

# Geotechnical Evaluation

Sycamore Canyon School

Building Additions

10201 Settle Road

Santee, California

Santee School District

9625 Cuyamaca Street | Santee, California 92071

May 30, 2019 | Project No. 108773001



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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Ms. Christina Becker  
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# 1 INTRODUCTION

In accordance with your authorization and Notice to Proceed dated March 8, 2019, we performed an update geotechnical evaluation for the proposed improvements at the Sycamore Canyon School campus located at 10201 Settle Road in Santee, California (Figure 1). The purposes of our services were to update our original geotechnical report (Ninyo & Moore, 2007) to the current 2016 California Building Code requirements and to update our infiltration testing report (Ninyo & Moore, 2017) to local guidelines for site infiltration to address storm water management. Additionally, this updated report addresses proposed site improvements that were not previously included in our original geotechnical report (Ninyo & Moore, 2007). Our geotechnical evaluation performed in general accordance with Chapter 18A of Title 24, Part 2, Volumes 1 and 2 of the 2016 California Building Code (CBC) and California Geological Survey (CGS) Note 48. This report presents the results of our field explorations and laboratory testing as well as our conclusions regarding the geotechnical conditions at the site and our recommendations for the design and construction of this project. The recommendations presented in this report supersede previous recommendations in our original geotechnical report (Ninyo & Moore, 2007).

## 2 SCOPE OF SERVICES

Our scope of services for this evaluation included the following:

- Reviewing readily available published and in-house geotechnical literature including our original geotechnical report (Ninyo & Moore, 2007), response to CGS review comments (Ninyo & Moore, 2008), and infiltration testing report for the school campus (Ninyo & Moore, 2017), topographic maps, geologic maps, fault maps, stereoscopic aerial photographs, and preliminary site plans (StudioWC, 2019).
- Performing a field reconnaissance to observe existing site conditions and to mark the locations of our exploratory borings.
- Coordinating with Santee School District personnel to locate underground utilities near our exploratory borings. Additionally, we notified Underground Service Alert (USA).
- Performing a subsurface exploration consisting of the drilling, logging, and sampling of seven (7) exploratory borings using a truck-mounted drill rig equipped with hollow-stem augers and manual techniques. Relatively undisturbed and bulk soil samples were obtained at selected intervals from the borings. The collected samples were transported to our in-house geotechnical laboratory for testing.
- Performing infiltration tests within two (2) boring locations.
- Performing geotechnical laboratory testing on representative soil samples to evaluate design parameters and soil characteristics.

- Compiling and performing engineering analysis of the data obtained from our background review, our subsurface evaluations, and geotechnical laboratory testing.
- Preparing this updated report presenting our findings, conclusions, and recommendations regarding the geotechnical aspects of the design and construction of the project.

### **3 SITE AND PROJECT DESCRIPTION**

The project site is situated within the existing school campus for the Sycamore Canyon School in Santee, California (Figure 1). In general, the campus is located on an irregular-shaped parcel bound by Settle Road to the east and single-family residential properties to the north, south, and west. The school site generally consists of school buildings, facilities, parking lots in the eastern portion of the campus, with an athletic field and hardscape areas in the western and northern portions of the property. The existing school buildings are at an approximate elevation of 425 feet above mean sea level (MSL), the athletic field and hardscape areas to the west of the buildings are at an approximate elevation of 410 feet above MSL. Slopes descend away from the site to the north, west, and south, and ascend from the site to the east. Review of historical topographic maps indicates that a canyon once existed at the southeastern portion of the site. This canyon feature has since been filled in to create the current topography at the site. The global coordinates of the project site are approximately 32.863°N Latitude and -117.004°W Longitude.

Our office previously performed a geotechnical evaluation at the project site (Ninyo & Moore, 2007). Based on our discussions with you and the Project Architect, and our review of the project plans (StudioWC, 2019), the size and location of the proposed building addition addressed in our original geotechnical evaluation report (Ninyo & Moore, 2007) has remained the same. The proposed building includes the construction of a new approximately 5,700 square-foot (sf), single story library building and an approximately 500 sf addition to the southeast corner of Building C.

The project will also include the construction of a parking lot expansion at the western end of the campus and a new access ramp and widened fire access lane near the central portion of the campus. The borings performed as part of our recent subsurface evaluation address these additional site improvements.

Additionally, our office previously performed four infiltration tests at the campus (Ninyo & Moore, 2017). This update evaluation includes the results of those infiltration tests and updates to local guidelines for site infiltration to address storm water management.

## 4 SUBSURFACE EVALUATION

Our recent subsurface exploration was conducted on April 22 and 23, 2019 and included the drilling, logging, and sampling of seven small-diameter borings (NM-1 through NM-5, IT-1, and IT-2). Borings NM-1 through NM-5 were drilled to depths up to approximately 18.5 feet using a truck-mounted drill rig equipped with 8-inch diameter hollow-stem augers. Borings IT-1 and IT-2 were manually excavated to depths up to approximately 5 feet using a 6-inch diameter hand auger. Our previous subsurface explorations were conducted in 2007 and 2017. Two small-diameter borings (B-3 and B-4) were drilled on July 10, 2007, two large-diameter borings (B-1 and B-2) were drilled from July 23 to 25, 2007, and four infiltration tests (SC-1 through SC-4) were excavated on April 13, 2017. Ninyo & Moore personnel logged the borings in general accordance with the Unified Soil Classification System (USCS) and ASTM International (ASTM) Test Method D 2488 by observing drill cuttings and drive samples. Representative bulk and in-place soil samples were collected at selected depths from within the exploratory borings and were transported to our in-house geotechnical laboratory for analysis. The approximate locations of the recent and previous exploratory borings are shown on Figure 2. Logs of the recent and previous borings are included in Appendix A.

## 5 LABORATORY TESTING

Geotechnical laboratory testing for this evaluation was performed on representative soil samples collected from our subsurface exploration. Testing included an evaluation of in-situ dry density and moisture content, gradation, consolidation, shear strength, expansion index, soil corrosivity, and R-value. The results of the in-situ dry density and moisture content tests are presented on the boring logs presented in Appendix A. The results of the other laboratory tests that we performed are presented in Appendix B. Laboratory test results from our previous 2007 evaluation are also included in Appendix B.

## 6 INFILTRATION TESTING

As a means of evaluating the infiltration characteristics of near-surface materials, infiltration tests were performed on April 22 and 23, 2019 at two locations designated IT-1 and IT-2 (Figure 2). Additionally, infiltration tests were performed at four locations designated SC-1 through SC-4 (Figure 2) on April 13 and 14, 2017 as part of a previous evaluation (Ninyo & Moore 2017). Borings IT-1 and IT-2 were manually excavated to depths up to approximately 5 feet using a 6-inch diameter hand auger. Borings SC-1 through SC-4 were manually excavated to depths up to approximately 3 feet using a 6-inch diameter hand auger. Following the excavation, both infiltration test locations were prepared by placing approximately 2 inches of gravel on the bottom, installing a 2-inch

diameter perforated polyvinyl chloride (PVC) pipe, and backfilling the annulus with pea gravel. As part of the test procedure, a presoak was performed on April 13, 2017 for SC-1 through SC-4 and on April 22, 2019 IT-1 and IT-2 to represent adverse conditions for infiltration. The presoak consisted of maintaining approximately 1 to 3.5 feet of water in each test boring for approximately 4 hours. The water level was then allowed to drop overnight.

Infiltration testing was performed on April 14, 2017 at borings SC-1 through SC-4, and on April 23, 2019 at borings IT-1 and IT-2 in general accordance with the City of Santee BMP Design Manual (2016). The infiltration test holes were filled with approximately 12 inches of water or the remaining water from the presoak was left in-place and the water depth was measured in 30-minute intervals for the duration of the tests. The test holes were refilled after the 30-minute intervals as needed to restore the initial water level.

Infiltration rates were calculated using the Porchet method. Infiltration tests at borings SC-1 through SC-4 indicated that the observed (i.e., unfactored) infiltration rates were 0.05 inches per hour or less. Borings IT-1 and IT-2 did not infiltrate. Infiltration test results and calculations are included in Appendix C and summarized in Table 1. Per the City of Santee BMP Design Manual Appendix D Section D.5.1, a suitability assessment factor of safety (FOS) of 2.0 was developed. A completed Worksheet C.4-1: Categorization of Infiltration Feasibility Condition Based on Geotechnical Conditions with the appropriate geotechnical aspects is presented in Appendix C. The rates presented in Table 1 are to be used for preliminary design purposes.

Infiltration Test	Approximate Test Depth (feet)	Description	Observed Infiltration Rate (in/hr)	Suitability Assessment Factor of Safety <sup>1</sup>	Reliable/ Factored Infiltration Rate <sup>2</sup> (in/hr)
SC-1	3.0	Sandy CLAY (Fill)	0.02	2.0	0.01
SC-2	3.0	Sandy CLAY (Fill)	0.01	2.0	<0.01
SC-3	3.0	Sandy CLAY (Fill)	0.03	2.0	0.02
SC-4	3.0	Sandy CLAY (Fill)	0.05	2.0	0.03
IT-1	5.0	SANDY Claystone (Friars Formation)	DNI	-	DNI
IT-2	5.0	Sandy CLAY (Fill)	DNI	-	DNI

**Notes:**

in/hr = inches per hour

DNI = did not infiltrate

<sup>1</sup> Design safety factor to be determined by the design engineer in accordance with Appendix D of the City of Santee BMP Design Manual (2016)

<sup>2</sup> Factored infiltration rate shall be divided by the design safety factor to obtain the design infiltration rate.



We note that the in-situ infiltration rates presented in Table 1 represent the infiltration rates at the specific locations and depths indicated in the table. Variation in the infiltration rates can be expected at different depths and/or locations from those shown in the table.

## **7 GEOLOGIC AND SUBSURFACE CONDITIONS**

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

### **7.1 Regional Geologic Setting**

The project site is situated in the coastal foothill section of the Peninsular Ranges Geomorphic Province. The 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. In general, the province 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 (Jennings, 2010). Several of these faults are considered 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 site. Major tectonic activity associated with these and other faults within the regional tectonic framework consists primarily of right-lateral, strike-slip movement. Specifics of faulting are discussed in the following sections of this report.

### **7.2 Site Geology**

Geologic units mapped at the site and encountered during our subsurface exploration include fill, Quaternary-age younger and older alluvial deposits, landslide deposits, and materials of the Tertiary-age Friars Formation (Kennedy and Tan, 2008). Generalized descriptions of the earth units encountered during our field reconnaissance and subsurface exploration are provided in the subsequent sections. Additional descriptions of the subsurface units are provided on the logs of the borings in Appendix A. The geology of the site is shown on Figure 3 and geologic cross sections are shown on Figures 4A and 4B.

## 7.2.1 Encountered Pavement Sections

Asphalt concrete (AC) pavements were encountered during our subsurface exploration in borings NM-1 and NM-2. The pavement sections consisted of AC over decomposed granite (DG) materials used as a base course. Table 2 summarizes the pavement sections as encountered in our borings.

Table 2 – Encountered Pavement Section Thicknesses			
Boring No.	Encountered AC Thickness (inches)	Encountered DG Materials Thickness (inches)	AC Notes
NM-1	4¾	18	<b>Two Layers of AC:</b> Top Layer 4-inches; Bottom Layer ¾-inches
NM-2	7½	-*	<b>Three Layers of AC:</b> Top Layer 3-inches; Second Layer 2½-inches; Bottom Layer 2-inches Petromat encountered between top and second layer.

**Notes:**

\* = Did not encounter

AC = asphalt concrete

DG = decomposed granite

## 7.2.2 Fill

Fill was encountered underlying the AC pavements or at the surface in our recent and previous borings and extended to depths up to approximately 15 feet below the ground surface. As encountered, the fill materials generally consisted of various shades of brown and gray, moist, soft to hard, sandy clay, and loose to very dense, silty sand, clayey sand, and clayey gravel. Scattered amounts of roots, gravel, and cobbles was encountered in the fill. Documentation regarding placement of these fills was not available for our review.

## 7.2.3 Younger Alluvium

Although not encountered in our recent or previous subsurface explorations, younger alluvium has been mapped at the site. These materials are generally expected to consist of firm, sandy clay and medium dense, sandy silt with scattered gravel.

## 7.2.4 Older Alluvium

Although not encountered in our recent or previous subsurface explorations, older alluvium has been mapped at the site. These materials are generally expected to consist of firm, silty clay, sandy clay, and medium dense to dense clayey sand with scattered gravel and cobbles.

### **7.2.5 Landslide Deposits**

Although not encountered in our recent or previous subsurface explorations, landslide deposits have been mapped at the site. These materials are generally expected to dense sandstone, claystone, pulverized claystone in a stiff clay matrix with cobbles.

### **7.2.6 Friars Formation**

Materials of the Tertiary-age Friars Formation were encountered in our recent and previous exploratory borings underlying the fill to the total depths explored. As encountered, the materials generally consisted of various shades of brown, gray and olive, moist to wet, moderately cemented, sandy, clayey, gravelly, cobble conglomerate, moderately to strongly indurated sandy, gravelly claystone and siltstone, and moderately cemented clayey and silty sandstone. Scattered amounts of gravel and cobbles was encountered in the Friars Formation.

## **7.3 Groundwater**

Groundwater was not encountered during our recent subsurface evaluation. Groundwater was encountered between 46 and 59 feet below existing grade during our previous evaluation (Ninyo & Moore, 2007). Groundwater levels can fluctuate due to seasonal variations, groundwater withdrawal or injection, and other factors. Additionally, perched water conditions may be present at the site due to the geologic contact between the fill and formational material and the presence of trench backfill and bedding materials for underground utilities, as these materials tend to act as a conduit for perched water conditions.

## **7.4 Flood Hazard**

Based on review of Federal Emergency Management Agency (FEMA) Mapping Information Platform website (2018), the site is not within a flood zone. Based on review of topographic maps, the site is located approximately 0.2 miles east of the Santee Recreational Lakes. The site is located at an elevation approximately 40 to 50 feet above the lakes. The site is also located approximately one mile north of the San Diego River bed that serves as a drainage for the El Capitan and San Vicente Reservoirs and Lake Jennings. The site is located at an elevation approximately 60 to 100 feet above the river bed. Based on this review and our reconnaissance, the potential for dam inundation and significant flooding at the site are not design considerations.

## 7.5 Landsliding

Per Tan (1995), the site is mapped as “most susceptible” to landsliding. In addition, the geologic map (Figure 3) indicates a mapped landslide in the eastern portion of the site. During our original geotechnical evaluation (Ninyo & Moore, 2007), two large diameter borings were drilled then observed and down hole logged by a State of California Certified Engineering Geologist from our firm. During our subsurface evaluation, no evidence of a landslide underlying the project was observed.

In our Response to CGS Comments (Ninyo & Moore, 2008), we discussed the possibility of unidentified landslide-related features in our down-hole logged, large-diameter borings, and in particular, could a claystone layer encountered in the borings represent a basal landslide rupture surface. The subject claystone layer was observed and logged at depth of roughly 15 feet below the existing ground surface in both of the large-diameter borings. Based on our observations, the claystone layer showed no signs that it was disturbed by landsliding. Such indications typically include sheared and polished surfaces, steepened, rotated, or truncated strata, and soft, plastic remolded clay seams. None of these features, or other indications of landsliding, were observed within the subject claystone bed or elsewhere in the large-diameter borings.

Based on our review of referenced geologic maps, literature, topographic maps, and stereoscopic aerial photographs, as well as our subsurface evaluation for the proposed improvements at Sycamore Canyon School, we conclude that the project site is not underlain by landslide deposits. It has been documented that landslides exist east of (upslope of) the project site, but that these landslides have been graded and stabilized. As such, the potential for significant large-scale slope instability at the site is not a design consideration.

## 7.6 Faulting and Seismicity

Based on our review of the referenced geologic maps and stereoscopic aerial photographs, as well as on our geologic review, the 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 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 structure. Figure 5 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 12.7 miles west of the site. Table 3 lists selected principal known

active faults that may affect the site and the maximum moment magnitude  $M_{max}$  calculated from the USGS National Seismic Hazard Maps - Fault Parameters website (USGS, 2008).

<b>Fault</b>	<b>Approximate Fault-to-Site Distance miles (kilometers)</b>	<b>Maximum Moment Magnitude (<math>M_{max}</math>)</b>
Rose Canyon	12.7 (20.4)	6.9
Coronado Bank	25.7 (41.4)	7.4
Elsinore (Julian Segment)	28.2 (45.4)	7.4
Newport-Inglewood (Offshore Segment)	31.9 (51.3)	7.0
Earthquake Valley	32.7 (52.6)	6.8
Elsinore (Temecula Segment)	32.9 (53.0)	7.1
Elsinore (Coyote Mountain Segment)	39.6 (63.7)	6.9
San Jacinto (Coyote Creek Segment)	49.1 (79.0)	7.0
San Jacinto (Clark Segment)	51.7 (83.2)	7.1
San Jacinto (Anza Segment)	51.7 (83.3)	7.3
San Jacinto (Borrego Segment)	52.1 (83.9)	6.8
Elsinore (Glen Ivy Segment)	54.9 (88.4)	6.9
Palos Verdes	60.2 (96.9)	7.3
San Jacinto (San Jacinto Segment)	61.2 (98.5)	6.9

### **7.6.1 Strong Ground Motion**

Based on our review of background information, data pertaining to the historical seismicity of the San Diego area are summarized in Table 4 below. This table presents historic earthquakes within a radius of 62 miles (100 kilometers) of the site with a magnitude 5.5 or greater.

<b>Date</b>	<b>Magnitude (M)</b>	<b>Approximate Epicentral Distance miles (kilometers)</b>
November 22, 1800	6.5	19 (31)
May 27, 1862	5.9	16 (25)
February 9, 1890	6.3	55 (88)
February 24, 1892	6.7	42 (68)
May 28, 1892	6.3	52 (83)
October 23, 1894	5.7	12 (20)
September 30, 1916	5.0	52 (84)
January 1, 1920	5.0	29 (46)
November 25, 1934	5.0	57 (92)
March 25, 1937	6.0	57 (91)
June 4, 1940	5.1	34 (55)
October 21, 1942	6.5	60 (96)
August 15, 1945	5.7	58 (94)
November 4, 1949	5.7	52 (84)
March 19, 1954	6.2	55 (89)



**Table 4 – Historical Earthquakes that Affected the Site**

Date	Magnitude (M)	Approximate Epicentral Distance miles (kilometers)
September 23, 1963	5.0	58 (94)
April 9, 1968	6.4	55 (89)
April 28, 1969	5.8	50 (81)
January 12, 1975	5.1	56 (90)
February 25, 1980	5.6	52 (83)
July 13, 1986	5.8	50 (81)
October 31, 2001	5.2	52 (84)
June 12, 2005	5.2	52 (84)

The 2016 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 0.40g using the Structural Engineers Association of California (SEAOC) and Office of Statewide Health Planning and Development (OSHPD) (SEAOC and OSHPD, 2019) seismic design tool (web-based).

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 0.39 using the OSHPD (SEAOC and OSHPD, 2019) seismic design tool that yielded a mapped  $MCE_G$  peak ground acceleration of 0.33g for the site and a site coefficient ( $F_{PGA}$ ) of 1.17 for Site Class D.

### 7.6.2 Ground Rupture

Based on our review of the referenced literature and our site reconnaissance, active faults are not known to cross the project vicinity. Therefore, the potential for ground surface rupture due to faulting at the site is considered low. However, lurching or cracking of the ground surface as a result of nearby seismic events is possible.

### **7.6.3 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 generally dense nature of the subsurface materials and the depth to groundwater, it is our opinion that the potential for liquefaction to occur at the site is not a design consideration.

### **7.6.4 Tsunamis**

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

## **8 CONCLUSIONS**

Based on our review of the referenced background data, subsurface evaluations, and laboratory testing, it is our opinion that construction of the proposed 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 overlying materials of the Friars Formation.
- The onsite material should be generally excavatable with heavy-duty earth moving equipment in good working condition. Zones containing gravel and cobbles may be encountered and additional efforts including heavy ripping should be anticipated.
- Due to the large amounts of cobble and the expansive nature of the clayey soils, fill materials encountered in our exploratory borings are considered unsuitable for structural support. Recommendations for removal of the existing fill materials within the building pad areas and replacement with imported granular soil are presented in the following sections.

- Fill and formational materials encountered at the site were observed to contain cobbles. Additional processing or screening of the material should be anticipated prior to reuse as compacted fill.
- Groundwater was not encountered during this subsurface evaluation. Groundwater was encountered between depths of approximately 46 and 59 feet below existing grade during our previous evaluation (Ninyo & Moore, 2007). However, perched water conditions may be encountered in such areas as existing utility trenches and the geologic contact between fill and the formational materials.
- The subject site is not located within a State of California Earthquake Fault Zone (Alquist-Priolo Special Studies Zone). However, the closest known major active fault is the Rose Canyon Fault, which is located approximately 12.7 miles west of the project.
- Based on the results of our limited geotechnical laboratory testing, the onsite soils exhibit a very low to medium expansion potential. Our recommendations presented herein include consideration for the expansive nature of onsite soils.
- Based on the results of our limited geotechnical laboratory testing the Caltrans (2018) corrosion guidelines, we consider the onsite soils to be corrosive.
- It has been documented that landslides exist east of (upslope of) the project site, but that these landslides have been graded and stabilized. As such, the potential for significant large-scale slope instability at the site is not a design consideration.

## 9 RECOMMENDATIONS

Based on our understanding of the project, the following recommendations are provided for the design and construction of the project. The proposed site improvements should be constructed in accordance with the requirements of the applicable governing agencies.

### 9.1 Earthwork

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

#### 9.1.1 Site Preparation

Site preparation should begin with the removal of flatwork, 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.

### 9.1.2 Excavation Characteristics

The results of our field exploration program indicate that the project site, as presently proposed, is underlain by fill, younger alluvium, older alluvium, and Friars Formation. These materials should be generally excavatable with heavy-duty earth moving equipment in good working condition. Zones containing gravel and cobbles may be encountered and additional efforts including heavy ripping should be anticipated. Excavations may generate oversize material and additional processing and handling of these materials, including screening and/or rock picking, should be anticipated.

### 9.1.3 Temporary Excavations

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

<i>Fill, Younger and Older Alluvium</i>	<i>Type C</i>
<i>Friars Formation</i>	<i>Type B</i>

Upon making the excavations, the soil classifications and excavation performance should be evaluated in the field by the geotechnical consultant in accordance with the OSHA regulations. Temporary excavations should be constructed in accordance with OSHA recommendations. For trench or other excavations, OSHA requirements regarding personnel safety should be met using appropriate shoring (including trench boxes) or by laying back the slopes to no steeper than 1.5:1 (horizontal to vertical) in fill, and younger and older alluvium, and 1:1 (horizontal to vertical) in Friars Formation. Excavations encountering seepage should be evaluated on a case-by-case basis. Onsite safety of personnel is the responsibility of the contractor.

### 9.1.4 Excavation Bottom Stability

The bottoms of the excavations may be soft and/or unstable especially if exposed to wetting. In general, soft and/or unstable bottom conditions may be mitigated by scarifying and recompacting the exposed bottom or overexcavating the excavation bottom to suitable depths (as evaluated in the field by Ninyo & Moore's representative) and replacing with granular fill or aggregate base materials reinforced with geosynthetic fabrics and/or geogrid. Specific recommendations for stabilizing excavation bottoms should be based on evaluation in the field by Ninyo & Moore at the time of construction.

### **9.1.5 Remedial Grading – Building Pads**

In order to provide suitable support for the proposed new building and addition, we recommend that the existing undocumented fills within the building pads be over-excavated and removed down to competent formational materials or to a depth of 2 feet below foundations, whichever is deeper. For the purposes of this report, the building pad is defined as the structural footprint of the building (including foundations for attached overhangs, canopies, and other building appurtenances) plus a horizontal distance of 5 feet, where feasible.

Subsequent to removal, the resulting surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompact 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 recompact, the overexcavation should be backfilled with generally granular soils that possess a very low to low expansion potential (i.e., an expansion index [EI] less than 50). Due to the presence of existing onsite soils that possess a medium potential for expansion, import soils will be needed to backfill the building pad overexcavations.

### **9.1.6 Remedial Grading – Retaining Wall**

We recommend that the existing foundation subgrade soils at the site be removed down to a depth of 1 foot below the bottom of retaining wall footings. This over excavation should extend to the horizontal limits of the retaining wall foundations. The lateral extents of the overexcavation may be modified in the field based on site constraints such as existing structures and property lines. The extent and depths of removals and overexcavations should be evaluated by Ninyo & Moore's representative in the field based on the materials exposed.

The resulting removal surface should be scarified to a depth of approximately 8 inches, moisture conditioned, and recompact to a relative compaction of 90 percent as evaluated by the ASTM D 1557 prior to placing new compacted fill. Once the resulting removal surface has been recompact, the overexcavation should be backfilled with onsite soils that possess a very low to low potential (i.e., an EI less than 50). These compacted fill soils should be placed at a relative compaction of 90 percent as evaluated by ASTM D 1557.



### **9.1.7 Remedial Grading – Pedestrian Concrete Paving**

In the proposed pedestrian concrete paving and exterior flatwork areas, we recommend that the onsite subgrade soils be scarified 8 inches, moisture conditioned, and recompact to a relative compaction of 90 percent as evaluated by ASTM D 1557. The proposed scarification and recompact should extend outward horizontally 2 feet from the exterior limits of the hardscaping, where feasible. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed.

In the event soils that possess a medium to high potential for expansion (i.e., an EI of 50 or more) are exposed at the subgrade elevation for proposed pedestrian concrete paving and exterior flatwork areas, we recommend that those expansive soils be overexcavated and removed. We recommend that the subgrade soils be overexcavated to a depth of 1 foot below the planned subgrade elevation for the proposed pedestrian concrete paving and exterior flatwork areas. The proposed overexcavations should extend outward horizontally 2 feet from the exterior limits of the pavement, where feasible. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed. The resulting surface should be scarified 6 inches, moisture conditioned, and recompact to a relative compaction of 90 percent as evaluated by ASTM D 1557. The removals should then be filled with soils that possess a low to very low potential for expansion (i.e., an EI less than 50). These compacted fill soils should be placed at a relative compaction of 90 percent as evaluated by ASTM D 1557.

Note, some onsite soils possess a medium potential for expansion. These materials are not considered suitable for reuse as compacted fill within 1 foot below the subgrade elevation for pedestrian concrete paving and exterior flatwork areas.

### **9.1.8 Remedial Grading – Vehicular Pavements**

In the proposed vehicular pavement areas, we recommend that the onsite soils be overexcavated to a depth of 1 foot below the planned subgrade elevation for the pavement. The proposed overexcavations should extend outward horizontally 2 feet from the exterior limits of the pavement, where feasible. The extent and depth of removals should be evaluated by Ninyo & Moore's representative in the field based on the material exposed. The resulting surface should be scarified 6 inches, moisture conditioned, and recompact to a relative compaction of 90 percent as evaluated by ASTM D 1557. The removals should then be filled with onsite soils suitable for reuse as compacted fill. The upper 12 inches of the subgrade materials should be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557.

### 9.1.9 Materials for Fill

Materials for fill may be obtained from onsite excavations or import sources. Fill soils should possess an organic content of less than approximately 3 percent by volume (or 1 percent by weight). In general, fill material should not contain rocks or lumps over approximately 3 inches in diameter, and not more than approximately 30 percent larger than ¾ inch.

As noted earlier, expansion index testing presented in Appendix B from our previous evaluation at the site (Ninyo & Moore, 2007) indicates that some of the onsite soils possess a medium potential for expansion. Soils that possess a medium to high potential for expansion (i.e., an expansion index of 50 or more) are not suitable for reuse within building pads, in the upper 1 foot beneath retaining wall footings, as retaining wall backfill, or as the upper 1 foot of subgrade soils beneath pedestrian concrete paving. These materials should be selectively graded for use in non-structural areas, may be blended with other soils to create a mixture that possesses an EI of less than 50, or removed and replaced.

Imported fill material should generally be granular soils with a very low to low expansion potential (i.e., an EI of 50 or less). Import fill material should also be non-corrosive in accordance with the Caltrans (2018) corrosion guidelines. Non-corrosive soils are soils that possess an electrical resistivity more than 1,100 ohm-centimeters (ohm-cm), a chloride content less than 500 parts per million (ppm), less than 0.15 percent sulfates, and a pH less than 5.5. Materials for use as fill should be evaluated by Ninyo & Moore's representative prior to filling or importing. To reduce the potential of importing contaminated materials to the site, prior to delivery, soil materials obtained from off-site sources should be sampled and tested in accordance with standard practice (DTSC, 2001). Soils that exhibit a known risk to human health, the environment, or both, should not be imported to the site.

Additionally, concrete and AC materials generated from the demolition of the existing improvements may be crushed and reused within the fill materials. These materials are considered suitable, provided they are processed and mixed with onsite soils to meet the gradation recommendations provided above. However, materials containing crushed AC should not be placed within the building pad. In areas of landscaping, the landscape architect should be consulted regarding the use of recycled AC and concrete materials within the fill.

### **9.1.10 Compacted Fill**

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 moisture contents generally at or slightly 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 this office 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 at or slightly 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 at or slightly above the laboratory optimum, 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 the subgrade materials beneath vehicular pavements should be compacted to a relative compaction of 95 percent relative density as evaluated by ASTM D 1557. Successive lifts should be treated in a like manner until the desired finished grades are achieved.

### **9.1.11 Pipe Bedding and Modulus of Soil Reaction ( $E'$ )**

It is our recommendation that new pipelines (pipes), where constructed in open excavations, 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 more.

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 1,400 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.

### **9.1.12 Pipe Zone Backfill**

The pipe zone backfill should be placed on top of the pipe bedding material and extend to 1 foot or more above the top of the pipe in accordance with the recent edition of the Standard Specifications for Public Works Construction (“Greenbook”). Pipe zone backfill should have a Sand Equivalent (SE) of 30 or more, and be placed around the sides and top of the pipe. Silts and clays should not be used as pipe zone backfill. 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.

### **9.1.13 Utility Trench Zone Backfill**

Onsite excavations may generate clayey soils that possess a medium potential for expansion. These expansive clay soils are generally not considered suitable for trench backfill beneath building pads, in the upper 1 foot beneath retaining wall footings, behind retaining walls, or as the upper 1 foot of subgrade soils beneath pedestrian concrete paving. Trench zone backfill material should be generally free of trash, debris, roots, vegetation, or deleterious materials. Additionally, onsite excavations may generate oversized materials. Trench zone backfill should generally be free of rocks or hard lumps of material in excess of 3 inches in diameter. Rocks or hard lumps larger than about 3 inches in diameter should be broken into smaller pieces or should be removed from the site. Oversize materials should be separated from material to be used as trench backfill. Moisture conditioning (including drying and/or mixing) of existing onsite materials is anticipated if reused as trench backfill. Trench zone backfill should be moisture-conditioned to generally at or slightly above the laboratory optimum. Trench zone backfill should be compacted to a relative compaction of

90 percent as evaluated by ASTM D 1557, except for the upper 12 inches of the backfill beneath vehicular pavements that should be compacted to a relative compaction of 95 percent as evaluated by ASTM D 1557. Lift thickness for backfill will depend on the type of compaction equipment utilized, but fill should generally be placed in lifts not exceeding 8 inches in loose thickness. Special care should be exercised to avoid damaging the pipe during compaction of the backfill.

#### **9.1.14 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 magnitude and distribution of passive lateral earth pressures presented on Figure 6. Thrust blocks should be backfilled with granular backfill material and compacted following the recommendations presented in this report.

#### **9.1.15 Drainage**

Roof, pad, and slope drainage should be directed such that runoff water is diverted away from slopes and structures to suitable discharge areas by nonerodible devices (e.g., gutters, downspouts, concrete swales, etc.). Positive drainage adjacent to structures should be established and maintained. Positive drainage may be accomplished by providing drainage away from the foundations of the structure at a gradient of 2 percent or steeper for a distance of 5 feet or more outside building perimeters, and further maintained by a graded swale leading to an appropriate outlet, in accordance with the recommendations of the project civil engineer and/or landscape architect.

Surface drainage on the site should be provided so that water is not permitted to pond. A gradient of 2 percent or steeper should be maintained over the pad area 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 5 presents the seismic design parameters for the site in accordance with the CBC (2016) guidelines and adjusted  $MCE_R$  spectral response acceleration parameters (SEAOC and OSHPD, 2019).

Seismic Design Factors	Value
Site Class	D
Site Coefficient, $F_a$	1.148
Site Coefficient, $F_v$	1.716
Mapped Spectral Acceleration at 0.2-second Period, $S_s$	0.879g
Mapped Spectral Acceleration at 1.0-second Period, $S_1$	0.342g
Spectral Acceleration at 0.2-second Period Adjusted for Site Class, $S_{MS}$	1.01g
Spectral Acceleration at 1.0-second Period Adjusted for Site Class, $S_{M1}$	0.587g
Design Spectral Response Acceleration at 0.2-second Period, $S_{DS}$	0.673g
Design Spectral Response Acceleration at 1.0-second Period, $S_{D1}$	0.391g

## 9.3 Foundations

Based on our understanding of the project, the proposed new buildings will be constructed at the site following demolition of the existing improvements. The new buildings are anticipated to be supported on shallow foundations. Recommendations for the shallow building foundations are presented in following sections. 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

Shallow, spread or continuous footings supported on compacted fill may be designed using an allowable bearing capacity of 3,000 pounds per square foot (psf). These allowable bearing capacities may be increased by one-third when considering loads of short duration such as wind or seismic forces. We recommend that shallow foundations for the new additions and building be founded 18 inches below the lowest adjacent grade. Continuous footings should have a width of 18 inches and spread footings should be 24 inches in width. The footings should be reinforced in accordance with the recommendations of the project structural engineer.

### **9.3.2 Lateral Resistance**

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

For frictional resistance to lateral loads, we recommend a coefficient of friction of 0.30 be used between soil and concrete. The passive resistance values may be increased by one-third when considering loads of short duration such as wind or seismic forces.

### **9.3.3 Static Settlement**

We estimate that the proposed structures, designed and constructed as recommended herein, and founded in compacted fill will undergo total settlement on the order of 1 inch. Differential settlement on the order of ½ inch over a horizontal span of 40 feet should be expected. These static settlements are considered to be in addition to the dynamic settlements presented in earlier sections of this report.

## **9.4 Site Retaining Walls**

Site retaining walls that are not a part of or are not connected to the buildings may be supported on continuous footings bearing on competent Friars Formation or compacted fill. The continuous footing should have a width of 24 inches or more and be embedded a depth of 18 inches or more. An allowable bearing capacity of 2,500 psf may be used for the design of site retaining wall foundations. The allowable bearing capacity may be increased by one-third when considering loads of short duration, such as wind or seismic forces.

For the design of a site yielding retaining wall that is not restrained against movement by rigid corners or structural connections, lateral pressures are presented on Figure 7. Site restrained walls (non-yielding) may be designed for lateral pressures presented on Figure 8. These pressures assume select backfill materials are used and free draining conditions. Measures should be taken to reduce the potential for build-up of moisture behind the retaining walls. A drain should be provided behind the retaining wall as shown on Figure 9. The drain should be connected to an appropriate outlet.

## 9.5 Interior Slabs-on-Grade

If interior concrete slabs-on-grade are used, we recommend that conventional, interior concrete slab-on-grade floors be underlain by compacted fill materials of generally very low to low expansion potential (i.e. an expansion index of 50 or less). Interior concrete slabs-on-grade should be 5 inches thick. 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 10-mil polyethylene (or equivalent) membrane placed over 4 inches of medium to coarse, clean sand or pea gravel. The slabs-on-grade should be reinforced with No. 4 reinforcing bars spaced 18 inches on center each way. The reinforcing bars should be placed near the middle of the slab. As a means to help reduce shrinkage cracks, we recommend that the slabs be provided with crack-control joints at intervals of approximately 12 feet each way. The slab reinforcement and expansion joint spacing should be designed by the project structural engineer.

## 9.6 Concrete Flatwork

We recommend that exterior concrete flatwork be 4 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 civil engineer. Before placement of concrete, the subgrade soils should be scarified to a depth of 6 inches, moisture conditioned to generally at or slightly above the laboratory optimum moisture content, and compacted to a relative compaction of 90 percent as evaluated by ASTM D 1557. Positive drainage should be established and maintained adjacent to flatwork.

## 9.7 Light Pole and Canopy Foundations

We recommend that light pole and canopy structures be supported on cast-in-drilled-hole (CIDH) piles. Light pole structures typically impose relatively light axial loads on foundations. Although we anticipate that pile dimensions will be generally controlled by the lateral load demand, we recommend that such drilled foundations have a diameter of 18 inches or more. The pile dimensions (i.e., diameter and embedment) should be evaluated by the project structural engineer.

The drilled pile construction should be observed by Ninyo & Moore during construction to evaluate if the piles have been extended to the design depths. 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. If the CIDH piles extend into groundwater or seepage, the contractor should consider appropriate measures during construction to reduce the potential for caving of the drilled holes, including the use of steel casing and/or drilling mud. In addition, we recommend concrete be placed by tremie method, to see that the aggregate and cement do not segregate during concrete placement, on the same day the CIDH piles are drilled.

For resistance of light pole footings to lateral loads, we recommend an allowable passive pressure of 300 psf per foot of depth be used, with an upper bound value of up to 3,000 psf. This value assumes that the light poles are designed to tolerate ½ inch of deflection at the surface and 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 0.35 be used between soil and concrete. The allowable lateral resistance values may be increased by 1/3 during short term loading conditions, such as wind or seismic loading.

## **9.8 Preliminary Flexible Pavement Design**

We understand that the project will include the construction of new pavements. Our laboratory testing of near surface soil samples at the project site indicated an R-values of 23 and 40. We have used an R-value of 23, along with estimated design Traffic Indices (TI) of 5, 6, and 7 as the basis of our preliminary flexible pavement design. Actual pavement recommendations should be based on R-value tests performed on bulk samples of the soils that are exposed at the finished subgrade elevations across the site at the completion of the grading operations. The preliminary recommended flexible pavement sections are presented in Table 6.

**Table 6 – Recommended Preliminary Flexible Pavement Sections**

Traffic Index (Pavement Usage)	Design R-Value	Asphalt Concrete Thickness (inches)	Aggregate Base Thickness (inches)
5 (Parking Stalls)	23	3	7
6 (Drive Aisles)	23	3	10
7 (Fire Lanes and Bus Lanes)	23	4	12

As indicated, these values assume TIs of 7.0 or less for site pavements. If traffic loads are different from those assumed, the pavement design should be re-evaluated. In addition, we recommend that the upper 12 inches of the subgrade and aggregate base materials be compacted to a relative compaction of 95 percent relative density as evaluated by the current version of ASTM D 1557.

We suggest that consideration be given to using Portland cement concrete pavements in areas where dumpsters will be stored and where refuse trucks will stop and load. Experience indicates that refuse truck traffic can significantly shorten the useful life of AC sections. We recommend that in these areas, 7 inches of 6000 pounds per square inch (psi) flexural strength Portland cement concrete reinforced with No. 4 bars, 18-inches on center, be placed over 6 inches or more of aggregate base materials compacted to a relative compaction of 95 percent.

## 9.9 Corrosion

Laboratory testing was performed on representative samples of the onsite earth materials to evaluate pH and electrical resistivity, as well as chloride and sulfate contents. The pH and electrical resistivity tests were performed in accordance with California Test (CT) 643 and the sulfate and chloride content tests were performed in accordance with CT 417 and CT 422, respectively. These laboratory test results are presented in Appendix B.

The results of the corrosivity testing indicated electrical resistivities of 740 and 3,790 ohm-cm, soil pH of 7.3 and 7.8, chloride contents of 225 and 260 ppm, and sulfate contents of 0.002 and 0.068 percent (i.e., 20 and 680 ppm). Based on a comparison with the Caltrans corrosion (2018) criteria, the onsite soils would be classified as corrosive. Corrosive soils are defined as soil with an electrical resistivity less than 1,100 ohm-cm, a chloride content more than 500 ppm, more than 0.15 percent sulfates (1,500 ppm), and/or a pH less than 5.5.

## 9.10 Concrete

Concrete in contact with soil or water that contains high concentrations of water-soluble sulfates that can be subject to premature chemical and/or physical deterioration. As noted, the soil sample tested in this evaluation indicated water-soluble sulfate contents of 0.002 and 0.068 percent by weight (i.e., 20 and 680 ppm). Based on the American Concrete Institute (ACI) 318 criteria, the potential for sulfate attack is negligible for water-soluble sulfate contents in soils less than about 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, II/V, or Type V cement for normal weight concrete in contact with soil.

## 10 PERMANENT INFILTRATION DEVICES

As previously discussed, the site subsurface soils at the Sycamore Canyon School campus had factored infiltration rates between less than 0.01 and 0.03 inches per hour within borings SC-1 through SC-4 and did not infiltrate within borings IT-1 and IT-2. Based on our infiltration test results, partial infiltration is not feasible. Therefore, we recommend that the bottom and sides of the infiltration media such as gravel reservoirs for infiltration devices be lined with an impermeable liner to avoid infiltrating storm water into the groundwater table. Additionally, we recommend that the sides the BMPs be lined and that the BMPs be connected to an appropriate outlet by a solid pipe. Additional recommendations and/or considerations should be provided by the project civil engineer.

As previously noted, our testing was specific to the locations and depths documented herein. Other areas of the site may or may not accommodate infiltration of storm water. Additional infiltration testing would be needed in these other areas to evaluate whether infiltration in these areas/depths are feasible. Additionally, the horizontal separations between the proposed basins and existing improvements should be evaluated to check whether the setback requirements presented in Section C.1 of the City of Santee BMP Design Manual (2016) are met.

## 11 PRE-CONSTRUCTION CONFERENCE

We recommend that a pre-construction meeting be held prior to commencement of grading. The owner or his representative, the agency representatives, the architect, the civil engineer, Ninyo & Moore, and the contractor should attend to discuss the plans, the project, and the proposed construction schedule.

## **12 PLAN REVIEW AND CONSTRUCTION OBSERVATION**

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

The recommendations provided in this report are based on the assumption that Ninyo & Moore will provide geotechnical observation and testing services during construction. In the event that it is decided not to utilize the services of Ninyo & Moore during construction, we request that the selected consultant provide the client and Ninyo & Moore with a Division of The State Architect (DSA) 109 form 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.

## **13 LIMITATIONS**

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

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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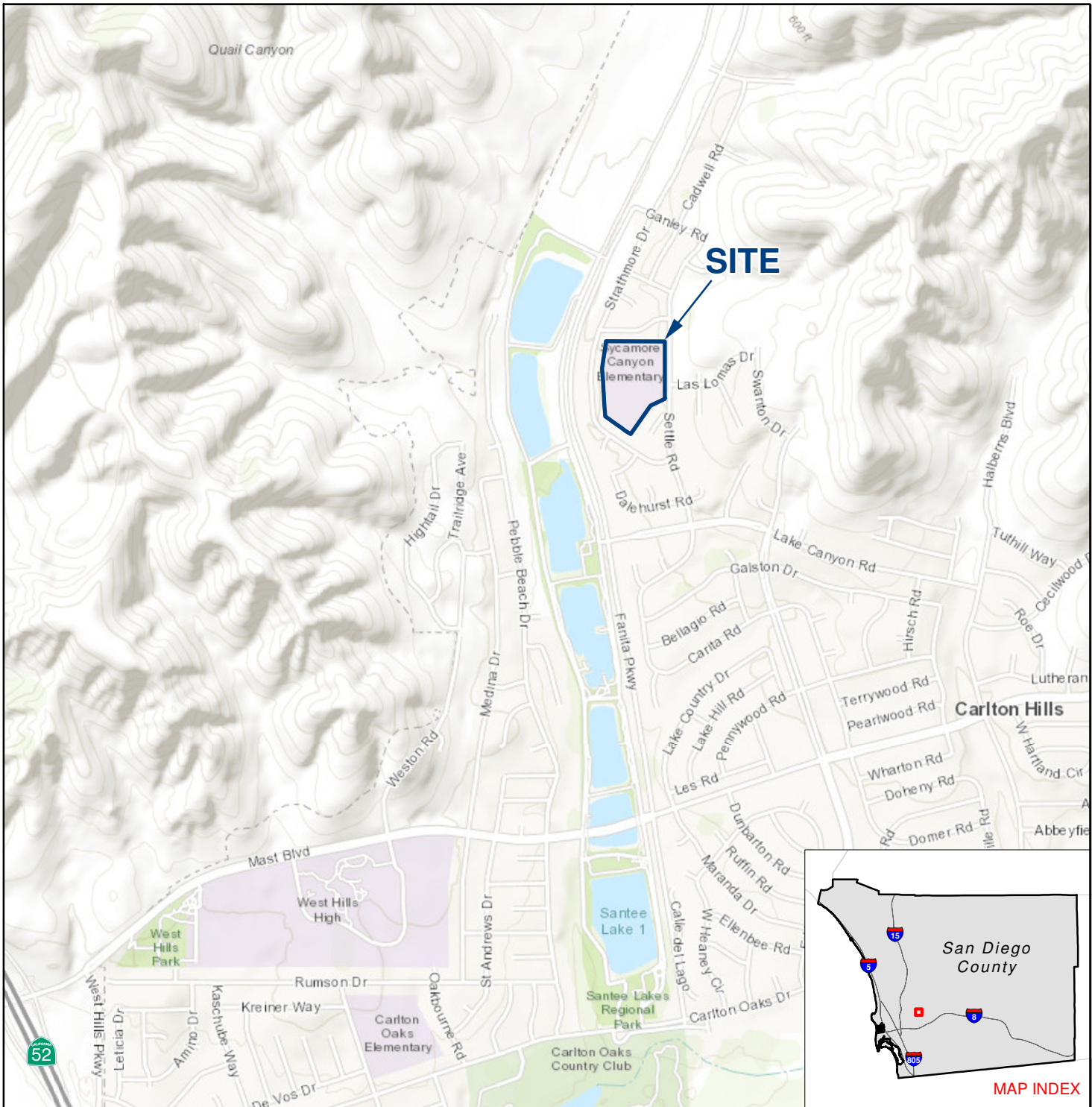
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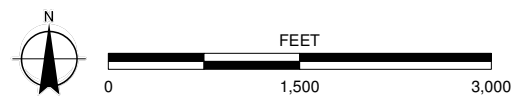


# FIGURES



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NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCE: ESRI WORLD TOPO, 2019



**FIGURE 1**

**SITE LOCATION**

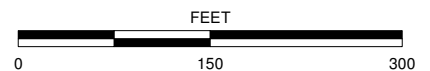
SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA





LEGEND	
<b>NM-5</b> TD=6.5 	BORING TD=TOTAL DEPTH IN FEET
<b>B-4</b> TD=15.4 	PREVIOUS BORING, (NINYO & MOORE, 2007) TD=TOTAL DEPTH IN FEET
<b>IT-2</b> TD=5.0 	INFILTRATION TEST TD=TOTAL DEPTH IN FEET
<b>SC-4</b> TD=3.0 	PREVIOUS INFILTRATION TEST, (NINYO & MOORE, 2017) TD=TOTAL DEPTH IN FEET
	PROPOSED BUILDING ADDITION
	SITE BOUNDARY
	CROSS SECTION
	CROSS SECTION (NINYO & MOORE, 2007)
	INDEX/INTERMEDIATE TOPOGRAPHIC CONTOUR
<b>Qaf</b>	FILL
<b>Tf</b>	FRIARS FORMATION

NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE. | SOURCES: GOOGLE EARTH, 2019; Topographic contours - 2014 USGS QL 2 Lidar, NOAA (coast.noaa.gov), 2019

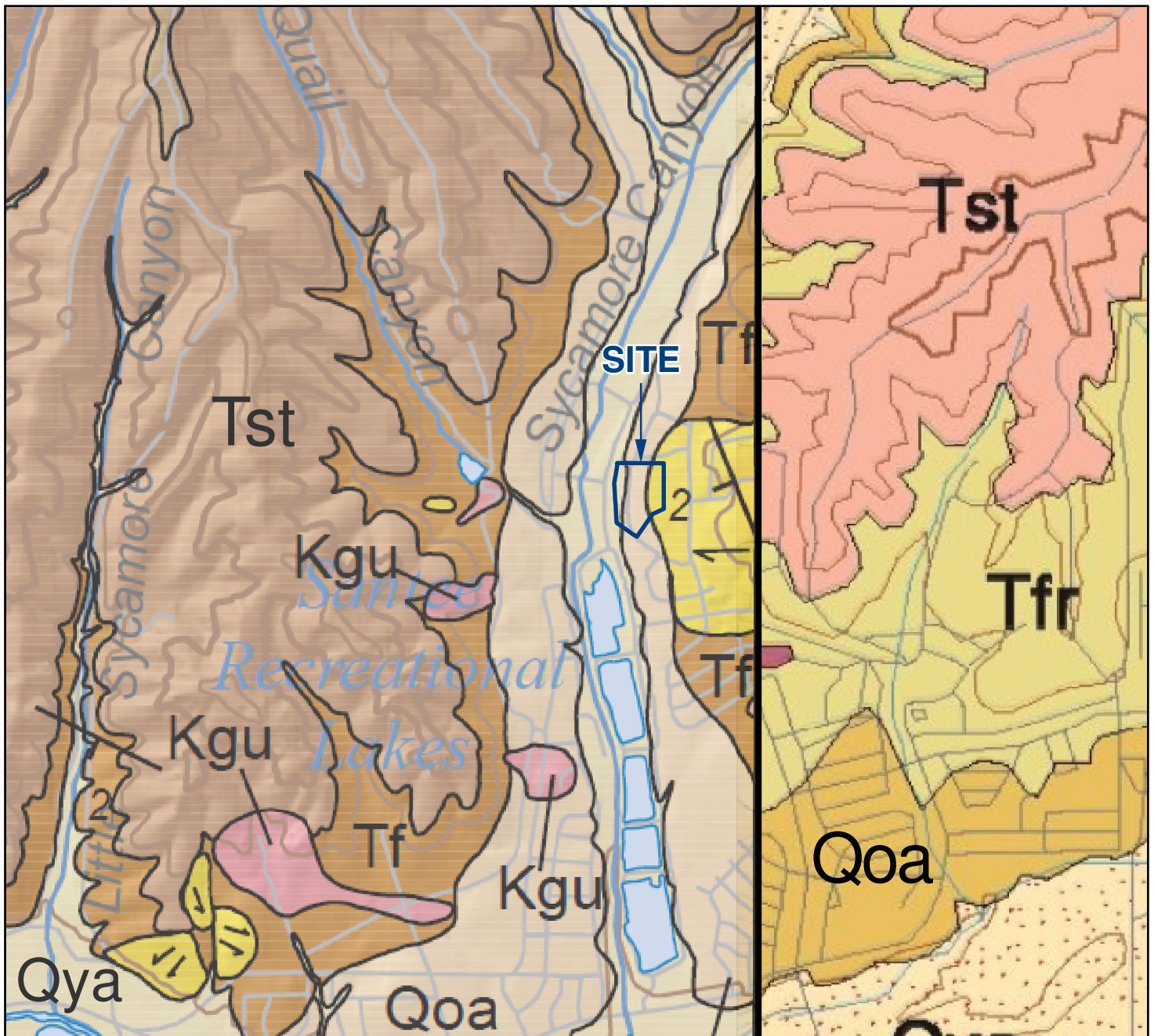


**FIGURE 2**

**BORING LOCATIONS**

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA

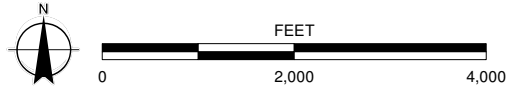




**LEGEND**

	Young alluvial flood-plain deposits (Holocene and late Pleistocene)		<b>Young alluvium (Holocene)</b> —Sand, silt, and gravel in modern streambeds and washes. Includes recent material accumulated on active alluvial fans.
	Old alluvial flood-plain deposits, undivided (late to middle Pleistocene)		<b>Older alluvium (Holocene and Pleistocene)</b> —Sand, silt, and gravel; moderately dissected terraces in stream valleys. Well to poorly bedded, unconsolidated. In places, modern streams incise older alluvium to as much as 15 m. In some areas, older alluvium grades into younger alluvium.
	Friars Formation (middle Eocene)		<b>Stadium Conglomerate (Eocene)</b> —Massive cobble conglomerate having dark-yellowish-brown, coarse-grained sandstone matrix. Moderately well sorted; sandstone beds and lenses make up 50% of unit. Dominant clast type is rhyolitic to dacitic volcanic rocks. Nonmarine in east, nearshore marine and lagoonal in west. Contains late(?) Eocene fossils.
	Mission Valley Formation (middle Eocene)		<b>Friars Formation (Eocene)</b> —Sandstone and claystone; nonmarine and lagoonal. Sandstone typically massive, yellowish-gray, medium grained, and poorly indurated. Conglomerate lenses are fluvial. Maximum thickness is 50 m. Contains marine and nonmarine fossils.
	Granodiorite and tonalite, undivided (mid-Cretaceous)		
	Landslide - Arrows indicate principal direction of movement. Queried where existence is questionable.		

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; TODD, V.R., 2004, PRELIMINARY GEOLOGIC MAP OF THE EL CAJON 30' X 60' QUADRANGLE, SOUTHERN CALIFORNIA,



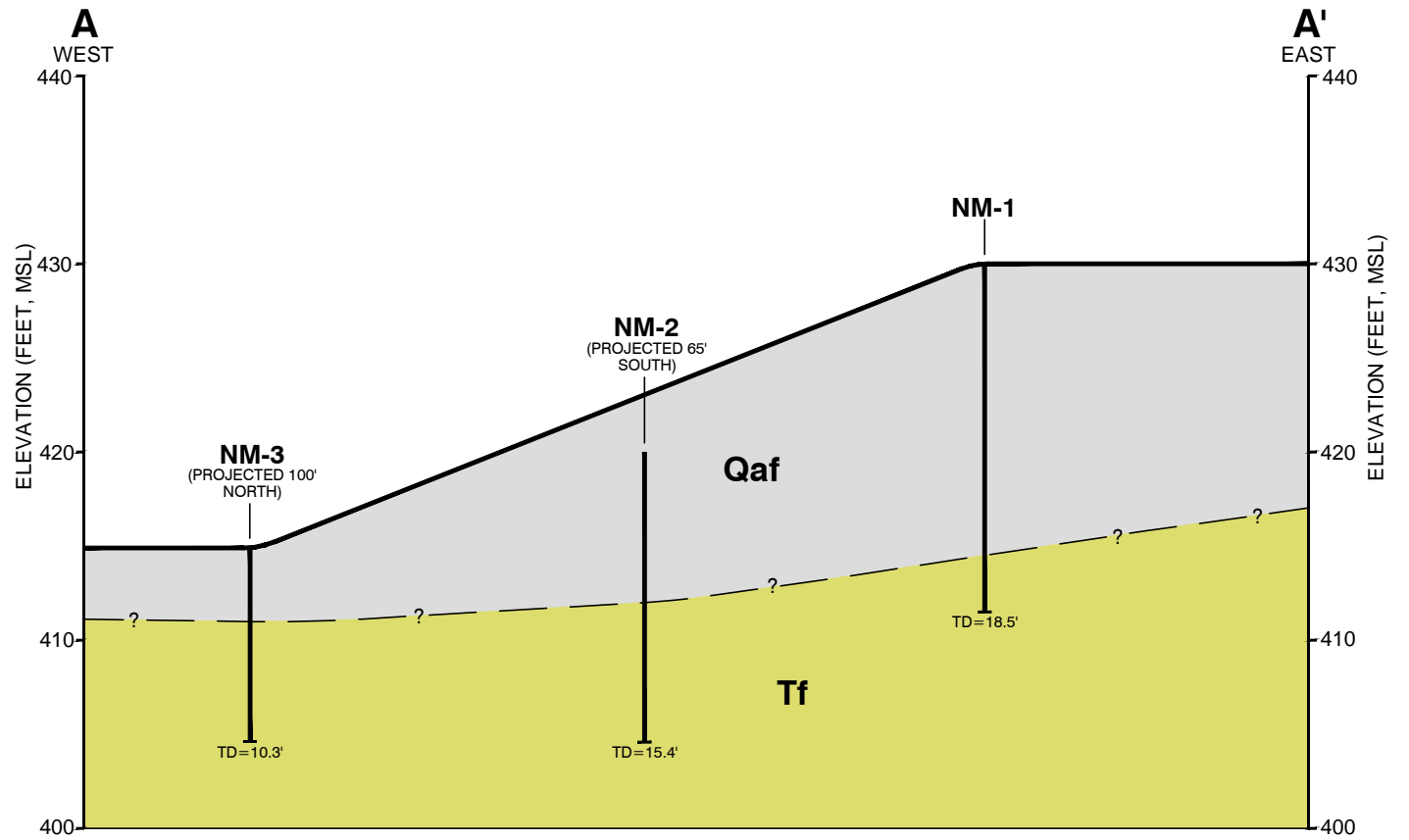
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**FIGURE 3**

**GEOLOGY**

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA

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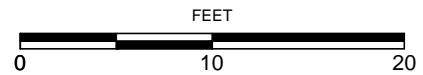
**LEGEND**

**NM-3** BORING  
 TD=TOTAL DEPTH IN FEET  
 TD=10.3'

**Qaf** FILL

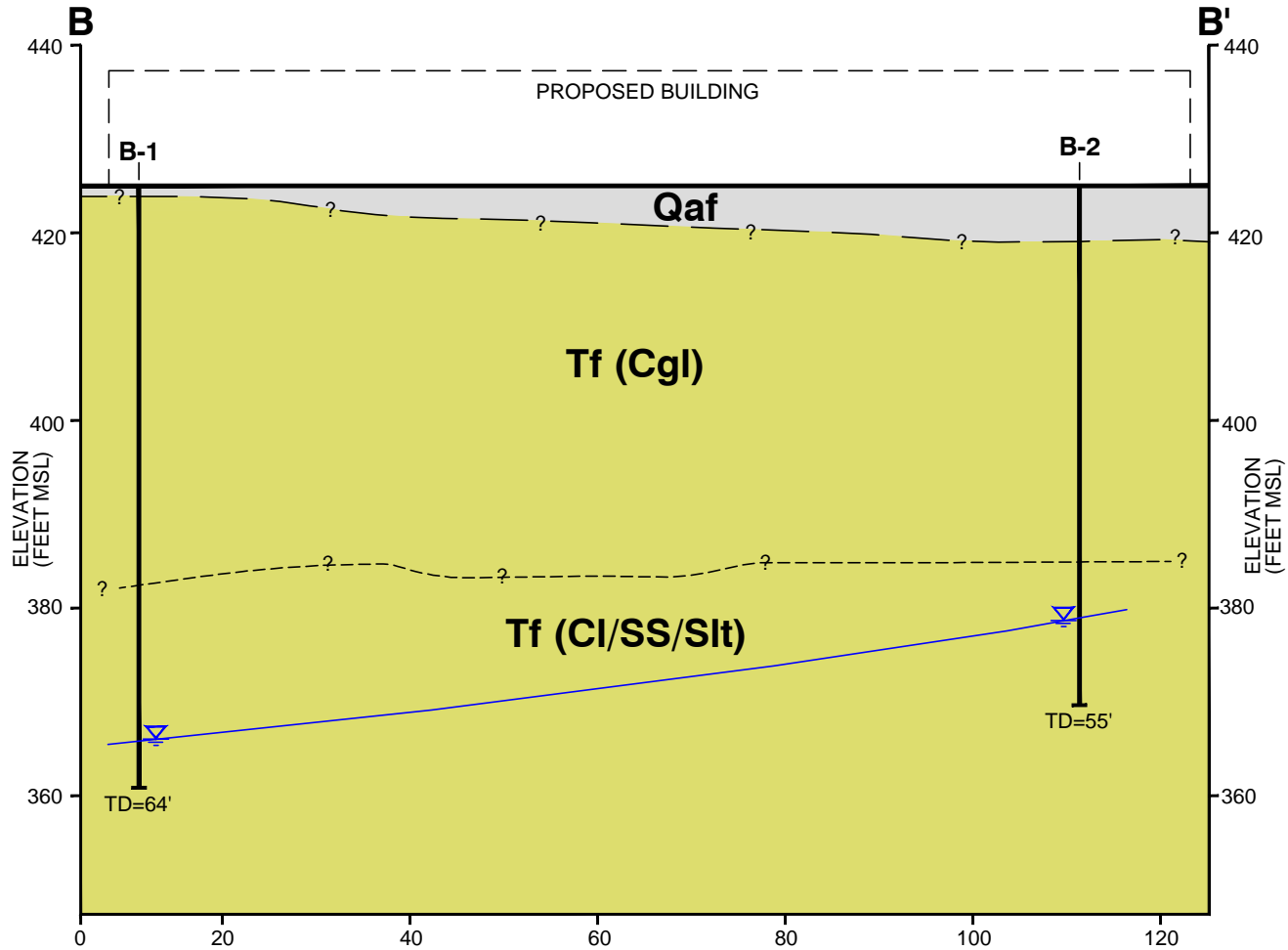
**Tf** FRIARS FORMATION

— ? — GEOLOGIC CONTACT, QUERIED WHERE UNCERTAIN

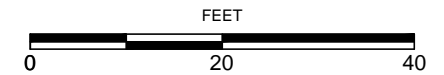


NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

FIGURE 4A



LEGEND	
<b>B-2</b> ↓ TD=55'	PREVIOUS BORING (NINYO & MOORE, 2007) TD=TOTAL DEPTH IN FEET
<b>Qaf</b>	FILL
<b>Tf (Cgl)</b>	FRIARS FORMATION - INTERBEDDED CONGLOMERATE, CLAYSTONE & SANDSTONE
<b>Tf (Cl/SS/Slt)</b>	FRIARS FORMATION - INTERBEDDED CLAYSTONE, SANDSTONE, AND SILTSTONE
— ? —	APPROXIMATE LOCATION OF GEOLOGIC CONTACT QUERIED WERE QUESTIONABLE
- - - ? - - -	APPROXIMATE LOCATION OF VARIATION IN LITHOLOGY WITHIN GEOLOGIC UNIT
— ▽ —	APPROXIMATE LOCATION OF GROUNDWATER ELEVATION



NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

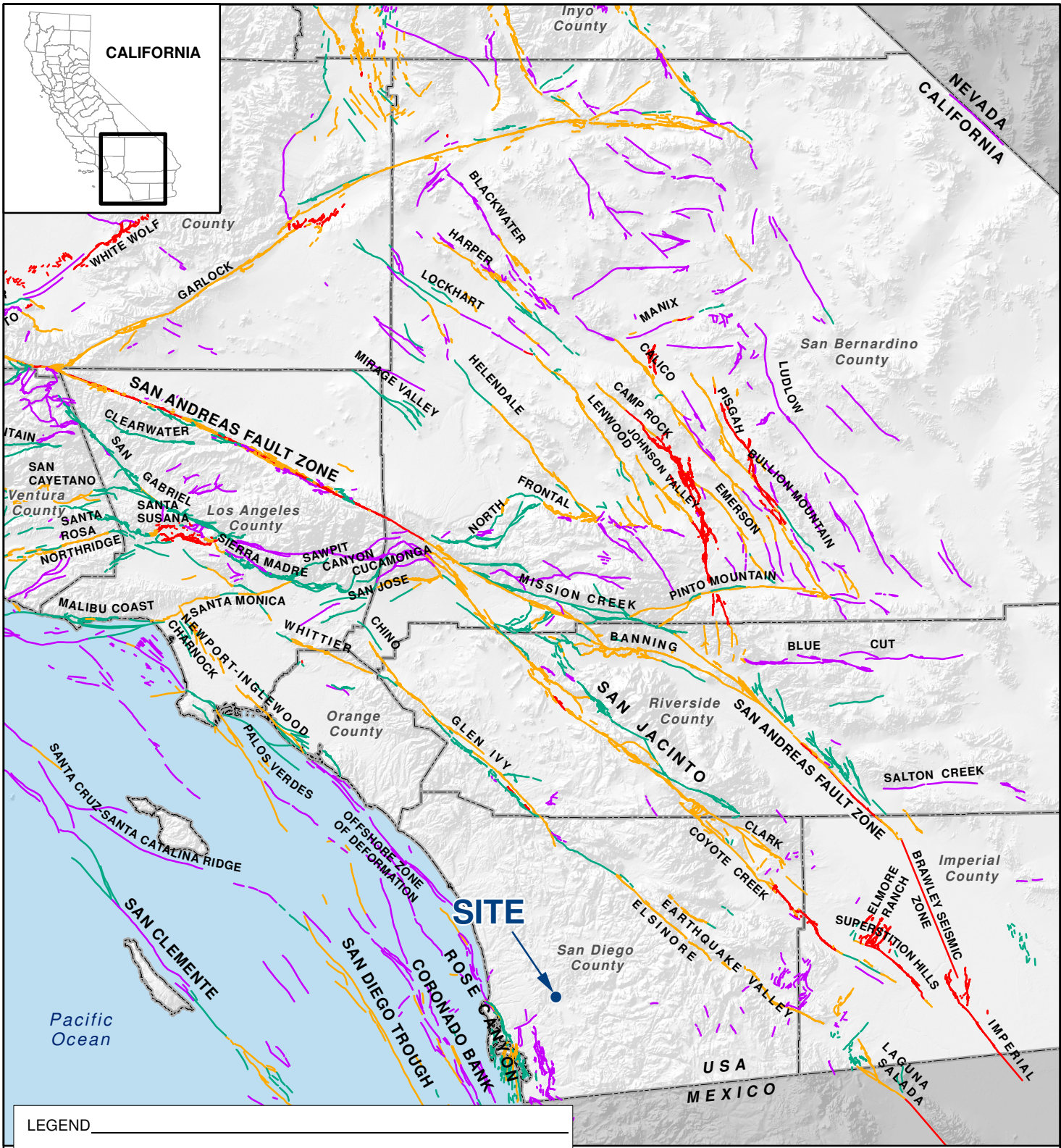
FIGURE 4B

**GEOLOGIC CROSS SECTION B-B'**






SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA

4B\_108773001\_CS\_B-B'.DWG

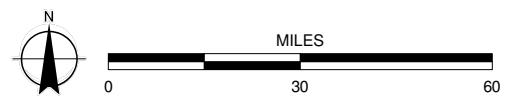




**LEGEND**

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

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



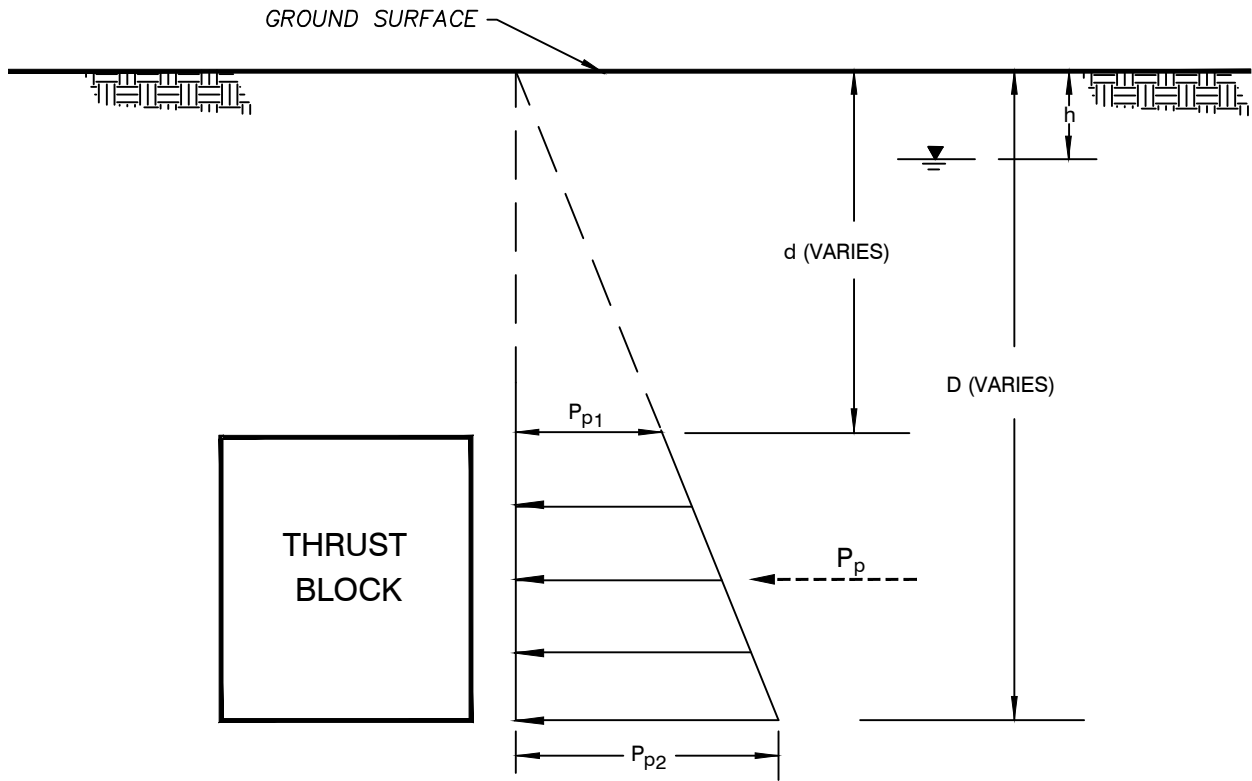
NOTE: DIRECTIONS, DIMENSIONS AND LOCATIONS ARE APPROXIMATE.

**FIGURE 5**

**FAULT LOCATIONS**

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA

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NOTES:

1. GROUNDWATER BELOW BLOCK  

$$P_p = 150 (D^2 - d^2) \text{ lb/ft}$$
2. GROUNDWATER ABOVE BLOCK  

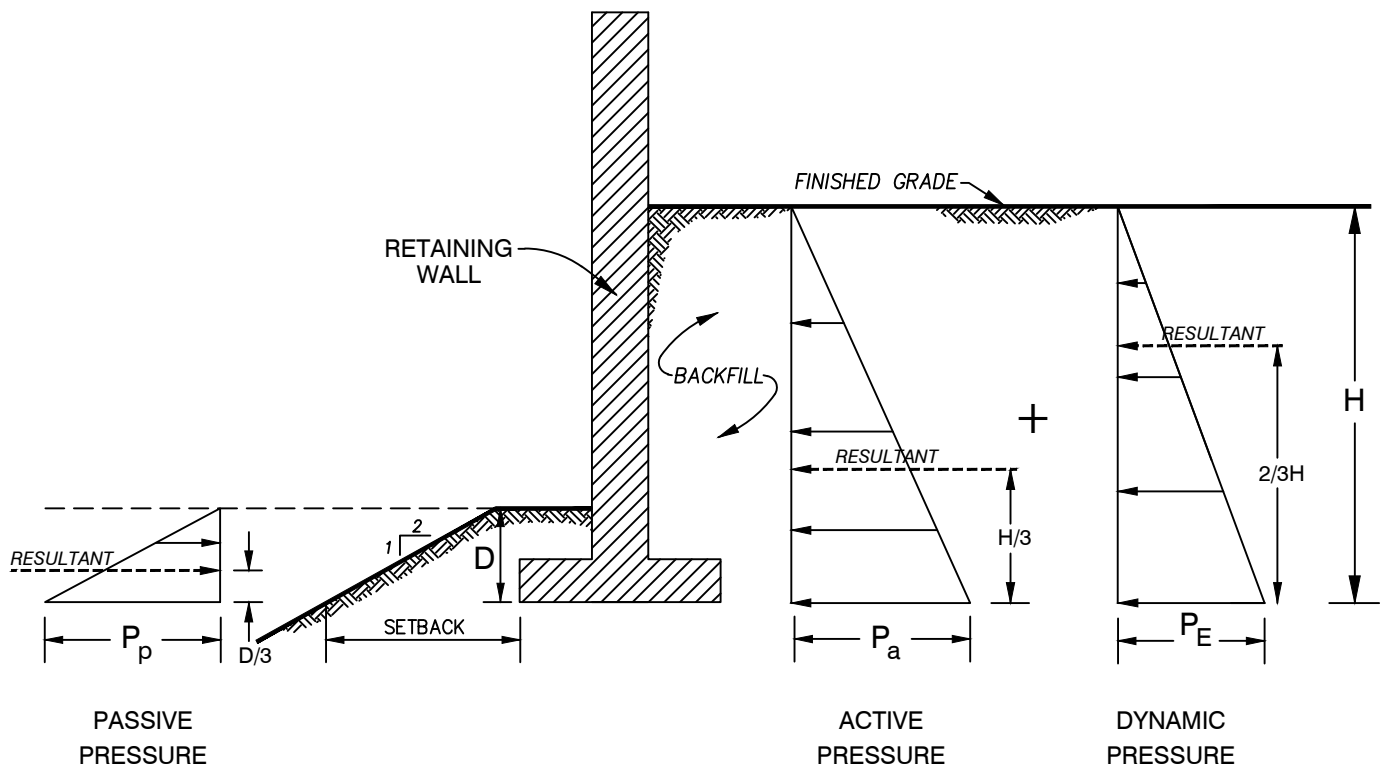
$$P_p = 1.8 (D - d) [124.8 h + 57.6 (D + d)] \text{ lb/ft}$$
3. ASSUMES BACKFILL IS GRANULAR MATERIAL
4. ASSUMES THRUST BLOCK IS ADJACENT TO COMPETENT MATERIAL
5. D, d AND h ARE IN FEET
6. GROUNDWATER TABLE

NOT TO SCALE

FIGURE 6

THRUST BLOCK LATERAL EARTH PRESSURE DIAGRAM

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
 10201 SETTLE ROAD, SANTEE, CALIFORNIA



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A PEAK GROUND ACCELERATION OF 0.39g
5.  $P_E$  IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7.  $H$  AND  $D$  ARE IN FEET
8. SETBACK SHOULD BE IN ACCORDANCE WITH THE CBC

NOT TO SCALE

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure (lb/ft <sup>2</sup> /ft) <sup>(1)</sup>	
$P_a$	Level Backfill with Granular Soils <sup>(2)</sup>	2H:1V Sloping Backfill with Granular Soils <sup>(2)</sup>
	38 H	58 H
$P_E$	16 H	
$P_p$	Level Ground	2H:1V Descending Ground
	300 D	140 D

NOTE: DIMENSIONS, DIRECTIONS AND LOCATIONS ARE APPROXIMATE.

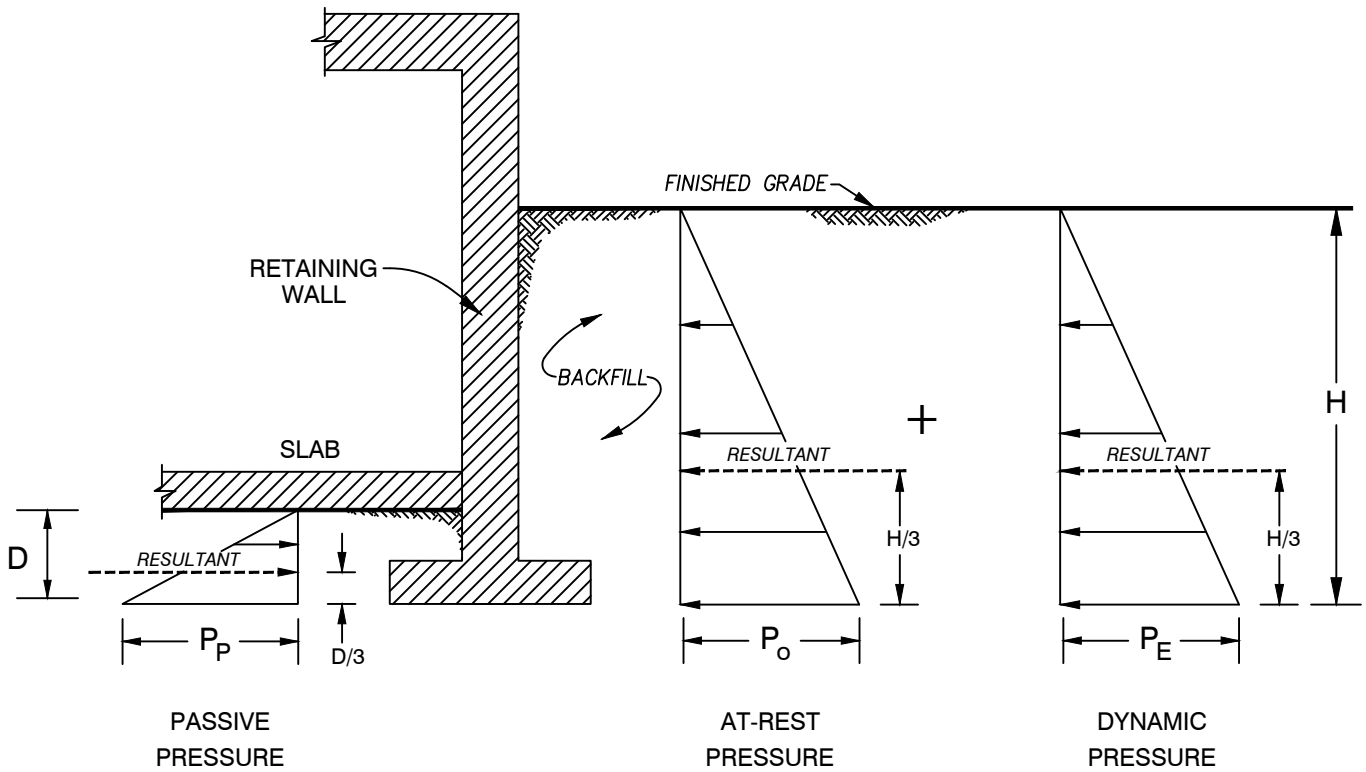
FIGURE 7

LATERAL EARTH PRESSURES FOR YIELDING RETAINING WALLS



Geotechnical & Environmental Sciences Consultants

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA



NOTES:

1. ASSUMES NO HYDROSTATIC PRESSURE BUILD-UP BEHIND THE RETAINING WALL
2. GRANULAR LOW EXPANSIVE BACKFILL MATERIALS SHOULD BE USED FOR RETAINING WALL BACKFILL
3. DRAINS AS RECOMMENDED IN THE RETAINING WALL DRAINAGE DETAIL SHOULD BE INSTALLED BEHIND THE RETAINING WALL
4. DYNAMIC LATERAL EARTH PRESSURE IS BASED ON A PEAK GROUND ACCELERATION OF 0.39g
5.  $P_e$  IS CALCULATED IN ACCORDANCE WITH THE RECOMMENDATIONS OF MONONOBE AND MATSUO (1929), AND ATIK AND SITAR (2010)
6. SURCHARGE PRESSURES CAUSED BY VEHICLES OR NEARBY STRUCTURES ARE NOT INCLUDED
7. H AND D ARE IN FEET

RECOMMENDED GEOTECHNICAL DESIGN PARAMETERS

Lateral Earth Pressure	Equivalent Fluid Pressure (lb/ft <sup>2</sup> /ft) <sup>(1)</sup>	
	Level Backfill with Granular Soils <sup>(2)</sup>	2H:1V Sloping Backfill with Granular Soils <sup>(2)</sup>
$P_o$	58 H	82 H
$P_e$	16 H	
$P_p$	Level Ground	2H:1V Descending Ground
	360 D	140 D

NOT TO SCALE

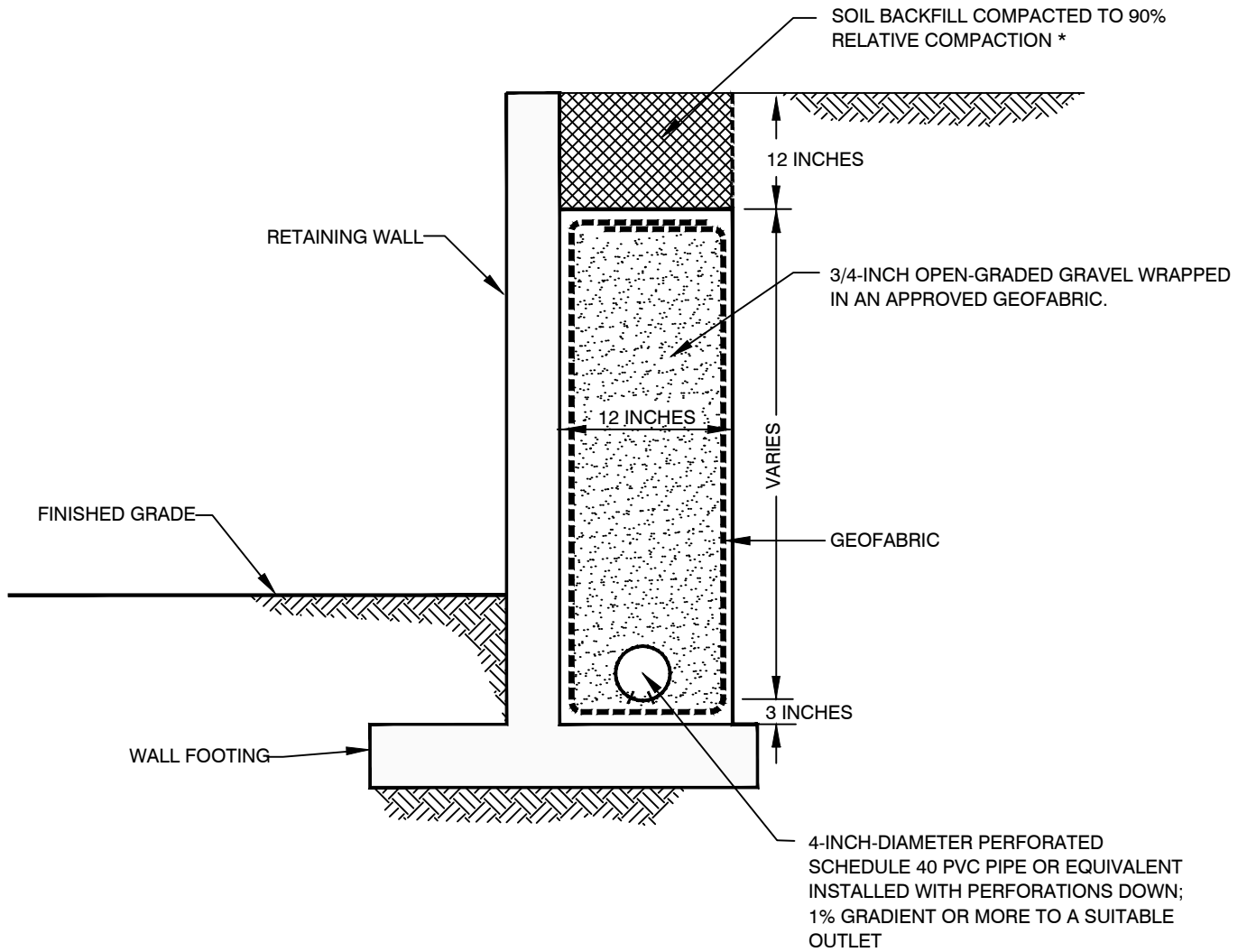
FIGURE 8

LATERAL EARTH PRESSURES FOR RESTRAINED RETAINING WALLS

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA



9\_108773001\_D-RWD.DWG



\*BASED ON ASTM D1557

NOT TO SCALE

FIGURE 9



# APPENDIX A

## Boring Logs

# APPENDIX A

## BORING LOGS

### **Field Procedure for the Collection of Disturbed Samples**

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

#### **Bulk Samples**

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

#### **The Standard Penetration Test (SPT) Sampler**

Disturbed drive samples of earth materials were obtained by means of a Standard Penetration Test sampler. The sampler is composed of a split barrel with an external diameter of 2 inches and an unlined internal diameter of 1 $\frac{3}{8}$  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 inches, was lined with 1-inch-long, thin brass rings with inside diameters of approximately 2.4 inches. The sample barrel was driven into the ground with the weight of a hammer or Kelly bar 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 or Kelly bar, 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.

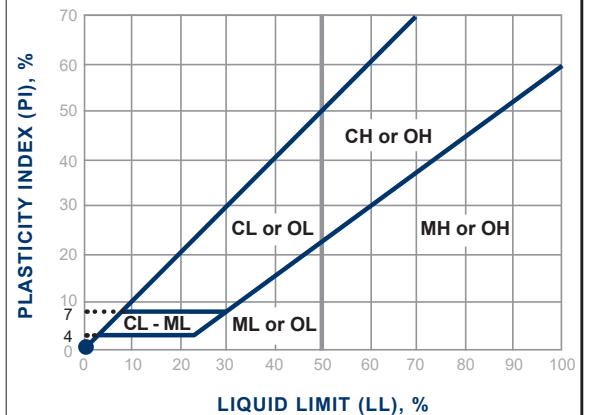
## Soil Classification Chart Per ASTM D 2488

Primary Divisions		Secondary Divisions			
		Group Symbol	Group Name		
<b>COARSE-GRAINED SOILS</b> more than 50% retained on No. 200 sieve	<b>GRAVEL</b> 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	
			GM	silty GRAVEL	
			GC	clayey GRAVEL	
		GRAVEL with FINES more than 12% fines	GC-GM	silty, clayey GRAVEL	
			SW	well-graded SAND	
	SP		poorly graded SAND		
	SW-SM		well-graded SAND with silt		
	<b>SAND</b> 50% or more of coarse fraction passes No. 4 sieve	CLEAN SAND less than 5% fines	SP-SM	poorly graded SAND with silt	
			SW-SC	well-graded SAND with clay	
		SAND with DUAL CLASSIFICATIONS 5% to 12% fines	SP-SC	poorly graded SAND with clay	
			SM	silty SAND	
			SC	clayey SAND	
			SC-SM	silty, clayey SAND	
		SAND with FINES more than 12% fines	CL	lean CLAY	
			ML	SILT	
<b>FINE-GRAINED SOILS</b> 50% or more passes No. 200 sieve	<b>SILT and CLAY</b> liquid limit less than 50%	INORGANIC	CL-ML	silty CLAY	
			ORGANIC	OL (PI > 4)	organic CLAY
				OL (PI < 4)	organic SILT
		<b>SILT and CLAY</b> liquid limit 50% or more		INORGANIC	CH
			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"	Thumb-sized to fist-sized
	Fine	#4 - 3/4"	Pea-sized to thumb-sized
Sand	Coarse	#10 - #4	Rock-salt-sized to pea-sized
	Medium	#40 - #10	Sugar-sized to rock-salt-sized
	Fine	#200 - #40	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

Consistency	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)	Bulk Driven SAMPLES	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).  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.
5	XX/XX		⊕				
10			⊕				
15					█	SM	MAJOR MATERIAL TYPE (SOIL): Solid line denotes unit change.  Dashed line denotes material change.
15					█	CL	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.

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven						4/22/2019	NM-1				
								GROUND ELEVATION	430' ± (MSL)	SHEET	1	OF	1
								METHOD OF DRILLING	8" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)				
								DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"		
								SAMPLED BY	GSW	LOGGED BY	GSW	REVIEWED BY	CAT
								<b>DESCRIPTION/INTERPRETATION</b>					
0								<b>ASPHALT CONCRETE:</b> Approximately 4-3/4 inches.					
							CL	<b>DECOMPOSED GRANITE (DG) FILL MATERIALS:</b> Approximately 1.5 feet thick.					
			39	5.4	102.3			<b>FILL:</b> Grayish brown to reddish brown, moist, stiff, sandy CLAY; scattered gravel and cobbles. Hard.					
								Cobbles from 7 to 9 feet.					
							SM	Brown, moist, dense, silty fine SAND; scattered gravel and cobbles; trace clay.					
10			30					Cobble.					
								Very dense.					
			50/5"					<b>FRIARS FORMATION:</b> Olive to light brown, moist, moderately cemented, gravel and cobble CONGLOMERATE in clay matrix.					
			50/3"					Total Depth = 18.5 feet. (Refusal) Groundwater not encountered during drilling. Backfilled and patched with black-dyed concrete shortly after drilling on 4/22/19.					
20			50/3"					<b>Note:</b> 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 accurate for preparing construction bids and design documents.					
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**FIGURE A-1**

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	
	Bulk	Driven						4/22/2019	NM-2	
								GROUND ELEVATION	SHEET	
								420' ± (MSL)	1 OF 1	
								METHOD OF DRILLING		
								8" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)		
								DRIVE WEIGHT	DROP	
								140 lbs. (Cathead)	30"	
								SAMPLED BY	LOGGED BY	REVIEWED BY
								GSW	GSW	CAT
								<b>DESCRIPTION/INTERPRETATION</b>		
0							SC	<b>ASPHALT CONCRETE:</b> Approximately 7-1/2 inches. Petromat encountered between top and middle layers. No aggregate base encountered.		
			53	12.0				<b>FILL:</b> Light to dark brown, moist, very dense, clayey SAND; scattered roots; gravel and cobbles. Hard.		
10			50/4"	10.7	99.9			<b>FRIARS FORMATION:</b> Reddish brown, moist, strongly indurated, sandy CLAYSTONE; scattered gravel and cobbles. Hard drilling at approximately 9 to 11 feet.		
			50/5"					Reddish brown to yellowish brown; gravelly. Total Depth = 15.4 feet. Groundwater not encountered during drilling. Backfilled and patched with black-dyed concrete shortly after drilling on 4/22/19.		
20								<b>Note:</b> 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 accurate for preparing construction bids and design documents.		
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**FIGURE A-2**



DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven						4/22/2019	NM-3				
								GROUND ELEVATION	415' ± (MSL)	SHEET	1	OF	1
								METHOD OF DRILLING	8" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)				
								DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"		
								SAMPLED BY	GSW	LOGGED BY	GSW	REVIEWED BY	CAT
								<b>DESCRIPTION/INTERPRETATION</b>					
0							SC	<b>FILL:</b> Brown to reddish brown, moist, medium dense, clayey SAND; scattered roots, gravel, and cobbles.					
			50/4"	5.1	108.9			<b>FRIARS FORMATION:</b> Olive to brown, moist, moderately to strongly indurated, sandy CLAYSTONE; scattered gravel and cobbles. Hard drilling on cobbles.					
10			50/3" 50/1"					Total Depth = 10.3 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 4/22/19.  <u>Note:</u> 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 accurate for preparing construction bids and design documents.					
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**FIGURE A-3**

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven						4/22/2019	NM-4				
								GROUND ELEVATION	415' ± (MSL)	SHEET	1	OF	1
								METHOD OF DRILLING	8" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)				
								DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"		
								SAMPLED BY	GSW	LOGGED BY	GSW	REVIEWED BY	CAT
								<b>DESCRIPTION/INTERPRETATION</b>					
0							SC	<p><b>FILL:</b> Brown to reddish brown, moist, medium dense, clayey SAND; scattered roots, gravel, and cobbles.</p>					
			50/2"	5.8	112.8			<p>Hard. Total Depth = 5.7 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 4/22/19.</p>					
10								<p><b>Note:</b> 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.</p>					
20								<p>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 accurate for preparing construction bids and design documents.</p>					
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**FIGURE A-4**

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven						4/22/2019	NM-5				
								GROUND ELEVATION	415' ± (MSL)	SHEET	1	OF	1
								METHOD OF DRILLING	8" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)				
								DRIVE WEIGHT	140 lbs. (Cathead)	DROP	30"		
								SAMPLED BY	GSW	LOGGED BY	GSW	REVIEWED BY	CAT
								<b>DESCRIPTION/INTERPRETATION</b>					
0							SC	<b>FILL:</b> Brown, moist, medium dense, clayey SAND; scattered roots, gravel, and cobbles.					
			50/5" 18					<b>FRIARS FORMATION:</b> Gray to olive, moist, moderately indurated, sandy CLAYSTONE. Changes to reddish brown. Total Depth = 6.5 feet. Groundwater not encountered during drilling. Backfilled shortly after drilling on 4/22/19.					
10								<b>Note:</b> 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 accurate for preparing construction bids and design documents.					
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**FIGURE A-5**

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	IT-1
	Bulk	Driven						4/22/2019		
								GROUND ELEVATION	430' ± (MSL)	SHEET 1 OF 1
								METHOD OF DRILLING	6" Diameter Hand Auger	
								DRIVE WEIGHT	N/A	DROP N/A
								SAMPLED BY	GSW	LOGGED BY GSW REVIEWED BY CAT
<b>DESCRIPTION/INTERPRETATION</b>										
0							SC	<b>DECOMPOSED GRANITE (DG) FILL MATERIALS:</b> Approximately 6 inches thick. <b>FILL:</b> Brown, moist, medium dense, clayey SAND; scattered roots, gravel, and cobbles.		
								<b>FRIARS FORMATION:</b> Light brown to gray, moist, moderately indurated, sandy CLAYSTONE; scattered gravel. Total Depth = 5 feet. Groundwater not encountered during drilling. Backfilled shortly after infiltration testing on 4/23/19.  <u>Note:</u> 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 accurate for preparing construction bids and design documents.		
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20										
30										
40										

**FIGURE A-6**

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	IT-2
	Bulk	Driven						4/22/2019	GROUND ELEVATION	SHEET
								430' ± (MSL)	1	1
								METHOD OF DRILLING 6" Diameter Hollow-Stem Auger (A-300) (Ingersoll-Rand)		
								140 lbs (Cathead)	DROP	30"
								SAMPLED BY GSW	LOGGED BY GSW	REVIEWED BY CAT
<b>DESCRIPTION/INTERPRETATION</b>										
0						CL		<p><b>DECOMPOSED GRANITE (DG) FILL MATERIALS:</b> Approximately 6 inches thick.</p> <p><b>FILL:</b> Reddish brown, moist, stiff, sandy CLAY; scattered gravel and cobbles.</p> <p>Fragments of claystone encountered at approximately 4 to 5 feet.</p>		
								<p>Total Depth = 5 feet. Groundwater not encountered during drilling. Backfilled shortly infiltration testing on 4/23/19.</p> <p><u>Note:</u> 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.</p> <p>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 accurate for preparing construction bids and design documents.</p>		
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20										
30										
40										

**FIGURE A-7**

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.		
	Bulk	Driven						7/23/07	B-1		
								GROUND ELEVATION	SHEET	OF	
								425' ± (MSL)	1	4	
								METHOD OF DRILLING 30" Diameter Bucket (Pacific Drilling)			
								DRIVE WEIGHT	Variable	DROP	N/A
								SAMPLED BY CAT LOGGED BY CAT/RDH REVIEWED BY RI			
								<b>DESCRIPTION/INTERPRETATION</b>			
0							SM	<p><b>FILL:</b> Dark brown, damp to moist, medium dense, silty SAND; trace of clay; roots up to 1" in diameter.</p> <p><b>FRIARS FORMATION:</b> Light brown, reddish brown, gray and yellowish brown, damp to moist, moderately cemented silty, clayey and sandy gravel and cobble CONGLOMERATE; some roots and plant debris in upper 3 feet; rounded cobbles up to 6" in diameter; some red staining.</p>			
5								More reddish brown, clayey sandy matrix (more clay).			
10								Increase in clay; sandy, clayey CONGLOMERATE.			
15								<p>Approximately 3" layer of gravelly CLAYSTONE; undulatory; continuous around hole.</p> <p>Mottled red brown to yellow brown, moist, sandy CONGLOMERATE.</p>			
20								<p>Approximately 3" layer of light tan, moist, weakly indurated, cobbly, sandy CLAYSTONE.</p> <p>Light gray brown, moist to wet, moderately cemented, clayey CONGLOMERATE; slightly sandy.</p>			



<b>BORING LOG</b>		
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DEPTH (feet)	SAMPLES	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.		
							7/23/07	B-1		
							GROUND ELEVATION	SHEET	OF	
							425' ± (MSL)	2	4	
							METHOD OF DRILLING 30" Diameter Bucket (Pacific Drilling)			
							DRIVE WEIGHT	Variable	DROP	N/A
							SAMPLED BY CAT LOGGED BY CAT/RDH REVIEWED BY RI			
							<b>DESCRIPTION/INTERPRETATION</b>			
20			~0				<b>FRIARS FORMATION: (Continued)</b> Light gray brown, moist to wet, moderately cemented, clayey CONGLOMERATE; slightly sandy.			
							Gray brown, wet, moderately cemented, clayey SANDSTONE.			
							Sharp, undulatory contact to mottled brown, saturated, moderately cemented sandstone; friable.			
25							Sandstone is finely laminated. Sharp contact to mottled light gray and reddish brown brown, moist to wet, clayey SILTSTONE.			
							Continuous clay layer (not remolded); finely laminated clay; approximately 1/2" thick; soft clay; more sand with depth. ATT: N5°W 5°E.			
							Grades to light gray, moist to wet, silty fine SANDSTONE; moderately cemented. Sharp, undulatory contact to mottled gray, reddish brown, moist to wet, sandy CONGLOMERATE; extends to 30.2'; approximately 1" thick layer gravelly sandstone; conglomerate (approximately 70% gravel and cobbles 6" to 8" in diameter).			
30										
35							Conglomerate with clayey sand matrix.			
40			~0							



**BORING LOG**

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FIGURE  
A-2



DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.				
	Bulk	Driven						7/23/07	B-1				
								GROUND ELEVATION	SHEET	OF			
								METHOD OF DRILLING	30" Diameter Bucket (Pacific Drilling)				
								DRIVE WEIGHT	Variable	DROP	N/A		
								SAMPLED BY	CAT	LOGGED BY	CAT/RDH	REVIEWED BY	RI
								<b>DESCRIPTION/INTERPRETATION</b>					
40								<b>FRIARS FORMATION: (Continued)</b> Mottled gray, reddish brown, moist to wet, clayey sandy CONGLOMERATE. @ 41': Conglomerate becomes strongly cemented.					
								@ 42.5': Sharp undulatory contact to olive greenish gray, damp, moderately indurated, silty CLAYSTONE.					
								@ 44': Grading to clayey SILTSTONE.					
45			20					@ 46.5': Olive gray, moist, moderately cemented, silty fine-grained SANDSTONE; no observed fractures.					
								@ 50': Dark olive gray, wet, strongly cemented, silty fine-grained sandstone; trace of clay.					
								@ 54': Continuous cemented bed: Light gray, wet, strongly cemented sandstone.					
								@ 54.8': Sharp contact to dark gray, wet, strongly cemented, silty fine grained sandstone; trace to slightly clayey.					
								Some yellowish brown interbedding.					
								@ 59': Standing water.					
60													



**BORING LOG**

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FIGURE  
A-3

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.	
	Bulk	Driven						7/23/07	B-1	
								GROUND ELEVATION	SHEET	OF
								METHOD OF DRILLING		
								DRIVE WEIGHT	Variable	DROP
								SAMPLED BY	CAT	LOGGED BY
									CAT/RDH	REVIEWED BY
										RI
								<b>DESCRIPTION/INTERPRETATION</b>		
60								<b>FRIARS FORMATION: (Continued)</b> Yellowish brown, strongly cemented, silty SANDSTONE; trace of clay; few cobbles and gravel.		
65								Total Depth = 64 feet. (Downhole logged to 59 feet) Seepage encountered at approximately 20.5 and 39.5 feet. Standing water at 59 feet at time of logging. Groundwater may rise to a level higher than that measured in the borehole due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with 2 feet of bentonite at bottom of hole; 6 inch of bentonite for every 5 feet of native soil; and 5 feet of bentonite at top of hole. Backfilled with a total of 61 cubic feet of bentonite shortly after drilling on 7/23/07. Drive Weight = 3500 lbs. from 0 to 27 feet; 2400 lbs. from 27 to 55 feet; 1300 lbs. from 55 to 85 feet.		
70										
75										
80										



**BORING LOG**

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FIGURE  
 A-4

DEPTH (feet)	SAMPLES Bulk Driven	BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED	BORING NO.		
							7/24/07	B-2		
							GROUND ELEVATION	SHEET	OF	
							425' ± (MSL)	1	3	
							METHOD OF DRILLING 30" Diameter Bucket (Pacific Drilling)			
							DRIVE WEIGHT	Variable	DROP	N/A
							SAMPLED BY CAT LOGGED BY CAT/RDH REVIEWED BY RI			
							<b>DESCRIPTION/INTERPRETATION</b>			
0						SC	<u>FILL:</u> Dark brown, damp to moist, loose to medium dense, silty, clayey, fine to medium SAND; few gravel; scattered plant debris.			
						CL	Olive, yellowish brown, and light gray (mottled), damp to moist, soft to firm, gravelly, cobbly, fine to medium sandy CLAY; scattered plant debris.			
5							<u>FRIARS FORMATION:</u> Brown, yellowish brown, reddish brown, and gray (mottled), moist, weakly cemented, clayey, fine- to coarse-grained sandy gravel and cobble CONGLOMERATE; cobbles up to 8" in diameter; matrix supported; fine roots down to 6.5'.  @ 8': Light gray matrix.			
							@ 9.5': Grades to brownish red (mottled), gravelly SANDSTONE.			
10							@ 11': Grades to CONGLOMERATE; matrix is clayey sand.			
							@ 13': Grades to gravelly SANDSTONE; cobbles.			
15							@ 15.5': Sharp near horizontal contact to reddish brown and pale reddish gray (mottled), moist, moderately indurated, silty CLAYSTONE; some fine sand.			
							@ 16.8': Grades over 3" to reddish gray and reddish brown, moist, clayey sandy CONGLOMERATE.			
20										



<b>BORING LOG</b>		
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DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>7/24/07</u>	BORING NO. <u>B-2</u>
	Bulk	Driven						GROUND ELEVATION <u>425' ± (MSL)</u>	SHEET <u>2</u> OF <u>3</u>
								METHOD OF DRILLING <u>30" Diameter Bucket (Pacific Drilling)</u>	
								DRIVE WEIGHT <u>Variable</u>	DROP <u>N/A</u>
								SAMPLED BY <u>CAT</u>	LOGGED BY <u>CAT/RDH</u> REVIEWED BY <u>RI</u>
<b>DESCRIPTION/INTERPRETATION</b>									
20				~0				<b>FRIARS FORMATION: (Continued)</b> Reddish gray to reddish brown, moist, clayey sandy CONGLOMERATE. @ 20': Minor seepage. Heavier on uphill side of hole. @ 21': Cobbles 6" to 8" in diameter (down to 23'). Cobbles are clay-lined; clay layers discontinuous.	
25								@ 26.5': Sharp near horizontal contact to reddish brown and light gray, moist, weakly cemented, silty fine SILTSTONE; finely laminated; no gravel or cobbles; extends down to 27.2 feet. @ 27.2': Reddish brown and gray, gravelly siltstone.	
30								@ 29.8': Sharp contact to reddish brown and gray (mottled), wet, sandy, CONGLOMERATE; clast supported; clasts up to 6 to 8" in diameter.	
35				~0				@ 36': Heavy seepage; heavier on uphill side (E side of hill). @ 36': Sand matrix is yellowish brown.	
40									



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FIGURE  
A-6

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>7/24/07</u>	BORING NO. <u>B-2</u>
	Bulk	Driven						GROUND ELEVATION <u>425' ± (MSL)</u>	SHEET <u>3</u> OF <u>3</u>
								METHOD OF DRILLING <u>30" Diameter Bucket (Pacific Drilling)</u>	
								DRIVE WEIGHT <u>Variable</u>	DROP <u>N/A</u>
								SAMPLED BY <u>CAT</u>	LOGGED BY <u>CAT/RDH</u> REVIEWED BY <u>RI</u>
<b>DESCRIPTION/INTERPRETATION</b>									
40								<b>FRIARS FORMATION: (Continued)</b> @ 40': Sharp contact to gray, wet, moderately indurated, sandy CLAYSTONE; contact @ 40' on E and 42' on W side. Below is massive olive gray; wet; moderately cemented; siltstone.	
45								@ 46': Standing water.	
								Dark olive gray, wet, moderately cemented SILTSTONE.	
50								Fine sandy; very strongly cemented; trace of clay; micaceous.	
55								Moderately cemented.	
								Total Depth = 55 feet. (Downhole logged to 46 feet) Seepage encountered at approximately 20 and 36 feet. Standing water encountered at approximately 46 feet at time of logging. Groundwater may rise to a level higher than that measured in the borehole due to seasonal variations in precipitation and several other factors as discussed in the report. Backfilled with 2 feet of bentonite at bottom hole; 6" of bentonite for every 5 feet of native soil; and 5 feet of bentonite at top of hole. Backfilled with a total of 44 cubic feet of bentonite shortly after drilling on 7/25/07. Drive Weight = 3500 lbs. from 0 to 27 feet; 2400 lbs. from 27 to 55 feet; 1300 lbs. from 55 to 85 feet.	
60									



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FIGURE  
A-7

DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>7/10/07</u> BORING NO. <u>B-3</u> GROUND ELEVATION <u>425' ± (MSL)</u> SHEET <u>1</u> OF <u>2</u> METHOD OF DRILLING <u>8" Diameter Hollow Stem Auger CME-75 (Baja Exploration)</u> DRIVE WEIGHT <u>140 lbs. (Auto-Trip Hammer)</u> DROP <u>30"</u> SAMPLED BY <u>BTM</u> LOGGED BY <u>BTM</u> REVIEWED BY <u>RI</u>		
	Bulk	Driven						DESCRIPTION/INTERPRETATION		
0							SC	<u>FILL:</u> Dark brown, moist, medium dense clayey SAND. Dark to light brown, moist, medium dense, clayey GRAVEL with cobbles. Light gray to light brown.		
						GC				
5			33	13.9	102.9			<u>FRIARS FORMATION:</u> Light brown and gray (mottled), moist, moderately cemented, sandy, clayey, gravel CONGLOMERATE.		
10			19					Brown, moist to wet, moderately to strongly indurated, sandy CLAYSTONE.		
15			40	19.5	107.5					
20			30							



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FIGURE  
A-8



DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>7/10/07</u> BORING NO. <u>B-4</u>	
	Bulk	Driven						GROUND ELEVATION <u>425' ± (MSL)</u>	SHEET <u>1</u> OF <u>2</u>
								METHOD OF DRILLING <u>8" Diameter Hollow Stem Auger CME-75 (Baja Exploration)</u>	
								DRIVE WEIGHT <u>140 lbs. (Auto-Trip Hammer)</u> DROP <u>30"</u>	
								SAMPLED BY <u>BTM</u> LOGGED BY <u>BTM</u> REVIEWED BY <u>RI</u>	
								<b>DESCRIPTION/INTERPRETATION</b>	
0							SC	<u>FILL:</u>	
							GC	Dark brown, moist, medium dense clayey SAND. Dark to light brown, moist, hard, sandy clayey GRAVEL with cobbles. Light gray to light brown.	
5			30	18.5	92.4			<u>FRIARS FORMATION:</u> Light brown and gray, moist, moderately cemented, sandy, clayey, gravel CONGLOMERATE.	
10			25						
15			40	20.2	103.9			Brown, moist, moderately to strongly indurated, sandy CLAYSTONE; scattered gravel.	
								Wet.	
20			80/9"						



**BORING LOG**

SYCAMORE CANYON SCHOOL  
SANTEE, CALIFORNIA

PROJECT NO. 106111001	DATE 8/07	FIGURE A-10
--------------------------	--------------	----------------



DEPTH (feet)	SAMPLES		BLOWS/FOOT	MOISTURE (%)	DRY DENSITY (PCF)	SYMBOL	CLASSIFICATION U.S.C.S.	DATE DRILLED <u>7/10/07</u> BORING NO. <u>B-4</u>	
	Bulk	Driven						GROUND ELEVATION <u>425' ± (MSL)</u>	SHEET <u>2</u> OF <u>2</u>
								METHOD OF DRILLING <u>8" Diameter Hollow Stem Auger CME-75 (Baja Exploration)</u>	
								DRIVE WEIGHT <u>140 lbs. (Auto-Trip Hammer)</u> DROP <u>30"</u>	
								SAMPLED BY <u>BTM</u> LOGGED BY <u>BTM</u> REVIEWED BY <u>RI</u>	
								<b>DESCRIPTION/INTERPRETATION</b>	
20								<p>Total Depth = 19.8 feet.  Groundwater not encountered during drilling.  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.  Backfilled with 3.9 cubic feet of bentonite shortly after drilling on 7/10/07.</p>	
25									
30									
35									
40									



**BORING LOG**

SYCAMORE CANYON SCHOOL  
SANTEE, CALIFORNIA

PROJECT NO.  
106111001

DATE  
8/07

FIGURE  
A-11



# APPENDIX B

## Geotechnical Laboratory Testing

# APPENDIX B

## GEOTECHNICAL 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 Figures B-1 through B-5 from our recent evaluation and Figures B-1 and B-2 from our previous evaluation (Ninyo & Moore, 2007). The 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 from our previous evaluation (Ninyo & Moore, 2007) 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 classifications in accordance with the USCS. The test results and classifications are shown on Figure B-3 from our previous evaluation (Ninyo & Moore, 2007).

### **Consolidation**

A consolidation test was performed on a selected relatively undisturbed soil sample in general accordance with ASTM D 2435. The sample was inundated during testing to represent adverse field conditions. The percent of consolidation for each load cycle was recorded as a ratio of the amount of vertical compression to the original height of the sample. The results of the test is summarized on Figure B-6 from our recent evaluation.

### **Direct Shear Test**

A direct shear test was performed on a relatively undisturbed sample in general accordance with ASTM D 3080 to evaluate the shear strength characteristics of the selected material. The sample was inundated during shearing to represent adverse field conditions. The results are shown on Figure B-7 from our recent evaluation.

### **Expansion Index Test**

The expansion index of selected materials were evaluated in general accordance with ASTM D 4829. The specimens were molded under a specified compactive energy at approximately 50 percent saturation. The prepared 1-inch thick by 4-inch diameter specimens were loaded with a surcharge of 144 pounds per square foot and was inundated with tap water. Readings of volumetric swell were made for a period of 24 hours. The results of this test are presented on Figure B-8 for this recent evaluation and Figure B-4 for our previous evaluation (Ninyo & Moore, 2007).

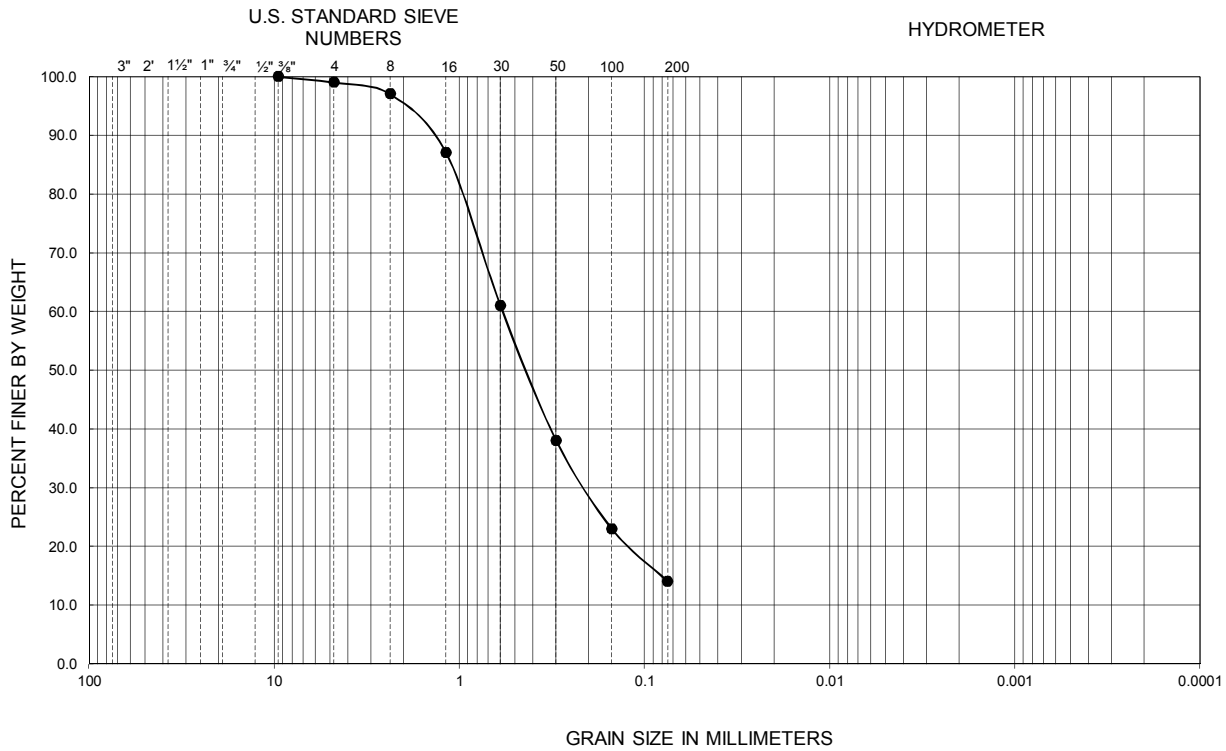
### **Soil Corrosivity Tests**

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 CT 422, respectively. The test results are presented on Figure B-9 from our recent evaluation and Figure B-5 from our previous evaluation (Ninyo & Moore, 2007).

### **R-Value**

The resistance value, or R-value, for site soils were evaluated in general accordance with CT 301. Samples were prepared and evaluated for exudation pressure and expansion pressure. The equilibrium R-value is reported as the lesser or more conservative of the two calculated results. The test results are shown on Figure B-10 from our recent evaluation.

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

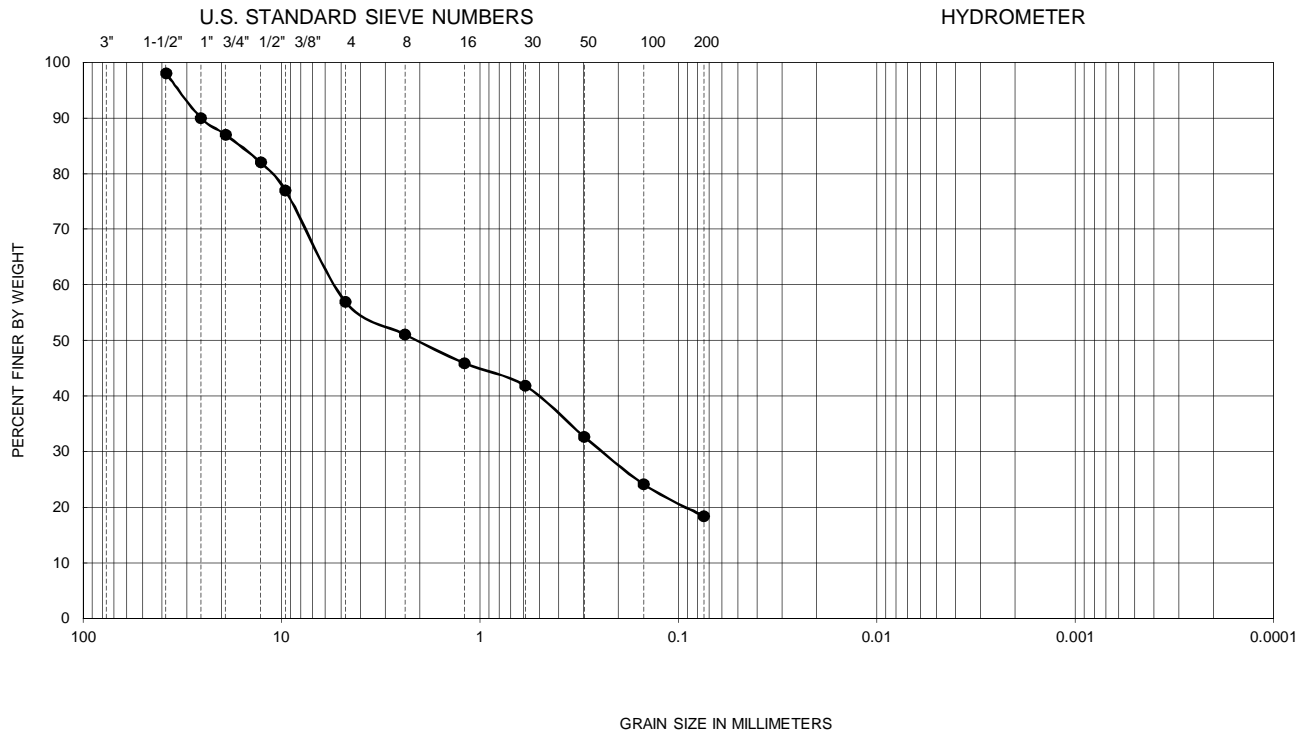


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>	Passing No. 200 (percent)	USCS
●	NM-1	10.0-11.5	--	--	--	--	--	--	--	--	14	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-1

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>	Passing No. 200 (percent)	USCS
●	NM-2	0.6-5.0	--	--	--	--	--	--	--	--	18	SC

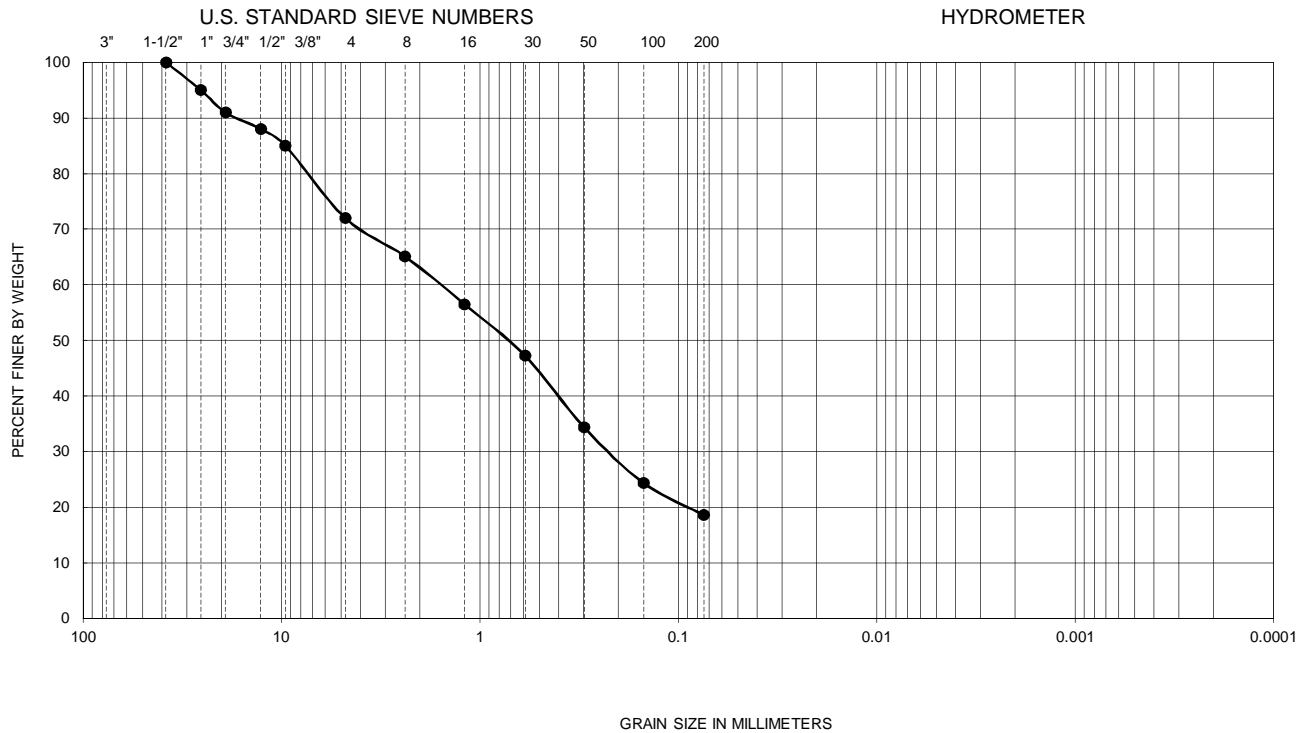
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-2

GRADATION TEST RESULTS

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>	Passing No. 200 (percent)	USCS
●	NM-3	0.0-5.0	--	--	--	--	--	--	--	--	19	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

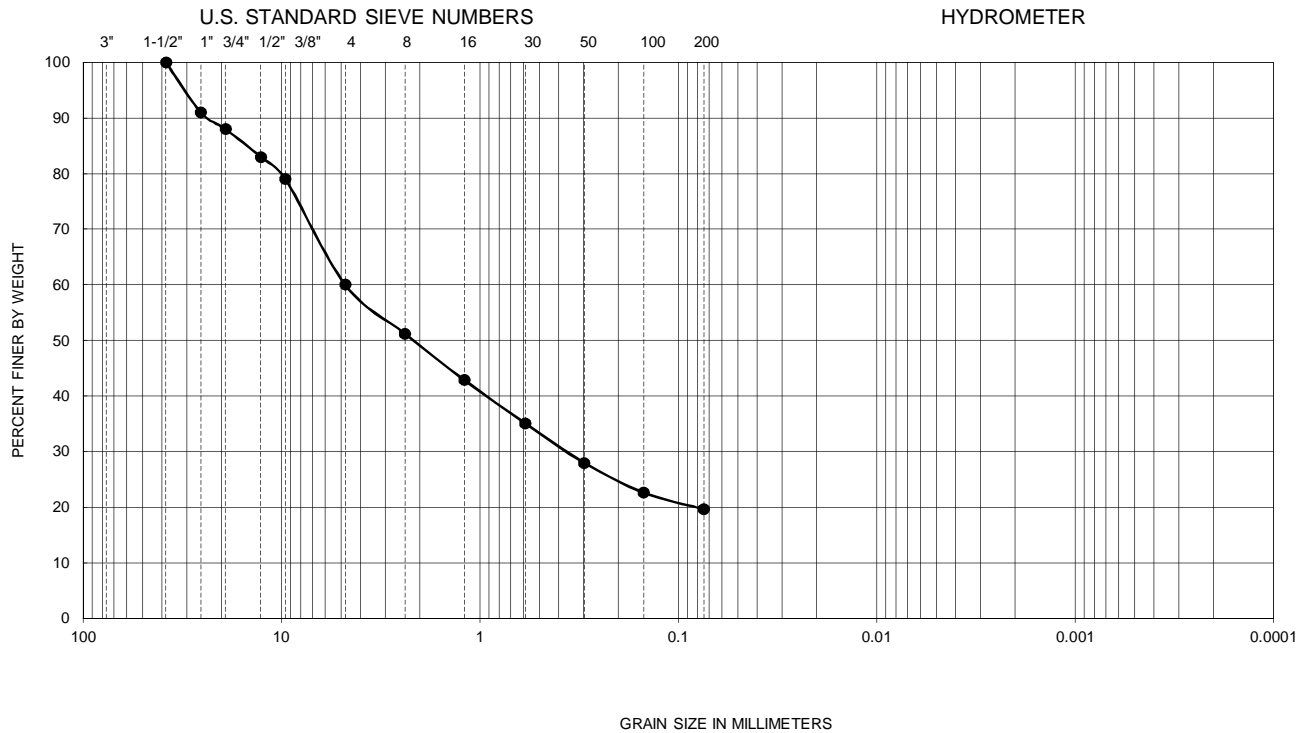
FIGURE B-3

GRADATION TEST RESULTS

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA



GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>	Passing No. 200 (percent)	USCS
●	NM-5	0.0-5.0	--	--	--	--	--	--	--	--	20	SC

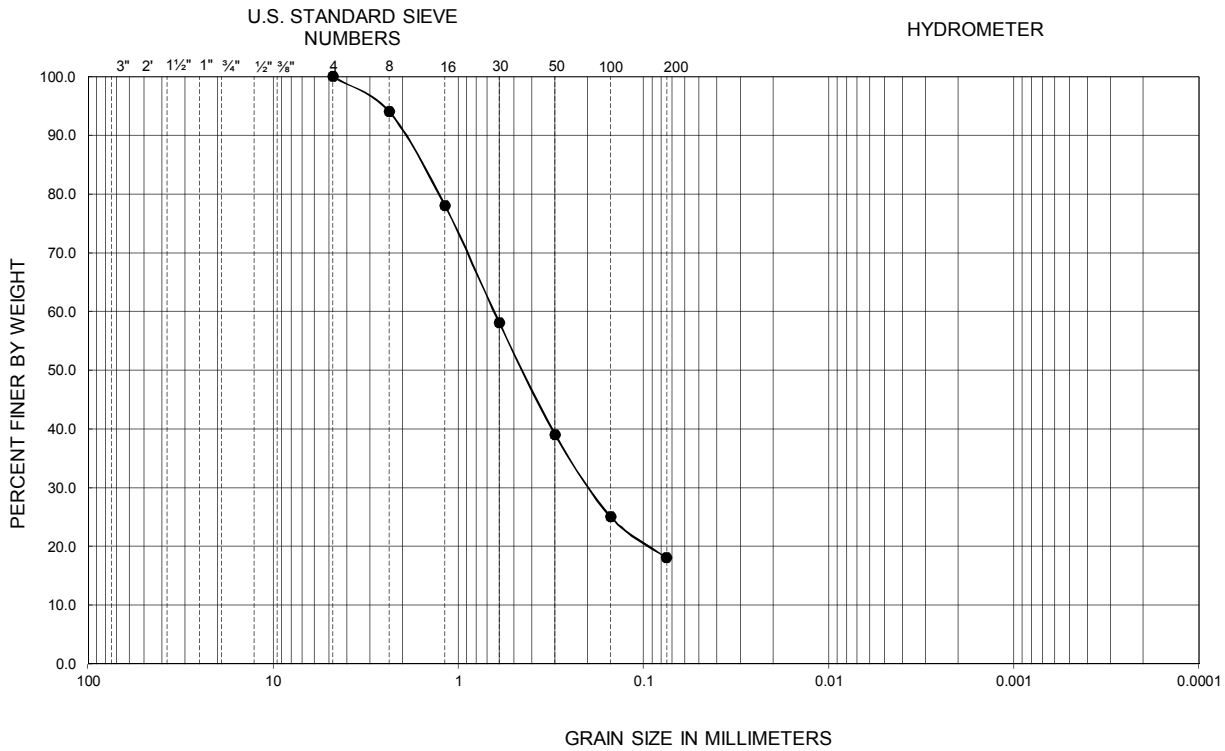
PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-4

GRADATION TEST RESULTS

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA

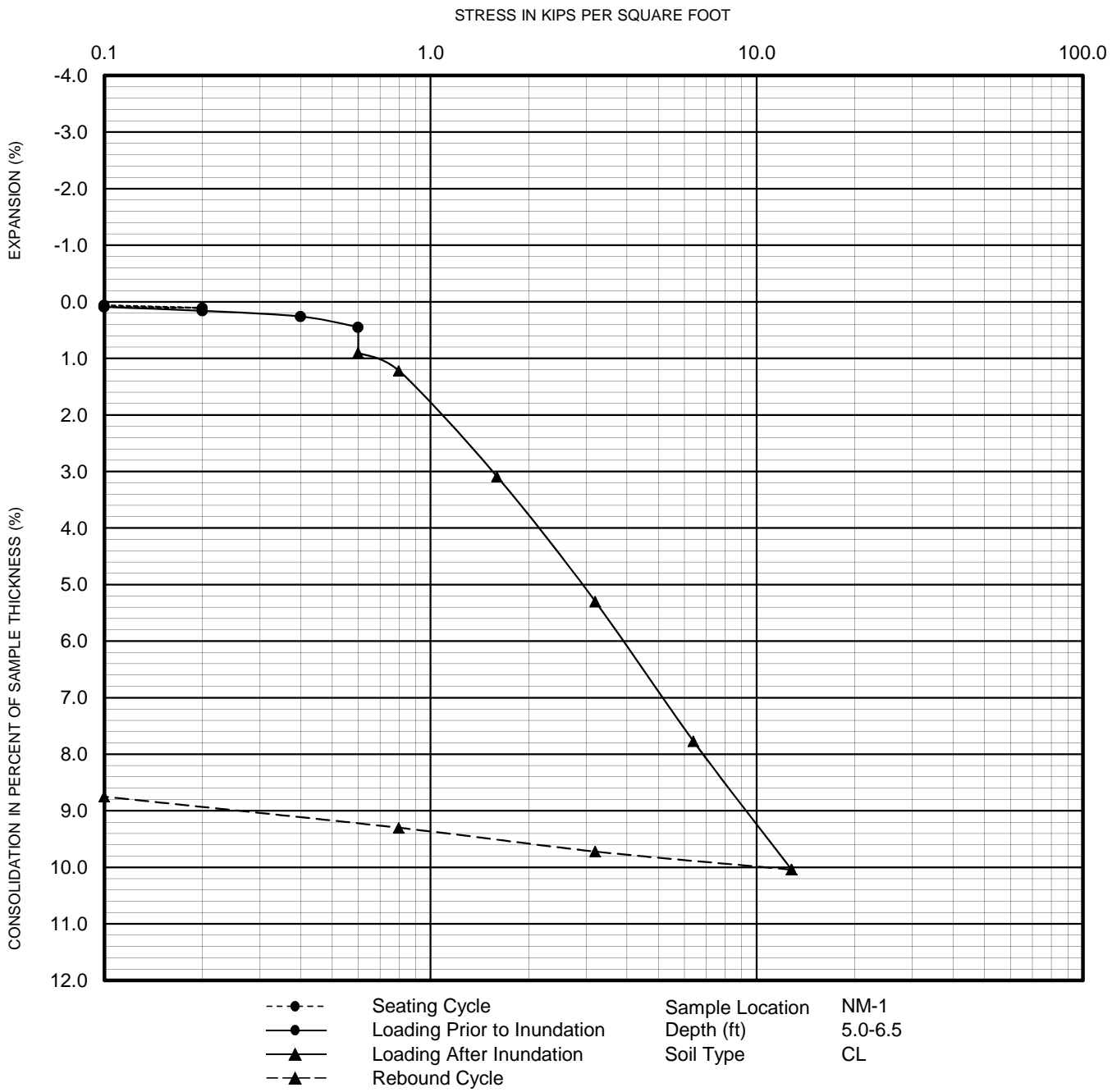
GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY



Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>	Passing No. 200 (percent)	USCS
●	IT-1	0.5-5.0	--	--	--	--	--	--	--	--	18	SC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422

FIGURE B-5



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2435

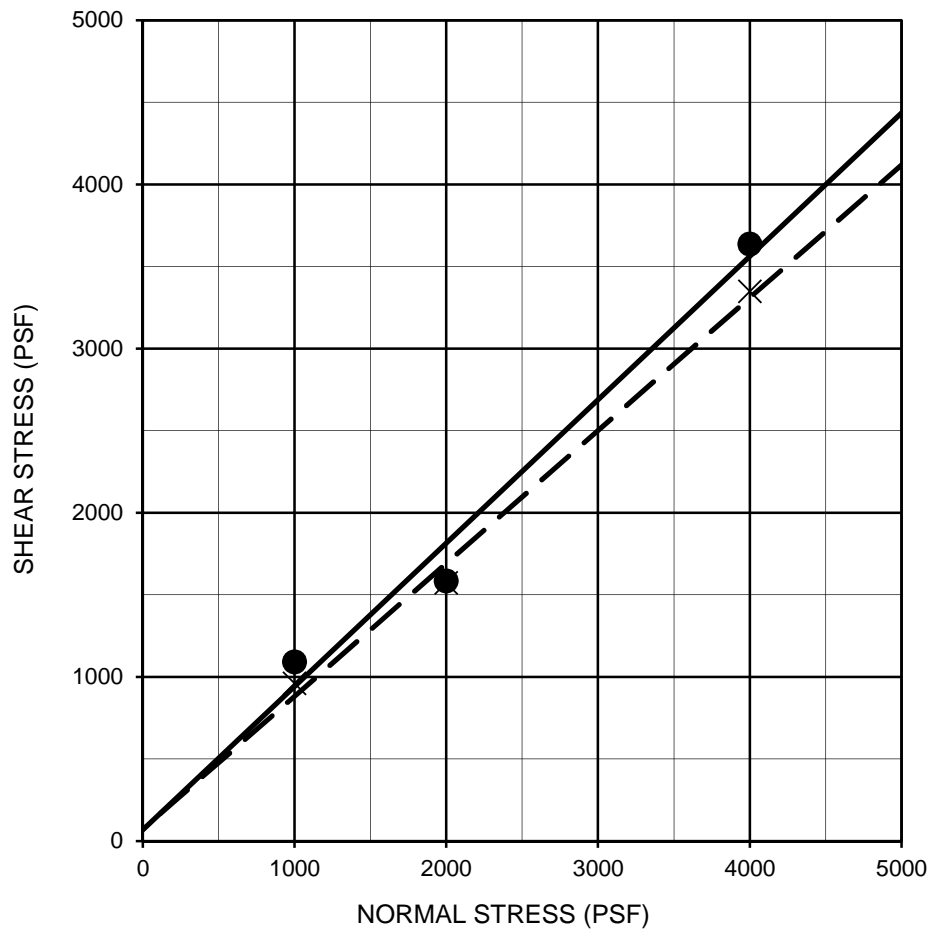


**FIGURE B-6**

**CONSOLIDATION TEST RESULTS**

SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
10201 SETTLE ROAD, SANTEE, CALIFORNIA

108773001 | 5/19



Description	Symbol	Sample Location	Depth (ft)	Shear Strength	Cohesion (psf)	Friction Angle (degrees)	Soil Type
Clayey SANDSTONE	●	NM-2	10.0-10.3	Peak	70	41	Formation
Clayey SANDSTONE	- - X - -	NM-2	10.0-10.3	Ultimate	70	39	Formation

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 3080

**FIGURE B-7**

SAMPLE LOCATION	SAMPLE DEPTH (ft)	INITIAL MOISTURE (percent)	COMPACTED DRY DENSITY (pcf)	FINAL MOISTURE (percent)	VOLUMETRIC SWELL (in)	EXPANSION INDEX	POTENTIAL EXPANSION
NM-1	2.0-5.0	9.5	110.8	22.1	0.005	5	Very Low
NM-3	0.0-5.0	9.5	111.0	21.5	0.021	21	Low
IT-2	0.5-5.0	10.0	110.4	20.6	0.005	5	Very Low

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

**FIGURE B-8**



**EXPANSION INDEX TEST RESULTS**  
 SYCAMORE CANYON SCHOOL BUILDING ADDITIONS  
 10201 SETTLE ROAD, SANTEE, CALIFORNIA

SAMPLE LOCATION	SAMPLE DEPTH (ft)	pH <sup>1</sup>	RESISTIVITY <sup>1</sup> (ohm-cm)	SULFATE CONTENT <sup>2</sup>		CHLORIDE CONTENT <sup>3</sup> (ppm)
				(ppm)	(%)	
NM-2	0.6-5.0	7.8	740	20	0.002	225

<sup>1</sup> PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

<sup>2</sup> PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

<sup>3</sup> PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

**FIGURE B-9**

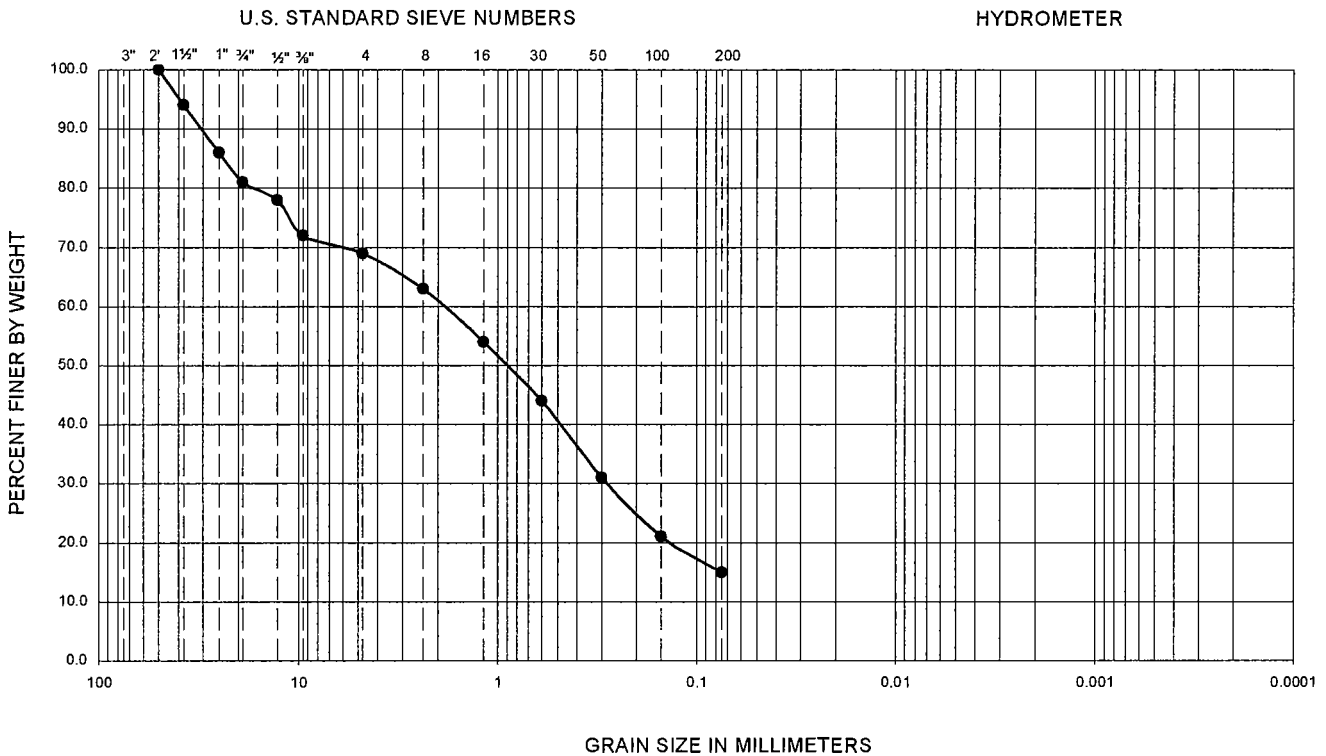


SAMPLE LOCATION	SAMPLE DEPTH (ft)	SOIL TYPE	R-VALUE
NM-1	2.0-5.0	Sandy CLAY (CL)	23
NM-4	0.5-5.0	Clayey SAND (SC)	40

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 2844/CT 301

**FIGURE B-10**

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	SILT	CLAY

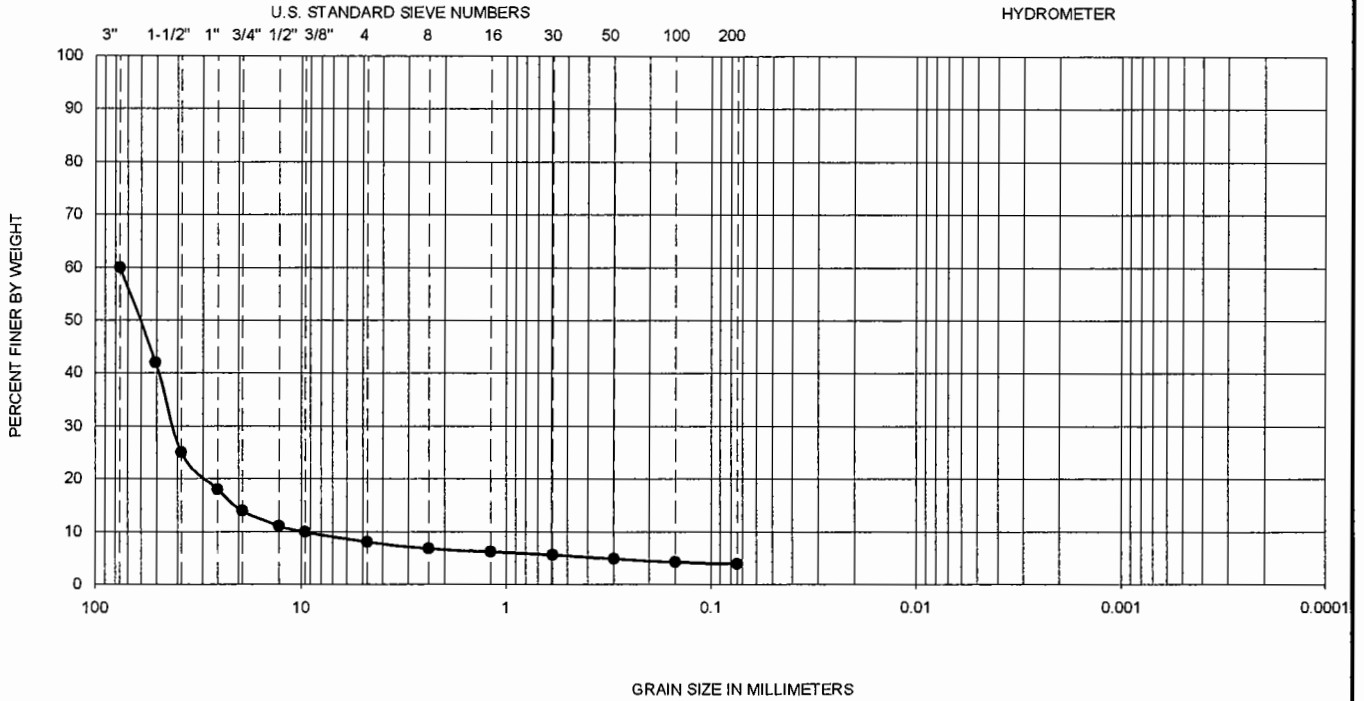


Symbol	Sample Location	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>	Passing No. 200 (%)	USCS
●	B-1	0.0-5.0	--	--	--	--	--	--	--	--	15	SM

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

<b>Ninyo &amp; Moore</b>		<b>GRADATION TEST RESULTS</b>		<b>FIGURE</b> <b>B-1</b>
PROJECT NO.	DATE	SYCAMORE CANYON SCHOOL SANTEE, CALIFORNIA		
106111001	8/07			

GRAVEL		SAND			FINES	
Coarse	Fine	Coarse	Medium	Fine	Silt	Clay



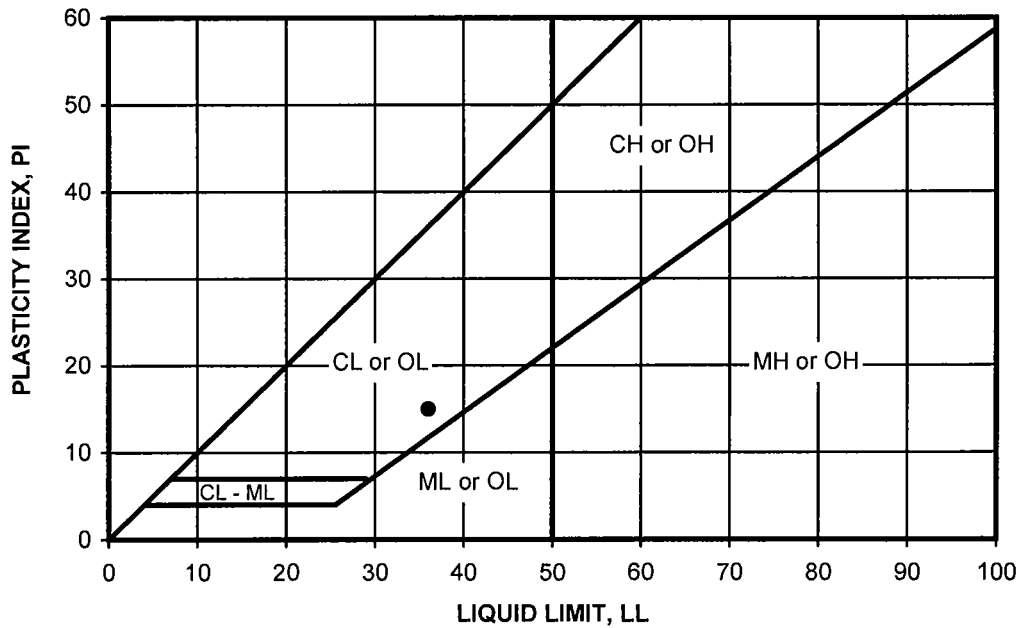
Symbol	Hole No.	Depth (ft)	Liquid Limit	Plastic Limit	Plasticity Index	D <sub>10</sub>	D <sub>30</sub>	D <sub>60</sub>	C <sub>u</sub>	C <sub>c</sub>	Passing No. 200 (%)	U.S.C.S
●	B-3	0.5-5.0	--	--	--	--	--	--	--	--	4	GC

PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 422-63 (02)

<b>Ningo &amp; Moore</b>		<b>GRADATION TEST RESULTS</b>		FIGURE <b>B-2</b>
PROJECT NO.	DATE	SYCAMORE CANYON SCHOOL SANTEE, CALIFORNIA		
106111001	8/07			

SYMBOL	LOCATION	DEPTH (FT)	LIQUID LIMIT, LL	PLASTIC LIMIT, PL	PLASTICITY INDEX, PI	USCS CLASSIFICATION (Fraction Finer Than No. 40 Sieve)	USCS (Entire Sample)
•	B-2	15.0-16.5	36	21	15	CL	CL

NP - INDICATES NON-PLASTIC



PERFORMED IN GENERAL ACCORDANCE WITH ASTM D 4318-05

<b>Ninyo &amp; Moore</b>		<b>ATTERBERG LIMITS TEST RESULTS</b>	FIGURE
PROJECT NO.	DATE	SYCAMORE CANYON SCHOOL SANTEE, CALIFORNIA	<b>B-3</b>
106111001	8/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	INITIAL MOISTURE (%)	COMPACTED DRY DENSITY (PCF)	FINAL MOISTURE (%)	VOLUMETRIC SWELL (IN)	EXPANSION INDEX	POTENTIAL EXPANSION
B-2	3.0-5.0	13.0	98.4	29.7	0.072	72	Medium

PERFORMED IN GENERAL ACCORDANCE WITH  UBC STANDARD 18-2  ASTM D 4829-03

<b><i>Ninyo &amp; Moore</i></b>		<b>EXPANSION INDEX TEST RESULTS</b>	FIGURE
PROJECT NO.	DATE	SYCAMORE CANYON SCHOOL SANTEE, CALIFORNIA	<b>B-4</b>
106111001	8/07		

SAMPLE LOCATION	SAMPLE DEPTH (FT)	pH <sup>1</sup>	RESISTIVITY <sup>1</sup> (Ohm-cm)	SULFATE CONTENT <sup>2</sup>		CHLORIDE CONTENT <sup>3</sup> (ppm)
				(ppm)	(%)	
B-2	3.0-5.0	7.3	3,790	680	0.068	260

<sup>1</sup> PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 643

<sup>2</sup> PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 417

<sup>3</sup> PERFORMED IN GENERAL ACCORDANCE WITH CALIFORNIA TEST METHOD 422

<b><i>Ninyo &amp; Moore</i></b>		<b>CORROSIVITY TEST RESULTS</b>	FIGURE
PROJECT NO.	DATE	SYCAMORE CANYON SCHOOL SANTEE, CALIFORNIA	<b>B-5</b>
106111001	8/07		



# APPENDIX C

## Infiltration Testing



Test Date:	4/14/2017	Infiltration Test No.:	SC-1
Test Hole Diameter, D (inches):	6.0	Excavation Depth (feet):	3.0
Test performed and recorded by:	GSW	Pipe Length (feet):	3.0

t <sub>1</sub>	d <sub>1</sub> (feet)	t <sub>2</sub>	d <sub>2</sub> (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H <sub>avg</sub> (feet)	Infiltration Rate (in/hr)
7:10	1.27	7:35	1.28	25	0.01	208.33	1.73	0.02
7:35	1.27	8:00	1.29	25	0.02	104.17	1.72	0.04
8:00	1.27	8:30	1.30	30	0.03	83.33	1.72	0.05
8:30	1.27	9:00	1.29	30	0.02	125.00	1.72	0.03
9:00	1.27	9:30	1.29	30	0.02	125.00	1.72	0.03
9:30	1.27	10:00	1.29	30	0.02	125.00	1.72	0.03
10:00	1.27	10:30	1.28	30	0.01	250.00	1.73	0.02
10:30	1.27	11:00	1.28	30	0.01	250.00	1.73	0.02
11:00	1.27	11:30	1.28	30	0.01	250.00	1.73	0.02
11:30	1.27	12:00	1.28	30	0.01	250.00	1.73	0.02
12:00	1.27	12:30	1.28	30	0.01	250.00	1.73	0.02
12:30	1.27	1:00	1.28	30	0.01	250.00	1.73	0.02

Test Date:	4/14/2017	Infiltration Test No.:	SC-2
Test Hole Diameter, D (inches):	6.0	Excavation Depth (feet):	3.0
Test performed and recorded by:	GSW	Pipe Length (feet):	3.0

t <sub>1</sub>	d <sub>1</sub> (feet)	t <sub>2</sub>	d <sub>2</sub> (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H <sub>avg</sub> (feet)	Infiltration Rate (in/hr)
7:12	0.91	7:37	0.92	25	0.01	208.33	2.09	0.02
7:37	0.91	8:02	0.93	25	0.02	104.17	2.08	0.03
8:02	0.91	8:32	0.93	30	0.02	125.00	2.08	0.03
8:32	0.91	9:02	0.92	30	0.01	250.00	2.09	0.01
9:02	0.91	9:32	0.93	30	0.02	125.00	2.08	0.03
9:32	0.91	10:02	0.92	30	0.01	250.00	2.09	0.01
10:02	0.91	10:32	0.92	30	0.01	250.00	2.09	0.01
10:32	0.91	11:02	0.92	30	0.01	250.00	2.09	0.01
11:02	0.91	11:32	0.92	30	0.01	250.00	2.09	0.01
11:32	0.91	12:02	0.92	30	0.01	250.00	2.09	0.01
12:02	0.91	12:32	0.92	30	0.01	250.00	2.09	0.01
12:32	0.91	1:02	0.92	30	0.01	250.00	2.09	0.01

Δt = change in time between initial and final water level readings

ΔH = change in depth to water or change in height of water column (i.e., d<sub>2</sub> - d<sub>1</sub>)

in/hr = inches per hour

I<sub>t</sub> = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

H<sub>avg</sub> = average head over the time interval, inches

<sup>1</sup> Based on the "Porchet Method" as presented in:  
Riverside County Flood Control, 2011, Design Handbook for Low Impact  
Development Best Management Practices: dated September.

$$I_t = \frac{\Delta H \times 60 \times r}{\Delta t(r + 2H_{avg})}$$

Test Date:	4/14/2017	Infiltration Test No.:	SC-3
Test Hole Diameter, D (inches):	6.0	Excavation Depth (feet):	3.0
Test performed and recorded by:	GSW	Pipe Length (feet):	3.0

t <sub>1</sub>	d <sub>1</sub> (feet)	t <sub>2</sub>	d <sub>2</sub> (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H <sub>avg</sub> (feet)	Infiltration Rate (in/hr)
7:18	1.95	7:43	1.95	25	0.00	---	1.05	DNI
7:43	1.95	8:08	1.96	25	0.01	208.33	1.05	0.03
8:08	1.95	8:38	1.96	30	0.01	250.00	1.05	0.03
8:38	1.95	9:08	1.96	30	0.01	250.00	1.05	0.03
9:08	1.95	9:38	1.96	30	0.01	250.00	1.05	0.03
9:38	1.95	10:08	1.96	30	0.01	250.00	1.05	0.03
10:08	1.95	10:38	1.96	30	0.01	250.00	1.05	0.03
10:38	1.95	11:08	1.96	30	0.01	250.00	1.05	0.03
11:08	1.95	11:38	1.96	30	0.01	250.00	1.05	0.03
11:38	1.95	12:08	1.96	30	0.01	250.00	1.05	0.03
12:08	1.95	12:38	1.96	30	0.01	250.00	1.05	0.03
12:38	1.95	1:08	1.96	30	0.01	250.00	1.05	0.03

Test Date:	4/14/2017	Infiltration Test No.:	SC-4
Test Hole Diameter, D (inches):	6.0	Excavation Depth (feet):	3.0
Test performed and recorded by:	GSW	Pipe Length (feet):	3.0

t <sub>1</sub>	d <sub>1</sub> (feet)	t <sub>2</sub>	d <sub>2</sub> (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H <sub>avg</sub> (feet)	Infiltration Rate (in/hr)
7:20	2.00	7:45	2.01	25	0.01	208.33	1.00	0.03
7:45	2.00	8:10	2.03	25	0.03	69.44	0.99	0.10
8:10	2.00	8:40	2.03	30	0.03	83.33	0.99	0.08
8:40	2.00	9:10	2.03	30	0.03	83.33	0.99	0.08
9:10	2.00	9:40	2.03	30	0.03	83.33	0.99	0.08
9:40	2.00	10:10	2.02	30	0.02	125.00	0.99	0.05
10:10	2.00	10:40	2.02	30	0.02	125.00	0.99	0.05
10:40	2.00	11:10	2.02	30	0.02	125.00	0.99	0.05
11:10	2.00	11:40	2.02	30	0.02	125.00	0.99	0.05
11:40	2.00	12:10	2.02	30	0.02	125.00	0.99	0.05
12:10	2.00	12:40	2.02	30	0.02	125.00	0.99	0.05
12:40	2.00	1:10	2.02	30	0.02	125.00	0.99	0.05

Δt = change in time between initial and final water level readings

ΔH = change in depth to water or change in height of water column (i.e., d<sub>2</sub> - d<sub>1</sub>)

in/hr = inches per hour

I<sub>t</sub> = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

H<sub>avg</sub> = average head over the time interval, inches

<sup>1</sup> Based on the "Porchet Method" as presented in:  
Riverside County Flood Control, 2011, Design Handbook for Low Impact  
Development Best Management Practices: dated September.

$$I_t = \frac{\Delta H \times 60 \times r}{\Delta t(r + 2H_{avg})}$$

Test Date:	4/24/2019	Infiltration Test No.:	IT-1
Test Hole Diameter, D (inches):	8.0	Excavation Depth (feet):	5.0
Test performed and recorded by:	GSW	Pipe Length (feet):	5.0

t <sub>1</sub>	d <sub>1</sub> (feet)	t <sub>2</sub>	d <sub>2</sub> (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H <sub>avg</sub> (feet)	Infiltration Rate (in/hr)
7:30	1.32	7:55	1.32	25	0.00	---	3.68	DNI
7:55	1.32	8:20	1.33	25	0.01	208.33	3.68	0.01
8:20	1.32	8:50	1.32	30	0.00	---	3.68	DNI
8:50	1.32	9:20	1.33	30	0.01	250.00	3.68	0.01
9:20	1.32	9:50	1.32	30	0.00	---	3.68	DNI
9:50	1.32	10:20	1.33	30	0.01	250.00	3.68	0.01
10:20	1.32	10:50	1.33	30	0.01	250.00	3.68	0.01
10:50	1.32	11:20	1.32	30	0.00	---	3.68	DNI
11:20	1.32	11:50	1.33	30	0.01	250.00	3.68	0.01
11:50	1.32	12:20	1.32	30	0.00	---	3.68	DNI
12:20	1.32	12:50	1.33	30	0.01	250.00	3.68	0.01
12:50	1.32	1:20	1.32	30	0.00	---	3.68	DNI

Test Date:	4/24/2019	Infiltration Test No.:	IT-2
Test Hole Diameter, D (inches):	8.0	Excavation Depth (feet):	5.0
Test performed and recorded by:	GSW	Pipe Length (feet):	5.0

t <sub>1</sub>	d <sub>1</sub> (feet)	t <sub>2</sub>	d <sub>2</sub> (feet)	Δt (min)	ΔH (feet)	Percolation Rate (min/in)	H <sub>avg</sub> (feet)	Infiltration Rate (in/hr)
7:31	1.45	7:56	1.45	25	0.00	---	3.55	DNI
7:56	1.45	8:21	1.46	25	0.01	208.33	3.55	0.01
8:21	1.45	8:51	1.45	30	0.00	---	3.55	DNI
8:51	1.45	9:21	1.46	30	0.01	250.00	3.55	0.01
9:21	1.45	9:51	1.45	30	0.00	---	3.55	DNI
9:51	1.45	10:21	1.46	30	0.01	250.00	3.55	0.01
10:21	1.45	10:51	1.45	30	0.00	---	3.55	DNI
10:51	1.45	11:21	1.46	30	0.01	250.00	3.55	0.01
11:21	1.45	11:51	1.45	30	0.00	---	3.55	DNI
11:51	1.45	12:21	1.46	30	0.01	250.00	3.55	0.01
12:21	1.45	12:51	1.45	30	0.00	---	3.55	DNI
12:51	1.45	1:21	1.46	30	0.01	250.00	3.55	0.01

Δt = change in time between initial and final water level readings

ΔH = change in depth to water or change in height of water column (i.e., d<sub>2</sub> - d<sub>1</sub>)

in/hr = inches per hour

I<sub>t</sub> = tested infiltration rate, inches/hour

ΔH = change in head over the time interval, inches

Δt = time interval, minutes

r = effective radius of test hole

H<sub>avg</sub> = average head over the time interval, inches

<sup>1</sup> Based on the "Porchet Method" as presented in:  
Riverside County Flood Control, 2011, Design Handbook for Low Impact  
Development Best Management Practices: dated September.

$$I_t = \frac{\Delta H \times 60 \times r}{\Delta t (r + 2H_{avg})}$$

## Appendix D: Approved Infiltration Rate Assessment Methods

**Worksheet 0-1: Factor of Safety and Design Infiltration Rate Worksheet**

Factor of Safety and Design Infiltration Rate Worksheet			Worksheet D.5-1		
Factor Category		Factor Description	Assigned Weight (w)	Factor Value (v)	Product (p) $p = w \times v$
A	Suitability Assessment	Soil assessment methods	0.25	2	0.5
		Predominant soil texture	0.25	3	0.75
		Site soil variability	0.25	2	0.5
		Depth to groundwater / impervious layer	0.25	1	0.25
		Suitability Assessment Safety Factor, $S_A = \sum p$			
B	Design	Level of pretreatment/ expected sediment loads	0.5		
		Redundancy/resiliency	0.25		
		Compaction during construction	0.25		
		Design Safety Factor, $S_B = \sum p$			
Combined Safety Factor, $S_{total} = S_A \times S_B$					
Observed Infiltration Rate, inch/hr, $K_{observed}$ (corrected for test-specific bias)					
Design Infiltration Rate, in/hr, $K_{design} = K_{observed} / S_{total}$					
<b>Supporting Data</b>					
<p>Briefly describe infiltration test and provide reference to test forms:</p> <p>Six infiltration tests were performed at the site, SC-1 through SC-4, IT-1, and IT-2 . In-situ rates (i.e., unfactored rates) ranged from 0.01 to 0.05 inches per hour for SC-1 through SC-4. Infiltration tests IT-1 and IT-2 did not infiltrate. SC-1 through SC-4 were each performed at a depth of approximately 3 feet below existing grade in fill material consisting of sandy clay.</p> <p>IT-1 was performed at a depth of approximately 5 feet below existing grade in formational material consisting of sandy claystone.</p> <p>IT-2 was performed at a depth of approximately 5 feet below existing grade in fill material consisting of sandy clay.</p> <p>Additional information regarding infiltration testing performed at the site is included in Ninyo &amp; Moore's geotechnical evaluation (2019).</p>					

## Appendix C: Geotechnical and Groundwater Investigation Requirements

### Worksheet 0-1: Categorization of Infiltration Feasibility Condition

Categorization of Infiltration Feasibility Condition		Worksheet C.4-1	
<p><b><u>Part 1 - Full Infiltration Feasibility Screening Criteria</u></b></p> <p><b>Would infiltration of the full design volume be feasible from a physical perspective without any undesirable consequences that cannot be reasonably mitigated?</b></p>			
Criteria	Screening Question	Yes	No
1	<p><b>Is the estimated reliable infiltration rate below proposed facility locations greater than 0.5 inches per hour?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.</p>		X
<p>Provide basis:</p> <p>Six infiltration tests were performed at the site, SC-1 through SC-4, IT-1, and IT-2 . In-situ rates (i.e., unfactored rates) ranged from 0.01 to 0.05 inches per hour for SC-1 through SC-4. Infiltration tests IT-1 and IT-2 did not infiltrate. SC-1 through SC-4 were each performed at a depth of approximately 3 feet below existing grade in fill material consisting of sandy clay. IT-1 was performed at a depth of approximately 5 feet below existing grade in formational material consisting of sandy claystone. IT-2 was performed at a depth of approximately 5 feet below existing grade in fill material consisting of sandy clay. Additional information regarding infiltration testing performed at the site is included in Ninyo &amp; Moore's geotechnical evaluation (2019).</p> <p>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</p>			
2	<p><b>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.</p>		X
<p>Provide basis:</p> <p>Reliable infiltration rates (i.e., factored) of 0.03 inches per hour or less, additionally two infiltration locations did not infiltrate. Underground utilities are within 10 feet of the proposed infiltration basins. Additional information regarding infiltration testing performed at the site is included in Ninyo &amp; Moore's geotechnical evaluation (2019).</p> <p>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</p>			

## Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 2 of 4			
Criteria	Screening Question	Yes	No
3	<p><b>Can infiltration greater than 0.5 inches per hour be allowed without increasing risk of groundwater contamination (shallow water table, storm water pollutants or other factors) that cannot be mitigated to an acceptable level?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</p>		X
<p>Provide basis:</p> <p>Reliable infiltration rates (i.e., factored) of 0.03 inches per hour or less, additionally two infiltration locations did not infiltrate. Additional information regarding infiltration testing performed at the site is included in Ninyo &amp; Moore's geotechnical evaluation (2019).</p> <p>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</p>			
4	<p><b>Can infiltration greater than 0.5 inches per hour be allowed without causing potential water balance issues such as change of seasonality of ephemeral streams or increased discharge of contaminated groundwater to surface waters?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</p>		X
<p>Provide basis:</p> <p>Reliable infiltration rates (i.e., factored) of 0.03 inches per hour or less, additionally two infiltration locations did not infiltrate. Additional information regarding infiltration testing performed at the site is included in Ninyo &amp; Moore's geotechnical evaluation (2019).</p> <p>Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability.</p>			
<b>Part 1 Result*</b>	<p>If all answers to rows 1 - 4 are “<b>Yes</b>” a full infiltration design is potentially feasible. The feasibility screening category is <b>Full Infiltration</b></p> <p>If any answer from row 1-4 is “<b>No</b>”, infiltration may be possible to some extent but would not generally be feasible or desirable to achieve a “full infiltration” design. Proceed to Part 2</p>		NO

\*To be completed using gathered site information and best professional judgment considering the definition of MEP in the MS4 Permit. Additional testing and/or studies may be required by City Engineer to substantiate findings.

## Appendix C: Geotechnical and Groundwater Investigation Requirements

### Worksheet C.4-1 Page 3 of 4

**Part 2 – Partial Infiltration vs. No Infiltration Feasibility Screening Criteria**

**Would infiltration of water in any appreciable amount be physically feasible without any negative consequences that cannot be reasonably mitigated?**

Criteria	Screening Question	Yes	No
5	<b>Do soil and geologic conditions allow for infiltration in any appreciable rate or volume?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2 and Appendix D.		X

Provide basis:

Reliable infiltration rates (i.e., factored) of 0.03 inches per hour or less, additionally two infiltration locations did not infiltrate. Additional information regarding infiltration testing performed at the site is included in Ninyo & Moore's geotechnical evaluation (2019).

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

6	<b>Can Infiltration in any appreciable quantity be allowed without increasing risk of geotechnical hazards (slope stability, groundwater mounding, utilities, or other factors) that cannot be mitigated to an acceptable level?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.2.		X
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Provide basis:

Underground utilities are within 10 feet of the proposed infiltration basins. Additional information regarding infiltration testing performed at the site is included in Ninyo & Moore's geotechnical evaluation (2019).

Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.

## Appendix C: Geotechnical and Groundwater Investigation Requirements

Worksheet C.4-1 Page 4 of 4			
Criteria	Screening Question	Yes	No
7	<p><b>Can Infiltration in any appreciable quantity be allowed without posing significant risk for groundwater related concerns (shallow water table, storm water pollutants or other factors)?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</p>	X	
<p>Provide basis:</p> <p style="margin-left: 20px;">Groundwater is anticipated between 46 and 59 feet below existing grade. Additional information regarding infiltration testing performed at the site is included in Ninyo &amp; Moore's geotechnical evaluation (2019).</p> <p style="margin-left: 20px;">Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</p>			
8	<p><b>Can infiltration be allowed without violating downstream water rights?</b> The response to this Screening Question shall be based on a comprehensive evaluation of the factors presented in Appendix C.3.</p>		X
<p>Provide basis:</p> <p style="margin-left: 20px;">Santee Lakes are located approximately 0.15 miles west of the project site. Sycamore Canyon School is located up-gradient from the Santee Lakes. Additional information regarding infiltration testing performed at the site is included in Ninyo &amp; Moore's geotechnical evaluation (2019).</p> <p style="margin-left: 20px;">Summarize findings of studies; provide reference to studies, calculations, maps, data sources, etc. Provide narrative discussion of study/data source applicability and why it was not feasible to mitigate low infiltration rates.</p>			
<b>Part 2 Result*</b>	<p>If all answers from row 1-4 are yes then partial infiltration design is potentially feasible. The feasibility screening category is <b>Partial Infiltration.</b></p> <p>If any answer from row 5-8 is no, then infiltration of any volume is considered to be <b>infeasible</b> within the drainage area. The feasibility screening category is <b>No Infiltration.</b></p>		No infiltration

\*To be completed using gathered site information and best professional judgment considering the definition of MEP in the Permit. Additional testing and/or studies may be required by Agency/Jurisdictions to substantiate findings





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