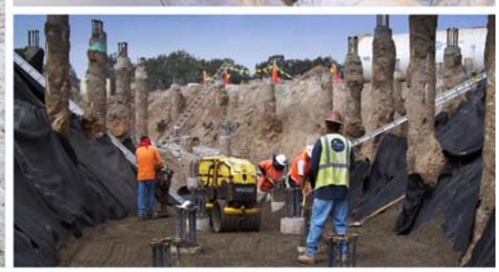
Geotechnical Evaluation

Neighborhood No. 1 Sewer Pump Station Replacement
16106 4S Ranch Parkway
San Diego, California

Infrastructure Engineering Corporation

14271 Danielson Street | San Diego, California 92064

February 21, 2018 | Project No. 108505001



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Ninyo & Moore

Geotechnical & Environmental Sciences Consultants

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February 21, 2018
Project No. 108505001

Mr. Patrick Mulvey, PE
Infrastructure Engineering Corporation
14271 Danielson Street
San Diego, California 92064

Subject: Geotechnical Evaluation
Neighborhood No. 1 Sewer Pump Station Replacement
16106 4S Ranch Parkway
San Diego, California

Dear Mr. Mulvey:

In accordance with your request and authorization, Ninyo & Moore has performed a geotechnical evaluation for the proposed Neighborhood No. 1 Sewer Pump Station Replacement located at 16106 4S Ranch Parkway in San Diego, California. This geotechnical evaluation has been performed in general accordance with Ninyo & Moore's proposal dated August 30, 2017. This report presents our findings and conclusions regarding the subject project.

We appreciate the opportunity to be of service of this project.

Sincerely,
NINYO & MOORE

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1 INTRODUCTION

In accordance with your request and authorization, Ninyo & Moore has performed a geotechnical evaluation for the proposed Neighborhood No. 1 Sewer Pump Station Replacement located at 16106 4S Ranch Parkway in San Diego, California (Figure 1). The purpose of this geotechnical evaluation was to assess the general geologic conditions at the site and to develop conclusions regarding potential geologic and seismic impacts associated with the project. This report presents a summary of our findings and conclusions regarding the geotechnical conditions within the project area and our recommendations regarding design and construction of the proposed improvements.

2 SCOPE OF SERVICES

Ninyo & Moore's scope of services for this study has included the review of geotechnical background materials, geologic reconnaissance of the project area, and geotechnical analyses. Specifically, we have performed the following tasks for our preliminary geotechnical study:

- Review of readily available topographic and geologic maps, published geotechnical literature, geologic and seismic data, groundwater data, 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).
- Acquiring a boring permit from the County of San Diego, Department of Environmental Health (DEH).
- Performing a subsurface exploration consisting of the drilling, logging, and sampling of two small diameter exploratory borings. The soil borings were drilled to depths of up to approximately 40 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.
- Performing geotechnical laboratory testing on representative samples to evaluate 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 site consists of a relatively flat-lying, irregular shaped parcel and is bounded to the north by the 4S Ranch Sports Park, and to the south, east, and west by undeveloped open space. Lusardi Creek is located approximately 400 feet south of the site and flows westward. The site currently supports an active sewer pump station that was built in 2002. An approximately 20-foot high, south-facing slope is located along the northern boundary of the site. Based on our review of readily available topographic and geologic maps and aerial photographs, the slope may have been constructed as part of the development of the adjacent sports park. Elevations at the pump station generally range from approximately 424 feet above mean sea level (MSL) in the eastern portion of the property to approximately 418 feet MSL in the western portion. An emergency overflow pond is located in the western portion of the site. The pond is constructed with approximately 2:1 (horizontal:vertical) descending slopes and the bottom of the pond is at an approximate elevation of 407 feet MSL.

Existing improvements at the pump station include a single-story, masonry block pump room, a generator pad, a wet well, a fuel tank, a chemical storage and feed system, and ancillary improvements. Based on our review of a conceptual design report (IEC, 2015) and a preliminary layout plan, we understand that the proposed improvements at the site will include construction of a new electrical building, a new dry well adjacent to the existing wet well and associated utilities.

4 SUBSURFACE EXPLORATION

Our subsurface exploration was performed on January 2, 2018 and consisted of the drilling, logging, and sampling of two small-diameter borings (B-1 and B-2). 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.

Borings B-1 and B-2 were drilled to depths of approximately 40 feet and 19½ feet, respectively, using a truck-mounted drill rig equipped with 8-inch diameter, continuous-flight, hollow-stem augers and hand augering equipment. 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 A.

5 GEOTECHNICAL LABORATORY TESTING

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

6 GEOLOGY

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

6.1 Regional Geology

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 Peninsular Ranges Province is traversed by a group of sub-parallel faults and fault zones trending roughly northwest. Several of these faults are considered to be active. 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 Zone, the nearest active fault system, has been mapped approximately 11 miles west of the project site.

6.2 Site Geology

The geologic units encountered during our reconnaissance and subsurface evaluation included fill, alluvium, and materials of the Friars Formation (Figure 3). Generalized descriptions of the encountered soils are provided in the following sections. Additional descriptions of the materials are provided in Appendix A.

6.2.1 Pavement Section

Pavement sections were encountered at the surface during our drilling operations. The encountered pavement sections were generally 12 inches thick and consisted of approximately 5 inches of asphalt concrete (AC) over approximately 7 inches of base material. The base material generally consisted of gray, dry, dense, silty gravel with sand.

6.2.2 Fill

Fill materials were encountered in our borings underlying the existing pavement section and extending to depths of up to approximately 8 feet. As encountered, these materials generally consisted of olive brown and dark gray, moist, firm to stiff, silty and sandy clay, with varying amounts of gravel. Documentation of the placement and compaction of existing fill was not available for our review.

6.2.3 Alluvium

Alluvium was encountered beneath the fill in our borings and extended to depths of up to 15½ feet. As encountered, the alluvium generally consisted of olive gray and brown, moist, stiff, silty clay.

6.2.4 Friars Formation

Materials of the Friars Formation were encountered in our borings beneath the alluvium and extended to the depths explored of up to 40 feet. As encountered, the Friars Formation generally consisted of various shades of brown and gray, dry to moist, weakly to moderately indurated sandy claystone, and weakly to moderately cemented clayey sandstone. Trace amounts of gravel and cobbles were encountered in the Friars Formation.

6.3 Groundwater

Groundwater was not encountered in our borings to the depths explored of up to approximately 40 feet. Fluctuations in the depth to groundwater will occur due to flood events, seasonal precipitation, variations in ground elevations, subsurface stratification, irrigation, groundwater pumping, storm water infiltration, and other factors.

Additionally, due to relatively impermeable clay soil layers, seepage and perched water conditions may be encountered and should be anticipated at the site. Backfill and bedding materials also tend to act as a conduit for water and perched water conditions may be present along 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 geologic hazards such as tsunamis and landsliding are discussed 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, 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 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 11 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 the center of the pump station site.

Table 1 – Principal Active Faults

Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (Mmax)
Rose Canyon	11 (18)	6.9
Newport-Inglewood (Offshore Segment)	21 (34)	7.0
Elsinore (Julian Segment)	24 (39)	7.4
Elsinore (Temecula Segment)	24 (39)	7.1
Coronado Bank	25 (41)	7.4
Earthquake Valley	33 (53)	6.8
Elsinore (Glen Ivy Segment)	44 (71)	6.9
Elsinore (Coyote Mountain Segment)	45 (73)	6.9
San Jacinto (Coyote Creek Segment)	46 (74)	7.0
San Jacinto (Anza Segment)	48 (77)	7.3

Table 1 – Principal Active Faults		
Fault	Approximate Fault-to-Site Distance miles (kilometers)	Maximum Moment Magnitude (Mmax)
San Jacinto (Clark Segment)	48 (78)	7.1
Palos Verdes	50 (81)	7.3
San Joaquin Hills	52 (84)	7.1
San Jacinto (San Jacinto Valley Segment)	53 (86)	7.0
San Jacinto (Borrego Segment)	55 (88)	6.8

The principal seismic hazard considerations at the site are surface ground rupture, ground shaking, and seismically induced liquefaction and/or dynamic settlement. A brief description of seismic and other geologic hazards and the potential for their occurrence on site are presented below.

7.2 Surface Rupture

Based on our review of the referenced literature and our site reconnaissance, no active faults are known to cross the project vicinity. Therefore, the potential for ground rupture at the site 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.

7.3 Ground Motion

The 2016 California Building Code (CBC) specifies that the Risk-Targeted, Maximum Considered Earthquake (MCER) ground motion response accelerations be used to evaluate seismic loads for design of buildings and other structures. The MCER 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 MCER for the site was calculated as 0.42 g 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 (MCEG) peak ground acceleration with adjustment for site class effects in accordance with the American Society of Civil Engineers (ASCE) 7-10 Standard. The MCEG peak ground acceleration is based on the geometric mean peak ground acceleration with a 2 percent probability of exceedance in 50 years. The MCEG peak ground acceleration with adjustment for site class effects (PGAM) was calculated as 0.40 g using the USGS (USGS, 2018) seismic design tool that yielded a mapped MCEG peak ground acceleration of 0.35 g for the site and a site coefficient (FPGA) of 1.15 for Site Class D.

7.4 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.

According to the County of San Diego Draft Liquefaction map (SANGIS, 2009), the site is located within an area mapped as being potentially susceptible to liquefaction. However, based on the absence of shallow groundwater and the relatively dense nature of the underlying Friars Formation, it is our opinion that the potential for liquefaction and seismically induced settlement is not a design consideration.

7.5 Landslides

The site is located in an area classified as generally susceptible to landslides and the northwestern portion of the site is located in an area classified as most susceptible to landslides (Tan, 1995). Based on our review of the referenced geologic maps, topographic maps, and stereoscopic aerial photographs, no landslides or indications of deep-seated landsliding were noted underlying the pump station site. A landslide is mapped northwest of the site (Figure 3; Kennedy and Tan, 2007). Portions of this landslide were removed during grading of the 4S Ranch Development (Ninyo & Moore, 2015; Geocon, 2004). As such the potential for significant large-scale slope instability at the site is not a design consideration.

7.6 Tsunamis

Tsunamis are long seismic sea waves (long compared to ocean depth) generated by sudden movements of the sea floor caused by submarine earthquakes, landslides, or volcanic activity. Based on the inland location of the site, the potential for a tsunami to impact the site is not a design consideration.

7.7 Flood Hazards

Based on review of Federal Emergency Management Agency (FEMA) Flood Insurance Rate Maps (FIRM), flood hazard mapping has not been published at the project site. Based on our review of the referenced geologic and topographic maps, seasonal flooding of Lusardi Creek may be anticipated.

8 CONCLUSIONS

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

- The site is underlain by fill soils, alluvium, and materials of the Friars Formation. The fill and alluvium are not considered suitable for structural support of the proposed improvements in their current condition.
- The proposed dry well structure will be supported on a mat foundation system with a deepened perimeter edge, and the southern portion of the proposed electrical building will be supported by the dry well structure. The northern portion of the electrical building is anticipated to overly fill and alluvium which are not considered suitable for structural support in their current condition. Further recommendations to mitigate these conditions are provided in the following sections.
- Groundwater was not encountered during our subsurface evaluation. Perched conditions and fluctuations in groundwater may occur due to variations in ground surface topography, subsurface geologic structure, rainfall, irrigation, and other factors.
- The on-site fill soils, alluvium, and Friars Formation can be excavated using heavy duty earthmoving equipment in good working condition. The contractor should anticipate caving and/or sloughing conditions when performing unbraced excavations.
- Based on the results of our laboratory testing, on-site soils possess medium expansive potential. Selective grading, mixing, and/or import of fill materials should be anticipated.
- There are no known active faults crossing the site, and the potential for surface ground rupture is considered low. Additionally, the Rose Canyon Fault is mapped approximately 11 miles west of the site.

- The site is located in an area considered susceptible to liquefaction. However, based on the dense nature of the soil and the absence of shallow groundwater, liquefaction hazards are not a design consideration.
- Due to the anticipated soil conditions across the foundation depth of the proposed improvements, special foundations recommendations are provided herein.
- Based on the results of our soil corrosivity tests, American Concrete Institute (ACI) 318, and Caltrans (2015) criteria, the on-site soils would be classified as corrosive.

9 RECOMMENDATIONS

Based on the results of our subsurface evaluation and our understanding of the proposed construction, we present the following general geotechnical recommendations relative to the design and construction of the proposed improvements. Based on our understanding of the project, the following recommendations are provided for the design and construction of the proposed project.

9.1 Earthwork

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

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 recommendations.

9.1.2 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.

9.1.3 Excavation Characteristics

The result of our field exploration program indicates that the project site is underlain by fill, alluvium and formational soils. Excavation of the subsurface materials should be feasible with heavy-duty excavation equipment in good working condition. However, the contractor should anticipate caving and/or sloughing conditions if performing unbraced excavations in fill or alluvial soils. Additionally, due to the presence of gravel, cobbles, and possible construction debris, the contractor may encounter difficulty in performing excavations, drilling, or pile driving when these materials are encountered.

9.1.4 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
Friars Formation	Type B

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 or alluvium and 1:1 in Friars Formation. 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.

9.1.5 Existing Utilities

Multiple existing utilities are present adjacent to the proposed dry well. It is imperative that the contractor take care to locate and protect existing utilities or other buried structures during construction. Shoring and excavation systems should be designed to support excavations and protect utilities in advance of excavating.

9.1.6 Shoring

Excavations below groundwater or in areas with limited space for construction, where temporary excavations may not be laid back at the recommended slope inclination, a shoring system may be incorporated to stabilize the excavation sidewalls during construction. Shoring systems should be constructed through the fill, alluvium, and Friars Formation materials. The shoring system should be designed using the magnitude and distribution of lateral earth pressures presented on Figure 5. The recommended design earth pressures are based on the assumptions that: (a) the shoring system is constructed without raising the ground surface elevation behind the shoring, (b) that there are no surcharge loads, such as soil stockpiles, construction materials, or vehicular traffic, and (c) that no loads act above a 1:1 plane extending up and back from the base of the shoring system. For shoring subjected to the above-mentioned surcharge loads, the contractor should include the effect of these loads on lateral earth pressures acting on the shoring wall.

Settlement of the ground surface may occur behind the shoring wall during excavation. The amount of settlement depends on the type of shoring system, the quality of contractor's workmanship, and soil conditions. Settlement may cause distress to adjacent structures, if present. To reduce the potential for distress to adjacent structures, we recommend that the shoring system be designed to limit the ground settlement behind the shoring to ½ inch or less. Possible causes of settlement that should be addressed include vibration during installation of the sheet piling, excavation for construction, construction vibrations, dewatering, and removal of the support system. We recommend that the potential settlement distress be evaluated carefully by the contractor prior to construction. Such an evaluation would include a precondition survey of existing site structures, which should include photographs and documentation of existing cracks, separations, and other features. Crack gauges may be installed at locations where necessary. Post-construction surveys of the existing structures should be performed and compared with the precondition survey.

The contractor should retain a qualified and experienced engineer to design the shoring system. The shoring parameters presented in this report are for preliminary design purposes and the contractor should evaluate the adequacy of these parameters and make appropriate modifications for their design. We recommend that the contractor take appropriate measures to protect workers. OSHA requirements pertaining to worker safety should be observed. We further recommend that the construction methods provided herein be carefully evaluated by a qualified specialty contractor prior to commencement of the construction.

9.1.7 Remedial Grading

Due to the presence of medium expansive soils at the site, we recommend remedial grading for interior slabs-on-grade, exterior flatwork, and equipment pads. The intent of this remedial grading is to provide suitable support for interior slabs-on grade and equipment pads and reduce the potential for differential vertical offsets and resulting trip hazards. We recommend that the existing soils within the areas described be over-excavated to a depth of 2 feet below finished grade and replaced with compacted, engineered fill.

The over excavation should extend to the horizontally 2 feet from the horizontal limits of the flatwork or equipment pad footprint, where feasible. The lateral extents of the overexcavation may be modified in the field based on site constraints such as existing buildings, structures, utilities, and property lines. 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 overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index of 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.

9.1.8 Construction Dewatering

Groundwater was not encountered during our subsurface exploration to the depths explored of approximately 40 feet. However, fluctuations in the depth to groundwater will occur and shallower groundwater should be considered. Accordingly, dewatering should be anticipated for the planned underground vault (dry well) excavation in order to perform work in a dry condition. The dewatering system design should be performed by a specialty dewatering contractor. Disposal of groundwater should be performed in accordance with guidelines of the Regional Water Quality Control Board.

9.1.9 Excavation Bottom Stability

The excavation bottom for the dry well is anticipated to expose dry to moist, weakly to moderately indurated claystone and moderately to strongly cemented sandstone of the Friars Formation. Although not anticipated, unstable bottom conditions may be encountered during construction. If encountered during construction, Ninyo & Moore should be contacted to evaluate unstable conditions and provide supplemental recommendations.

9.1.10 Materials for Fill

Materials for fill may be selectively graded from on-site excavations or may be import materials, provided they meet the following criteria. Fill soils should generally be granular soils with a very low to low expansion potential (i.e., an EI of 50 or less) and 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 Caltrans (2015) corrosion guidelines and ACI 318, which is defined as a soil with an electrical resistivity value greater than 1,000 ohm-centimeters (ohm-cm), a chloride content of less than 500 parts per million (ppm), a sulfate content of less than 1,000 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.

9.1.11 Compacted Fill

Prior to placement of compacted fill, the contractor should request an evaluation of the exposed ground surface by the project geotechnical consultant. Unless otherwise recommended, the exposed ground surface should then be scarified to a depth of approximately 8 inches and watered or dried, as needed, to achieve moisture contents generally above the optimum moisture content. The scarified materials should then be compacted to a relative compaction of 90 percent as evaluated 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 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 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.

9.1.12 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 1,000 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 1,200 psi may be used for trenches deeper than 5 feet.

9.1.13 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.

9.1.14 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 6. Thrust blocks should be backfilled with granular backfill material and compacted in accordance with recommendations presented in this report.

9.1.15 Drainage

Surface drainage on the site should be provided so that water is not permitted to pond adjacent to footings or pavements. A gradient of 2 percent or steeper should be maintained away from structures and drainage patterns should be established to divert and remove water from the site to appropriate outlets.

Care should be taken by the contractor during final grading to preserve any berms, drainage terraces, interceptor swales or other drainage devices of a permanent nature on or adjacent to the property. Drainage patterns established at the time of final grading should be maintained for the life of the project. The property owner and the maintenance personnel should be made aware that altering drainage patterns might be detrimental to foundation performance.

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

Seismic Design Factors	Value
Site Class	D
Site Coefficient, F_a	1.127
Site Coefficient, F_v	1.671
Mapped Spectral Response Acceleration at 0.2-second Period, S_s	0.931g

Table 2 – 2016 California Building Code Seismic Design Criteria	
Seismic Design Factors	**Value**
Mapped Spectral Response Acceleration at 1.0-second Period, S_1	0.365g
Spectral Response Acceleration at 0.2-second Period Adjusted for Site Class, S_{MS}	1.050g
Spectral Response Acceleration at 1.0-second Period Adjusted for Site Class, S_{M1}	0.609g
Design Spectral Response Acceleration at 0.2-second Period, S_{DS}	0.700g
Design Spectral Response Acceleration at 1.0-second Period, S_{D1}	0.406g

9.3 Foundations

We understand the proposed dry well structure will be supported on a mat foundation system with a deepened perimeter edge, and the southern portion of the proposed electrical building will be supported by the dry well structure. The northern portion of the electrical building is anticipated to overly fill and alluvium which are not considered suitable for structural support in their current condition. Therefore, we recommend that the portion of the proposed electrical building that will not be supported by the dry well structure be founded on cast-in-drilled-hole (CIDH) piles. The following sections present geotechnical recommendations for mat foundations and deep foundations that are bearing on Friars Formation.

Design of foundations should also be designed in accordance with structural considerations. In addition, requirements of the governing jurisdictions, practices of the Structural Engineers Association of California, and applicable building codes should be considered in the design of structures. Should the configuration of the building change prior to construction alternative foundation recommendations may be provided.

9.3.1 Cast-In-Drilled-Hole Piles

We recommend that the proposed structure be supported on drilled foundations having a diameter of 2 feet or more. Additionally, we recommend that the CIDH piles extend through the existing fill and alluvial materials and be embedded 5 feet into competent Friars Formation. As indicated on our borings logs in Appendix A, the fill and alluvium thicknesses vary across the site. Accordingly, the length of piles will also vary based on the thickness of fill and alluvium. The pile dimensions (i.e., diameter and embedment) and spacing should be evaluated by the project structural engineer.

We recommend that the 24-inch CIDH piles embedded 5 feet into competent Friars Formation be designed using an allowable axial capacity of 50 kips in downward compression and 40 kips in uplift. The allowable downward axial capacity may be increased by 3 kips per additional foot of embedment into the Friars Formation and the uplift axial capacity may be increased by 1 kips per additional foot of embedment into the Friars Formation. These allowable downward and uplift capacities are based on a factor of safety of 2.0 and 1.5, respectively.

We recommend that the 24-inch CIDH piles embedded 5 feet into competent Friars Formation with lengths of 20 feet or more be designed for lateral capacities as shown on Table 3.

Table 3 – Lateral Load Capacity of 24-Inch Diameter CIDH Pile

Design condition	Free-Head	Fixed Head
Pile Length*	20 feet or more*	
Allowable Deflection	1/4-inch at Pile Head	
Lateral Capacity, kips	21.3	46.5
Max. Positive Moment, ft-kip	69.6	67.27
Max. Negative Moment, ft-kip	--	-176.9
Depth to Max. Positive Moment, ft	6.2	9.6
Depth to Max. Negative Moment, ft	--	0
Depth to 1st Point of Zero Deflection, ft	10.9	13.3

Note:
*Depth is measured from the bottom of pile cap (top of the pile) to the pile tip.

Drilled pile excavations may be difficult to perform due to the presence of gravel in the fill and alluvial materials, and/or concretions and cemented zones within the Friars Formation. The drilled pile installation should be observed by Ninyo & Moore during construction to evaluate if the piles have been extended to the design depths and embedments. 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. The center-to-center spacing of piles should be no less than three times the nominal diameter of the pile.

Due to the presence of loose gravel within the fill, we recommend that the contractor consider taking appropriate measures during construction to reduce the potential for caving of the drilled holes, including the use of steel casing. Additionally, the contractor should clean the bottoms of the excavations with either a cleanout plate/bucket or vacuum to remove loose materials from the bottom of the excavation. We recommend placement of concrete by tremie method to see that the aggregate and cement do not segregate during concrete placement.

9.3.2 Mat Foundations

As noted above, mat foundations are anticipated to be used for the support of the dry well. Based on the geotechnical data disclosed by our subsurface exploration, along with the proposed locations of dry pit, we anticipate that the mat foundations for the proposed improvements will be supported on the Friars Formation. To provide consistent bearing conditions for the mat foundations, we recommend that no utilities, piping, or duct banks be constructed within 3 feet of the zone of influence of the bottom of each mat 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 mat.

A net allowable bearing pressure of 4,000 pounds per square foot (psf) may be assumed for the mat foundations bearing in competent (i.e., firm and unyielding) Friars Formation. This value is based on an embedment of 12 inches. The bearing capacity for the Friars Formation may be increased by 250 psf per additional foot of embedment beyond a 12-inch embedment, up to a maximum of 6,000 psf. This net allowable bearing capacity may be increased by one-third when considering loads of a short duration such as wind or seismic forces. Thickness and reinforcement of the mat foundation should be in accordance with the recommendations of the project structural engineer.

The total and differential settlements corresponding to the net allowable bearing pressures presented above are estimated to be less than 1 inch over a horizontal span of 40 feet, respectively.

Mat foundations typically experience some deflection due to loads placed on the mat and the reaction of the soils directly underlying the mat. A design modulus of subgrade reaction (K) of 275 pounds per cubic inch (pci) should be used for the Friars Formation in evaluating such deflections. This value is based on a unit square foot area and should be adjusted for large mats. Adjusted values of the modulus of subgrade reaction, Kv, can be obtained from the following equation for mats of various widths:

$$K_v = K[(B+1)/2B]^2 \quad \text{(pci)}$$

B in the above equation represents the width (i.e., the lesser dimension of the width and length) of the mat in feet.

For frictional resistance to lateral loads on mat, we recommend a coefficient of friction of 0.35 at the concrete-soil interface. For a mat with an embedment depth shallower than 2 feet, passive earth pressure should be ignored while evaluating lateral resistance; only frictional resistance should be considered. For mats with embedment depths greater than

2 feet, passive earth pressure may be combined with frictional resistance to evaluate the total lateral resistance. In such cases, the 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 resistance. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

9.4 Underground Structures

Underground structures may be designed for lateral pressures represented by the pressure diagram on Figure 7. It is recommended that the exterior of underground walls and horizontal and vertical construction joints be waterproofed, as indicated by the project civil engineer and/or architect. For pipe wall penetrations into the drywell, lift station, vaults, and other structures, standard “water-tight” penetration design should be utilized. To reduce the potential for relative pipe to wall differential settlement, which could cause pipe shearing, we recommend that a pipe joint be located close to the exterior of the wall. The type of joint should be such that minor relative movement can be accommodated without distress.

9.5 Interior Concrete Slabs-on-Grade

We recommend that conventional, interior slab-on-grade floors, be underlain by compacted fill materials of generally very low to low potential for expansion. As presented earlier, we recommend that the dry pit be supported on a concrete mat foundation. The mat foundation will also serve as the interior concrete slab-on-grade floor for a portion of the building. For the interior slab-on-grade in the portion of the building that does not overly the dry well, a structural slab that connects the slab and foundation should be considered. The slab reinforcement and expansion joint spacing should be designed by the project structural engineer so as to neglect support from underlying soils.

If moisture sensitive floor coverings are to be used, we recommend that slabs be underlain by a vapor retarder and capillary break system consisting of a 15-mil polyethylene membrane or Stego Wrap products (or equivalent) be placed over 4 inches of medium to coarse, clean sand or pea gravel.

9.6 Exterior Concrete Flatwork

Exterior concrete flatwork should be 5 inches in thickness and should be reinforced with No. 3 reinforcing bars placed at 24 inches on-center both ways. A vapor retarder is not needed for exterior flatwork. To reduce the potential manifestation of distress to exterior concrete flatwork due to movement of the underlying soil, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the structural engineer. Before placement of concrete, the subgrade soils should be scarified to a depth of 6 inches, moisture conditioned to generally above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557.

9.7 Pavement Reconstruction

In general, pavement repair to the driveway should match the existing pavement section and should conform to the requirements of the appropriate governing agency. Aggregate base material and asphalt concrete should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557.

9.8 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 an electrical resistivity of 590 ohm-cm, a soil pH of 8.2, a chloride content of 255 parts per million (ppm), and a sulfate content of 0.047 percent (i.e., 470 ppm). Based on the laboratory test results, ACI 318, and Caltrans (2015) corrosion criteria, the project site would be classified as corrosive. A corrosive soil environment is defined as a soil having an electrical resistivity value less than 1,000 ohm-cm, a chloride content of more than 500 ppm, a sulfate content more than 0.2 percent, and/or a pH less than 5.5. We recommend that a corrosion engineer be consulted with during design to address mitigation of corrosion of the proposed improvements.

9.9 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 0.047 percent by weight (i.e., about 470 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, consideration should be given to using Type II/V cement for normal weight concrete in contact with soil.

9.10 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 one exploratory boring. 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.

10 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

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2017

FIGURE 1

SITE LOCATION

OMWD NEIGHBORHOOD NO. 1 SEWER PUMP STATION
16106 4S RANCH PARKWAY, SAN DIEGO, CALIFORNIA

108505001 | 2/18

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2_108505001_BL.mxd 2/19/2018 AOB

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: GOOGLE EARTH, 2017

FIGURE 2

BORING LOCATIONS

OMWD NEIGHBORHOOD NO. 1 SEWER PUMP STATION
16106 4S RANCH PARKWAY, SAN DIEGO, CALIFORNIA

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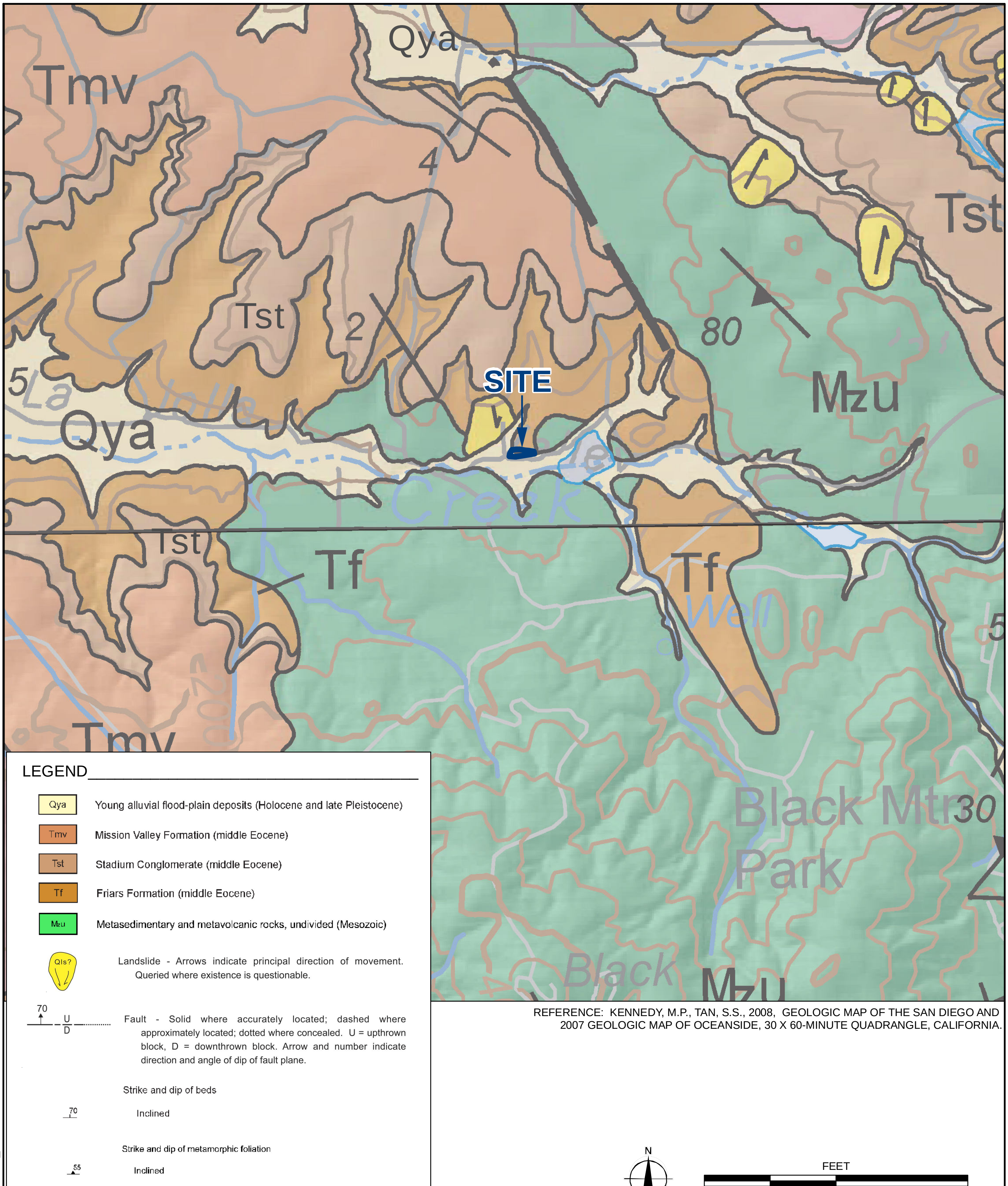

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N

FEET

0 2,000 4,000

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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 3

GEOLOGY

OMWD NEIGHBORHOOD NO. 1 SEWER PUMP STATION
16106 4S RANCH PARKWAY, SAN DIEGO, CALIFORNIA

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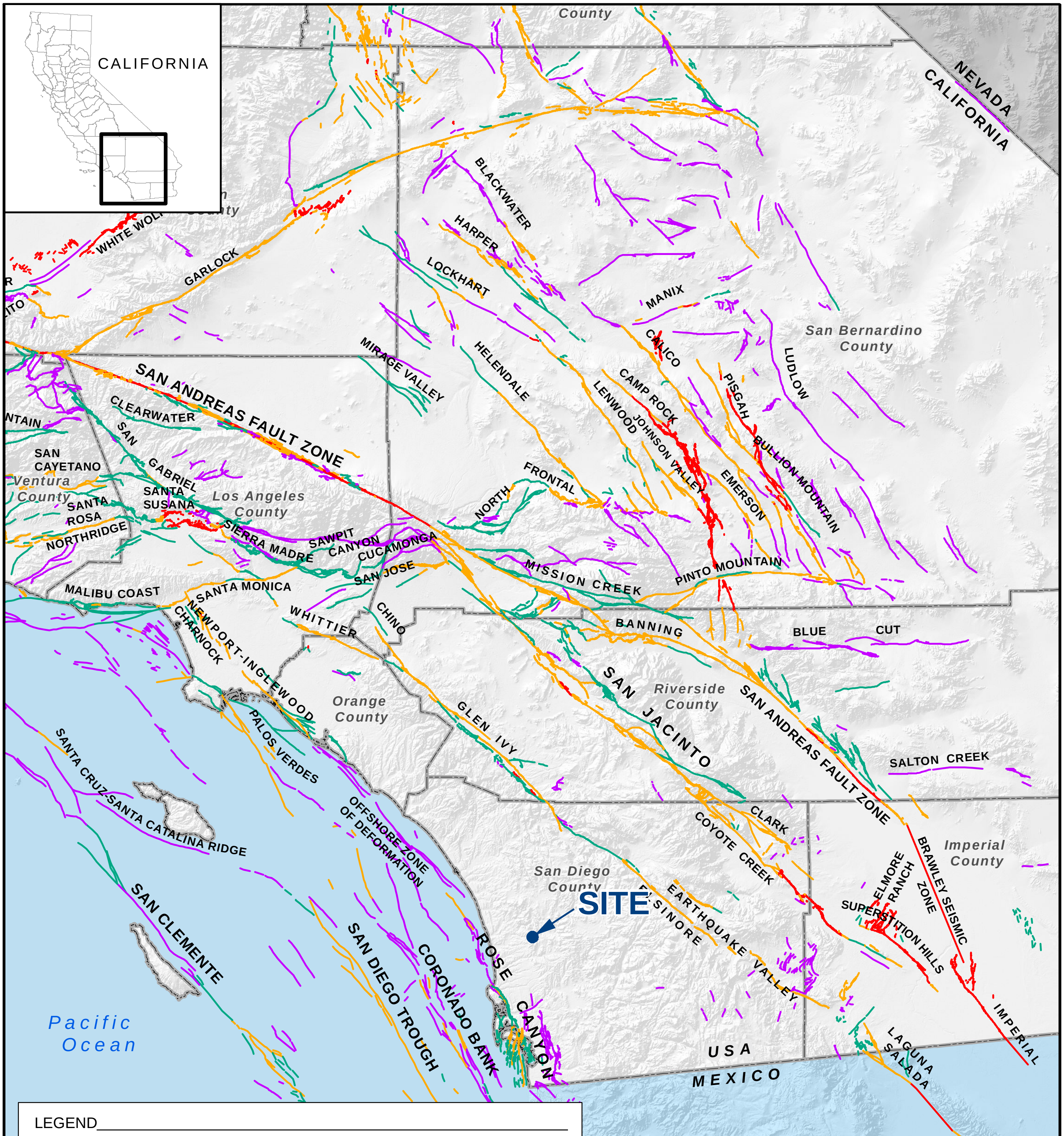

LEGEND

CALIFORNIA FAULT ACTIVITY

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

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

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MILES

0 30 60

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

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FIGURE 4

FAULT LOCATIONS

OMWD NEIGHBORHOOD NO. 1 SEWER PUMP STATION
16106 4S RANCH PARKWAY, SAN DIEGO, CALIFORNIA

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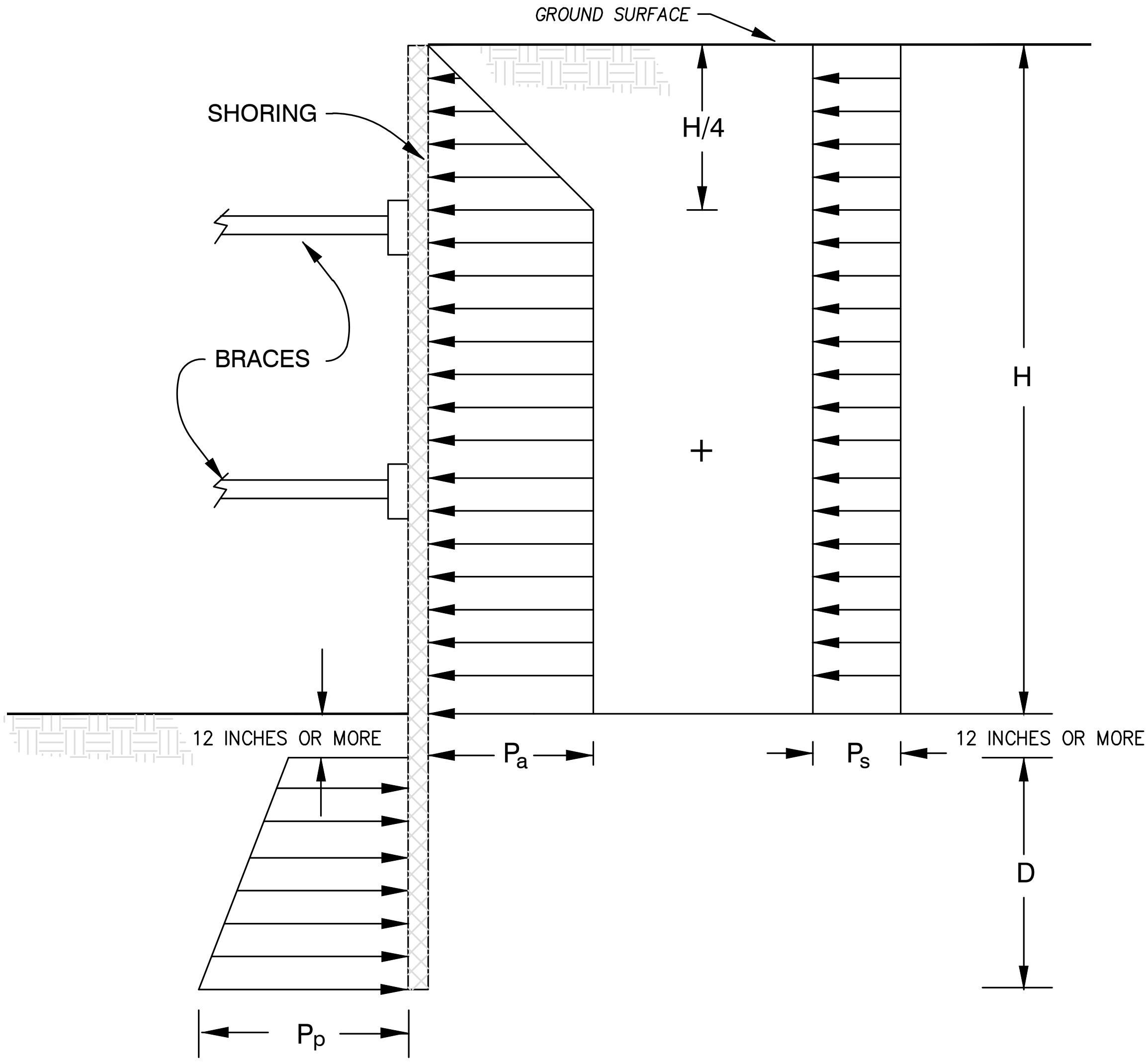

NOTES:

1. APPARENT LATERAL EARTH PRESSURE, P_a
   P_a = 45 H psf
2. CONSTRUCTION TRAFFIC INDUCED SURCHARGE PRESSURE, P_s
   P_s = 120 psf
3. PASSIVE LATERAL EARTH PRESSURE, P_p
   P_p = 1,950 psf
4. ASSUMES GROUNDWATER IS NOT PRESENT
5. SURCHARGES FROM EXCAVATED SOIL OR CONSTRUCTION MATERIALS ARE NOT INCLUDED
6. H AND D ARE IN FEET

NOT TO SCALE

5 108505001 D-BE.DWG

FIGURE 5

LATERAL EARTH PRESSURES FOR BRACED EXCAVATION (SOFT TO FIRM CLAY)

OMWD NEIGHBORHOOD NO. 1 SEWER PUMP STATION
16106 4S RANCH PARKWAY, SAN DIEGO, CALIFORNIA

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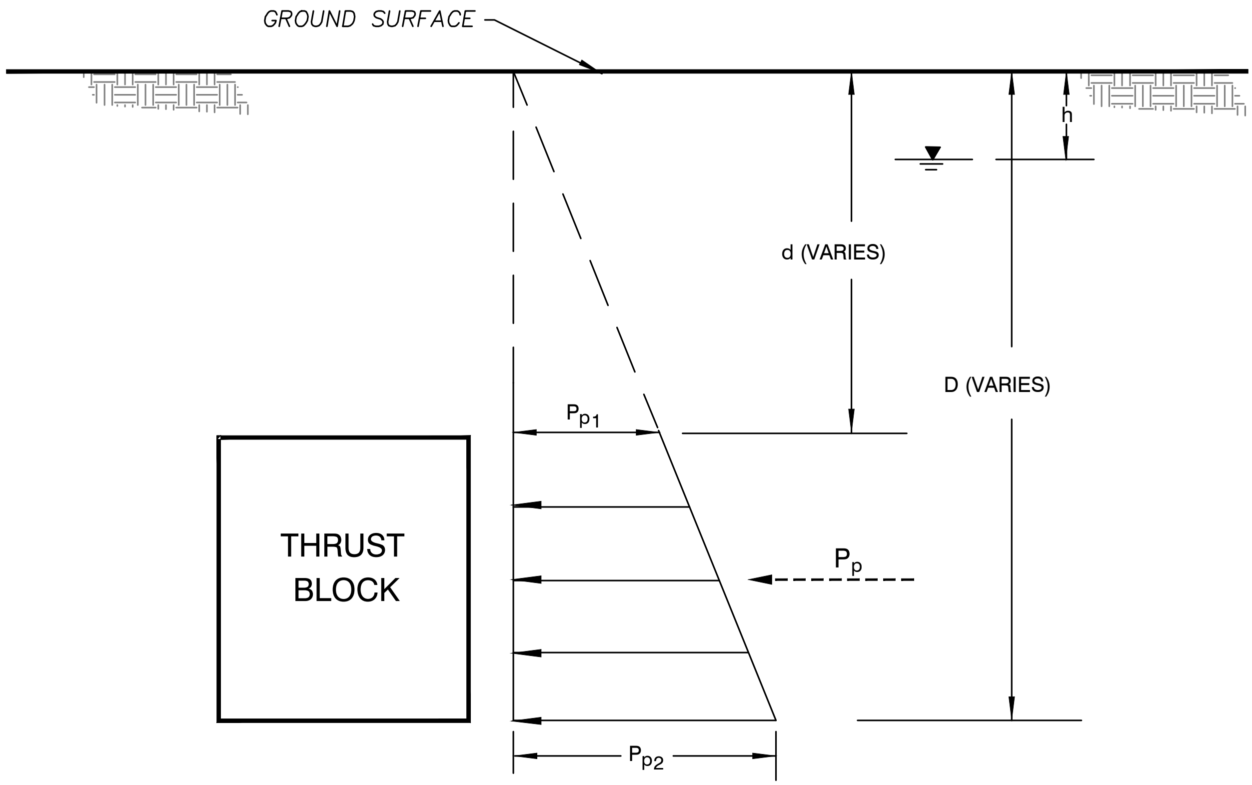
Geotechnical & Environmental Sciences Consultants



NOTES:

1. GROUNDWATER BELOW BLOCK

   $P_p = 180\ (D^2 - d^2)$ 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

6 108505001 D-TB.DWG AOB

NOT TO SCALE

FIGURE 6

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

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OMWD NEIGHBORHOOD NO. 1 SEWER PUMP STATION
16106 4S RANCH PARKWAY, SAN DIEGO, CALIFORNIA

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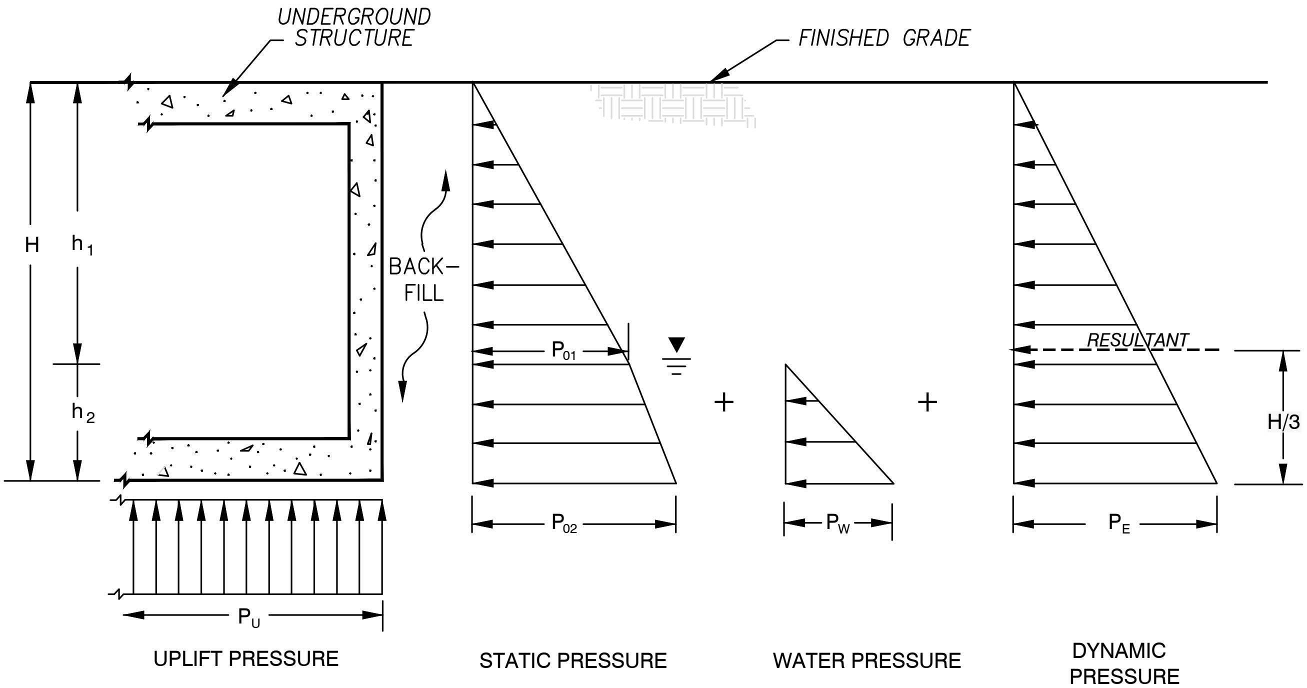

NOTES:

1. APPARENT LATERAL EARTH PRESSURES, P_{01} AND P_{02}
   $P_{01} = 60\ h_1$ psf
   $P_{02} = 60\ h_1 + 29\ h_2$ psf
2. WATER PRESSURE, P_w
   $P_w = 62.4\ h_2$ psf
3. DYNAMIC LATERAL EARTH PRESSURE, P_E, IS BASED ON A PEAK GROUND ACCELERATION, OF 0.42 g
   $P_E = 20\ H$ psf
4. P_E IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010).
5. UPLIFT PRESSURE, P_u
   $P_u = 62.4\ h_2$ psf
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H, h_1 AND h_2 ARE IN FEET
8. ▼ GROUNDWATER TABLE

NOT TO SCALE

FIGURE 7

LATERAL EARTH PRESSURES FOR UNDERGROUND STRUCTURES

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16106 4S RANCH PARKWAY, 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 methods.

Bulk Samples

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

The Standard Penetration Test (SPT) Sampler

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1⅜ inches. The sampler was driven into the ground 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 3.0 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.

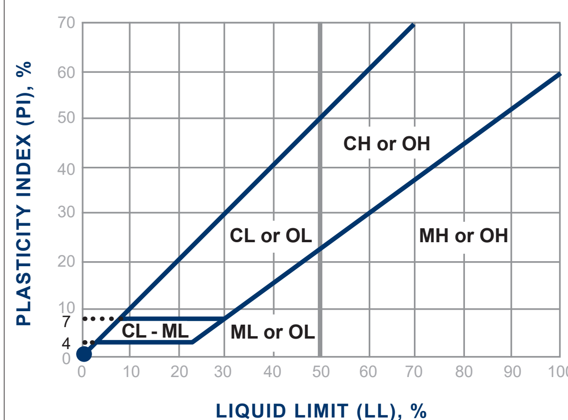
Soil Classification Chart Per ASTM D 2488

Primary Divisions			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

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

DATE DRILLED	1/02/18	BORING NO.	B-1	
GROUND ELEVATION	420' ± (MSL)	SHEET	1	OF 2
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (Baja Exploration)			
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"	
SAMPLED BY	ZH	LOGGED BY ZH	REVIEWED BY	NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0								ASPHALT CONCRETE: Approximately 5 inches thick.
							GP	AGGREGATE BASE: Gray, dry, medium dense, poorly graded GRAVEL; approximately 7 inches thick.
							CL	FILL: Olive brown, moist, stiff, silty CLAY; trace coarse sand and fine gravel.
			11					
							CL	ALLUVIUM: Olive gray, moist, stiff, silty CLAY; trace coarse sand; trace gravel-sized chunks of claystone.
10			7	17.1				
			56	14.7	112.6			FRIARS FORMATION: Olive brown, moist, moderately indurated, sandy CLAYSTONE; some oxidation staining.
								Light brown and olive brown; weakly to moderately indurated; mottled; trace sand.
20			73/11"					
								Light brown and olive brown, moist, moderately to strongly cemented, clayey SANDSTONE; oxidation staining.
			71/9"	12.0	119.7			
30			50/6"					Light brown; trace oxidation-staining.
			50/5"					Light gray; dry to moist; trace cobble.
40								

FIGURE A- 1

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OMWD NEIGHBORHOOD PUMP STATION NO.1
16016 4S RANCH PARKWAY, SAN DIEGO, CALIFORNIA

DATE DRILLED	1/02/18	BORING NO.	B-1
GROUND ELEVATION	420' ± (MSL)	SHEET 2 OF 2	
METHOD OF DRILLING	8" Diameter Hollow Stem Auger (Baja Exploration)		
DRIVE WEIGHT	140 lbs. (Auto-Trip)	DROP	30"
SAMPLED BY	ZH	LOGGED BY ZH	REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
40			50/2"					FRIARS FORMATION: (Continued) Light gray, dry, strongly cemented, clayey SANDSTONE. Total Depth = 40.25 feet. Groundwater not encountered during drilling. Backfilled with approximately 13 cubic feet of grout and capped with black-dyed concrete shortly after drilling on 1/02/18. 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.
50								
60								
70								
80								

DATE DRILLED 1/02/18 BORING NO. B-2

GROUND ELEVATION 420' ± (MSL) SHEET 1 OF 1

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

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

SAMPLED BY ZH LOGGED BY ZH REVIEWED BY NMM

DEPTH (feet)	SAMPLES Bulk	SAMPLES Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	CLASSIFICATION U.S.C.S.	DESCRIPTION/INTERPRETATION
0							ASPHALT CONCRETE: Approximately 5 inches thick.
						GP	AGGREGATE BASE: Gray, dry, medium dense, poorly graded GRAVEL; approximately 7 inches thick.
	X					CL	FILL: Dark gray, moist, firm, silty and sandy CLAY; trace fine gravel. @ 3': Olive brown; trace coarse sand; trace oxidation-staining. @ 4': Olive and brown; mottled.
		X	6			CL	ALLUVIUM: Brown, moist, stiff, silty CLAY. @ 6': Gray.
10		X	22				
							FRIARS FORMATION: Light brown and olive brown, moist, weakly indurated, sandy CLAYSTONE; mottled; weathered.
		X	42				Olive brown; moderately indurated; trace gravel; oxidation-staining.
		X	92/10"	14.2	116.7		Light brown and olive brown; mottled.
20							Total Depth = 19.5 feet. Groundwater not encountered during drilling. Backfilled with approximately 6 cubic feet of grout and capped with black-dyed concrete shortly after drilling on 1/02/18.
							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.
30							
40							

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 borings in Appendix A.

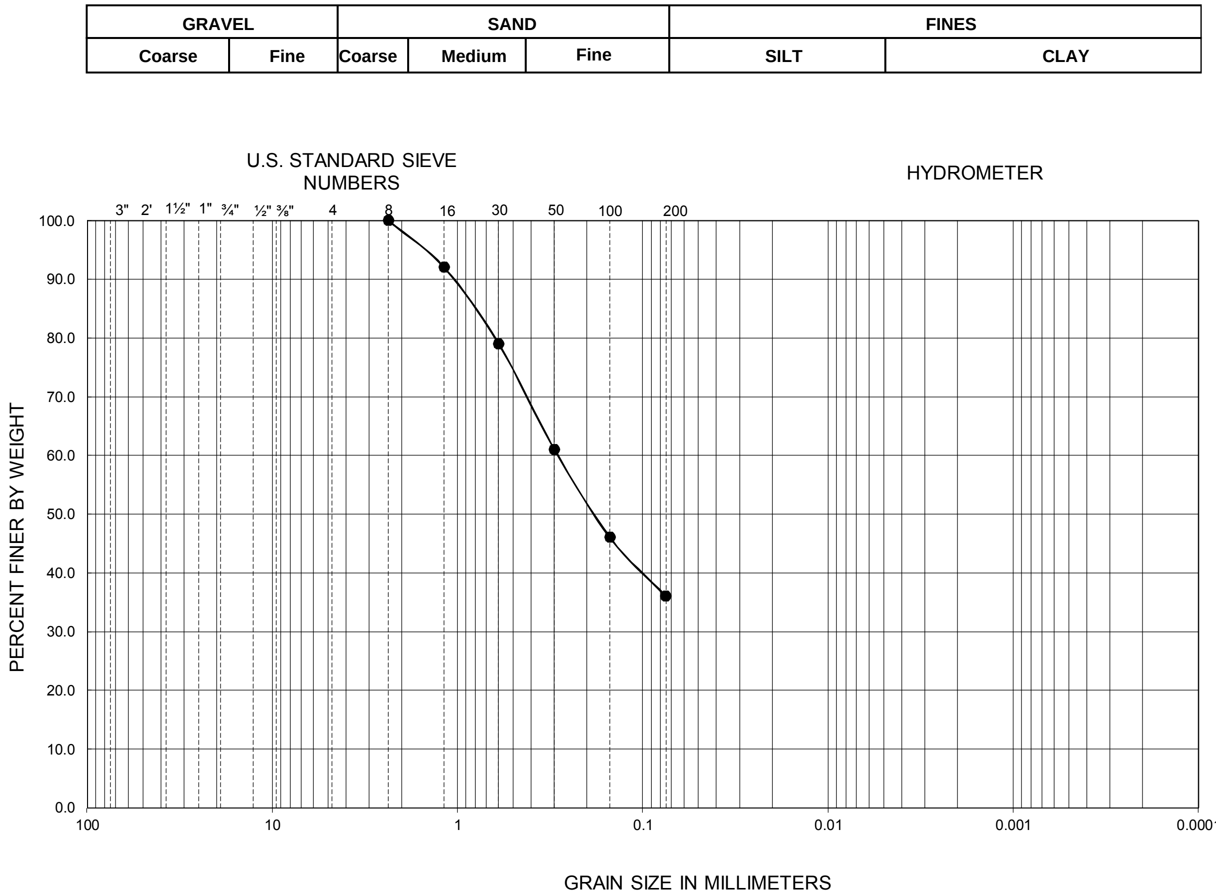
In-Place Moisture and Density Tests
The moisture content and dry density of relatively undisturbed samples obtained from the exploratory borings were evaluated in general accordance with ASTM D 2937. The test results are presented on the logs of the exploratory borings in Appendix A.

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

Direct Shear Tests
Direct shear tests were performed on relatively undisturbed samples in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected materials. The samples were inundated during shearing to represent adverse field conditions. The results are shown on Figure B-2.

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 pounds per square foot and were inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of these tests are presented on Figure B-3.

Soil Corrosivity Tests
Soil pH and resistivity tests were performed on representative samples in general accordance with CT 643. The soluble sulfate and chloride contents of the selected samples were evaluated in general accordance with CT 417 and CT 422, respectively. The test results are presented on Figure B-4.



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	25.0-26.5	--	--	--	--	--	--	--	--	36	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

GRADATION TEST RESULTS

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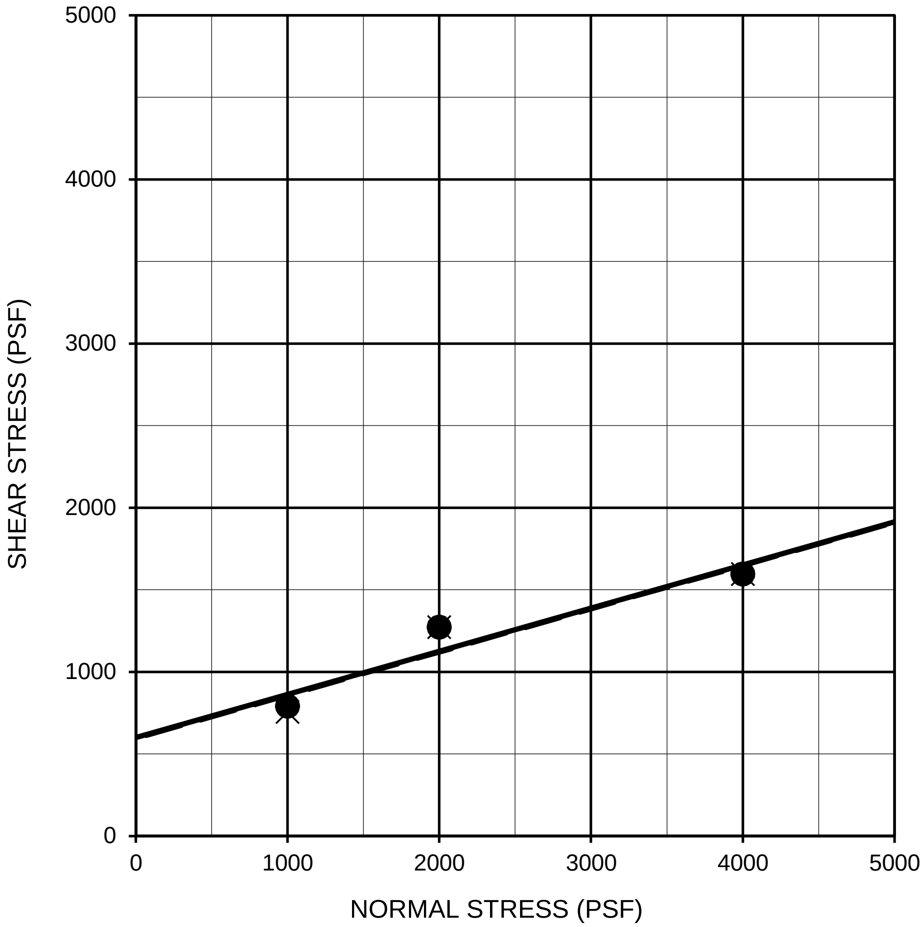

Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Sandy CLAY	——●——	B-1	5.0-6.5	Peak	600	15	CL
Sandy CLAY	– – X – –	B-1	5.0-6.5	Ultimate	590	15	CL

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

FIGURE B-2

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DIRECT SHEAR TEST RESULTS

OMWD NEIGHBORHOOD PUMP STATION NO. 1
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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-1	1.0-5.0	9.5	110.0	24.3	0.060	60	Medium

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

FIGURE B-3

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EXPANSION INDEX TEST RESULTS

OMWD NEIGHBORHOOD PUMP STATION NO. 1
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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	1.0-5.0	8.2	590	470	0.047	255

[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-4

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CORROSIVITY TEST RESULTS

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