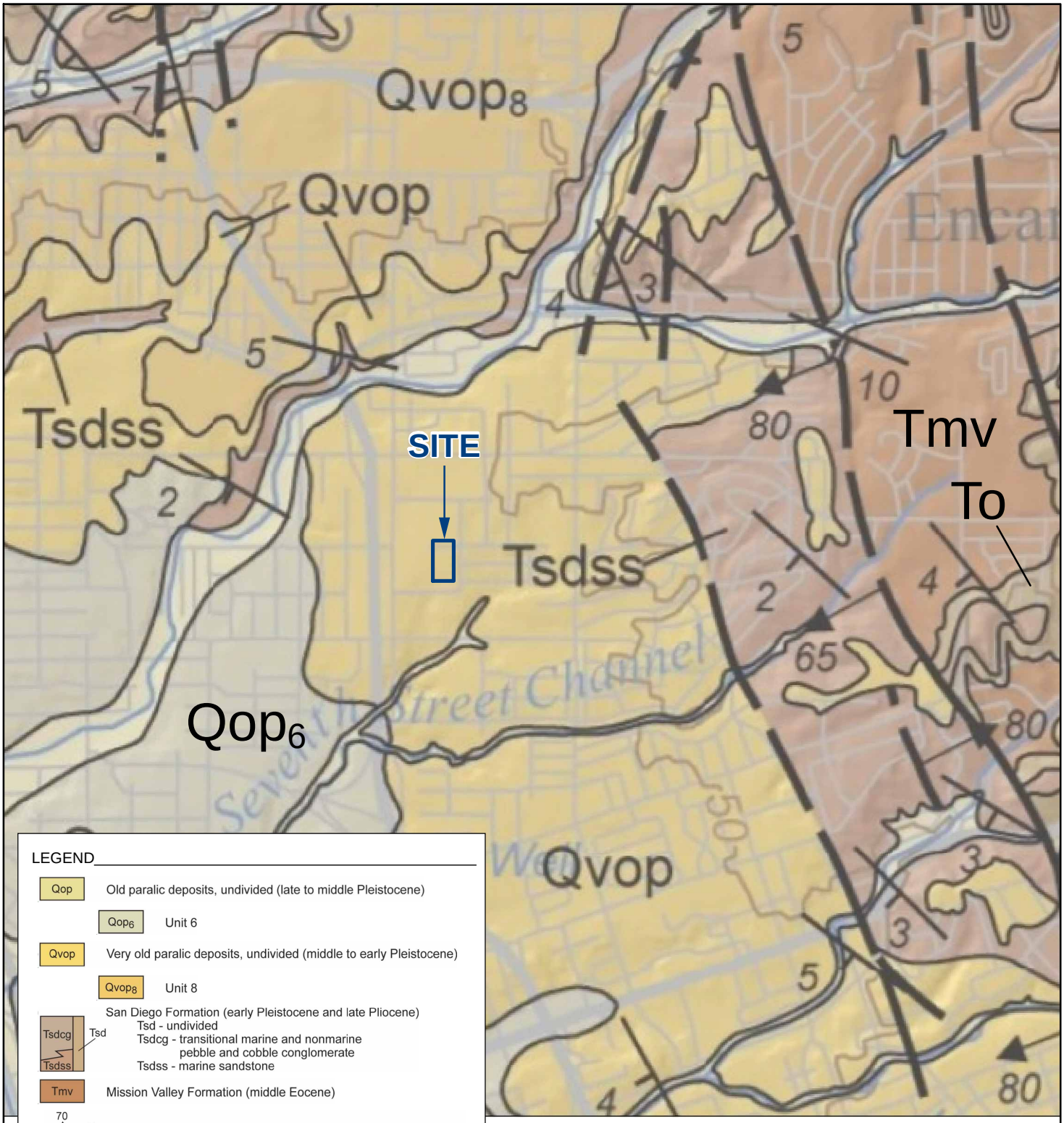

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCES: KENNEDY, M.P., AND TAN, S.S., 2008, GEOLOGIC MAP OF THE SAN DIEGO 30' X 60' QUADRANGLE, CALIFORNIA

3_108602001_G.mxd 7/13/2018 JDL

FIGURE 3

GEOLOGY

JFK NEIGHBORHOOD PARK IMPROVEMENTS, 4825 OCEAN VIEW BOULEVARD
SAN DIEGO, CALIFORNIA

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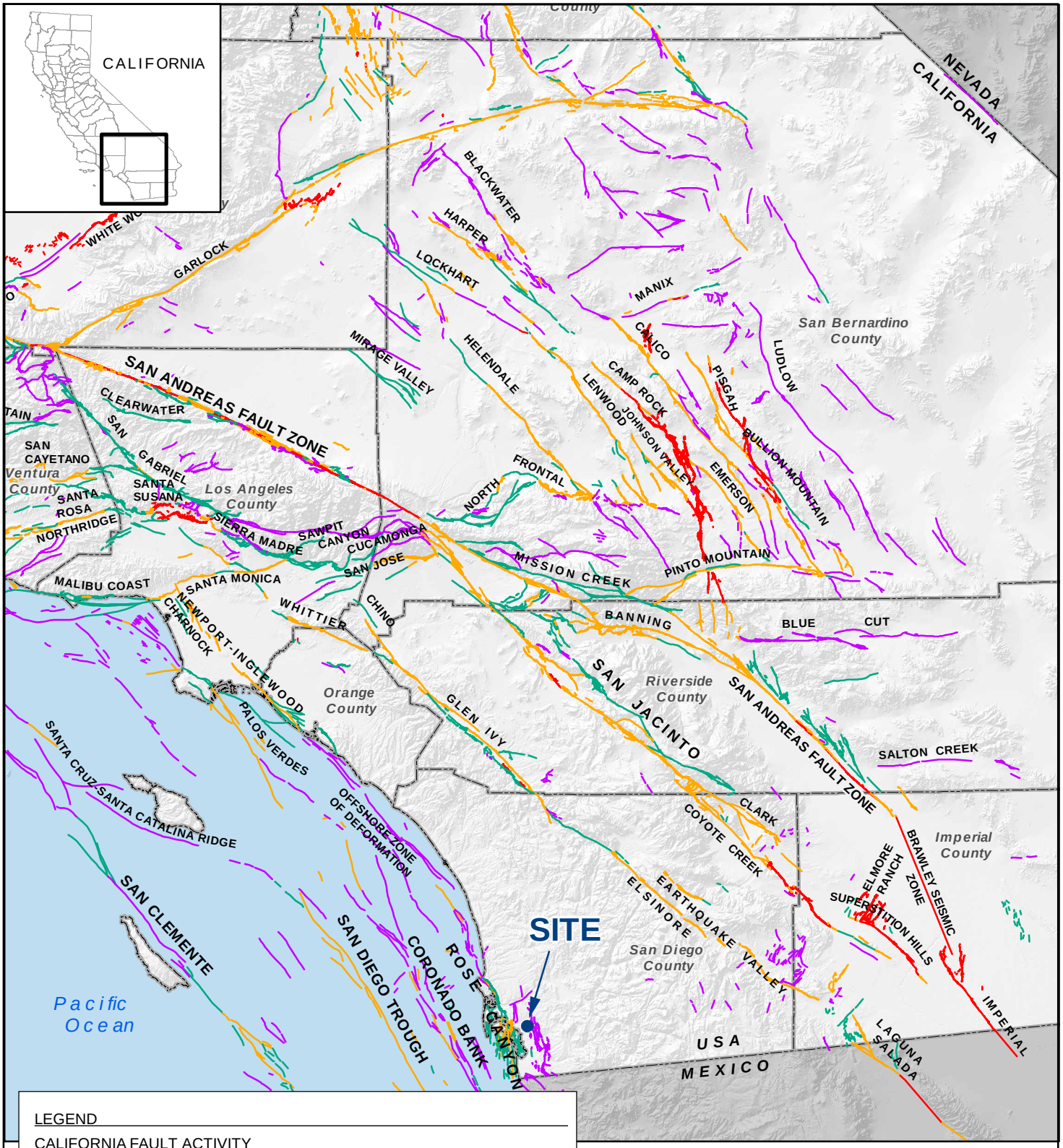

LEGEND

CALIFORNIA FAULT ACTIVITY

- HISTORICALLY ACTIVE
- HOLOCENE ACTIVE
- LATE QUATERNARY (POTENTIALLY ACTIVE)
- QUATERNARY (POTENTIALLY ACTIVE)
- STATE/COUNTY BOUNDARY

N

MILES 0 30 60

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2018

4_108602001_FL.mxd 8/2/2018 JDL

FIGURE 4

FAULT LOCATIONS

JFK NEIGHBORHOOD PARK IMPROVEMENTS, 4825 OCEAN VIEW BOULEVARD
SAN DIEGO, CALIFORNIA

108602001 | 8/18

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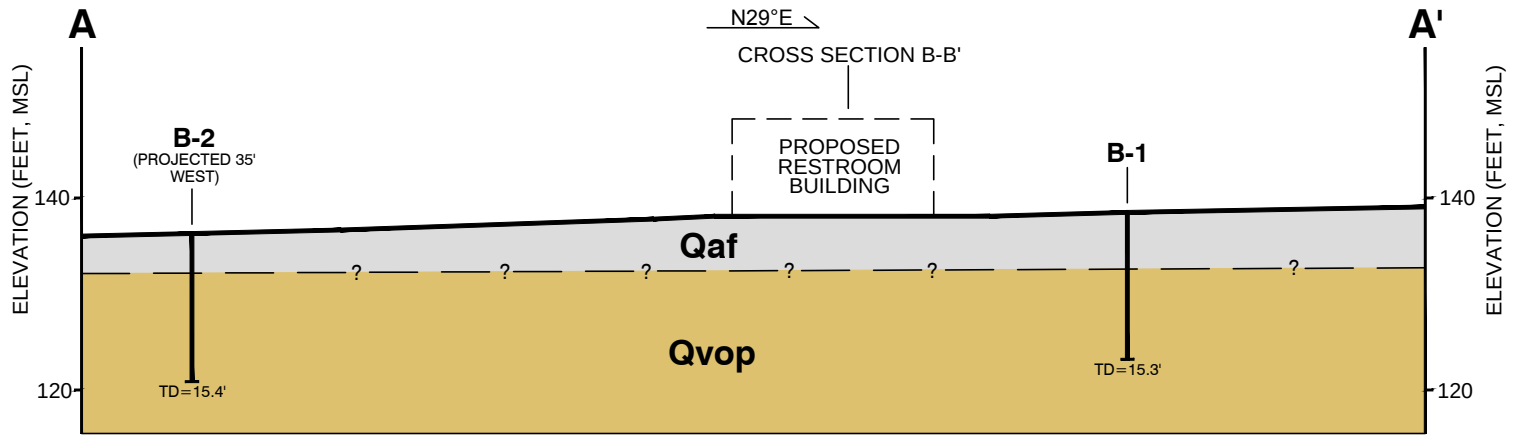

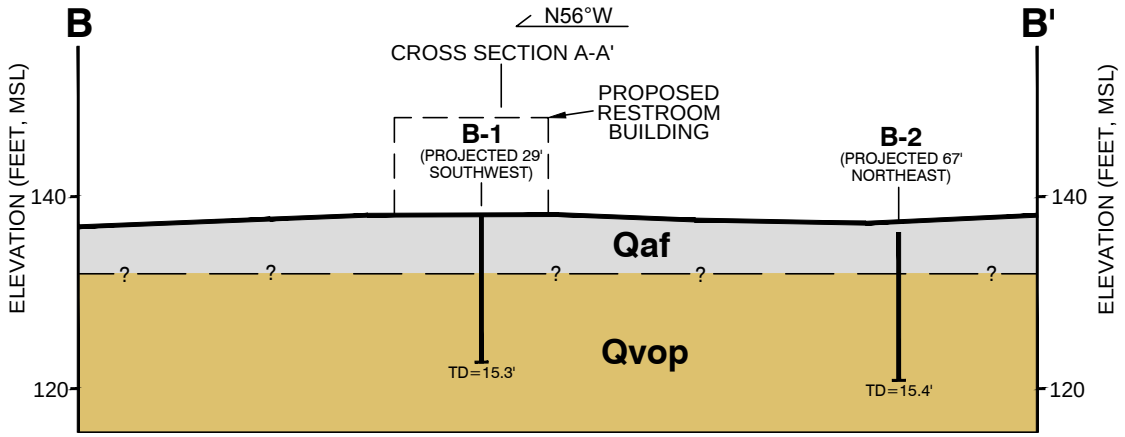

LEGEND

B-2 (TD=15.4')	BORING TD=TOTAL DEPTH IN FEET
Qaf	FILL
Qvop	VERY OLD PARALIC DEPOSITS
—— ? ——	GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FEET
0 20 40

5 108602001 CS A-A' B-B'.DWG

FIGURE 5

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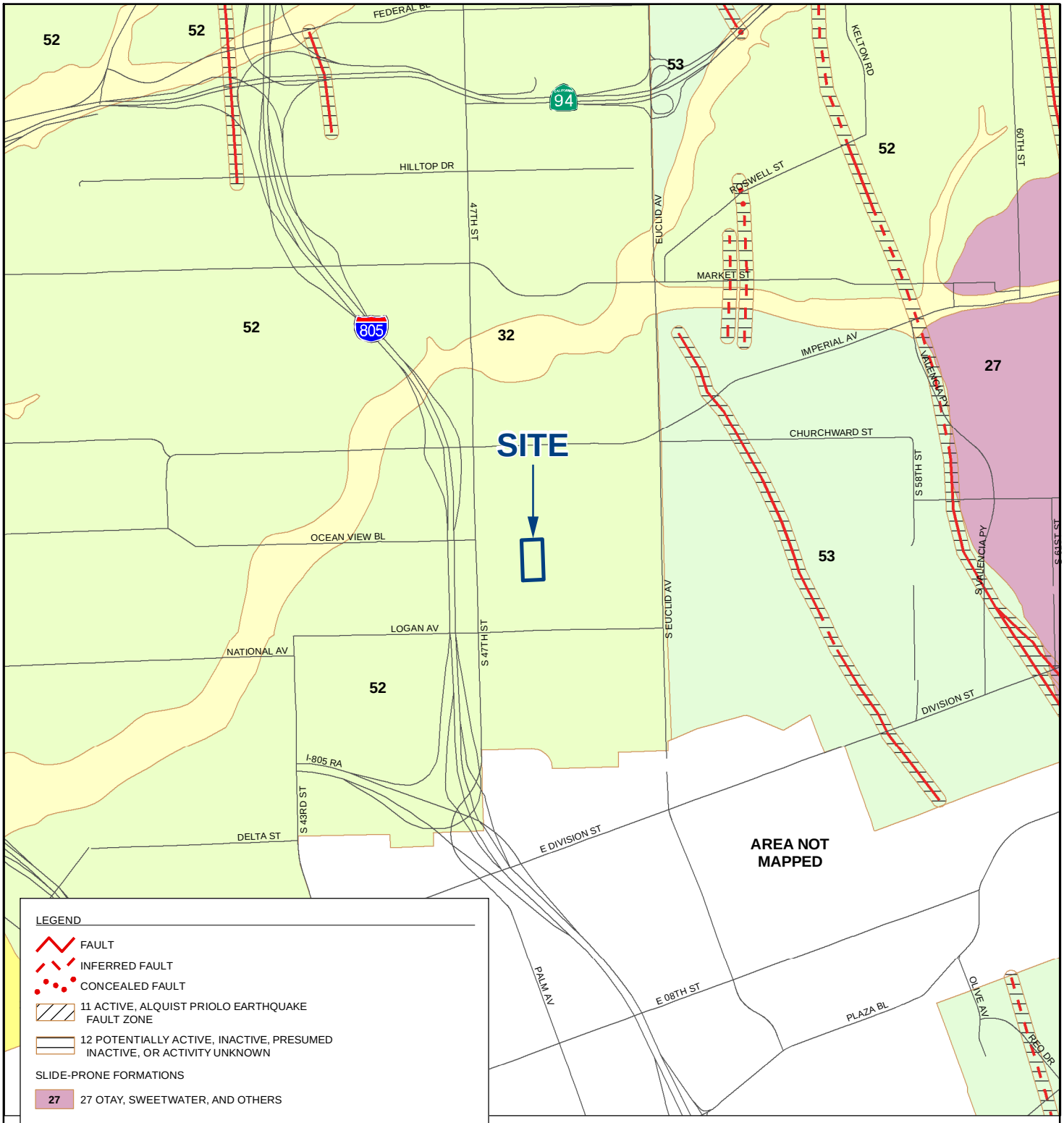
GEOLOGIC CROSS SECTION A-A' AND B-B'

JFK NEIGHBORHOOD PARK IMPROVEMENTS
4825 OCEAN VIEW BOULEVARD, SAN DIEGO, CALIFORNIA

108602001 I 7/18



NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: SANGIS, 2008, CITY OF SAN DIEGO SEISMIC SAFETY STUDY GEOLOGIC HAZARDS AND FAULTS.

6_108602001_GH.mxd 8/2/2018 JDL

FIGURE 6

GEOLOGIC HAZARDS

JFK NEIGHBORHOOD PARK IMPROVEMENTS, 4825 OCEAN VIEW BOULEVARD
SAN DIEGO, CALIFORNIA

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APPENDIX A

Boring Logs

APPENDIX A

BORING LOGS

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

Bulk Samples
Bulk samples of representative earth materials were obtained from the exploratory test pits. The samples were bagged 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
Relatively undisturbed soil samples were obtained in the field using the Modified Split-Barrel Drive Sampler. The sampler, with an external diameter of 3.0 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 140-pound hammer, in general accordance with ASTM D 3550. The driving weight was permitted to fall freely. The approximate length of the fall, the weight of the hammer, and the number of blows per foot of driving are presented on the boring logs as an index to the relative resistance of the materials sampled. The samples were removed from the sample barrel in the brass rings, sealed, and transported to the laboratory for testing.

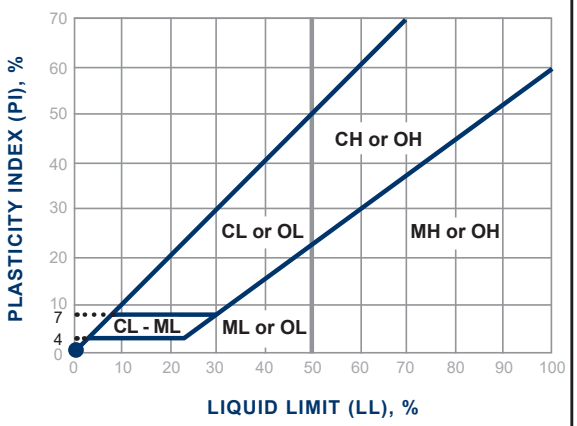
Soil Classification Chart Per ASTM D 2488

Primary Divisions			Secondary Divisions	
			Group Symbol	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		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	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		Automatic Trip Hammer	
	SPT (blows/foot)	Modified Split Barrel (blows/foot)	SPT (blows/foot)	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

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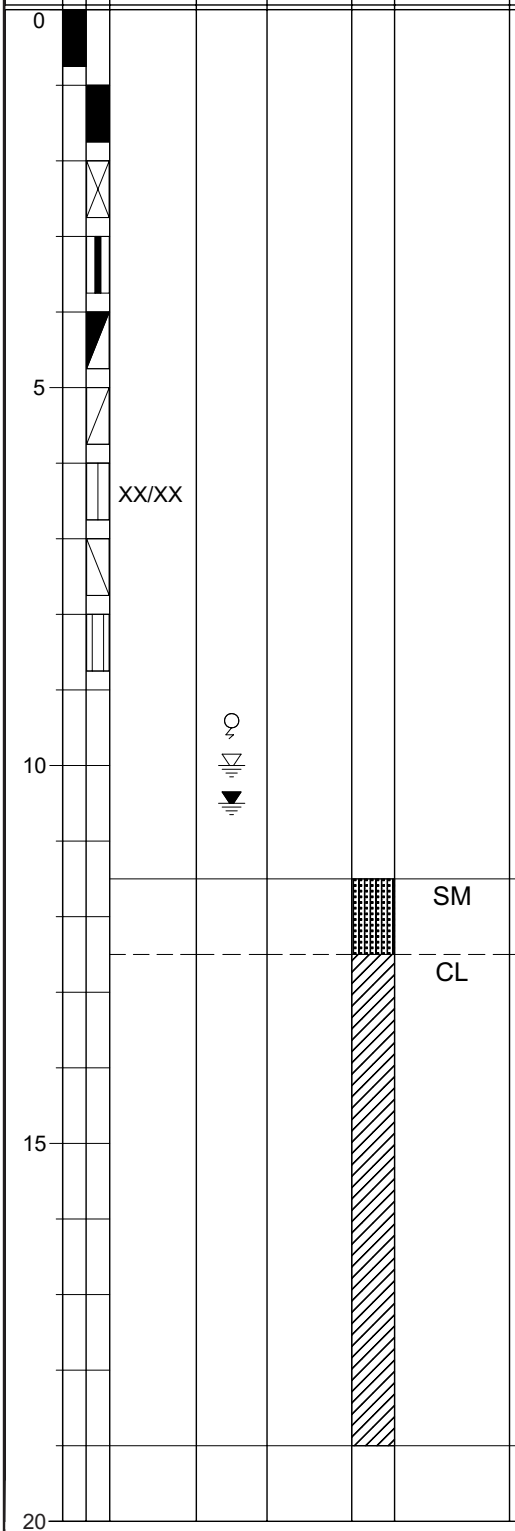
USCS METHOD OF SOIL CLASSIFICATION

BORING LOG EXPLANATION SHEET



Bulk sample.

Modified split-barrel drive sampler.

No recovery with modified split-barrel drive sampler.

Sample retained by others.

Standard Penetration Test (SPT).

No recovery with a SPT.

Shelby tube sample. Distance pushed in inches/length of sample recovered in inches.

No recovery with Shelby tube sampler.

Continuous Push Sample.

Seepage.

Groundwater encountered during drilling.

Groundwater measured after drilling.

MAJOR MATERIAL TYPE (SOIL):

Solid line denotes unit change.

Dashed line denotes material change.

Attitudes: Strike/Dip
b: Bedding
c: Contact
j: Joint
f: Fracture
F: Fault
cs: Clay Seam
s: Shear
bss: Basal Slide Surface
sf: Shear Fracture
sz: Shear Zone
sbs: Shear Bedding Surface

The total depth line is a solid line that is drawn at the bottom of the boring.

DATE DRILLED 7/02/18 BORING NO. B-1

GROUND ELEVATION 138' ± (MSL) SHEET 1 OF 1

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

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

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY ZH

DEPTH (feet)	SAMPLES (Bulk / Driven)	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0						SM	FILL: Dark brown, moist, loose to medium dense, silty fine to medium SAND; scattered grass and roots.
						SC	Grayish brown, moist, medium dense, clayey fine to coarse SAND; few gravel up to approximately 1 inch.
			11.2				
5		45	11.0	114.9			Brown; silty.
							VERY OLD PARALIC DEPOSITS: Light brown, moist, moderately cemented, silty fine-grained SANDSTONE; massive.
10		81/6"					Light brown and light reddish brown (mottled); scattered gravel up to approximately 1-1/2 inches.
15		50/3"					Scattered gravel and cobbles.
							Total Depth = 15.3 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 7/02/18.
							Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report.
							The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
20							

FIGURE A- 1

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JOHN F. KENNEDY NEIGHBORHOOD PARK IMPROVEMENTS
4825 OCEAN VIEW BOULEVARD, SAN DIEGO, CALIFORNIA

DATE DRILLED 7/02/18 BORING NO. B-2

GROUND ELEVATION 136' ± (MSL) SHEET 1 OF 1

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

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

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY ZH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0	■			16.8			SM / SC	FILL: Dark brown, moist, loose to medium dense, silty fine to medium SAND; scattered roots and grass. Grayish brown, moist, medium dense, clayey fine to coarse SAND; scattered gravel up to approximately 1 inch.
5		■	34					VERY OLD PARALIC DEPOSITS: Light brown, moist, weakly to moderately cemented, silty fine-grained SANDSTONE.
10			50/0"					Scattered gravel.
15		■	50/5"					Scattered gravel and cobbles; clayey.
20								Total Depth = 15.4 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 7/02/18. Note: 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.

FIGURE A- 2

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4825 OCEAN VIEW BOULEVARD, SAN DIEGO, CALIFORNIA

DATE DRILLED 7/02/818 BORING NO. B-3/IT-1

GROUND ELEVATION 138' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 6" Diameter Hand Auger (Manual Excavation)

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY ZH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0				14.5			SM	FILL: Brown, moist, medium dense, silty fine to medium SAND; scattered grass and roots; some gravel. Some clay.
								VERY OLD PARALIC DEPOSITS: Light brown, moist, moderately to strongly cemented, silty SANDSTONE.
5								Total Depth = 3 feet. Groundwater not encountered during drilling. Infiltration test set on 7/02/18. Backfilled on 7/038/18. Note: Groundwater, though not encountered at the time of drilling, may rise to a higher level due to seasonal variations in precipitation and several other factors as discussed in the report. The ground elevation shown above is an estimation only. It is based on our interpretations of published maps and other documents reviewed for the purposes of this evaluation. It is not sufficiently accurate for preparing construction bids and design documents.
10								
15								
20								

FIGURE A- 3

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JOHN F. KENNEDY NEIGHBORHOOD PARK IMPROVEMENTS
4825 OCEAN VIEW BOULEVARD, SAN DIEGO, CALIFORNIA

DATE DRILLED 7/02/818 BORING NO. B-4/IT-2

GROUND ELEVATION 136' ± (MSL) SHEET 1 OF 1

METHOD OF DRILLING 6" Diameter Hand Auger (Manual Excavation)

DRIVE WEIGHT N/A DROP N/A

SAMPLED BY CAT LOGGED BY CAT REVIEWED BY ZH

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0				15.0			SM	FILL: Brown, moist, medium dense, silty fine to medium SAND; scattered grass and roots; scattered gravel and cobbles. Scattered roots up to approximately 1/2 inches in diameter. Clayey.
								Total Depth = 2.5 feet. Groundwater not encountered during drilling. Infiltration test set on 7/02/18. Backfilled on 7/03/18. Note: 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.
5								
10								
15								
20								

FIGURE A- 4

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JOHN F. KENNEDY NEIGHBORHOOD PARK IMPROVEMENTS
4825 OCEAN VIEW BOULEVARD, SAN DIEGO, CALIFORNIA

APPENDIX B

Laboratory Testing

APPENDIX B

LABORATORY TESTING

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

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

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 Figure B-1. These test results were utilized in evaluating the soil classifications in accordance with the USCS.

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

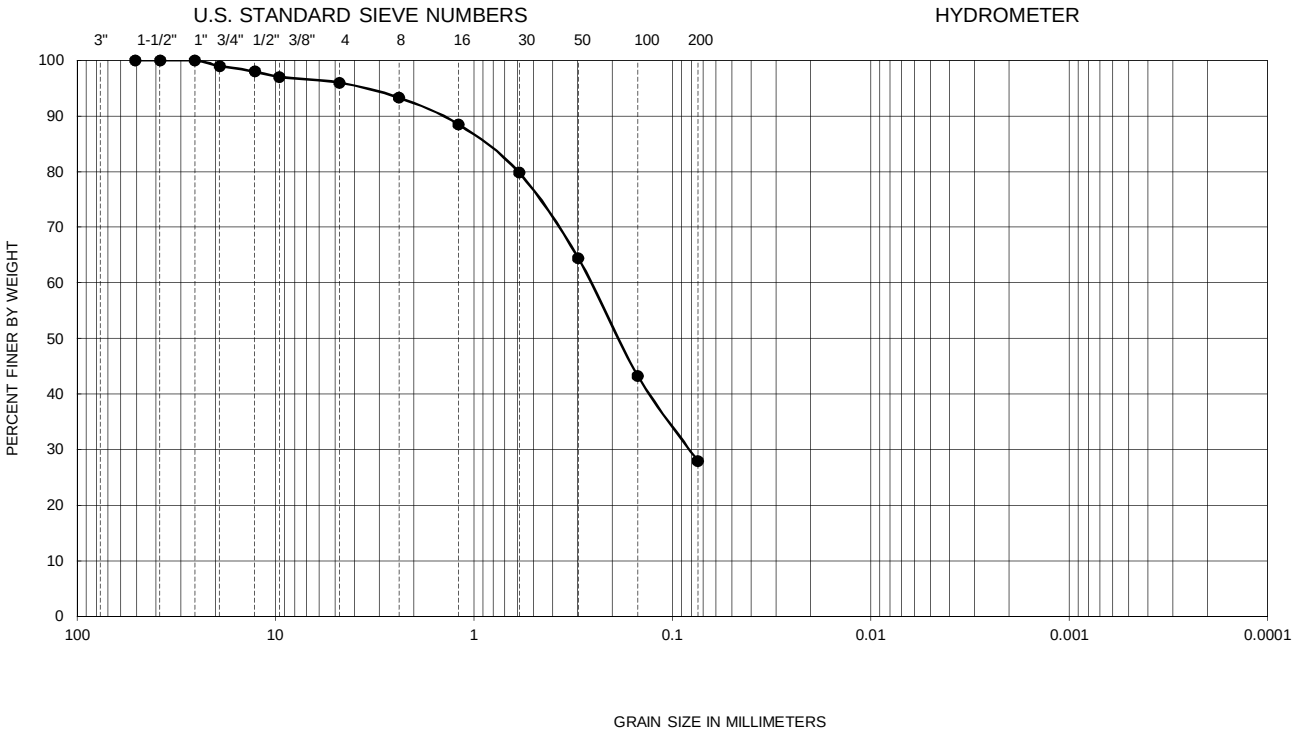
Direct Shear Tests
Direct shear tests were performed on remolded samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figure B-3.

Expansion Index Tests
The expansion index of selected materials was evaluated in general accordance with Uniform Building Code (UBC) Standard No. 18-2 (ASTM D 4829). Specimens were molded under a specified compactive energy at approximately 50 percent saturation (plus or minus 1 percent). The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 psf and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure B-4.

Proctor Density Tests
The maximum dry density and optimum moisture content of selected representative soil samples were evaluated using the Modified Proctor method in general accordance with ASTM D 1557. The results of these tests are summarized on Figure B-5.

Soil Corrosivity Tests
Soil pH and electrical resistivity tests were performed on a representative sample in general accordance with CT 643. The sulfate and chloride contents of the selected sample were evaluated in general accordance with CT 417 and 422, respectively. The test results are presented on Figure B-6.

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	0.0-4.0	--	--	--	--	--	--	--	--	28	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

GRADATION TEST RESULTS

JOHN F. KENNEDY NEIGHBORHOOD PARK IMPROVEMENTS
4825 OCEAN VIEW BOULEVARD, SAN DIEGO, CALIFORNIA

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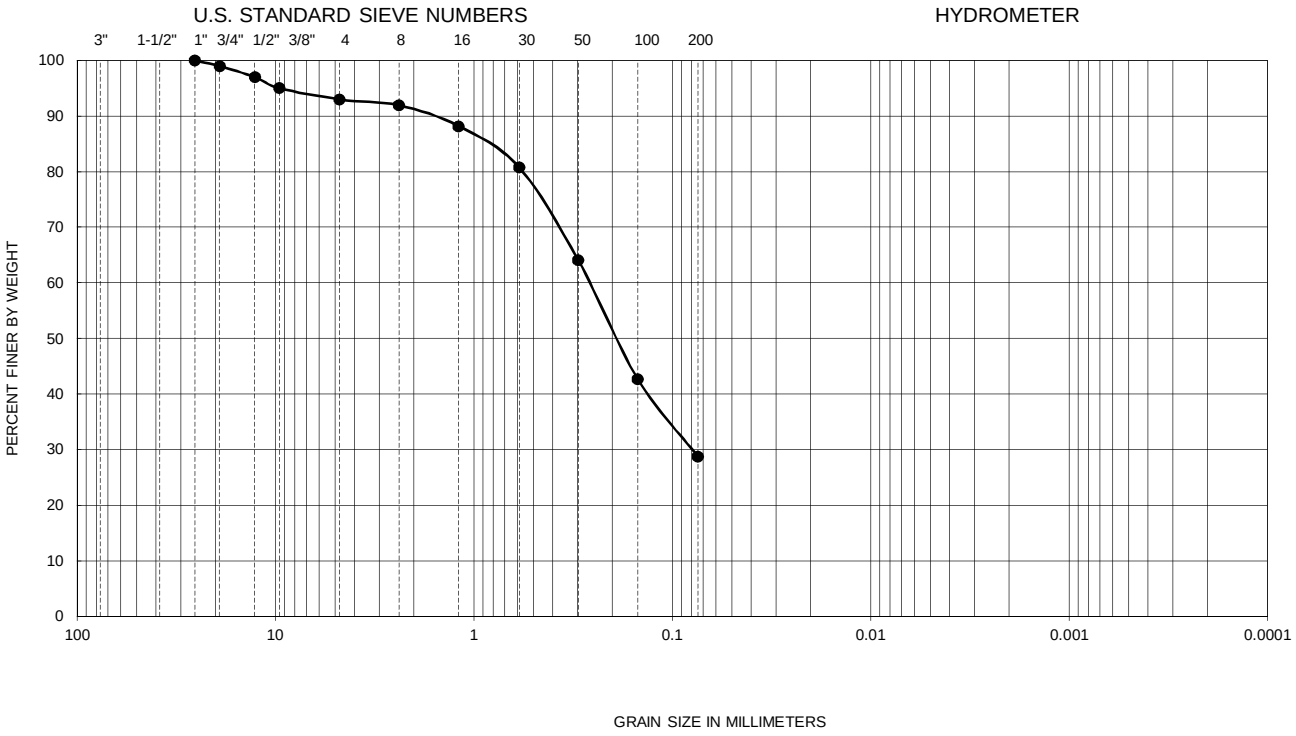
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



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	0.0-3.0	--	--	--	--	--	--	--	--	29	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

GRADATION TEST RESULTS

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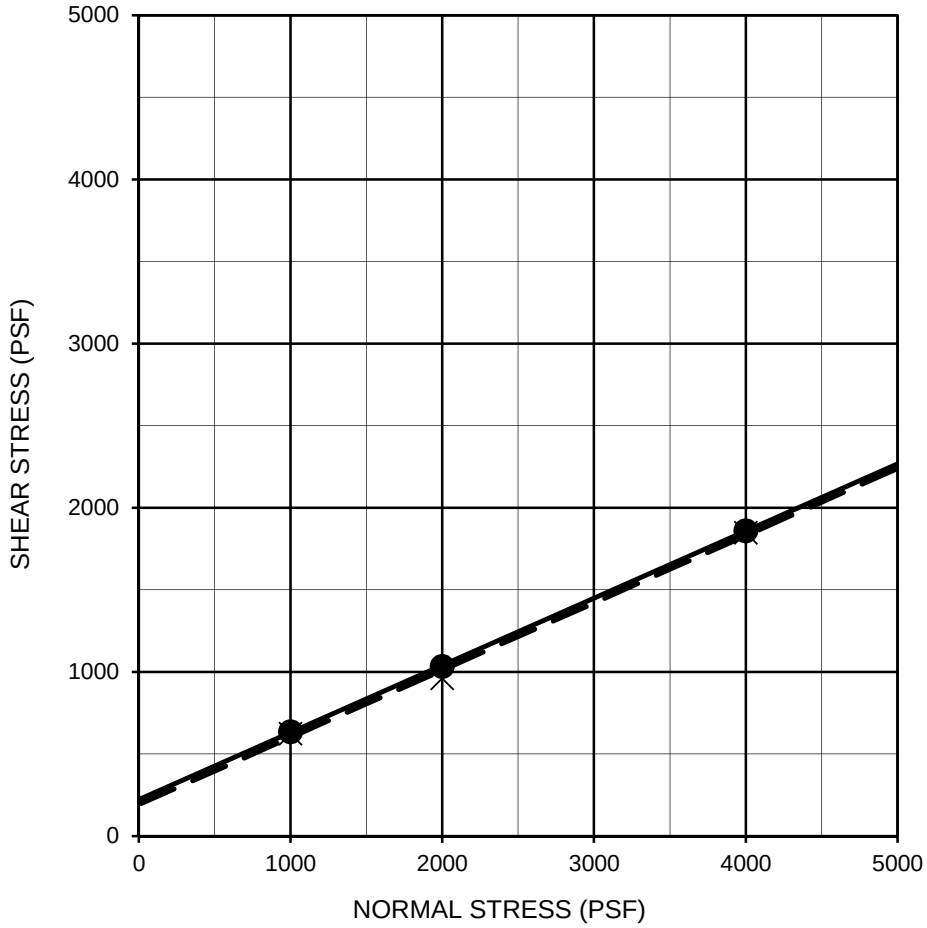

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Remolded @ 90% Relative Compaction	—●—	B-1	0.0-4.0	Peak	220	22	SC
	– – X – –	B-1	0.0-4.0	Ultimate	190	22	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-3

DIRECT SHEAR TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
B-2	0.0-3.0	9.0	112.1	-	0.050	50	Low

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

FIGURE B-4

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EXPANSION INDEX TEST RESULTS

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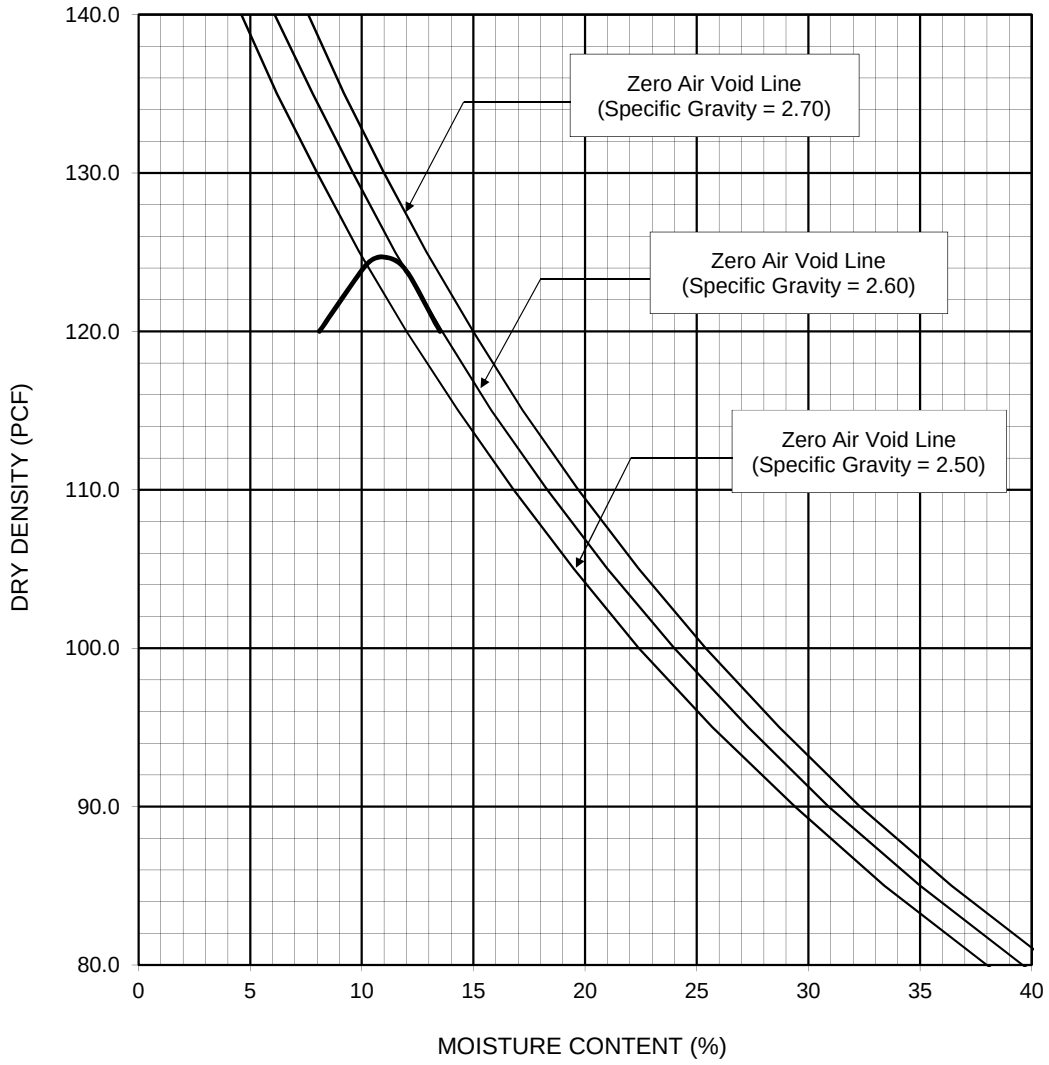

Sample Location	Depth (ft)	Soil Description	Maximum Dry Density (pcf)	Optimum Moisture Content (percent)
B-1	0.0-4.0	Clayey SAND	124.7	10.9

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

FIGURE B-5

PROCTOR DENSITY TEST RESULTS

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SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH [1]	RESISTIVITY [1] (ohm-cm)	SULFATE CONTENT [2]		CHLORIDE CONTENT [3] (ppm)
				(ppm)	(%)	
B-1	0.0-4.0	7.5	410	190	0.019	485

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

FIGURE B-6

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CORROSIVITY TEST RESULTS

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